
Actions from waves and currents on coastal structures

Effets des vagues et des courants sur les structures côtières



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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

International Standards are drafted in accordance with the rules given in the ISO/IEC Directives, Part 2.

The main task of technical committees is to prepare International Standards. Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights.

ISO 21650 was prepared by Technical Committee ISO/TC 98, *Bases for design of structures*, Subcommittee SC 3, *Loads, forces and other actions*.

Introduction

This International Standard, which deals with the actions from waves and currents on structures in the coastal zone and in estuaries, is the first of its kind. Waves and currents and actions from waves and currents on structures in deeper water, especially structures for the petroleum industry, are dealt with in ISO 19901-1 and ISO 19902, ISO 19903 and ISO 19904-1. Some of the structural elements for deeper water structures and coastal structures are the same, especially elements with cylindrical shapes. There will thus be, to some extent, an overlap between this International Standard and other ISO standards on the wave and current actions on cylindrical structural elements. There is though, a difference in wave conditions and wave kinematics between coastal waves and deeper water waves.

Actions from waves and currents on coastal structures

1 Scope

This International Standard describes the principles of determining the wave and current actions on structures of the following types in the coastal zone and estuaries:

- breakwaters:
 - rubble mound breakwaters;
 - vertical and composite breakwaters;
 - wave screens;
 - floating breakwaters;
- coastal dykes;
- seawalls;
- cylindrical structures (jetties, dolphins, lighthouses, pipelines etc.).

For the rubble mound structures it is not possible to determine the forces on and the stability of each individual armour unit because of the complex flow around and between each armour unit. But there are formulae and principles to estimate the necessary armour unit mass given the design wave conditions. Coefficients in these formulae are based on hydraulic model tests. Since the rubble mound structures are heavily used, they are included in this International Standard, although they may not be treated exactly in accordance with ISO 2394.

This International Standard does not include breakwater layout for harbours, layout of structures to manage sediment transport, scour and beach stability or the response of flexible dynamic structures, except vortex induced vibrations.

Design will be performed at different levels of detail:

- concepts;
- feasibility;
- detailed design.

This International Standard is aimed at serving the detailed design.

It is pointed out that the annexes are only informative and are not guidelines/manuals. The annexes have no regulatory power.

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.

2 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

- 2.1**
actions
force (load) applied to the structure by waves and/or currents
- 2.2**
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
- 2.3**
annual maximum method
method of estimating extreme wave heights based on a sample of annual maximum wave heights
- 2.4**
armour layer
protective layer on a breakwater, seawall or other rubble mound structures composed of armour units
- 2.5**
armour unit
relatively large quarry stone or concrete shaped unit that is selected to fit specified geometric characteristics and density
- 2.6**
astronomical tide
phenomenon of the alternate rising and falling of sea surface solely governed by the astronomical conditions of the sun and the moon, which is predicted with the tidal constituents determined from harmonic analysis of tide level readings over a long period
- 2.7**
breakwater
structure protecting a shore area, harbour, anchorage and/or basin from waves
- 2.8**
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
- 2.9**
chart datum
CD
reference level for soundings in navigation charts
- 2.10**
core
inner portion of a breakwater, dyke and rubble mound structures, often with low permeability
- 2.11**
crest
1. highest point of a coastal structure
2. highest point of a wave profile
- 2.12**
crown wall
concrete superstructure on a rubble mound

2.13**datum level**

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

2.14**deep water**

water of such a depth that surface waves are little affected by bottom topography, being larger than about one-half the wavelength

2.15**design water level****DWL**

water level selected for functional design, structural design and stability analysis of marine structures

NOTE Generally it is the water level that mostly affects the safety of the structures/facilities in question. DWL is chosen in view of the acceptable level of risk of failure/damage.

2.16**density driven currents**

currents induced by horizontal gradients of water density generated by changes in the salinity and/or temperature, which are caused by the influx of fresh water from run-off from land through an estuary, heat flux from coastal power stations, or other reasons

2.17**diffractions coefficient**

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

2.18**directional spreading function**

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

2.19**directional wave spectrum**

function expressing the energy density distribution of waves in the frequency and directional domains, being expressed as the product of frequency wave spectrum and the directional spreading function

2.20**drag coefficient**

coefficient used in the Morison equation to determine the drag force

2.21**dyke berms**

nearly horizontal area in the seaward and landward dyke slope which are primarily built to provide access for maintenance and amenity and which reduce wave run-up and overtopping

2.22**dyke toe**

part of a dyke that terminates the base of the dyke on its seaward face

NOTE Various toe constructions are used to prevent undermining of the dyke.

2.23**extreme sea state****extreme waves**

state of waves occurring a few dozen times a year to once in many years, expressed with the significant wave height and the mean or significant wave period at the peak of storm event

2.24

filter

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

2.25

floating breakwater

moored floating object to reduce wave heights in the area behind the floating breakwater

2.26

foreshore

shallow water zone near the shore on which coastal dykes, seawalls and other structures are built

NOTE In beach morphology the term foreshore is used to denote the part of the shore lying between the crest of the seaward berm and the ordinary low water mark.

2.27

frequency wave spectrum

function expressing the energy density distribution of waves in the frequency domain

2.28

geotextile

synthetic fabric which may be woven or non-woven used as a filter

2.29

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

2.30

highest wave height

height of the highest wave of a given wave record or that in a wave train under a given sea state

2.31

impulsive wave pressure

water pressure of high peak intensity with a very short duration induced by the collision of the front surface of a breaking wave with a structure or the collision of a rising wave surface with a horizontal or slightly inclined deck of a pier

2.32

inertia coefficient

coefficient used in the Morison equation to determine the inertia force

2.33

international marine chart datum

IMCD

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

2.34

jetty GB

pier US

deck structure supported by vertical and possibly inclined piles extending into the sea, frequently in a direction normal to the coastline

2.35**lift coefficient**

coefficient used to determine the lift force

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

2.37**mean high water springs****MHWS**

average height of high waters, occurring at the time of spring tides

2.38**mean low water springs****MLWS**

average height of low waters occurring at the time of the spring tides

2.39**mean sea level****MSL**

average height of the sea level for all stages of the tide over a 19-year period, generally determined from hourly height readings

2.40**mean water level****MWL**

average elevation of the water surface over a given time period, usually determined from hourly tidal level readings

NOTE The monthly mean water level varies around seasons by a few tens of centimetres.

2.41**mean wave period**

average period of all waves among a given wave record

NOTE The mean wave period is often estimated from the spectral information obtained from a wave record. See 5.2.1.

2.42**moorings**

ropes, wires or chains to hold a floating object in position

2.43**overtopping**

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

NOTE This definition could serve as a general definition and should not be given individually for each structure.

2.44**parapet**

low wall built along the crest of a seawall

2.45

peaks-over-threshold method

POT method

method of estimating extreme wave heights based on a sample of peak heights of storm waves exceeding some threshold level

2.46

peak wave period

period corresponding to the peak of frequency wave spectrum

2.47

permeability

capacity of bulk material (sand, crushed rock, soft rock *in situ*) in permitting movement of water through its pores

2.48

pipeline

structure for carrying water, oil, gas, sewage, etc.

2.49

pipng

erosion of closed flow channels caused by water flowing through soil usually underneath the dyke body

NOTE Soil particles are carried about by seepage flow, thus endangering the stability of the dyke.

2.50

pore pressure

interstitial pressure of water within a mass of soil or rock

2.51

porosity

percentage of the total volume of a soil and/or granular material occupied by air/gas and water

2.52

pulsating wave pressure

wave pressure with a period comparable with the wave period

2.53

refraction coefficient

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

2.54

reflection coefficient

ratio of the height of reflected waves to the height of incident waves

2.55

revetment

cladding of concrete slabs, asphalt, clay, grass and other materials to protect the surface of a sea dyke against erosion

2.56

rip-rap

usually, well-graded quarry stone, randomly placed as an armour layer to prevent erosion

2.57

rock

aggregate of one or more minerals

2.58**run-up/run-down**

phenomenon of waves running up and down the seaward slope of a sloping structure, their height being measured as the vertical distance from the still water level

2.59***R*-year wave height**

extreme wave height corresponding to the return period of *R* years

NOTE When used, the specific value of *R* is indicated such as 100-year wave height.

2.60**scour**

removal of underwater sand and stone material by waves and currents, especially at the base or toe of a structure

2.61**sea state**

condition of sea surface within a short time span, being expressed with characteristic wave heights, periods and directions

2.62**seaward dyke slope**

slope of the dyke on the seaward side that is generally flatter than 1:4 to reduce wave run-up, protected by a revetment made of clay and grass, concrete slabs, asphalt, or stones to prevent erosion

2.63**shallow water**

water of such a depth that surface waves are noticeably affected by bottom topography, being less than about one-half the wavelength

NOTE Region of water in which waves propagate is sometimes classified into three categories of 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.

2.64**shoaling coefficient**

ratio of the height of waves affected by the depth change in shallow water to their height in deep water with the refraction effect eliminated

2.65**shoreward dyke slope**

slope of dyke on the landward side, generally no steeper than 1:3 to prevent erosion by wave overtopping

NOTE It is generally protected by a revetment made of clay/grass.

2.66**significant wave height**

average height of the one-third highest waves of a given wave record

NOTE The significant wave height is often estimated from the spectral information obtained from a wave record. See 5.2.1.

