PORT WORKS DESIGN MANUAL
PART 4

Guide to Design of Seawalls and Breakwaters

Civil Engineering Office
Civil Engineering Department
The Government of the Hong Kong Special Administrative Region

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FOREWORD
(Continuously updated e-version Jun 2023)

This continuously updated e-version of the Port Works Design Manual has incorporated the previously issued Corrigenda No. 1/2006 (June), No. 1/2014 and No. 1/2018 to facilitate the designers and industry practitioners to carry out coastal design in a more convenient manner.

Practitioners are encouraged to comment at any time to the Civil Engineering Office on the contents of this document, so that improvements can be made to future editions.

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Head, Civil Engineering Office
June 2023

FOREWORD

The Port Works Design Manual presents recommended standards and methodologies for the design of marine works in Hong Kong. It consists of five separate volumes, namely, Part 1 to Part 5. Part 1 mainly covers design considerations and requirements that are generally applicable to various types of marine works. Part 2 to Part 5 are concerned with specific design aspects of individual types of works including piers, dolphins, reclamations, seawalls, breakwaters and beaches. This Manual supersedes the Port Works Manual prepared in the 80's.

This document, Port Works Design Manual: Part 4, gives guidance and recommendations on the design of seawalls and breakwaters. It was prepared by a working committee comprising staff of the Civil Engineering Office and Special Duties Office with reference to the latest local and overseas design standards and experiences in consultation with other Government departments, engineering practitioners and professional bodies. Many individuals and organizations made very useful comments, which have been taken into account in drafting the document. An independent review was undertaken by experts before the document was finalized. All contributions are gratefully acknowledged.

Practitioners are encouraged to comment at any time to the Civil Engineering Office on the contents of this document, so that improvements can be made to future editions.

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May 2003

Working Committee of Port Works Design Manual : Part 4 **[First published Version in 2003]**

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1. INTRODUCTION

1.1 Purpose

The purpose of the Port Works Design Manual (the Manual) is to offer guidance on the design of marine works and structures normally constructed by the Government of the Hong Kong Special Administrative Region. Such works and structures include public piers, ferry piers, dolphins, reclamations, seawalls, breakwaters, pumphouses, beaches and associated marine facilities. The Manual has been written with reference to the local conditions and experience. Therefore, it may also provide a source of useful data and design reference for other marine works and structures constructed by other organizations or parties in Hong Kong.

The Manual is issued in five separate parts. The titles of these parts are :

- Part 1 – General Design Considerations for Marine Works
- Part 2 – Guide to Design of Piers and Dolphins
- Part 3 – Guide to Design of Reclamation
- Part 4 – Guide to Design of Seawalls and Breakwaters
- Part 5 – Guide to Design of Beaches

The recommendations given in the Manual are for guidance only and should not be taken as mandatory. Compliance with these recommendations does not confer immunity from relevant statutory and legal requirements. Because of the variable nature of the marine environment, the design of marine works and structures relies particularly on the use of sound engineering judgement and experience. Practitioners should be aware of the limitations of the assumptions employed in a particular theoretical or computational method. Since the marine environment is a field where active research and development are continuing, it is beyond the scope of the Manual to cover all analysis and design methods. Practitioners should be prepared to explore other methods to suit a particular problem and should also realize that many of the methods will continue to evolve.

This part (Part 4) of the Manual gives guidance and recommendations on the design of seawalls and breakwaters, covering aspects on the choice of types and layouts of structures, foundation, hydraulic performance, structural stability, construction and maintenance. It also includes design of minor marine structures and facilities normally associated with the construction of seawalls and breakwaters. Worked examples are provided in Appendix D to illustrate the application of the design methods. In using this part of the Manual, readers should refer to other parts of the Manual on particular aspects, as necessary.

1.2 Definitions and References

The definitions of terms and meanings of symbols for the purpose of this part of the Manual are given in the Glossary of Terms and Glossary of Symbols at the end of this document. Meaning of symbols not shown in the glossary is given in each case in the text.

The titles of the publications referred to in this part of the Manual are listed in the reference section. Readers should consult these original publications for more detailed coverage of particular aspects. For Works Bureau Technical Circulars (WBTC) or Environmental, Transport and Works Bureau Technical Circular (Works) which are updated regularly, reference should be made to their latest issues.

2. TYPES OF STRUCTURES

2.1 General

This chapter discusses the characteristics of various types of breakwaters and seawalls, and provides general guidance on the selection of an appropriate structural form for these structures.

2.2 Breakwaters

2.2.1 Functions

A breakwater is a structure employed to reflect and dissipate the energy of water waves and thus prevent or reduce wave action in a water area it is desired to protect. Breakwaters may be constructed to form a harbour or typhoon shelter and create sufficiently calm water, thereby providing protection for safe navigation, berthing and mooring of vessels, and other harbour activities. Breakwaters may sometimes serve as aids to navigation or shore protection or as both. There are three main types of breakwaters, namely, rubble mound breakwater, vertical breakwater and composite breakwater.

2.2.2 Rubble Mound Breakwaters

Rubble mound breakwater is a commonly used type of breakwater structure in Hong Kong (see Figure 1). It is typically constructed with a core of quarry-run stone that is protected from wave action by one or more rock underlayers and an outer layer composed of massive rocks or specially shaped concrete armour units (Figure 2). A concrete crest structure may be constructed on the mound to provide access or, with the incorporation of a wave wall, to prevent or reduce wave overtopping.

Figure 1 indicates the components of a typical rubble mound breakwater. Their functions are summarized as follows:

- Foundation – Provides embankment stability.
- Scour protection apron – Prevents erosion.
- Core – Provides bulk of structure and reduces wave transmission.
- Toe mound – Supports the main armour and prevents toe scouring.
- Underlayer – Acts as filter between core and armour layer and bedding for placement of armour.

- Rear face armour – Protects core from overtopping waves and against wave action inside the harbour.
- Main armour – Provides wave protection.
- Concrete crest structure – Provides access and reduces wave overtopping.

The properties of armour rock should comply with the requirements given in Section 21 of the General Specification for Civil Engineering Works (GS) (Hong Kong Government, 1992). For armour design, it is recommended that the specific gravity of the rock, if obtained locally, should be taken as 2.6. This figure corresponds to the minimum requirement of specific gravity given in Section 21 of the GS. A value higher than 2.6 should not be used for design without extensive testing, both prior to construction, where a rock source has been identified, and during construction for quality control.

2.2.3 Vertical Breakwaters

A vertical breakwater is one in which wave attack is resisted primarily by a vertically faced structure extending directly from seabed level. Structures comprising reinforced concrete caissons are common forms of vertical breakwaters. They are usually designed for floating into position from a dry dock or a floating dock and sinking to the seabed foundation. Typical sections of caisson type vertical breakwaters are shown in Figure 1.

2.2.4 Composite Breakwaters

A composite breakwater is a combined structure consisting of a vertical structure placed on a rubble mound that is submerged at all tidal levels. Typical cross section of a composite breakwater with reinforced concrete caisson is shown in Figure 1. This type of structure may be used as a breakwater in very deep water where the volume of rock required for a rubble mound structure is not available or when it is not practicable to design a vertical face structure to carry the design wave loading to the full depth.

2.2.5 Selection

The following factors should be considered when selecting the type of structures:

- Layout of breakwaters.
- Environmental conditions.

- Operational conditions.
- Navigation requirements.
- Construction conditions and periods.
- Construction cost.
- Availability of construction material.
- Maintenance.
- Effect of climate change.

In general, it is necessary to compare the merits and costs of different types of structure under the respective site conditions and project constraints before a decision is made. A general comparison of the applications of the three types of breakwater is shown in Table 1.

2.3 Seawalls

2.3.1 Functions

A seawall can be used as a soil retaining structure of a reclamation or as an armouring structure to protect a shoreline from erosion against wave and current actions. Seawalls may be vertical or sloping. Vertical seawalls have the advantage that they can provide marine frontage for vessel berthing and cargo handling. If necessary, wave absorption units can be included on vertical seawalls to reduce wave agitation inside a harbour.

2.3.2 Concrete Blockwork Seawalls

Concrete blockwork seawalls are gravity structures made up of precast concrete blocks. Typical layout of a concrete blockwork seawall is shown in Figure 3.

Concrete blockwork structures are commonly used in Hong Kong. They have the following advantages:

- Relatively low cost of construction.
- Long history of satisfactory performance with negligible need for maintenance.
- Flexibility to cope with some differential foundation settlement.
- Damage from vessels in accidents is usually minor.
- Incorporation of landings, pumphouses and drainage outfalls is relatively simple.

Disadvantages of concrete blockwork structures relate mainly to the relatively long construction period required, and the need for a large casting yard and stacking area with marine frontage. These disadvantages, however, can generally be reduced in significance with adequate project planning, as many such blocks can now be cast in the Mainland and delivered to site when required. Another disadvantage is that vertical walls reflect waves, with the consequence that wave activity in an adjacent area is increased.

2.3.3 Caisson Seawalls

Apart from precast concrete blocks, the earth retaining function of a seawall can be provided by means of concrete caissons as shown in Figure 3. The caissons are usually cast in a dry dock or on a floating dock and transported to the site by floatation before sinking into the designated locations. Because of the relatively high mobilization cost for a caisson seawall, it is usually not economical to use caissons for a short seawall or in limited water depth.

2.3.4 Wave Absorption Vertical Seawalls

Vertical seawalls with solid face are highly reflective of wave energy. This may not be acceptable inside a harbour as wave agitation will affect vessel operation and navigation.

Wave reflection can be reduced by introducing wave absorption units on the vertical seawalls. A wide variety of wave absorption vertical seawall have been developed over the years under different wave conditions and application constraints in different places. An example of a wave absorption seawall is shown in Figure 3. It contains a wave absorption chamber with perforated front wall that allows flow into and out of the chamber. The degree of wave absorption capacity depends very much on the size of the wave absorption chamber relative to the incoming wavelength. Normally, wave reflection is minimized when the width of the chamber is 10% to 20% of the incoming wavelength, provided the perforation ratio, defined as the ratio between the area of the perforations and the total area of the front wall, is around 30%. The suitability of the application of the seawall at a particular site should be subject to model tests.

2.3.5 Rubble Mound Sloping Seawalls

A typical cross-section of a rubble mound sloping seawall is shown in Figure 4. The slope of the seawall is generally protected by rock armour. If the wave condition renders the rock size not economically available in the market, concrete armour units can be used as an alternative to protect the slope of the seawall.

The advantage of a rubble mound sloping seawall are :

- Construction generally simpler and faster than a vertical seawall.
- More tolerable to differential settlement.
- Reduced reflected wave height due to dissipation of wave energy on the slope of the structure.
- Less wave overtopping than a vertical wall with a solid face.
- Easier to carry out maintenance.

A sloping seawall may not be a suitable form of construction if marine frontage for vessel berthing or cargo handling is required. However, a piled deck structure can be constructed over the rubble mound to form a berth for vessels. Another drawback is that a wider clearance has to be provided for marine traffic due to the underwater slopes, which may sometimes be not practicable when water space is limited.

2.3.6 Selection

Factors to be considered in selecting the type of seawall are similar to those for breakwaters listed in Section 2.2.5, with due consideration of the relative merits and demerits of individual types of seawalls discussed in Sections 2.3.2 to 2.3.5. If reinforced concrete is used, reference should be made to Chapter 6 of Part 1 of the Manual on the concrete specification and corrosion protection measures.

3. LAYOUT CONSIDERATIONS

3.1 General

This chapter provides general guidelines on designing the layout of breakwaters and seawalls, in particular on the setting and alignment of these structures.

3.2 Breakwaters

3.2.1 General

The layout of breakwaters for typhoon shelter or harbour basin should be determined by considering the following factors :

- Required sheltered conditions for vessels at berth or anchorage.
- Maneuvering areas for vessels within the sheltered area.
- Adequate stopping distance for vessels entering the entrance at a safe navigating speed.

Analysis should be carried out when determining the layout of breakwaters to evaluate the extent of wave penetration, the requirements of port operation and navigation, and the environmental impact. Since the size of the sheltered area is determined by manoeuverability, vessel characteristics, berthing and mooring requirements, Marine Department and users should be consulted in designing the layout.

3.2.2 Wave Penetration

Wave diffraction through the entrance of breakwaters will affect the degree of shelter provided and spread of waves into the basin. Hence, it is first necessary to establish the wave conditions just outside the entrance, then to determine the effect of the entrance in permitting waves to enter the sheltered area, and finally to determine the responses at critical positions. Wave direction is important and, whilst the greatest shelter should be provided against the largest waves, less critical wave conditions from other directions should also be considered in the layout. Some important points that should be noted are summarized below :

- The layout of the heads of the main and lee breakwaters should preferably be

- designed to give an overlap to prevent direct penetration of the most severe waves into the protected area (See Figure 5).
- The overlap of the main and lee breakwaters against the direction of wave propagation should ensure that no direct penetration of the incident waves will reach the anchorage areas for small vessels.
- Wave transmission through the structure can occur with a very porous rubble mound, for example, one constructed only of large rocks, where the degree of transmission increases appreciably with wave period. Therefore, this type of structure should be avoided for breakwaters of harbour basin or typhoon shelter.
- The effect of waves generated from vessels in adjacent fairways should be considered in locating the entrance of a harbour for small vessels. Normally, ship waves do not interfere the navigation and anchorage of ocean-going vessels.
- The entrance location should avoid penetration of swells or long period waves that may induce possible resonance motion on vessels inside the basin.
- Where wave overtopping is a problem, a wave wall may be constructed on the structure to reduce the overtopping quantity. For vertical and composite breakwaters, the wave wall can be constructed of concrete as an integral part of the breakwaters. There is no joint between the wave wall and the concrete of the caissons. The wave wall is not subject to uplift, and the horizontal wave force acting on the wave wall is added to the wave force acting on the caisson part for the examination of the stability of the upright sections.

A preliminary estimate of the degree of diffraction in a sheltered area may be estimated using the diagrams in Figures 6 and 7. A fairly flat seabed is assumed in these figures. For more realistic estimate of the wave conditions, mathematical wave modelling may be applied. Guidance on mathematical wave modelling is given in Chapter 2 of Part 1 of the Manual – General Design Consideration for Marine Works.

3.2.3 Port Operation and Navigation

Currents can be generated across an entrance of the harbour basin or typhoon shelter as a result of the deflection of currents around the head of the breakwater. A wide entrance may ease navigation but this will be in conflict with the objective of limiting wave penetration. Some compromise may be necessary, and the advice of Marine Department and experienced mariners is essential in determining the optimum layout of breakwaters at the entrance, taking into account any limits on navigation and port operation.

Reflection from the seaward face of a solid face vertical breakwater can set up standing wave patterns which can result in increased wave agitation and affect navigation in front of the breakwater. This effect may be reduced if the alignment is convex outward in the seaward side instead of a straight one. A concave alignment, which will create severe wave concentration on the seaward side of the structure, should be avoided. Wave absorption chamber may be constructed on vertical breakwaters to reduce wave reflection. A wave study on the change in wave climate due to new breakwaters should be carried out to ascertain the effect on port operation and navigation.

3.2.4 Environmental Effect

A breakwater may cause change in the hydrodynamic regime. Hence, it is necessary to undertake hydraulic study and environmental impact assessment over the life of the structure, including the effects of climate change, to ensure that the changes in flow and wave climate during and after construction will have no unacceptable effects on :

- Tidal flushing and water quality.
- Ecology.
- Siltation and seabed scouring.
- Sediment transport and shoreline stability of existing beaches.

An example of the impact on sediment transport is illustrated in Figure 5, showing the possibility of up-drift sediment accretion and down-drift erosion of the shoreline after the construction of breakwaters. Up-drift accretion can eventually cause the formation of a bar across the entrance of breakwaters which will then require frequent maintenance dredging. Down-drift erosion can lead to loss of beaches and the need for coastal protection measures, which can extend a long way from the harbour. Such impact should be carefully assessed when longshore sediment transport is a major feature of the shoreline. Examples of how beaches will behave after construction of breakwaters on a sandy coast are given in OCDI (2000).

The construction of breakwaters 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 basin or typhoon shelter, resulting in pollution and settlement of sediment in the sheltered water. Openings or culverts may be provided at suitable positions along the breakwaters to increase flow circulation. The effect of wave penetration should be assessed when determining the positions of these openings.

3.3 Seawalls

Seawalls are usually edge structures of reclamation. The determination of their layout with respect to alignment, crest level, operation, navigation and environment are related to the reclamation design, and is covered in Part 3 of the Manual – Guide to Design of Reclamation. Specific aspects are listed as follows :

- Wave reflection from solid face vertical seawalls can lead to wave agitation in the harbour, affecting port operation and navigation. Vertical seawalls with wave absorption units or rubble mound sloping seawalls should be considered to reduce the effect of reflection at a particular site.
- Vertical seawalls are generally required where marine frontage is required for vessel berthing and cargo handling, or where water space is not sufficient to accommodate the underwater slope of rubble mound seawalls.
- Seawalls built to protect land from wave actions may be provided with wave walls to minimize the amount of wave overtopping. A wave wall or a parapet wall can be constructed as an integral part of a seawall. It also serves to prevent people promenading at the waterfront from falling into the sea.

3.4 Eco-shoreline

A properly designed artificial seawall should be able to serve the stabilization and protective function while offering a larger variety and surface area of intertidal habitats for increasing biodiversity in the area. Where applicable, design of eco-shoreline should be considered instead of traditional seawall with a view to enhancing the ecological value of seawall. Eco-shoreline can also promote water-friendly culture and improve marine environment for public enjoyment.

There are some designs of eco-shoreline which have been successfully applied in other countries, including:

- The use of sloping/terraced seawall or creation of artificial tidal pool/ mudflat to provide additional habitat for intertidal organisms and increase the biodiversity.
- Incorporation of different structures (e.g. protrusions, cavities, pools, artificial reefs, etc.) into the seawall to increase the complexity. This also helps to mimic the natural shoreline and provide a microhabitat for organisms to settle in.
- Shoreline rehabilitation with suitable vegetation (e.g. mangrove, marsh plants, etc) to improve visual amenity, stabilize the foreshore and control sediment erosion.

4. FOUNDATIONS

4.1 General

The structure and its foundation should be designed so that, during the design life, foundation displacements and movements are kept within the limits that the structure can tolerate without affecting its structural integrity and functional capability. This chapter gives general guidance on the design of foundations for seawalls and breakwaters. GEO Publication No. 1/2006 – Foundation Design and Construction (GEO, 2006), GEO Technical Guidance Note No. 41 (TGN 41) – Amendments to British Standards References in Technical Guidance Documents for Migration to Eurocodes (GEO, 2014) and Code of Practice for Foundations (BD, 2004) may provide useful reference for the foundation design of marine structures. One important aspect in the design of foundation is the stability of the seabed and the possibility of scour and undermining around the structure under wave and current actions. This is covered in Chapter 6 of this part of the Manual.

4.2 Site Investigation

Reference should be made to Geoguide 2 (GEO, 2017a) for guidance on good site investigation practice, Geoguide 3 (GEO, 2017b) for guidance on description of rocks and soils in Hong Kong, and Geospec 3 (GEO, 2017c) for model specification for soil testing. Specific details of site investigation and soil testing for marine works are given in Chapter 4 of Part 1 of the Manual.

Difficult ground conditions generally refer to the existence of unfavourable subsoil strata on site. The presence of such conditions, if not properly handled, may lead to both problems at the construction stage and during the future use of seawalls, breakwaters and reclamation. Guidelines for site investigation in such conditions are given in the report “Study on Coastal Subsoil Geotechnical Conditions” (CED, 2003). A summary of the guidelines, including the spacing of the points of exploration, depth of penetration and vertical intervals of in-situ tests and soil sampling, is shown in Appendix A.

4.3 Stability

4.3.1 Factor of Safety against Soil Shear Failure

The global factor of safety should be used when designing the foundation of marine works against slip failure. It may be taken as the ratio of average available shear strength of the soil along the critical slip surface to that required for maintaining equilibrium. Where soil properties have been tested, the following minimum factors of safety are recommended :

Loading Conditions	*Factor of Safety against Soil Shear Failure*
Normal	1.3
Extreme	1.2
Accidental	1.2

For temporary loading conditions, the factor of safety against soil shear failure should be assessed for each individual case by the designer.

The loads for calculating the factors of safety should be unfactored values with no allowance for partial safety factors (see Chapter 5 of Part 1 of the Manual).

4.3.2 Soil Conditions

The factor of safety should be determined on the basis of a full knowledge of the soil properties at the site. The values of geotechnical parameters for design should be determined from careful assessment of the range of values of each parameter. Particular attention should be given to the quality of ground investigation and the adequacy of test data with respect to the inherent variability of the materials encountered. Reasonably conservative selected values should be adopted and sensitivity checks within the upper and lower limits of design parameters should be carried out if the level of confidence is low. If sensitivity analysis results are not conclusive, additional investigation and testing should be carried out to obtain more reliable information.

For structures founded on silty/clayey material (low permeability), consolidation takes a long time and the most critical period for stability is during construction and just after completion. The undrained shear strength of the founding strata is the controlling critical factor for overall stability. The designer should determine the appropriate undrained shear strength as well as the long term (drained) parameters, and assess the foundation stability under all conditions.

Undrained shear strength of silty/clayey soil can be determined from in-situ vane shear tests, using a reduction factor on the measured vane shear value, where appropriate. Unconsolidated undrained triaxial tests can also be used, provided samples are obtained using sampling techniques which avoid disturbance during sampling. However, the results of unconsolidated undrained tests may not be very reliable due to possible disturbance during sampling. Hence, they should be used to supplement the in-situ soil strength obtained from the field tests. Consolidated undrained tests can simulate the long-term performance of the soil samples and their results can be used to assess the long-term stability of the structures. In view of the comparatively poor consolidation characteristics of clayey/silty soil, care should be exercised in adopting the consolidated undrained test results in the analysis of short-term stability. In-situ vane shear test results should be used for such analysis as far as possible.

Field and laboratory tests to determine suitability of the founding material should be identified at the design stage. Validity of the design assumptions should be checked during construction by incorporating requirements for appropriate tests in the contract documents. Advice should be sought from geotechnical specialists, where appropriate.

4.3.3 Loading

All of the appropriate loads and loading conditions described in Chapter 5 of Part 1 of the Manual and the various loading stages on the structure under the most severe load combinations should be examined. If it is expected that other loading conditions could be critical, they should also be investigated.

In seawall design, the live load should be determined according to the designated land use behind the seawall. Temporary surcharge preloading on the seawall may be more critical than the permanent loads or future live loads. This should be checked in the design.

Particular attention should be paid to fill placement behind the structure when clayey/silty deposits remain under the foundation. Rapid fill placement may induce instability on the foundation as the excess pore water pressure due to the fill loading will take some time to dissipate completely. The effect of the filling rate or the stages of loading on stability should be investigated with respect to the shear strength of the underlying soil at the time of construction.

The induced pore pressures must be measured during construction and further filling must not be allowed to proceed before the required dissipation has been achieved. Provision for suitable instrumentation should be specified in the contract.

4.4 Settlement

The settlement expected during the design life of seawalls and breakwaters should be assessed to ensure that it is acceptable to the proposed use of the structures. In general, the residual settlement after completion of construction should be limited to not more than a maximum between 150 mm and 300 mm, depending on the type, importance, stability and usage of the structure and the site condition. For settlement-sensitive installations or facilities, more stringent requirement may be needed and should be determined in consultation with the client and users. In addition, consideration of settlement of proposed or adjacent developments should be taken account of when estimating overtopping or tidal flooding effects.

4.5 Types of Foundation

4.5.1 Dredging

Dredging for the foundation of seawalls or breakwaters may involve totally or partially removing the marine deposits and replacement with sand or rubble fill in order to provide adequate foundation stability and to prevent excessive settlement. Normally, dredging is stopped when a firm stratum has been reached. This method, though relatively simple, requires the disposal of dredged sediments, in particular when the quantity is large. In addition, removal of soil is generally discouraged unless there is strong justification (see ETWB TCW 34/2002 (ETWB, 2002)).

Partial removal of marine deposits, leaving the stiffer or stronger deposits in place, reduces the dredging and fill quantities compared to the full-dredge method. Partial dredging may be carried out in conjunction with installation of vertical drains and staged construction. The main purpose of vertical drains is to accelerate the consolidation of the remaining soil so that the target settlement due to primary consolidation can be achieved within shorter period. Staged construction allows sufficient time for the marine deposits to consolidate and gain strength between stages of construction. The extent of marine deposits to be left is subject to thorough ground investigation, soil testing and detailed design. Partial dredging normally requires longer construction period for consolidation to take place. This aspect should be account for when assessing the cost and programme implications.

4.5.2 Deep Cement Mixing

The principle of deep cement mixing (DCM) is based on chemical reactions between clay and chemical agents. Lime and Portland cement are the two most commonly used admixture stabilizers. The purpose of mixing chemical additives with the soil is to improve the stability, strength and stress-strain properties of the soil. The stabilization mechanism generally involves the following chemical reaction processes :

- Cement reacts with the pore water of soft clay to form a series of hydrates.
- Hydrates exchange ions with clay particles and form large conglomerates.
- Clay particles react with the excess calcium ions from the hydration process and form non-soluble compounds.

DCM is implemented in the field by machines with rotation blades that supply chemical agent into the soil for in-situ mechanical mixing to form DCM piles. The DCM stabilized soil can take the form of pile, wall or block as shown in Figure 8, which is summarised as follows :

- Pile Type - This is formed by placing DCM piles at grid pattern. It is usually adopted when the superstructure is relatively light and differential settlement is not a problem. Piles with depth up to 60 m have been used in Japan.
- Wall Type - When DCM piles are constructed close together in one direction with overlapping, the wall type DCM foundation is formed. It is usually adopted for superstructures with large length to height ratio and sensitive to differential settlement.
- Block Type - When DCM piles are constructed close together in perpendicular directions with overlapping, the block type DCM foundation is formed. It is usually adopted for heavy superstructure with stringent differential settlement requirement.

The advantages of DCM are :

- By stabilizing native soil using chemical additives, DCM does not require dredging and filling to form the foundation as in the conventional dredging method.
- The operation of DCM would not cause lateral displacement of the soil being treated. Therefore, effect on adjacent structures or foundations is minimal.
- The weight of DCM-treated soil is basically unchanged. Therefore, no additional surcharge will be induced on the underlying soil strata.
- DCM is flexible in application because the amount of stabilizing agent and form of treatment can be adjusted to suit different soil properties and engineering requirements.

The following limitations should be considered in the choice of the method :

- Its cost may be several times higher than that of a conventional dredging scheme.

- Stringent quality control and monitoring is required during the mixing process to ensure that the required strength is developed in the soil. It may be necessary to carry out field trials to obtain an optimal site-specific soil to cement ratio for practical application.
- The rotating blades of the DCM machine may not work properly if obstructions of size larger than 250 mm are encountered during the mixing process.
- Investigations should be carried out to assess the possible environmental impacts associated with marine application of DCM and to determine if mitigation measures are necessary for a particular site.
- It does not work well in certain soils, notably those which have a high organic content and acidic soils (Suzuki, 1982).

4.5.3 Stone Columns

Stone columns is a grid of densely packed columns of gravel installed in the soil (see Figure 9). Their diameter generally ranges from 0.6 m to 1.0 m and the size of gravel normally ranges from about 75 mm to 100 mm. By constructing stone columns in a square, rectangular or triangular grid pattern, the ground is transformed into a composite mass of vertical, compacted granular cylinders with intervening soil. This method provides the advantages of increasing the average shear strength and decreasing the compressibility of the treated soil. Since gravel is a good drainage material, installation of stone columns in clayey soil also accelerates the dissipation of excess pore water pressure and hence the consolidation.

