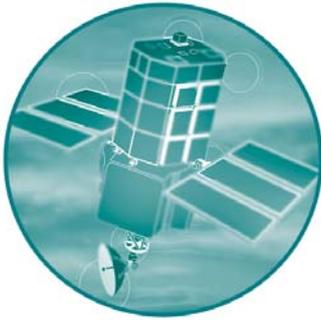


Defra/Environment Agency Flood and Coastal Defence R&D Programme



Low Cost Rock Structures for Beach Control and Coast Protection Practical Design Guidance

R&D Technical Report

FD2409

**Defra/Environment Agency
Flood and Coastal Defence R&D Programme**

**Low Cost Rock Structures for Beach Control and
Coast Protection**

Practical design guidance

R&D Technical Report FD2409

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This document provides information on the design and assessment of low cost rock structures for beach control and coast protection and constitutes an R&D output from the Joint DEFRA / Environment Agency Flood and Coastal Defence R&D Programme. This report also serves as HR Wallingford report SR 631.

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EXECUTIVE SUMMARY

Coastal rock structures are widely used in coastal engineering for a variety of purposes, including controlling the morphological development of beaches and providing protection against coastal erosion or flooding by wave overtopping. Strict adherence to existing design guidance has resulted in many of these structures being built using multiple layers of different rock sizes, high quality imported rock and carefully prepared foundations. Some innovative structures have, however, used locally available rock with simpler cross-sections placed on unprepared foundations, apparently without significant reduction to the overall performance of the scheme.

This report gives guidance from a short research project which examined practical experience on rock structures from around the UK, with particularly emphasis on those that depart from conventional design rules. The report demonstrates that there are opportunities for lower cost rock structures for beach control and coast protection. Established design guidance provides a good degree of confidence in predictions of performance of coastal structures, but it is widely perceived that simple design rules can be overly prescriptive, particularly for nearshore structures in shallow water depths.

The opportunities for lower cost structures principally relate to improved assessment of armour size for depth-limited waves, reduction in armour size for closer armour packing, and the need for complex underlayer / filters. The report emphasises the need to understand the performance of individual structures in the context of the overall scheme and ultimately national objectives, which provides an incentive to re-explore the balance between cost and structure performance. It also encourages the consideration of cost issues during the design of rock structures.

Although the use of lower cost structures may also provide safety and environmental advantages, the structures described are envisaged to be of greatest benefit in locations where conventional structures would not be economically justified.

CONTENTS

| | |
|---|-----------|
| EXECUTIVE SUMMARY | i |
| 1. Introduction | 1 |
| 1.1 Common structure types and functions | 1 |
| 1.2 Rationale for low cost rock structures | 1 |
| 1.3 Description of the project | 4 |
| 1.4 Outline and use of this report | 5 |
| 2. Design and assessment | 6 |
| 2.1 Performance and scheme requirements | 6 |
| 2.2 Failure and adaptability | 7 |
| 2.3 Environmental impacts | 9 |
| 2.4 Safety considerations | 9 |
| 3. Established design guidance and techniques | 11 |
| 3.1 Geometrical design | 11 |
| 3.2 Structural design | 14 |
| 3.3 Rock Materials | 21 |
| 4. Understanding cost issues | 25 |
| 4.1 Design and assessment | 25 |
| 4.2 Rock armour | 26 |
| 4.3 Construction | 29 |
| 4.4 Monitoring, maintenance and repair | 31 |
| 4.5 Programming expenditure | 32 |
| 5. Lower cost rock structures | 33 |
| 5.1 Less material | 33 |
| 5.2 More efficient construction | 35 |
| 5.3 Cheaper rock | 35 |
| 5.4 Reducing Excavation | 37 |
| 6. Design and detailing for lower cost | 38 |
| 6.1 Appropriate armour sizing | 38 |
| 6.2 Foundation and toe details | 48 |
| 6.3 Monitoring and maintaining low cost rock structures | 54 |
| 7. Conclusions | 57 |

| | |
|-------------------------|-----------|
| Acknowledgements | 58 |
| References | 59 |
| Notation | 64 |

Tables

| | | |
|----------|---|----|
| Table 1 | Wave transmission over low-crest breakwaters..... | 11 |
| Table 2 | Values of K_D for use in Hudson equation..... | 15 |
| Table 3 | Design values of S_d for $2D_{n50}$ thick armour layer..... | 16 |
| Table 4 | Rock shape coefficients..... | 17 |
| Table 5 | Guide to rock durability..... | 23 |
| Table 6 | Suggested grading ranges..... | 24 |
| Table 7 | Potential sources of rock armour for south coast scheme..... | 27 |
| Table 8 | Transport methods..... | 28 |
| Table 9 | Characteristic dimensionless wave heights..... | 40 |
| Table 10 | Performance levels for rock structures..... | 55 |

Figures

| | | |
|-----------|---|----|
| Figure 1 | Conventional breakwater cross-section..... | 2 |
| Figure 2 | Conventional rock groyne cross-section..... | 2 |
| Figure 3 | Elmer breakwater cross section..... | 3 |
| Figure 4 | 'Reef' breakwater cross section..... | 3 |
| Figure 5 | Simplified rock groyne cross section, Mudeford Sandbank..... | 3 |
| Figure 6 | Performance hierarchy for flood and coast defence..... | 6 |
| Figure 7 | Public use of a rock structure at Lyme Regis for recreation..... | 10 |
| Figure 8 | Wave transmission performance for low crest breakwaters..... | 12 |
| Figure 9 | Notional permeability factor..... | 16 |
| Figure 10 | Filter performance / failure modes..... | 20 |
| Figure 11 | Rosin-Rammler theoretical equation for stone gradation and fragmentation..... | 27 |
| Figure 12 | Shallow-water significant wave heights for uniform foreshore slopes..... | 39 |
| Figure 13 | Selective placement in the toe of a nearshore breakwater at Sidmouth..... | 44 |
| Figure 14 | Comparison of stability prediction methods for rock armour and concrete slabs..... | 47 |
| Figure 15 | Toe detail from a fishtailed breakwater at West Shore, Llandudno..... | 49 |
| Figure 16 | Falling apron toe detail..... | 49 |
| Figure 18 | Permeable Rock groyne at Mappleton..... | 52 |

Boxes

| | | |
|-------|---|----|
| Box 1 | Rock groynes at Highcliffe, Dorset..... | 8 |
| Box 2 | Fishtail breakwaters at Westshore, Llandudno..... | 34 |
| Box 3 | Rock groynes at Hengistbury Head..... | 36 |
| Box 4 | Determination of design wave heights..... | 40 |
| Box 5 | Tight packing for a revetment..... | 42 |
| Box 6 | Variation in armour along the length of a groyne..... | 45 |
| Box 7 | Paddys Gap Rock Revetment Works..... | 53 |

1. INTRODUCTION

Whilst rock has been a natural feature on coasts throughout history it is only relatively recently that it has been widely used around the UK coastline for beach control and coast protection structures. A wide range of different types of rock structures have been designed and constructed, particularly since the late 1970s. These various structures have been developed to satisfy different functional and performance requirements at particular locations and whilst some have borrowed from design techniques used for large harbour breakwaters, others have been developed on the basis of trial and refinement, often using locally available materials.

1.1 Common structure types and functions

Coastal rock structures can be broadly categorised by type and function; this study is focused on applications to control the morphological development of beaches and provide protection against coastal erosion. The types of structures most commonly used for this function are briefly described below.

- Nearshore breakwaters: structures intended to reduce wave energy reaching the shoreline, usually aligned parallel to the coast located either individually at transition points or in groups over longer lengths, with appropriate spacing between individual breakwaters
- Revetments: sloping structures used to protect an embankment, natural coast or shoreline against erosion by absorbing wave energy, which also reduces wave over-topping
- Groynes: structures generally aligned perpendicular to the shoreline intended to interrupt or reduce longshore currents and / or transport of sediment

Other types of structures have also been applied; on duned coasts rock headlands may be used to protect the toe of eroding dunes over short lengths, allowing the dunes between the headlands to erode naturally to form an increasingly embayed shoreline and, providing the spacing has been correctly determined, enabling a stable plan shape to develop. Some structures are variations on (or composites of) those described above. Rock sills or reefs are similar to submerged nearshore breakwaters, placed at the toe of a natural or nourished beach, reducing offshore transport and causing the biggest waves to break, most effective where there is a small range in possible water levels. Other structures combine two of the above types, for example fish-tail breakwaters combine aspects of nearshore breakwaters with those of rock groynes.

1.2 Rationale for low cost rock structures

Most existing literature on the design and assessment of coastal rock structures, including much of the 'Rock Manual' (CIRIA/CUR 1991) and 'Coastal Engineering Manual'(USACE 2003), is based on research primarily directed towards the large, relatively deep water structures which shelter ports and maritime facilities from wave disturbance and are often designed for minimal maintenance. In general the degree of complexity of coastal rock structures will be a function of the accessibility of the site, cost and acceptability of maintenance – that is the acceptability with which integrity and performance can be compromised to reduce cost. Since beach control and coast protection structures are often relatively easily accessible and the long term scheme

performance is seldom critically dependent on the short term integrity of the structure, simple structures which are inexpensive to construct but require more frequent maintenance are likely to be suitable. The prescriptive application of design guidance developed for large offshore breakwaters to these smaller, predominantly beach based structures, can result in excessively complex and costly structures which may also be dangerous to build and increase environmental damage.

Whilst there is only limited guidance available on the design of simpler structures, a wide range of different structures have been constructed. Examples below illustrate the range of complexity starting with the conceptual cross section adopted for conventional deep-water rock mound structures, Figure 1, and extending to simpler structures in Figures 2-5.

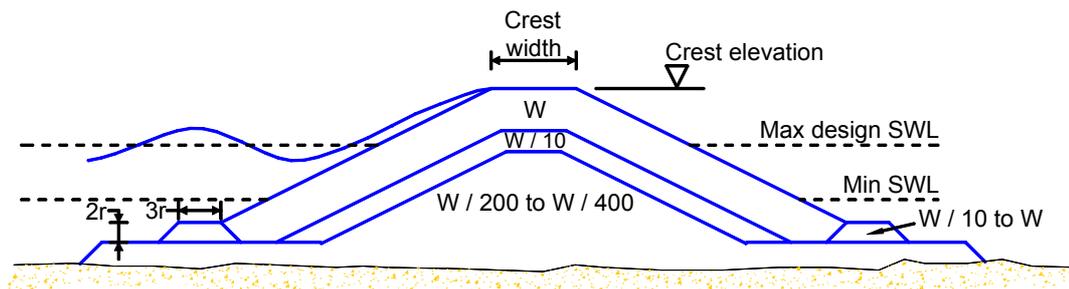


Figure 1 Conventional breakwater cross-section (after SPM 1984)

The CIRIA/CUR Rock Manual edited by Simm (1991) shows a groyne cross-section (Figure 2 below) from the Atlantic Coast of North Carolina, USA which follows a broadly similar pattern with armour and underlayers following filter rules (although in this case the underlayer is W/15) and a bedding layer provided to ensure a uniform foundation with minimal settlement into the substrate.

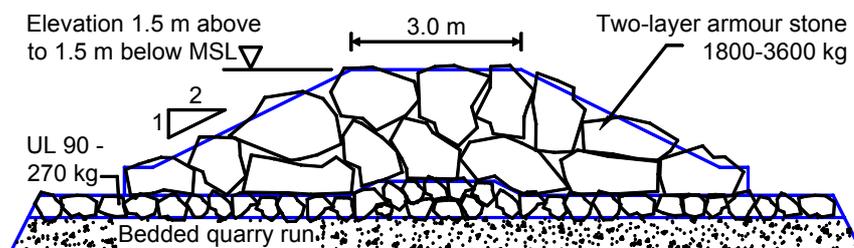


Figure 2 Conventional rock groyne cross-section (after CIRIA/CUR 1991)

In practice some designers have adopted simpler structures using a single grading of armour, sometimes with a bedding layer and sometimes placed directly onto hard substrate. One example of this is the nearshore breakwaters at Elmer which are illustrated in Figure 3. The armour is from the 'standard' Rock Manual 6-10 tonne grading and was sized using the van der Meer equations for static stability during the design event.

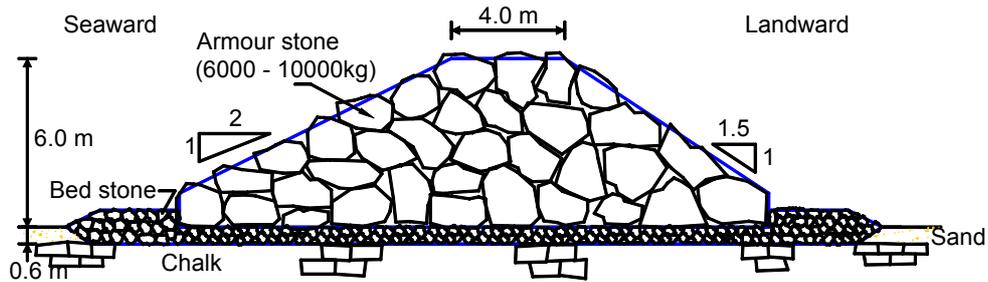


Figure 3 Elmer breakwater cross section (after Holland & Coughlan 1994)

The ‘reef’ breakwaters developed in the USA and described by Ahrens (1989) and Chasten et al (1993) are illustrated in Figure 4. They use a similar cross-section to that applied at Elmer, but have a much lower crest elevation and are designed to be dynamically stable with the armour grading typically encompassing the range of material that would be used for the primary armour and first underlayer of a conventional statically stable structure. These structures have a low crest and high porosity which increases the armour stability (but will also increase the amount of wave energy passing the structure) and are resilient to damage – since they have no core they will not suffer catastrophic failure. Once constructed they are expected to adjust and deform under wave loading, but are designed such that they will continue to provide the required performance.

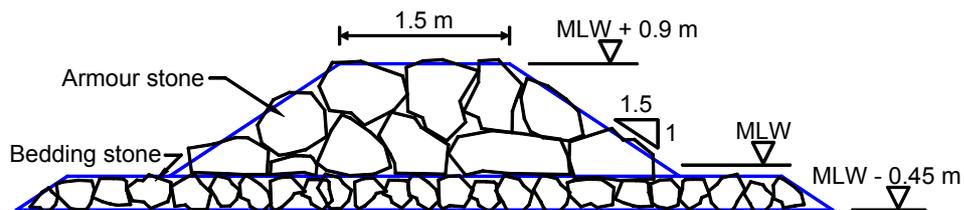


Figure 4 ‘Reef’ breakwater cross section (after Chasten et al 1993)

A similar approach has been used for groynes in the UK, although in the example illustrated in Figure 5 the bedding layer was omitted since the structure was founded on a hard stratum underlying the beach.