2.67**significant wave period**

average period of the one-third highest waves of a given wave record

2.68**slamming actions**

actions when a water surface and a structure suddenly collide

2.69

still water level

SWL

level of water surface in the absence of any wave and wind actions, is also called the undisturbed water level

2.70

stone

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

2.71

storm surge

phenomenon of the rise of the sea surface above astronomical water level on the open coast, bays and on estuaries due to the action of wind stresses on the water surface, the atmospheric pressure reduction, storm-induced seiches, wave set-up and others

2.72

swell

wind-generated waves that have advanced out of the wave generating area and are no longer affected by winds

2.73

tidal currents

alternative or circulating currents associated with tidal variation

NOTE Tides and tidal currents are generally strongly modified by the coastline.

2.74

toe

lowest part of sea- and port-side breakwater slope, generally forming the transition to the seabed

2.75

total sample method

method of estimating extreme wave heights by extrapolating a distribution of all the wave heights measured at a site of interest

2.76

tsunami

long waves with the period of several minutes to one hour and the height up to a few tens of meters, which are generated by the vertical movement of sea floor associated with a submarine earthquake, by plunging of large mass of earth into water by land slide or volcanic eruption, and other causes

2.77

uplift

upward water pressure exerted up the base of a structure or pavement due to waves, excluding buoyancy

2.78

vortex induced vibration

VIV

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

2.79

wave climate

description of wave conditions at a particular location over months, seasons or years, usually expressed by the statistics of significant wave height, mean or significant wave period, and wave direction

2.80

wave induced currents

currents in the nearshore zone, which are induced by the horizontal gradient of wave energy flux being attenuated by wave breaking

2.81**wave pressure**

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

2.82**wave set-up**

rise of water level near the shoreline associated with wave decay by breaking

NOTE Wave set-up may amount to more than 10 % of the offshore significant wave height.

2.83**wave transmission coefficient**

ratio of the height of waves transmitted behind a structure to the height of incident waves

2.84**wind waves**

waves generated by and/or developed by wind

2.85**wind driven current**

currents induced by the wind stress on the sea surface

NOTE In coastal waters, wind driven currents are influenced by the bottom topography and the presence of the coastline.

2.86**wind set-up**

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

3 Symbols

$H_{1/3}$	significant wave height or the average height of highest one-third waves
$H_{\max}$	highest wave height
H_{m0}	significant wave height estimated from wave spectrum
m_n	n -th moment of wave spectrum such as m_0 and m_2
$T_{1/3}$	significant wave period
T_m	mean wave period
$T_{m0,2}$	mean wave period estimated from the zero-th and second moments of wave spectrum
T_p	period corresponding to the peak of frequency wave spectrum

4 Basic variables for actions from waves and currents**4.1 Water levels****4.1.1 Tides**

The astronomical tide levels at a design site shall be calculated with the tidal constituents obtained through the harmonic analysis of a long-term tide record at the site or those estimated from a nearby tide station.

The highest and lowest water levels that have occurred at or near the site should be taken into account in the evaluation of the actions from waves and currents.

The datum level for maritime structures shall be established with reference to the International Marine Chart Datum and/or the national geodetic datum levels.

4.1.2 Storm surges and tsunamis

The characteristics of storm surges at a design site should be duly investigated and be taken into consideration in evaluation of the action of waves and currents.

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.

Sets of storm surge water levels and/or storm tides should statistically be analysed for extreme distribution functions so as to determine R -year storm surge levels.

In the locality where the action of a tsunami is not negligible, tsunami characteristics at the site should be duly investigated by means of data collection and hindcasting of tsunamis in the past, and/or numerical evaluation of hypothetical tsunamis in the future.

4.1.3 Joint probability of waves and high water level

Evaluation of the action of waves should be made with due consideration for the joint probability of wave height and water level, especially at a site where the water is relatively shallow and breaker heights are controlled by the depth of water under influence of the tide.

The wave measurement data obtained at the location where the largest wave height is limited by the water depth should not be used for extreme statistical analysis for the estimation of storm wave conditions at the water deeper than the site of measurements.

4.2 Waves

4.2.1 Wave heights and periods

The characteristic heights of wind waves and swell for evaluation of the action of waves should be the significant wave height $H_{1/3}$ and the highest wave height $H_{\max}$, which are defined by the zero-crossing method in the time domain analysis. Other definitions of wave heights may be used as the characteristic wave heights when a method of evaluation requires the use of such wave heights. The significant wave height may be estimated from the zero-th moment of wave spectrum, m_0 , as being equal to $4,0 m_0^{1/2}$. When this estimation is employed, the symbol H_{m0} should be used instead of $H_{1/3}$ so as to clarify the estimation method of the significant wave height, because they may differ by several percent or more (see B.1.2).

The characteristic periods of wind waves and swell for evaluation of the action of waves are the significant wave period $T_{1/3}$ and the mean period T_m , which are defined by the zero-crossing method in the time domain analysis, and the spectral peak period T_p , which is obtained from the frequency-domain analysis. The mean period may be estimated from the zero-th and second moments of wave spectrum as being equal to $(m_0/m_2)^{1/2}$. When this estimation is employed, the symbol $T_{m0,2}$ should be used so as to clarify the estimation method of the mean wave period, because the spectrally estimated mean period is generally smaller than the individually counted mean period.

Because of the random nature of wind waves and swell, the heights and periods of individual waves in a given sea state are distributed over broad ranges of variation. Statistical distributions of individual wave heights and periods should be taken into consideration when evaluating actions from waves in shallow water (see B.1).

4.2.2 Wave spectrum

Characteristics of wind waves and swell may also be represented with the directional wave spectrum, which is expressed as the product of the frequency spectral density function and the directional spreading function (see B.2).

When evaluating the action of waves, the information on the wave spectrum being employed should be clearly stated.

The extent of the directional spreading of waves becomes narrower in shallow water than in deep water because of the wave refraction effect. This change should be taken into consideration when evaluating the action of waves in shallow water.

Where wind waves and swell coexist, wave spectra exhibit multiple peaks. Wave heights may be estimated from the zero-th moment of the wave spectrum (see B.1.2). 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 of waves of multi-peaked spectra can be made by calculating contributions of components, constructed by superimposing the spectra of wind waves and swell in question.

4.2.3 Statistics of extreme sea state

Statistics of extreme sea state at a specific site should be established on the basis of instrumentally measured wave data and/or hindcasted wave data, coupled with necessary refraction/shoaling analysis, which cover the duration as long as possible and not less than 15 y (see B.4.1).

The method of wave hindcasting should have successfully been calibrated with several storm wave data by instrumental measurements around the site of interest.

Caution should be taken for the water depth at which waves have been measured, because a shallow water depth imposes an upper limit to the largest wave height owing to wave decay by breaking.

The preferred method of producing the data set of extreme waves is the peaks-over-threshold (POT) method. The annual maximum method may be employed, but the use of the total sample method is discouraged.

When estimating the wave height corresponding to a given return period, the confidence interval to account for sample variability should be evaluated and reported.

The wave period associated with the return wave height can be determined by referring to empirical joint distributions of wave height and period of extreme wave data.

The highest wave height corresponding to a given return period can be estimated from the result of extreme statistical analysis for the significant wave height, by converting the latter to the former on the basis of the Rayleigh distribution of individual wave heights and the wave transformation analysis.

4.2.4 Wave transformation

4.2.4.1 General

Waves undergo various transformation processes while travelling from deep water toward the shore. The processes include wave shoaling, refraction, diffraction, reflection, transmission, breaking and others. When waves propagate into a region with currents of appreciable strength, the wave heights and direction change. Considerations to be given to these wave transformation processes are described in 4.2.4.2 to 4.2.4.8.

4.2.4.2 Wave shoaling

The process of wave shoaling may be evaluated using the linear wave theory. The shoaling coefficient of wind waves and swell can be calculated by means of either the monochromatic wave method or the spectral method, because the difference between the results by the two methods is a few percent at most.

When evaluating wave loading on structures however, it is preferable to take into account the wave non-linearity effect that can cause a large increase of wave height beyond the prediction by the linear wave theory.

4.2.4.3 Wave refraction

Wave transformation by refraction should be evaluated by the directional spectral calculation. For preliminary analysis however, the calculation with monochromatic waves can be employed for the cases of simple bathymetry because of a relatively small difference between the two calculation methods for such cases (see B.5.2).

4.2.4.4 Wave diffractions

Wave transformation by diffraction behind barriers such as islands and breakwaters shall be evaluated using the directional spectral calculation. Diagrams of multidirectional random wave diffractions can be referred to for the purpose of preliminary analysis. Care should be taken for the directional spreading characteristics of wind waves and swell at the site of interest, because they are the governing factor of random wave diffraction.

When it is expected that wave diffraction takes place in association with wave refraction over shoals, an appropriate method of numerical analysis and/or hydraulic model tests should be employed (see B.5.3).

4.2.4.5 Wave reflection and transmission

The coefficients of wave reflection and transmission of a maritime structure can be estimated by means of hydraulic model tests and/or the knowledge gained through model tests of similar structures in the past.

The influence of reflected waves on harbour tranquillity, structural stability and others should be examined when evaluating the action of waves.

4.2.4.6 Wave breaking

Decay and variation of wave height caused by breaking in the nearshore zone shall be evaluated by taking into account the random nature of waves.

The nearshore zone is characterized by gradual changes in the functional shape of wave height distribution, rise of the mean water level (called wave set-up) and its long-period fluctuation (called the surf beat) by wave actions, and non-zero wave height at the initial shoreline of zero depth. A numerical model for random wave breaking in the nearshore zone should be capable of reproducing such features.