The technique utilizes the vibroflot equipment for forming cylindrical holes in the soil. For marine application, stone columns are generally formed by penetrating the vibroflot to the desired depth and gravel is pumped through a supply duct to the bottom of vibroflot where the gravel is forced out by air pressure through a mud protection shield as the vibroflot is lifted. The vibroflot also compacts the gravel and displaces the gravel outwards, hence mobilizing the lateral resistance of the soil against the displaced gravel. Compaction is continued until the lateral resistance to the displacement of the soil by the gravel is fully developed. The maximum practical length of stone columns is about 30 m.

The advantages of stone columns are :

- Stone columns share the external loads with the native soil in the form of a composite foundation, and hence the method effectively utilizes the original ground without dredging in principle.

- It immediately increases the rate of settlement of the soil in the presence of gravel that acts as drainage material.
- It is flexible in application because the diameter and spacing of the stone columns can be easily adjusted to suit different site conditions.

The limitation of the method are :

- The method is more costly than the conventional dredging method, although it may be cheaper than the DCM method.
- Stone columns may not be feasible if the strength of the soil to be treated is too low.
- Stringent quality control is required during the installation process as the integrity of the stone columns is crucial in the whole system.
- Installations of stone columns may cause lateral or upward soil displacement and result in heaving of the seabed. The extent should be investigated in the design.
- The soil in the vicinity of the stone columns may be disturbed to a certain extent during installation. The effect of strength reduction should be included in design.

4.5.4 Comparison of Foundation Types

A comparison of the application of the above three types of foundation is given in Table 2.

4.6 Design Approach

4.6.1 Dredging

The extent and depth of dredging should be determined by means of a thorough slip surface analysis. Guidance on the use of such methods may be found in the Geotechnical Manual for Slopes (GCO, 1984) and Works Bureau Technical Circular 13/99 (WB, 1999).

Chapter 6 of Part 1 of the Manual has indicated that, when decomposed granite is used as fill for underwater foundations, the deposited layer should normally not exceed 15 m thick and should not contain Grade VI materials as defined in Table 4 of Geoguide 3 (GEO, 2017b). The purpose is to limit excess pore pressures within the construction period for maintaining the stability of the structure. Further details are given in GEO Report No. 33 entitled "An Evaluation of the Suitability of Decomposed Granite as Foundation Backfill for Gravity Seawalls in Hong Kong" (GEO, 1993).

For settlement assessment, reference may be made to the principle given in Chapter 4 of Part 3 of the Manual.

4.6.2 Deep Cement Mixing

The DCM treated soil normally has large shear strength and deformation modulus with very small strain at failure compared to the original soil. Therefore, the DCM treated soil may be considered as a rigid structure. A feasible DCM scheme for marine gravity structures will generally involve analysis of the following :

- Analysis of the overall stability against shear failure, both through the stabilised foundation and beneath it.
- External stability against sliding, overturning and bearing capacity at the bottom surface of the stabilized body under the design external loads acting on the boundary of the stabilized body (See Figure 10).
- Internal stability against the internal stresses (including compressive, tensile and shear) induced by the external loads on the stabilized body; the strength being dependent on the soil properties and the soil-cement mixing ratio.
- Amount of reduction of settlement as compared with the original soil.

The design of DCM foundation requires specialist knowledge and experience. Specialist input should be sought if this type of foundation is adopted. Reference may be made to Ye et al. (1997) for further details of the design methodology.

4.6.3 Stone Columns

Design of stone-column foundation involves the determination of the diameter, length spacing and pattern of the stone columns, and the size of gravel for forming the columns. The design process will involve analysis on the following :

- Bearing capacity of individual columns and the stone-column group against vertical stresses from the structure.
- Overall stability including slip failure analysis of the composite ground made up of the stone columns and the soil.

- Assessment of settlement of the composite ground so that the residual settlement after completion of the works is within acceptable limit.

The design of stone-column foundation requires specialist knowledge and experience. Specialist input should be sought if this type of foundation is adopted. Reference may be made to Mitchell and Matti (1981) and Ye et al. (1997) for further details of the design methodology.

5. HYDRAULIC PERFORMANCE

5.1 General

This chapter provides general guidance on assessing the hydraulic performance of the structures on wave run-up, overtopping, transmission and reflection. Some empirical methods of estimating the magnitude of these parameters for simplified structural configurations and wave conditions are given. These methods, mostly based on results of laboratory testing, provide an estimate of the order-of-magnitude of the parameters only. Where complicated situations are encountered and the predictions are less reliable than are needed, physical model tests should be conducted to confirm the hydraulic performance of the structures.

5.2 Wave Run-up

Wave action on a structure will cause the water surface to oscillate over a vertical range generally greater than the incident wave height. The extreme high level reached by waves on a structure is the wave run-up. It is the vertical height above the still water level to which water from an incident wave will run up the face of the structure. Variations in wave run-up throughout structure design life as result of climate change (including variations both water level and increased wind speed) should be checked at the beginning and end of the design life of the structures. In case of vertical structures, the run-up height is that of the crest of standing waves in front of them. The run- up level can be used to assess the required level of the crest of the structure or as an indicator of the occurrence of wave overtopping.

For design purpose, the amount of wave run-up is often indicated by $R_{u2\%}$, and is defined as the run-up level exceeded by 2% of the incident waves. Over most wave conditions and slopes, a rubble slope will dissipate more wave energy and result in less run-up than a smooth or non-porous slope does. This reduction is influenced by the permeability of the armour, filter and underlayers, and by the steepness and period of the waves. Methods to estimate the amount of wave run-up for rubble mound structures are given in Appendix B. Designers should take note of the range of testing conditions on which these methods are based.

5.3 Wave Overtopping

5.3.1 Mean Overtopping Rate

In the design of seawalls and breakwaters, the controlling hydraulic response is often the wave overtopping. If the crest level of a structure is exceeded by the wave run-up, wave overtopping will occur. Overtopping is not a continuous process but an intermittent occurrence at times of attack of individual high waves varying from one wave to another. The degree of wave overtopping is normally measured by the mean rate of overtopped water per metre run of the structure (m^3/s/m). Methods to estimate the overtopping rate for rubble mound and vertical structures are given in Appendix B.

Wave overtopping is affected by many factors; even a small modification of the geometry of a structure may change the amount of overtopping. Variations in wave overtopping throughout structure design life as result of climate change (including variations both water level and increased wind speed) should be checked at the beginning and end of the design life of the structures. Although there is no reliable conclusion, the increase of wave overtopping by an onshore wind is large when the quantity of overtopping is small and the wind effect decreases gradually as the overtopping rates increases. Hence, the methods given in Appendix B can only be used to provide general indication of the order of magnitude of the overtopping rate. More accurate estimate of the overtopping rate should be determined through hydraulic model tests.

Designers should determine the amount of overtopped water that would flow into the existing drainage system behind the seawall. Appropriate drainage provisions will need to be considered to avoid flooding or overloading the existing drainage system due to the overtopping wave. Designers should take particular attention in determining the overtopping rate at locations where sharp change in alignment of seawall or change in types of seawall occurs. Physical model or computer model may be used to determine the overtopping rate and the hydraulic performance of the structures when complicated situations are encountered.

5.3.2 Permissible Overtopping Rate

Wave overtopping can cause inconvenience or danger to personnel and vehicles, interruption to operations and flooding, and can induce instability to the crest and rear amour of the structure. The permissible rate of overtopping water depends on the usage of the crest of the structure or the land behind the structure, the strength of pavement against the impact of falling water mass, and the capacity of drainage facilities. Suggested limits of overtopping are (CIRIA, 1991):

Safety Considerations	*(m^3/s/m)*
Danger to personnel	3×10^{-5}
Unsafe to vehicle	2×10^{-5}
Damage to unpaved surface	5×10^{-2}
Damage to paved surface	2×10^{-1}

The above values are mean overtopping rates; peak values can be up to 100 times the average.

If there is pedestrian and vehicle movement or other operations on or near the structures, the permissible overtopping rates for personnel and vehicle should be satisfied for normal environmental conditions. For extreme environmental conditions, the checking of the overtopping discharge against the permissible rates for personnel and vehicle may not be necessary if operations such as pedestrian and vehicle movements cease at the structure. However, if the usage on or near the structure in extreme environmental conditions is critical, designers should determine on individual situations whether the permissible values for personnel and vehicle have to be met in extreme environmental conditions.

Damage to surface behind the structure due to repeated wave overtopping under extreme environmental conditions can affect the structural safety due to loss of fill from the core of the structure by erosion and leakage. The permissible overtopping rates for damage to unpaved or paved surface should be checked for extreme environmental conditions.

5.4 Wave Reflection

5.4.1 Reflected Wave Height

All coastal structures reflect some portion of the incident wave energy. The amount of wave reflection is often described by a reflection coefficient, C_r, defined in terms of the incident and reflected wave heights, H_i and H_r, or the incident and reflected wave energies, E_i and E_r :

$$C_r = \frac{H_r}{H_i} = \sqrt{\frac{E_r}{E_i}}$$

The reflection coefficient of solid vertical structure is normally greater than 0.9 whereas the reflection coefficient of rubble mound structure can vary from about 0.3 to 0.6, depending on the wave steepness and the slope of the structure. Empirical formulae which may be used to estimate the reflection coefficient for rubble mound structures are given in Appendix B.

The total wave height, H_{total}, due to the incident and reflected waves may be calculated by the principle of summation of energy components :

$$H_{total} = \sqrt{H_i^2 + H_r^2}$$

The theoretical basis of the above equation is that the significant wave height is proportional to the square root of the total wave energy, irrespective of the shape of the wave spectrum. The equation, however, is not applicable in the immediate vicinity of structures because of the fixed phase relationship between the incident and reflected waves. The equation is only applied to a distance of about one wavelength or more from the reflective structure as the phase interference cancels out among the various components of random sea waves.

5.4.2 Wave Reflection in the Harbour

When waves are reflected by a structure, the reflected waves causes increased agitation of the water in front of the structure and can affect vessel navigation and operations. New marine structures in the Victoria Harbour should be designed to achieve a reflection coefficient less than 0.5 for waves with periods less than 5 s to reduce the impact of reflected waves on vessels.

5.4.3 Wave Absorption Structures

Waves acting on a vertical structure can be absorbed by introducing wave absorption unit to reduce the reflected wave energy. The performance of the wave absorption structures is related to the incident wave period and should be determined by physical model testing. An example of wave absorption structure is shown in Figure 11. It consists of a wave chamber with perforated front wall. The main cause of energy dissipation is the energy loss of the water jets through the perforations at their outlets. Once the water jets are ejected from the outlets of the perforations, their kinetic energy is consumed by turbulence and eddies and cannot be recovered into the form of kinetic energy again by the entropy principle. The speed of water jets or the amount of the kinetic energy is controlled by the water level difference between the outside and the inside of the wave chamber, or the phase lag between the incident and reflected waves.

Physical model testing should be carried out to determine the most appropriate layout of the perforations, including the width and depth of wave absorption chamber and perforation ratio of the front wall. Perforation ratio is defined as the ratio between the total area of the perforations and the total area of the front wall. The model tests should cover different wave heights, periods and directions as well as water levels that occur at a particular site.

Wave absorption structure, if adopted within the Victoria Harbour, should be designed to cater for vessel waves with short periods in the range of 2 to 5 s.

5.5 Wave Transmission

Wave transmission is applicable to breakwater constructed with low crest level where waves overtop and transmit wave energy into sheltered waters. Long period waves transmitted through the breakwaters can cause movement of vessels and affect operations within the harbour behind the breakwaters.

Wave transmission is described by the coefficient of transmission, C_t, defined in terms of the incident and transmitted wave heights, H_i and H_t , or the incident and transmitted wave energies, E_i and E_t :

$$C_t = \frac{H_t}{H_i} = \sqrt{\frac{E_t}{E_i}}$$

The transmission performance of low-crested breakwaters is dependent on the structure geometry, principally the crest freeboard, crest width, water depth, permeability, and on the wave conditions, principally the wave period. Some empirical formulae based on the results of hydraulic model tests to estimate the transmission coefficient are given in Appendix B.

6. STRUCTURAL STABILITY

6.1 General

This chapter provides general guidance on assessing the structural stability of breakwaters and seawalls. However, as each design rule has its limitations, it may be necessary to perform physical model studies to verify the design for critical structures exposed to unfavourable environmental conditions.

Guidance on the determination of loads, loading conditions and combinations for the design of breakwaters and seawalls can be found in Chapter 5 of Part 1 of the Manual – General Design Considerations for Marine Works.

6.2 Rubble Mound Structures

6.2.1 General

The stability of rubble mound structures relies on whether the armour units can remain stable on the slope to protect the inner core of the structure under wave action. The underlayers bedding layers, core, toe protection and geometry of the structure such as crest width, height, slope and layer thickness interplay with the armour to provide the necessary stability of the structure as a whole. The design of these elements is discussed in this section. Guidance on checking the foundation stability against slip failure is given in Chapter 4 of this part of the Manual.

The definition sketch for rubble mound breakwaters and seawalls is shown in Figure 12.

6.2.2 Weight of Armour Units

Common methods to determine the weight of armour units include the Hudson formula and the Van der Meer formulae, details of which are given in Appendix C. Reader can make reference to Van Gent M.R.A., Smale A.J. and Kuiper C. (2003) which discusses stability of rock slopes with shallow foreshores. General comments on the application of these formulae on rock armour are given below.

(1) Hudson Formula

The Hudson formula, developed for rock armour, was derived from results of regular wave tests for armour stability in conditions when the crest of the structure is high enough to prevent major overtopping. The formula has been widely used because of its simplicity and the long period of application. The formula, however, does not take account of many factors such as wave period and spectrum, angle of incident wave, shape, type and interlocking of armour units, method of placing armour units, size and porosity of underlayer material, and effect of the crest elevation relative to wave height. The formula should not be used for a low crest structure.

(2) Van der Meer Formula

The Van der Meer formulae were established from the results of a series of model tests using irregular waves which better reflect the real conditions of the sea state. These formulae are based on a wide set of model data and are considered as the most widely applicable of the prediction methods currently available. The Van der Meer formulae are more complex than the Hudson formula and take account of the following variables which are not included in the latter :

- Wave period.
- Breaker parameter.
- Duration of storm.
- Permeability of the core of the structure.
- Damage level.
- Breaking wave conditions.

Details of the formulae and range of applicability are described in Appendix C. In the formulae, the permeability of the structure is represented by a notional permeability factor *P* (see Figure 13). The suggested values of *P* range from 0.1 for a relatively impermeable core to 0.6 for a virtually homogeneous rock structure. Designers should note that the values of *P* are only assumed and not related to the actual core permeability. For good design practice, the formulae should not be used for conditions outside those given in Appendix C, and sensitivity of the calculated rock weight should be performed for all parameters in the formulae, including the full range of wave period.

(3) Crest and Rear Face Armour

The stability of armour on the crest of a rubble mound structure may be less than the stability of those on the seaward slope because of the reduced interlocking among armour units on the crest. For breakwaters, wave overtopping may also induce instability on the rear face armour. No analytical methods are available for determining the size of these armour units.

For determining the size of these armour units, reader can make reference to Coastal Engineering Manual (CEM, 2002) and The Rock Manual (CIRIA, 2007) which discuss analytical methods for sizing crest and rear face armour. Physical model tests are recommended for severely overtopped or submerged structures to determine the required size of the armour.

(4) Concrete Armour Units

Information on the use and design of particular concrete armour units should be obtained from literature published by the originator or licensee of the unit. BS 6349:Part 7:1991 (BSI, 1991) also provides some general guidance on the use of these units.

6.2.3 Thickness and Extent of Armour Layer

The thickness of the armour layer t_a may be obtained from the following formula :

$$t_a = nk_\Delta \left(\frac{W_a}{\gamma_a} \right)^{1/3}$$

where W_a = Weight of an individual armour unit (N).
n = Number of armour layers.
k_Δ = Layer thickness coefficient.
γ_a = Unit weight of armour unit (N/m^3).

The average number of armour units per unit area N_a may be determined by the following formula :

$$N_a = nk_\Delta \left(1 - \frac{p}{100} \right) \left(\frac{\gamma_a}{W_a} \right)^{2/3}$$

where p = Volumetric porosity.

The thickness of randomly placed rock armour should normally be designed to contain a double layer of rocks (n = 2), with layer thickness coefficient equal to 1.15 and volumetric porosity equal to 0.37. The average number of armour units per unit area should be specified to ensure that sufficient units are placed on the structure. For concrete armour units, two layers of units are normally provided but in any case the method of placing should been based on careful testing or as recommended by the originator or licensee of the concrete armour units.

The armour layer should extend below the lowest design water level to a depth equal to 2 times $H_{1/3}$. For deep water structures, the slope below the level at which the primary armour terminates should be protected by rock having a size not less than that required for the underlayer. In shallow water where the waves break, the armour in the primary layer should be extended over the entire slope.

6.2.4 Underlayers and Core

The weight of the underlayer rock should normally be taken as not less than one-tenth of the weight of the armour. The size of individual underlayer rock should be within ± 30% of the nominal weight selected. This applies where the armour layer is made up of rock. For concrete armour units, recommendations on the weight of underlayer rock can be found in BS 6349:Part 7:1991.

The thickness of the underlayer t_u should contain at least two layers of rock and may be determined from the following formula :

$$t_u = nk_\Delta \left(\frac{W}{\gamma_r} \right)^{1/3}$$

where W = Weight of a rock in the underlayer (N).
n = Number of rock layers.
k_Δ = Layer thickness coefficient, equal to 1.15 for rock.
γ_r = Unit weight of rock (N/m^3).

For the filter action between successive underlayers and between the lower underlayer and the core, the filter criteria given in BS 6349:Part 7:1991 (BSI, 1991) may be used to determine the size of the underlayers in relation to the core :

$$D_{15u}/D_{85c} \leq 4 \text{ to } 5$$

$$4 \leq D_{15u}/D_{15c} \leq 20 \text{ to } 25$$

where D is the nominal size of an equivalent cube.
Suffix 'c' refers to core.
Suffix 'u' refers to underlayer.
Suffixes '15' and '85' refer to the percentage of material passing through that size.

When applying the above criteria, some disturbance of the finer material and possible migration through the overlying material due to varying wave induced water movements is still possible. A conservative approach should be adopted in the design of the filter.

When the rubble mound structure is protecting a reclamation, adequate filter should also be provided to prevent loss of fine material through the core. The following filter criteria is given in BS 6349:Part 7:1991 :

$$D_{15(\text{larger})}/D_{85(\text{smaller})} \leq 4 \text{ to } 5$$

$$4 \leq D_{15(\text{larger})}/D_{15(\text{smaller})} \leq 20 \text{ to } 25$$

$$D_{50(\text{larger})}/D_{50(\text{smaller})} \leq 25$$

where D is the nominal size of an equivalent cube.
Suffixes '15', '50' and '85' refer to the percentage of material passing through that size.

The following points should be noted when designing the filter layer between the rubble mound structure and the reclamation fill :

- No filter layer should contain more than 5% of material by weight passing 63 μm sieve and that fraction should be cohesionless.
- Filter material should be well graded within the specified limits and its grading curve should have approximately the same shape as the grading curve of the protected material.
- Where the retained fill material contains a large proportion of gravel or coarser material, the filter should be designed on the basis of the grading of that proportion of the protected material finer than a 20 mm sieve.
- Where the retained fill is gap graded, the coarse particles should be ignored and the grading limits for the filter should be selected on the grading curve of the finer soil.
- Where a filter protects a variable soil, the filter should be designed to protect the finest soil.
- The thickness of filter layers should be ample to ensure integrity of the filter

- when placed underwater. In practice, the thickness of filter layer at 1 m below and 0.5 m above water level should be the minimum thickness of $4D_{85}$(filter layer).
- The filters should cover the full depth of the structure.

6.2.5 Slope of Structure

The slope angle of the structure depends on hydraulics and geotechnical stability, and should generally be not steeper than 1 (vertical) : 1.5 (horizontal).

6.2.6 Crest

The crest elevation should be determined from wave run-up and overtopping considerations. An allowance for the settlement that will occur in the design life of the structure and its foundation, including the effects from any adjacent reclamation should also be included in determining the crest elevation.

The crest width should be sufficient to accommodate any construction, operation and maintenance activities on the structure. For rubble mound breakwaters, the minimum crest width B should be sufficient to accommodate at least three crest armour units and may be determined from the following formula :

$$B = 3k_{\Delta}\left(\frac{W_a}{\gamma_a}\right)^{1/3}$$

where W_a = Weight of an individual armour unit (N).
k_{Δ} = Layer thickness coefficient.
γ_a = Unit weight of armour unit (N/m^3).

6.2.7 Crest Structures

A crest structure may be constructed on the structure to provide access or act as a wave wall to prevent or reduce overtopping. Typical form of crest structures for rubble mound breakwaters are shown in Figure 14. The underside of the crest structure may be keyed into the underlying material to increase sliding resistance.

6.2.8 Toe Protection

Wave action in front of the structure can cause severe turbulence at the seabed. In particular, the toe of the structure can be exposed to the action of breaking waves in shallow water, leading to erosion of seabed material and scouring of toe. Figure 15 shows different toe details for rubble mound structures under different wave and ground conditions. The extent of toe protection and the rock size at toe may be determined from Figure 16 for the case of rubble mound in front of vertical and composite breakwaters. Where currents are combined with wave action, it is suggested that the weight of the rock for protection against wave scour should be increased by 50% (BSI, 1991). Alternatively, the shear stresses due to the combined effect of waves and currents may be calculated to determine the required toe protection.

Fine material at the seabed is liable to be scoured. The design may place rubble to act as a falling apron as shown in Figure 17 for toe protection.

6.2.9 Breakwater Head

The breakwater head may be more exposed than other parts of the structure for the following reasons :

- The head is usually exposed to attack by waves approaching from a wider range of directions.
- Increased wave disturbance can arise due to reflection or diffraction by the structure or by the other breakwater at the entrance of the typhoon shelter or harbour basin, or due to the effect of the slope around a breakwater head on wave refraction, or by the effect of the presence of dredged channel or change in seabed level as a result of littoral drift or bar formation.
- Currents can be more pronounced than other parts of the breakwater.
- The curvature of a breakwater with roundhead construction can reduce the interlock between the armour units. The wave action at the roundhead will result in higher water velocities over parts of the rear slope than elsewhere; it is often found that this is the region of the least armour stability.

BS 6349:Part 7:1991 recommends that the breakwater head should be designed with greater strength than the breakwater trunk in order to achieve comparable stability under the same wave conditions. This can be achieved by :

- Using larger armour units or flatter slope, or by a combination of both.
- Increasing the thickness, and hence the permeability, of the armour layer.
- Increasing the crest width.

Such measures should be applied around the head and along both sides of the trunk for a distance of typically 1 to 2 times the overall height of the breakwater tip. A smooth transition should be provided between the roundhead and the trunk. A typical breakwater roundhead construction is shown in Figure 18.

Some types of concrete armour units, such as Tetrapod and Dolos, are less stable under oblique waves than under waves perpendicular to the structure (BSI, 1991). The above measures should be adopted when units displaying such characteristic are used.

The measures at the breakwater head should also be considered at the following conditions :

- Where the breakwater has sharp changes in direction.
- At the ends of the breakwater where there is a junction with a vertical structure.
- Where other types of construction or structure such as extensive culvert wing walls have been incorporated into a length of the breakwater.

The length of structure to be considered as corresponding to head conditions is dependent on site conditions, crest level and armour slope, and must be decided by the designer in each case. For small structures with significant junctions or discontinuities where head conditions apply, it may be justified to use configuration corresponding to head conditions for the full structure length.

The above guidance should also be applied to rubble mound seawalls if similar breakwater head conditions are encountered.

6.3 Vertical Structures

6.3.1 General

Vertical structures derive their stability largely from their self-weight. Failure by overturning occurs when the overturning moment due to the disturbing forces exceeds the restoring moment due to the weight of the structure. Sliding takes place when the frictional resistance between the base of the structure and the foundation is insufficient to withstand the disturbing forces. Bearing capacity failure occurs when the contact pressure beneath the base of the structure exceeds the bearing capacity of the foundation. The wave action can lead to toe scour or undermining, affecting the stability of the structure. If slip surface is developed in the structure or foundation, slip failure will occur. Recommended minimum factors of safety against soil shear failure are given in Chapter 4 of this part of the Manual. Minimum factors of safety against overturning, sliding and bearing capacity are given in this section.

6.3.2 Overturning, Sliding and Bearing Capacity

The following minimum factors of safety against overturning, sliding and bearing capacity failure of a vertical structure under various loading conditions are recommended :

Loading Conditions	*Overturning*	*Sliding*	*Bearing Capacity*
Normal	2.0	1.75	2.5
Extreme	1.5	1.5	2.0
Accidental	1.5	1.5	2.0

For overturning, it is recommended that the resultant should lie within the middle third of the base width under normal loading conditions when transient loads are ignored.

For sliding, the recommended factors of safety also apply to sliding at horizontal block interfaces in the case of concrete blockwork seawall. The coefficient of friction at the interface of two concrete blocks and at the interface of a concrete block and a levelled rubble mound foundation may be taken as 0.6.

The factors of safety for temporary loading conditions should be assessed by the designer for each individual case.

The methods of calculating the above factors of safety for vertical seawalls and breakwaters are given in Figures 19 and 20. The factors of safety should be assessed under the most severe combinations of loading, wave positions and water levels.

Chapter 5 of Part 1 of the Manual recommends that a tidal lag of not less than 0.7 m and 1.0 m above the still water level under normal loading conditions and extreme loading conditions respectively may be applied in relatively simple ground conditions behind a seawall. On the basis of this assumption, typical water levels shown in Table 3 should normally be considered in seawall design. However, it should be noted that for different types of structures, different loading cases and conditions, the critical still water level may be the minimum, maximum or some intermediate levels within those shown in Table 3, and therefore should be assessed by the designers for each case. The ground water level should take into consideration the worst credible ground water conditions, for example, in the case where flow from land sources is significant. Tidal lag is not applicable to breakwaters.

The major lateral loads acting on a vertical seawall and a vertical breakwater are different. The critical lateral loads for a vertical seawall may include the lateral earth pressure due to fill and surcharge behind the seawall and the wave suction in front of the structure under the effect of a wave trough. For a vertical breakwater, as the structure is surrounded by water, the critical lateral load may be the wave load due to a wave crest acting on the seaward face of the structure with gentle wave condition inside the shelter. This should be noted in the design.

6.3.3 Design Wave Height

The design wave height for assessing the structural stability should be taken as the maximum wave height H_{max} experienced as a result of increased wind speed variation throughout design life of structure.

In deepwater, the most probable maximum value of H_{max}, as mentioned in Chapter 2 of Part 1 of the Manual, is given by :

$$H_{\max} \approx \left(0.706\sqrt{\ln N_0}\right)H_{1/3} \approx (1.6\sim2.0)H_{1/3}$$

where $H_{1/3}$ is the significant wave height.
N_0 is the number of waves during a peak of storm events.

For design purpose, to assess the wave pressure under wave crest, H_{max} is generally taken as $1.8H_{1/3}$ if the structure is located seaward of the surf zone. Within the surf zone where wave breaking takes place, the design wave height is taken as the highest of the random breaking waves H_{max} at the location of a distance equal to $5H_{1/3}$ seaward of the structure as given by the Goda method in Appendix A of Part 1 of the Manual. The design wave period can be taken as the significant wave period. The corresponding wave pressure formulae according to Goda are given in Section 5.10.3 of Part 1 of the Manual.

To assess the wave pressure under wave trough, the maximum wave height H_{max} is taken to be $1.8H_{1/3}$. It should be noted that the solution for wave pressure under a wave trough, in particular that of breaking waves, has not yet been fully developed. But as far as the pressure of standing waves is concerned, the wave pressure distribution under the trough may be determined according to the Sainflou theory as given in Section 5.10.3 of Part 1 of the Manual.

Reference should be made to Section 5.10.2 of Part 1 of the Manual regarding the wave conditions to be considered in design. Typical wave conditions with respect to water levels are given in Table 3.

