

Maritime structures —

Part 7: Guide to the design and construction of breakwaters

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- British Ports Federation and the National Association of Ports Employers
- British Steel Industry
- Concrete Society
- Department of the Environment (Property Services Agency)
- Department of Transport (Marine Directorate)
- Federation of Civil Engineering Contractors
- Health and Safety Executive
- Institution of Civil Engineers
- Institution of Structural Engineers
- Oil Companies International Marine Forum

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Publication(s) referred to	Inside back cover

Foreword

This Part of BS 6349 has been prepared under the direction of the Civil Engineering and Building Structures Standards Policy Committee.

This Part of BS 6349 consists of six sections providing guidance for the design and construction of breakwaters as follows.

- *Section 1: General;*
- *Section 2: Layout planning;*
- *Section 3: General design of breakwater structures;*
- *Section 4: Rubble mound structures;*
- *Section 5: Vertical face structures;*
- *Section 6: Composite structures.*

It has been assumed in the drafting of this British Standard that the execution of its provisions is entrusted to appropriately qualified and experienced people, for whose guidance it has been prepared. It provides information and guidance, not all of which may be directly verifiable. Depending upon the extent of information and knowledge gained in this field in the coming years, it is possible that this guide could be updated as a code of practice.

The seven Parts of BS 6349 are as follows.

- *Part 1: General criteria;*
- *Part 2: Design of quay walls, jetties and dolphins;*
- *Part 3: Design of dry docks, locks, slipways and shipbuilding berths, shiplifts and dock and lock gates;*
- *Part 4: Design of fendering and mooring systems;*
- *Part 5: Code of practice for dredging and land reclamation;*
- *Part 6: Design of inshore moorings and floating structures;*
- *Part 7: Guide to the design and construction of breakwaters.*

Parts 1 to 6 have been written as codes of practice and contain recommendations on good, accepted practice as followed by competent practitioners. Part 7 has been written as a guide.

A number of the figures and tables in this Part of BS 6349 have been provided by individual organizations who own the copyright. The details of the sources are given at the foot of each figure and BSI acknowledges with appreciation permission to reproduce them.

The full list of the organizations which have taken part in the work of the Technical Committee is given on the inside front cover. The Chairman of the Committee was Mr P Lacey CEng, FICE, FIMStructE, FIHT, FRSA and the following were members of the Technical Committee.

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Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 84, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

Section 1. General

1.1 Scope

This Part of BS 6349 provides guidance on the design and construction of breakwaters.

Breakwaters are structures which provide protection to harbours and structures such as sea intakes against wave action and this Part of BS 6349 gives guidance on the main types of breakwater. Floating breakwaters are not included.

Coastal structures such as groynes, revetments and training walls are not covered, although certain aspects of design may be found to be relevant to them.

NOTE The titles of the publications referred to in this British Standard are listed on the inside back cover. The numbers in square brackets used throughout the text relate to the bibliographic references given in Appendix A.

1.2 Definitions

For the purposes of this Part of BS 6349, the definitions in BS 6349-1 apply together with the following.

1.2.1

rubble mound breakwater

a structure composed primarily of rocks dumped or placed upon the sea bed. An outer layer, or layers, of more massive rock or precast concrete units provides an armour layer to protect the less massive rock core from wave attack. A concrete crest structure which contributes to the function of the breakwater may be constructed on the mound

NOTE Examples of rubble mound breakwaters are shown in Figure 6.

1.2.2

vertical face breakwater

a breakwater in which wave attack is resisted primarily by a vertically faced structure extending directly from sea bed level

NOTE Examples of vertical face breakwaters are shown in Figure 18.

1.2.3

composite breakwater

a submerged rubble mound foundation or breakwater surmounted by a vertically faced structure projecting above sea level

NOTE Examples of composite breakwaters are shown in Figure 30.

Section 2. Layout planning

2.1 General

This section considers the planning of breakwater layout to achieve the harbour protection function. Guidance is given on navigational aspects, wave penetration, environmental effects and data collection.

2.2 Harbour layout

2.2.1 General

Wave energy can enter a harbour by penetration through the entrance between the breakwaters, by overtopping and by transmission through permeable breakwater structures. The types of breakwater structures used and their detailed design therefore influence the wave climate within the harbour, and for this reason breakwater layout cannot be entirely separated from design of the structures; an iterative process is often needed in determining the optimum solution.

Port planning requirements for the number, size and locations of cargo handling facilities will determine the overall dimensions of the harbour. These considerations are outside the scope of this Part of BS 6349. References are given in **2.1.1** of BS 6349-2:1988.

Breakwaters can also be required to protect an approach channel from littoral drift or to stabilize or train the alignment of a tidal entrance.

The siting and layout of the breakwaters to provide the necessary degree of protection to the harbour are determined by the need for the following:

- a) sheltered conditions for ships at berth or anchorage;
- b) manoeuvring and turning areas for ships within the harbour;
- c) an adequate stopping distance for ships entering the harbour entrance at a safe navigating speed.

2.2.2 Navigational aspects

Criteria for depth and width of approach channels are given in clause **18** of BS 6349-1:1984, criteria for manoeuvring inside harbours are given in clause **19** of BS 6349-1:1984, and criteria for the acceptable wave conditions for moored boats and ships are given in clauses **30** and **31** of BS 6349-1:1984. Suitable conditions should also be provided to enable tugs and mooring vessels to work satisfactorily.

The presence of the breakwaters produces special navigation conditions at the harbour entrance. Currents can be generated across a harbour entrance as a result of the deflection of currents and by wave diffraction around the head of the breakwater. Wave reflections can occur from the breakwaters, and as a vessel moves from the open sea to sheltered water there are significant changes in environmental conditions affecting the vessel over a short distance.

A wide harbour entrance, to ease navigation, conflicts with the objective of limiting wave penetration, and some compromise is needed. Navigation is not always possible in exceptional wind and wave conditions.

The advice of experienced mariners is essential in determining the optimum layout of breakwaters at the harbour entrance, taking into account the economic aspects of cost and any limits on navigation and port operation.

Models and ship simulators, described in clause **18** of BS 6349-1:1984, can be valuable aids to the planning of the harbour entrance and breakwater layout.

2.2.3 Wave penetration

The most important determinant of harbour response is wave penetration through the entrance. It is first necessary to establish wave conditions just outside the entrance, then to determine the effect of the entrance in permitting waves to enter the harbour and finally to determine the response at critical positions within the harbour.

Guidance on establishing the offshore wave climate is given in clause **22** of BS 6349-1:1984, and methods of deriving inshore wave conditions at the harbour entrance are given in clause **23** of BS 6349-1:1984. Wave direction is important and, whilst the greatest shelter to the harbour area should be provided against the largest waves, lesser wave conditions from different directions can be important in designing the layout.

Consideration should be given to fairly frequent wave conditions as well as to rare events, as the former can affect down-time and economy of operation whereas the latter will affect safety. Acceptable limits on ship movement are given in **31.4** of BS 6349-1:1984.

Wave action should be investigated at different water levels caused by tide or surge effects. Water level will normally modify incident wave energy and can particularly affect wave direction at the entrance (see clause 25 of BS 6349-1:1984). Changes in sea bed contours can also have significant effects. An example would be the creation of a dredged channel outside the harbour, as referred to in 23.2.3 of BS 6349-1:1984.

Wave diffraction at the harbour entrance will determine the degree of shelter provided by the breakwaters and the spread of waves into the harbour basin. It is necessary to consider the extent to which waves can be reflected or absorbed within the harbour and, where depths vary, whether shoaling, refraction and bottom friction need to be considered in determining the harbour response. Guidance on these aspects and on the use of physical and computational models is given in clause 29 of BS 6349-1:1984.

Long period waves, over approximately 30 s, are difficult to exclude from a harbour and can cause ranging of moored ships. Long period waves can also cause harbour resonance, on which guidance is given in 29.4 of BS 6349-1:1984. This undesirable phenomenon is aggravated by the use of reflecting faces within the harbour and such faces should be avoided if possible. Absorbing faces sometimes have to be provided.

2.2.4 Wave overtopping and transmission

Wave overtopping and transmission and their effect on harbour layout and response are determined by the design of the breakwater structure. It can be very costly to prevent waves overtopping a breakwater, because increasing the height to achieve this can greatly increase the forces on the structure. The extent of overtopping which can be allowed should be considered very carefully.

A distinction should be drawn between mass overtopping and wind-carried spray. In the case of a rubble mound breakwater the mass overtopping may be prevented or controlled by appropriate design of the seaward face and crest; wind-carried spray cannot be controlled. When reclaimed areas and installations are located behind a breakwater, overtopping and wind-carried spray can cause serious inconvenience or danger to personnel and vehicles, interrupt operations and cause flooding.

Suggested limits for overtopping are given in 3.5.2.4. These relate to the passage of vehicles and people. Even quite severe overtopping will rarely have a significant effect on wave action generally within the harbour, except in the special case of a breakwater designed with a very low crest. The effect then becomes important and is referred to in 4.9 with respect to rubble mounds, and in 5.3.2 with respect to vertical face structures.