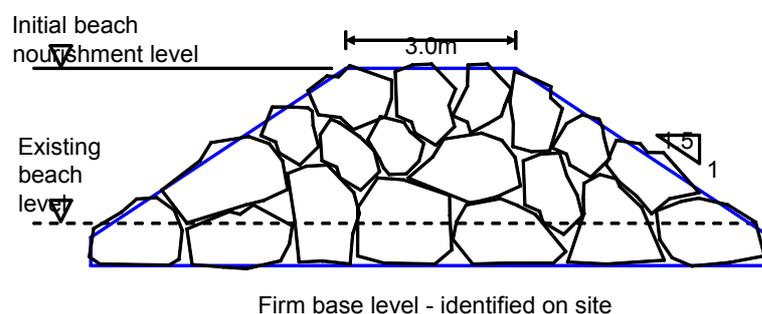


Figure 5 Simplified rock groyne cross section, Mudford Sandbank

The application of unconventional structures has largely been the desire for lower cost structures combined with an acceptance that this will result in reduced, or at least less predictable, performance. Other attributes of these structures include:

- Easier (and often quicker and cheaper) construction

- Increased construction safety
- Reduced environmental impact / damage
- More adaptable structures which can be altered in response to changing conditions or requirements, but to do so, will need regular inspection and (possibly) maintenance

The compromise between cost and performance is likely to be most acceptable where maintenance is relatively easy and a reduction in the integrity or performance of the structure will not jeopardise the success of the scheme. The former will require plant and materials to be readily available and easy access to the structure, the latter will require an understanding of the functioning and performance of the scheme as a whole and good monitoring arrangements. Most beach control and coast protection structures function in a predominantly morphological manner with the structures influencing the long term development of the beach, but providing minimal protection in the event of a storm. However, structures such as revetments protecting tidal flood embankments, provide critical performance during storm events and failure is likely to be catastrophic. In these situations application of the empirical guidance contained in this document will not be appropriate and conventional design procedures are recommended.

1.3 Description of the project

An earlier scoping study by Crossman & Allsop (2000) included a consultation workshop and identified issues restricting the adoption of lower cost structures including the following.

- Pressures on designers from requirements for professional indemnity insurance, documented best practice and competitive fee bidding
- Clients lacking the knowledge or resources to adequately maintain structures
- Inappropriate allocation of the risks and benefits of innovation
- Difficulties in adequately predicting the performance of rock structures
- Public perception of structures requiring maintenance as having failed

This guidance document was prepared by Matt Crossman, Silvia Segura-Domínguez and Prof. William Allsop of HR Wallingford in association with a Steering Group comprising the following rock structure experts, contractors and owners:

- Dr Philip Barber, Shoreline Management Partnership
- Prof. Andrew Bradbury; Southampton University / New Forest District Council
- Mr Ron Gardner, Boskalis Westminster
- Dr Jentsje van der Meer, Infram
- Mr Michael Owen, Independent Consultant, Defra/Environment Agency Project Manager for this project.
- Mr Will Shields, Dean & Dyball
- Mr Jonathan Simm, HR Wallingford

The project was based on case study analysis of the design, construction and performance of existing rock structures and commenced with an initial review of rock structures around the coast of Britain. This was used to identify candidate structures about which more information could be collected: however, useful information was

found to be difficult to obtain and most of the case studies were constrained by the limited availability of design and / or performance data. As a result the analysis focused on more qualitative aspects than had originally been hoped, this did, however, include consideration of a broad range of associated issues including environmental impact, maintenance construction and safety.

During the course of the study, results came available from a research project on the use of closely packed and optimally shaped rock armour, Stewart et al 2002 and Stewart et al 2003. The results from that research, allowing (perhaps) thinner armour layers and smaller armour sizes, have been incorporated in the guidance presented here.

1.4 Outline and use of this report

This report is intended to provide information and guidance on the use of low cost rock structures in Britain. Chapter 2 provides an introduction to the design and assessment of coastal rock structures, which is followed in Chapter 3 by a summary of existing design guidance and techniques, intended to capture common practice and provide a wider context for coastal rock structures. Chapter 4 discusses cost issues, providing an explanation of the way in which scheme costs are influenced by different factors. The philosophy and drivers behind lower cost structures are explained in Chapter 5 and details of how aspects of such structures may be designed described in Chapter 6. Chapter 7 presents the conclusions from the study.

This document is not intended to replace any aspect of existing design references, but rather to complement them by describing a different approach which may be appropriate in certain situations. The guidance is not intended to be prescriptive and due to the nature of the research cannot be considered comprehensive, but it is hoped that it will advance best practice by providing a checklist of issues to be considered in developing lower cost coastal rock structures.

Much of the data collected, outlines of the analysis spreadsheets used, and a more detailed description of design rules are given by Segura-Domínguez & Allsop (2003) in the project record.

2. DESIGN AND ASSESSMENT

The design and assessment of coastal rock structures requires an understanding of a range of different issues including:

- economics (both scheme costs and benefits)
- technical feasibility and engineering principles
- performance
- safety
- environmental impacts

Established design guidance and techniques are largely based on empirical evidence, including scheme performance and small scale model testing, rather than scientific theories or reasoning. Much of the guidance provides the designer with arbitrary limits on allowable parameters for acceptable performance, but gives no understanding of how performance is changed if the parameter is varied. This guidance has already been well documented, but is summarised in Chapter 3 for convenience and completeness.

Since many of the design decisions for low cost rock structures seek to explore compromises between cost and performance these topics require particular consideration. Cost issues are described in Chapter 4 whilst summary information on performance, environmental and safety issues is provided below.

2.1 Performance and scheme requirements

A clear understanding of performance requirements for the schemes and individual structures is vital if coastal rock structures are to be both efficient and effective. Unconventional structures may not be intended to deliver the same performance as more established (and expensive) designs, but they must enable the overall scheme requirements to be met. This is best understood in a hierarchical way relating to the system of the flood and coast defence which is being delivered. The hierarchy encompasses national goals and high level targets, scheme objectives and functional requirements of individual scheme components as illustrated in Figure 6 below.

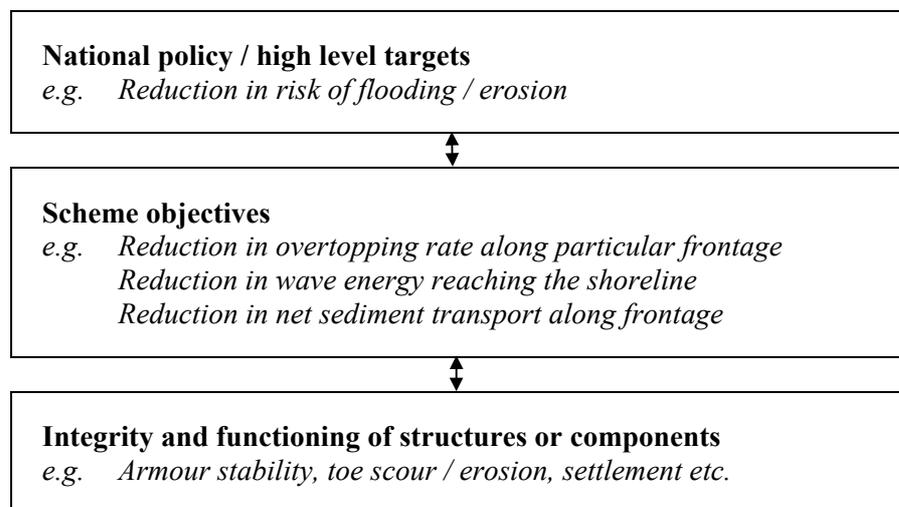


Figure 6 Performance hierarchy for flood and coast defence

It should be noted that performance at each of the different levels is interrelated. The challenge in reducing the cost of structures is to identify more precisely the particular performance that is necessary for the overall scheme to function. This can lead to a considerable relaxation in the performance requirements for individual scheme elements. For example, in the case of a scheme comprising beach nourishment and rock groynes, the main function of the groynes is to retain the beach. They will provide only minimal protection in the event of a short storm, but maintain the beach volume during morphological events. If the beach will need substantial renourishment or reinstatement following significant storm events it may be appropriate to design the rock structures so that they are also reconstructed or repaired at this time, rather than requiring them to be able to last for the whole design life of the scheme without any maintenance or repair.

2.2 Failure and adaptability

Damage of largely homogenous structures is unlikely to be catastrophic; indeed it may be expected to reduce with time if the structure will adopt a progressively more stable form or a protective beach is built up reducing the load on the structure. Since monitoring, maintenance and repair of beach based structures is often relatively simple and inexpensive, there is thought to be considerable potential to remove much of the safety margin normally incorporated in the design of coastal structures. This will of course necessitate arrangements to be made for regular monitoring and appraisal of the structures, but can lead to considerable benefits including an improved understanding of the performance of the structure which may in turn feed into improved design practice.

Such monitoring could also provide the impetus for structures which are not performing in the manner intended, or for which the loading and / or functional requirements have changed, to be adapted to perform in accordance with requirements. Whilst the ability to adjust rock structures is often cited as a major advantage, it is frequently complicated by the presence of multiple gradings of rock and elaborately prepared foundations. The ease with which simpler structures may be adapted and extended to provide the performance required over time is a considerable advantage and discussed further in Chapter 5.

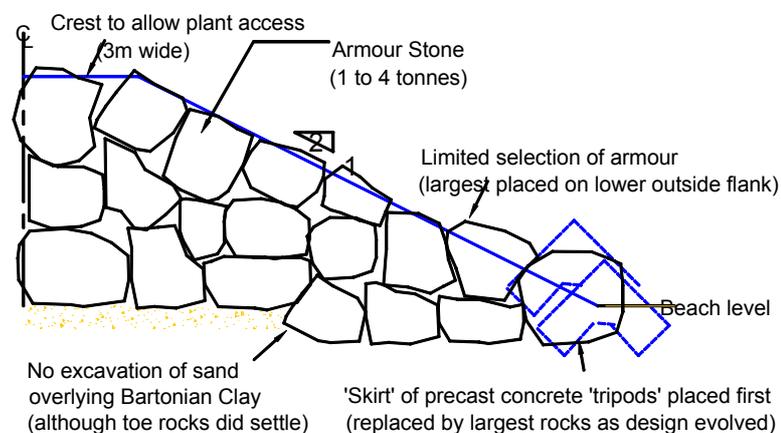
Some of the issues associated with performance requirements are illustrated in Box 1.

Box 1 Rock groynes at Highcliffe, Dorset



Rock groynes were constructed in 1992 to replace life-expired timber groynes at the base of an unconsolidated sandy clay cliff in Christchurch Bay, on top of which Highcliffe Castle is located. In order to minimise the cost of the works whilst maintaining the existing groyne positions and spacing, alternating long (80m) and short (60m) rock groynes were used to retain the required width of shingle beach.

The groynes were designed to make use of locally available Purbeck limestone (of between 1 and 7 tonnes) which is readily available and relatively inexpensive, but does deteriorate in the exposed coastal conditions. The crest width was designed to allow access for construction plant (the rock is delivered by trucks which reverse along the length of the groyne with smaller rocks temporarily placed as kerbing for safety) and the side slopes at 1 in 2 are as steep as was practicable for stability. Concrete tripods were originally used to provide a stable toe, but were found to be unnecessary and selected larger rocks were used as 'toe stones' in structures constructed later. The groynes were built directly on the thin (approximately 30cm thick at the head of the structures) sand beach, but some of the material (particularly the concrete tripods) quickly settled through this onto the underlying clay layer.



Cross section of rock groynes at Highcliffe

The groynes are inspected on a monthly basis and maintenance works (predominantly for public safety) cost approximately 0.5% of the present value of capital works per year (including the importation of limited quantities of rock to compensate for the deterioration). The performance is considered to be appropriate to the location and constraints and a small team of experienced council engineers manages the ongoing monitoring and maintenance programme.

2.3 Environmental impacts

Environmental issues associated with coastal rock structures include visual / landscape impacts, recreational benefits and the provision of marine habitat. The latter is well described in ‘Design criteria for enhancing marine habitats within coastal structures: a feasibility study’ (Halcrow et al 2001) which describes how ecological benefits may be facilitated by:

- Improved shelter from currents and waves
- Inclusion of preferred internal gallery structure
- Increased roughness and range of cracks / fissures within structural units
- Spreading of isolated structure units, set apart from the main reef structure

The report also notes that the promoters of coastal schemes have a duty under the EU Habitats Directive and Planning Policy Guidance Note 9 to actively seek environmental enhancements associated with new structures and schemes. This presents significant potential for the development of multi-purpose structures possibly resulting in additional economic benefits and thus increased justification.

The total structure size and range of internal void spaces are likely to be smaller for lower cost structures than conventional structures, allowing the potential use of a greater portion of a quarry’s yield. Such structure may however be disrupted (damaged and repaired) more frequently, but they should require less disturbance of the beach during initial construction activities and will have a lower impact on the visual landscape and wider environment through the use of less material. Taken overall, it is likely that lower cost rock structures will have a similar (or perhaps smaller) environmental impact when compared with conventional rock structures.

2.4 Safety considerations

In considering safety both the general public and those constructing and maintaining structures must be considered. Public safety of access to various coastal structures was considered in detail in an earlier study, see Halcrow (1996) and Heald (2002). Although public access is seldom a functional requirement for coastal rock structures, designers must be aware that the public are likely to use anything to which they can gain access for amenity (even if signage suggests that this is unwise).



Figure 7 Public use of a rock structure at Lyme Regis for recreation

For coastal rock structures, public safety will be primarily related to access to and on the structure(s). Some hazards are inherent to a greater or lesser degree in all coastal rock structures including:

- potential for falls
- slip / trip hazards
- trapped limbs
- cut / stab hazards
- entrapment in tidally inundated structures
- marooning on shore-detached structures

The adoption of lower cost structures offers a significant opportunity to improve public safety. The use of smaller armour and / or overall structure dimensions together with tighter packing and increased care with which the armour is placed should reduce the effect of falls (since the structures are likely to be smaller) and lead to smaller voids between armour stones reducing the risk of trips, slips and trapped limbs. Public safety could, however, be adversely affected by damage resulting in displaced and / or unstable armour – in such circumstances it may be necessary to restrict access to the structure(s) until repairs are effected. Decreased reliability in the performance of a structure (possibly leading to increased cliff erosion for example) may also threaten public safety during service.

The safety of those building lower cost structures will generally be improved. Since they are built more quickly overall exposure to danger during construction will be reduced. As lower cost structures generally require less foundation preparation there is also likely to be little or no requirement for deep excavation within the beach and the use of fewer armour gradings will mean there is less need to check the levels of layers. The omission of underwater geotextile or complex granular filters requiring diver operations would also substantially increase construction safety.

3. ESTABLISHED DESIGN GUIDANCE AND TECHNIQUES

A variety of guidance documents exist for coastal engineers undertaking the design of coastal rock structures. Documents such as the ‘Rock Manual’ (CIRIA / CUR 1991 and CUR / RWS 1995), Coastal Engineering Manual (USACE 2003) and national / international standards provide advice on the design process and issues. This chapter summarises existing design methods and rules for rock structures. The content covers geometrical design, structural design and material selection. Definition of functional and performance requirements were covered in Chapter 2.