4.2.4.7 Wave transformation by currents

Changes in the heights and directions of waves by currents depend on the current strength and the angle of encounter. Appropriate numerical models and/or hydraulic model tests should be used to evaluate these changes when changes are expected to be significant.

4.2.4.8 Other transformations

Other processes of wave attenuation by bottom friction, soft subsoil damping, and others may be taken into account as necessary when evaluating the action of waves.

4.2.5 Wave crest elevation and wave kinematics

4.2.5.1 Wave crest elevation

The height of a wave crest above the still water level is larger than one half of the wave height owing to the non-linear nature of water waves. Non-linear wave theories and/or reliable laboratory test data should be referred to when estimating the crest elevation of design waves. The theory and/or laboratory data of monochromatic waves may be applied to the highest individual wave of random waves for estimation of highest wave crest elevation.

4.2.5.2 Wave kinematics

The wave kinematics, or the orbital velocities and accelerations of water particles under the action of waves, should be evaluated by means of non-linear wave theories of high accuracy, because the linear wave theory underestimates the orbital velocities especially around the wave crest.

When waves are expected to break at a location at which the action of waves are to be evaluated, special consideration should be taken when evaluating the kinematics and the form of the waves because they can be quite different to those of non-breaking waves. Use of hydraulic model tests and/or advanced numerical models is recommended for the evaluation.

4.2.5.3 Wave and current kinematics

When currents of appreciable strength coexist with waves, the vector sum of the current velocity and the orbital velocities of particles by waves may be employed in evaluating the kinematics of water particles.

4.3 Currents

4.3.1 General

Currents may have an effect on structures, directly and indirectly. Directly they exercise the drag and lift forces on the structure. Indirectly they interfere with the waves and modify the wave kinematics and thus affect the actions from waves and currents. Thus the current-wave interactions should be considered when evaluating the action of waves and currents unless the currents are weak.

Currents in coastal waters may be divided into tidal currents, wind-driven currents, density-driven currents and wave-induced currents.

Currents in coastal waters may be affected by the current in the adjacent ocean. The current velocities are in general stronger in coastal waters than in the deeper oceans.

4.3.2 Current velocity

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.

Current velocities at a design site should preferably be investigated by field measurements for a sufficiently long duration time. Where tidal currents are not negligible, measurements should be made at several elevations in the water, because current velocities vary vertically.

When field measurements are not feasible, numerical computations may be carried out for gaining information on currents. However, calibration of the computations model should have been made with the field measurement data at several sites in the region of the same coastal waters.

5 Wave and current action on structures

5.1 Wave action on mound breakwaters

5.1.1 Definitions

Mound breakwaters are characterized by a seaward sloping front and a porous structure. The rear side might be a slope, a vertical face structure or reclaimed land. While the core is most often made of relatively small size wide-graded stone material, the slope surfaces are generally armoured with larger well-sorted rocks or concrete blocks of various shapes. Core and armour layers are separated by filter layers. A monolithic concrete crown wall, sometimes fully or partly sheltered by armour blocks, is used for the crest when access roads are needed or by some other reason.

Berm breakwaters are a special type of rubble mound breakwater which allow a certain degree of deformation of slope surfaces under wave action and reshape themselves to gain stability against further wave actions. A berm is formed around the mean sea level on the seaward side. A horizontal berm is built at construction stage and is allowed to reshape into an S-shape.

5.1.2 Types of wave action

Waves break on the sloping front resulting in loading on the armour units, run-up, run-down as well as related pore pressure variations and porous flow inside the structure.

The impact of waves on a breakwater depends on the stage of instability of the waves, i.e. the actions from non-breaking, breaking and broken waves are different given the same significant wave height and wave steepness.

5.1.3 Wave action on seaward armour units

The stability of armour units on slope surfaces against the effect of wave actions shall be examined. The wave action on the seaward armour layer is affected by the wave reflection from the structure. The armour stability increases with the increase in porosity and permeability. Further, a low-crested structure where significant overtopping occurs experiences less wave loading on the armour units than a structure with a high freely extending crown wall exposed to direct wave actions, other things being equal.

Due to the complexity of the flow around each armour unit, it has not been possible to evaluate the wave action's effect on individual armour units. Instead the required mass for the individual armour units to be stable has been determined through some semi-theoretical concepts leading to formulae with unknown coefficients. These coefficients have been determined through model tests and in some cases prototype observations.

Assessment of armour stability may be based on semi-empirical formulae (see D.1) for less exposed structures falling into the validity range for the formulae, provided that the uncertainties of the formulae are taken into account. Otherwise hydraulic model tests should be performed.

The stability of a berm breakwater depends on the equilibrium of seaward slope shape against the action of waves during its design working life. The capability of the trunk section in maintaining stability may be checked with empirical formulae but should preferably be examined by hydraulic model tests (see D.2).

5.1.4 Wave actions on seaward toe

The stability of a seaward rubble mound toe shall be examined in relation to its supporting function. The stability is mainly affected by the wave-induced flow during run-down. In case of non-depth limited waves, the most critical situation will generally be at low water levels. Special consideration should be taken where breaking waves can act directly on the toe and where seabed scour can endanger the stability of the toe.

Empirical formulae for assessment of toe block stability, based on model tests, can be used within their validity range for standard toe solutions where no wave breaking takes place on the toe (see D.1). However, model tests are generally recommended.

5.1.5 Wave overtopping

The effect of overtopping water and spray should be considered in relation to the function of the breakwater and the activities on and behind the breakwater. Attention should be given to wave transmission, danger to traffic and vessels moored behind the breakwater and damage to infrastructure and goods on hinterlands.

Empirical formulae based on model tests for assessment of average overtopping discharge can be used within their validity range (see D.1). Where overtopping is a critical factor it is recommended to perform model tests, because the existing formulae have large uncertainties and provide no information on the distribution in time and space of the overtopping water.

5.1.6 Wave action on rear slope armour

The stability of the rear slope armour layer should be considered. Overtopping water hitting the rear slope might cause damage to the slope and thus endanger the stability of the breakwater crest. Large pore pressure gradients can enhance the effect by causing a push-out load on the rear side surface blocks. This effect is usually enhanced by the presence of crown wall structures.

Assessment of rear slope armour stability should in general be based on model tests due to the lack of reliable formulae.

5.1.7 Influence of wave action on geotechnical failures

Wave loading on the slopes together with wave-induced pore pressures in the mound, and on and in the seabed, should be considered when examining the stability against geotechnical failures.

Wave loading can be approximated by the weight of water under a wave crest above the design water level, being negative if under a trough, or by the wave pressure acting on the slopes.

Pore pressures in the mound and in the seabed can be assessed by the use of calibrated numerical models. Pore pressures in the mound can be assessed by the use of model tests with due consideration of scale effects.

5.1.8 Wave action on crown walls

A freely standing part of a crown wall can experience impulsive wave loads when directly exposed to wave actions, 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 (nose for reduction of overtopping). For assessment of the overall stability of the crown wall against sliding and slip failures, the most critical simultaneous combination of the front loading and the uplift forces should be identified.

Impulsive pressures on the crown wall front are characterized by high peak values of very short duration. The influence of impulsive pressures on the overall stability of the crown wall should be evaluated with due consideration of the dynamic interactions with the foundation. Moreover, the influence of significant air inclusions in the breaking waves hitting the crown wall might be taken into account when transforming model test loadings into prototype.

For the assessment of strength of the crown wall, the load distributions giving the largest tensile stresses in the various parts of the structure should be used. The load results from wave-induced pressures, wave loads transmitted through armour units resting against the wall, dead loads from armour units resting against the wall, and reaction forces from the foundation.

The loadings on crown wall should preferably be determined from model tests with due consideration of 3-dimensional effects caused by obliqueness of the waves and of scale effects. Formulae for wave loads of crown walls are of empirical character and based only on 2-dimensional model tests (see D.1).

5.1.9 Wave action on filter layers

Filter layers should be designed to resist migration of finer mound materials into coarser mound materials caused by wave-induced pressure gradients. Empirical formulae for gradation of filters, based on physical tests and prototype observations, can be used (see D.1).

5.1.10 Stresses in armour units

Wave action imposes direct hydraulic loads on armour units with subsequent movements like rocking and displacements of some of the units. This, together with gravity forces, impose stresses which result in breakage when stresses exceed the material strength. The problem, which is more pronounced for slender types of unreinforced concrete units, should be examined in the design.

Calibrated semi-empirical formulae for stresses can be used (see D.1).

Abrasion and breakage of armour rocks can occur where wave action causes repeated movements of the rocks. The long-term performance should be considered for such cases.

Stones for building berm breakwaters should be hard and have sufficient resistance to crushing, because stones on the seaward side are forced to roll over large distances by wave actions and are subject to abrasion and crushing.

5.1.11 Seabed scour due to waves and currents

Waves and currents may cause scour close to and around structures on an erodible seabed. The scour depth should be considered and scour protection should be provided as necessary.

5.2 Wave action on vertical and composite breakwaters

5.2.1 Definition of vertical and composite breakwaters

A vertical breakwater is a structure of rectangular or nearly rectangular cross-sections having a vertical or nearly vertical front wall extending directly from the seabed or built on top of a thin bedding layer. A composite breakwater is a combined structure with a main body of rectangular or nearly rectangular cross-section, placed on a rubble mound that is submerged at all tidal levels. The main body is made of masonry works, concrete block works or a reinforced concrete caisson filled with sand, etc. A superstructure made of placed-in concrete is constructed on top of it. The cross-section of superstructure can be different from a rectangular shape and have a sloped front or other shape. The rubble foundation has berms in front and rear of the main body, and the berms and slopes are covered with armour units for protection against wave action.