Transmission through the structure can occur with a very porous rubble mound, e.g. one constructed only of large rocks, where the degree of transmission increases appreciably with wave period. For long period waves, over approximately 30 s, the effects on harbour response can be pronounced.

Vertical face breakwaters do not permit wave transmission except in the case of perforated wave screens, which are not generally applicable in harbour works but can be used in mild wave climates such as sheltered yacht marinas.

2.2.5 Breakwater alignment

The dimensions of the harbour and alignment of breakwaters should be determined using the guidance in 2.2.1 to 2.2.4. A variety of options would usually be considered, and by taking advantage of favourable features of the coastline and sea bed topography, considerable economies can be achieved. For example, it could be possible to:

- a) site the root of the breakwater at a rocky headland to reduce the risk of scour at this location;
- b) choose a layout which will minimize the length and depth of construction for a given port area and facilities;
- c) select a breakwater position and alignment such that there is a reduction in the height of the waves which the breakwater has to resist.

The last arrangement could involve taking advantage of offshore reefs or sandbanks which would cause the higher waves to break before reaching the breakwater. In such cases it is particularly necessary to assess the effect of wave refraction, which can increase wave height due to wave concentration at some location along the breakwater. Wave height at the breakwater can also be increased by oblique wave attack causing a build-up of waves running along the breakwater.

It is important to determine whether the presence of the breakwater will cause changes, e.g. the deepening of offshore shoals which can then expose the breakwater to greater wave attack, as discussed further in 2.3.

Reflections from the seaward face of a breakwater can set up standing wave patterns which can result in increased wave attack in some sections. This effect is reduced if a convex alignment is adopted instead of a straight one. A concave curvature can create very severe wave concentrations and should be avoided.

The layout of the heads of the main and lee breakwaters are often designed to give a substantial overlap which will prevent direct penetration of the most severe waves into the harbour.

It is sometimes possible to design a main breakwater to resist the most severe wave attack and provide a lee breakwater of lighter construction as shown in Figure 1(a). This could enable harbour facilities to be concentrated along the lee breakwater, which would permit overtopping of some magnitude to be accepted along the main breakwater with consequent economy.

A different design of harbour, as shown in Figure 1(b), usually more appropriate to a river mouth where training of currents is important, has little breakwater overlap and permits more wave action to penetrate the entrance. Spending beaches inside the main breakwaters absorb a large proportion of the waves and a narrower secondary harbour entrance leads to the berths.

2.2.6 Physical and computational modelling

Much experience has been gained evaluating harbour layouts by means of physical models. Guidance is given in 29.5 of BS 6349-1:1984.

Computational models are now available for assessing the effects of different layouts. They can also be used to determine the qualitative effects of different layouts but do not necessarily give sufficiently detailed information for all aspects required. Guidance on the use of computational models is given in 29.6 of BS 6349-1:1984.

Current best practice for major projects is to use both physical and computational models.

Attention is drawn to the need for the early collection of site data to enable the model to be constructed and the test programme planned.

The physical models referred to above are often of too small a scale in a wave basin covering the whole harbour area to study breakwater structure stability. However, such models can give useful guidance on the parts of the breakwater which will suffer the most severe wave attack, and these can be used to guide the planning of hydraulic model testing, referred to in 3.6.

2.3 Environmental effects

2.3.1 General

Construction of breakwaters involves one of the largest changes which can be imposed on a coastal regime. Considerable attention should be given not only to the effects of the environment on the breakwater but also to the effects of the breakwater on the environment. Factors which could arise are indicated in 2.3.2 to 2.3.4.

2.3.2 Hydrodynamic regime and sediment transport

A breakwater will cause changes to the sea state (see clause 28 of BS 6349-1:1984). The resulting changes arising from the movement of mobile bed material by tidal or wave induced currents need careful evaluation even though considerable uncertainty attends the results. Clause 14 of BS 6349-1:1984 gives general advice on sediment transport and the limitations of both physical and mathematical modelling of the processes.

In the simplest case it is to be expected that up-drift accretion and down-drift scour will occur after construction of a coastal harbour, as indicated in Figure 1. Up-drift accretion could eventually cause the formation of a bar across the entrance, which can require maintenance dredging. Down-drift erosion could lead to loss of beaches and the need for coastal protection measures, which can extend a long way from the harbour.

The effects on the breakwater structure need to be assessed. For example shoaling can reduce wave action on the up-drift breakwater and toe scour can increase the risk of instability of the down-drift breakwater.

Where littoral drift is a major feature of the coastline and the consequences of interruption of the drift are likely to have serious effects on the adjacent shoreline it can be necessary to provide a bypassing facility for the drift material.

Bypassing can be achieved by the training of tidal currents if these are of sufficient strength or by trapping drift material where it can be pumped past the entrance or dredged and dumped to feed the beaches on the down-drift side.

2.3.3 Pollution

The creation of a coastal harbour will result in an area of water relatively undisturbed by waves and currents. As far as is practicable no major drainage sources should be allowed to discharge into the harbour as pollution and settlement of sediment could occur in the quiescent water.

Openings or culverts can be provided at suitable positions in breakwaters to increase flow inside a harbour, where the tidal range is small, e.g. in the Mediterranean Sea.

Outside the harbour the changes in the hydraulic regime due to the breakwaters can affect the dispersion of pollutants.

2.3.4 Ecological considerations

Breakwaters generally have no harmful effects on the ecology of the area unless changes they cause in the regime affect the local habitat (see **13.5.2** of BS 6349-1:1984).

2.4 Data collection

2.4.1 Meteorology and climatology

Data on wind, temperature and barometric pressure are required for breakwater design and construction, and for assessment of wave climate and extreme water levels. Details of sources of data and the meteorological and climatological considerations to be taken into account are given in clause **7** of BS 6349-1:1984.

2.4.2 Waves

The design and construction of breakwaters requires detailed knowledge of wave activity and persistence under all conditions. Data on extreme wave heights are required for design of structures, while data on seasonal and annual variations are required for layout of harbours, for assessing the effect on harbour operations and for construction planning. At some locations, particularly where there is a persistent swell, data on long period wave activity will be needed. Details of data sources, wave recording and analysis are given in clause **26** of BS 6349-1:1984.

2.4.3 Bathymetry and coastal topography

Details of sea bed and coastline are required for determining the alignment of breakwaters and assessing the effect of the sea bed and coastal features on wave propagation in the area under consideration. The extent of the area to be surveyed and the specification of the survey work needed can be determined from a study of available charts. Additional bathymetric information can be obtained from collector charts held by The Hydrographer of the Navy¹⁾. The Hydrographic Department holds records not only for the UK but also for many other countries. Similar facilities are available in other countries.

The extent of the offshore bathymetry needed for wave refraction studies is related to wave period. The area covered to seawards of the proposed breakwater should normally extend to locations where the depth is about half the wavelength. Methods of carrying out bathymetric surveys are described in clause **8** of BS 6349-1:1984.

2.4.4 Water levels

Data on variations in water levels due to tidal fluctuations and predictions of extreme water levels due to barometric effects, storm surges and wave set-up are required.

Methods of water level recording and the meteorological effects causing changes in levels are described in clause **10** of BS 6349-1:1984 and guidance on storm surges is given in clause **25** of BS 6349-1:1984.

In areas where tsunamis occur a thorough investigation of historical records is sometimes necessary to determine their severity and probability of occurrence. For data in the Pacific Ocean reference can be made to the US Weather Centre in Hawaii.

2.4.5 Water movement

Data should be collected on the velocity and the pattern of currents at the site of the breakwater so that the effect of the proposed structure on the hydrodynamic regime can be investigated. The same data is needed to study navigational aspects. Some information on currents can be obtained from the Admiralty Pilots published by The Hydrographer of the Navy and other similar bodies in other countries. Current directions and velocities also appear on charts but the information is usually insufficient for detailed planning purposes. Local fishermen and yachtsmen are often a valuable source of information, particularly during the preliminary stages of design, but quantitative information from such sources should be treated with caution.

Methods of measuring current velocities are described in **11.2** of BS 6349-1:1984.

2.4.6 Sediment transport

The effects of alterations to the hydrodynamic regime due to breakwater construction and the subsequent changes in sediment transport should be considered, as discussed in **2.3.2**.

¹⁾ The Hydrographer of the Navy, Taunton, Somerset TA1 2DN.