3.1 Geometrical design

3.1.1 Breakwaters – wave transmission

The transmission performance of low-crested breakwaters depends on the structure geometry, principally the crest freeboard (R_c), crest width and water depth, the armour and core permeabilities, and on the wave conditions, wave height (H_s) and period. The CIRIA/CUR (1991) Manual summarised data obtained from hydraulic model tests by Seelig (1980), Ahrens (1987) and Van der Meer (1990):

Table 1 Wave transmission over low-crest breakwaters

| Range of validity | $C_t =$ |
|---------------------------|---------------------|
| $-2.00 < R_c/H_s < -1.13$ | 0.80 |
| $-1.13 < R_c/H_s < 1.2$ | $0.46 - 0.3R_c/H_s$ |
| $1.2 < R_c/H_s < 2.0$ | 0.10 |

A structure under design conditions (with regard to stability) with $R_c/H_s > 1$ will always show transmission coefficients smaller than 0.1. For the range of low wave heights compared to the stone diameter and $R_c/H_s > 1$, Ahrens (1987) gave the following relationship:

$$C_t = 1.0 / (1.0 + X^{0.592})$$

where $X = H_s A_t / L_p D_n^{50^2}$
 A_t = cross-section area of structures
 L_p = local peak wave length

Random wave results are also presented by Powell & Allsop (1985), although without a prediction equation. Careful inspection of this graph reveals that, over the main region of interest, the slope of the wave transmission response can be given by the slope implicit in Table 1:

$$\Delta(C_t) = -0.3 \Delta(R_c/H_s)$$

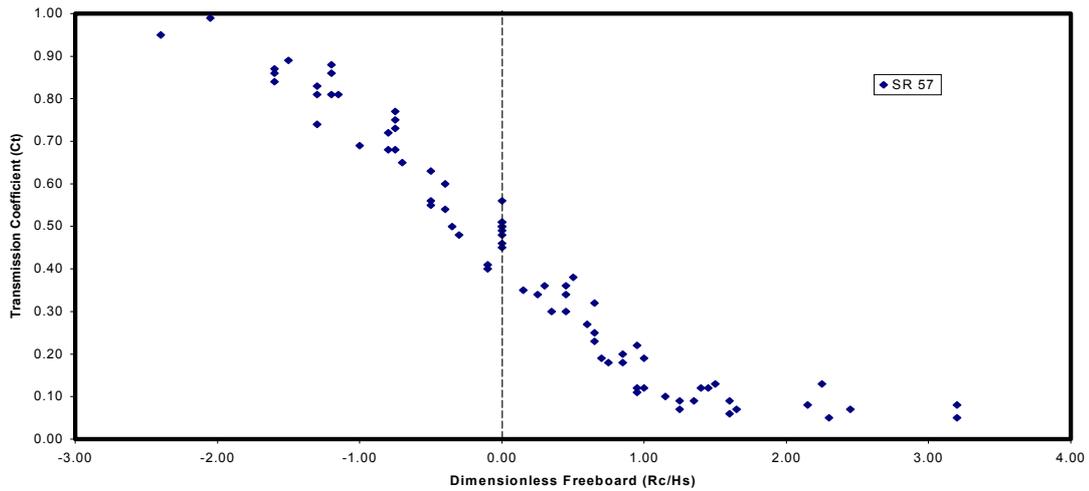


Figure 8 Wave transmission performance for low crest breakwaters, after Powell & Allsop (1985)

It should be noted that the measure of efficiency is the ability of the breakwater to provide shelter in the area behind it, i.e. 100% efficiency corresponds to zero wave transmission.

Design guidance on the use of ‘reef’ breakwaters is also provided in ‘Engineering design guidance for detached breakwaters as shoreline stabilization structures’ (Chasten et al 1993) which cites work by van der Meer (1991). Van der Meer analysed several data sets and assumed a linear relationship between the transmission co-efficient and relative crest height R_c/D_{n50} , resulting in the following formula:

$$C_t = a R_c / D_{n50} + b$$

Where $a = 0.031 H_i / D_{n50} - 0.24$ and

$b = -5.42 s_p + 0.0323 H_i / D_{n50} - 0.0017 (B/D_{n50})^{1.84} + 0.51$ for conventional breakwaters or

$b = -2.6s_p - 0.05 H_i/D_{n50} + 0.85$ for reef breakwaters

The minimum and maximum transmission co-efficients for reef breakwaters are given as 0.15 and 0.6 whilst those for conventional breakwaters are 0.075 and 0.75. The formula is valid for

$$1 < H_i / D_{n50} < 6 \text{ and } 0.01 < s_p < 0.05$$

3.1.2 Revetments - overtopping

In the design of most seawalls and many revetments the controlling response is the overtopping discharge, Q . The estimation of overtopping is described in some detail in the EA manual and ‘Wave overtopping of Seawalls’ (Besley, 1999). This describes a number of methods to calculate both mean overtopping discharge, Q and the maximum

volume likely to overtop, V_{\max} . The report (Besley, 1999) also gives tables showing the two measures of overtopping that are used for different types of performance. The tables provide the different levels of damage that are likely to happen for different amounts of overtopping discharge, for buildings, embankments seawalls and revetment seawalls as follows: for embankment seawalls no damage would be suffered if $Q < 0.002 \text{ m}^3/\text{s}/\text{m}$ and they would suffer damage, even if fully protected, if $Q > 0.05 \text{ m}^3/\text{s}/\text{m}$.

To assess the safety of pedestrians and vehicles the probability of at least one overtopping event occurring during a storm is estimated using the following relationship:

$$P(\text{overtopping}) = 1 - (1 - N_{ow}/N_w)^{N_w}$$

Information on calculation of the ratio of overtopping waves (N_{ow}/N_w) and the maximum overtopping volume is provided in the EA overtopping manual by Besley (1999). Besley (1999) suggests that for pedestrian and vehicle safety on structures with public access the risk of an overtopping event occurring during a sequence of 1000 waves should be less than 1%.

Empirical methods to estimate mean overtopping discharge

Most estimates of overtopping for a particular structure geometry, water level and wave condition are based on empirical equations fitted to hydraulic model test results. A number of different methods have been developed. For plain and bermed smooth slopes Owen (1980) relates a dimensionless discharge parameter, $Q^* = q/(gT_m H_s)$, to a dimensionless freeboard parameter, $R^* = R_c/T_m(gH_s)^{0.5}$, by:

$$Q^* = a \exp(-b R^*/r)$$

This equation is valid for $0.05 < R^* < 0.3$ and values for typical roughness coefficients, r , and for coefficients a and b are given in Besley (1991).

Effect of armour placement on wave overtopping

In most design work, it is assumed that the effect of the armour slope on overtopping can be given by the simple roughness or relative run-up coefficient in Owen's equation. Typical values of r are summarised in Besley (1991).

3.1.3 Groynes – sediment trapping

There is little definitive guidance available regarding the design and performance of groynes. The 'Guide to the use of groynes in coastal engineering' (Fleming, 1990) was based on analysis of a large quantity of prototype data. Physical model studies have also investigated the effectiveness of different types of groynes (Coates 1994, Coates & Lowe 1993, HR Wallingford 1986). The present state of design knowledge is summarised in the Beach Management Manual (Simm et al 1996) and presented in the sections below:

Groyne design for shingle beaches

Bastion groynes were found to be most effective for a given length of groyne, but require a greater volume of material for construction. In order for the groynes to act as a significant barrier to longshore transport under storm conditions the crest should be set

about 1m above the design storm beach profile across the breaker zone. There is little knowledge of the variation in transport rates over groynes with crest height above the beach, but it is suggested that for a wave height of $H_s=2\text{m}$ a crest freeboard of $R_c=0.25\text{m}$ does not provide a significant barrier to drift whilst transport can still take place over a crest 1m above the beach. The berm of the groyne should be set at the same level of the natural berm of the beach, as no significant transport will take place above this level.

It is further suggested that groynes are only effective to the point at which the crest intersects with a depth of approximately $0.75H_b$ below the water level.

Groyne design for sand beaches

The crest level of the groyne should be set at about 1m above the upper limit of the estimated beach profile for the range of design events. An initial estimate of appropriate length may be determined by calculating the position of the breaker zone for a moderate summer swell wave at mean high water level and ensuring that the groyne extends beyond this length.

3.2 Structural design

During the periods up to about 1990, most guidance on coastal structures was taken from the Shore Protection Manual (USACE 1973, 1977, 1984). Since about 1990, substantially better guidance has become available. Even so, design guidance is still sometimes contradictory, and is limited to relatively few structure configurations and responses. In practice, most design calculations or design decisions are based on guidance in the CIRIA/CUR (1991) Manual, the British Standard (BS 6349 Parts 1 and 7), the Revetment manual by McConnell (1998) or the Shore Protection Manual. Some chapters of the future Coastal Engineering Manual (to succeed the Shore Protection Manual) are becoming available during 2001 - 2002.

The main design methods for coastal structures are based on satisfying three primary performance criteria, described in more detail in this chapter:

- Armour stability under direct waves
- Overtopping and crest response
- Separation / filtering at granular layers under hydraulics gradients

3.2.1 Static armour stability

The two most commonly used design methods for determining statically stable main armour sizes are:

- Hudson method
- Van der Meer's equations

These methods focus on calculation of median rock mass, M_{50} , or D_{n50} defined in terms of M_{50} and the rock density ρ_r .

Hudson's method to estimate armour mass has the advantage of simplicity, and wide experience in its use:

$$M_{50} = \rho_r H^3 / K_D \cot \alpha \Delta^3$$

The method was developed from regular waves, so the wave height to be used was assumed to be some maximum wave height. This was later changed to $H = H_{1/10}$, but comparison with other formulae and model test data suggests that this is extreme. Design practice in Europe is generally to use $H = H_s$ if the Hudson formula must be used, although most UK / European designers prefer the van der Meer equations described below.

Values of the Hudson stability coefficient, K_D , in Table 2 correspond to a “no damage” condition where up to 5% of the armour may be displaced. Values of K_D distinguish between breaking and non-breaking wave conditions at the structure (Shore Protection Manual, 1973, 1977, 1984). In this context a breaking wave is defined as one which breaks, due to the foreshore in front of the structure, directly onto the armour layer. The use of K_D does not always best describe the effect of the slope angle. BS 6349 particularly comments that this method is not generally applicable to flat slopes.

Table 2 Values of K_D for use in Hudson equation

| | Non-breaking waves | Breaking waves |
|--------------|--------------------|----------------|
| Rough stone | 4.0 | 2.0 |
| Smooth stone | 2.4 | 1.2 |

Van der Meer’s formulae take account of more variables than are included in Hudson’s formulae including:

- Wave period (within the wave steepness);
- Surf similarity parameter, includes both structure slope and wave steepness;
- Type of wave breaking, given by a distinction between plunging and surging conditions
- Duration of the storm, given by the number of waves, N_z ;
- Permeability of the overall construction to wave penetration;
- Level of damage that may be permitted in the design, $S_d = A_e / D_{n50}^2$
where A_e = cross sectional area of erosion around still-water level

These formulae incorporate the stability number, $H_s / \Delta D_{n50}$, which is used to classify structure types. Statically stable structures usually have a stability number between 1 and 4.

For plunging waves
$$H_s / \Delta D_{n50} = 6.2 P^{0.18} (S_d / \sqrt{N_z})^{0.2} \xi_m^{-0.5}$$

For surging wave
$$H_s / \Delta D_{n50} = 1.0 P^{-0.13} (S_d / \sqrt{N_z})^{0.2} \sqrt{\cot \alpha} \xi_m^P$$

Where α is the angle of the armour slope to the horizontal.

In using these formulae, Van der Meer recommended that the number of waves should not exceed $N_z=7500$, because after this number the structure has more or less reached on equilibrium. In practice, many UK designers limit the use to $N_z= 3000$ to 5000 (BS 6349). The relationships are valid for wave steepness between $s_m=0.005$ and 0.06 and for armour with a mass density between $\rho_r=2000\text{kg/m}^3$ and 3100kg/m^3 .

The suggested values of the damage co-efficient are given in Table 3 below. These values were developed for conventional structures with different layers, which may suffer from catastrophic failure if the core or underlayer is exposed.

Table 3 Design values of S_d for $2D_{n50}$ thick armour layer

| Slope | Initial damage | Intermediate damage | Failure |
|-------|----------------|---------------------|---------|
| 1:1.5 | 2 | 3-5 | 8 |
| 1:2 | 2 | 4-6 | 8 |
| 1:3 | 2 | 6-9 | 12 |
| 1:4 | 3 | 8-12 | 17 |
| 1:6 | 3 | 8-12 | 17 |

The transition from plunging to surging waves is calculated using a critical value of ξ

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

Notional permeability factor P values for various structures are shown Figure 9 (particular structures should be compared with the given structures in order to estimate the P factor).

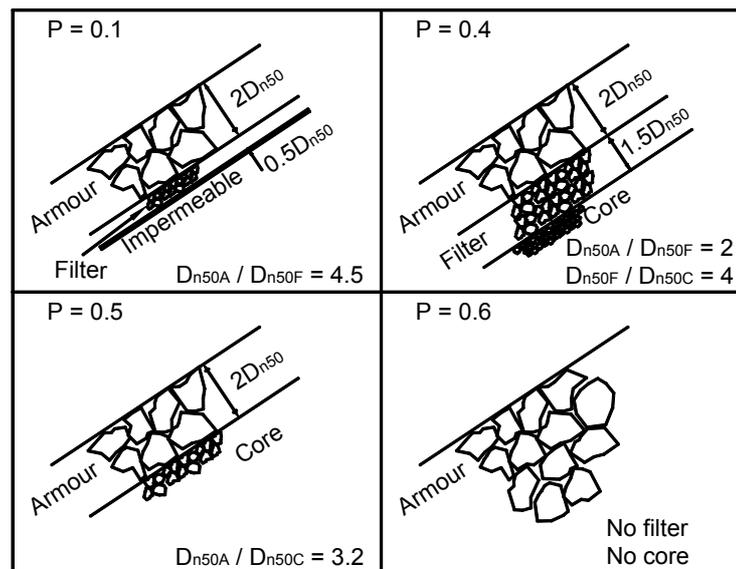


Figure 9 Notional permeability factor (after van der Meer 1988)

The CIRIA/CUR (1991) Manual recommends different parameters for van der Meer's formulae taking account of the effect of depth-limited situations and the influence of grading and shape on stability.

Influence of stone shape and layer thickness on armour stability

The thickness of random placed rock armour should normally be sufficient to contain a double layer of rocks. The CIRIA/CUR (1991) Manual and BS 6349 give formulae to determine armour layer thickness t_a based on rock shape, number of layers, n , volumetric porosity and method of placing:

$$t_a = n k_t D_{n50}$$

where t_a = thickness of the layer
 n = number of layers
 k_t = layer thickness coefficient.

Values of the layer thickness coefficient are given in the CIRIA / CUR (1991) Manual. The coefficient depends of the shape of the rock and the method of placement (special or random). In general terms, for the same rock shape, the coefficient varies significantly (around 40%-60%) depending on the placement method, being smaller when the rocks are placed without any special specification (random placement). On other hand, the layer coefficient suffers little variation (around 10%) depending on the shape of the rock, that can be irregular, semi-round, equant or very round.

Later research by Bradbury et al (1988) studied stability of different armour shapes with a narrow grading of $D_{85}/D_{15} = 1.25$ and with individual placement of the armour blocks to give two layers. This placement gave armour layer thicknesses of $t_a = 1.5-1.7 D_{n50}$.

That study gave higher damage than predicted by van der Meer for shape classes of material of similar shape characteristic. Further tests yielded adaptations of van der Meer's equations for different armour shapes with the introduction of two shape coefficients, C_{pl} and C_{su} into equations for:

plunging waves $H_s / \Delta D_{n50} = C_{pl} P^{0.18} (S_d/\sqrt{N})^{0.2} \xi_m^{-0.5}$

surging waves $H_s / \Delta D_{n50} = C_{su} P^{-0.13} (S_d/\sqrt{N})^{0.2} \sqrt{\cot\alpha} \xi_m^P$

The values of the shape co-efficients are provided in Table 4.

Table 4 Rock shape coefficients.

| Shape |  |  |  |  |  |
|-------------|---|---|--|---|---|
| Description | Elongate or tabular | Irregular or fresh | Equant | Semi-round | Very round |
| C_{pl} | 6.72 | 6.32 | 6.24 | 5.96 | 5.88 |
| C_{su} | 1.30 | 0.81 | 1.09 | 0.99 | 0.81 |

For conditions where the size (D_n) or shape of the armour change, and/or the layer thickness (t_a) reduces, these methods can be used to calculate any increase in damage S_d arising from changes to the rock armour shape, or from loss of layer thickness.