The front of the main body can be covered with a mound of concrete blocks to reduce the action of waves on the main body and reduce wave reflection.

A front part of the main body can have an open chamber that is connected with the seaside water through perforation in the front wall for the purpose of partially absorbing wave energy and decreasing the degree of wave reflection by the breakwater.

5.2.2 Types of wave action on vertical and composite breakwaters

The main action of waves on vertical and composite breakwaters are the wave pressure on the front wall and the uplift on the bottom of the main body, which govern the overall stability of the main body against sliding, overturning, and foundation failure as well as the integrity of the structural elements.

Vertical and composite breakwaters can endure a certain degree of wave overtopping without endangering their structural integrity. Composite breakwaters are susceptible to the instability of armour units on rubble foundations caused by wave-induced flows in front of their main bodies. Wave-induced flow may cause scour of the seabed at the toe of the rubble mound, which can cause damage to the rubble foundation.

5.2.3 Wave pressure, uplift and buoyancy

The action of waves on the main body of a vertical or composite breakwater should be evaluated for the wave of largest height among random trains of individual waves.

The wave pressure and uplift exerted upon the main body should be evaluated by means of hydraulic model tests or appropriate calculation models, e.g. the extended Goda formula (see E.2). The wave pressure on its rear wall may need to be considered if the waves diffracted from the head towards the rear of the breakwater are not of negligible height. Care should be taken for the difference between the wave pressures for the evaluation of overall stability and those for the design of structural elements. The buoyancy of the immersed part of the main body below the design water level shall be taken into account in the stability analysis of the breakwater.

Under a certain combination of wave conditions, breakwater geometry and bathymetric features, impulsive breaking wave pressure can be exerted on the breakwater. The impulsive pressure is characterized by high peak intensity and a very short duration. When the action of impulsive pressures on the whole or structural elements of the main body needs to be considered in the design of breakwaters, it should be evaluated by taking its duration into account together with its peak intensity.

5.2.4 Wave overtopping

Depending on the use of the area behind a breakwater, the amount of wave overtopping and the height of waves transmitted behind the breakwater should be examined by determining the crest elevation of a vertical or composite breakwater and the configuration of its superstructure.

5.2.5 Wave action on armour units of rubble foundation

Armour units covering the surface of a rubble foundation shall have the capacity to remain in position under the actions from waves. The minimum mass of armour units required should preferably be determined by means of hydraulic model tests. Three-dimensional tests are recommended for examination of the stability of armour units around a breakwater head. For empirical formulae for estimation of the required mass, see E.3.

5.2.6 Influence of wave action on geotechnical failures

The main body of a vertical or composite breakwater is subject to the wave pressure on its front wall, superstructure and the rear wall, the uplift on its bottom, and the buoyancy to its immersed part. These loadings, together with its self-weight, give an eccentric and inclined load on the seabed or the surface of the rubble foundation. The eccentric and inclined load can cause geotechnical failures within the rubble mound and/or through the seabed foundation when the bearing capacity of the rubble mound and the seabed foundation is insufficient.

Appropriate methods for stability analysis against slip failures should be employed to ensure the safety of vertical and composite breakwaters (see E.4).

5.2.7 Seabed scour due to waves and currents

Waves and currents may cause scour close to and around structures on an erodible seabed. The scour depth should be considered and scour protection should possibly be provided.

5.3 Wave actions on coastal dykes and seawalls

5.3.1 Coastal dykes

5.3.1.1 Definition

Coastal dykes are man-made sloped soil structures parallel to the shore to protect the hinterland against erosion and flooding. They may be sea dykes along coastal shorelines and estuary dykes in a river estuary. These dykes are characterized by mild slopes on the seaward side and on the shoreward side. Very often, the seaward and/or shoreward sides of the dykes have a berm which provides access for maintenance, and/or reduces wave run-up and wave overtopping (seaward side). Coastal dykes are generally built of sand and/or clay and are covered by different materials such as grass, asphalt, stone or concrete revetments, etc.

5.3.1.2 Types of wave actions

Wave and current actions on coastal dykes include:

- wave loading (including impact loading by breakers) on the seaward slope of the dyke;
- wave run-up and run-down on the seaward slope of the dyke (including layer thickness and velocities);

- wave overtopping over the dyke crest;
- infiltration due to wave run-up and run-down in the dyke core;
- internal pressure on an impermeable top layer (with open toe) on the seaward side of the dyke (in case of a high water level inside the dyke).

Wave actions on coastal dykes shall be evaluated for the design wave height and wave period (see 4.2) and for a design water level (DWL) taking into account the water depth and tidal variations in front of the dyke. If relevant, the influence of the shape of wave spectra should be accounted for.

5.3.1.3 Wave action on seaward slope

The run-up height of the seaward slope is the governing factor for the determination of the crest elevation. This run-up height shall be assessed with due consideration of wave breaking (breaker index). The run-up and run-down flow (layer thickness and velocity), infiltration, phreatic water level in the dyke body, wave induced uplift forces underneath the revetment or cover layer, and wave impact loads on the seaward side of the dyke should be considered (see Annex F) when relevant.

5.3.1.4 Wave action on seaward toe

The stability of a seaward toe at a sea dyke should be examined to ensure the support of the dyke body. Therefore, the stone mass required should be accounted for by hydraulic model tests, empirical formulae or experience. Wave-induced run-up and run-down during lower water levels will mainly affect the stability of the toe and thus need to be considered (see Annex F). In addition, the width of the toe may be important in relation to the scour hole and hence should be accounted for.

5.3.1.5 Wave overtopping

Wave-induced overtopping over the dyke crest shall be assessed since overtopping waves can induce damage to the shoreward side of the dyke. Empirical formulae can be used to predict mean and peak overtopping discharges (see Annex F). Damage to the dyke body or hazards resulting from wave overtopping can be avoided when the dyke is designed for allowable overtopping rates taking into account local conditions of the dyke and the hinterland. Where overtopping is a critical factor, hydraulic model tests are recommended since individual overtopping rates, which induce damage to the dyke body, differ significantly from mean overtopping rates.

5.3.1.6 Wave action on dyke crest and shoreward slope

The crest height of the dyke shall be sufficiently large to prevent wave overtopping rates higher than the allowable rate under the design condition. Overtopping or overflowing water may result in considerable damage of the rear slope of the dyke and may eventually lead to total collapse of the dyke body. Hydraulic model tests should be performed to assess the stability of the rear slope by providing information on the mean wave overtopping rate, individual overtopping volumes per wave, the layer thickness and velocity of the overtopping bore on both the dyke crest and inner slope. Empirical formulae to assess these parameters are given in Annex F.

5.3.1.7 Influence of wave action on geotechnical failures

Geotechnical failures may be induced by wave action on the dyke and therefore need to be examined as they affect the overall stability of the dyke body. Semi-empirical and analytical models are available to account for these processes (see Annex F).

5.3.1.8 Seabed scour due to waves and currents

Waves and currents may cause scour close to and around structures on an erodible seabed. The scour depth should be considered and scour protection should be provided as necessary.

5.3.2 Seawalls

5.3.2.1 Definition

Seawalls are onshore or foreshore structures generally parallel to the shoreline. They are built as vertical face structures such as gravity concrete walls, steel or concrete sheet pile walls, and stone filled cribworks or as sloping structures with revetment typically made of concrete slabs, concrete armour units or rock armour. 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.

5.3.2.2 Classification and structural components of seawalls

With respect to the effect of wave actions, two main categorizations of seawalls should be distinguished:

- a) sloping or vertical seawalls;
- b) porous or non-porous seawalls.

Irrespective of the type of seawall, three main structural components shall be distinguished:

- the body (which includes the front face and the core);
- the toe;
- the crest (which includes the rear face).

5.3.2.3 Types of wave action

Wave action on seawalls shall be evaluated for the design wave height and wave period (see 4.2) and for a design water level (DWL) taking into account the water depth and tidal variations in front of the seawall. The crest height should be evaluated under due consideration of the total allowable overtopping rate and/or the wave run-up height.

The following factors associated with the stability of the structure components including body (front face, filter layer, core), toe and crest (different types of crests and rear face) shall be considered:

- a) horizontal wave forces (positive and negative);
- b) wave up-lift forces;
- c) hydraulic stability of revetments, including filters;
- d) internal water pressure in the body and seepage flow.

5.3.2.4 Wave reflection

Seawalls will reflect some proportion of the incident waves and these reflections may have significant impact on the wave pattern and the sediment transport on the coastal regime in front of the sea wall. The influence of wave reflections from seawalls should therefore be investigated. Reflection coefficients can be estimated from empirical and semi-empirical formulae for perpendicular wave attack (see Annex F).

5.3.2.5 Wave action on seaward slope

Wave action on the seaward slope of a seawall are to a large extent dependent on the type of armour used for its protection. Wave climate and type of protection shall be examined together, and armour material shall be selected accordingly. Hydraulic model tests may be required to assess the stability of the selected material on the seaward slope. Empirical or semi-empirical formulae can also be used (see Annex F).

Wave-induced run-up and run-down need to be estimated since excessive run-up leads to severe overtopping over the wall crest and both run-up and run-down may cause erosion damage on the seaward slope. Empirical and semi-empirical formulae are given in Annex F.

5.3.2.6 Wave action on seaward toe

The main purpose of the toe structure of seawalls is to prevent undermining of the body of the seawall. Failure of the toe may lead to total collapse of the seawall so that wave actions on the toe should be assessed in hydraulic model studies.