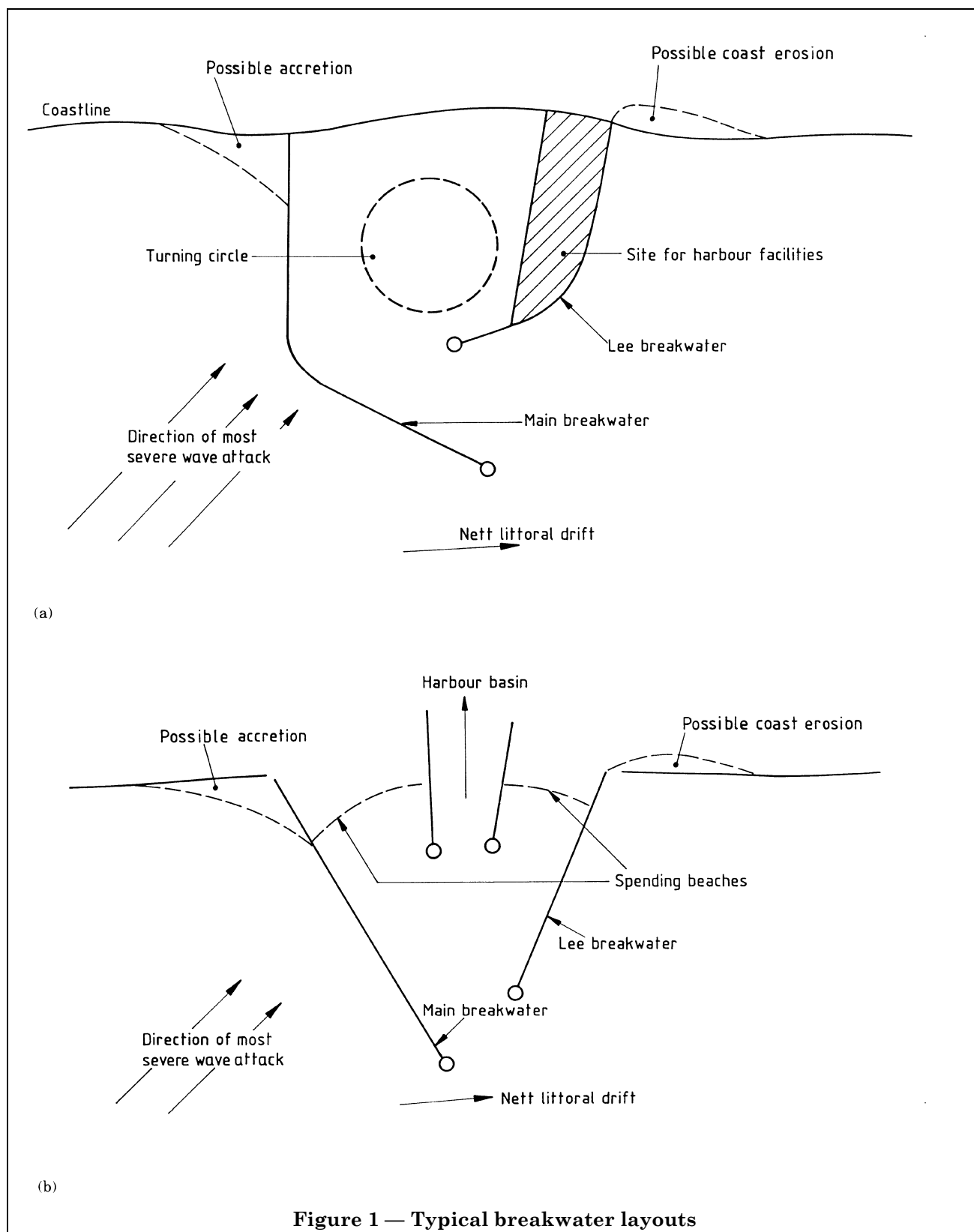


Figure 1 — Typical breakwater layouts

Comparative study of old charts, and aerial photographs, can give an indication of the extent of bed movement which has occurred in the past. Sediment transport and methods of measuring sediment load, accretion and scour due to wave transport are discussed in clause 14 of BS 6349-1:1984.

2.4.7 Geotechnical aspects

Guidance on site investigations required to determine subsurface conditions is given in clause 49 of BS 6349-1:1984. Borehole investigations should, where appropriate, be preceded by side scan sonar and geophysical surveys to obtain a preliminary picture of coverage of surface and subsurface changes throughout the area under investigation.

Careful consideration should be given to the timing, extent and detail of marine site investigations. At the early stages of design the precise location of breakwaters can be uncertain, suggesting coverage of a wide area in limited detail. For final design of the structures a narrower area requires more detailed exploration.

The sequential development of knowledge of the subsoils has to be tailored for each individual project, being dependent on the ground variability and the cost of setting up one or more stages of investigation. It is also largely affected by the remoteness of the site and the severity of wave exposure.

2.4.8 Construction materials

Large volumes of rock and concrete aggregates are usually required for breakwater construction. The potential sources and qualities of material should be determined at an early stage. The work includes the assessment of available maps, photographs and reports, followed by site investigation.

Tests for quality of primary armour rock are described in 57.2 of BS 6349-1:1984. Estimates of the yield of rock of different sizes can be made by experienced engineers and geologists and some guidance on this is given by Allsop, Bradbury and others [1]. Further information on particular aspects of rock material usage in rubble mound breakwaters is given in 4.10.1.

Section 3. General design of breakwater structures

3.1 General

This section considers the philosophy of the design of breakwater structures, the factors which affect the selection of design criteria and the derivation of design wave conditions. Considerations which affect the choice of structure type are described and the use of hydraulic models is discussed, followed by a review of risk analysis.

3.2 Design philosophy

3.2.1 General

The structural design is determined by the function of the breakwater, the site topography, environmental conditions and economic considerations. The principal factors in design are wave loading and foundation conditions.

The essential comparison of imposed wave loadings and the structural resistance to such loadings is complex. The reasons for this are that wave loads are stochastic in nature, and structural response to waves is not fully understood.

The practical approach adopted differs between vertical face and rubble mound structures, and the current design philosophy can be briefly described as follows for each case.

Vertical walls are regarded as rigid structures and are designed by a quasi-static analysis, in which an assessment is made of extreme wave conditions at the structure, from which pressures, loads and movements are computed from formulae. These applied loadings are compared with the resistance of the structure to confirm that the design has appropriate factors of safety. The uncertainty in design is largely concerned with the wave conditions and the validity of the formulae used.

For rubble mounds, which are regarded as flexible structures, there is similar uncertainty over wave conditions, but in addition the nature of wave/structure response is less well understood. The design is thus based on a concept of tolerable damage or movement of the main armour layer, using empirical relationships to assess the design of the main armour for given wave conditions. Other elements of the rock mound are empirically related to the main armour layer. There is no quasi-static concept of overall safety factors in current design philosophy, although advances are being made in the understanding of probabilistic design of rubble mounds.

The extreme wave conditions selected for the design of a breakwater must be carefully assessed in each case. It is common practice to regard a design wave as a single value of wave height with a low probability of exceedence during the intended service life or design life of the structure. However, the descriptive parameter of wave height for a given sea state can vary according to the design method used, as described in sections 4, 5 and 6. For example the maximum wave height $H_{\max}$ is usual for vertical walls, whereas the significant wave height H_s or the mean of the highest one-tenth of wave heights $H_{1/10}$ is used for rubble mounds. In addition other sea state parameters such as wave period, spectral energy, direction and whether waves are breaking are important in the design process.

It is nevertheless convenient to make the following observations generally in terms of wave height. It is the major parameter in assessing the severity of wave action, and in a particular sea area an increased wave height on a structure will generally lead to an increased probability of failure.

Failure is defined as having occurred when the breakwater no longer substantially fulfils its function of providing protection to a harbour or land area or if the cost of damage repair, including interference with commercial operations, is unacceptable. This is the ultimate limit state.