6.3.4 Impulsive Wave Pressure

An impulsive wave pressure will be exerted on a vertical wall when incident waves begin to break in front of the wall and collide with it, having a wave front which is almost vertical. The impulsive pressure caused by breaking waves is much greater than the pressure usually adopted in the design of vertical structures mentioned above. Hence, these structures should be located in such a way to avoid direct exposure to impulsive breaking wave pressure. A rubble mound breakwater may be more suitable in such a situation. If space is limited or if little wave transmission is to be allowed, a vertical breakwater protected by a mound of rock or concrete blocks of the energy-dissipating type may be an alternative design.

It is difficult to describe precisely the occurrence condition of the impulsive breaking wave pressure but the possibility of its generation may be judged to a certain extent with reference to the guideline given in Table 4. It should be noted that the guideline is of a rather qualitative nature, and many cases may fall in the border zone. This uncertainty is inevitable because the phenomenon is affected by many factors in a complex and delicate manner. Physical model testing should be carried out if in doubt. Further guidance on the assessment of the impulsive breaking wave pressure can be found in Goda (2000).

6.3.5 Toe Protection

The extent of toe protection and the rock size at the toe may be determined in accordance with the guidance given in Section 6.2.8 of this part of the Manual.

6.3.6 Breakwater Head

In contrary to rubble mound breakwaters, upright sections of vertical and composite breakwaters at their head sections may be designed in the same manner as for their trunk sections. However, the bermstones at the breakwater heads are more susceptible to scour, because they are exposed to strong wave-induced currents around the corners of the upright sections. The effects of scour may be reduced by :

- Providing an outer face curved on plan for upright section at the breakwater head.
- Increasing the anti-scour protection : the width of the protection and the weight of the rock or blocks may be increased by at least 50%; such protection should be continued along the main face for a suitable distance.

6.4 Vertical Wave Absorption Seawalls

The loading and stability analysis of vertical wave absorption seawalls should generally follow those guidelines for vertical structures mentioned in previous sections. In the presence of wave absorption chamber, the wave pressure on various structural elements of the chamber should be assessed to determine the worst loading combinations. For side walls and bottom slabs, the Extended Goda Formulae that take in account the effect of impulsive wave pressure may be applied to estimate the wave loading. Details of the Extended Goda Formulae are given in Tsinker (1997).

Information on the magnitude of wave pressure on top slabs of wave absorption seawall is limited. Laboratory model testing had once been carried out for wave absorption seawall (HKU, 1998) with removable panels on the front face of the seawall. The testing results indicated that the uplift pressure increased significantly when the front panel was removed. Without the front panel, waves directly impinged onto the rear wall and caused higher run-up along the rear wall and against the top slab, and resulted in significant increase in the uplift. The testing conditions and results are summarized in Table 5. However, there is still much uncertainty about wave impact pressures and the physical processes that govern it. Most researchers believe that small-scale experiments tend to over-predict impact pressures. It should be noted that both the overtopping pressure and the uplift are highly sensitive to the difference in elevation between the still water level and the slab soffit. Designers should exercise great care when trying to apply test results under different design wave conditions and levels of slab soffit.

7. CONSTRUCTION

7.1 General

This chapter covers aspects requiring attention during supervision of the construction of seawalls and breakwaters. The General Specification for Civil Engineering Works (GS) (HKSARG, 2020) should be referred to for information on general construction requirements.

7.2 Foundation Dredging

7.2.1 General

The quality of the remaining material at and below the bottom of the dredged trench is an important consideration in determining the dredging level. The stability of trench side slope is also important as it is required to be stable until the trench has been filled with foundation material. These dredging parameters are related to the overall stability of the structure and should be determined through a thorough stability analysis mentioned in Chapter 4 of this part of the Manual.

7.2.2 Samples of Dredged Materials

Sampling of dredged material should commence when the depth of dredging has reached about 5 m above the design founding level. Samples should then be taken at regular depth intervals of approximately 2 m to identify any change in stratum or material quality. Each sample should have a mass of about 1 kg and be labeled with location, depth, level, date, time and dredging method. The sample should preferably be taken from the centre of a grab or bucket load. For a trailer suction dredger, the sample should be taken from the pipe discharging into the hopper.

7.2.3 Dredging Profile and Depth

Normally, the level of dredged foundation trench is determined by relying on the rule of thumb of 70% sand content (by weight) in the dredged material. The rationale is to ensure that the sand content of material in-situ below the dredging level is not smaller than that of the backfill material so as to avoid undue stability and long term settlement problems.

For foundation dredging in particular where clayey or silty materials remain at the trench bottom, measures to determine suitability of the founding material, including field and laboratory tests, should be identified in the design stage and specified in the contract. Field tests to assess the in-situ soil strength includes vane shear test, standard penetration test and cone penetration test as described in Chapter 4 of Part 1 of the Manual. To ensure the compatibility between design assumptions and actual site conditions, comparison should be made between information on soil stratification and strength parameters obtained during the design stage and those revealed during construction, taking into account the following information :

- Soil strata according to the information revealed from samples collected from dredged material.
- Vertical and lateral variability of the soil profiles along the foundation of the structures.
- Results of the in-situ sand content tests, hand vane shear tests or other field tests.
- Difficulty or obstruction encountered in the dredging works.

If there are substantial deviations in soil strata from the design assumptions or if suitable soil stratum is encountered prior to reaching the design depth, stability calculations should be reviewed to determine if the dimensions of the dredged trench are adequate.

If the soil at the designed depth is not suitable, dredging should be continued until a suitable stratum is reached. Alternatively, instead of dredging further downwards, the width of the dredged trench may be widened, subject to further stability calculation based on the strength reflected from the in-situ field tests. A combination of widening and deepening the foundation trench may be adopted to optimize the dredging effort.

In case of doubt, further ground investigation and field or laboratory testing should be conducted to confirm the soil conditions.

7.2.4 Disposal of Dredged Materials

Disposal requirements of dredged materials are given in ETWB TCW 34/2002 on Management of Dredged/Excavated Sediment (ETWB, 2002).

It is generally not necessary to physically ensure that dredged materials are disposed of at designated disposal ground, as this is a legal requirement of the dumping permit issued by and policed by the Environmental Protection Department. However, periodic checks should be made that the contractor's barges are properly licensed and have appropriate dumping permits. The periods between the barges leaving full from the site and returning empty should also be checked to ensure they are compatible with the time that the trip to the appropriate disposal ground should take.

7.3 Soil Strengthening

When applying deep cement mixing or stone-column technique, stringent quality control and monitoring are required to ensure that the required strength is developed in the soil. These measures may include :

- Trial soil treatment on site to ascertain the soil strengthening parameters with respect to actual soil conditions before full-scale construction of the foundation. Performance control of the treatment process and depth of treatment; for example, the gravel consumption rate and compaction effort versus depth in the construction of stone columns; or in deep cement mixing, the consumption of stabilizing agents, the penetration time and withdrawal velocity of mixing equipment versus depth.
- Water quality monitoring to detect if there is leakage of stabilizing agents to the
- water environment in deep cement mixing or intermixing of soil and gravel with seawater during the installation of stone columns.
- Monitoring of the stability of adjacent seabed or structures.
- Undertaking verification and acceptance tests of the treated soils.
- Post construction monitoring on the stability and settlement behaviour of the
- foundation.

Specialist input is required in drafting the specification for these techniques and supervising their application on site.

7.4 Fill Placement

The rate of fill placement behind a just-completed seawall should be controlled. In particular, for seawall foundation resting on silty or clayey soil, the excess pore water pressure developed during filling may not be able to dissipate if the rate of filling is high, and this will induce instability to the seawall. On the other hand, if the loading condition at construction, during which the undrained shear strength of the silty or clayey soil before consolidation takes place is critical, has been checked in the design, the rate of fill placement will probably be not of prime concern in principle. Nevertheless, as a good engineering practice, placement of large quantity of fill behind a just-completed seawall within a short period should be avoided and the dissipation of excess pore water pressure in the founding material should be monitored during construction.

The presence of tension cracks or rapid increase in settlement on newly reclaimed land may indicate the possibility of movements of the founding strata, which could lead to the failure of a seawall. When such sign is observed on site, all works including the filling operation and the construction of seawalls should be stopped, and the project engineer should immediately initiate a thorough investigation to identify the cause and develop a remedial plan, if necessary. The construction work should be resumed only upon the rectification of the cause of potential failure.

To allow for subsequent settlement during the construction period, the levelling rock fill at the top of the foundation may be raised above the required design level. The amount depends on many variables, including the characteristics of the underlying foundation material, the thickness of any sand and rock filling, the mass of the works to be constructed on the foundation, and the expected construction period. This amount of set-up should be specified in the contract.

Where fill will be deposited in a foundation trench, it is important to check that there has been no significant deposition or accumulation of soft deposits in the bottom of the trench between completion of dredging and the start of filling. This is particularly important when there has been a period of high waves during a storm. Such checking can be carried out by diver, grab sampling or repeating the survey, or a combination of these as appropriate. No fill should be placed until the dredged profile is agreed and approved.

7.5 Rock Armour and Underlayers

Rock in armour layers and underlayers in rubble mound construction should normally be placed from the bottom to the top of a section, in such a manner and sequence that individual rock pieces interlock and do not segregate and the interstices are kept free of small rock fragments. These requirements are particularly important as they relate directly to design assumptions covering stability against wave attack and wave run-up. There should be no free pieces on the surface of a completed layer, and all pieces should be wedged and locked together so that they are not free to move without disturbing adjacent pieces in the same layer.

Armour is the most important layer for the stability of the rubble mound. The armour layer should be placed as soon as possible following the placement of underlayer to avoid damages to these layers, which may be difficult to repair. It is advisable to keep a sufficient number of rock armour in stock on site to ensure rapid placement in case of an unfavorable weather forecast. Each armour should be placed individually, after inspection to ensure that it is within the specified weight range, uncracked and of acceptable shape.

The core and underlayers are liable to damage by wave action during construction. If continuous rough weather is expected, it may be necessary to cease work before the onset of rough weather and provide temporary protection to the unfinished work. It is advisable to limit the extent to which the core is constructed ahead of the underlayer, and the underlayer ahead of the armour, to reduce the risk of storm damage and consequent delay.

For rock armour layers and underlayers above water level, final visual inspections from the top of the slope and by boat from the bottom of the slope should be carried out in addition to the normal profile check by survey. Below water level, a final visual inspection by diver is recommended where possible, depending on visibility, particularly for rock armour layers. If any significant holes or areas with infilled interstices are detected, whether above or below water level, it will be difficult for these to be satisfactorily rectified without almost complete reconstruction of the adjacent areas.

The method of survey should be agreed with the surveyor before the work starts to ensure that readings are taken at truly representative points but that any high and low spots are also identified. It should be noted that it is unable to fully control the thickness of armour layer by sounding or levelling surveys. The number of rock for a stated area specified in the drawings should therefore be checked to ensure adequate coverage and thickness of the armour layer.

7.6 Concrete Armour

Concrete armour units are in general mass concrete and only occasionally contain reinforcement. Opinions are divided on the effectiveness of reinforcement in armour units as, if the steel corrodes, the adverse effect on durability can outweigh any advantages in using it (BSI, 1991). High quality concrete should always be used, but caution should be exercised on the use of high cement contents because of the risk of shrinkage cracking, particularly with large armour units. Concrete mixes for large units should be designed to reduce temperature differentials and moulds should be designed to avoid cracking of concrete due to thermal stresses. Low heat cement is advisable. Concrete production, casting, curing, stripping of formwork, delivery to stockyard, transporting and placing should be arranged and programmed to minimize stresses. Sufficient number of armour units may be kept on site to enable rapid placement to protect the underlayers and the core in case of an unfavourable weather condition.

Concrete armour units may be placed randomly or in a regular pattern. They range from massive approximately cubical units such as cubes to the more complex forms such as Tetrapods and Dolosse. The massive types are intended to function in a way similar to natural rock, while the more complex units depend upon the interlocking between units to achieve the hydraulic stability. True random placing is difficult to achieve, and inevitably results in some units not being as well interlocked as others. Although placing to a predetermined layout is usually specified for interlocking units, this is also difficult to achieve except under favourable conditions of good underwater visibility and calm seas. The result may be a semi-random pattern. Specific recommendations on the placing method should be checked with the originator or licensee.

Cracks resulting from stresses arising during construction, delivery and placement can significantly reduce the capacity of the concrete armour units to resist wave loads and therefore they should be handled with great care under close supervision. Full scale dynamic loading tests can be carried out on site to check the impact resistance of the units. These can take the form of drop tests in which a unit is dropped from varying heights onto a concrete or rubble surface. Results of these tests have shown that the flexural strength may be reduced by 60% after 6 to 10 impacts. Further information, including suggested maximum sizes of concrete armour units, can be found in BS 6349:Part 7:1991.

The conditions of the concrete armour units should be closely checked on site. Cracked or broken units should not be used as armour. A thorough inspection should be made on each unit immediately before and after placing. Damaged units should be removed immediately even though it has been placed on the slope.

7.7 Bermstones

Bermstones should be placed as soon as practicable to protect the toe of the structure against scouring due to waves and currents. Early placement is particularly important when one or more of the following conditions apply :

- The location is subject to strong currents.
- The location is exposed to wave attack.
- When works are carried out in season during which tropical storms may be frequent.
- The water depth in front of the structure is shallow.

Underwater inspection is important to ensure that bermstones have been placed over the foundation width required and that the gap between bermstones are kept to the minimum.

7.8 Concrete Seawall Blocks

Precast blocks for concrete blockwork structures are normally made of mass concrete with a characteristic strength of 20 MPa. The ease and accuracy of construction is dependent on the accuracy of the shape and size of the blocks being used, and the accuracy and consistency of the levelling stones on top of the foundation. It is important for the levels of the rails or other profile marks to be checked by surveyor before laying of the levelling stones starts, and for the levelling stones to be inspected by diver before any block setting.

Daily records for the casting and setting of blocks should be kept. In addition, record drawings giving the date of setting of each block should be kept in the site office. After the setting of each layer of blocks has been completed, a diving inspection should be carried out to check such matters as the accuracy of setting, joint widths, infilling of gaps between adjacent blocks and cleanliness of the top surface for receiving the next layer of blocks.

7.9 Facing Stones and Copings

The construction of in-situ concrete copings and the pointing of facing stones of a seawall should preferably be carried out as late as possible in the construction programme in order to allow for the effects of settlement. Subject to user requirements, the works of these two items may be delayed until towards the end of the construction period of a project.

7.10 Caissons

Concrete caissons consist of open-topped cells prefabricated in the dry and are usually floated to their final location for sinking onto prepared foundation in the seabed. Caissons are generally of rectangular shape in plan and subdivided into cells for strength and for control of stability during towing, sinking and filling when in the final position.

Filling should be carried out as soon as the caisson is positioned for the sake of stability. For breakwaters, compartments are completely filled for stability under wave loading. For seawalls, the seaward compartments may be better left either empty or partially filled to adjust the overall centre of gravity and reduce bearing pressures if the front wall is not used for vessel berthing. Fill compaction can be carried out to provide a secure foundation to the superstructure. Lean concrete may be used in seawall compartment where necessary to provide increased resistance to impact loads such as vessel berthing loads. The capping should not be cast until the caissons have been filled. Scour protection against wave and current actions should also be completed as soon as possible after placing of the caissons.

7.11 Joints for Seawall Caissons

Movement joints should generally be provided in the reinforced concrete capping of seawalls at centres not exceeding 30 m. The capping should be effectively anchored to the wall and to the counterforts.

Gaps between caissons for seawalls should generally be closed to prevent water movement and to protect the bedding layer from scour by high velocity currents caused by wave action. The joint seal on the seaward face should be made as close as practicable to the seaward face to keep the depth of the gap between caisson walls to a minimum. Where storm wave action is possible at any time during construction, the joint should be completed as soon as possible.

Key joints are sometimes necessary to transmit load between caissons to avoid relative movement and should be capable of shear transmission of 25% of the maximum horizontal load on either caisson to the adjacent unit (BSI, 1991). Except where caissons are placed on a rock foundation, some relative settlement is likely to take place and joints should provide for vertical movement.

Where differential settlement between caissons is possible, the joint faces should be painted with slip coat such as bitumen to avoid bond between the joint plug and the caissons. The gap can be sealed at the face using a grout sock or tube, with tremie concrete being used to form the joint plug.

8. MARINE AND MAINTENANCE FACILITIES

8.1 General

This chapter gives general guidance on the provision of marine and maintenance facilities on seawalls and breakwaters.

8.2 Marine Facilities

Marine facilities should be provided on seawalls and breakwaters after consultation with the Director of Marine and other users. Where there is berthing requirement, public landings, handrails, ladders and covers may be required to facilitate cargo handling or passenger loading and unloading. Fenders are normally provided at public landings; but for public waterfronts and public cargo working areas, fenders are usually not required.

Depending on the cope or crest level, public landings should consist of intermediate landings to cater for different tidal levels. To prevent passengers from walking on slippery surface, rough cast finish with thickness of 25 mm should be provided on the landing steps. Capping units should be provided on the top of the fenders to close the gap between the landing and the vessel. Landings should be furnished with stainless steel handrails.

Use of timbers as fendering system is not environmentally friendly and not recommended. If rubber fenders are used, general guidance can be found in Chapter 6 of Part 1 of the Manual.

Bollards and mooring eyes are required to allow vessels to berth and moor against the structure. Standard 10-tonne bollards at about 8 m to 10 m centres are usually provided for vessels up to 2000 t displacement tonnage. However, bollards of 30 t or higher may be required, for example, in public cargo working areas, to cater for larger vessels. Mooring eyes are provided on the vertical face of structures for mooring of small vessels with small freeboard not practical to have mooring ropes fixed to a bollard. The mooring eyes should be recessed into the structures so that they will not affect the movement of vessels induced by waves and tidal variations.

Depending on the use of the structures, the following facilities should also be provided :

- Cranes and mechanical handling equipment.

- Area lighting.
- Stainless steel handrails.
- Navigation light.
- Notice boards.
- Fire-fighting equipment.
- Water and electricity services.
- Marine structure identification plate showing the marine structure number.

The marine structure number should be obtained from the Civil Engineering Department for structures maintained by this department.

For walkway constructed on the crest of the structure but not designed for public access, security measures and adequate notices should be provided to avoid misuse by the public.

8.3 Maintenance Facilities

Typical inspection and maintenance accessories to be provided on seawalls and breakwaters include :

- Stainless steel catladders for access to vertical seawalls and breakwaters.
- Access steps, which may be in the form of precast concrete blocks, constructed on the slope of rubble mound breakwaters or sloping seawalls, from low tide level to the crest of the structures.
- Handrails and lifting hooks as appropriate.

Unlike conventional seawalls or breakwaters, structures with wave absorption chambers require a number of maintenance related considerations that designers should address in the design, as detailed below.

Due to the presence of perforations, the structural strength of the front panel is significantly weaker than a solid wall. Hence, if the structure is to be used for berthing of vessels, particular attention should be paid to the installation of fenders in front of the structure and the front panel should be checked against the berthing forces. For ease of maintenance, the front panels may be designed to be readily removable. The construction of a short section of conventional blockwork landing step or pier structure amid a long length of wave absorption seawall may be required to meet the berthing requirement.

The perforations on the structure may cause floating debris to be trapped inside the wave chamber due to the effect of tidal and wave action. Adequate access and facilities should therefore be provided for routine clearance work. A continuous walkway with anti-slip finish cantilevered from the rear face of the wave chamber may be constructed as working platform. Manholes should be specified for access to the walkway. Manhole covers should be designed to be watertight and fixed by steel bolts designed to withstand the uplift wave pressure acting on the top slab. To facilitate underwater inspection, openings should also be provided at the cross wall of the structure. Suitable anchors and lifting hooks should be provided for fastening of safety belts and for easy maneuvering of maintenance materials within the wave chambers.

The actual requirements of maintenance facilities are dependent on the nature and type of the structures and should be agreed with the maintenance authority.

9. MISCELLANEOUS STRUCTURES

9.1 General

This chapter provides general guidelines on the design of marine structures that may be associated with the construction of seawalls and breakwaters. These include pumphouses, slipways, ramps, outfalls, intakes and beacons.

9.2 Pumphouses

9.2.1 General

Pumphouses covered by this Manual include sets of individual small units, interconnected small units and larger units for installation of pumps to provide salt water for buildings such as those for air-conditioning purposes.

9.2.2 Layout and Location

In the design and construction of pumphouses, the requirements of the size, layout, facilities and fittings should be agreed in advance with the client. The following points should be noted when selecting a site for a pumphouse :

- The intake should be remote from sewage outfalls and other sources of contamination and debris, and also from salt water outlets which discharge heated water.
- The seabed should be sufficiently deep to accommodate the intake, after allowance for silting.
- The water in front of the intake should not be stagnant and the adjacent seawall should not be used for berthing.

9.2.3 Structure and Design

Pumphouses normally consist of reinforced concrete units, precast where placed below water level and cast in-situ above water level. They should be designed to BS 6349: Part 2 (BSI, 2010) and BS EN1992-1-1:2004 (BSI, 2004). The concrete and steel reinforcement parameters given in Table 41 of Part 1 of the Manual should be adopted. Inparticular, the exposure condition for salt water pumphouses should be XS3 in accordance with Table 4.1 of BS EN 1992-1-1:2004 (BSI, 2004).

Pumphouse units are usually constructed as part of a seawall. To avoid possible future settlement problems, it is important that the underlying ground is consolidated, for example, by preloading, before the setting of the pumphouse units. This is particularly important where a pumphouse is to be constructed as an extension to or immediately behind an existing seawall. The pumphouse units may be connected to the sea by intakes formed in special precast concrete blocks. To ensure satisfactory operation of the pumps in all tidal and wave conditions, it is recommended that the crown of the intake should generally be at a level not higher than –0.75 mPD.

For ease of construction and to minimize the number of joints, precast pumphouse units should be individual self-contained units with walls formed to as high a level as possible, subject to weight limitations, and preferably to a level between mean sea level and mean higher high water level for harbour locations. For larger pumphouses, sets of units can be interconnected above the junction between the precast and in-situ concrete level. It is usual for precast pumphouse units to be cast on a waterfront site, lifted by crane, transported to the pumphouse site by barge, and set in position by crane. For this method of construction, the weight of an individual unit is limited by the lifting capacity of available plant; units within the weight range of 500 kN to 1000 kN are relatively common. Another method of construction is for the unit to be launched on a slipway after casting, floated, towed to the pumphouse site and set in position by crane or crane barge.

When using the construction method referred to above which involves transport by barge, it is usual to test each unit at the casting yard for watertightness by filling the unit with water and leaving it filled for at least 24 hours. Although this method of testing does not fairly reflect normal water pressures during pumphouse operation, it is far simpler and less expensive than immersing the unit in water. Whichever method of testing is adopted, it is important that, during the design stage, the test loading condition is also checked, with the reinforcement designed and detailed accordingly. Water or sand is usually used as ballast during the placing of the precast pumphouse units to guard against buoyancy. Such ballast should not be removed until a careful design check is made on the buoyancy of the structure.

9.2.4 Ties and Waterstops

Ties used to secure and align formwork should not pass completely through any liquid-retaining part of the structure, unless effective precautions can be taken to ensure water tightness after their removal. The ends of any embedded ties should have cover equal to that required for the reinforcement. The gap left from the end of the tie to the face of the concrete should be effectively sealed. Although it has been common practice to provide central waterstops and keys at construction joints between the precast units and in-situ concrete sections, sections, waterstops are not usually required for construction joints with complete continuity in water-retaining structure. Central waterstops can be difficult to fix and held in position during concreting, and problems can be experienced when placing and compacting concrete around the waterstop. Whether or not a central waterstop is used, extreme care should be taken during surface preparation for construction joints in pumphouse unit walls.

9.2.5 Screens, Guides and Fittings

Pumphouse intake screen guides may be stainless steel or cast iron sections bolted onto the outside of the concrete intake blocks, or formed directly as a recess in the concrete intake. For the former case, the guides should be protected from damage by vessel impact using securely fixed fenders. For the latter case, the concrete nib between the recess and the outer face should be detailed with care, with stainless steel sections being used as necessary to protect and line the recess.

Internal and external steel fittings and fixtures, such as ladders, gratings, guide covers and runway beams, should be stainless, galvanized or painted with coal tar epoxy, as agreed with the users. To protect the internal fittings and to guard against the entry of silt and other deposits, a temporary stopper should be provided to block the intake pipe.

9.3 Slipways and Ramps

9.3.1 Location and Basic Dimensions

A slipway is a structure, consisting of a rail track, cradle and haulage device, used in ship building and ship repair work for the movement of vessels to and from the sea. The cradle is used to support the vessel and runs along the rail track, usually of standard flat-bottomed rails in two, three or four parallel lengths. Wire ropes are usually used to haul the vessel by means of a winch. Useful information on slipways is given by Grove & Little (1951).

Slipways should be located, where possible, at sites well protected from wave action. The slipway dimensions will depend on the size of the largest vessel to be slipped; in general the length of track above high water should exceed the vessel length, and the lower end of the track should extend to a depth adequate to allow the cradle to clear the vessel at lowest tide. The overall slipway width should be at least one and a half times the width of the largest vessel, and the gradient of the track within the range of 1 in 10 to 1 in 25, with about 1 in 15 being normal.

9.3.2 Slipway Design

To a large extent, slipway design will depend on the method of construction. Construction in the dry within a cofferdam may be more expensive in terms of initial cost than construction underwater, but will enable better quality of construction and tighter tolerances, resulting in a significant reduction in likely long term maintenance costs. With piled foundations, differential settlement will be controlled. With rubble mound foundations, it is essential that pre-loading is carried out to limit future differential settlement. Track support beams should be connected by cross-ties to maintain track gauge. Rail track fixing details should allow for possible relevelling and realignment during the design life of the structure, and also possible replacement of the upper lengths due to corrosion. Setting tolerances for line and level will depend on the cradle design, but will normally be significantly tighter than for general marine works. A tolerance of ± 10 mm for line and level is considered typical, but is often difficult to achieve for underwater work.

For the design of the rail track support beams, the main problem relates to the assessment of the load distribution as the vessel ceases to be waterborne and becomes carried on the loading cradle. At the start of slipping, with the cradle at the bottom of the slipway, the vessel is warped into position until bearing is obtained on the first section of the cradle. As slipping commences, by hauling up the cradle, gradually more and more weight is taken by the first section, and this load reaches the maximum just as the second section begins to take a share of the weight. Thereafter, all sections progressively take some load until the vessel is clear of the water and bearing uniformly over the whole cradle length. The exact value of the maximum load bearing on the first section, or 'sue' load, depends on the draft and outline of the vessel concerned, but as a guide can be taken to be about one third of the vessel weight. Since the sue load is only effective over a relatively short length, it is unnecessary to design the full slipway length for this load. The lowest length need only be designed to carry the weight of the cradle plus vessel uniformly distributed. The intermediate length should be designed for the full sue load or a proportion of the full sue load increasing from the lower end to the full sue load at the upper end as appropriate.

Care should be taken in estimating the cross distribution of load. With a cradle carried on two rails only, it is safe to regard the load as being equally divided between them, but where three or four rails are involved, such an assumption is not recommended due to possible rail settlement causing the cradle to carry loads unevenly. It is recommended that each rail should be designed for at least one half of the load.

9.3.3 Ramp Design

In comparison with a slipway, a ramp is a relatively simple structure. It consists essentially of a concrete slab sloping from about lowest tide level to above high tide level, for the movement of vehicles, usually from vessels to the shore. Design criteria should be agreed with the client. Design axle loads are typically 50 kN to 100 kN with a maximum of about 120 kN, with a normal ramp width of about 8 m and a slope of about 1 in 12.