Influence of low-crest sections on armourstone size

Lower crest levels may allow increased wave overtopping, reducing the chance of damage to the armour on the front face, but increasing (potential) damage on the crest and rear face. The CIRIA/CUR manual gives a simple correction factor, f_i , to be applied to the size of the armour on the front face when described by its nominal diameter, D_n :

$$f_i = 1 / (1.25 - 4.8R_p^*)$$

Which is valid over the range $0 < R_p^* < 0.052$ where $R_p^* = (R_c/H_s)(s_p/2\pi)^{0.5}$

Sizing of crest and rear armour for stability

There is little guidance to relate rear face armour size to overtopping. In preliminary design, it may be assumed that armour on the crest or rear face should not be smaller than the main armour size. Structures that are high enough to limit overtopping can however use smaller crest and rear armour than on the front slope. Pilarczyk (1990) is cited in the CIRIA/CUR (1991) Manual and recommends a simple method to estimate the rear face and crest rock armour size:

$$H_s/\Delta D_{n50} = 2.25 \cos\alpha_i \xi p^{-0.50}/[1-(R_c/R_{u2\%})]$$

where α_i is the slope angle on the lee side over the length requiring protection, L_s ,

$$L_s = 0.2 \Psi T_p \sqrt{[g(R_{u2\%}-R_c)]} \geq L_{\min}$$

where

Ψ = factor related to the importance of the structure and its function ($\Psi \geq 1$)
 L_{\min} , the practical minimum width protection is equal to about $3D_{n50}$, also considered as the minimum transition length from slope to crest.

An alternative approach (which shows rather different trends than that given above) might be to calculate rear armour sizes subject to overtopping flows using a method ascribed to Klein-Breteler & Pilarczyk (1998). This approach uses an indicative flow rate (q_{\max}) and slope angle (α_r) to calculate a rear slope nominal armour size, D_n using the fractional density $\Delta = (\rho_r / \rho_w) - 1$:

$$\Delta D_n = (\sin \alpha_r)^A (q_{\max}^2 / (g B^2))^{1/3}$$

where $A = 0.78$ and $B = 0.11$ for rock armouring.

3.2.2 Toe Armour

Most front-face armour layers are supported by toe armour. It is assumed that if toe armour has the same dimensions as the main armour, then the toe will be stable. In most cases, however, the designer will wish to reduce the size of the armour in the toe.

Van der Meer et al (1998) presents a method based on tests which concluded that toe berm stability is affected by wave height, water height at the top of the toe berm, width of the toe berm, and block density. Wave steepness did not appear to be critical. Toe armour size is given by an improved formula where the toe depth was given as h_t/D_{n50} .

$$H_s/\Delta D_{n50} * N_{od}^{-0.15} = 2 + 6.2 (h_t/h)^{2.7}$$

where N_{od} is the damage level, actual number of displaced stones related to a width, along the longitudinal axis of the structure, of one nominal diameter.

For “standard” toe size of about 3-5 stones wide and 2-3 stones high:

- $N_{od} = 0.5$, start of damage
- $N_{od} = 2.0$, some flattening out
- $N_{od} = 4.0$, damage

This method can be used in the range: $0.4 < h_t/h_m < 0.9$ and $3 < h_t/D_{n50} < 25$

For wide toes, it may be possible to accept higher damage levels.

3.2.3 Underlayers and filters

Rock armoured structures in coastal and shoreline protection are normally constructed with an armour layer or layers, and with one or more underlayers, one of which may be termed the filter. In some instances, an underlayer will be required to act as a filter.

Filter rules

Granular filter are frequently applied in civil engineering. Filters rules for irregular-shaped particles developed originally by Terzaghi are cited in the CIR/CURIA Rock Manual:

To prevent migration

For uniformly graded materials

$$D_{50f}/D_{50b} < 5$$

For wide graded materials

both $D_{15f}/D_{85b} < 5$

and $5 < D_{50f}/D_{50b} < 20-60$

To ensure adequate permeability

$$D_{20f}/D_{20b} > 5$$

where the sub-script f denotes filter sizes, and sub-script b denotes base material, core or foundation as appropriate.

Terzaghi’s general filter rules are then qualified by a number of other texts. The Shore Protection Manual (1984) recommends an underlayer stone size (mass) of 1/10 to 1/15 of the armour mass. This criterion is regarded as stricter than the geotechnical filter rules, as for narrow gradings it would give $D_{n50armour}/D_{50filter} < 2.1-2.5$, or with realistic gradings $D_{n50armour}/D_{85filter} < 4-5$.

BS 6349 recommends a modified version of Terzaghi’s filter criteria.

$$D_{15f} / D_{85b} \leq 4 \text{ to } 5$$

$$4 < D_{15f} / D_{15b} < 20-25$$

For detached breakwaters where armour is placed direct on a bedding layer, Chasten et al (1993) give two methods to select the size of bedding stone or filter in relation to the armour given:

$$D_{50a} / D_{50b} \leq 4 \text{ to } 5 \quad \text{based on guidance after van der Meer \& Pilarczyk, (1987)}$$

or

$$D_{15a} / D_{85b} \leq 4 \text{ based on Ahrens (1975), for which } D_{n50a} / D_{n50b} \leq 6.8$$

Effect of gradation

Similar criteria for rip-rap or wider graded protection are expressed:

$$\begin{aligned} D_{15f} / D_{85b} &\leq 4 \\ D_{50f} / D_{85b} &\leq 7 \\ D_{15f} / D_{85b} &\leq 7 \end{aligned}$$

Van der Meer (1988) suggests that a filter with moderate grading ($D_{85} / D_{15} = 2.5$) will be as stable as a narrow grading if well mixed and the armour is suitable. Thanikachalam & Sakthivadivel (1974) cited in the Rock Manual suggest less strict filter rules which take account of gradation given by ratios D_{60b}/D_{10b} for the base material, and D_{60f}/D_{10f} for the filter:

$$\begin{aligned} D_{10f}/D_{10b} &< 2.50 D_{60b}/D_{10b} + 5.00 \\ D_{60f}/D_{10f} &< 0.94 D_{10f}/D_{10b} - 5.65 \\ D_{50f}/D_{50b} &< 2.41 D_{60f}/D_{10f} + 8.00 \end{aligned}$$

The Rock Manual does not reveal whether these rules have been tested under strongly reversing conditions.

Effect of hydraulic loading

Pilarczyk (1984) suggests additional guidance which is based on severe hydraulic loadings. He defines $N_f = n_f D_{15f} / D_{50b}$, where n_f = filter porosity. Pilarczyk recommends that N_f should ideally be less than 1, and must not exceed 5. The failure modes for filters are illustrated in Figure 10.



Figure 10 Filter performance / failure modes (after Pilarczyk 1984)

A monogram for filter stability under hydraulic conditions is shown in the CUR/CIRIA (1991) Rock Manual, for filters at least five times D_{50f} thick (in order to let the gradients be a realistic average). Using this monogram and following from parallel grading, slope steepness, porosity and grain size of the base material one obtains the required filter material.

Under cyclical loading caused by wave action it is probable that reversal of flow within a filter layer will cause some disturbance of the finer material and possible migration through the overlying material.

Suffusion

The processes of fine material being washed out of a mixture (usually gap-graded) is termed suffusion, and whilst differing from filter failure, follows many similar processes.

Kovacs (1981) suggested a very simplified rule for a single grading based on the uniformity coefficient $U_c = D_{60} / D_{10}$:

| | |
|--------------------|---|
| $U_c < 10$ | No suffusion |
| $10 \leq U_c < 20$ | Transition, suffusion depends on hydraulic gradient |
| $20 < U_c$ | Suffusion is probable |

This is however too simple for many cases, and fails to identify the onset of the process. Rather better guidance, although more complicated, is discussed by Allsop et al (1990, 1991) based on work by Kenney & Lau (1985). They start from the premise that a small particle of diameter D_f can pass through the constrictions in a coarser filter formed by particles of size $= 4D_f$. This hypothesis is used to analyse gradings to test whether any significant fraction shows such a size difference. The gradient of the size grading is calculated over a size ratio of 4. An F-H diagram is plotted over the range of sizes in the grading for F, the fraction smaller than D, and for H, the fraction with sizes between D and 4D. Kenney & Lau argue that stability can be estimated from:

| | |
|----------------------|--|
| $1.3 F < H$ | Stable, no suffusion |
| $1.3F \geq H > 0.6F$ | Semi-stable, suffusion depends on hydraulic gradient |
| $H \leq 0.6F$ | Unstable, suffusion is probable |

This does not however indicate the levels of hydraulic gradients at which this process might start. Using data from filter box tests at Delft Geotechnics, den Adel et al (1988) suggest a tentative relationship between the minimum value derived from the H/F curve, $(H/F)_{\min}$, and the critical hydraulic gradient for onset of suffusion:

$$i_{cr} = 0.5 (H/F)_{\min}$$

Tests reported by Allsop & Williams (1991) and by Allsop & Shih (1990) suggest however that significant suffusion was observed for gradients of about half that predicted by den Adel, suggesting that a safer limit might be given by:

$$i_{cr} = 0.25 (H/F)_{\min}$$

3.3 Rock Materials

3.3.1 Rock properties

In service, rock armour must retain its original size / mass and shape. The effect on armour layer stability arising from reductions in armour size, or perhaps changing armour shape, might be predicted by methods in Section 3.2.1. To avoid those changes, armour must resist abrasion, solution, and atmospheric weathering. Some rock types are potentially more suitable for armour than others. Some rock types with lower

resistance may be used in underlayers or core where direct abrasion processes are reduced. These requirements may be summarised as follows:

Armour layers and facing

- High strength
- Durability characteristics
- Stringent geometric

Underlayers and filters

- Dissipate wave energy by turbulent flow thorough void spaces
- Provide structural support for armour against direct wave and geotechnical forces
- Prevent hydraulic removal of particles from the lower layers

Core materials

- Provide structural support for armour / underlayers against geotechnical forces and (indirect) wave forces

The selection of appropriate rock materials for use in a marine structure, whether from extension of an existing quarry, from development of a new quarry, or from a commercial supply source, must be made on assessment of the rock's physical properties, geological considerations at the quarry, and the requirements for durability of the rock in service. A set of idealised rock properties are given in the CUR/CIRIA (1991) Manual. The idealised properties given are rock density, porosity, water absorption, discontinuity spacing, weathering grade, uniaxial compressive strength and seismic rock quality designation, for armour, underlayers and core/fill. Other tests may be necessary to explore resistance to some weathering processes.

Water absorption of a rock material gives the most important single indication of in-service durability. The value of water absorption for armour rock should be not more than 3%, but if greater than 0.5% then the provisional European Standard (prEN 13383-2:2001) recommends that loss of mass in the Magnesium sulphate soundness test $\leq 25\%$. Where freeze-thaw is significant, tests must show $< 0.5\%$ loss of mass or formation of open cracks.

In reality, the sources of rock available will not always conform fully to those ideal limits, and some structures do not always require rock of the greatest resistance. It is sometimes therefore necessary to categorise rock by potential durability, as in Table 5.

Table 5 Guide to rock durability (after CIRIA/CUR 1991)

| Test | Excellent | Good | Marginal | Poor | Comments |
|--|-----------|-------------|-------------|---------|--|
| Rock density (oven dry) (t/m^3) | > 2.9 | 2.6-2.9 | 2.3-2.6 | < 2.3 | Physical property affecting hydraulic stability. Good indicator of durability except dense but weathered basic rocks. |
| Water absorption (% by mass) | < 0.5 | 0.5-2.0 | 2.0-6.0 | > 6.0 | Most important indicator for weathering. Can be misleading for porous limestone with free-draining pores. |
| Magnesium sulphate soundness BS812/BS 6349 (%) | < 2 | 2-12 | 12-30 | > 30 | Resistance to weathering. Important for porous sedimentary rocks in hot dry climate. Good correlation with water absorption. |
| Freeze/Thaw FT (%) | < 0.1 | 0.1-0.5 | 0.5-2.0 | > 2.0 | Important test for freezing winter climates. Good correlation with water absorption. |
| Methylene blue absorption (g/100 g) | < 0.4 | 0.4-0.7 | 0.7-1.0 | > 1.0 | Indicate presence of deleterious clay minerals |
| Fracture toughness ($Mpam^{1/2}$) | > 2.2 | 1.4-2.2 | 0.8-1.4 | < 0.8 | Indicates resistance to type 2 breakage. Good correlation with abrasion resistance. Can mislead for impact resistance of large blocks. |
| Point load index (MPa) | > 8.0 | 4.0-8.0 | 1.5-4.0 | < 1.5 | Indicates resistance to type 2 breakage. Quick, but high variability. Can mislead for impact resistance of large blocks. |
| Wet dynamic crushing value (%) | < 12 | 12-20 | 20-30 | > 30 | Indicates resistance to type 2 breakage. Quick test |
| Mill abrasion Resistance (loss/1000 revs) | < 0.002 | 0.002-0.004 | 0.004-0.015 | > 0.015 | Indicates resistance to abrasion by mutual grinding of saturated rock surface. |
| Block integrity drop test I_d (%) | < 2 | 2-5 | 5-15 | > 15 | Indicates resistance to type 1 breakage of large blocks. |

The form of the rock properties presented above has been superseded by BS EN 13383 which specifies requirements for aggregates for use in armourstone that differ from those found in the CIRIA/CUR (1991) manual on the use of rock in coastal and shoreline engineering. The Published Document PD 6682-7:2003 giving guidance of the use of the BS EN 13383 will be published shortly after the publication of this practical design guidance.

3.3.2 Colour

Other aspects of the rock will also influence its acceptability, and performance in service. Rock colour, despite changing significantly once installed, may be judged to be important in the acceptability of the scheme. Aesthetic requirements in amenity areas can therefore lead to demands for particular rock colour, and this may modify the rock type that would be chosen with reference to durability properties. It should however be noted that weathering, abrasion by beach material, and/or biological colonisation will rapidly alter the intertidal zone appearance. It is also noted that colour is unlikely to be important for the lower armour or underlayers.

3.3.3 Grading

Wide graded sizes are not encouraged for armour where smaller sizes could more easily be removed by direct wave action. Wider gradings may however be more useful in core or underlayers where an increase in grading width permits easier sourcing, and improves filter performance. Particle weight distributions are conveniently presented by cumulative weight or diameter curves. Table 6 shows ranges recommended for grading widths:

Table 6 Suggested grading ranges (after CIRIA/CUR 1991)

| | $D_{85}/D_{15} = (W_{85}/W_{15})^{1/3}$ | W_{85}/W_{15} |
|-------------------------------------|---|-----------------|
| Narrow or 'single-sized' gradation | < 1.5 | 1.7 - 2.7 |
| Wide gradation | 1.5 - 2.5 | 2.7 - 16 |
| Very wide or 'quarry run' gradation | 2.5 - 5.0+ | 16 - 125+ |

3.3.4 Construction induced properties

Armour stability and hydraulic responses depend on placement techniques. The placement density, thickness and surface smoothness / irregularity result from selection of armour unit, and effort taken in packing the armour layer. Some designers require a minimum number of point contacts that each block makes with its neighbours, often 3, sometimes 5. Armour layer porosity normally lies within the range 25-40% and the use a measured trial panel (as described in CIRIA / CUR 1991) will provide a means of resolving potential disputes over porosity values and a reference for individual projects.

4. UNDERSTANDING COST ISSUES

The cost of coastal rock structures is dependent on a large number of factors, only some of which will be within the influence of engineers. Since the importance of different factors will vary with location and timing of works it is not possible to quantify the impact of different issues on cost within this document. Instead this text is intended to inform designers about the issues and prompt consideration of costs during the design development for individual projects.

It is important when considering costs that there is a good understanding of the working methods likely to be adopted and the influence of different issues on construction. This may be best developed by a partnership approach, involving the contractor from an early stage in the scheme development process. Indeed in many cases it is likely that the full benefits of lower cost structures will only be achieved by establishing a partnership approach between the client, designers and contractors so that savings that are difficult to quantify on paper can be realised and lessons learnt feed back into the design process.