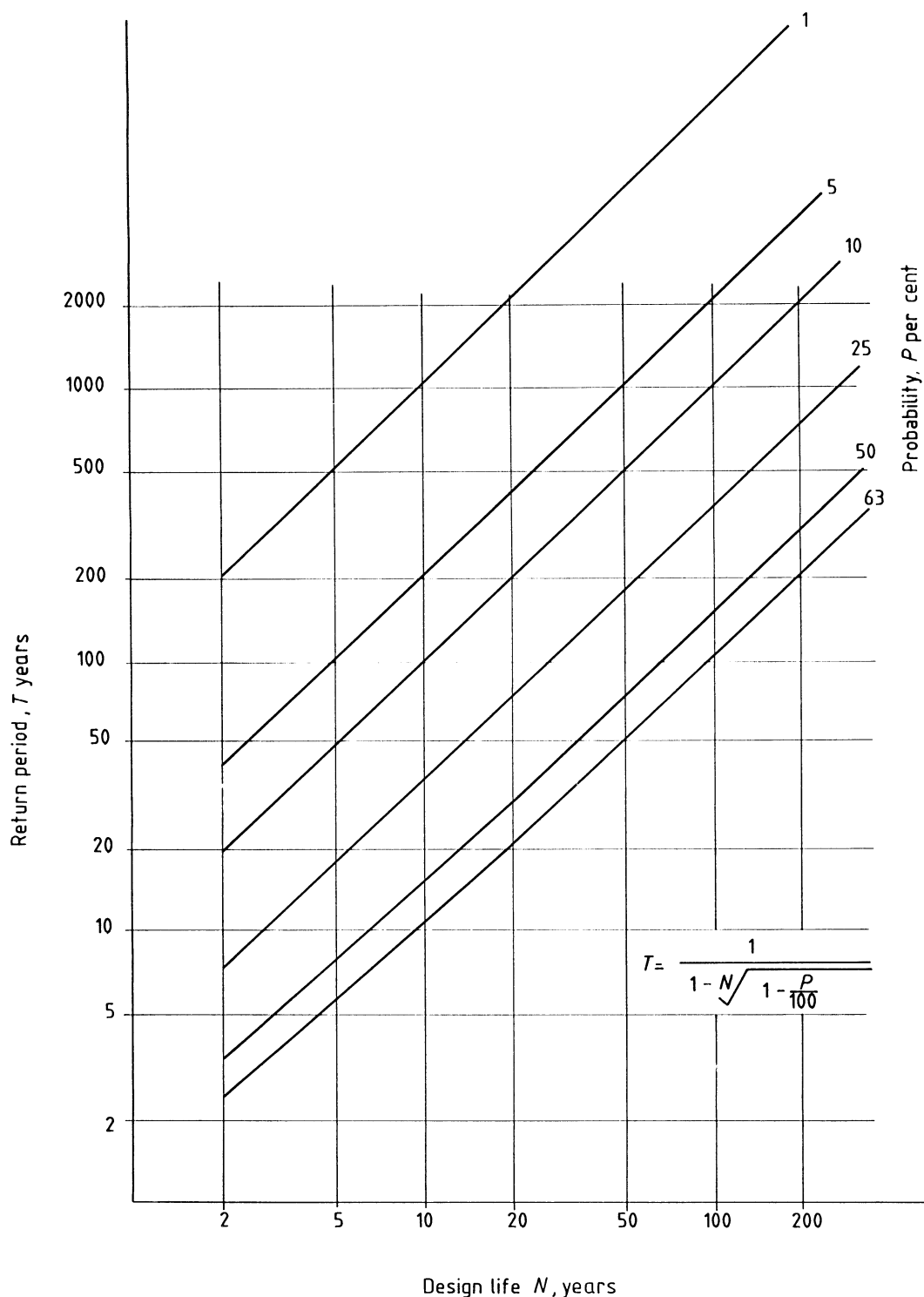
The serviceability limit state exists when damage to the breakwater of considerable magnitude has occurred but it is still possible to carry on most normal operations inside the harbour.

The acceptable probability of failure or the acceptable degree of damage during the life of the structure should be decided at an early stage of the design. The cost of the repair should be estimated and included in the assessment of the economic feasibility of the project. It will be evident that at the early stages of design this will be very imprecise, but should be improved as the project proceeds and more information becomes available.

3.2.2 The design wave

The design methods described in this Part of BS 6349 are based on the assumption that some waves that occur during the life of a breakwater may be higher than the design wave.

Design life is discussed in clause 16 of BS 6349-1:1984, but the choice should be determined chiefly by the function of the project. A service life of 50 to 100 years is often assumed, but the design wave should normally have a much longer return period for the reasons set out below.



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

Figure 2 — Relationship between design life, return period and probability of exceedence

If a breakwater is designed to resist a wave whose return period equals the design life, there is a 63 % probability that the design wave will be exceeded during the design life (see clause 21 of BS 6349-1:1984).

The relationships between design life, return period and probability of exceedence are shown in Figure 2. If a 5 % probability of the design wave being exceeded were to be acceptable for a design life of 50 years, it would be necessary for the design wave to have a return period of 1 000 years.

It is therefore necessary to balance the probabilities and consequences of damage against the costs of avoiding or reducing these risks. It is recommended that the stability of the structure should be checked under a wave which has a probability of exceedence during the design life of only 5 %. This is not necessarily a no-damage condition.

The value to be ascribed to the height (and other parameters) of a design wave with a return period considerably longer than the design life is site specific. Where there are no natural limits of wave action due to shallow water or limited fetch length, an extrapolation of return periods by the methods described in clause 27 of BS 6349-1:1984 can be used. However, generally in shallow water conditions, there can be a physical limit to wave action so that the 1 000-year return period wave can be little different from the 50-year return period wave. In determining such cases it is necessary to take account of the combined probabilities of storm waves and high water levels due to tide and surge.

3.2.3 Factors contributing to failure

Significant modes of failure are indicated in Figure 10 (for rubble mounds) and Figure 31 (for composite breakwaters). However, where failures have occurred it has frequently not been possible to identify a single cause with certainty. Factors which are believed to have contributed to failure, and which should be considered during the design of a breakwater, include the following:

- a) underestimation of design wave due to inadequate information about wave climate or designing for too short a return period;
- b) insufficient allowance for local concentration of waves due to localized feature of sea bed contour;
- c) inadequacy of design techniques and knowledge of the behaviour of the structure resulting in hydraulic instability of the structure and its component parts;
- d) inadequacies in carrying out and in interpreting the results of hydraulic model tests;
- e) geotechnical instability of the structure or its foundations;

f) insufficient control and supervision of construction, particularly in placing of underwater elements;

g) poor quality of materials used in construction, or insufficient appreciation of material behaviour in service, e.g. inadequate resistance to corrosion, abrasion, weathering; fatigue of concrete armour units and variations in quality.

3.3 Design development

Figure 3 illustrates a logic diagram for the design process from the pre-feasibility stage to the construction stage [2].

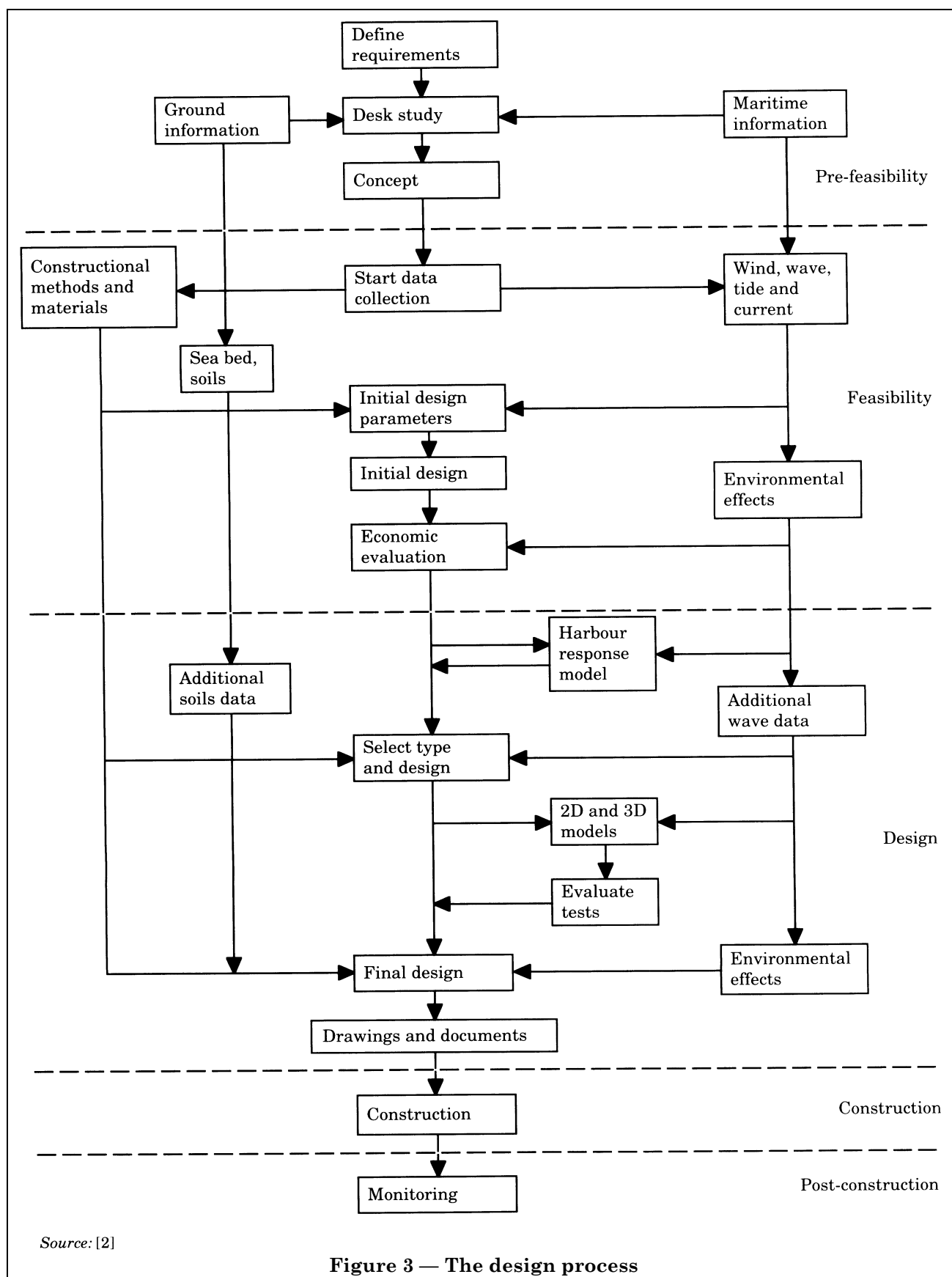
An assessment of wave climate is the first requirement as wave action is the most important consideration in design. It can be based at the pre-feasibility stage on storm wave prediction using formulae involving wind speed, duration of storm and fetch as described in 22.2.1 to 22.2.5 of BS 6349-1:1984. This is refined at the feasibility stage by analysis of additional wind records and using the results of wave records. The process is described further in 3.5.