A simple rubble foundation, at least 3 m thick, is usually satisfactory for a ramp, as settlement problems are not usually significant. The section within the lower tidal range is usually constructed using precast concrete blocks for ease of construction. The upper section is usually a normal in-situ concrete slab, typically 0.3 m thick, either reinforced for crack control or unreinforced with joints at 4 m to 5 m centres. Care should be taken to ensure that the rubble foundation at the lower end and sides is trimmed, and checked by a diver, to ensure no projection of rubble above the slab line which might cause damage to vessel approaching the ramp.

9.4 Outfalls and Intakes

Outfalls should be located well clear of pumphouses, intakes and landing steps, and where possible, should not be located immediately adjacent to suspended deck structures because of possible future dredging access problems during desilting. The determination of the invert levels of stormwater outfalls should take into account possible problems with adjacent vessels, hydraulic requirements and visual impact. Advice from Drainage Services Department should be sought.

Outfalls through seawalls are usually made of precast concrete units. For large box culverts, it may be necessary to form two units with a horizontal joint at about mid-wall height to reduce unit weights to a reasonable level. Wherever possible, lifting hooks for precast concrete outfall units should be formed in recesses which can be filled with suitable grout or concrete after unit setting; in this way, lifting hooks need not be removed and are available for future use in demolition or modification. Seals between outfall units are usually not necessary but shear keys are often provided. Where outfalls are constructed in advance of drainage pipes or box culverts, they should be temporarily sealed by timber boards, brickwork, concrete or steel plates as appropriate; the loads on the temporary seals due to waves, water pressure and soil pressure should be assessed.

Intakes are usually formed in seawalls to provide seawater for pumping stations, and are usually constructed concurrently with the seawalls. Size and location of the intake will be determined by the client. The invert level should be designed to ensure a continuous supply of water, unaffected by waves, tides, currents and water temperature variations. The usual method of construction is to use precast concrete units for the base slab and lower walls, and cast in-situ concrete for the upper walls and roof slab. Joints between precast concrete units are usually required by the client to be sealed.

9.5 Beacons

Beacons include lit and unlit beacons located offshore, on the foreshore or rock outcrops and on land, and navigation lights on marine structures. Lights can be mains- or battery-powered as appropriate to the location and as required by the Director of Marine. A beacon located offshore can either be a piled structure, similar to a dolphin in design, or a precast reinforced concrete gravity structure with enlarged base and rubble foundation, depending on the seabed conditions and water depth. Beacons located on the foreshore or rock outcrops can usually be simple precast or cast in-situ concrete structures doweled to underlying sound rock where possible. They will be topped with steel light posts for final light connection for lit beacons, or simple steel/concrete marker posts for unlit beacons. Beacons located on land and navigation lights on structures will generally only be subject to dead and wind loads, and simple mass concrete foundations for the light posts or marker posts will usually be adequate.

Ladders, fenders and mooring eyes as appropriate should be provided for beacons located offshore. Beacons located on the foreshore, rock outcrops and land should be provided with landing facilities, either incorporated into the beacon structure or built separately. Fitting and fixtures such as ladders, handrails and mooring eyes should be stainless steel. Steel light posts and marker posts should preferably be galvanized and painted after fabrication.

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TABLES

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Table 1 Comparison of Types of Breakwater

	Rubble Mound Breakwater	Vertical Breakwater	Composite Breakwater
Wave Reflection	Rubble mound absorbs part of the wave energy and reduces the amount of wave reflection.	Waves are nearly fully reflected from the vertical face.	Same as vertical breakwaters.
Water Depth	A large rubble mound will be required in deep water.	May not be practicable to design a vertical breakwater to carry the wave loading in very deep water.	May be suitable for very deep water where the quantity of rock required for a rubble mound is not available or when it is not practicable to design a vertical breakwater in deep water.
Settlement	Able to tolerate settlement.	A certain control on settlement is required.	A certain control on settlement is required.
Berthing	Berthing facilities should be provided separately.	The vertical face of the structure can allow vessel berthing.	Same as vertical breakwater.
Construction Materials	Large quantity of rock should be available particularly in deep water.	May be suitable if sufficient rock quantity is not available.	May be suitable in deep water if sufficient rock quantity is not available for large rubble mound.
Construction Methods	Specialized plant is not necessarily required.	Specialized plant is required for delivery and placing of caissons.	Same as vertical breakwaters.
Maintenance	Regular monitoring is required and repair is necessary for dislocated armour units.	Repair is necessary for damaged concrete. Monitoring of displacement of upright section is required after severe storms.	A combination of rubble mound breakwaters and vertical breakwaters.

Table 2 Comparison of Types of Foundation

Methods	Principles	Properties of Treated Soil	Advantages and Limitations
Dredging (Full or Partial Dredging)	Marine mud or soft alluvial deposit to be totally or partially removed and replaced by suitable fill material.	Marine mud or soft alluvial deposit is completely or partially replaced by fill of better engineering properties.	The method is relatively simple but problematic for soil disposal, in particular for contaminated soil. Less dredging for partial dredging but more detailed investigation and design, close monitoring as well as longer construction period may be required.
Deep Cement Mixing	Lime and cement introduced into native soil through rotating auger or special in-place mixer.	Solidified soil piles or walls with relatively high strength.	No dredging involved normally, no lateral displacement of native soil and no additional surcharge on underlying soil. Stringent quality control required. Cannot work if large obstruction is encountered. Study on possible environmental impact required.
Stone Columns	Holes jetted into soil and backfilled with densely compacted gravel.	Increased bearing capacity and reduced settlements.	Limited bearing capacity enhancement. Stringent quality control required. Not effective for sensitive clay. Lateral and upward displacement of soil. May not be applicable for soft soil.

Table 3 Typical Water Levels in Seawall Design

<table>
<tr><th>Loading Conditions</th><th>Wave Condition</th><th>Still Water Level in front of Seawall</th><th>Ground Water Level behind Seawall</th></tr>
<tr><td rowspan="3">Normal/ Accident</td><td rowspan="3">Wave condition at tropical cyclone signal no. 3 or within the first few hours of hoisting of tropical cyclone signal no. 8</td><td>Sea water level at return period of 2 years</td><td rowspan="2">Sea water level at return period of 2 years</td></tr>
<tr><td>Sea water level at return period of 2 years minus 0.7 m</td></tr>
<tr><td>Mean lower low water level</td><td>Mean lower low water level plus 0.7 m</td></tr>
<tr><td rowspan="7">Extreme</td><td rowspan="2">Wave condition at return period of 100 years</td><td>Sea water level at return period of 10 years</td><td rowspan="2">Sea water level at return period of 10 years</td></tr>
<tr><td>Sea water level at return period of 10 years minus 1.0 m</td></tr>
<tr><td rowspan="2">Wave condition at return period of 10 years</td><td>Sea water level at return period of 100 years</td><td rowspan="2">Sea water level at return period of 100 years</td></tr>
<tr><td>Sea water level at return period of 100 years minus 1.0 m</td></tr>
<tr><td rowspan="2">Wave condition at return period of 50 years</td><td>Sea water level at return period of 50 years</td><td rowspan="2">Sea water level at return period of 50 years</td></tr>
<tr><td>Sea water level at return period of 50 years minus 1.0 m</td></tr>
<tr><td>Wave condition at return period of 100 years</td><td>Mean lower low water level</td><td>Mean lower low water level plus 1.0 m</td></tr>
<tr><td colspan="4">Notes : 1. The water levels for temporary loading conditions should be determined by designers.
2. The critical still water level may be some intermediate levels of the quoted water levels in this table and should be assessed by designers for each case.
3. Designers should take into account the worst credible ground water conditions when determining the ground water levels behind the seawall. Hence, the design ground water level may be higher than the levels given in this table.</td></tr>
</table>

Table 4 Assessment of Possibility of Impulsive Breaking Wave Pressure

A-1	Is the angle between the wave direction and the line normal to the breakwater less than 20°?	No → Little Danger
	↓ Yes	
A-2	Is the rubble mound sufficiently small to be considered negligible?	No → Go to B-1
	↓ Yes	
A-3	Is the sea bottom slope steeper than 1/50?	No → Little Danger
	↓ Yes	
A-4	Is the steepness of the equivalent deepwater wave less than about 0.03?	No → Little Danger
	↓ Yes	
A-5	Is the breaking point of a progressive wave (in the absence of a structure) located only slightly in front of the breakwater?	No → Little Danger
	↓ Yes	
A-6	Is the crest elevation so high as not to allow much overtopping	No → Little Danger
	↓ Yes	
	Danger of Impulsive Pressure Exists	
B-1	(Continued from A-2) Is the combined sloping section and top berm of the rubble mound broad enough?	No → Little Danger
	↓ Yes	
B-2	Is the mound so high that the wave height becomes nearly equal to or greater than the water depth above the mound?	No → Little Danger
	↓ Yes	
B-3	Is the crest elevation so high as not to cause much overtopping?	No → Little Danger
	↓ Yes	
	Danger of Impulsive Pressure Exists	

Source : Reproduced from “Random Seas and Design of Maritime Structures” by permission of Prof. Y. Goda.

Table 5 Measured Wave Pressure on Top Slab of Wave Absorption Chamber

	Extreme Condition (10-year Return Period)	Extreme Condition (100-year Return Period)
<u>With Front Panel</u> :		
Maximum local uplift pressure	$1.9\rho gH_{max}$	$1.5\rho gH_{max}$
Average uplift pressure	$0.9\rho gH_{max}$	$0.8\rho gH_{max}$
Average overtopping pressure	No overtopping	$0.2\rho gH_{max}$
<u>Without Front Panel</u> :		
Maximum local uplift pressure	$3.5\rho gH_{max}$	$1.7\rho gH_{max}$
Average uplift pressure	$1.7\rho gH_{max}$	$1.3\rho gH_{max}$
Average overtopping pressure	No overtopping	$0.2\rho gH_{max}$

Notes :
1. The wave pressure on the top slab is for reference only, and is determined from physical model testing of seawall with a wave absorption chamber and removable perforated front wall (HKU, 1998). The dimension of the wave chamber (measured between the inner face of the front wall and the rear wall of the chamber) is equal to 3 m. The wave chamber is extended to a depth of –2.65 mPD.
2. The perforation ratio of the front wall with uniformly spaced circular perforation of 700 mm is about 26%.
3. The surface and soffit levels of the top slab in the test are respectively +4.35 mPD and +3.65 mPD.
4. The still water level is +3.05 mPD in 10-year return period and +3.45 mPD in 100-year return period.
5. The significant wave height is +0.81 m in 10-year return period and +1.31 m in 100-year return period.
6. Caution should be exercised if these figures are adopted, as the extreme water levels and wave heights vary in different areas, and chamber dimensions, perforation layout and soffit level of top slab may be different.
7. ρ is the density of seawater.

FIGURES

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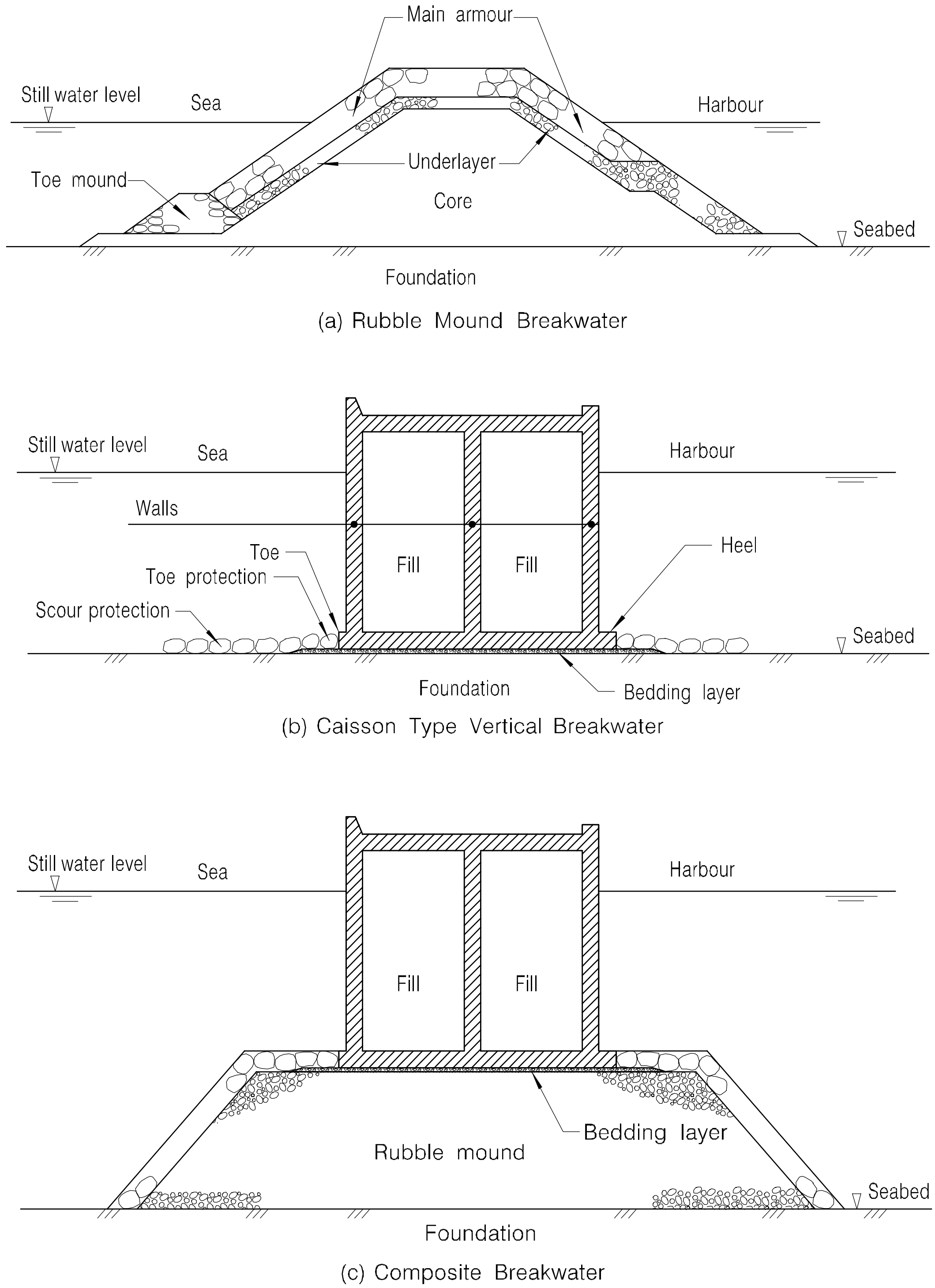


Note:

1. Caisson type structure can be replaced by blockwork structure.

Figure 1 – Type of Breakwaters

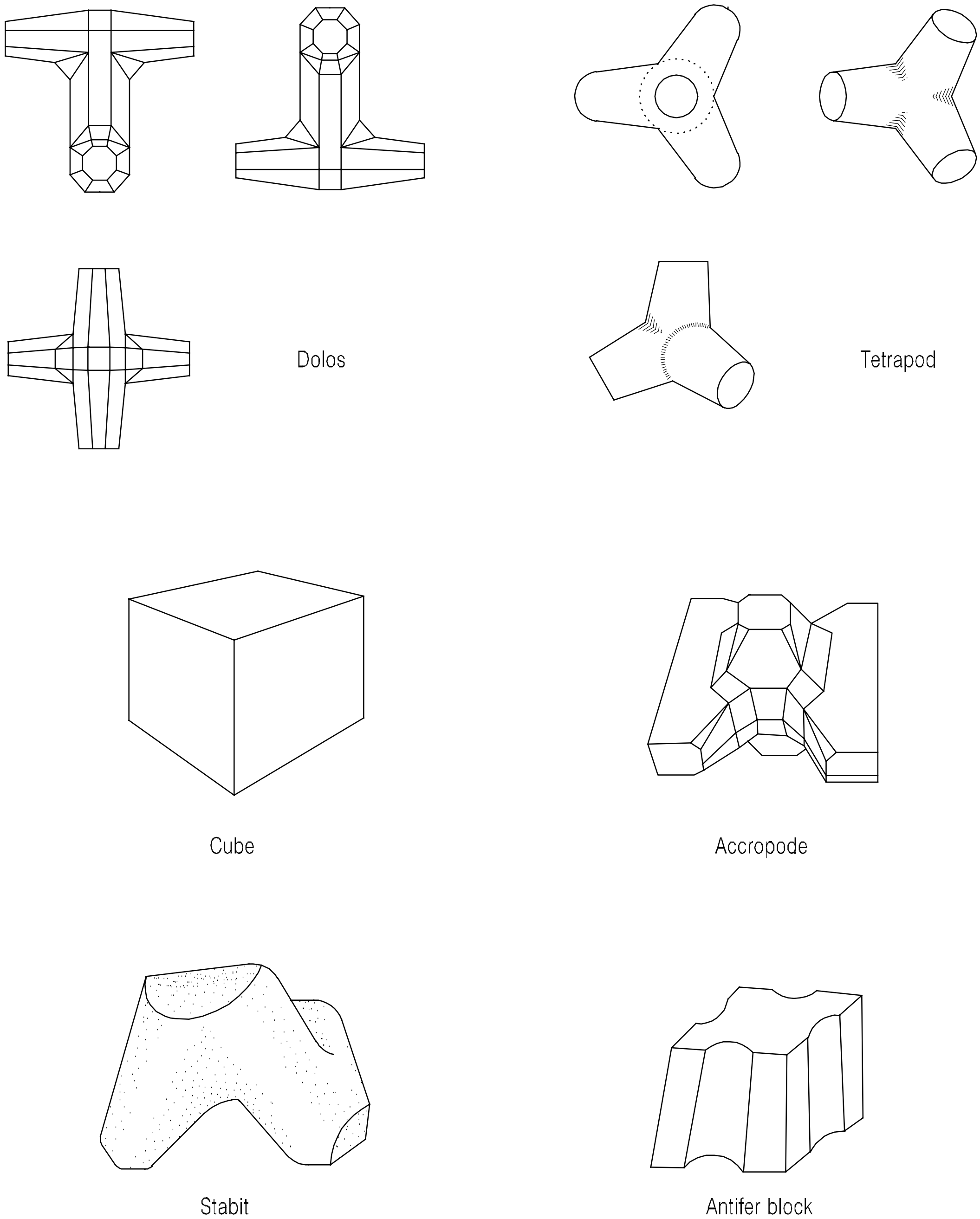
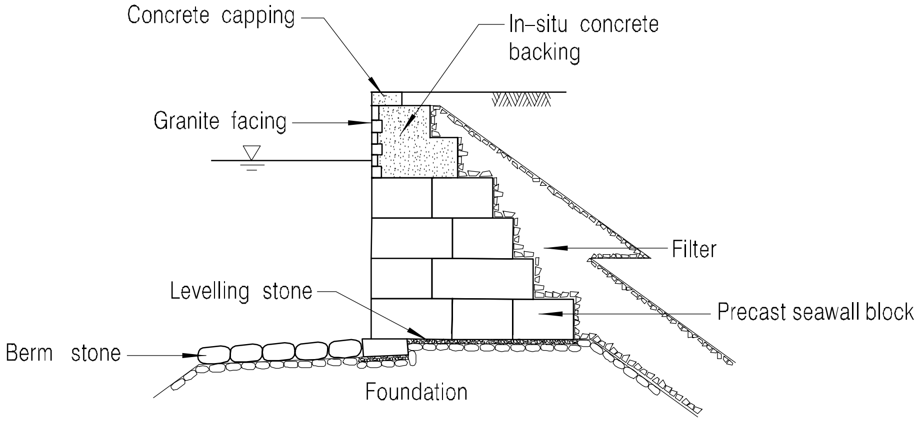
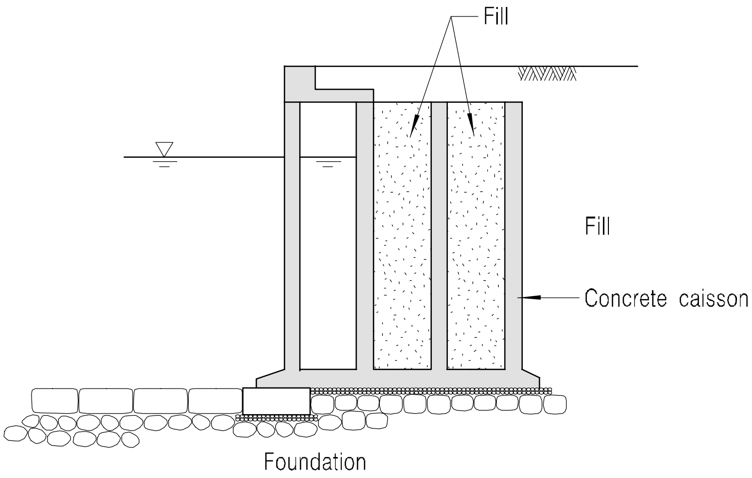


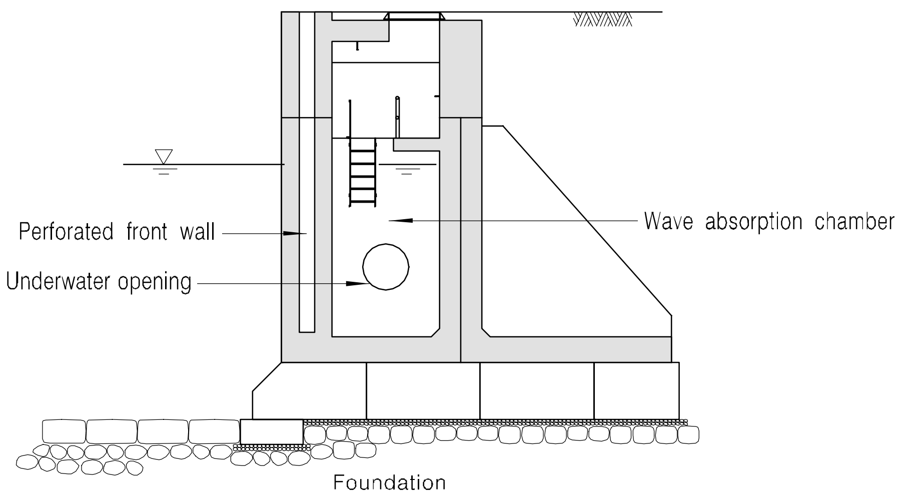
Figure 2 – Precast Concrete Armour Units