The cost of rock structures can be broadly separated into the following four categories each of which are discussed in more detail below.

- Design and assessment
- Rock armour
- Construction
- Monitoring, maintenance and repair

4.1 Design and assessment

These costs will include analysis to determine the appropriate performance and function, geometry and structural arrangement and are also likely to include at least some modelling (numerical and / or physical) to verify and optimise aspects of the design. This stage may also include preparation of contract documents and administration of the tender process.

Early decisions concerning the function, performance, type and arrangement of structures can have significant impacts on the overall cost of the structures. Similarly, unless the potential for lower cost rock structures is identified at an early stage in the project an alternative option may be taken forward. The barriers to the wider implementation of innovative rock structures identified in an earlier study (HR Wallingford 2000) included several issues relating to the motivation and ability of designers to innovate. It must be recognised that lower cost structures may necessitate higher design costs, or at least an equitable apportionment of risks and benefits.

Additional design and assessment costs can contribute to increased confidence in the performance of the structure and may enable total costs to be optimised, however for smaller schemes where lower cost structures are most likely to be appropriate, the costs of detailed mathematical or physical modelling can quickly exceed any resulting savings. In these situations, providing the original concept is sound, the adoption of adaptable structures able to be developed through a process of trial and refinement is likely to be more appropriate. In developing coastal rock structures experience and a

good understanding of the physical processes at the site are invaluable. They may be relatively inexpensively acquired by local coastal managers over a number of years, but will be much more expensive to quantify and document over the short timescales in which engineering schemes are normally developed.

4.2 Rock armour

The cost of supplying rock armour is normally included in the construction contract, but since it forms a large proportion of the total scheme costs it is appropriate to consider it separately. Although principally dependent on the accessibility of the site and distance to the quarry, the cost of rock, like that of any other commodity, is subject to market forces. If there is considerable demand (e.g. due to a number of coastal defence schemes being constructed simultaneously) there will be a degree of competition for both the available rock and means of delivering it to site, which is likely to result in higher costs and less reliable delivery schedules.

In most instances assessment of the cost and availability of suitable rock forms one of the initial scheme development activities and depending on the results the structure is often designed with a particular source of material in mind. This can present significant problems if the properties are later found to be different (for example the maximum armour size is much less than anticipated) or the source becomes uneconomic. It is therefore critical that the process is iterative and that potential sources of material are reviewed prior to any major commitment of expenditure.

4.2.1 Sources and quality of rock

Rock armour for coastal defence projects in Britain is typically required in relatively small quantities and as a result is seldom supplied from a dedicated quarry, but rather a by-product of other quarrying activities such as aggregate production. The cost of rock at the quarry floor is of the order of one tenth of the price paid for delivery of rock to site with the differential being a reflection of the effort required to sort, store and transport the rock to site. The cost will also be influenced by the proportion of the rock produced that can be used for armour. This is often of the order of 10-15% of the total quarry yield, although the grading achieved from a particular location will be dependent on geological issues and the method of production. Unless there is also a market for the rest of the yield (such as roadstone or concrete), there may be considerable logistical difficulties and cost implications. This is illustrated in Figure 11, which shows a theoretical fragmentation curve where 1-3t constitutes only 7% of the yield, and even 0.5-3t is less than 16%. Depending on the demand for the different gradings produced it is often appropriate to adopt wide gradings of armour to make best use of the quarry yield.

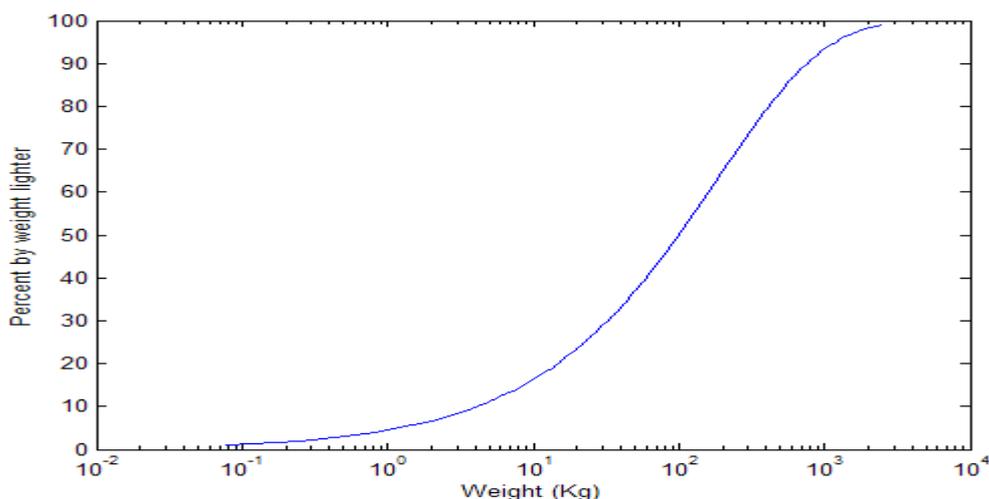


Figure 11 Rosin-Rammler theoretical equation for stone gradation and fragmentation.

Since the rock used for coastal schemes is commonly a by-product of quarrying for construction materials, its availability and cost is variable depending on how much is stockpiled at a particular quarry at a certain time. In most parts of Britain rock of varying size and quality is available from a number of different sources. In selecting the source of armour it is sensible to consider quality of the material and the grading(s) available as well as the cost for a number of different sources, an example is shown in Table 7 below.

Table 7 Potential sources of rock armour for south coast scheme

| Quarry location | Dorset | Cornwall | Somerset | France | Norway |
|------------------------------|-----------|----------|-----------|-----------|---------|
| Rock type | Limestone | Granite | Limestone | Limestone | Syenite |
| Max size (T) | 10-15 | 6-10 | 3-6 | 10-15 | 20 + |
| Density (Mg/m ³) | 2.6 | 2.7 | 2.8 | 2.7 | 2.8 |
| Soundness | ? | ✓ | ✓ | ✓ | ✓ |

Armourstone provided from local quarries, which can supply rock with small transport distances, is likely to be relatively inexpensive. However, such quarries may only be able to supply a limited grading or poor quality material, and may experience difficulty in producing material at the rate required for economical construction. Nevertheless such material may be advantageous from an environmental perspective (particularly if it is similar to native beach materials). It may be possible to design the structure, using procedures described in Chapter 6 to facilitate the use of such material.

4.2.2 Rock gradings, underlayers and filters

Large breakwaters often utilise dedicated quarries and structure cross sections are optimised to make best use of quarry yield. However, rock used in coastal defence structures is often a by-product of aggregate (normally <50mm) production. The availability of some classes of rock commonly specified for under- or filter-layers is very restricted because material between approximately 50kg and 500kg is commonly crushed for aggregate production. Where it is available, the additional processing required (particularly if any quality control is applied) generally outweighs the savings

in handling and transport, thus underlayers may be at least as difficult to produce, and as expensive as the primary armour rock.

The Rock Manual (CIRIA / CUR 1991) established a series of standard gradings which greatly eased the identification and procurement of armour for many coastal rock projects. Whilst these provide a useful starting point they may need to be adjusted to match the size of rock required or available for particular projects. It should be noted that whilst the rock manual standard gradings are wider than those within the Shore Protection Manual they are comparable to those presented in the Coastal Engineering Manual and may be extended provided the structure slopes are not too steep. Where a local quarry is being used to supply armour it may be appropriate to use a wider grading than would normally be the case, so as to accommodate a greater proportion of the quarry yield.

4.2.3 Delivery methods

Constraints on the method of delivering rock to site can have a significant impact on the cost of the material, the rate at which it can be delivered and the source that it is obtained from. Over the last 10 or 15 years most rock used in coastal defence schemes in Britain has been delivered to site by sea since this is relatively economic for large volumes and is often less intrusive on the local environment than road transport. Rail transport has also been used to a limited extent, but is unlikely to be feasible for smaller, lower cost projects.

The selection of delivery method should be carefully considered prior to any constraint being imposed on the contractor. The advantages and disadvantages of the two principal means of transport are discussed below, and in practice they may be combined, with road transport used to provide the final volume of material required to complete a structure or scheme after the bulk was delivered by sea.

Table 8 Transport methods

| | Road | Sea |
|---------------|---|---|
| Advantages | <ul style="list-style-type: none"> considerable flexibility where the volume of rock is not precisely defined or additional supplies are required | <ul style="list-style-type: none"> often cheaper and less environmentally damaging for large quantities of materials wider range of sources of supply |
| Disadvantages | <ul style="list-style-type: none"> may be difficulties in achieving a rate of delivery costs may be excessive where the site is far from the quarry | <ul style="list-style-type: none"> often limited to (single or multiple) large shipments may be delayed by severe weather |

Alternative designs were produced for a rock revetment at Barton on Sea allowing for two different grades of material, but comparable long-term performance. One design allowed for static stability, using a high-grade armourstone (Mendip Limestone). The other utilised a lower grade of material (Portland Stone) and was designed to allow for degradation of the armourstone, whilst maintaining stability. This was achieved by building the structure to a flatter slope. The tender process consistently provided a cost saving resulting from use of the high-grade material, on this occasion. This approach is mainly sensitive to the proximity of armourstone quarries to the site.

4.3 Construction

Cost will also be influenced by the approach taken to construction and the plant and equipment available. Construction activities usually include stockpiling of armour delivered to site, excavation and preparation of foundations (including supply and placement of geotextiles if required) re-handling and placement of rock armour within the structure and maintenance until contractual handover. For all but the smallest projects they are also likely to include establishment of appropriate site facilities, administration / supervision and overheads such as insurance.

The cost of construction activities is directly related to the volume of the structure, but other important issues include whether the structure can be built using land based plant, the size of armour and dimensions of the structure, the requirements for placing armour and the complexity of the structural cross section. Risk has a significant role in construction activities and contractors rates will include components for any risk they perceive to be significant. The following sections describe some of the most significant issues associated with construction costs.

4.3.1 Access to the structure

The location of the structure on the beach can have a significant impact on the method and ease of construction. Where the structure is founded considerably below mean low water springs (MLWS) marine plant may need to be used (although alternatives include end tipping from a causeway, or the use of land plant adapted to operate in up to 1 or 1.5m of water). As soon as the foundation level approaches MLWS the use of land based plant becomes more favoured as it is often significantly less expensive than marine plant. Even where construction must take place below water, the constructor will endeavour to conduct construction using land based plant. This approach reduces risks and construction costs. However, at levels approaching MLWS working periods will be limited to a short duration, possibly available on only a few tides per month. If bad weather or a surge prevents a critical activity the whole construction project may be delayed.

The toe of structures are often most difficult to construct, since they are often designed with at least some excavation to prevent undermining through scour. This excavation can be particularly difficult and dangerous, often filling with water and collapsing during the limited working window. In these and marine situations the problems and safety implications associated with placing a geotextile layer, which is relatively easy and cheap in the dry, may mean that it cannot be justified and even where it is placed it may be damaged or otherwise compromised.

Some access problems may be overcome by providing for construction from or over parts of the structure itself. This can be particularly helpful when access on the beach is difficult (either due to beach materials or short tidal windows) or may cause excessive damage to the substrate. Access may be provided over the (often partially completed) crest of the structure by 'blinding' the surface with concrete or smaller rock, however, this needs to be limited to avoid total cover of underlayer stone, which would affect structure performance. The ability of plant to track over armourstone depends upon shape as well as weight, whilst most plant will have difficulty moving over a layer of

rock much larger than approximately 1 tonne, some tracked plant can manoeuvre (albeit with difficulty) over armour as large as 3-6 tonnes. Alternatively any filter or bedding layer may be extended beyond the base of the structure to form an access route and, following construction, provide protection against scour.

Marine plant access must also be considered as vessels may be damaged by rocky outcrops or structures on the beach. Where barges are used to deliver rock to the site care must be taken to ensure that the beaching location is suitable as the barge may be seriously damaged if it is not evenly supported.

4.3.2 Armour placement

Plant requirements (and costs) are determined by the armour size and structure geometry. In general land based plant capable of handling individual rocks of up to about 8 tonnes at a reasonable radius is readily available, but as the mass of rock or radius is increased, more specialised (and expensive) plant is required. A similar threshold exists for marine plant at about 12 tonnes, although it is less pronounced. Packing requirements will influence both the rate of production and quantity of rock armour required for the structure.

Many of the risks associated with rock placing have traditionally been borne by contractors and included in their rates for the works. These risks are well described by Simm & Cruickshank (1998) which provided a commentary and possible control strategies for the following:

- General rock placing and buildability
- Availability of plant
- Stone variability and void ratio
- Ability to achieve 'payment' profile
- Experience and flexibility of the client's representative on site
- Variations in the formation levels
- Stability of the partially completed works
- Settlement
- Wastage due to breakage
- Core material loss
- Barge beaching risks
- Damage to vessels
- Public safety
- Exchange rate risk

Many of these risks will not be significantly improved by the adoption of unconventional structures, however some will be substantially removed, such as the loss of core materials, stability of partially completed works and general buildability. Others, such as settlement may be increased and will need to be shared equitably between the different parties. Different measures will be needed to address risks such as the void ratio – in this example, payment by weight of rock rather than volume of construction more accurately reflects the contractor's costs and may allow the client's representative to achieve the desired packing required without penalising the contractor excessively.

Marine plant is generally much more expensive than land-based plant with considerable mobilisation periods and costs. It is thus unlikely that the high level of maintenance required for lower cost land-based structures will be economic with maritime plant. There may be scope for the increased use of nearshore dynamically stable structures, however. These have been developed by the US Army Corps of Engineers (see Chasten et al 1993) along broadly similar lines to Scandinavian berm breakwaters. They are generally low crested and designed to accommodate a degree of armour movement and settlement. To date such structures have only been constructed in the USA with land-based plant, however there is thought to be scope for the bulk placement of bedding layers and armour around or below MLWS using specialised side stone dumping vessels, sometimes used for the transport of rock to beaches.

4.3.3 Timing of construction

Coastal defences are often constructed on or adjacent to popular tourist beaches and many promoting authorities are understandably reluctant to undertake works at these locations during the popular summer months for public safety and economic reasons. However, requirements for winter working are likely to increase the risk of downtime and delays due to severe weather. This factor becomes more significant if the rock is being delivered to site by sea or construction utilises marine plant.

Such downtime is often quantified in the preparation of construction tenders and much of the risk is incorporated into the construction costs (although some forms of contract make allowance for the contractor to be compensated in the most severe events). Winter working can also increase significantly construction safety hazards, which may outweigh any decrease in public safety due to summer construction. More information on the risks associated with the timing of rock works is provided by Simm & Cruickshank (1998) in the form of graphs showing the percentage exceedence for various wave heights at different locations around the coast of the UK. These show that for the south and east coasts the percentage time exceedence for a significant wave height 1m is approximately 30% during the summer months (June – August) and 80% during winter (December – February).

There is obviously a need not to have ‘closed’ seasons for coastal rock construction (this would increase costs by limiting the utilisation of plant and equipment) but designers should consider the implications of the timing of works carefully and it may be appropriate to provide a generous contract period for works taking place over the winter or continuity of works over a number of years so that there is an opportunity for downtime to be ‘averaged out’.

4.4 Monitoring, maintenance and repair

Monitoring maintenance and repair activities take place after the initial construction, but are required to maintain the integrity and performance of the structure. Many conventional structures are designed using existing guidance such that there is a low probability of damage during the scheme life. Although this results in structures with relatively large capital costs, little or no allowance is made for monitoring, maintenance or repair of the structures. Conversely lower cost structures often require regular monitoring and periodic maintenance / repair.