A number of alternative preliminary designs should be prepared and compared during the period when more detailed site-specific information is collected and analysed to ensure that all required data is obtained. The main elements to be considered in design and the procedures which are available are described in sections 4, 5 and 6.

All except the simplest of breakwater designs should be based upon hydraulic model testing (see 3.6). The sections to be tested should be chosen after comparing alternative designs and selecting the optimum.

Physical hydraulic model testing is the most efficient and reliable way of determining the stability of a breakwater design and recent developments in laboratory techniques enable most hydraulic aspects of stability to be investigated. A comprehensive series of model tests should be carried out to refine the design and determine the safety of the structure under extreme conditions. Because of limitations of time and cost it is rarely possible to test all options and the test programme should be carefully prepared to obtain the greatest benefit from testing and to assist interpretation of the results.

Computational models have not yet been developed for examining the hydraulic stability of breakwater elements although there have been some advances in research into modelling the hydraulic behaviour inside a rubble mound and its effects on overall geotechnical stability.



The final stage of each design process should comprise a thorough analysis of the expected risks and consequences of damage with a view to balancing the costs of acceptable damage against capital investment required, while at the same time seeking to provide adequate factors of safety against those forms of damage which constitute failure of the project as a whole to fulfil its function.

This final design stage will take into account the extent and reliability of the data collected, the results of hydraulic model testing and its limitations, the availability and cost of construction materials, construction methods and the risk of damage and its repair. Risk analysis is discussed in 3.7 and should be reviewed fully at the final stage of design. However, the need for adequate margins of safety against ultimate failure during design life and application of the philosophy of risk analysis should be considered by the designer throughout the design process.

3.4 Design wave climate

3.4.1 Derivation of wave climate

3.4.1.1 General

Wave data are often expressed in terms of significant wave height H_s for storms of different return periods. This is now recognized as being only part of the description of wave conditions, as mentioned in 3.2.1, and wave period, spectral energy, wave direction and breaking need also to be considered.

Wave grouping effects, as described in 24.3 of BS 6349-1:1984 can occur. The long waves associated with wave grouping are of importance in harbour response, but the effect of wave groups on structures (particularly rubble mounds) is less certain.

Methods of predicting wave parameters are given in clauses 22 and 23 of BS 6349-1:1984; wave recording and analysis are discussed in clause 26 of BS 6349-1:1984 and extrapolation of wave data in clause 27 of BS 6349-1:1984. The effects of breakwaters and sea walls on sea states are discussed in clause 28 of BS 6349-1:1984.

3.4.1.2 Wind records

It is rare for long term recorded wave data to be available and short term records, which are often for only one year, are not necessarily representative of long term conditions.

Wind records can be used for hindcasting over many years to assist in extrapolating wave records and to determine whether the period of wave recording occurred during a period of low, average or high wind (and therefore wave) activity. Design wave conditions can then be determined by comparing recorded heights with the estimated wind-generated heights during the period of recording and during extreme storm conditions.

Wind records are often the only available means of determining offshore wave directions. When waves travel into shallow water their characteristics are altered and should be assessed as described in clause 23 of BS 6349-1:1984.

The UK Meteorological Office provides overland wind records which can be used to estimate overwater wind speeds. Table 1 gives approximate relationships for anemometer stations within 16 km of the coast. A check should be made to establish whether wind speeds recorded at the station are affected by major topographical features.

It should be borne in mind, however, that local winds may be quite different from those in the area where waves are generated. Wave forecasting from synoptic charts may be of greater value [3].

Table 1 — Wind speed adjustment, nearshore

Wind direction	Location of recording station	Ratio ^a
Onshore	3 km to 5 km offshore	1.0
Onshore	At coast	0.9
Onshore	8 km to 16 km inland	0.7
Offshore	At coast	0.7
Offshore	16 km offshore	1.0

^a Ratio of wind speed at location to overwater wind speed (both at 9 m above sea level).

Knowledge of offshore wind conditions is not always available for prediction of wind conditions in sufficient detail for use over long fetches. Use should then be made of the published wind and wave observations made by mariners. Information on availability of records is now held by Marine Information and Advisory Services (MIAS) and the Meteorological Office have developed methods for enhancing the reliability of visual wave data which have been validated with measured wave records. These methods are based on functional modelling of the statistical relationships between wave height, wave period and wind speed and are incorporated in a computer programme NMIMET from which the wave climate may be synthesized [4, 5].

Since the raw data observations are concentrated along shipping lanes it is essential to review the likely spatial variability at the site of the breakwater.

3.4.1.3 Wave hindcasting

Wave hindcasting can be used for estimating wave conditions from historical wind data. Methods of wave hindcasting and forecasting make use of a wind model and a wave model. The wind model for hindcasting can use as an input not only synoptic weather charts at the time of the event but also other wind observations which have been collected before or after it. For further information on the use of these techniques see Battjes [6] and Mynett, De Voogt and Schmeltz [3].

3.4.1.4 Tropical storms

Tropical storms are revolving storms with very high winds of 33 m/s (64 kn) or greater and are known as hurricanes, typhoons, tornadoes, tropical cyclones and by other local names.

A fully developed hurricane can have a diameter of 1 000 km. At the average travel rate a hurricane will take about two days to pass. The worst conditions for shipping normally last only a small fraction of the two days and the period of most severe weather, including the eye, will seldom exceed 6 h. However, the effect on build-up of waves at a breakwater can be quite different depending on the direction in which the storm is travelling. A steady build-up of waves over a number of days is possible and the aftermath of swell can persist for a long time afterwards [7]. Swell can affect areas remote from the track of the hurricane.

Tropical storms cannot be fitted into the same statistical projection as normal wind/wave records and should be considered as a separate data set.

Tropical storms approaching a coastline will have an effect on water level due to surge, wind and wave set-up.

Further information can be obtained from the *Shore Protection Manual* [8] and the Meteorological Office.

3.4.1.5 Storm duration

The duration of storm wave attack will affect the extent of damage which can occur and is particularly important for a rubble mound and should be assessed. The probability of occurrence of two consecutive storms should also be investigated. If repairs needed as a result of the first storm cannot be executed before the arrival of the second storm, this could represent a worse case than a single storm that is more severe than either of the others.

Storm duration is less important for vertical face breakwaters where the critical condition is that which causes the worst case to occur, taking account of water level.

It should be noted that the longer a storm persists, the higher the maximum individual wave will be, as described in 27.3 of BS 6349-1:1984.

3.4.1.6 Influence of water level

Since the maximum height of waves which can attack a breakwater is dependent on the depth at or near the toe, a knowledge of tidal levels and extreme levels due to storm surges is necessary. The combined probability of a storm with high water levels should be assessed. With storm surges the meteorological conditions causing the rise in water levels are sometimes but not always the same as those causing maximum wave attack. In some cases the two conditions will be independent variables; in others they can be positively or negatively related. The likelihood of occurrence can be assessed by joint probability computations [9].

In some instances the low water case can be important as the form of wave attack can alter from non-breaking to breaking, due to the reduction in water depth. This can also increase toe scour.

3.4.1.7 Tsunamis

In areas where tsunamis can occur their effect should also be considered. Although the probability of coincidence with extreme storms is very low, wave shoaling, as mentioned in 24.4 of BS 6349-1:1984 can have a severe effect on these waves and their effect on the structure can be pronounced.

3.4.2 Design wave conditions

3.4.2.1 Methods for assessment

A review of methods to establish the wave climate for breakwater design is given by Battjes [6]. The particular wave conditions which are critical in the design will depend upon the type of breakwater. Each part of the structure should be considered independently when determining design wave criteria.

3.4.2.2 Wave height

The deep water where waves at the structure are not depth limited and an unbroken wave can reach the structure, the selection of the design wave height should be based on a probability of exceedence within the design life of the structure. The probability of exceedence should be chosen taking into account the factors outlined in 3.2 and will involve both owner and designer. Extrapolation of wave data is described in clause 27 of BS 6349-1:1984.

Probabilities of waves beyond the design wave height should be predicted for use in a sensitivity analysis of behaviour under less likely conditions.

3.4.2.3 Wave spectrum

The spectrum for the design wave cannot be predicted by extrapolation from short term wave records and it will rarely be possible to record a spectrum which is relevant to extreme conditions.

The reliability of wave spectra obtained from site records should be considered in relation to the length of records and whether these include any storms approaching the design wave in height. Adoption of an arbitrary one-dimensional spectrum such as JONSWAP (Joint North Sea Wave Project) in a fetch-limited situation, or Pierson–Moskowitz where a fully developed sea occurs, is sometimes the only solution (see 22.2.5 of BS 6349-1:1984).

3.4.2.4 Wave refraction and diffraction

Both graphical and computer methods are available for determining the effects of wave refraction and diffraction (see clause 23 of BS 6349-1:1984). Back tracking will be the best method when considering wave conditions in the entrance or at the roundhead, but tracking the advancing wave can be more suitable when considering possible wave concentrations along the length of the breakwater.