5.3.2.7 Wave overtopping

Wave-induced overtopping over seawalls may induce significant damage to the structure itself or any objects behind the wall or may generate hazards for people living or working immediately behind the structure and should thus be examined in detail. Due to the complexity of some seawall geometries where no overtopping formulae are available, hydraulic model tests or numerical modelling may be required to assess the mean overtopping rate as well as individual overtopping volumes. Semi-empirical formulae based on hydraulic model tests and prototype investigations are given in Annex F.

5.3.2.8 Wave-induced forces

Seawalls with a vertical or steep front face may be exposed to wave loadings that might endanger the overall stability of the wall. Depending on the foreshore and berm geometry, seawalls might experience both pulsating and impulsive wave loadings. Therefore, the type of wave action should be identified, followed by thorough investigation of relevant failure modes such as sliding, overturning, slip and bearing capacity failures. Critical load combinations of horizontal forces, uplift forces and high water tables shall be considered. For further relevant issues such as duration of wave impact forces and strength of the seawall see 5.1.8.

Wave-induced forces may be assessed using semi-empirical formulae for 2D situations as given in Annex F for simple geometries (vertical walls with and without berms) but need to be investigated by hydraulic model tests for more complex geometries.

5.3.2.9 Seabed scour due to waves and currents

Waves and currents may cause scour close to and around structures on an erodible seabed. The scour depth should be considered and scour protection should be provided as necessary.

5.4 Wave and current action on cylindrical members and isolated cylindrical structures

5.4.1 Definition

Cylindrical structures may be single isolated cylinders to support a platform of some kind (e.g. lighthouses) or structures composed of vertical cylindrical members supporting a platform (e.g. jetty) or a truss structure with vertical, inclined and horizontal cylindrical members, supporting a platform (e.g. oil platform) or a pipeline close to or on the seabed.

5.4.2 Types of wave and current action

Waves and currents passing a cylindrical member induce dynamic pressures different from the hydrostatic pressures on the surface of the cylinder. These pressures when integrated over the whole surface of the cylinder result in a net force on the cylinder. The wave and current actions are the drag actions, the inertia actions, diffraction actions and the vortex-shedding induced actions (vortex-induced vibrations).

5.4.3 Current action

Current action on a cylinder should be calculated with a drag force formula with an appropriate drag coefficient.

5.4.4 Wave and current action on vertical cylinders from non-breaking waves

The action of non-breaking waves and currents on small diameter cylinders and cylindrical structural elements should basically be calculated using the Morison equation with appropriate wave kinematic formulation and appropriate drag and inertia coefficients.

The action of non-breaking waves on large diameter cylinders and cylindrical structural elements may be calculated using wave diffraction theories, by numerical methods or hydraulic model tests.

5.4.5 Wave and current action on vertical, inclined and horizontal cylinders from breaking waves

Wave and current action of breaking waves on vertical, inclined cylinders in shallow water and on reefs should be calculated using appropriate formulae. Special considerations should be made regarding any wave slamming action. Hydraulic model tests should be carried out on important and critical structures on rapidly varying bathymetry.

5.4.6 Slamming action of waves on horizontal and inclined cylinders

For horizontal and inclined cylinders entering and leaving the water surface as waves pass by, slamming action should be considered.

5.4.7 Wave action on decks

Wave action, including slamming, on platform or jetty decks, shall be considered when there is a possibility of the deck being hit by waves.

5.4.8 Wave action on small diameter pipelines

Wave action and current action on small diameter pipelines on or close to the seabed should be calculated using a Morison type equation for the horizontal force and a similar equation without the inertia term for the lift force.

5.4.9 Current and wave-induced vibrations

For slender structures, e.g. pipelines in free spans, current and wave-induced, vortex-shedding induced vibrations shall be considered from the strength and the fatigue resistance point of view.

5.4.10 Seabed scour at cylinders due to waves and currents

Waves and currents may cause scour close to and around structures on an erodible seabed. The scour depth should be considered and scour protection should be provided as necessary.

5.5 Wave interaction with floating breakwaters

5.5.1 Definitions

Floating breakwaters are moored floating objects installed to reduce the height of waves approaching harbours, marinas, etc. The cross sectional form of the object may vary. The most commonly cross sectional form is the rectangular form, thus forming a box type structure of a certain length. Several such boxes may be placed in a row thus forming a breakwater of required length. Other cross sectional forms are also used.

The wave height reducing function by floating breakwaters is mainly dependent on the ratio wave length:width of the breakwater. Floating breakwaters are thus mainly used in relatively sheltered areas where only relatively short period waves are present. On open coasts with ocean waves and swell, the floating breakwater concept is not economical. To give any reasonable wave-reducing effect, the size of the breakwater will be excessively large on the open coast. An exception to this is the case in which reduction of only the short period waves of the wave spectrum is of significance, e.g. protection of fish farming devices.

The moorings of a floating breakwater are normally wires and/or chains, which are fixed at the seabed to anchors, piles or concrete units. These moorings often have non-linear force-motion characteristics.

5.5.2 Types of wave and current action

Wave action on a floating breakwater induces forces on the floating "box" and in the moorings of the breakwater. Hence the dynamic system "floating breakwater/mooring" lines shall be considered. The bottom anchors, piles or concrete units should provide sufficient resistance against the tension from the mooring lines.

The floating breakwater will undergo motions in its six degrees of freedom: surge, sway, heave, roll, yaw and pitch. The wave transmission, the wave forces on, and the motion response of, the breakwater should be calculated by numerical methods or hydraulic model tests, taking into account the force-motion characteristics of the mooring system.

Slowly varying wave drift forces induce generally large horizontal motions and are the governing factor of the mooring forces of floating breakwaters.

Current actions should be calculated with a drag force formula. The value of the drag coefficient should be appropriately selected by taking the draft:water depth ratio into consideration.

If necessary combined wave-current actions should be considered.

5.6 Wave action on wave screens

5.6.1 Definition

Wave screens include a wide range of structures consisting of one or more thin vertical walls used to form a fixed or rigid breakwater to protect a harbour or marina from wave actions. Wave screens consist of thin wall panel sections, used to limit wave actions, which are then attached to a supporting structure, usually a pier or separate pile-supported structure. Wave screens can be solid or permeable, and extend to mid-depth with a gap at the bottom or to the seabed without a gap. Other names for these structures include wave barriers, wave fences, wave "skirts", baffle breakwaters and curtain walls.

5.6.2 Type of wave action

The main action of waves on wave screens are the oscillatory wave pressures on the front and back of the screen. These integrated pressures result in horizontal forces and related moments which have to be taken by the panels and support structures. Wave loading is generally due to wind-waves or boat-generated waves.

5.6.3 Wave forces

Horizontal wave forces on a wave screen may be computed by an integration of dynamic pressures from mathematical models for wave-structure interactions, by verified empirical formulae or by extrapolation of results from physical model tests. Wave forces should be evaluated for the wave of largest height among trains of individual waves for the design wave conditions (see Annex I). Wave screens may be considered "brittle" compared to vertical or rubble mound breakwaters and due consideration should be taken when selecting safety factors. Vertical loads due to buoyant forces shall also be included.

5.6.4 Wave slamming action

Wave screens are usually designed for situations where slamming loads do not occur and where wave loading is periodic in nature. Wave slamming action from breaking waves should be examined in physical model tests if such conditions are expected to occur.

5.6.5 Wave transmission, reflection and overtopping

The degree of wave transmission under or through the wave screen, the wave reflection and the degree of overtopping, should be examined when determining the draft, porosity, and crest elevation of a wave screen. Effects of waves diffracted from the ends of wave screens should also be considered. Wave interactions with wave screens should be examined using mathematical models or hydraulic model tests. An evaluation should consider irregular waves and should consider both low and high water level conditions.

5.6.6 Seabed scour at wave screens due to waves and currents

Waves and currents may cause scour close to and around structures on an erodible seabed. The scour depth should be considered and scour protection should be provided as necessary.

6 Probabilistic analysis of performance of structures exposed to action from waves and currents

6.1 Examination of uncertainties related to wave and current action

Action from waves and currents is classified as variable action by the definition given in ISO 2394. The related basic variables are the hydraulic parameters characterizing water levels, waves and currents, given in Clause 5. They are all random variables given by probability distributions, except the astronomical tide level which can be predicted accurately once the time and the location are specified. However, it should be treated as a random variable when the times of wave and current action are uncertain. For design of coastal structures, the action of waves and currents appear in calculations either as hydrodynamic loading in terms of pressure on the structures, or only indirectly as a response of the structure. The first case applies to loading on walls and piles, whereas the second case applies to structural response such as rubble mound armour stability and integrity, seabed scour, and to hydraulic response such as wave run-up and overtopping, wave transmission and reflection. In both cases the methods of estimation of action and the response to action contain uncertainty besides the random variability of the hydraulic basic variables. Major sources contributing to the uncertainties are as follows (see J.1 for discussion).

- Statistical variability of basic natural variables.
- Errors related to measurement, hindcast or visual observation of hydraulic basic variables.
- Sample variability due to limited sample size of the available data of basic variables.
- Choice of distribution as a representative of the unknown true long-term distributions for hydraulic basic variables.
- Variability of parameter estimation for a distribution function fitted to basic hydraulic data.
- Accuracy of models for prediction of storm surge water levels.
- Accuracy of models for wave forecasting and/or hindcasting.
- Accuracy of models for transformation of waves and currents, for example from deep to shallow water.
- Accuracy of models for prediction of action of waves and currents.
- Accuracy of models for prediction of structural and hydraulic response.
- Reliability of the results of physical model tests for the estimation of loadings and structural and hydraulic response.
- Variability of structural parameters.