(a) Concrete Blockwork Seawall



(b) Caisson Seawall



(c) Wave Absorption Seawall

Figure 3 – Vertical Seawalls

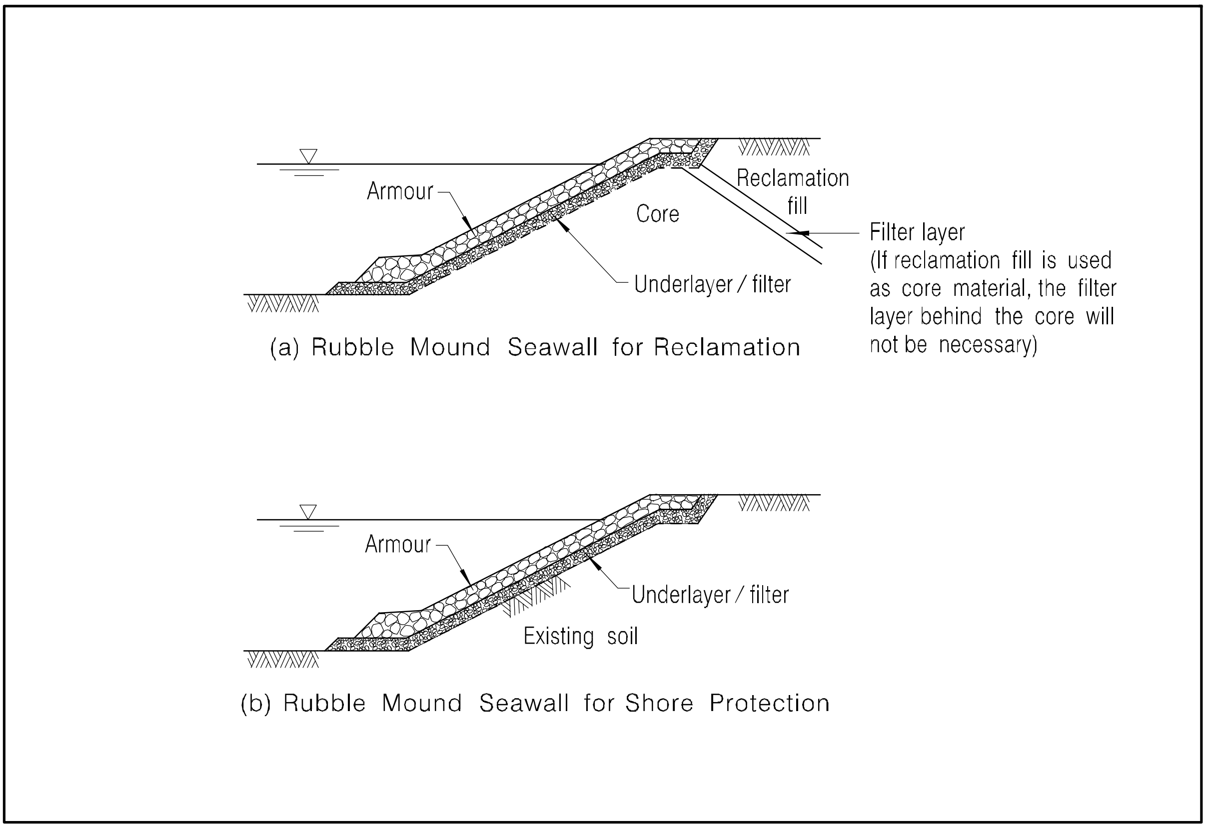


Figure 4 – Rubble Mound Seawalls

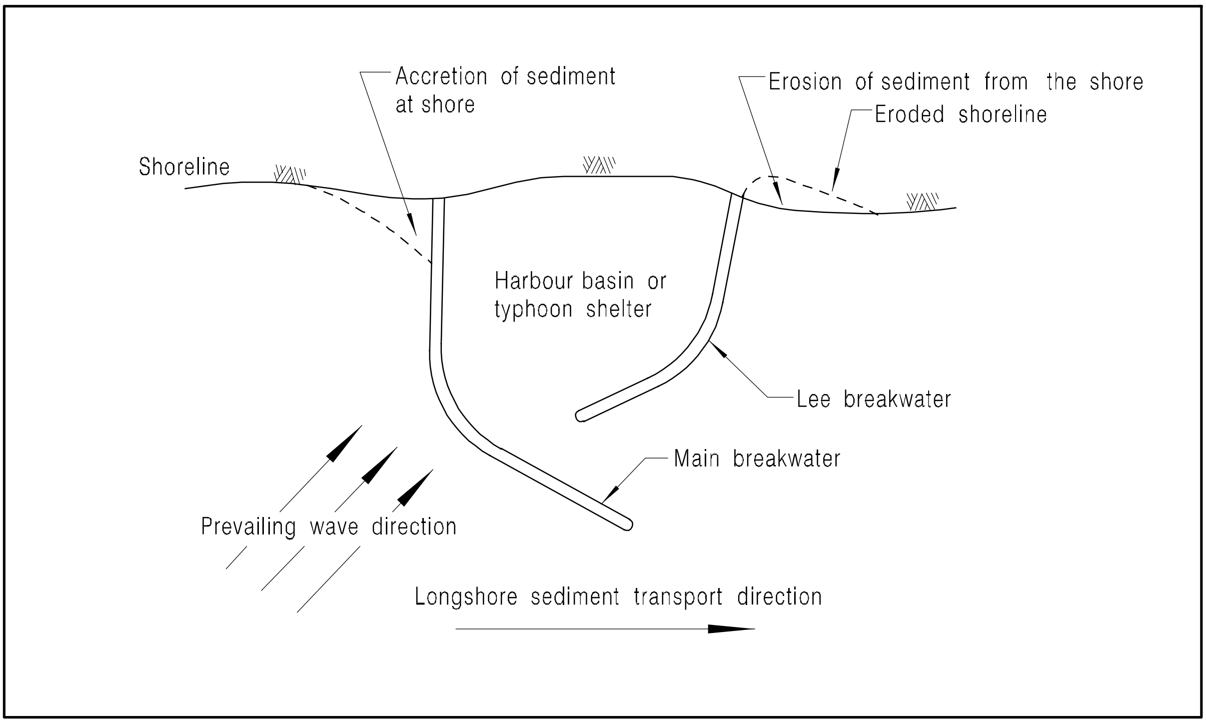
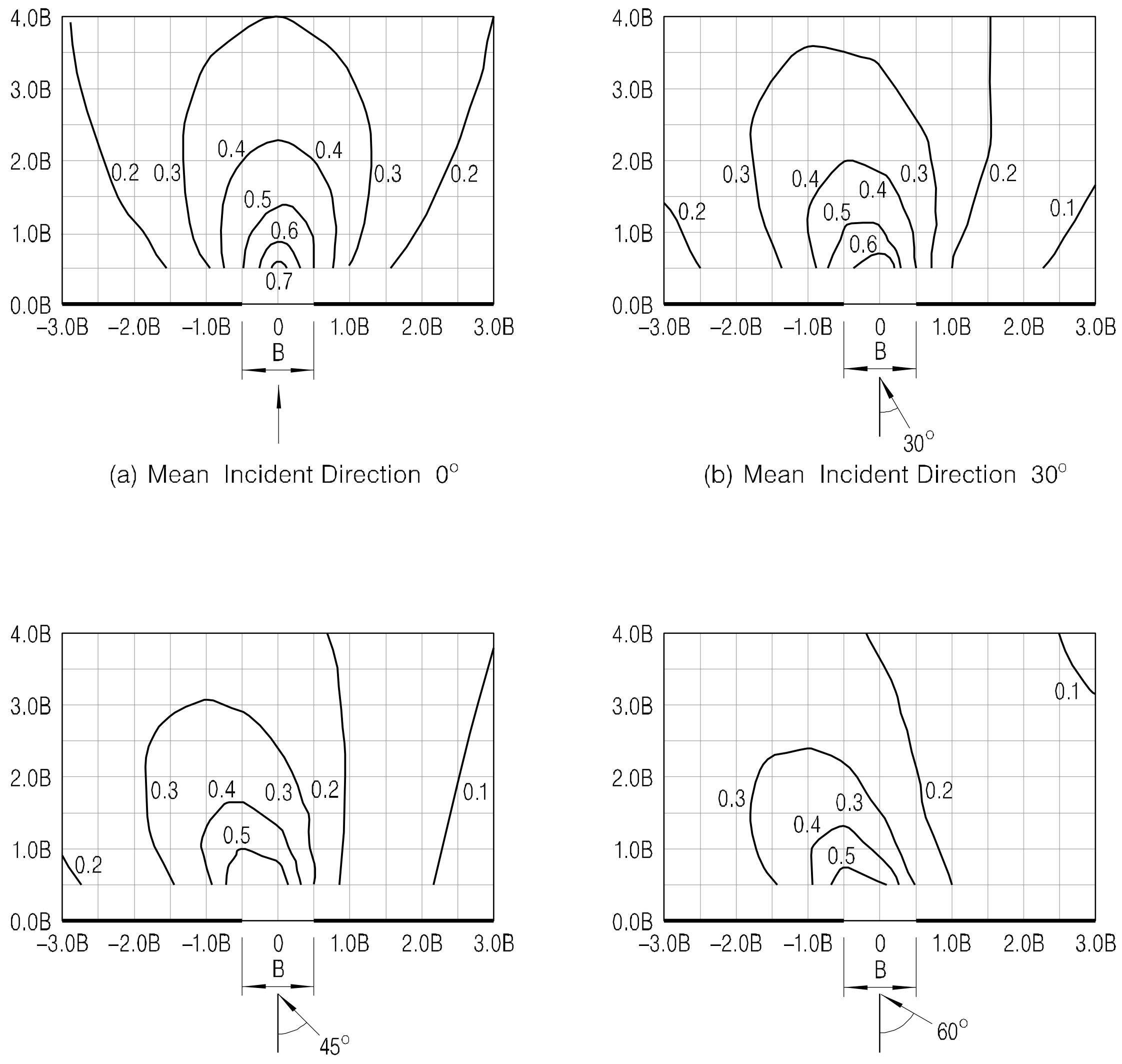


Figure 5 – Breakwater Layout



(a) Mean Incident Direction 0°

(b) Mean Incident Direction 30°

(c) Mean Incident Direction 45°

(d) Mean Incident Direction 60°

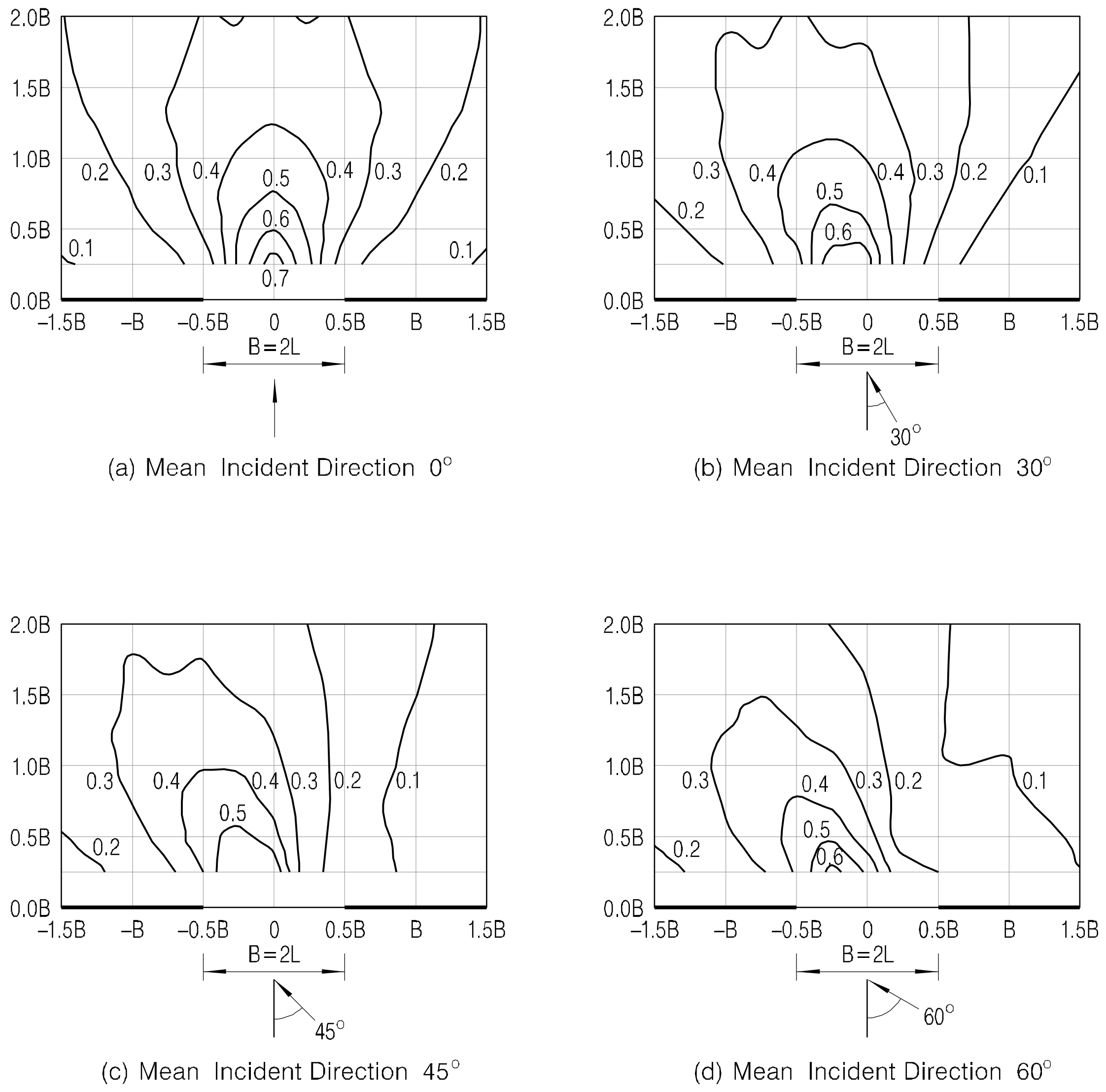
Notes:

1. Wave length corresponds to peak period of wave spectrum.

2. Length B = one wave length, Pierson–Moskowitz spectrum, $\cos^2$ directional spread.

Source: BS 6349 : Part 1 (2000) (Acknowledgement: HR Wallingford)

Figure 6 – Diffraction Coefficients for Breakwater Gap (Sheet 1 of 2)



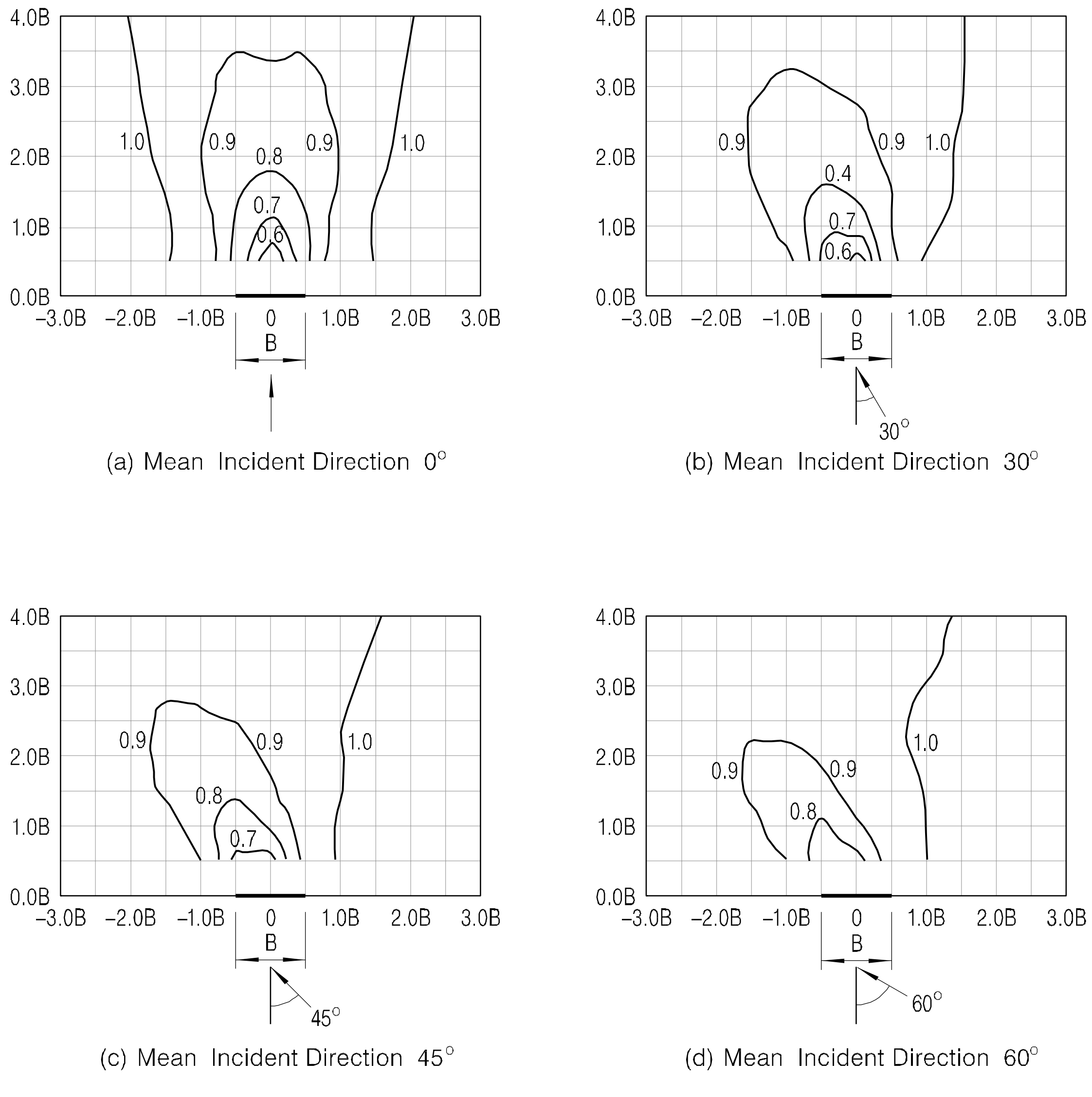
Notes:

1. Wave length corresponds to peak period of wave spectrum.

2. Length B=two wave lengths, Pierson–Moskowitz spectrum, $\cos^2$ directional spread.

Source: BS 6349 : Part 1 (2000) (Acknowledgement: HR Wallingford)

Figure 6 – Diffraction Coefficients for Breakwater Gap (Sheet 2 of 2)



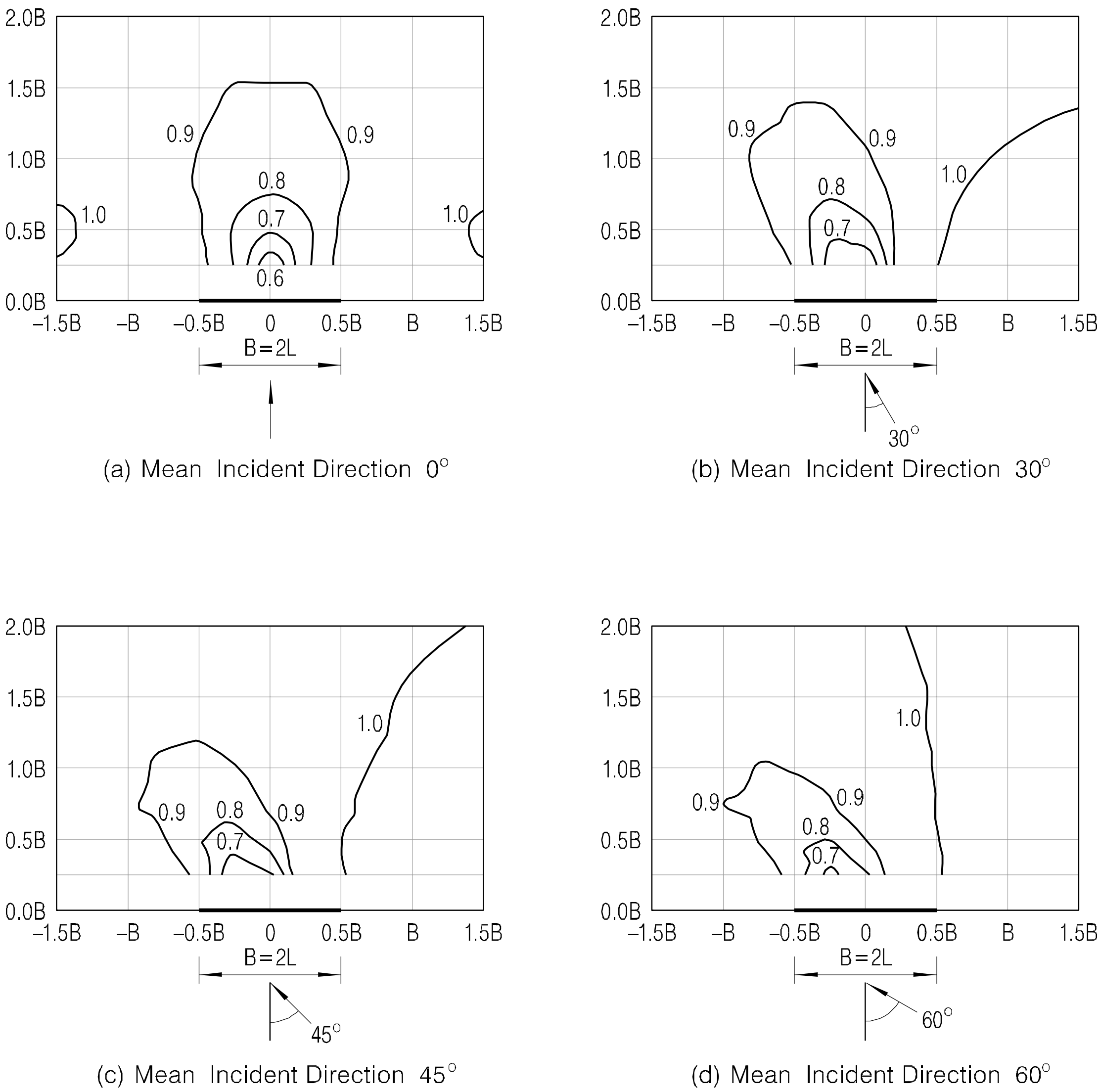
Notes:

1. Wave length corresponds to peak period of wave spectrum.

2. Length B = one wave length, Pierson–Moskowitz spectrum, $\cos^2$ directional spread.

Source: BS 6349 : Part 1 (2000) (Acknowledgement: HR Wallingford)

Figure 7 – Diffraction Coefficients for Island Breakwater (Sheet 1 of 2)

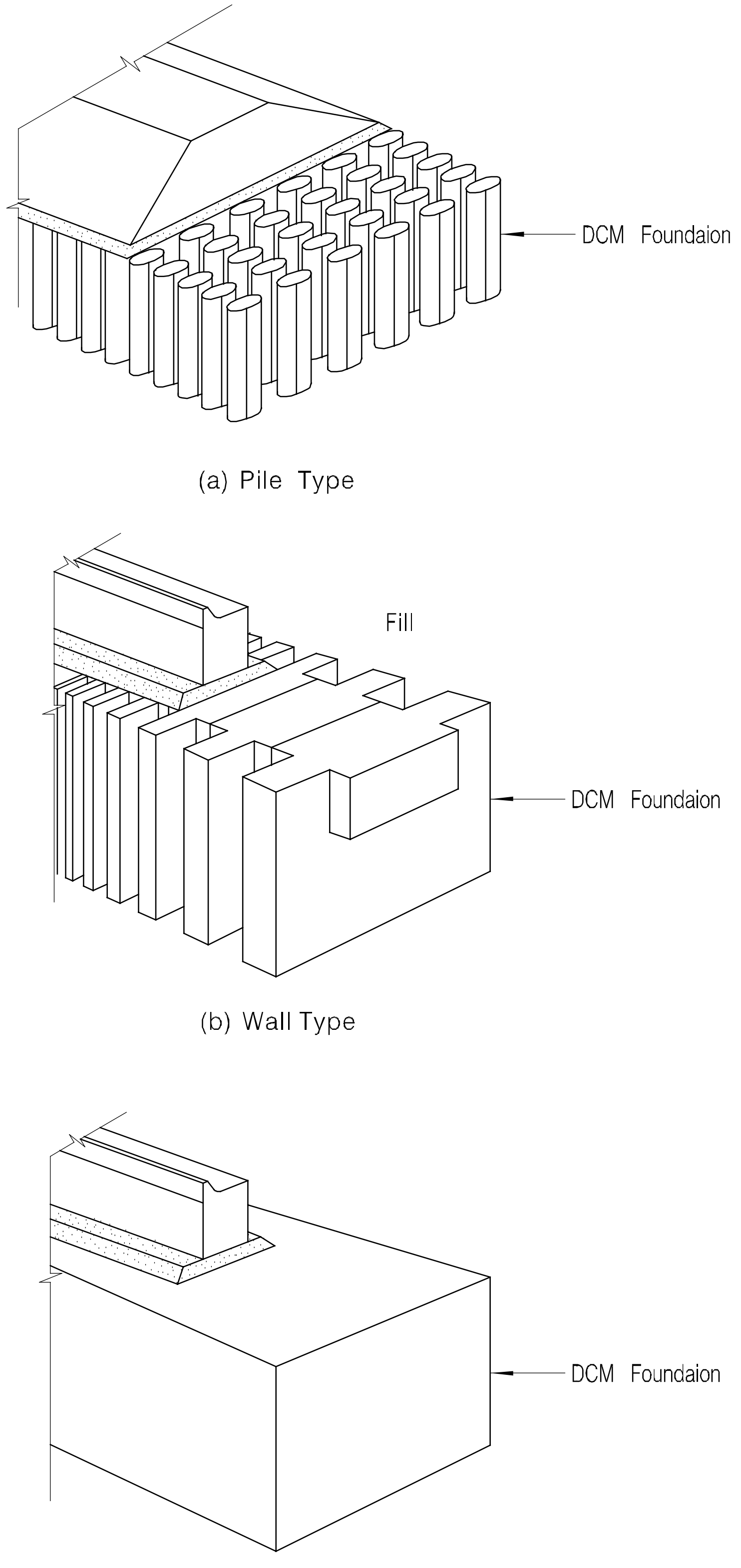


Notes:

1. Wave length corresponds to peak period of wave spectrum.
2. Length B=two wave lengths, Pierson–Moskowitz spectrum, $\cos^2$ directional spread.

Source: BS 6349 : Part 1 (2000) (Acknowledgement: HR Wallingford)

Figure 7 – Diffraction Coefficients for Island Breakwater (Sheet 2 of 2)

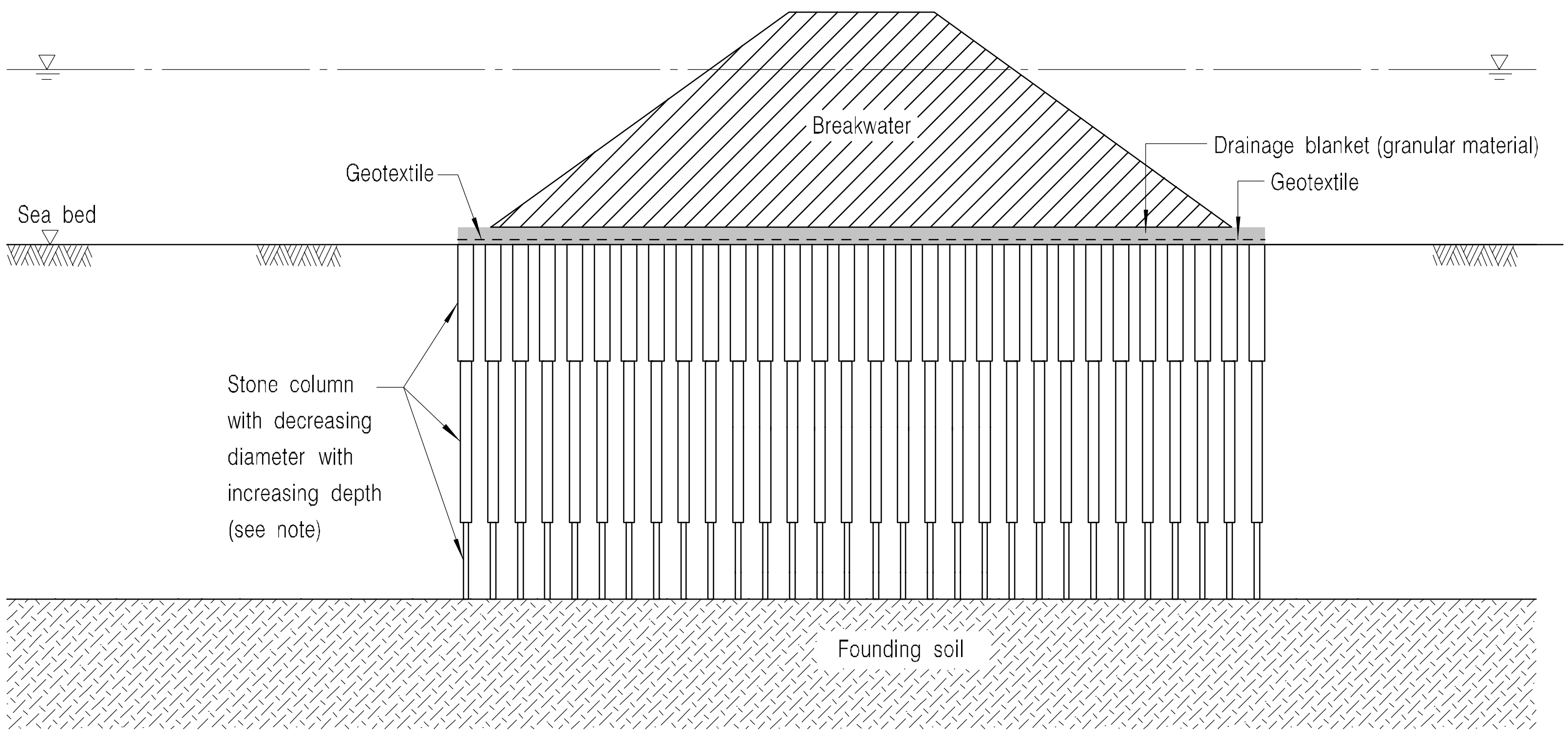


(a) Pile Type

(b) Wall Type

(c) Block Type

Figure 8 – Layout of Deep Cement Mixing Foundation



Notes :

1. Reduction of diameter with depth as shown, which is due to the increase in confining stress with depth, is for illustration purpose only. The actual amount of reduction will depend on the strength–depth relationship of respective soil stratum.

2. This figure illustrates the general principle of stone columns only; details of the structure such as toe protection are not shown.