Deep water incident waves are, at some sites, attenuated by diffraction around headlands. The graphical method described in 29.2 of BS 6349-1:1984 for a flat sea bed can be used to give an approximate solution. Computer methods are also available.

3.5 Choice of type of structure

3.5.1 Types of structure

1.2 defines the three main types of breakwater, the design of which is discussed with examples in sections 4, 5 and 6. In some cases general consideration, discussed in 3.5.2, will enable a decision to be made in favour of one type; in other cases it will be necessary to compare the cost of a number of designs of different types.

3.5.2 Factors affecting choice

3.5.2.1 General

Some of the main factors which will affect the choice of type of structure are described in 3.5.2.2 to 3.5.2.10. These often result in conflicting preferences, and an important part of the design process will be giving each the appropriate weight in arriving at a compromise.

3.5.2.2 Other functions

Besides providing sheltered water the breakwater can be required to protect a reclamation area, to provide a berth on the inner face or to support access and services. Both the risks of damage and its consequences are increased by designing for this type of dual function. Overtopping can be critical, and could have to be minimized at considerable extra cost.

3.5.2.3 Navigation

Wave reflections from vertical face and composite breakwaters can cause confused seas at harbour entrances, rendering navigation dangerous. Energy absorbing rubble mounds in front of vertical walls, as illustrated in Figure 26, can be used to reduce such reflections. However, mariners need to give a wide clearance to underwater slopes, where navigation width is not so clearly defined as with vertical structures.

3.5.2.4 Wave overtopping

Overtopping can be minimized with some types of breakwater by providing sufficient freeboard. In this respect rubble mound breakwaters are particularly suitable as most of the wave energy is dissipated in the armour layer before reaching and passing over the crest. With vertical face structures a column of water is thrown up by wave action with little reduction in energy, even with a high freeboard. The column could collapse partly on the structure and partly on the water area behind it, creating secondary waves. The secondary waves will have a shorter period than the incident wave.

Clause 28 of BS 6349-1:1984 discusses overtopping and Table 2 gives guidance on permissible volumes of overtopping water in relation to inconvenience or danger to personnel and vehicles.

Table 2 — Overtopping water: safety considerations

Consideration	(m ³ /m)/s
Inconvenience to personnel	4×10^{-6}
Inconvenience to vehicles	1×10^{-6}
Danger to personnel	3×10^{-5}
Impassable for vehicles	2×10^{-5}
Source: [10, 11]	

The discharges given in Table 2 are average values for personnel and vehicles about 3 m behind the wave wall; peak values can be up to 100 times the average.

3.5.2.5 *Wave transmission*

A permeable rubble mound breakwater consisting only of large rocks has good stability when partly built but has the disadvantage that waves, particularly those with periods of 20 s or more, are transmitted through the structure. As discussed in 2.2.4, the effects on harbour response need to be considered.

3.5.2.6 *Environmental effects*

The type of structure will influence the effects on the coastal regime as discussed in 2.3. The principal effects will be caused by the reflective characteristics of a vertical wall compared with the reduced reflections of waves from a rubble mound, both outside and inside the harbour.

3.5.2.7 *Foundation conditions*

Where poor foundations exist a rubble mound breakwater can be more suitable than a vertical face breakwater because it is better able to tolerate settlement. If it is necessary to remove poor foundation material and replace it with better quality materials, or where excavation would expose a firm foundation material, a vertical face or composite breakwater can be more economical than a rubble mound because the foundation width is less.

Sheet pile structures require good foundation conditions in order to develop resistance against rotational failure (see clause 51 of BS 6349-1:1984). Wave action on breakwaters can cause erosion of the sea bed at the toe of the structure particularly where there are also significant currents. A rubble mound structure will generally cause less erosion than a vertical face structure.

3.5.2.8 *Construction materials*

Rubble mound construction requires large volumes of rocks of many sizes. When large rock cannot be obtained for armouring, concrete armour units can be used instead. If supplies of suitable rock are limited, a vertical face breakwater can be used, although in deep water it is usual to select a composite type. Although a composite breakwater requires less rock and of a smaller size than a rubble mound type, volumes may still be considerable.

Sand or gravel fill could also be required for replacement of weak foundation material and as ballast for caissons. If large quantities are required an offshore source should be located as extraction, transport and placing by dredger can be more economical than a land source.

For concrete caissons aggregates of the required quality are needed to ensure durability of reinforced concrete. Large quantities of good aggregate could also be needed in concrete armour units and cap structures on rubble mound breakwaters.

3.5.2.9 *Construction methods*

Rubble mound breakwaters in shallow waters do not necessarily require specialized construction plant. For bigger structures a large crane with long outreach will be required when placing armour from the crest of the breakwater and for placing core material between natural end tip face and final slope. Alternatively, the crane can be mounted on a jack-up barge or large pontoon but use of these will be affected by wave conditions.

Vertical face and composite structures usually require preparation of the sea bed and upper surface of the rubble foundation. This underwater work can be slow and requires calm sea conditions.

Vertical face and composite structures are often built by floating in caissons. These can be constructed in a dry dock. Alternatively the lower part of the caisson can be built onshore and lifted or launched into the water for completion afloat.

Caissons can also be lifted into position, which can reduce the period of risk of storm damage during construction, but in this case the availability of very heavy lifting equipment will be a major consideration. However, there are a wide variety of other forms of construction which can be considered depending upon circumstances. The degree of protection of the breakwater site will be an important factor (see 5.4).

Most forms of construction using steel sheet piling are very vulnerable to damage by wave action until each section has been completed, particularly cellular straight web sheet pile structures.

3.5.2.10 *Damage and maintenance*

Because of the variable and unpredictable nature of the wave forces to which a breakwater is subjected the possibility of damage during its life should be accepted. The method of repair should be considered in design.

Some rubble mound breakwaters can suffer a considerable amount of damage before their function is seriously impaired. With rock armour, a displacement of up to 5 % of the units is often considered acceptable before repair is necessary. With concrete armour units, depending on interlock for their stability, a lower level of allowable damage is advisable as fully effective repair is not always economically viable. In all cases repair work will require the use of heavy lifting equipment and permanent access along the breakwater should be provided if repair from floating plant is likely to be impracticable or too costly.

The availability and cost of mobilization of suitable plant and materials for repair is very important when considering what risk of damage should be accepted.

Limited movement of vertical face structures is acceptable if their sole function is wave protection. When other facilities are incorporated in or connected to the breakwater this can be unacceptable. Damage to joints between caissons whether caused by movement of the caissons or otherwise should be repaired as soon as possible in order to prevent further deterioration.

Serious damage to the rubble mound of a composite breakwater can lead to collapse of the structure above and total failure of the breakwater.

3.6 Hydraulic model testing

3.6.1 Introduction

Physical hydraulic model testing is the most reliable method of assessing the hydraulic performance of a breakwater. The reliability of test results depends upon the quality of the input data.

With reliable data it is reported that rock armoured rubble mound sections in a flume give almost complete agreement between model and prototype [12].

The primary object of testing is to check the stability of the breakwater up to and exceeding the design state and its hydraulic performance in respect of run-up, overtopping, wave transmission and reflection.

The use of computational models for planning breakwater layouts is described in 29.6 of BS 6349-1:1984, and the general principles given are applicable to breakwater stability tests.

As referred to in 2.2.6, the scales for a harbour response model are usually too small to represent properly the structure stability (see also 3.6.2), but depending on the size of wave basin available and the breakwater dimensions it is sometimes possible to combine structural testing in three dimensions with a harbour response model. This possibility should be borne in mind when planning the test programme.

There are three methods of physically modelling a breakwater structure in stability testing. The principal, easiest and most frequent method is to model a cross section of the breakwater in a flume, which gives normal wave incidence with unidirectional sea states. A programmable random wave generator should be used rather than one generating regular waves.

The cross section modelled will normally be chosen to represent that part of the breakwater with the most severe exposure, but in some cases it can be desirable to model more than one cross section at different locations.

Where oblique wave attack is expected, such as at roundheads, the use of a wave basin is to be recommended. Such a basin will contain part or all of the breakwater and will be used for tests with random long-crested waves, usually with a wave generator which can be moved to provide different incident wave directions. The sea state generator is unidirectional, as in a flume, but the effects of bed topography can cause wave concentrations and changes of wave direction at particular places along the breakwater, so that the results are more nearly representative of true wave conditions at the site than in a flume.

However, the long-crested seas generated are not fully representative of the multidirectional and short-crested waves which occur in nature, particularly in deep water. In order to study such effects on a structure it is necessary to generate multidirectional waves in a specially designed basin. Such a procedure is difficult and has rarely been used in breakwater engineering. Results available are insufficient to establish the value of this procedure, although studies on a rock mound have indicated that, for the same incident wave energy, a multidirectional sea produces less damage than a unidirectional sea normal to the breakwater [13].

The objectives of model testing, the methods to be used and the programming should be discussed with experienced hydraulic laboratories to ensure that the most suitable procedures are adopted.