The bias and standard deviation of each factor of uncertainty should be investigated and be duly taken into account in the evaluation of the action of waves and currents.

6.2 Reliability assessment of structures

Structures subject to the action of waves and currents should be assessed for their reliability at the serviceability and ultimate limit states with due consideration for their economic and social functions, environmental influences, and the consequences of failure. The nature and extents of the uncertainties in 6.1 should be duly taken into account when assessing the reliability of structures during their design working life.

All failure modes and failure mode interactions of importance for the performance of the structure should be considered.

The probability of failure during the design working life should preferably be assessed and confirmed to be less than the minimum value assigned to a specific class of structure, which is to be preset or approved by responsible agencies.

The probability of failure may be evaluated by the use of the reliability index method or with direct calculation by numerical integration of their probability density functions or Monte Carlo simulations.

For a structure that permits a certain degree of deformation at the serviceability and ultimate limit states, the expected amount of deformation should preferably be evaluated.

NOTE A caisson breakwater can function as a wave barrier even after its main body has slid over a small distance. A berm breakwater allows reshaping of its seaward slope by wave actions until it reaches a stable shape.

Annex A (informative)

Water levels

A.1 Tide levels

Because the astronomical tide level at any location varies continuously with time, certain representative tide levels are used to provide the information on the range of tide levels. Most frequently used ones are the mean high (or low) water springs, which is the average height of high (or low) water at the time of spring tides obtained over a long period of time. It is equal to the elevation above (or below) the mean sea level by the amount equal to the sum of the largest two constituents of the same family, usually the M_2 and S_2 . Because the heights of two high (or low) waters in a day usually differ, they are not the highest (lowest) water levels at spring tides. The near highest high water level is calculated by adding the amount equal to the sum of the amplitudes of the four main tidal components M_2 , S_2 , K_1 and O_1 to the mean sea level. The near lowest low water level is calculated by subtracting the same amount from the mean sea level. The latter is also called the low water Indian springs and has been used as a chart datum in some countries before the International Hydrographic Organization resolved to adopt the lowest astronomical tide (LAT) as the chart datum.

In addition to the above high (or low) water levels, the mean monthly highest (or lowest) water level, which is the annual mean of the highest (or lowest) water levels within five days after the day of new or full moon in respective months, may be used as representative of the high (or low) water levels of tides.

The water level at a given location can vary beyond the range of astronomical tide levels due to the 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.

A.2 Design water level

When evaluating the action of waves and currents on structures, the water level at the time of their action needs to be specified. It is called the design water level. The principle of selecting the design water level is to use the combination of waves, currents and water level which is most unfavourable to the stability and/or safety of the structure and/or facilities under design.

For a certain type of structure such as retaining walls, exceptionally low water at the ebb of tsunami may cause its seaward collapse owing to the earth and residual water pressures behind it. When scour of the seabed in front of a structure is apprehended, a low water level can become a critical condition. Impulsive breaking wave pressures may be exerted on a vertical or composite breakwater when the water level is intermediate or low, depending on the geometry of the breakwater. However, most cases of structural design set the design water level at a rather high elevation.

The methodology of selecting the design water level at high elevation is not established yet, depending on the available data, design practices in use, local situations and others. Several methods are cited in a) to d).

a) Use of the highest record of high water level

This method is sometimes used for the case of tsunamis with the addition of a certain allowance, because the statistical analysis of tsunami heights is difficult owing to the rare occurrence of tsunamis.

b) Extreme statistical analysis of storm surge levels

This can be done for both the absolute level of the highest water above the datum level and the deviation of highest water level from the astronomical tide at the time of storm surges. The analysis can yield the

return water level corresponding to a designated return period. When the R -year storm surge deviation 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.

c) Numerical simulation of the past worst storm surge or tsunami, around the locality

The storm surge or tsunami that inflicted the worst damage on 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. Selection of the astronomical tide level when a design storm passes depends on the analyst in charge.

d) Probabilistic analysis

All the basic variables related to design of structures including the water level are given 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.

Annex B (informative)

Wave action parameters

B.1 Wave variability and wave parameters

B.1.1 Variability of waves in the sea

Wind waves and swell in the sea are essentially random in time and space. Heights and periods of individual waves in a short time span vary randomly over wide ranges. It is necessary to define individual waves with some criteria and calculate characteristic wave heights and periods in a statistical manner in the time domain analysis. The zero-upcrossing or zero-downcrossing method is the standard technique to define individual waves, in which the two successive crossing points of the wave surface profile with the mean water level (zero-line) mark the start and the end of one individual wave, respectively. The significant wave height $H_{1/3}$ is the mean of the highest one-third waves thus defined.

The state of wave activities on the surface of the sea varies gradually with time. The condition of the sea surface in a short time span is termed the sea state, which is generally expressed with the significant wave height and the mean wave period or other period parameter, and the principal wave direction. Statistical analysis of the sea state over months, seasons and years yields a long-term description of wave conditions at a particular site, which is called the wave climate. The subject is discussed in Clause B.3.

Analysis of individual waves of a wave record is generally made for a time span of 20 min or so. The length of time span is chosen as the result of compromise between the requirement of a short duration to guarantee the constancy of sea state during the recording time and that of a long duration to have a sufficient number of waves necessary for reliable estimates of significant wave height and other characteristic wave heights and periods.

B.1.2 Marginal distribution of individual wave heights in a short time span

The distribution of individual wave heights in a short time span can be approximated using the Rayleigh distribution. The Rayleigh distribution is approximately applicable to any sea state inclusive of wind waves, swell, and coexisting state of both, regardless of frequency spectral shapes; e.g. see Goda^[88] (pp. 40-41 and pp. 261-265).

The assumption of the Rayleigh distribution enables the estimation of significant wave height from spectral information as below

$$H_{1/3} = H_{m0} = 4,0 \eta_{\text{rms}} = 4,0 \sqrt{m_0} \quad (\text{B.1})$$

where η_{rms} denotes the root-mean-square surface elevation and m_0 is the zero-th moment of frequency wave spectrum defined by

$$m_n = \int_0^\infty f^n S(f) df \quad (\text{B.2})$$

where $S(f)$ denotes the frequency spectral density function.

Although the Rayleigh distribution provides a good approximation to the wave height distribution, the latter in deep water is slightly narrower than the Rayleigh, resulting in the empirical relationship of $H_{1/3} \cong 3,8 \eta_{\text{rms}}$ or $H_{1/3} \cong 0,95 H_{m0}$. In relatively shallow water outside the nearshore zone, on the other hand, the wave height distribution becomes as broad as or broader than the Rayleigh owing to the enhanced effect of wave non-linearity.

In the outer part to middle of the nearshore zone where waves begin to break randomly owing to depth-controlled limitation to large wave heights, the marginal distribution of wave heights becomes much narrower than the Rayleigh. As waves proceed toward the shoreline however, the distribution of wave heights becomes broad again owing to the stationary rise of the mean water level (wave set-up) and its temporal variation (surf beat).

B.1.3 Highest wave height in relation to significant wave height

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. Its ensemble mean is the function of the number of waves within duration of a given sea state and expressed relative to the significant wave height as given below.

$$\left(\frac{H_{\max}}{H_{1/3}} \right)_{\text{mean}} \cong 0,706 \left[\sqrt{\ln N_0} + \frac{0,577}{\sqrt{\ln N_0}} \right] \quad (\text{B.3})$$

where N_0 denotes the number of waves.

The most probable value is given by the first term of the right-hand side of Equation (B.3).

Equation (B.3) is based on the Rayleigh distribution of wave heights. There are some reports in which extremely large waves were observed with heights beyond what might be expected from the statistical distribution of $H_{\max}/H_{1/3}$. These waves are called freak waves. It is an unsettled question whether the probability of freak wave appearance remains within a range predicted by the theory of Rayleigh distribution of wave heights or if they are caused by some effects not accounted for in currently available theories of wave statistics.

B.1.4 Distribution of individual wave periods in a short time span

The distribution of individual wave periods needs to be examined in the light of the joint distribution between wave heights and periods, because the characteristic wave periods such as the significant wave period are defined in association with the wave heights ranked by their magnitudes. The joint distribution of individual wave heights and periods is strongly affected by the functional shape of the frequency wave spectrum. No reliable theoretical model is available for the joint distribution of wave heights and periods for versatile spectral shapes. 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.

For wind waves with single-peak spectra, the following empirical relationships hold for the wave periods defined by the zero-upcrossing or zero-downcrossing method.

$$T_{\max} \cong T_{1/3} \cong 1,2T_m \quad (\text{B.4})$$

where $T_{\max}$, $T_{1/3}$, and T_m denote the periods of highest wave, significant wave, and the mean wave period, respectively. The relationship between the significant wave period and the spectral peak period is affected by the spectral shape. For fully-grown wind waves, the mean relationship of $T_{1/3} \cong 0,9T_p$ is applicable. Further information can be found in Table 2.4 of Goda^[88].

The mean wave period can be estimated from the frequency wave spectrum with the zero-th and second moments as below.

$$T_m \cong T_{m0,2} = \sqrt{m_0 / m_2} \quad (\text{B.5})$$

where the spectral moments m_0 and m_2 are calculated by Equation (B.2).