Figure 9 – Layout of Stone–Column Foundation

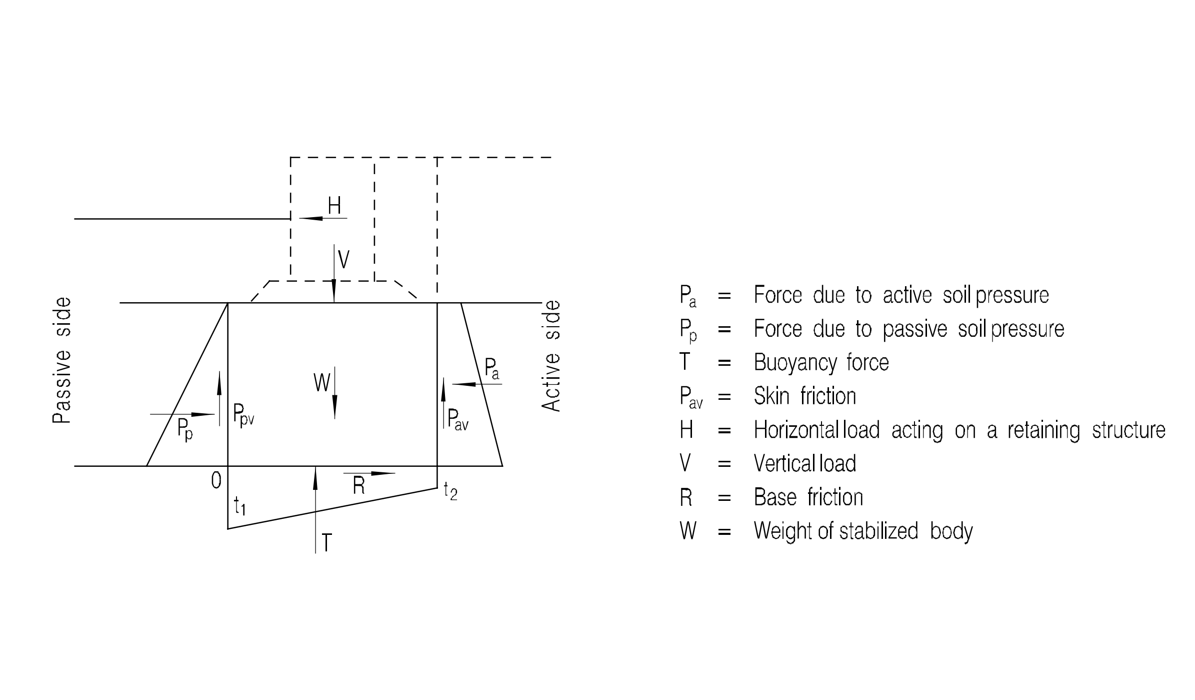


Figure 10 – External Forces on Soil Body Stabilized by Deep Cement Mixing

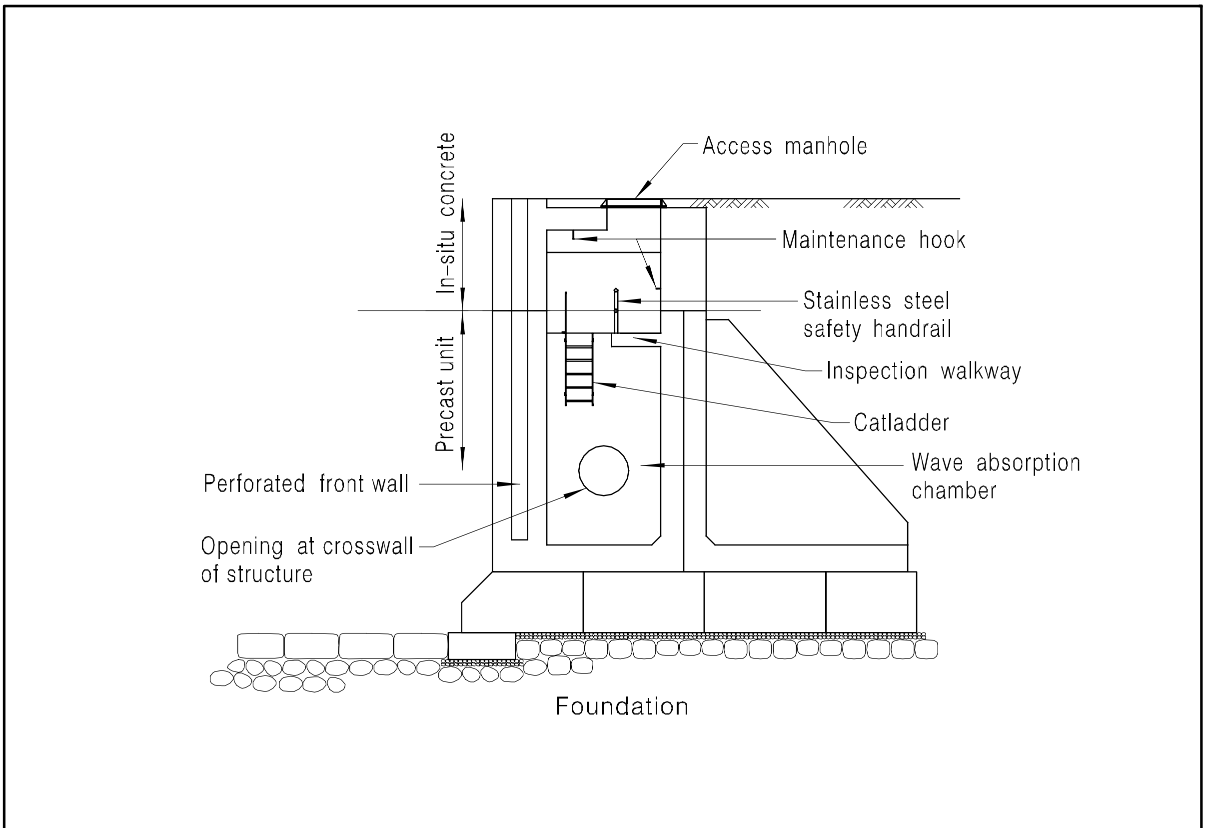
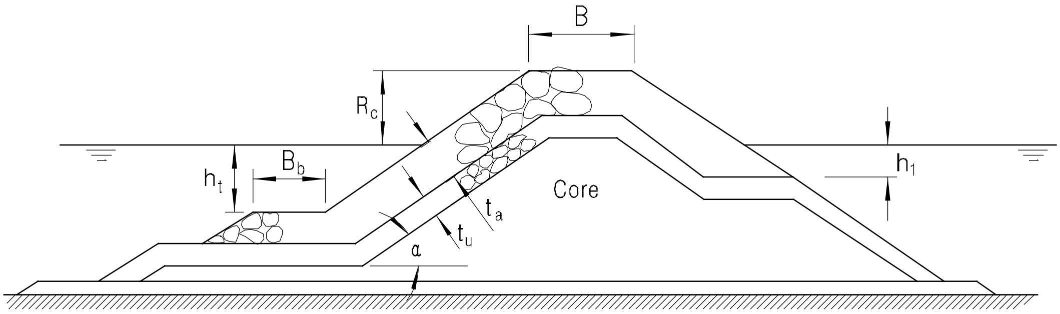
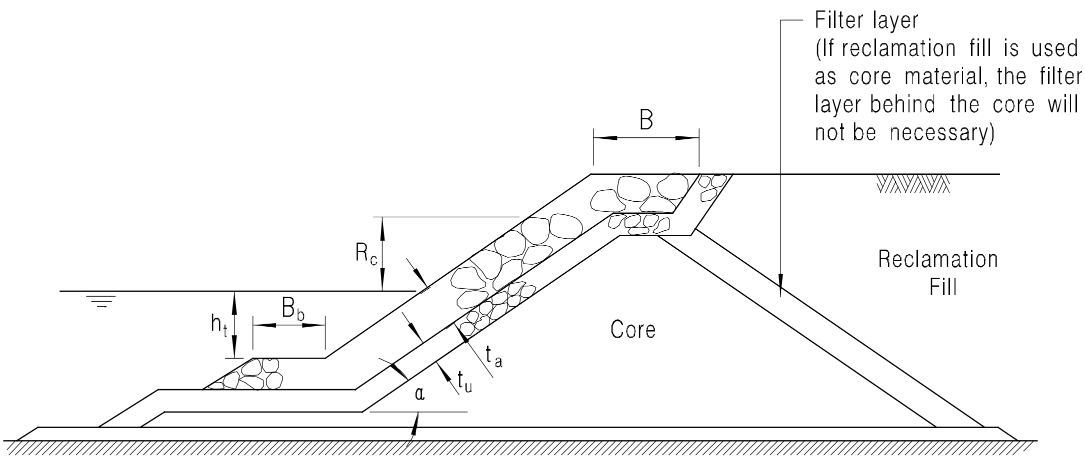


Figure 11 – General Layout of Wave Absorption Seawall



(a) Breakwater



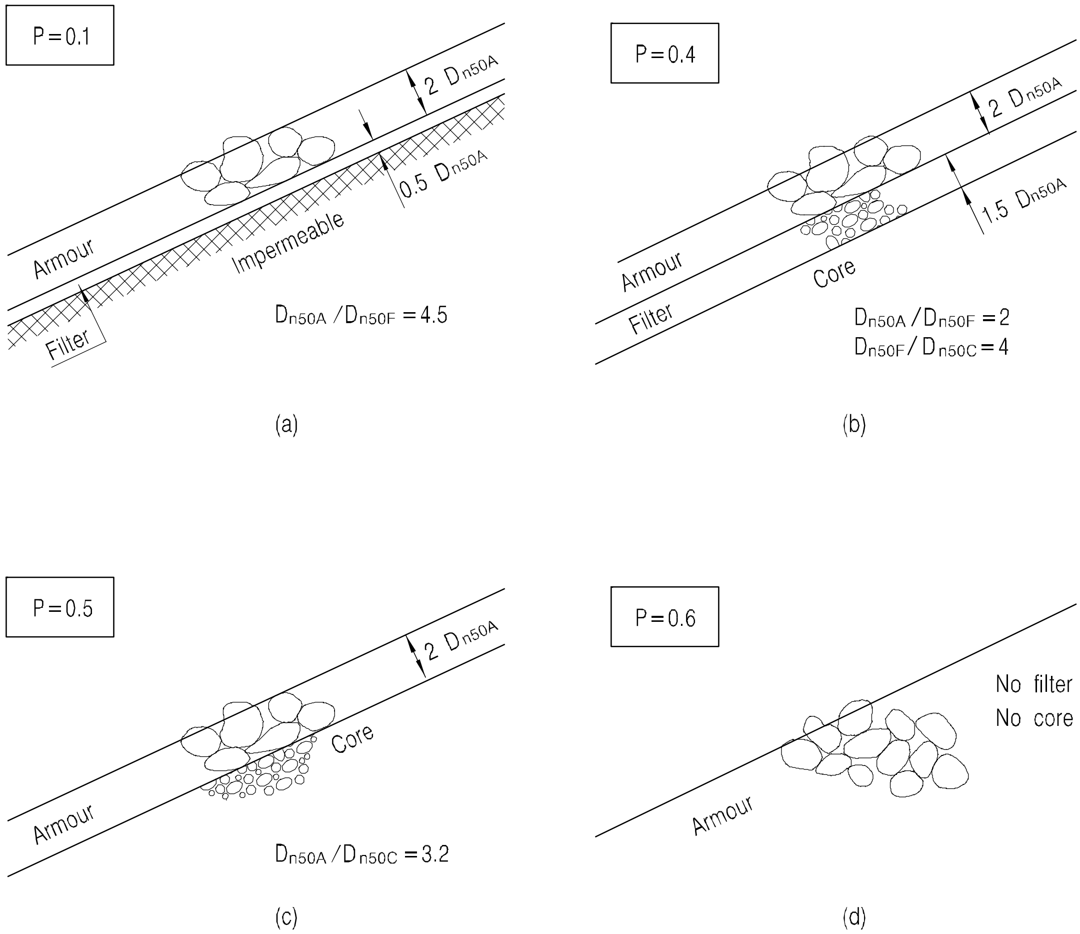
(b) Seawall

B_b = Crest width
B = Top apron width
R_c = Crest free board
t_a = Thickness of armour layer
t_u = Thickness of underlayer
α = Slope of structure
h_t & h_1 = Extent of armour layer on slope

Note:

1. h_t should extend down to the entire slope of the structure in shallow water where waves break.

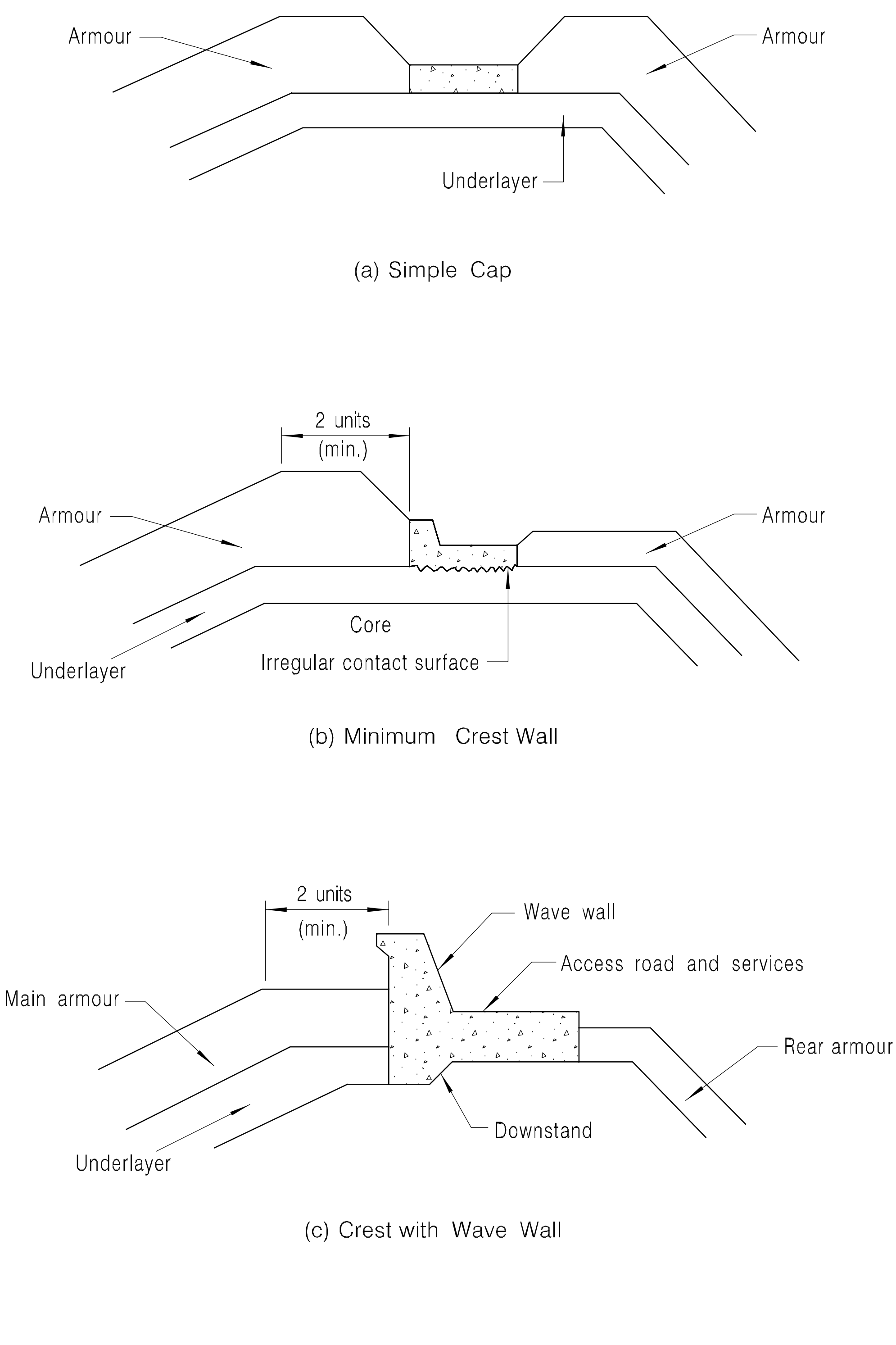
Figure 12 – Definition Sketch for Rubble Mound Breakwaters and Seawalls



P = Notional permeability factor
D_{n50A} = Nominal diameter armor
D_{n50F} = Nominal diameter filter
D_{n50C} = Nominal diameter core

Source: CIRIA (1991)

Figure 13 – Notional Permeability Factor



Source: BSI (1991)

Figure 14 – Typical Crest Structures for Rubble Mound Breakwaters

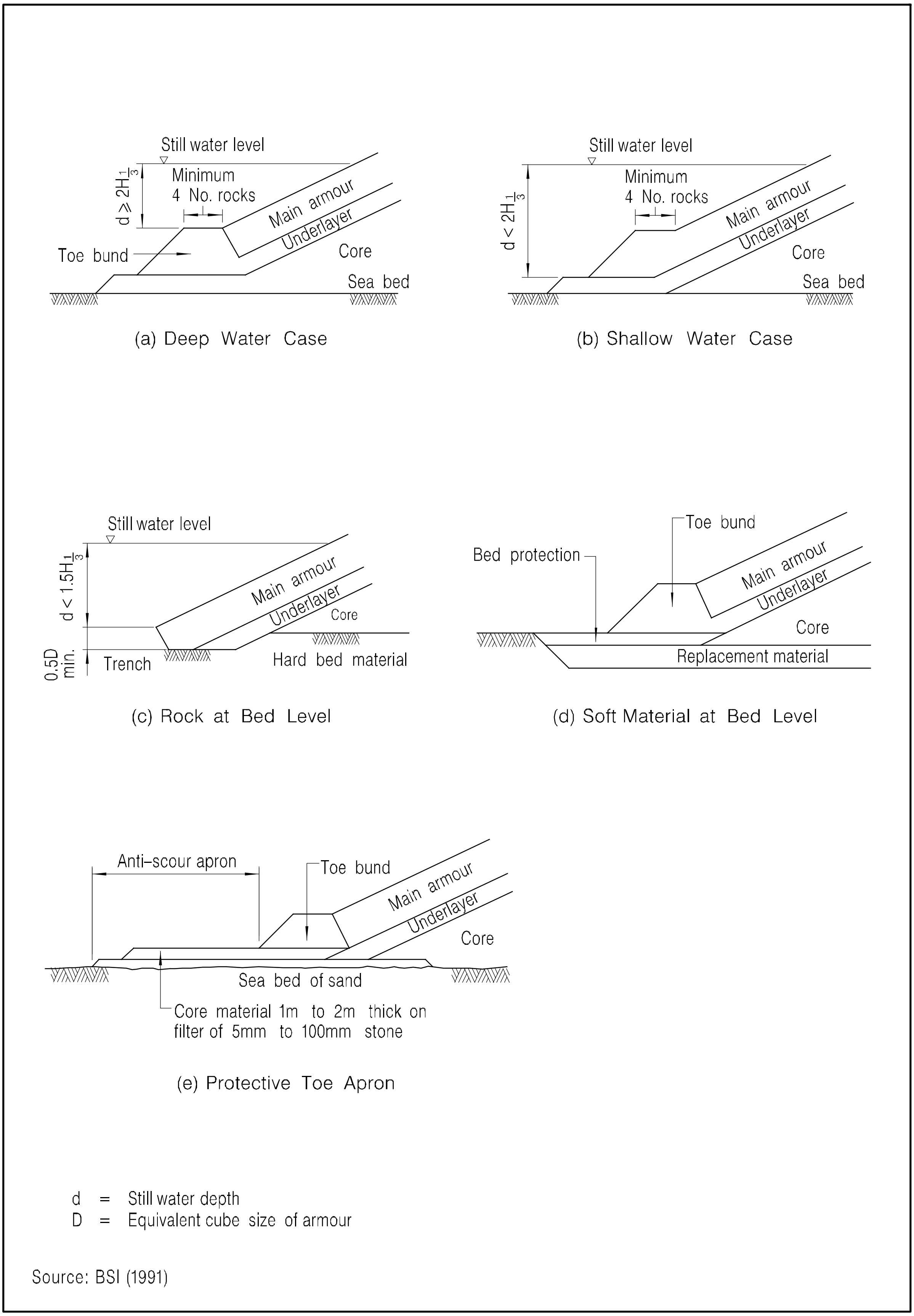


Figure 15 – Toe Details for Rubble Mound Structures

$$W = \frac{w_r H^3}{N^3 (s_r - 1)^3}$$

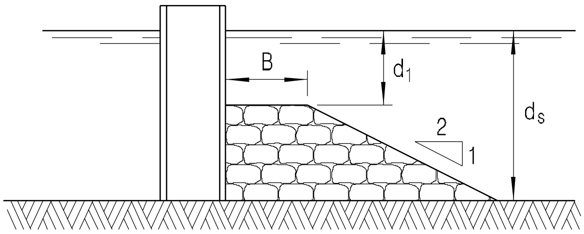
$B \geqslant 0.4d_s$

B = 2H or 4 times size of rock whichever is greater

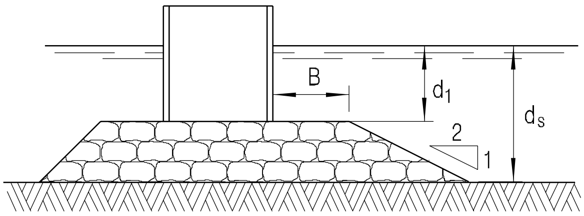
H = Design wave height

w_r = Unit weight of rock

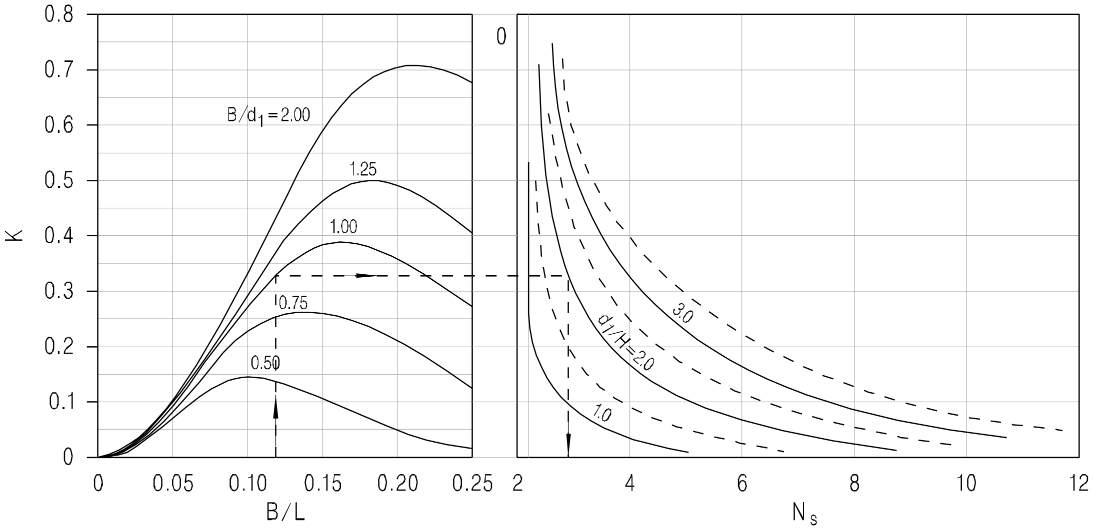
$$s_r = \frac{\rho_{rock}}{\rho_{water}}$$



(a) Rubble Toe Protection



(b) Rubble Foundation



where:

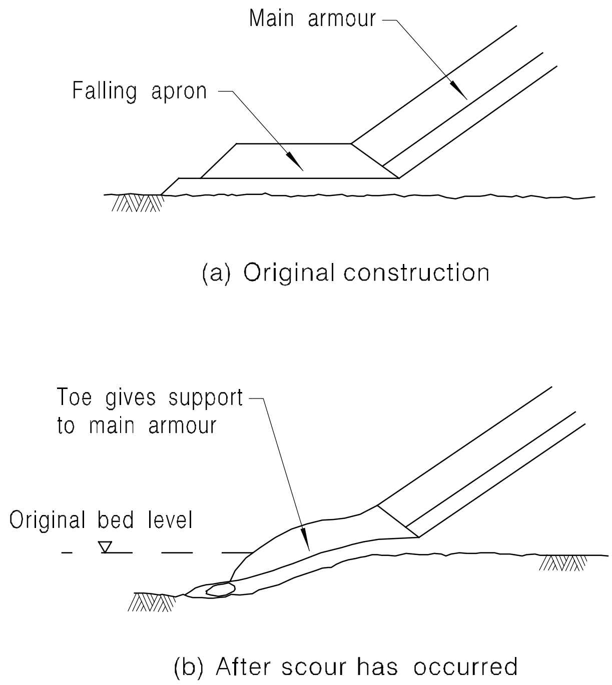
- K = parameter representing the combined effects of the relative water depth and the relative distance from the vertical wall on the maximum horizontal velocity at the bottom.
- H = design wave height associated with depth d_s
- L = wavelength associated with the depth d_1
- d_s = depth at structure
- B = toe apron width

Note:

1. For critical structures at open exposed sites where failure would be disastrous, and in the absence of reliable wave records, the design wave height should be the $H_{1/100}$ during an extreme event at the structure corrected for refraction and shoaling. If breaking might prevent the $H_{1/100}$ wave from reaching the structure, the maximum wave that could reach the structure should be taken for the design value of H. For less critical structures, design wave height could be taken between $H_{1/10}$ and $H_{1/100}$.

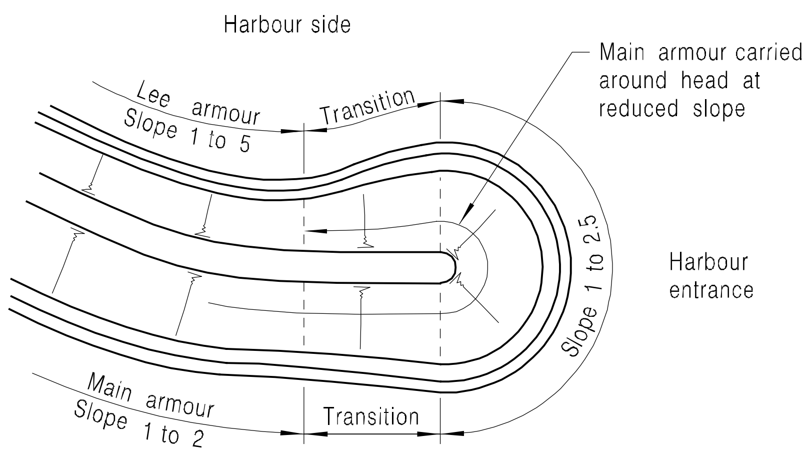
Source: CETN (1988) and Tanimoto et al (1982)

Figure 16 – Toe Protection



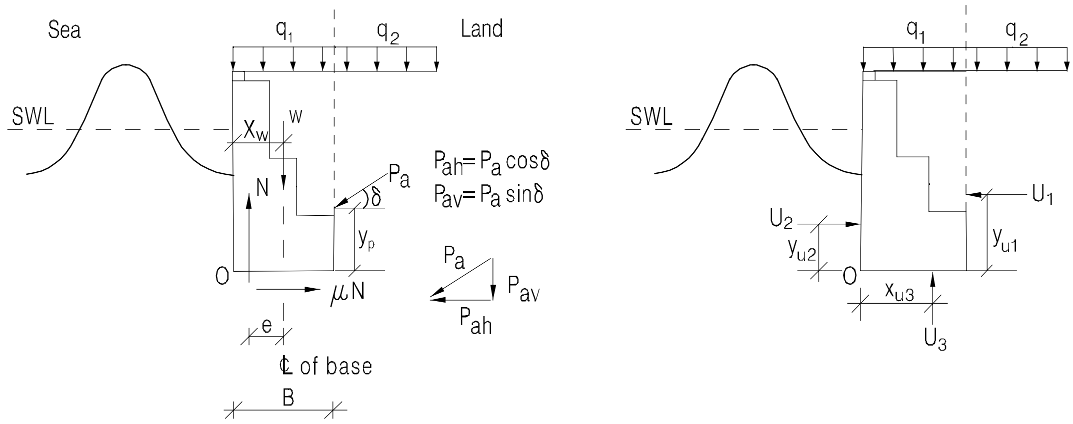
Source: BSI (1991)

Figure 17 – Falling Apron for Rubble Mound Structures



Source: BSI (1991)

Figure 18 – Typical Breakwater Roundhead Construction



(a) Forces on a Seawall
(Wave and water forces shown in (b))

(b) Wave and Water Forces on a Seawall
(Wave condition with wave trough
at seaward face)

SLIDING FAILURE

Activating force $F_a = P_{ah} + U_1 - U_2$

Resisting force $F_r = \mu N$ where $N = W + P_{av} - U_3$

FOS against sliding $= F_r / F_a$

OVERTURNING FAILURE

Overturning moment $M_o = P_{ah} y_p - P_{av} B + U_1 y_{u1} - U_2 y_{u2}$

Resisting moment $M_r = W x_w - U_3 x_{u3}$

eccentricity $e = \frac{B}{2} - \frac{M_r - M_o}{N}$

FOS against overturning $= M_r / M_o$

BEARING CAPACITY FAILURE

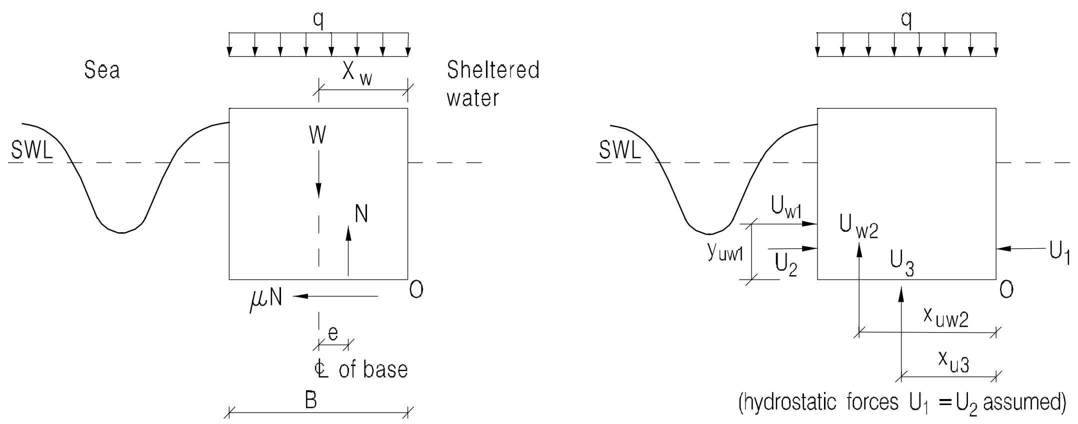
Effective normal load $Q_n = N + q_1 B$

FOS against bearing capacity failure $= Q_{ult} / Q_n$

Notes :

1. Other loads not indicated in the figure, if any, should also be taken into account.
2. The total weight W includes the weight of the structure and that of the soil behind the structure up to the virtual back.
3. The soil pressure P_a includes the active pressure behind the virtual back due to soil and the surcharge live load q_2.
4. U_1 refers to the hydrostatic force exerting on the virtual back; U_2 refers to the combined hydrostatic and wave pressure on the seaward face; U_3 refers to the uplift due to combined effect of buoyancy and wave. For details of U_1, U_2 and U_3, see Port Works Design Manual: Part 1 – Figure 17.
5. Live load q_1 to be considered only for checking against bearing capacity failure; otherwise $q_1 = 0$.
6. Coefficient of friction μ may be assumed to be 0.6 for both block /foundation and block / block interfaces.
7. Q_{ult} is the ultimate bearing capacity.