Costs associated with monitoring and maintenance are highly variable depending on the nature of the structure and the ease of access. Where the structure is a component of a wider system with easy access and it performs similarly to other aspects of the system (such as a rock groyne suffering damage only during events, which also require the beach to be reinstated) the additional costs of the structure will be limited. Where the structure is isolated and detailed surveys or special maintenance are required the costs are likely to be much more significant.

4.5 Programming expenditure

To allow direct comparison between the costs and benefits of coastal rock structures with other scheme options future values are discounted to present values as described within the official guidance (MAFF 1999). In order that an appropriate decision can be made it is important that adequate allowance is made for all costs over an appropriate scheme life. Such costs are unlikely to be limited to the rock structure itself (which would include capital works, maintenance and repair as well as any decommissioning costs and / or residual values), but will also be related to the performance of the structure since other scheme elements (such as the provision of beach nourishment or recycling) will also need to be included in the cost estimate.

The effect of discounting is to reduce the value of future benefits and expenditure, it is thus important that works are appropriately timed to maintain the standard of defence, but it will seldom be appropriate to increase the standard of defence (with respect to sea level rise for example) substantially before it can be justified by increased benefits. Due to the discount rates used in economic analysis, schemes with lower capital cost and significant maintenance during latter years, are often of economic benefit.

5. LOWER COST ROCK STRUCTURES

Lower cost rock structures arise from a reconsideration of the balance between performance, capital and maintenance costs. The approach normally results in structures which have lower whole life costs, reduced environmental impacts and are more easily adaptable for changing situations or requirements. This chapter describes adjustments to conventional performance requirements which may lead to a reduction in the cost of rock structures.

There are four fundamental ways in which the cost of coastal rock structures can be reduced:

- Using less rock armour, including adaptable construction
- Making construction more efficient
- Using cheaper rock
- Reducing excavation and underwater working

Each has advantages and disadvantages, which are discussed in the following sections.

5.1 Less material

The size and shape of rock structures (their ‘geometrical design’) is determined primarily by functional and performance requirements such as the length and degree to which a beach is protected or the proportion of longshore transport trapped by a groyne. However within the geometrical envelope some aspects of the structure design are dependent solely on the size of the rock armour, for example the crest width and layer thickness are often defined as two or three rocks. If analysis or alternative structural configurations (particularly the use of permeable core materials) can increase the confidence that a smaller grading of armour will exhibit the required stability, the volume of rock required can be significantly reduced providing both cost and environmental savings.

Other strategies for reducing the quantity of rock used are to replace parts of the rock with alternative materials. This might be the use of alternative materials for the core of the structure (waste materials such as tyres may be particularly suitable for this) or employing composite structures where rock is used for some parts of the structure and other structural configurations adopted where they provide savings. An example would be the replacement of the landward end of a rock groyne with a vertical (possibly timber) structure or the introduction of armourstone extensions or replacements to seaward ends of selected timber groynes.

The increased monitoring and adaptability of rock structures may also enable designs to be less conservative, initially using only the minimum quantity of rock expected to provide the required performance in the knowledge that the structure will be monitored and can be enhanced if required.

5.1.1 Adaptability

It has long been claimed that rock structures can easily be adjusted, repaired or modified during the scheme life of the structure. In reality the use of different gradings, multiple

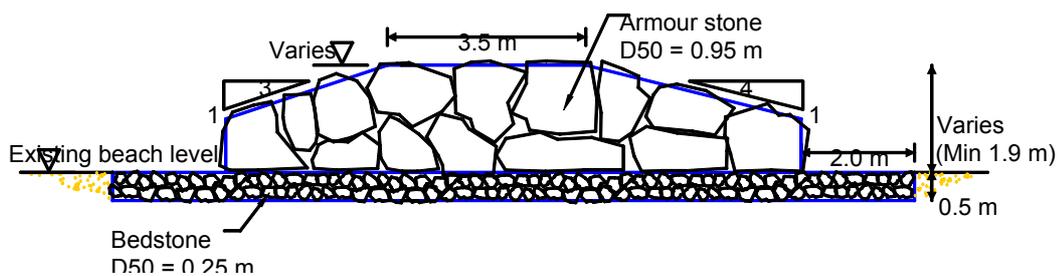
layers and prepared foundations can result in this being difficult and uneconomic. Thus whilst timber groynes may be adjusted annually, rock groynes have largely been perceived as unchanging throughout the scheme life.

Box 2 Fishtail breakwaters at Westshore, Llandudno



The scheme, comprising three fishtailed breakwaters and beach nourishment, was completed at West Shore, Llandudno in 1992. Within a few years it became apparent that the breakwaters were retaining a higher and wider sand beach than had originally been envisaged and local residents were increasingly concerned by the problems of wind-blown sand. This was initially remedied by beach management works, recycling sand along the foreshore, but after regular monitoring it became apparent that the removal of part of the central breakwater would be more economical and have minimal impact on the integrity of the defences. This work was carried out in 1997, with the armour rock stockpiled adjacent to the structure and the bedstone left in place on the beach. Further beach management was undertaken in 1998 and a performance appraisal carried out in Autumn 1999 recommended further reduction in the size of the central breakwater.

Since the structures comprised a single grading of armour placed on a layer of bedstone extending beyond the armour (see figure below) the adaptation was carried out quickly and economically using land based plant with no damage to the beach. If found to have adversely affected performance the changes can be quickly and easily reversed.



Typical cross section of breakwaters at West Shore, Llandudno

Increasing the ease with which the structures could be modified and accepting an ongoing requirement for monitoring, repair and adjustment provides several advantages:

- Design of structures could become much less conservative
- Structures may be optimised to provide the desired performance as requirements or conditions change
- Temporary structures could be used to provide short term or seasonal protection at 'hotspots'

Increased adaptability is particularly valuable for coastal structures due to the inherent variability and uncertainty in both loading conditions and performance. This will, however, require the ability to deliver relatively small quantities of rock to sites to allow enhancements to take place or stockpiling of rock at the time of initial construction (which might negate some of the cost savings). Adaptation of structures can also be useful where functional requirements change over time, as illustrated in Box 3 below.

5.2 More efficient construction

Careful design and detailing of rock structures, together with reduction and appropriate allocation of risks can significantly reduce construction time and costs. It should, however, be noted that this reduction is unlikely to be fully acknowledged during the conventional competitive tendering process, but will be fostered by effective communication and a long term relationship between all parties.

The use of simpler cross sections, with fewer different gradings of rock will reduce the number of construction operations required and the degree of checking. This in turn will make construction quicker and the use of a single grading of armour will also reduce the risk of damage to the structure during construction. Construction can also be assisted by the use of simple, easily visualised structures which are quick to 'set out'. The value of easily understood drawings and an understanding of the designers requirements (often best facilitated by the attendance of the designer on site during the initial phases of the work) should not be underestimated.

Reducing the contractors risks by agreeing to payment by weight of rock armour or enabling a clear definition of what is required at the start of the project (through the use of a trial panel of measured density incorporated into the works) is also likely to reduce construction costs as will the sensible programming of works. Construction duration often has a significant impact on construction costs and, where possible, opportunities for enabling the programme to be accelerated, through night and tidal working (on at least the most restricted or critical elements of the scheme) should not be limited.

5.3 Cheaper rock

The cost of rock supplied to site can vary by as much as $\pm 25\%$ of the average depending on the state of the market. The more choice of supply the greater the likelihood will be of obtaining economic materials. Limiting the size of armour and developing alternative designs to accommodate constraints imposed by local quarries can enhance choice.

Box 3 Rock groynes at Hengistbury Head

Five rock groynes were constructed at Hengistbury Head in 1986-7 to protect the eroding cliffs, however since they were located at the down-drift end of the frontage with ample longshore transport, beach nourishment was not provided but the groynes were intended to trap some of the longshore transport since this would have only minimal impact on the adjacent frontage.



Rock groynes with temporary additional crest layer in place

To ensure that an adequate beach was built within the groyne bays a row of large rocks was placed along the crest of each groyne to increase their protection against oblique wave attack and their effectiveness in retaining the beach during severe events. This proved successful and the beach built rapidly after which the extra layer of rock was removed from the groynes and stockpiled on site for reuse during maintenance works.

The rock used in the groynes was sourced from local quarries and whilst the armour does degrade over time, periodic maintenance has proved sufficient to maintain the integrity and functioning of the structures.

Where there are local quarries within a short distance of the site they may be able to supply rock at an attractive rate, but there may be concerns, particularly with the quality of rock, grading of the armour and rate of production. Many of these concerns can be overcome, either by stockpiling armour prior to the commencement of the main contract, widening the armour grading to utilise a greater proportion of the quarry yield or relaxing the quality requirements and making provision for importing additional armour during the life of the scheme. Where there is difficulty in obtaining a sufficiently large grading of armour, selective placement can be used (this may include the use of a larger sub-grading of armour in locations where the greatest damage is

expected such as the crest and toe of structures or the outer end of a groyne) providing the increased cost and difficulty of construction does not outweigh the benefits of local materials

In some situations the choice of rock available for coastal works is limited to poor quality materials by economic constraints. In this situation attention to the selection of the most suitable of the materials available will be rewarded, but the consequences that the material will have on the expected useful life of the structure should be fully accepted at the outset (see for example Fookes and Thomas 1986). The use of some materials such as Portland stone, which do not fully comply with standard quality requirements for rock armour can also be justified when there is historical evidence to suggest that their durability is actually acceptable (Clarke 1988). Less durable materials such as Purbeck stone can also be justified, providing they reduce whole life costs.

5.4 Reducing Excavation

Excavation, often for the toe or other foundation of rock structures is a difficult and expensive operation. Excavation at sea by marine based plant seems to be avoided by engineers as much as possible. (The most common reason why it is occasionally required relates to poor bearing capacity in the ground conditions and the need to replace the existing soil with better, often granular, material.)

It is in excavation using land based plant that the greatest savings can be made in the future. Costs are high for such excavation because the excavation often contains water and creating stable side slopes requires either shallow side slopes or strutting and propping. In addition excavation on beaches tends to infill rapidly on high tides due to sediment being washed in by waves and currents. The effect of this infill means that construction of rock structures in excavations often needs to be carried out in short sections, involving excavating and replacing the excavation with rock material on the same tide. Such working in short sections is inefficient and time consuming. The deeper such excavations go, the more water they are likely to contain and this adds to time and cost and increases safety risks.

6. DESIGN AND DETAILING FOR LOWER COST

The use of low cost structures requires a modified design approach from the conventional guidance outlined in Chapter 3. Such an approach could rely on physical or numerical modelling combined with the application of expert judgement, however, given that the principal incentive is reduce costs extensive modelling may be counter productive. The use of physical modelling to fine tune design, is that this technique incorporates many natural processes and is widely accepted as a proven design methodology, that will be considered acceptable best practice design. It is, however, possible to take advantage of past experience and recent research to develop appropriate design techniques which are described in the following sections.

6.1 Appropriate armour sizing

The assessment of appropriate armour size is critical for the success of all rock structures, but developing low cost structures requires a greater degree of understanding of design issues as there is much less scope to make 'conservative' decisions. About half of the structures investigated during the course of this project have armour sizes substantially smaller than would be suggested from the use of the nearshore wave climate. This may be partly attributable to variation in the location of the nearshore wave prediction point, but does demonstrate the scope for appropriate armour sizing.

6.1.1 Design for depth-limited locations

Within the 'Rock Manual' (CIRIA/CUR 1991) it is suggested that $H_{2\%}$ wave height may be more appropriate than H_s for use in depth-limited conditions, and van der Meer suggests revisions to his equations using the relationship between $H_{2\%}$ and H_s for deep water. The suggestion is based on the assumption that it is the largest waves that cause structural damage and the benefit of adopting this method is that in progressively more 'depth-limited' situations the ratio between H_s and $H_{2\%}$ reduces from approximately 1.4 in deep water to 1.2 in shallow. This reduction in $H_{2\%}$ due to wave breaking can be significant in justifying smaller armour size than would otherwise be used.

The manual went on to suggest that $H_{2\%}$ could be estimated for depth limited locations from Goda's equations (which give H_s and $H_{0.4\%}$). However, whilst estimating $H_{2\%}$ from Goda's H_s and $H_{0.4\%}$ is possible it is not entirely satisfactory. An alternative is the use of numerical models, which are often used to assess beach profile response, to assess variation in wave energy across the beach profile. These may be used in combination with research into wave height distributions on shallow foreshores, recently presented by Battjes & Groenendijk (2000). Where numerical models are not available the use of design graphs generated for standard cases may also be of benefit and are clearly far superior to the adoption of a simple breaker criteria (such as $H_s=0.55h$). Such graphs are presented in Figure 12 and include the influence of both shoaling and wave breaking (van der Meer, 1990c, e). The graphs provided are for wave steepnesses of 0.05, 0.04, 0.03, 0.02 and 0.01; if the steepness falls between the values for two different graphs breaker indices should be found using both and the final value interpolated from these.

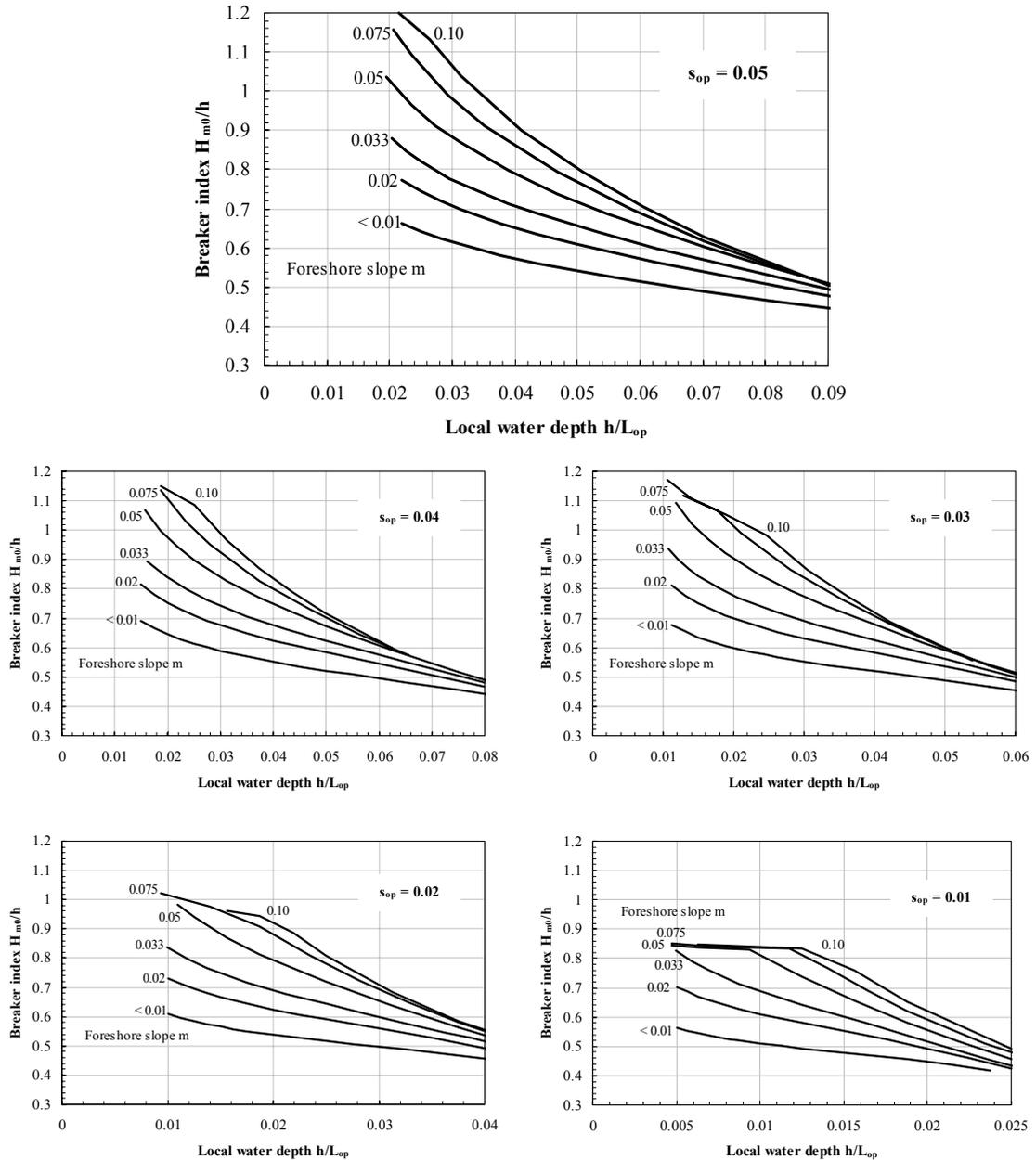


Figure 12 Shallow-water significant wave heights for uniform foreshore slopes

The method to calculate maximum breaking significant wave heights is based on energy methods, which means that the significant wave height used is H_{m0} . Since significant wave heights H_{m0} and $H_{1/3}$ are almost equal in deep water this should not present undue difficulty provided wave information is obtained for deep water. Based on the value of H_{m0} for a particular depth of water, the significant wave $H_{1/3}$ and the 2% wave height $H_{2\%}$ can be calculated according to the method described by Battjes and Groenendijk (2000). A summary of the method follows:

$$H_{m0} = 4 \sqrt{(m_0)}$$

$$H_{rms} = (2.69 + 3.24 \sqrt{(m_0)/h_m}) \sqrt{(m_0)}$$

where h_m = local water depth

$$H_{tr} = (0.35 + 5.8 \tan m) h_m$$

where $\tan m$ = the slope of the foreshore in front of the breakwater

$$\hat{H}_{tr} = H_{tr}/H_{rms}$$

In the table below values of \hat{H}_{tr} are given with corresponding values for the dimensionless significant wave height $\hat{H}_{1/3}$ and 2% $\hat{H}_{2\%}$