Equation B.5 is based on the condition that the wave spectrum is comprised of all the free linear frequency components. Because actual waves contain a certain amount of non-linear spectral components in the high frequency ranges, the mean period $T_{m0,2}$ estimated by Equation (B.5) becomes shorter than the mean period T_m calculated by the zero-crossing technique; the difference is enhanced in shallow water and can reach up to 20 %.

B.2 Wave spectra

B.2.1 Directional wave spectra

A detailed structure of wind waves and swell is represented with the directional wave spectrum, which can be expressed as the product of the frequency wave spectrum $S(f)$ and the directional spreading function $D(\theta|f)$. The term f denotes the frequency and θ is the azimuth measured from some fixed axis of direction.

The frequency wave spectrum or the frequency spectral density function, $S(f)$, expresses the distribution of wave energy density (being divided by the unit weight of water) in the frequency domain and has the dimension of m^2/s or equivalent units. The directional spreading function, $D(\theta|f)$, expresses the distribution of wave energy density in the directional domain at a specific frequency f , 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.

Details of directional wave spectra can only be examined through multiple-component wave measurements. However, some standard functional forms are available for evaluation of the actions from waves as described in B.2.2 to B.2.4. Directional spectral functions of wind waves and swell can be expressed with the input of characteristic wave height, period and direction. For the sea state of coexisting wind waves and swell, the respective spectral densities can be linearly superposed so as to yield the directional wave spectrum of the combined sea state.

Spectral analysis of wind waves and swell is based on the concept of linear superposition of component waves. When the characteristic 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. Use of wave spectra in very shallow water should be made with due caution for the effect of non-linear components on wave actions.

B.2.2 Frequency spectra of wind waves and swell

Various functional forms of frequency wave spectra have been analysed on the basis of many field measurements, and several standard functions have been proposed. Some frequency spectra are expressed as the function of wind speed for the purpose of wave forecasting. They include the Pierson-Moskowitz, the JONSWAP and the TMA spectra. For evaluation of the action of waves, it is more convenient to express a frequency spectrum as the function of characteristic wave height and period.

For fully-grown wind waves, the Bretschneider-Mitsuyasu frequency spectrum, as follows, can be employed:

$$S(f) = 0,257 H_{1/3}^2 T_{1/3}^{-4} f^{-5} \exp[-1,03 (T_{1/3} f)^{-4}] \quad (\text{B.6})$$

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 $H_{1/3} = 4,0 \sqrt{m_0}$ and the relationship of $T_p = 1,05 T_{1/3}$, which was derived by Mitsuyasu based on his field measurements of wind waves with the presence of some low frequency components.

For versatile functional shapes of single-peak wave spectra, the modified JONSWAP, as follows, is often employed:

$$S(f) = \beta_J H_{1/3}^2 T_p^{-4} f^{-5} \exp[-1,25 (T_p f)^{-4}] \gamma^{\exp[-(T_p f - 1)^2 / 2\sigma^2]} \quad (\text{B.7})$$

in which

$$\beta_J = \frac{0,0624}{0,230 + 0,0336\gamma - 0,185(1,9 + \gamma)^{-1}} [1,094 - 0,01915 \ln \gamma] \quad (\text{B.8})$$

$$T_p \cong T_{1/3} / [1 - 0,132(\gamma + 0,2)^{-0,559}] \quad (\text{B.9})$$

$$\sigma \cong \begin{cases} 0,07 & : f \leq f_p \\ 0,09 & : f \geq f_p \end{cases} \quad (\text{B.10})$$

in which $f_p = 1/T_p$ denotes the frequency at the spectral peak.

The term γ is called the peak enhancement factor, which was given the value between 1 and 7 in the original proposal of the JONSWAP spectrum. In using Equation (B.7) however, the value of γ can be chosen between 1 and 10, depending on the wave characteristics. For fully-grown wind waves, $\gamma = 1$ is appropriate because it yields the frequency spectrum equivalent to the Pierson-Moskowitz spectrum. For swell, the value should be increased in proportion to the distance of travel, but $\gamma = 10$ can be regarded as appropriate for swell having travelled over several thousand kilometres (see Goda^[88], p. 30).

Equation (B.7) is due to Goda^[86], who also derived the functional form of the modified Wallops spectrum having a free parameter of the exponent of the power of frequency.

B.2.3 Directional spreading function

B.2.3.1 Standard directional spreading functions

Various functional forms have been proposed for the directional spreading function of wind waves. Some functions are based on the analysis of field data, while some others are without validation by field data. Due to an insufficient number of field measurements of directional wave spectra, selection of a directional spreading function is left to the judgment of an analyst of the wave action. In this subclause, two functional forms are introduced, but other functional forms can also be used in the analysis.

The directional spreading function based on field data is the Mitsuyasu type as follows:

$$D(\theta|f) = D_0 \cos^{2s} \left(\frac{\theta - \theta_0}{2} \right) \quad (\text{B.11})$$

in which

$$D_0 = \left[\int_{\theta_{\min}}^{\theta_{\max}} \cos^{2s} \left(\frac{\theta - \theta_0}{2} \right) d\theta \right]^{-1} \quad (\text{B.12})$$

where θ_0 denotes the principal wave direction measured from a given axis of direction and s is the spreading parameter that varies with the frequency as in the following:

$$s = \begin{cases} (f/f_p)^5 s_{\max} & : f \leq f_p \\ (f/f_p)^{-2,5} s_{\max} & : f \geq f_p \end{cases} \quad (\text{B.13})$$

According to Mitsuyasu et al.^[155], the peak value of spread 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^[88] (p. 34) and OCDI^[163] (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 case of deep-water waves.

The directional spreading function employed in some multi-directional model tests is the wrapped-normal type as expressed below.

$$D(\theta) = \frac{1}{2\pi} + \frac{1}{\pi} \sum_{n=1}^N \exp \left[-\frac{(n\sigma_\theta)^2}{2} \right] \cos n\theta \quad (\text{B.14})$$

in which σ_θ represents the angular standard deviation defined by

$$\sigma_\theta^2 = \int_{\theta_{\min}}^{\theta_{\max}} (\theta - \theta_0)^2 D(\theta) d\theta \quad (\text{B.15})$$

The number of serial terms, N , should be sufficiently large to ensure convergence of the finite series.

B.2.3.2 Mutual relationships among standard spreading functions

The Mitsuyasu type spreading function dictates the frequency dependency of directional spreading as expressed by Equation (B.13), while the wrapped-normal spreading function assumes directional spreading independent of frequency. However, mutual comparison between these spreading functions is possible by means of the angular standard deviation σ_θ . When the overall directional energy spreading of the Mitsuyasu type function is calculated by the integration of the directional spectrum with respect to the frequency, the spectrum with $s_{\max} = 10$ approximately yields $\sigma_\theta = 33^\circ$, while $s_{\max} = 75$ yields $\sigma_\theta = 14^\circ$. For more details of mutual comparison of several directional spreading functions, see Goda^[87].

B.2.3.3 Directional spreading functions in shallow water

Owing to wave refraction effects, directional spreading of wind waves and swell becomes narrow in shallow water compared with that in deep water. Numerical analysis has been made for the refraction of directional random waves on a coast with straight, parallel depth-contours. A directional wave spectrum composed of the Bretschneider-Mitsuyasu frequency spectrum and the Mitsuyasu type spreading function was employed in the analysis. From the result of this analysis, an equivalent value of spreading parameter $s_{\max}$ was evaluated as a function of the ratio of water depth to the deepwater wavelength. A diagram for the estimation of the equivalent value of the spreading parameter has been prepared and is listed in Goda^[88] (p. 36) and OCDI^[163] (p. 39).

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.

B.2.4 Representative height and period of combined sea state

When a sea state is composed of wind waves and swell, the directional wave spectrum exhibits multiple peaks at different frequencies and azimuths. If the information of the characteristic 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 can be estimated using the formulae in the previous subclauses. The directional wave spectrum of the combined sea state is obtained by linearly superimposing the directional spectral densities of these wind waves and swell. The action of these waves is then analysed by calculating contributions of components of the directional wave spectrum thus obtained.

The representative height of the combined sea state can be estimated as follows:

$$H_s = \sqrt{H_1^2 + H_2^2 + \dots + H_n^2} \quad (\text{B.16})$$

where H_s denotes the characteristic wave height of combined sea state and H_1 to H_n are the characteristic heights of wave groups. Any definition of wave height such as the significant wave height or the highest wave height is applicable to this formula, because the distribution of wave heights of the combined sea state can be approximated by the Rayleigh distribution as discussed in B.1.2.

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^[163] (p.70). It was proposed by Tanimoto et al.^[218] for the purpose of evaluating wave loading on vertical face breakwaters.

B.3 Wave climate statistics

B.3.1 Statistical representation of wave climate

As mentioned in B.1.1, a long-term description of wave conditions at a particular site is called the wave climate. Continual wave measurements carried out every few hours provide the basic source of wave climate. The significant wave height is the characteristic height of wave climate statistics, while the characteristic wave period is either the mean period, significant wave period or spectral peak period, depending on the method of wave record analysis.

Wave climate at a particular site can be described in many ways. Time-history diagrams of characteristic height, period and direction over a month, a season or a year visualize a general trend of wave climate at a particular 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 characteristic 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 also important in assessing tranquility of a harbour basin and littoral sand transport rate along a coastline. Duration statistics of calm seas and rough seas are also examined for analysis of the workability and operational efficiency of maritime facilities.