3.6.2 Model scales

Models should be constructed to as large an undistorted geometric scale as is practicable. Scales are usually in the range 1 : 30 to 1 : 80, modelled according to the Froude scaling law for which the scale factors are given in 29.5 of BS 6349-1:1984.

For rubble mound breakwaters the behaviour depends partly on the flow of water through, into or out of the structure. Flow through the voids depends on Reynolds number which, as water is used in the model, is not correctly scaled and therefore flow and permeability cannot be correctly reproduced at all locations in the model.

It has been found in practice that scale effects are insignificant if the armour unit Reynolds number R_e is greater than about 3×10^4 , although some recent studies suggest that it can be as low as 8×10^3 . Reynolds number is defined as follows:

$$R_e = \frac{(W/\gamma_a)^{1/3} (gH)^{1/2}}{\nu} \quad (1)$$

where

W is the armour unit weight (in t);

γ_a is the density of armour (in t/m³);

H is the wave height at incipient failure (in m);

ν is the kinematic viscosity (in m²/s);

g is the acceleration due to gravity (9.81 m/s²).

When impacts due to breaking waves on vertical face structures are measured, the same scaling laws do not apply to all types of impact mainly because air entrainment cannot be modelled at suitable scales. Generally the Froude relationship is used as there is evidence to suggest that this gives a conservative result [14].

Erodible bed material is rarely modelled because of the difficulty of correctly modelling such material (see 14.6 of BS 6349-1:1984). Instead a rigid bed model is used and bed protection material modelled to Froude scale.

3.6.3 Model concrete armour units

When concrete armour units are used for rubble mound breakwaters it is generally not practicable to model all the material properties of the unit. Model units of concrete, mortar or plastic, correctly scaled for density and friction, are stronger than is required for similitude between model and prototype. This is particularly important with slender units such as Dolosse where the effects of rocking and displacement can cause breakage in the prototype, but breakage does not occur under similar wave conditions in the model.

The following methods have been used to study the behaviour of model concrete units. They are not generally adopted and add to the cost and duration of any model studies.

a) Instrumentation of model concrete units has been used in a few instances at large scales (1 : 5). Accelerometers have been used to measure the change of velocity on impact and the forces and stresses developed have been determined by analysis [15].

b) Model concrete units made of a material with scaled-down tensile strength have been inserted at critical locations in the model. Failure at these locations due to hydrodynamic and impact forces has then been revealed by testing [15].

c) A plastic material, steel-fibre-reinforced epoxy resin, has been used with the correct density but with an elasticity such that the strains induced on the unit during testing can be measured by model instrumentation. The strains measured during tests have been used to derive, by numerical analysis, loads which could be used for structural design [15].

Because of random placing and random wave attack a sufficient number of special model units should be provided to enable a statistical analysis of the results to be made.

3.6.4 Model construction

The model should be constructed to an undistorted scale and the relative density, coefficient of friction and shape of all materials correctly reproduced. If uplift pressures under crest structures, caissons or similar vertical face structures are to be measured it could be necessary to modify rock sizes to reproduce flow through the voids more correctly.

Model construction techniques should be comparable with those which will be employed in the prototype breakwater and care should be taken to see that over-compaction does not occur. In particular the method of placing armour units should be similar to that which will be used in the prototype in order to ensure that the correct packing density is achieved.

If placing is expected to take place during moderate sea and swell conditions, the appropriate sea states should be generated during armour placing. The armour layer should be reconstructed for each test series. Ideally the underlayer should also be reconstructed.

The sea bed should in general be modelled for a distance of about five wavelengths in front of the structure so that the effects of shoaling on the waves are reproduced. This is not always practicable in the flume available, and the effects on the structure need to be considered in deciding the scale and techniques appropriate in the particular case.

3.6.5 Test programme

3.6.5.1 General

A test programme should consist of a series of preliminary tests designed to identify critical features of the proposed design and the effect of modifications. The preferred test section should then be subjected to a more comprehensive series of tests.

Regular waves should not be used for testing. A random deep water sea state should be produced by a wave generator which can reproduce the design wave energy/frequency spectrum and wave height group characteristics. For this purpose the generator should be able to reproduce a sequence of at least 1 000 waves so that random distributions are automatically generated. The sequence should be repeatable for comparative purposes.

Ideally at least five tests should be carried out for each design condition in order to allow for random variations in damage results.

When funds and time available for testing are limited, it is important that any limitations on the test results are understood and the test programme is carefully planned, within the constraints, to provide the information which will best enable a safe and economical design to be prepared.

It is important that variations in the design parameters of wave climate and water level should be examined particularly when water level variation can result in a change from non-breaking to breaking wave conditions. It is not normally possible to vary water level during a test as wave generator response will change.

In location with appreciable tidal ranges water levels can change significantly during a storm and therefore it will be necessary to design the testing programme to take account of this. It is usual to test at a high water level for overtopping and upper armour stability under the largest waves, with further tests at low water for toe scour and possibly for waves breaking seaward of the structure.

In all cases the relationships between wave conditions, water level and structure response need to be considered carefully in designing the test programme.

3.6.5.2 Rubble mound breakwaters

3.6.5.2.1 General

For rubble mound breakwaters a series of tests is often carried out increasing in steps, e.g. from 50 % to 120 % or more of design wave height with corresponding increases in wave period to simulate the build-up of storm conditions and to check the margin of safety in the test design. The most logical approach to deciding the steps and increasing severity of test conditions is to model waves with progressively lower probabilities of occurrence during the design life, to the limit discussed in 3.2.2. In depth limited conditions higher waves will tend to be modified or break before the structure and it can be impossible for a wave of 120 % of the design wave height to exist at the breakwater, although the use of increased wave period can affect stability.

Whenever larger waves are generated, the need or otherwise to raise standing water levels should be considered. Wave set up is normally reproduced in the model, and tide conditions can be regarded as independent of wave conditions. However, joint probability of increased storm surge with increased wave height will sometimes need to be taken into account (see clause 25 of BS 6349-1:1984).

The duration of each prototype test would normally be about 3 h but, when the test level reaches the design wave, prototype testing should be increased to 6 h. This will simulate storm build-up but if the storm profile is known, or has been deduced, this may be modelled.

In order to assess the stability of armour units their movement should be recorded by photography, cine film or video so that displacement can be measured or acceleration estimated. There are as yet no standardized methods of recording results and this can lead to difficulties when comparing published results.

The amount of movement which can be tolerated by a concrete armour unit before breakage occurs will vary with each type and with its size. Categories of movement which have been used for a number of unit types are given in Table 3.

A descriptive method of classifying overall damage is given in Table 4.

3.6.5.2.2 Reflection and transmission

Coefficients of reflection from the breakwater and wave transmission through and over the structure can be measured by wave probes in front of and behind the breakwater.

3.6.5.2.3 Overtopping

With random seas the rate of overtopping varies considerably. Measurement of quantity should be made over periods of about 50 waves to 100 waves and this should be repeated to obtain sufficient results for statistical analysis of the overtopping discharge.

Spray cannot be correctly modelled because of the effects of viscosity, and wind is not usually modelled in a flume.

Table 3 — Movement of concrete armour units in models

Classification	Description
0	No discernible movement
R	Units seen to be rocking but not permanently displaced
1	Unit displaced by up to $0.5d$
2	Unit displaced by more than $0.5d$ and up to $1.0d$
3	Unit displaced by more than $1.0d$
<i>Source:</i> Hydraulics Research Ltd. NOTE d is normally the equivalent cube size of the unit, but other characteristic dimensions such as height of armour unit have been used.	

Table 4 — Damage classification in model breakwaters

Damage	Description
Destroyed	Core of breakwater affected
Serious	Core of breakwater visible
Much	Large gaps in primary layer; 5 % of units displaced
Moderate	Gaps in primary layer; 3 % of units displaced
Little	2 % of units displaced
Slight	1 % of units displaced
Hardly	No damage
<i>Source:</i> Delft Hydraulics Laboratory	

3.6.5.2.4 Toe scour

The sea bed profile is usually modelled as a fixed bed and model tests should be designed to determine the size of stone necessary to resist movement. Quantitative observation of sea bed scour is not possible but observation of currents above the bed will assist in determining the extent of protection necessary to reduce erosion to acceptable limits.

3.6.5.2.5 Forces on crest structures

Measurement of wave forces on crest structures can be carried out using the same methods as for vertical faced breakwaters (see 3.6.5.3). The work is difficult to execute and interpret because of the effect of the armour in front of the crest structure, and the uncertainty of modelling the flows and pressures in the rock under the cap. Crest structures have been modelled at reduced relative density to investigate whether there is a margin of safety in the design [16].

3.6.5.3 Vertical face breakwaters

The model testing of vertical face breakwaters is mainly concerned with the evaluation of pressures and forces on the structure rather than the determination of acceptable thresholds of damage, although the latter can apply in the case of scour protection. Measurement of overtopping and reflections will usually be important.

A series of tests should be carried out, using increasing steps in wave height as for rubble mound breakwaters (see 3.6.5.2).