Table 9 Characteristic dimensionless wave heights

| | | | | | | | | | | |
|-----------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| \hat{H}_{tr} | 0.05 | 0.5 | 1 | 1.2 | 1.35 | 1.5 | 1.75 | 2 | 2.5 | 3 |
| $\hat{H}_{1/3}$ | 1.279 | 1.28 | 1.324 | 1.371 | 1.395 | 1.406 | 1.413 | 1.415 | 1.416 | 1.416 |
| $\hat{H}_{2\%}$ | 1.548 | 1.549 | 1.603 | 1.662 | 1.717 | 1.778 | 1.884 | 1.985 | 1.978 | 1.978 |

The actual $H_{1/3}$ and $H_{2\%}$ can then be calculated:

$$H_{1/3} = H_{rms} \cdot \hat{H}_{1/3}$$

$$H_{2\%} = H_{rms} \cdot \hat{H}_{2\%}$$

It remains important, however, to assess wave conditions at an appropriate location relative to the structure. This is because total wave energy is not immediately decreased on wave breaking, but continues to propagate inshore for some distance. Within Goda's formulae the water depth at a distance of five times H_s from the point of interest is used although van der Meer recommends the application of his equations using wave conditions at the toe of the structure.

Box 4 Determination of design wave heights

The determination and application of design wave heights is illustrated in the following example:

A groyne is to be constructed at a site with the following characteristics:

Beach slope approximately 1:100

Design offshore wave conditions $H_{s0} = 4.0\text{m}$, $T_m = 5.5\text{s}$, $T_p = 7\text{s}$

Design water level = mean sea level +3.0m

Based on the offshore wave conditions it is possible to calculate an armour size using van der Meer's equations. Using $S_d = 2$, $\alpha = 1.5$, $N = 3000$ and $P = 0.4$ this gives $W_{n50} = 7.7$ tonnes.

However, because the groyne is located on the beach, some of the wave energy will be dissipated by the seabed before reaching the structure. If the minimum beach level at the head of the groyne is assessed to be -2m MSL (including any allowance for scour or erosion) the maximum wave height for the design depth may be determined using the design graphs:

Since offshore wave conditions assume $H_{m0} = H_{1/3} = H_s$

h/L_{op} where $L_{op} = gT_p^2/2\pi = 9.81*7^2/2*3.14 = 76.5\text{m}$ thus $h/L_{op}=5/76.5 = 0.065$

$$s_{op} = H_s / L_{op} = 4.0 / 76.5 = 0.052$$

Since $s_{op}=0.05$ and $h / L_{op} = 0.065$, then from Figure 12 $H_s/h = 0.50$

$$\text{Thus } H_{m0}=0.50 * 5 = 2.5\text{m}$$

Since $H_{m0} = 4 \sqrt{(m_0)}$ thus $\sqrt{(m_0)} = 2.5 / 4 = 0.625$

$$H_{rms} = (2.69 + 3.24 \sqrt{(m_0)/h}) \sqrt{(m_0)} = (2.69 + 3.24 * 0.625/ 5) * 0.625 = 1.93\text{m}$$

$$H_{tr} = (0.35 + 5.8 \tan m) h = (0.35 + 5.8/100) * 5 = 2.04$$

$\hat{H}_{tr} = H_{tr}/H_{rms} = 2.04 / 1.93 = 1.06$ therefore from interpolation in the table of characteristic dimensionless wave heights (Table 9):

$$\hat{H}_{1/3}=1.34 \text{ and } \hat{H}_{2\%} = 1.62 \text{ and } H_{1/3} = 2.59\text{m} \text{ and } H_{2\%} = 3.13\text{m}$$

Using the same values as before within van der Meer's equations with relevant C_{pl} and C_{su} :

$$H_{1/3} \text{ gives } D_{n50} = 0.93 \text{ and } W_{n50} = 2.09 \text{ tonnes}$$

$$H_{2\%} \text{ gives } D_{n50} = 0.85 \text{ and } W_{n50} = 1.61 \text{ tonnes}$$

Thus a smaller grading of armour can be justified which may facilitate the use of cheaper, locally available rock and smaller plant. The total volume of the structure will also be reduced if the structure dimensions are a function of the armour. For example if the layer thickness is $0.91 * D_{50}$, the crest width is three rocks and the groyne height is three layers with side slopes of 1:1.5, the cross sectional area of the groyne trunk will be 16% smaller using the $H_{2\%}$ calculations in this example. The increased stability required at the head of the groyne may result in even more significant savings at this location.

It should however be noted that the effective design of rock structures in depth limited locations is dependent on a robust analysis of the joint probability of waves and waterlevels, and that the designer must consider the increase in wave height due to sea-level rise over the life of the structure. It should also be remembered that the structure is likely to be exposed to conditions close to the design standard on a much more regular basis than would be the case if it was not depth limited (since the difference between routine and severe wave heights will be much less).

6.1.2 Performance of tightly packed armour layers

Research has also recently been completed by Stewart (2002, 2003) into the hydraulic performance of tightly packed armour layers. This study demonstrated that the stability of armour layers increases significantly if the armour layer is placed carefully to achieve a tight packing and that the dissipation of wave energy was not greatly affected.

The study concluded that the parameters given in van der Meer's equations for bulk placed rock ($C_{pl}=6.2$ and $C_{su}=1.0$) are appropriate for armour layers where the void porosity approaches $n_v = 40\%$. If, however the porosity reduces to below $n_v = 35\%$, then stability increases significantly and revised values ($C_{pl}=7.8$ and $C_{su}=1.8$) are more appropriate. It should be noted that the minimum porosity tested for armour stability was $n_v = 34.5\%$ and it is not appropriate to extrapolate higher values for the stability coefficients from lower porosity values.

In order to apply the revised stability coefficients it is recommended that designers should satisfy themselves that the following criteria will be met:

- Rocks should be individually placed with good orientation control above water. In practice this implies placement by a grab mounted on a hydraulic excavator, not dumped into position or lowered on a sling by a crane.
- A porosity of less than $n_v = 35\%$ will be achieved for the armour layer
- Rocks should not be excessively round; if blockiness measurements are available there should be few or no rocks with a blockiness coefficient of less than 50%

The effects of increased packing density on the quantity of armour and cost are illustrated in Box 5 below.

Box 5 Tight packing for a revetment

If a revetment is constructed in deep water at the toe of a cliff with a front slope of 1:1.5 and the same loading characteristics as described in Box 4, the armour size determined using conventional van der Meer equations and parameters is 7.7 tonnes or $D_{n50} = 1.43\text{m}$. If, however, tight packing is adopted (thus $C_{pl} = 7.8$ and $C_{su} = 1.8$) the required armour size is reduced to 3.9 tonnes or $D_{n50} = 1.14\text{m}$.

Assuming that the tight packing reduces the 'as constructed' porosity from 40% to 30% the quantity of rock required to construct the revetment for tight packing will increase by 10 %. The time and cost for constructing the works is likely to increase further since the additional effort in selecting and placing the armour more tightly will outweigh any savings from the use of smaller plant.

The savings provided by tighter packing of the armour layer will relate to the reduced layer thickness resulting from the smaller armour, which is directly proportional to the D_{n50} , thus, in this case approximately 20%. Additional savings from reduced crest width are unlikely to be significant in terms of the total armour volume, but may reduce the volume of fill required under the revetment.

It should be noted that savings could be more significant for a structure where the volume is a function of the square of the layer thickness (for example a groyne) but may not be advantageous (indeed the additional density may be detrimental) for structures such as nearshore breakwaters where the performance or functional requirements mean that the crest elevation or width cannot be reduced.

The study also provided indicative porosity and layer coefficients for use in preliminary design from the extensive laboratory and prototype measurements.

Porosity $n_v = 32.1\%$ and layer thickness, $K_t = 0.91$

It should be noted that these values are heavily influenced by construction requirements and techniques and may not be universally applicable or achievable.

It is suggested that the porosity of a double layer can be further reduced by approximately 2% if the ‘dense packing’ approach is adopted. This is described by Stewart (2003) as follows:

‘The rocks were rotated until the orientation that was likely to produced the maximum number of points of contact in the layer (and minimum volume of voids) was achieved. Individual rocks were removed and replaced several times if necessary.’

The ‘standard’ approach was:

‘Each rock was placed with the orientation that it had naturally adopted in the stockpile. The only placement criterion that had to be satisfied was that the rock should have a minimum of three points of contact in the layer in which it was placed. Only if three points of contact could not be achieved with the rock’s natural orientation, was the rock rotated.’

Clearly the dense packing approach will considerably reduce the rate of production compared to the standard approach (initial indications suggest that this might be of the order of 10%) but the benefits of increased stability (or smaller armour sizes) combined with a perceived increase in public safety may result in it becoming widely adopted.

The report by Stewart (2003) recommends that *‘As with all designs that diverge from standard procedures, it is recommended that model tests are conducted’*. It should also be noted that all of the tests were carried out on straight revetment sections. The practicality of applying dense packing to complex structures such as roundheads, or where other criteria such as a defined crest level must also be achieved have not been investigated.

6.1.3 Notional Permeability factor

The notional permeability factor P is an important element in van der Meer’s stability equations and there is some concern that the concept is not fully understood by all designers. The factor P is intended to relate to the penetration of water through the armour during run-up and it should be noted that what may appear to be relatively insignificant changes in structural configuration can have major consequences for armour stability. For example a decision taken on site to replace a granular filter underneath an armour layer by a geotextile might change P from the design value of $P = 0.4$ to $P = 0.1$ necessitating a significant increase in the size or the rock armour to maintain stability.

Similarly it is possible that structures built on a beach will have a varying value of P for different parts of the structure. It is possible that the configuration of armour in the toe, or accumulation of sediment within the base of the structure will result in a value of P for the armour in this area closer to $P = 0.1$. However, the top of the structure may be homogenous and have a permeability closer to $P = 0.6$. In practice the wave exposure at the toe will be greatly limited by the beach, but the design should investigate the stability requirements for each part of the structure.

6.1.4 Using wider gradings

When using gradings wider than normal ($D_{85}/D_{15} < 1.5$), there is potential for the smallest rocks to become dislodged from the body of the structure. On shallow slopes (less than 1:4) this may lead to moving rocks falling back, known as self-healing, but on steep slopes (steeper than 1:3), this will ultimately lead to a decrease in the stability of the armour layer. Anecdotal evidence suggests that providing the grading is not very wide ($D_{85}/D_{15} < 2.5$) and the structure slopes are not too steep ($\text{Cot } \alpha > 2.5$ or 3) the use of wide gradings is unlikely to result in significant problems. Allsop (1990) showed that extremely wide grades ($2.5 < D_{85}/D_{15} < 4$) were not desirable at slopes 1:1,5-1:4 due to the high risk of spatial variation and local failure.

The majority of the structures studied during the course of this project did not use the 'standard gradings' of armour as defined by the 'Rock Manual' (CIRIA/CUR, 1991). Some were designed before the publication of the 'standard gradings', but others reflect the willingness of the designers to base the structure on materials available locally, possible at an advantageous price. None of the gradings used were very wide.

6.1.5 Selective placement of armour

One technique which has been associated with the use of wider gradings, but which may also be used where there is a concern that the armour stability is marginal is the selective placement of the largest rocks in the parts of the structure where greatest damage is likely to occur. In most cross sections this will be at the crest and / or toe of the structure as shown in Figure 13 below.