B.3.2 Marginal distributions of characteristic wave height and period

The Weibull and log-normal distributions are often fitted to the marginal distributions of characteristic wave height and period of wave climate statistics. Because the sea state varies gradually, the wave heights and periods measured at intervals of several hours are mutually correlated. Even with a time lag of 24 h, the correlation coefficient between successive significant wave heights can maintain a value of 0,3 or higher. Thus the data set for the marginal distribution of characteristic wave height or period does not constitute a sample of statistically independent data.

The upper tails of the marginal distributions often exhibit 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. A simple extrapolation of a marginal distribution of significant wave height for an estimation of extreme wave height, such as 100 year wave height, which is called the total sample method, should not be made in the evaluation process for design wave heights because of the inherent inaccuracy involved.

B.3.3 Joint distribution of characteristic wave height and period

The pattern of the joint distribution of characteristic wave height and period is highly dependent on the nature of wave climate at locality. 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 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. Caution should be taken against simple application of theoretical models such as double log-normal distributions to the joint distribution data of characteristic wave height and period.

A joint distribution of characteristic wave height and period over many years can be convoluted using the Rayleigh distribution for individual wave heights so as to yield the marginal distribution of whole individual wave heights or actions during the design working life of a maritime structure. 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 using the characteristic wave period. These waves are given respective heights according to the Rayleigh 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 of interest, further refinement can be achieved.

B.4 Extreme wave statistics

B.4.1 Data set for extreme wave analysis

The database for extreme wave analysis is a long record of instrumental wave measurements and/or results of wave hindcasting projects. The accuracy of wave hindcasting is affected by the reliability of both the hindcasting model itself and the meteorological information for wind field estimation. It is necessary to employ 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.

The length of data record is preferably 30 y or longer. A long record is needed so as 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 y.

The measured and/or hindcasted extreme waves should preferably be classified according to types of meteorological disturbance so that the data sets can be constructed for respective storm types. Extreme wave data of respective storm types may constitute samples from different populations of extreme waves. When an extreme wave analysis is made on a data set of mixed populations, the prediction of extreme wave height may not be reliable.

A set of extreme wave data can be prepared by two methods. One is to take the maximum significant waves in every year, and the other is to take the waves at a peak of every storm event that is defined by the level by which the significant wave height exceeds a preset threshold level. The former is called the annual maximum method, and the latter is the peaks-over-threshold (POT) method. 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. Therefore, the POT method is a preferred technique of data analysis. It should be remembered that 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.

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 storm types.

B.4.2 Extreme distribution functions for storm wave heights

Because no consensus has been established on the population distribution of storm wave heights and the POT wave data are not the extreme data in a strict sense, several distributions are employed as the candidates for fitting to the data set of extreme wave heights.

Commonly employed distributions in extreme wave analysis are the Fisher-Tippett type I (double exponential or Gumbel), the Fisher-Tippett type II (Frechét) and the Weibull distributions (see Chapter 11 of Goda^[88] for their functional forms). However, other distributions such as the Generalized Extreme Value and log-normal distributions can also be used.

B.4.3 Data fitting and selection of extreme distribution function

A data set of extreme wave heights, or a sample, is fitted to a candidate distribution for parameter estimation. The least squares method (LSM), the maximum likelihood method (MLM), and other valid methods may be employed for distribution fitting. When applying the LSM, the shape parameter of the Fisher-Tippett type II or the Weibull distribution is often fixed at a predetermined value so as to transform it into a two-parameter distribution. Care should be taken to employ the non-bias plotting position formulae for distribution functions when using the LSM.

Appropriate criteria of best fitting and/or rejection should be chosen and applied for the data set, depending on the methodology of data fitting.

B.4.4 *R*-year wave height and confidence interval

Once the distribution best fitted to the data set is selected, the distribution is assumed to represent the population of extreme wave heights at the site of interest. The wave height corresponding to a given return period, or *R*-year wave height, can be estimated by a standard procedure of extreme statistics.

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 locality. Even with absence of climatic changes, a data set covering different but equal lengths of time will constitute a sample of the same distribution but with different statistical characteristics. This is called the sample variability of data set. Because of this variability, each sample will yield different estimates of *R*-year return wave height. A range of confidence interval should be estimated and indicated for every estimate of return wave height, even though the methodology of estimation is left to analysts.

Furthermore, there is no way to know the true population distribution of storm wave heights in general. A misfit of an extreme wave data set to a distribution different from the true population will yield a bias in the estimate on *R*-year return 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.^[93]).

B.4.5 *R*-year height of highest wave

The highest wave height corresponding to a given return period is usually estimated from the *R*-year return significant wave height by multiplying it with a certain factor based on the Rayleigh distribution or others.

In offshore engineering, efforts are often made to estimate the *R*-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 Rayleigh distribution, as discussed in B.3.3.

B.4.6 Wave period associated with *R*-year wave height

Information on a wave period associated with the *R*-year return wave height is often needed when evaluating the action of waves. However, no established method is currently available to estimate such a wave period. Often a joint distribution of storm wave heights and periods is prepared to find out a meaningful correlation between the height and period.

For fully-grown wind waves in deep water, the following mean relationship can be quoted:

$$T_{1/3} \cong 3,3 H_{1/3}^{0,63} \quad (\text{B.17})$$

in which the units of $T_{1/3}$ are seconds and those of $H_{1/3}$ are metres. The above relationship is due to Goda^[90], based on Wilson's^[267] formula for wind wave forecasting.

B.5 Wave transformations

B.5.1 Processes of wave transformation

During the propagation of waves and swell, they experience various processes of wave transformation, by which the height, period, direction and spectrum are changed. The processes mainly considered in evaluation of the actions from waves and currents are shoaling, refraction, diffraction, reflection, transmission and breaking. Wave shoaling denotes the process of changes in wavelength, wave celerity, wave height, etc. when waves propagate in water of decreasing depth. Wave refraction is the process by which wave direction and height change when the waves propagate obliquely to the depth contour. Wave diffraction is the phenomenon that waves propagate into the geometric shadow zone behind a barrier. When waves encounter a man-made or natural barrier to their propagation, waves are partially or fully reflected and there may be some waves transmitted behind the barrier. These are the phenomena of wave reflection and transmission. When the height of a wave becomes large beyond a certain threshold, which is expressed either in terms of the ratio of wave height to wavelength or the ratio of wave height to water depth, the wave cannot maintain its kinematic stability and loses a part of its energy through breaking.

When waves encounter currents, they are refracted by them if the directions of the waves and the currents are different. When waves meet the opposing currents, wave heights increase and wavelengths are shortened. When waves propagate riding on the following currents, wave heights decrease and wavelengths are elongated.

Most of these processes are linear in the sense that the wave height after transformation is linearly proportional to the wave height before a transformation, and are analysed by linear wave theories. When the relationship between the wave heights after and before a transformation is not linear, the process is called non-linear. Wave transformation by breaking is a typical non-linear process. Wave shoaling exhibits some non-linear features. The effect of wave non-linearity on wave refractions is small and neglected in most analyses.

Estimation of wave heights, periods and directions by wave transformations involves a certain degree of uncertainty due to the variability of transformation processes and the reliabilities of estimation models. When evaluating the action of waves, such uncertainty should be taken into consideration.

B.5.2 Wave shoaling

B.5.2.1 Linear shoaling coefficient

The linear wave theory provides the basis for calculating the wavelength, celerity, group velocity and shoaling coefficient in shallow water. The shoaling coefficient denotes the ratio of the height of waves having been affected by the depth change in shallow water to their height in deep water with the refraction effect eliminated. The shoaling coefficient by the linear wave theory is calculated with a closed-form function of the ratio of the water depth to the local wavelength.

For shoaling of random waves, the shoaling coefficient is first computed for a range of spectral frequency components and the results at various frequencies are utilized to construct the frequency spectrum of waves in shoaling water. The characteristic wave height in shoaling water is estimated from the zero-th moment of resultant wave spectrum by Equation (B.1) and the random shoaling coefficient is calculated by using the spectrally evaluated wave heights. The shoaling coefficient of random waves differs from the shoaling coefficient of monochromatic waves, but the difference is a few percent at most in many cases.

B.5.2.2 Nonlinear shoaling coefficient

When waves with a large height or non-linear waves propagate into quiet shallow water, the wave profile takes the form of a sharp crest and flat trough. The potential and kinetic energies for a given wave height become less than those of linear waves. Conversely, non-linear waves can have a height larger than the height of linear waves for the same energy flux. Because of this feature, the shoaling coefficient of non-linear waves becomes larger than that of linear waves. Several theories are available for evaluation of the non-linear shoaling coefficient, among which the theory by Shuto^[197] is often referred to. A diagram based on this theory is listed in BS^[31], Goda^[88] (p.77), and OCDI^[163] (p.75).

The non-linear shoaling coefficient should be employed when evaluating wave loading on structures, because prediction of such wave loading is usually made with the input of local wave height having been affected by non-linear shoaling. However, an increase of wave height by non-linear shoaling beyond the linear shoaling process does not represent a net increase of wave energy density, as explained in the above. Therefore, the non-linear shoaling coefficient should not be used in the calculation of wave energy flux, radiation stresses, longshore currents and other energy related phenomena.

B.5.3 Wave refraction

Upon entering a region of shallow water, wind waves and swell undergo the process of wave refraction together with wave shoaling. Changes in the direction of wave propagation and wave height are often analysed using the wave ray method or equivalent methods by computer, which have been developed for regular (monochromatic) waves with a single period and direction. In the coast of simple bathymetry, such methods of wave refractions analysis can be utilized for the purpose of preliminary analysis. In principle however, wave refraction should be analysed for multidirectional random waves with the input of directional