Figure 19 – Stability Calculation for Vertical Seawalls



(a) Forces on a Breakwater
(Wave and Water forces shown in (b))

(b) Wave and Water Forces on a breakwater
(Wave condition with crest at the seaward face and calm sea condition at the sheltered side)

SLIDING FAILURE

Activating force $F_a = U_{w1}$
Resisting force $F_r = \mu N$ where $N = W - U_3 - U_{w2}$
FOS against sliding $= F_r / F_a$

OVERTURNING FAILURE

Overturning moment $M_o = U_{w1} y_{uw1} + U_{w2} x_{uw2}$
Resisting moment $M_r = W x_w - U_3 x_{u3}$
eccentricity $e = \frac{B}{2} - \frac{M_r - M_o}{N}$
FOS against overturning $= M_r / M_o$

BEARING CAPACITY FAILURE

Effective normal load $Q_n = N + qB$
FOS against bearing capacity failure $= Q_{ult} / Q_n$

Notes :

1. Other loads not indicated in the figure, if any, should also be taken into account.
2. The total weight W includes the weight of the structure.
3. U_1 and U_2 refer to the hydrostatic forces exerting on the seaward face and sheltered face respectively ($U_1 = U_2$ assumed); U_3 refers to the buoyancy force; U_{w1} and U_{w2} refer to the horizontal force and uplift due to the waves at the seaward side. For details of U_{w1} and U_{w2} , see Port Works Design Manual: Part 1 – Figures 15 and 16.
4. Live load q to be considered only for checking against bearing capacity failure; otherwise q = 0.
5. Coefficient of friction μ may be assumed to be 0.6 for both block / foundation and block / block interfaces.
6. Q_{ult} is the ultimate bearing capacity.

Figure 20 – Stability Calculation for Vertical Breakwaters

APPENDIX A

MARINE GROUND INVESTIGATION IN DIFFICULT GROUND AREAS

APPENDIX A MARINE GROUND INVESTIGATION IN DIFFICULT GROUND AREAS

1. General

Difficult ground conditions generally refer to the existence of unfavourable subsoil strata on site. The presence of such conditions, if not properly handled, may lead to both problems at the construction stage and during the future use of seawalls, breakwaters and reclamation.

This Appendix provides suggested guidelines for marine ground investigation in areas with difficult ground conditions or likely to possess difficult ground conditions for seawalls, breakwaters and reclamation, based on the findings of the "Study on Coastal Subsoil Geotechnical Conditions" (CED, 2003).

The study identifies the following categories of difficult ground conditions in the Territory :

- Ground conditions that are difficult – These are difficult ground conditions with very thick marine deposit and/or extensive/thick interbedded soft alluvium below –35 mPD.
- Ground conditions that are likely to be difficult – These are ground conditions where marine deposit and/or soft alluvium are shown to exist in some borehole logs at about –35 mPD or below, or where the soil strata are variable but the available ground investigation information is not sufficient to lead to a definite conclusion.

2. Points of Exploration

2.1 Seawalls and Breakwaters

The spacing of the points of exploration, which may include a combination of boreholes and points of in-situ tests, may be taken as 75 m to 100 m if the structures are located in areas with thick, uniform marine or alluvial deposit layers. As an example of investigation arrangement, the points of exploration may include boreholes at 200 m spacing with cone penetration tests undertaken approximately halfway between boreholes. Additional cone penetration tests, about 5 to 10 % of the total number of boreholes, should be carried out adjacent to boreholes for calibrating the results of the cone penetration tests. Alternatively, cone penetration tests undertaken halfway between the boreholes may be replaced by boreholes as appropriate to the site conditions.

If interbedded soft deposits are expected, the spacing of the points of exploration may be further reduced to 50 m or less in order to identify the locations and extents of the soft material. The investigation may include boreholes at 100 m spacing with cone penetration tests undertaken approximately halfway between boreholes. Similarly, additional cone penetration tests, about 5 to 10 % of the total number of boreholes, should be carried out adjacent to boreholes for calibrating the results of the cone penetration tests. Cone penetration tests undertaken halfway between the boreholes may be replaced by boreholes as appropriate to the site conditions.

At locations where highly variable soft deposits exist and where the soil strength is critical to the stability of structures, the double-hole sampling approach may be considered. A borehole is first sunk to obtain continuous profile of the soil strata for inspection and a second borehole adjacent to the first borehole is then sunk to undertake vane-shear tests at close intervals to ascertain the type, nature and strength of the soil. Attention should be paid to locate the second borehole at a sufficient distance away from the first borehole to avoid testing the disturbed ground caused by the drilling of the first borehole. Additional boreholes should be sunk if the collected information is not sufficient to ascertain the ground conditions.

2.2 Reclamation

The spacing of the points of exploration, which may include a combination of boreholes and points of in-situ tests, may be taken as 100 m, if interbedded soft deposits are expected in the subsoil profiles. As an example of investigation arrangement, the points of exploration may include boreholes at 200 m spacing with cone penetration tests undertaken approximately halfway between boreholes. Additional cone penetration tests, about 5 to 10 % of the total number of boreholes, should be carried out adjacent to boreholes for calibrating the results of the cone penetration tests. Alternatively, cone penetration tests undertaken halfway between the boreholes may be replaced by boreholes to suit the site conditions. Additional boreholes should be sunk if the collected information is not sufficient to ascertain the ground conditions.

3. Depth of penetration

The investigation should reach a depth of 5 m into the underlying Grade V weathered rock to determine the thickness of the marine and alluvial deposits, in order to allow an estimate of the stability and settlement of the structures and reclamation. In addition, 10% of the boreholes should be penetrated 5 m into Grade III rock to ascertain the location of firm bearing stratum.

4. In-situ Field Tests and Soil Sampling for Laboratory Testing

For in-situ testing and sampling, the following schedule should be applied :

(a) Vane shear tests and piston samples should be undertaken alternatively at 2 m intervals for clayey/silty soil. If double-hole sampling is carried out, vane shear tests should be continuously undertaken at 1 m intervals in the second borehole.

(b) Standard penetration tests (with liner samples) and U100 or Mazier samples should be undertaken at 2 m intervals for soils of sandy nature.

(c) For cone penetration tests, the measurement can be made at depth intervals of 0.2 m. The types of reading to be taken include the tip resistance and, if available, sleeve friction and pore pressure. Classification charts based on tip resistance, sleeve friction and/or pore pressure are available for estimation of soil types. For more accurate assessment of the soil properties, the test results should be calibrated with the information of an adjacent borehole.

The designer should prepare a schedule of laboratory testing for determining the grading, moisture content, density, strength deformation and consolidation characteristics of the soil. The following aspects should be noted :

(a) The laboratory testing conditions should resemble the field conditions in which the works or structures will be constructed and operate at various stages. The initial state of the samples as well as the state of the soils in the construction and operation should be clearly specified, taking into account the depth, soil permeability and future stress conditions.

(b) Unconsolidated and consolidated undrained triaxial tests should be carried out for soil samples taken along the potential slip surface of marine structures. However, the results of unconsolidated undrained tests may not be very reliable due to possible disturbance during sampling. Hence, they should be used to supplement the in-situ strength obtained from the field tests. Consolidated undrained tests can simulate the long-term performance of the soil samples and their results can be used to assess the long-term stability of the structures. In view of the comparatively poor consolidation characteristics of clayey/silty soil, care should be exercised in adopting the consolidated undrained test results in the analysis of short-term stability. In-situ vane shear test results should be used for such analysis as far as possible.

(c) Oedometer tests should be carried out for soil left below the foundation of structures and reclamation. The number and interval of the samples to be tested should be determined according to the variability of the subsoil profiles, the layout of the foundation as well as the extent of dredging or soil treatment works.

5. Reference

CED (2003). Special Project Report No. SPR 1/2003 – Study on Coastal Subsoil Geotechnical Conditions. Civil Engineering Office, Civil Engineering Department, Hong Kong.

APPENDIX B

ASSESSMENT OF HYDRAULIC PERFORMANCE

CONTENTS

APPENDIX B ASSESSMENT OF HYDRAULIC PERFORMANCE

B.1 General

This appendix discusses some methods of assessing run-up, overtopping, reflection and transmission due to waves on a structure. These methods are empirical based on simplified configurations and should not be regarded as exhaustive. The results of calculations should only be treated as quick estimate of the order of magnitude of the hydraulic parameters. Further details of these methods can be found in Besley (1999), CIRIA (1991) and Goda (2000). Where complicated situations are encountered, or if more accurate results are required, physical model tests should be carried out to determine the hydraulic performance of the structure.

B.2 Wave Run-up

For simple armoured rubble slopes, Van der Meer (1988) has given prediction formulae for rock slopes with an impermeable core having permeability factor P = 0.1 and porous mounds of relatively high permeability given by P = 0.5 and 0.6. The prediction formulae are :

$$R_{ui} / H_{1/3} = a\xi_m \qquad \text{for } \xi_m < 1.5$$

$$R_{ui} / H_{1/3} = b\xi_m{}^c \qquad \text{for } \xi_m > 1.5$$

The run-up for permeable structures (P > 0.4) is limited to a maximum :

$$R_{ui} / H_{1/3} = d$$

where R_{ui} = Run-up at i % exceedance level (m).
$H_{1/3}$ = Significant wave height (m).
ξ_m = Surf similarity parameter based on mean wave period $= \tan\alpha / \sqrt{s_m}$.
α = Average slope angle (degree).
s_m = Offshore wave steepness based on mean wave period = $2\pi H_{1/3} / gT_m{}^2$.
T_m = Mean wave period (s).

Values of the coefficients a , b, c and d for exceedance levels of i equal to 1%, 2%, 5%, 10%

and significant run-up levels are given in Table B1.

When subject to oblique waves, the wave run-up behaviour will be different for short-crest waves and long-crested waves (CIRIA, 1991). For short-crested waves, the run-up is maximum for normal incidence and the reduction of run-up for large wave angles is not more than a factor of 0.8 compared with normal incidence. For long-crested waves, the increase in run-up is only present when the incident wave angle is about 10 to 30 degrees.

B.3 Wave Overtopping

B.3.1 Armoured Rubble Slope

Owen (1980) has derived the following formulae to estimate the mean overtopping discharge for rough impermeable and rough permeable structures :

$$R_* = R_c/(T_m (gH_{1/3})^{0.5}) \qquad (0.05 < R_* < 0.30)$$
$$Q_* = A \exp(-BR_*/r)$$
$$Q = Q_* T_m g H_{1/3}$$

where R_c = Freeboard between still water level and crest of structure (m).
$H_{1/3}$ = Significant wave height at the toe of the structure (m).
T_m = Mean wave period at the toe of the structure (s).
r = Roughness coefficient given in Table B2.
g = Acceleration due to gravity (m/s^2).
A,B = Empirical coefficients dependent on cross-section (see Table B3).
Q = Mean overtopping discharge rate per metre run of seawall (m^3/s/m).
Q_* = Dimensionless mean overtopping discharge.
R_* = Dimensionless freeboard.

For a permeable crest, a reduction factor C_r may be applied to the overtopping discharge as calculated above (Besley, 1999) :

$$C_r = 3.06 \exp(-1.5C_w/H_{1/3})$$

where C_w = Crest width of the structure (m).

If $C_w/H_{1/3}$ is less than 0.75, C_r may be assumed as 1.

If the incident waves are not normal to the structures, the overtopping rate may further be multiplied by a reduction factor O_r (Besley, 1999) :

$$O_r = 1 - 0.000152\,\beta^2$$

where β = Angle of wave attack to the normal, in degrees.

The formula is valid for $0^o < \beta \le 60^o$. For angles of approach greater than 60^o, it is suggested that the result for $\beta = 60^o$ be applied.

B.3.2 Vertical Structures

When the toe of a vertical structure is close to the seabed level, the overtopping rate may be estimated using the diagrams in Figures B1 and B2 (Goda, 2000). These diagrams are compiled by Goda from the results of a series of random wave tests with allowance of wave deformation in the surf zone. Equivalent deepwater wave steepness of 0.012, 0.017 and 0.036, and seabed slopes of 1/10 and 1/30 are covered.

Besley (1999) also suggests method for calculating the amount of wave overtopping discharge for vertical walls, which is given in the following paragraphs.

Reflecting waves predominate when $d_* > 0.3$, in which case the following equation applies :

$$d_* = (d/H_{1/3})(2\pi d/(gT_m^2))$$
$$Q^\# = 0.05 \exp(-2.78\, R_c/H_{1/3}) \qquad \text{(Valid for } 0.03 < R_c/H_{1/3} < 3.2\text{)}$$
$$Q = Q^\# (gH_{1/3}^3)^{0.5}$$

where
d_* = Dimensionless depth parameter.
d = Water depth at the toe of the structure (m).
$H_{1/3}$ = Significant wave height at the toe of the structure (m).
g = Acceleration due to gravity (m/s^2).
T_m = Mean wave period (s).
$Q^\#$ = Dimensionless discharge.
Q = Mean overtoping discharge rate per metre run of seawall (m^3/s/m).
R_c = Freeboard (height of crest of the wall above still water level) (m).

If the incident waves are at an angle to the normal of the seawall,

$$Q^{\#} = 0.05 \exp \{(-2.78/\gamma)(R_c/H_{1/3})\}$$

γ is the reduction factor for angle of incident waves and is given by :

$\gamma = 1 - 0.0062\beta$ for $0° < \beta \le 45°$

$\gamma = 0.72$ for $\beta > 45°$

where β = Incident wave angle relative to the normal, in degrees.

Impact waves predominate when $d_* \le 0.3$, in which case the following equation applies :

$Q_h = 0.000137 R_h^{-3.24}$ (Valid for $0.05 < R_h < 1.00$)

where Q_h = Dimensionless discharge = $\{Q/(gh^3)^{0.5}\}/d_*^2$

R_h = Dimensionless crest freeboard = $(R_c/H_{1/3})\ d_*$

No data is available to describe the effect of oblique wave incidence on the mean discharge when waves are in impacting mode.

B.4 Wave Reflection

There are various formulae for the coefficient of wave reflection of armoured slopes. It will be useful to compare the results of these formulae when assessing the coefficient of reflection of rubble mound structures.

For a rough permeable slope, the following formula was given by Seelig and Ahrens (CIRIA, 1991) to estimate the coefficient of reflection :

$$C_r = a\xi_p^{\ 2}/(b + \xi_p^{\ 2})$$

where ξ_p = Surf similarity parameter based on peak wave period.

C_r = Coefficient of reflection.

$a = 0.6$ and $b = 6.6$ for a conservative estimate of rough permeable slopes.

Postma (1989), taking into account Van der Meer (1988) data for rock slopes and Seelig and

Arhens formula, derived the following formula for C_r :

$$C_r = 0.14\xi_p^{\ 0.73} \qquad \text{with standard deviation of } C_r = 0.055$$

Postma also treated the slope angle and wave steepness separately and derived another relationship :

$$C_r = 0.071P^{-0.082}(\cot\alpha)^{-0.62}s_p^{\ -0.46} \qquad \text{with standard deviation of } C_r = 0.036$$

where P = Notional permeability factor.
α = Slope of structure face.
s_p = Offshore wave steepness based on peak wave period.

The results of random wave tests by Allsop and Channell (1989), analyzed to give values for the coefficients *a* and *b* in Seelig and Ahrens formula, but with ξ_m instead of ξ_p, are shown below. The slopes used armour rock in one or two layers with an impermeable slope covered by underlayer rock equivalent to notional permeability factor P equal to 0.1 :

Rock, 2-layer	$a = 0.64$	$b = 8.85$
Rock, 1-layer	$a = 0.64$	$b = 7.22$

The range of wave conditions for which the coefficients may be used is given by :

$$0.004 < s_m < 0.052 \qquad \text{and} \qquad 0.6 < H_{1/3}/(\Delta D_{n50}) < 1.9$$

where s_m = Offshore wave steepness based on mean wave period.
D_{n50} = Nominal rock diameter equivalent to that of a cube.
Δ = Relative mass density.
= (mass density of rock/mass density of seawater) – 1

Postma (1989) also reanalyzed the data of Allsop and Channell and modified his previous formula for coefficient of reflection as follows :

$$C_r = 0.125\xi_p^{\ 0.73} \qquad \text{with standard deviation of } C_r = 0.060$$

For structures with no-porous and steep faces, approximately 100% of the wave energy incident on the structure will be reflected.

B.5 Wave Transmission

Van der Meer (1990) re-analysed the hydraulic model test results of various researchers and suggested a prediction method for wave transmission :

Range of Validity	*Equation*
$-2.00 < R_c/H_{1/3} < -1.13$	$C_t = 0.80$
$-1.13 < R_c/H_{1/3} < 1.20$	$C_t = 0.46 - 0.3R_c/H_{1/3}$
$1.20 < R_c/H_{1/3} < 2.00$	$C_t = 0.10$

These formulae give a very simplistic description of the data available but will usually be used for preliminary estimate of the performance.

For the range of low wave heights compared to rock diameter and $R_c/H_{1/3} > 1$, Ahrens (1987) gave a formula relating the coefficient with wavelength, rock size and cross-sectional area of the structure :

$$C_t = 1.0/(1.0+X^{0.592}) \qquad \text{for } R_c/H_{1/3} > 1$$

where $X = H_{1/3}A_t/(L_pD_{n50}^2)$

A_t = Cross-sectional area of structure

L_p = Local wave length

B.6 References

Allsop, N.W.H. and Channell, A.R. (1989). Wave Reflections in Harbours : Reflection Performance of Rock Armoured Slopes in Random Waves. Report OD 102. Hydraulics Research Ltd, Wallingford.

Ahrens, J.P. (1987). Characteristics of Reef Breakwaters. Technical Report CERC-87-17. US Army Corps of Engineers, Coastal Engineering Research Centre, Vicksburg.

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Technical Rport W 178. Hydraulics Research Ltd, Wallingford.

CIRIA (1991). Manual on the Use of Rock in Coastal and Shoreline Engineering. Construction Industry Research and Information Association, United Kingdom, 907p.

Goda, Y. (2000). Random Seas and Design of Maritime Structures. World Scientific Publishing Co Pte Ltd, Singapore, 443p.

Owen, M.W. (1980). Design of Seawalls allowing for Wave Overtopping. Report Ex 924. Hydraulics Research Ltd, Wallingford.

Postma, G.M. (1989). Wave Reflection from Rock Slopes under Random Wave Attack. Delft University of Technology.

Van der Meer (1988). Rock Slopes and Gravel Beaches under Wave Attack. Doctoral thesis. Delft University of Technology.

Van der Meer (1990). Data on Wave Transmission due to Overtopping. Report H986. Delft Hydraulics.

LIST OF TABLES

Table B1 Wave Run-up Coefficients

Exceedance Levels i	*a*	*b*	*c*	*d*
1%	1.01	1.24	0.48	2.15
2%	0.96	1.17	0.46	1.97
5%	0.86	1.05	0.44	1.68
10%	0.77	0.94	0.42	1.45
Significant	0.72	0.88	0.41	1.35
Note : These are coefficients used in the Van der Meer wave run-up prediction formulae.				

Table B2 Roughness Coefficients

Type of Slope	Roughness Coefficient *r*
One layer of rock armour on impermeable base	0.80
One layer of rock armour on permeable base	0.55 - 0.60
Two layers of rock armour	0.50 - 0.55

Table B3 Wave Overtopping Coefficients

Front Face Slope of Structure	*A*	*B*
1 : 1	0.00794	20.1
1 : 1.5	0.00884	19.9
1 : 2	0.00939	21.6
1 : 2.5	0.0103	24.5
1 : 3	0.0109	28.7
1 : 3.5	0.0112	34.1
1 : 4	0.0116	41.0

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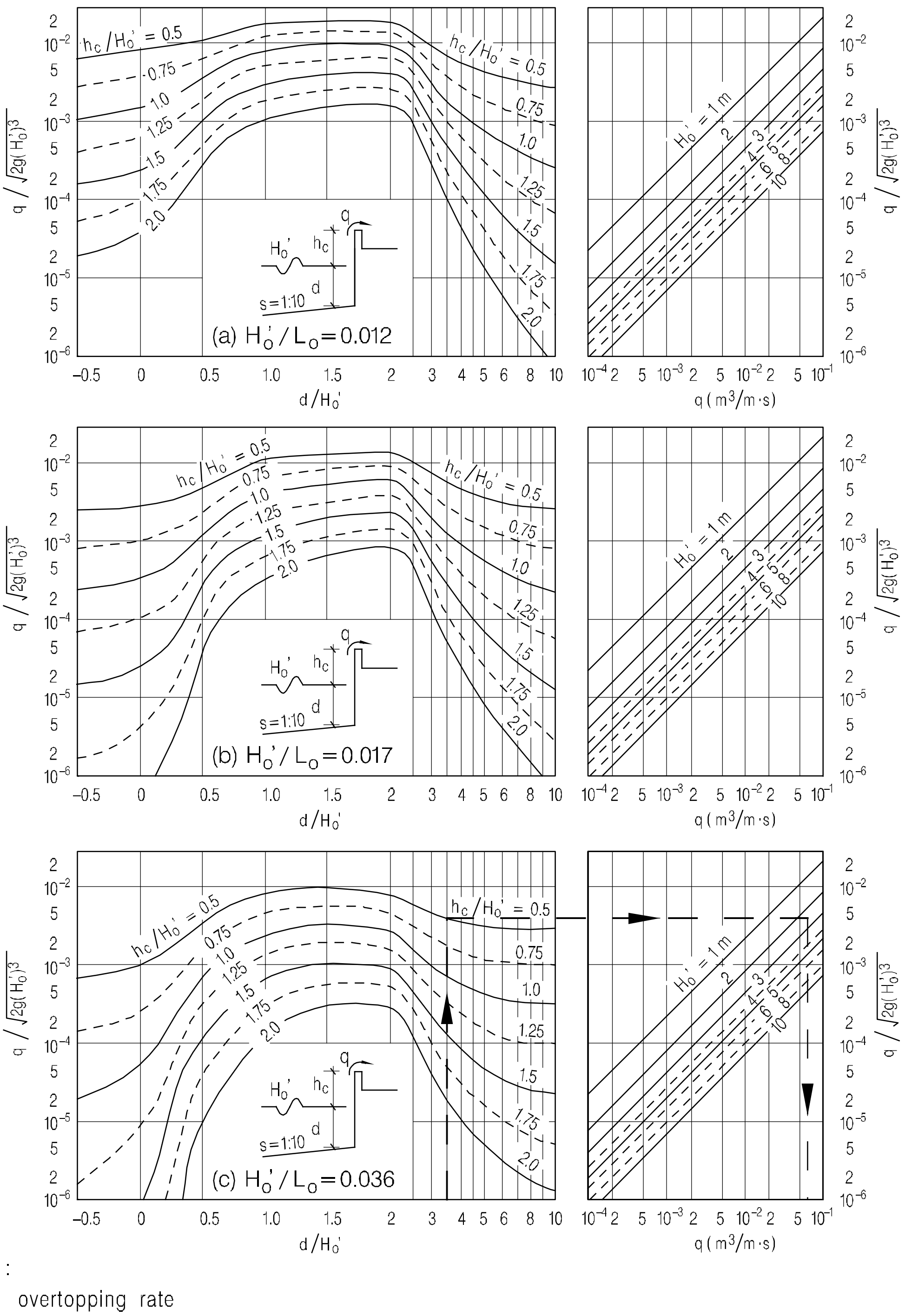


Legend :

q = overtopping rate

H_o' = equivalent deep-water wave height

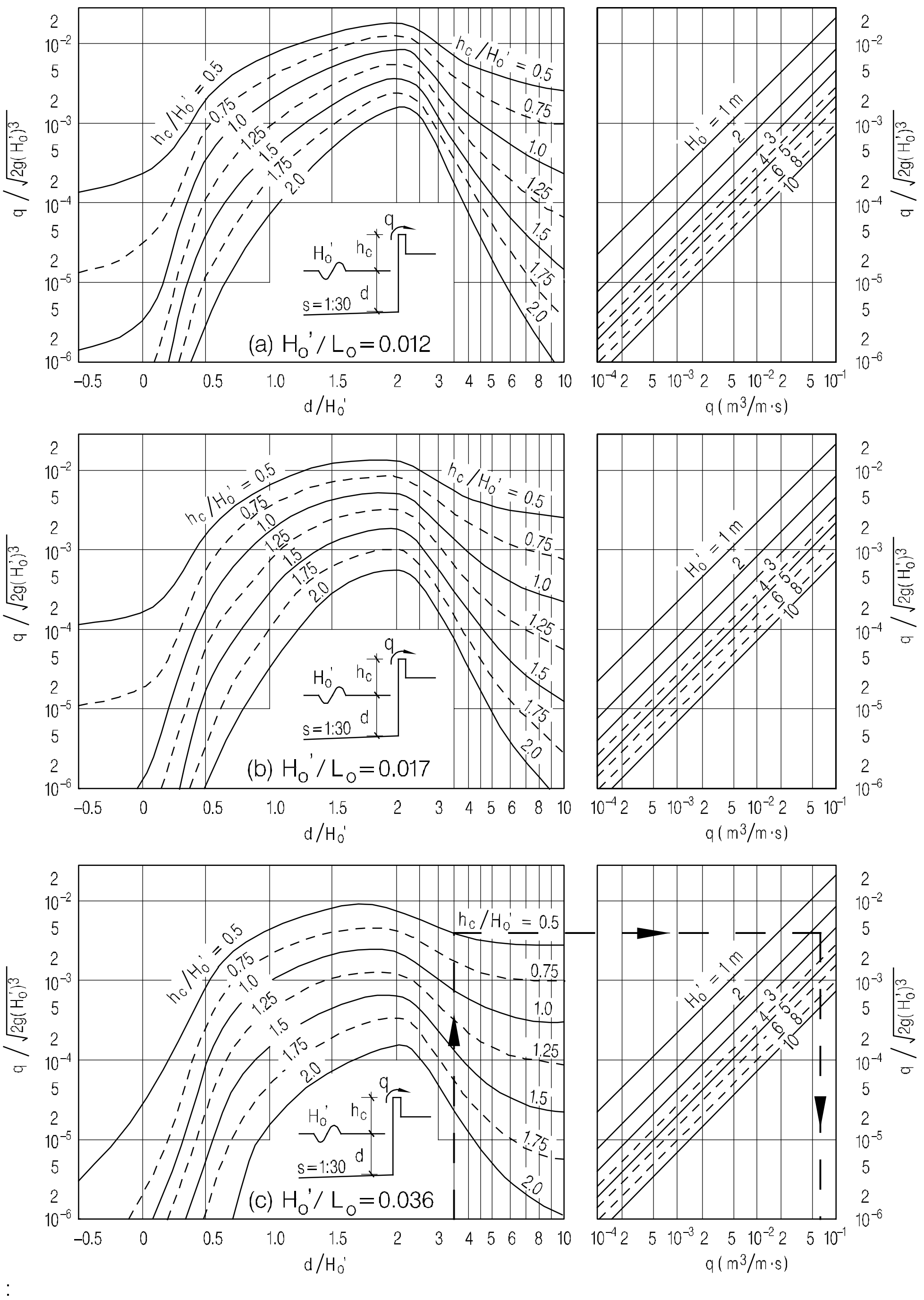
L_o = deep-water wavelength corresponding to significant wave period i.e. $L_o = gT_s^2/2\pi$

h_c = height of top of wall above still water level

s = seabed slope (tangent of angle relative to horizontal)

Source : Goda (2000)

Figure B1 – Prediction of Overtopping Rates for Vertical Structures – Seabed Slope 1 / 10



Legend :

q = overtopping rate

H_o' = equivalent deep–water wave height

L_o = deep–water wavelength corresponding to significant wave period i.e. $L_o = g T_s^2 / 2\pi$

h_c = height of top of wall above still water level

s = seabed slope (tangent of angle relative to horizontal)

Source : Goda (2000)

Figure B2 – Prediction of Overtopping Rates for Vertical Structures – Seabed Slope 1 / 30

APPENDIX C

DETERMINATION OF SIZE OF ARMOUR

CONTENTS

APPENDIX C DETERMINATION OF SIZE OF ARMOUR

C.1 General

This appendix discusses the Hudson Formula and the Van der Meer Formulae for calculating the size of rock armour of rubble mound structures.