It is particularly important to consider the full range of wave heights and water levels, as there can be considerable differences between the various types of wave impact on a vertical face (see 3.9.4 of BS 6349-1:1984).

Measurements of force or pressure should be made with continuously recording instruments so that fluctuations of magnitude and duration can be measured and exceedance curves calculated.

Total forces can be estimated by simultaneous measurement of pressure at a number of locations on the structure and integrating the results. Total forces on a rigid structure can also be measured using a force frame.

Wave forces and pressures are subject to rapid fluctuations and instrumentation should be sensitive enough to record this as maxima can exist for less than 0.1 s.

In order to indicate a margin of safety for a vertical face gravity structure such as a caisson, it is possible to model the structure at reduced relative density; this can also be done to assess temporary construction stability before filling is completed. However, because of variation in buoyancy with wave action, and the uncertainty of modelling uplift pressures, any threshold of movement will not necessarily provide a reliable measure of a true factor of safety.

For wave reflection, overtopping and toe scour, see 3.6.5.2.

3.6.5.4 Composite breakwaters

Testing of composite breakwater structures should combine the test programme and measurements outlined in 3.6.5.2 and 3.6.5.3.

Particular attention should be paid to the stability of the top of the rubble mound where wave reflections from the vertical face can cause stability problems.

3.6.5.5 Temporary conditions during construction

It is desirable to carry out tests on the partially completed stages of breakwater construction especially when storms expected to occur during construction can make it impossible to continue work during a winter season; some form of temporary protection can be justified.

Tests would be designed to assess the damage which might occur during a storm and find means of preventing or reducing it.

A construction wave climate which is appropriate to the risk should be used. Particular features which should be examined are as follows.

- a) For rubble mound breakwaters: stability of core and underlayer, effect of overtopping before crest is completed, scour at working end.
- b) For vertical face and composite breakwaters: effect of scour at working end and in front of vertical face, stability of partly completed structure.

3.7 Risk analysis

3.7.1 Limit states

The design of a breakwater should be such that, during its construction and throughout its intended service life, it has an acceptably low probability of failure, as defined in 3.2.1. In order to obtain the best assessment of probability, a risk analysis is carried out. The analysis involves quantifying, on the one hand, the probability of occurrence of an undesirable event (e.g. partial collapse of a breakwater) and, on the other, the consequences of the occurrence of that event (e.g. interruption of port operations).

A risk analysis requires the preparation of an inventory of the hazards and response mechanisms, i.e. the manner in which the breakwater responds to hazards. A distinction is drawn between ultimate limit state and serviceability limit state (see 3.2.1). The material damage and non-material loss for each state is assessed. The risk can be evaluated by multiplying the loss by the probability of occurrence during the service life of the structure.

3.7.2 Choice of level of risk

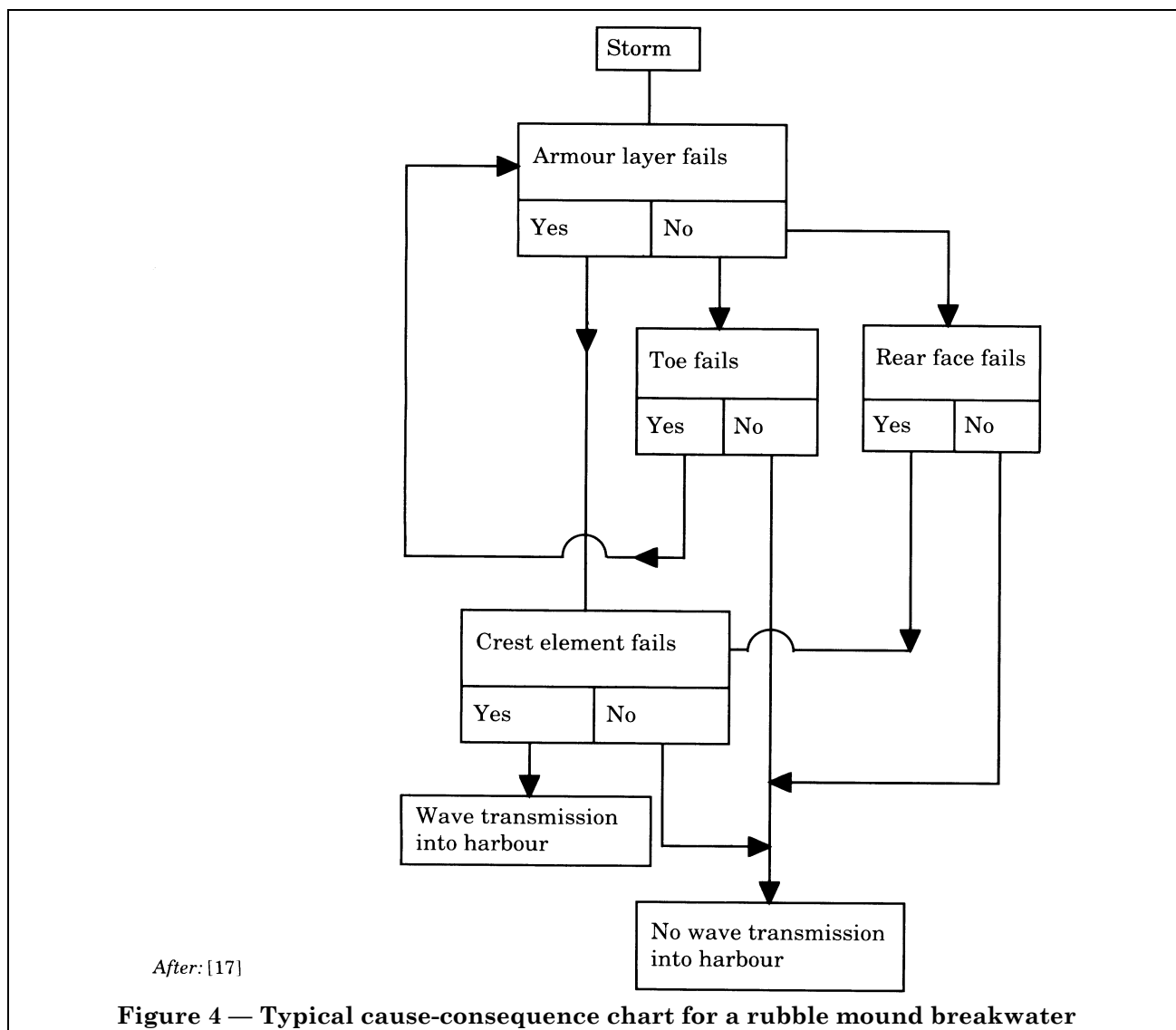
The choice of level of risk depends on the following factors.

- a) *The characteristics of the type of breakwater.* A rubble mound structure can suffer only partial damage when the design conditions have been exceeded. With vertical face or composite breakwaters or slender concrete armour units, the exceedence of design conditions can involve their total destruction.
- b) *The purpose of the breakwater.* The permissible risk level should reflect the importance of the breakwater, the function for which it has been designed and its value to commercial operations. The risk level for failure (ultimate limit state) should be different from that for damage (serviceability limit state).
- c) *The reliability of the data used for design.* If there is doubt about the data or the assumptions adopted, the philosophy should be to adopt a conservative approach and to design for a low risk.
- d) *Quality control and tolerances achieved in construction.*
- e) *Whether sufficient model testing has been carried out to be able to determine the failure conditions for the proposed design.*

Although it is possible to make a classification according to security or risk criteria in a qualitative way for different situations, the quantitative determination of the values to be adopted is a matter of decision for the person responsible for the project.

3.7.3 Fault trees

In assessing the safety of a breakwater the system should be considered as a whole, as the structure is composed of several components, each of which is subject to hazards and mechanisms, and the failure of any one component can lead to a different consequence. For industrial installations the system can be represented by diagrams such as fault trees and event trees. For breakwaters and other civil engineering projects, where problems of a continuous character are a fairly frequent occurrence a cause-consequence diagram may be more appropriate. An example is shown in Figure 4.



3.7.4 Method of analysis

There are two approaches in determining the probability of failure by a particular response mechanism. One is to prepare a computational model of the mechanism in order to establish a reliability function for the limit state considered; the other is to make a direct estimate of the probability on the basis of experience [17].

There are various techniques which can be applied to determine the probability of failure for a given reliability function and for given statistical characteristics of the basic variables, generally described as follows.

a) Level III comprises calculations in which the complete probability density functions of the stochastic variables are introduced and the possibly non-linear character of the reliability function is fully accounted for.

b) Level II comprises a number of approximate methods in which the reliability function is linearized and all probability density functions are approximated by normal distributions.

c) Level I comprises calculations based on characteristic values and safety factors. This is not strictly a probabilistic approach because uncertainty is not included.

Full physical descriptions of all the possible mechanisms which can lead to failure are not yet possible, nor can the probability density functions of the load and strength variables be formulated.

Methods of risk analysis are currently being developed, mainly at level II. In the meantime a factor of safety approach (level I) linked to qualitative comparisons based on fault tree and cause-consequence diagrams should be used to compare designs with respect to the risks to be adopted. This is a matter for consideration by both owner and designer.