C.2 Hudson Formula

The Hudson formula was derived from a series of regular wave tests using breakwater models. The formula is given by :

$$W = \frac{\rho_r g H^3}{K_D \Delta^3 \cot\alpha}$$

where W = Weight of an armour unit (N).
H = Design wave height at the structure (m).
K_D = Dimensionless stability coefficient.
α = Slope angle of structure.
ρ_r = Mass density of armour (kg/m^3).
g = Acceleration due to gravity (m/s^2).
Δ = Relative mass density of armour = (ρ_r / ρ_w) – 1
ρ_w = Mass density of seawater (kg/m^3).

ρ_r and ρ_w may be taken as 2600 kg/m^3 for rock and 1025 kg/m^3 for seawater respectively for design purposes.

For non-breaking wave conditions, the design wave height may be taken as $H_{1/10}$ at the site of the structure. For conditions where $H_{1/10}$ will break before reaching the structure, the wave height used in design should be the breaking wave height or the significant wave height, whichever has the more severe effect (BSI, 1991).

Suggested values of K_D for rock armour at the trunk and head of structures under non-breaking and breaking wave conditions can be found in BS6349:Part 7:1991 (BSI, 1991). These quoted values do not take account of the differences in factors such as wave period and spectrum, shape of armour rock, placement method, interlocking, angle of wave incidence, size of underlayer and porosity which will have influence on the stability. They should not be used without careful reviews of the factors involved.

C.3 Van der Meer Formulae

Van der Meer derived two formulae for plunging and surging waves. These formulae take account of the influence of wave period, storm duration, armour grading, spectrum shape, groupiness of waves, core permeability and damage level on rock armour, and therefore they are described as practical design formulae for rock armour. The formulae are (BSI, 1991) :

For plunging waves,

$$\frac{H}{\Delta D_{n50}}\sqrt{\xi_m} = 6.2P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}$$

For surging waves,

$$\frac{H}{\Delta D_{n50}} = 1.0P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2}(\sqrt{\cot\alpha})\ \ \xi_m{}^{P}$$

where H = Design wave height, taken as the significant wave height (m).
D_{n50} = Nominal rock diameter equivalent to that of a cube (m).
Δ = Relative mass density of armour = $(\rho_r / \rho_w) - 1$
P = Notional permeability factor (see Figure 13).
α = Slope angle of structure.
N = Number of waves.
S = Damaged level = $A / D_{n50}{}^2$
A = Erosion area in a cross-section (m^2).
ξ_m = Surf similarity parameter for mean wave period = $(\tan\alpha)/\sqrt{s_m}$
s_m = Offshore wave steepness based on mean period = $2\pi H / gT_m{}^2$
T_m = Mean wave period (s).
g = Acceleration due to gravity (m/s^2).

The transition from plunging to surging waves is calculated using a critical value of ξ_c (CIRIA, 1991) :

$$\xi_c = (6.2P^{0.31}\sqrt{\tan\alpha})^{1/(P+0.5)}$$

Depending on the slope angle and permeability, this transition lies between ξ_c = 2.5 to 3.5. When the value of surf similarity parameter is greater than ξ_c, the formula for surging waves should be used. For $\cot\alpha \geq 4$, the transition from plunging to surging does not exist and for these slopes, only formula for plunging waves should be used.

The notional permeability factor P should lie between 0.1 for a relatively impermeable core to 0.6 for a virtually homogeneous rock structure. The choice of P depends on designer's judgement. Where data are not available for a detailed assessment, P may be taken as 0.3 for rock armoured breakwater, unless an open core is to be provided. If in doubt, it is recommended that the permeability be underestimated rather than over-estimated.

The damage level S is the number of cubic stones with a side of D_{n50} being eroded around the water level with a width of one D_{n50}. The limits of S depend mainly on the slope of the structure. For a two-diameter thick armour layer, the lower and upper damage levels have been assumed to be the values shown in Table C1. The start of damage of S = 2 to 3 is the same as that used by Hudson, which is roughly equivalent to 5% damage. Failure is defined as exposure of the filter layer.

The formulae can be used when the number of waves N, or storm duration, is in the range of 1000 to 7000. For N greater than 7000, the damage tends to be overestimated. Unless data are available for more detailed assessment, values of N from 3000 to 5000 may be used for preliminary design purpose (BSI, 1991).

The slope of the armour structure, $\cot\alpha$, should lie between 1.5 and 6. The wave steepness s_m should be within the range of 0.005 and 0.06. Waves become unstable when the steepness is greater than 0.06.

For shallow water conditions, the parameter ($H_{2\%}$/1.4) should be used in the above Van der Meer formulae instead of significant wave height $H_{1/3}$. This is based on the analysis of some test results of breaking waves on the foreshore of a structure. These results indicated that if the structure is located in relatively shallow water and that if the wave height distribution is truncated, the 2% value of the wave height exceedance curve gives the best agreement with results showing a Rayleigh distribution (Van der Meer, 1990).

Some further remarks on the use of the formulae are also given here. A deterministic design procedure is followed if various design parameters are input in the formulae to determine the size of rock armour and if a sensitivity analysis is carried out on the various parameters. Another design procedure is the probabilistic approach in which the formulae are rewritten to so-called reliability functions and all the parameters can be assumed to be stochastic with an assumed distribution. For details of the latter approach, reference can be made to CIRIA (1991).

C.4 References

BSI (1991). Maritime Structures – Part 7 : Guide to the Design and Construction of Breakwaters (BS 6349:Part 7 : 1991). British Standards Institution, London, 88p.

CIRIA (1991). Manual on the Use of Rock in Coastal and Shoreline Engineering. Construction Industry Research and Information Association, United Kingdom, 907p.

Van der Meer, J.W. (1990). Rubble Mounds – Recent Modifications, Handbook of Coastal and Ocean Engineering, Volume 1, edited by J.B. Herbich, Gulf Publishing Company, Houston, pp. 883-894.

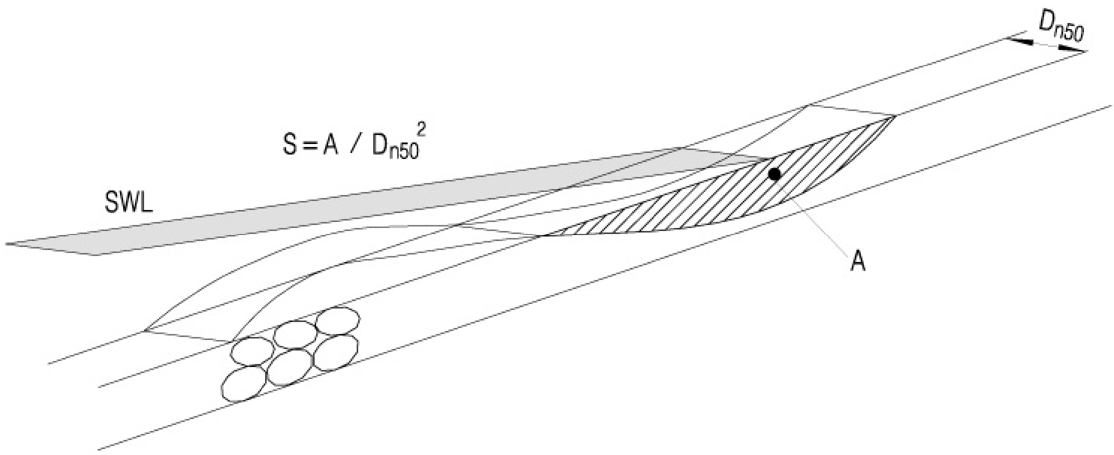
LIST OF TABLES

Table C1 Damage Levels for Two-Diameter Thick Rock Slopes

Slope of Structure A	Damage Level S at Start of Damage	Damage Level S at Failure
1:1.5	2	8
1:2.0	2	8
1:3.0	2	12
1:4.0	3	17
1:6.0	3	17

Note : 1. Damage Level $S = A/D_{n50}^2$, where A is the eroded area of the cross-section of the structure and is the hatched area as shown in the figure below.

2. Source : CIRIA (1991).



APPENDIX D

WORKED EXAMPLES

CONTENTS

D.1 WAVE OVERTOPPING OF RUBBLE MOUND SEAWALL

Reference Section 5.3 and Appendix B.3.

Given

A rubble mound seawall with two layers of rock armour.
Crest level = +4.5 mPD
Slope of seawall (front face) = 1 : 2
Sea level = +3.2 mPD
Significant wave height at seawall = 2.0 m
Mean wave period = 4.4 s
Angle of incident wave to the normal of the seawall = 30 degrees

Find

Mean overtopping rate of the rubble mound seawall.

Solution

Take g = 9.81m/s^2 and use Owen's formulae in Appendix B.3.1

Dimensionless crest freeboard R_*

$= R_c/(T_m(gH_{1/3})^{0.5})$

$= \dfrac{(4.5-3.2)}{4.4\times\sqrt{9.81\times2.0}}$

$= 0.067$

Dimensionless mean discharge Q_*

$= A \exp(-BR_*/r)$

From Table B3, for slope of seawall (front face) = 1:2, take empirical coefficients A and B to be 0.00939 and 21.6 respectively.

From Table B2, for two layers of rock armour, take roughness coefficient r to be 0.5.

$Q_* = 0.00939 \exp(-21.6\times0.067/0.5)$

$\quad = 5.2\times10^{-4}$

$Q = Q_* T_m g H_{1/3}$

= $5.2 \times 10^{-4} \times 4.4 \times 9.81 \times 2.0$

= 0.045 m^3/s per meter run of the seawall

Reduction factor for incident waves not normal to the structures O_r

O_r = $1 - 0.000152\ \beta^2$

= $1 - 0.000152\ (30)^2$

= 0.86

Therefore, mean overtopping discharge

= $Q \times O_r$

= 0.045×0.86

= 0.039 m^3/s per meter run of the seawall

This overtopping rate is nearly equal to the suggested limit of the damage to unpaved surface, 5×10^{-2} $m^3/m/s$ listed in Section 5.3.2 of this part of the Manual.

D.2 WAVE OVERTOPPING OF SOLID FACE VERTICAL SEAWALL

Reference Section 5.3 and Appendix B.3.

Given

A solid face vertical seawall with toe level close to the seabed level.
Crest level = +4.5 mPD
Sea level = +3.2 mPD
Seabed level = –6.0 mPD
Significant wave height at seawall = 2.0 m
Mean wave period = 4.4 s
Incident wave angle : normal to seawall
Seabed slope = 1:30

Find

Mean rate of wave overtopping of the vertical seawall.

Solution 1

Based on the method mentioned by Besley (1999) in Appendix B.3.2 :

Water depth $d = 3.2 - (-6.0) = 9.2$ m
Height of top of wall above still water level R_c
$= 4.5 - 3.2$ m
$= 1.3$ m

Dimensional parameter d_*
$= (d/H_{1/3})(2\pi d/(gT_m^2))$
$= (9.2/2.0)(2\pi\, 9.2/(9.81 \times 4.4^2))$
$= 1.4$

As $d_* > 0.3$, reflecting waves predominate, and $R_c/H_{1/3} = 1.3/2.0 = 0.65$. The following equations should apply.

$Q^\# = 0.05 \exp(-2.78\, R_c/H_{1/3})$

where $Q^\#$ is the dimensionless discharge, given by $Q/(gH_{1/3}^3)^{0.5}$

$Q^\# = 0.05 \exp(-2.78 \times 1.3 / 2.0) = 8.2 \times 10^{-3}$

Mean overtopping discharge
$= Q^{\#} (gH_{1/3}{}^3)^{0.5}$
$= 8.2 \times 10^{-3} \times (9.81 \times 2.0^3)^{0.5}$
$= 0.073$ m^3/s per meter run of seawall

<u>Solution 2</u>
Based on the diagram by Goda (2000) in Appendix B.3.2 :

Equivalent deepwater wave height $H_0' \approx H_{1/3} = 2.0$ m
Significant wave period $T_{1/3} \approx 1.2\ T_m = (1.2)(4.4) = 5.3$ s
Wave steepness $= H_0'/((g/2\pi) \times T_{1/3}{}^2) \approx 2.0/((9.81/2/3.1459) \times 5.3^2) = 0.046$
Dimensionless depth parameter $d/H_0' \approx d/H_{1/3} = 9.2/2.0 = 4.6$
Dimensionless crest parameter $h_c/H_0' \approx R_c/H_{1/3} = 1.3/2.0 = 0.65$

By using Figure B2 (c) for the wave steepness $H_0'/L_0 = 0.036$ as having the steepness nearest to the design condition, and reading off the diagram, the dimensionless overtopping rate is obtained as :

$Q/[2g(H_0')^3]^{1/2} \approx 2\times10^{-3}$

Mean overtopping rate
$= 2 \times 10^{-3} \times (2\times9.81\times2.0^3)^{1/2}$
$= 0.025$ m^3/s per meter run of seawall

Even though the above estimate differs from the previous estimate of Solution 1 by a factor of 3, such diversity should be expected because the phenomenon of wave overtopping involves a large spread of data.

D.3 REFLECTION COEFFICIENT OF RUBBLE MOUND SEAWALL

Reference Section 5.4 and Appendix B.4.

Given

A rubble mound seawall with two layers of rock armour.
Slope of seawall = 1 : 2
Significant wave height = 2.0 m
Mean wave period = 4.4 s

Find

Reflection coefficient of the rubble mound seawall.

Solution

Assume notional permeability factor P = 0.3

Peak wave period $T_p = 1.1\times T_{1/3} = 1.1\times 1.2\times T_m = 1.1\times 1.2\times 4.4 = 5.8$ s
(See Section 2.5.3 of Part 1 of this Manual)

Offshore wave steepness based on peak wave period s_p

$= 2\pi H_{1/3}/(gT_p^2)$

$= 2\pi\times 2.0/(9.81\times 5.8^2)$

$= 0.038$

Surf similarity parameter based on peak wave period ξ_p

$$= \frac{\tan\alpha}{\sqrt{s_p}}$$

$$= \frac{1/2}{\sqrt{0.038}}$$

$= 2.56$

(a) Seelig and Ahrens formula

Coefficient of reflection C_r $= a\xi_p^2/(b+\xi_p^2)$

$= 0.6\times 2.56^2/(6.6+2.56^2)$

$= 0.30$

(a=0.6 and b=6.6 as given by the formula)

(b) Postma formula

Coefficient of reflection C_r $= 0.14\xi_p^{0.73}$

$= 0.14 \times 2.56^{0.73}$

$= 0.28$

(c) Postma formula with slope angle and wave steepness treated separately

Coefficient of reflection C_r $= 0.071P^{-0.082}(\cot\alpha)^{-0.62}s_p^{-0.46}$

$= 0.071(0.3)^{-0.082}(2)^{-0.62}(0.038)^{-0.46}$

$= 0.23$

(d) Postma formula modified with Allsop and Channel data

Coefficient of reflection C_r $= 0.125\xi_p^{0.73}$

$= 0.125 \times 2.56^{0.73}$

$= 0.25$

D.4 ROCK ARMOUR OF RUBBLE MOUND BREAKWATER

Reference Section 6.2 and Appendix C.

Given

A conventional rubber mound breakwater in deepwater with two-diameter thick armour layer.
Slope of breakwater = 1 : 2
Significant wave height = 2.0 m
Mean wave period = 5.0 s
Damage level : Only start of damage is allowed

Find

Size of rock armour.

Solution

Mass density of rock armour ρ_r = 2600 kg/m^3
Mass density of seawater ρ_w = 1025 kg/m^3
Acceleration due to gravity g = 9.81 m/s^2

(a) Hudson's formula

Relative mass density of armour Δ
$=(\rho_r / \rho_w) - 1$
= (2600/1025) – 1
= 1.54

Assume non-breaking wave condition as the breakwater is in deepwater. For non-breaking waves, design wave height at structure is taken as $H_{1/10}$.

$$H_{1/10} = 1.27 H_{1/3} = 1.27 \times 2.0 = 2.54 \text{ m}$$

From Table 7 of BS6349:Part 7:1991, for trunk of structures with two layers of rough angular rock under non-breaking wave condition, dimensionless stability coefficient K_D = 4.0 .

Therefore, weight of armour unit

$$W = \frac{\rho_r g H^3}{K_D \Delta^3 \cot\alpha} = \frac{(2600)(9.81)(2.54)^3}{(4.0)(1.54)^3 (2)} = 14305\text{N} = 14.3 \text{ kN}$$

(b) Van der Meer formula

The breakwater is not in shallow water.
Take design wave height as significant wave height $H_{1/3}$ = 2.0 m.
Relative mass density of armour Δ
$= (\rho_r / \rho_w) - 1$ = 1.54

Offshore wave steepness based on mean period s_m

$$= \frac{2\pi H_{1/3}}{g T_m{}^2} = \frac{2 \times \pi \times 2.0}{9.81 \times 5.0^2} = 0.051$$

Surf similarity parameter for mean wave period ξ_m

$$= \frac{\tan\alpha}{\sqrt{s_m}} = \frac{1/2}{\sqrt{0.051}} = 2.21$$

Only start of damage is allowed and slope of breakwater = 1 : 2.
Therefore, from Table C1, damage level S = 2.

Assume number of waves N = 4000 and notional permeability factor P = 0.3.

Critical value of ξ_c

$$= (6.2 P^{0.31} \sqrt{\tan\alpha})^{1/(P+0.5)} = [(6.2)(0.3)^{0.31} \sqrt{0.5}]^{1/(0.3+0.5)} = 3.98$$

Since $\xi_m < \xi_c$, the formula for plunging waves should be used.

$$\frac{H_{1/3}}{\Delta D_{n50}} \sqrt{\xi_m} = 6.2 P^{0.18} (\frac{S}{\sqrt{N}})^{0.2}$$

Thus, nominal rock diameter D_{n50}

$$= \frac{H_{1/3}\sqrt{\xi_m}}{\Delta} / \left[6.2 P^{0.18} (\frac{S}{\sqrt{N}})^{0.2} \right] = \frac{2.0\sqrt{2.21}}{1.54} / \left[(6.2)(0.3)^{0.18} (\frac{2}{\sqrt{4000}})^{0.2} \right] = 0.77 \text{ m}$$

Nominal mass of rock armour = $(0.77)^3(2600)$ = 1187 kg

Weight of rock armour = 11.6 kN

D.5 UNDERLAYER OF RUBBLE MOUND BREAKWATER

Reference Section 6.2.4.

Given

A conventional rubber mound breakwater with two-diameter thick armour layer.
Nominal mass of rock armour = 2000 kg
D_{15} of rock armour = 0.83 m

Find

Size of underlayer rock.

Solution

Take the number of rock layers of the underlayer $n = 2$
For rock, layer thickness coefficient $k_{\Delta} = 1.15$
Mass density of rock = 2600 kg/m^3

The nominal mass of rock in the underlayer should be at least 1/10 of the nominal mass of rock armour, i.e. > 2000/10 = 200 kg.

The nominal rock size of the underlayer $D_{50} > (200/2600)^{1/3} = 0.425$ m

To prevent smaller rocks in the underlayer from being taken out through the armour layer by wave action, the following filter criteria are checked.

$$D_{15(\mathrm{armour})} / D_{85(\mathrm{underlayer})} \leq 4$$
$$4 \leq D_{15(\mathrm{armour})} / D_{15(\mathrm{underlayer})} \leq 20$$

$D_{15(\mathrm{armour})} = 0.83$ m

Therefore, $D_{85(\mathrm{underlayer})} \geq 0.21$ m
$0.04 \text{ m} \leq D_{15(\mathrm{underlayer})} \leq 0.21$ m

Note :
The filter requirement of the underlayer should also be checked with the size of core material of the breakwater, although this is not shown in this worked example.

D.6 TOE PROECTION

Reference Section 6.2.8 and Figure 16.

Given

A critical vertical seawall located in an open exposed area.

Sea level = +3.2 mPD

Seabed level = –5.0 mPD

Top level of toe protection = –4.0 mPD

Slope of rubble toe protection = 1 : 2

Significant wave height at seawall = 2.0 m

Mean wave period = 4.4 s

Find

Rock size and width of toe protection.

Solution

Referring to Figure 16,

$d_1 =$ 3.2 – (–4.0) = 7.2 m

$d_s =$ 3.2 – (–5.0) = 8.2 m

For intermediate water depth (i.e. $\frac{1}{25} < \frac{d}{L} < \frac{1}{2}$), the wavelength associated with depth d_1 is:

$$L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi d_1}{L}\right)$$

$$L = \frac{9.81 \times 4.4^2}{2\pi}\tanh\left(\frac{2\pi \times 7.2}{L}\right)$$

By iteration, L = 27.9 m

$$\frac{d}{L} = \frac{7.2}{27.9} = 0.258$$

Therefore, the assumption of intermediate water depth is justified.

As the seawall is situated at open exposed site, the design wave height H is taken to be $H_{1/100}$ according to Figure 16.

H = 1.67 $H_{1/3}$ = 1.67 x 2.0 = 3.3 m

The width of toe protection is given by the following:

$B \geq 0.4 d_s = 0.4 \times 8.2 = 3.3$ m

$B \geq 2H = 2 \times 3.3 = 6.6$ m

For B = 6.6 m

$$\frac{B}{L} = \frac{6.6}{27.9} = 0.24$$

$$\frac{B}{d_1} = \frac{6.6}{7.2} = 0.92$$

$$\frac{d_1}{H} = \frac{7.2}{3.3} = 2.18$$

From Figure 16, $N_s = 3.8$

The mass of rock required for toe protection is:

$$\frac{\rho_r H^3}{N_s{}^3 (s_r - 1)^3} = \frac{(2600)(3.3)^3}{(3.8)^3 (2600/1025 - 1)^3} = 469\,\mathrm{kg}$$

The width of toe protection is checked with the following:

$B \geq$ 4 times size of rock = $4 \times (469/2600)^{1/3} = 2.3$ m

This requirement is also satisfied. Therefore, width of toe protection = 6.6m.

GLOSSARY OF TERMS AND SYMBOLS

GLOSSARY OF TERMS

Armour layer. The outermost protective layer of a rubble mound structure composed of armour units which are either quarry rock or specially shaped concrete units.

Bermstones. The protective layer laid in front of the toe of the structure to prevent scouring of foundation material due to waves and currents.

Surf similarity parameter. Being defined as the ratio of tangent of slope angle to the square root of wave steepness, it has often been used to describe the form of wave breaking and to predict wave runup on a sloping beach or structure.

Breakwater head. The end of a breakwater which is more vulnerable to wave attack at all directions. The design of which requires special attention and a more robust structure is required.

Breakwater trunk. The body of the breakwater other than the structure head.

Coping. The uppermost in situ concrete portion of a vertical seawall. It is constructed in the late stage of the construction programme for minimizing the effects of wall settlement upon completion.

Core. The innermost material of a rubble mound breakwater, the permeability of which determines the extent of wave transmission to the leeward side of the breakwater due to long period wave. The more porous is the core material, the higher will be the degree of wave transmission.

Filter. Intermediate layer to prevent fine materials of an underlayer from being washed through the voids of an upper layer.

Freeboard. The height of a structure above still water level.

Longshore sediment transport. The sediment that is transported in the alongshore direction in the nearshore zone by waves and currents.

Overtopping. Water passing over the top of the seawall.

Plunging waves. A kind of breaking waves which occur on mildly to steeply sloping beaches or structures and are characterized by the crest of the wave curling over forward and impinging onto part of the wave trough. The wave itself is spectacular when air escapes by bursting through the back of the wave or by blowing out at a nonbreaking section of wave crest.

Still water level. Water level which would exist in the absence of waves.

Surging waves. A kind of breaking waves which occur on very steeply sloping beaches or structures and are characterized by narrow or nonexistent surf zones and high reflection.

Run-up. The rush of water up a structure as a result of wave action.

Toe of structure. The base of the structure on its seaward face.

Underlayer. A granular layer between the armour layer and the core material, and functions as separation and filter. It also provides a foundation for placement of armour layer.

Volumetric porosity. The ratio of void volume to total volume.

Wave steepness. A ratio of the wave height to the wavelength. The limiting wave steepness in deep water is about 0.142 which occurs when the water particle velocity at the wave crest just equals the wave celerity. A further increase in steepness would result in particle velocities at the wave crest greater than the wave celerity and breaking starts to occur.

Wave wall. A structure built on the seawall or breakwater to reduce wave overtopping.

GLOSSARY OF SYMBOLS

A	Erosion area in a cross-section
B	Crest width
C_r	Wave reflection coefficient
C_t	Wave transmission coefficient
D	Nominal size of an equivalent cube
D_{15}	15% of the material passing through that size
D_{50}	50% of the material passing through that size
D_{85}	85% of the material passing through that size
d	Water depth
E_i	Incident wave energy
E_r	Reflected wave energy
E_t	Transmitted wave energy
g	Acceleration due to gravity
H	Wave height
$H_{1/3}$	Significant wave height, also denoted as H_s in other literatures
H_i	Incident wave height
H_{max}	Maximum wave height
H_o'	Equivalent deepwater significant wave height
H_r	Reflected wave height
H_{total}	Total wave height
K_D	Dimensionless stability coefficient in Hudson's formula
k_Δ	Layer thickness coefficient
L	Wavelength
L_o	Deepwater wavelength
N	Number of waves
N_a	Average number of armour units per unit area

N_o	Number of waves during a peak of storm events
n	Number of armour layers or number of rock layers
P	Notional permeability factor
p	Volumetric porosity
Q	Mean overtopping discharge rate per meter run of structure
R_c	Freeboard between still water level and crest of structure
$R_{u2\%}$	The run-up level exceeded by 2% of the incident waves
R_{ui}	The run-up at i % exceedance level.
r	Roughness coefficient
S	Damaged level
s	Slope of seabed
s_m	Offshore wave steepness based on mean wave period
s_p	Offshore wave steepness based on peak wave period
T	Wave period
$T_{1/3}$	Significant wave period, also denoted as T_s in other literatures
T_m	Mean wave period
t_a	Thickness of armour layer
t_u	Thickness of underlayer
W_a	Weight of an armour unit
W	Weight of a rock in the underlayer
α	Slope angle of structure
β	Incident wave angle relative to normal of structure
γ_a	Unit weight of armour unit
γ_r	Unit weight of rock
γ_w	Unit weight of water
ξ_c	Critical surf similarity parameter for transition from plunging to surging waves
ξ_m	Surf similarity parameter based on mean wave period

ξ_p	Surf similarity parameter based on peak wave period
ρ , ρ_w	Mass density of seawater
ρ_r	Mass density of rock
Δ	Relative mass density e.g. for rock Δ = (mass density of rock/mass density of water) – 1