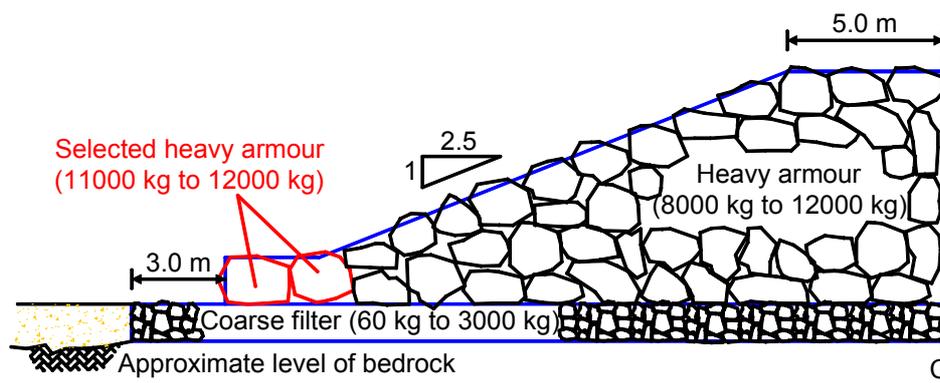
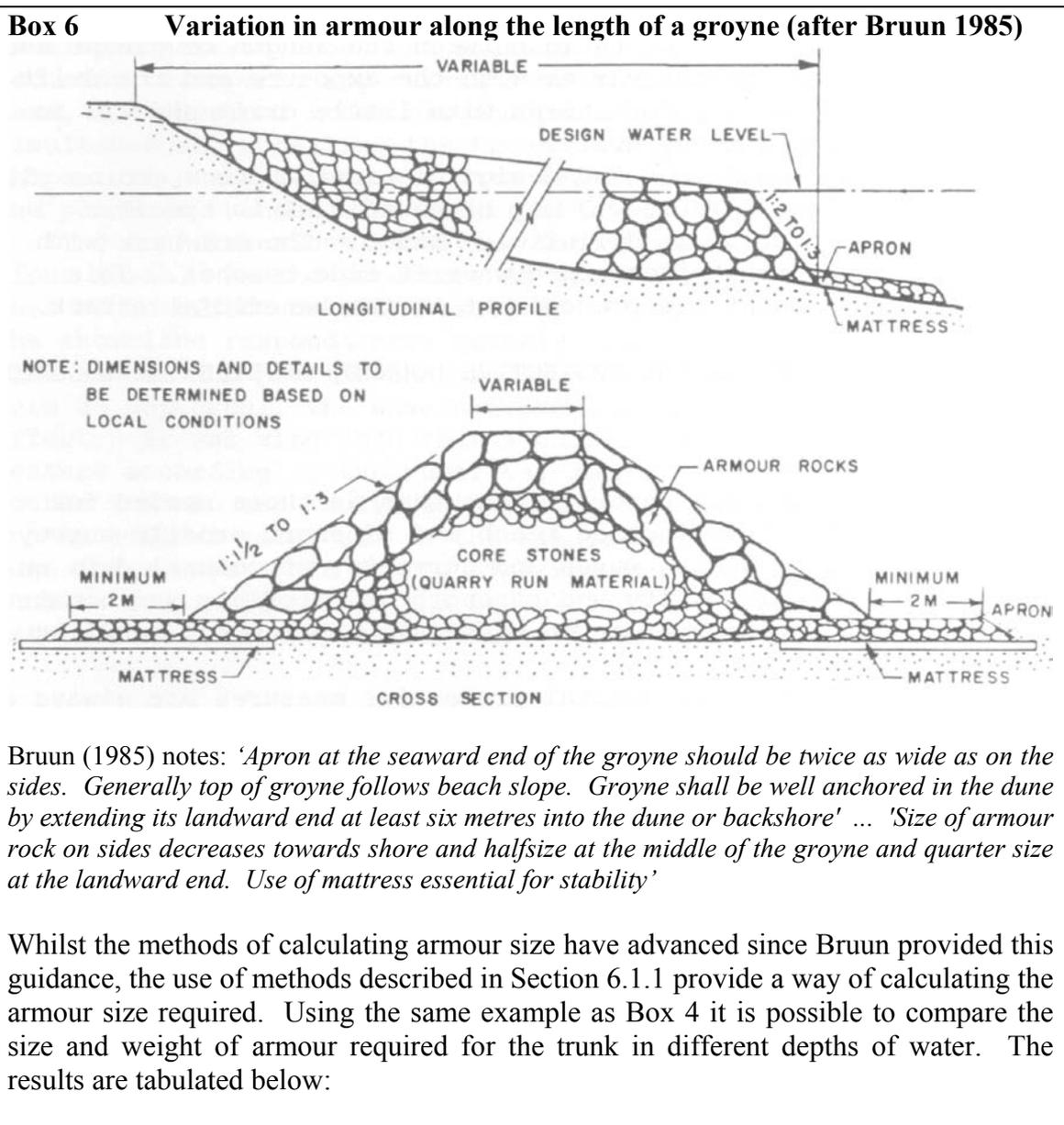


Figure 13 Selective placement in the toe of a nearshore breakwater at Sidmouth

It is important that the contractor is fully aware of such a proposal at the time of tendering and the selection and setting aside of the largest rocks can incur significant effort during construction and the requirement to place large rocks at the crest of the

structure may have implications for the selection of plant. The specification of such selective placement may best be formalised through the creation of a sub-grading for the largest rocks which can be separately priced if appropriate. Where rock of the required size for a particular part of the structure is not available the use of simple (cube or tripod) concrete armour units may be appropriate.

A similar approach can also be applied to the plan shape of some structures. For example the head of a rock groyne is often subject to much greater loading than the trunk or root of the structure and if the supply of the largest armour is severely limited it may be sensible to vary the grading along the length of the structure as shown in Box 5 below.



| Depth of water (m) | | 6 | 5 | 4 | 3 | 2 | 1.5 |
|--------------------|-----------------------|-------|-------|-------|-------|-------|-------|
| H _{1/3} | D ₅₀ (m) | 1.134 | 0.927 | 0.799 | 0.789 | 0.631 | 0.525 |
| | W _{n50} (kg) | 3,822 | 2,090 | 1,336 | 1,288 | 658 | 378 |
| H _{2%} | D ₅₀ (m) | 0.933 | 0.849 | 0.760 | 0.649 | 0.519 | 0.434 |
| | W _{n50} (kg) | 2,131 | 1,605 | 1,151 | 716 | 366 | 215 |

In this example the armour weight required is reduced to about one third when the depth of water is halved. Depending on the configuration of the groyne and beach profile there is clearly significant potential to reduce the size of armour as the maximum depth of water decreases. It should be noted that these calculations make no allowance for the increased stability required at the head of the groyne.

This concept may also be applied to the side slopes of the structure if it is decided to keep the armour size and grading constant for logistical reasons. The slope might vary from 1:3 at the head of the groyne, to 1:2 for the outer trunk and 1:1.5 for the root of the structure.

6.1.6 Use of single layers of rock armour

Whilst the vast majority of coastal rock structures use double layers of rock, a number of single layer structures have been used in Britain for coast protection revetments at the toe of cliffs and also in developing countries, again largely in revetments. Whilst the use of tight fitting concrete blocks, basalt columns or interlocking armour units can successfully be applied in single layers, the use of single layers of rock armour must take account of the potentially “brittle” failure where the removal of a single stone can lead a catastrophic failure. Single layer revetments were investigated in recent research into armour packing. Stewart (2003) used Hudson’s formula with $H=H_{1/10}$ to analyse stability, but found K_D factors for no damage varying from 1.5 to 23. It was clear to Stewart (2003) that his studies (and similar tests by Camfield in the USA) showed situations where tightly fitted rock (down to $n_v \approx 25\%$) is significantly more stable (high values of K_D). Both UK and USA studies however also showed examples where rock was extracted for relatively low values of K_D . These may have been due to local variations of rock packing, or to locally high wave velocities or pressures. This variability in test results suggest that the use of simple layer armour may give significant benefits, but can not be recommended for safety-critical structures without more detail studies, or very tight quality control on placement.

Types of simple layer armouring where there is already more information, and less scatter is of blockwork revetments. Here rock or concrete blocks are placed very tightly (with porosity below say $n_v < 15\%$).

A helpful approach in understanding this is suggested by Yarde et al (1996) who extended the general method by Klein Breteler & Bezuijen (1991) by including more directly the effects of underlayer material, for which Klein Breteler & Bezuijen had simply presented upper and lower bound coefficients. Yarde et al (1996) suggested the following modified equation:

$$H_s / \square t_a = S_c / \square_p$$

Where the stability coefficient, S_c , is described as a function of the dimensions and permeabilities of the cover layer and under-layer:

$$S_c = 3.3 \ln((\sqrt{A_s} / t_f) (w / D_{f15})^{0.1}) + 4.0$$

and A_s is the slab or block top surface area
 t_f is the thickness of the filter layer
 w is the gap between slabs representing drainage area or cover layer permeability
 D_{f15} is the 15% non-exceedance diameter of the filter layer material (obtained from the grading curve). This indicates the relative permeability of the filter layer.

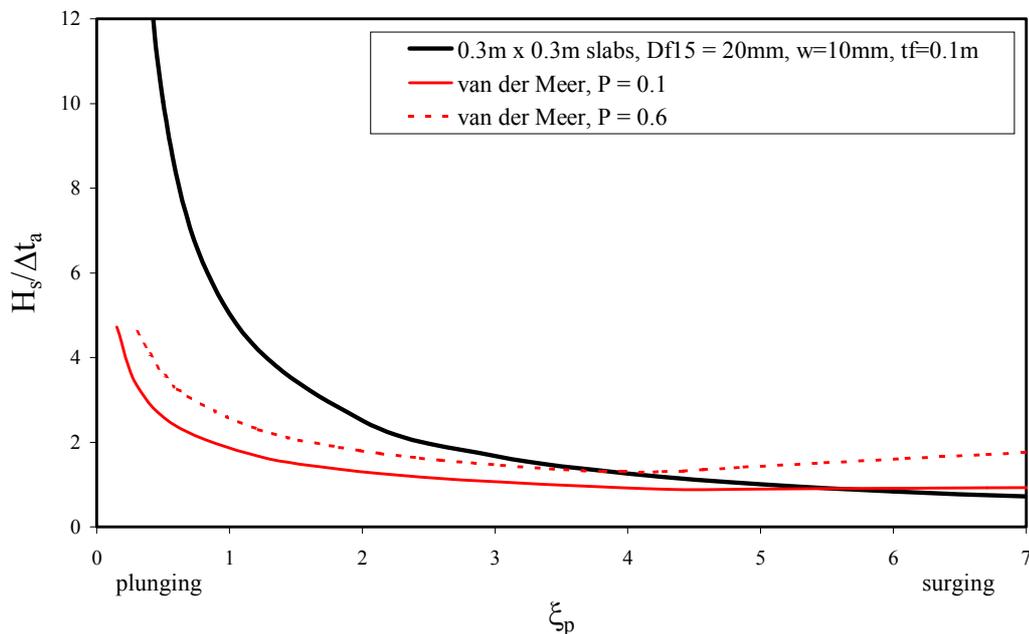


Figure 14 Comparison of stability prediction methods for rock armour and concrete slabs (after McConnell & Allsop 1999)

Use of this method for single layer low-permeability armouring was discussed by McConnell & Allsop (1999) who compared Yarde et al's results to those from the established method for two-layer random placed armour by van der Meer (1988). Both methods are used in Figure 14 to calculate the total layer thickness of armour, t_a . Yarde's method is used here for small blocks of approximately 0.3m side on top surface. The comparison shows that there will not be large savings in layer thickness and hence in armour volume for longer waves with slope Iribarren number $\xi_p > 2.5$, but that for waves of shorter periods and $\xi_p < 2.5$, then significant savings in armour thickness / volume may be possible by close packing.

6.1.7 Roundheads

No reliable generic studies have been reported on the stability under random waves of rock armour on roundheads, although it is well established that armour on some zones of a roundhead will be less stable than along a trunk section. The Shore Protection

Manual suggests reduced values of K_D for structure heads under breaking waves, from which a relative increase in armour unit mass may be estimated (Allsop & Jones, 1994):

| Structure | K_D | Relative increase in M_{50} |
|------------------------|-------|-------------------------------|
| Trunk, slope 1:1.5-3.0 | 2.0 | 1.0 |
| Roundhead, slope 1:1.5 | 1.9 | 1.2 |
| Roundhead, slope 1:2.0 | 1.6 | 2.0 |
| Roundhead, slope 1:3.0 | 1.3 | 3.6 |

Armour stability in roundheads may also be increased by reducing the slope of the structure locally, again the application of Hudson's stability formulae allow the extent required to be estimated.

Roundheads are often particularly difficult to construct: since the structural form is curved it is difficult for the plant operator to visualise the surface required and surveyors or engineers to set out. Roundheads are often constructed on the lowest part of the beach where the tidal window is shortest and potential wave loading most significant. Where a roundhead is not required to refract incoming waves it may be possible to avoid altogether by adopting a sharply tapering section at the head of the groyne. This is likely to reduce construction difficulties and will also reduce the volume of rock required

6.2 Foundation and toe details

In addition to the appropriate sizing of rock armour, the design of the foundation and toe details are of critical importance for the performance and integrity of coastal rock structures.

Foundation conditions may be broadly classified into those with:

- Substantial exposed strata
- Minimal mobile beach (<0.3m) overlying a substantial strata
- Moderate mobile beach (0.3-2.5m) overlying a substantial strata
- Deep mobile beach

The potential for settlement, scour and liquefaction will need consideration for each of the situations although the significance of each will vary to some extent with the wave loading, water levels, beach and substrate materials.

6.2.1 Toe details

Rock structure toe details deserve particular attention; they are often exposed to the most severe wave climates, and are most difficult to construct (often with a very small working window due to the returning tide or ground water within the beach flooding any excavation). The base of the toe detail will be dependent on an assessment of the maximum depth of scour and the depth of mobile beach in front of the structure. Where the beach is relatively thin and over lies a strata which will not easily be eroded it may be possible to adopt a relatively simple toe detail as shown in Figure 15. This uses a selected larger rock to provide stability and a good starting point for the structure at the toe.

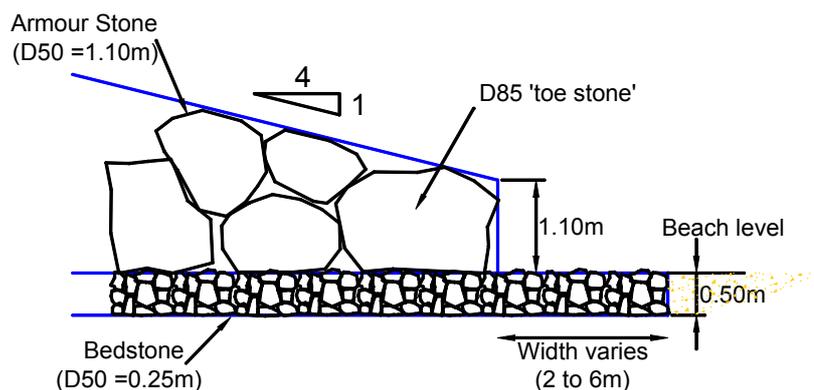


Figure 15 Toe detail from a fishtailed breakwater at West Shore, Llandudno

The large toe stone can, however, result in significant reflections of waves leading to increased erosion or scour in-front of the structure. This can be avoided by extending a bedstone apron some distance in front of the structure to protect the beach. The apron has the dual purpose of providing access beside the structure preventing the construction plant from damaging the beach.

Where there is a significant beach above the impermeable strata the front slope of the structure has often just been extended downward such that the top of the base rock is below the expected depth of scour (d_s). Whilst this solution is reliable and robust (providing of course that the depth of scour has been correctly estimated) the toe can require considerable excavation, which itself will increase the probability / extent of scour, and construction can be dangerous (particularly if geotextile is laid in the foundations). An alternative approach is illustrated in Figure 16 which provides a stockpile of armour at the surface of the beach which should settle into any scour-hole, protecting the structure from undermining. The latter solution might increase initial scour (due to reflections from the stockpile) and may not be as inherently robust as the buried toe, but provides a useful, cheaper and more adaptable alternative.

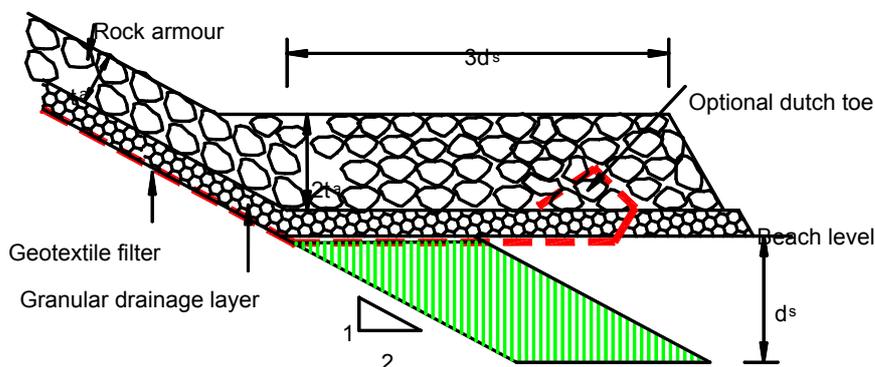


Figure 16 Falling apron toe detail

Where the structure is founded directly onto a hard strata (generally those which need special effort to excavate using standard plant) it may be necessary to excavate a shallow toe trench to provide additional toe stability against local slipping. If this is the

case care should be taken not to unduly weaken or fissure the underlying strata. Other toe details (such as the timber piles illustrated in Figure 17a) are sometimes used.

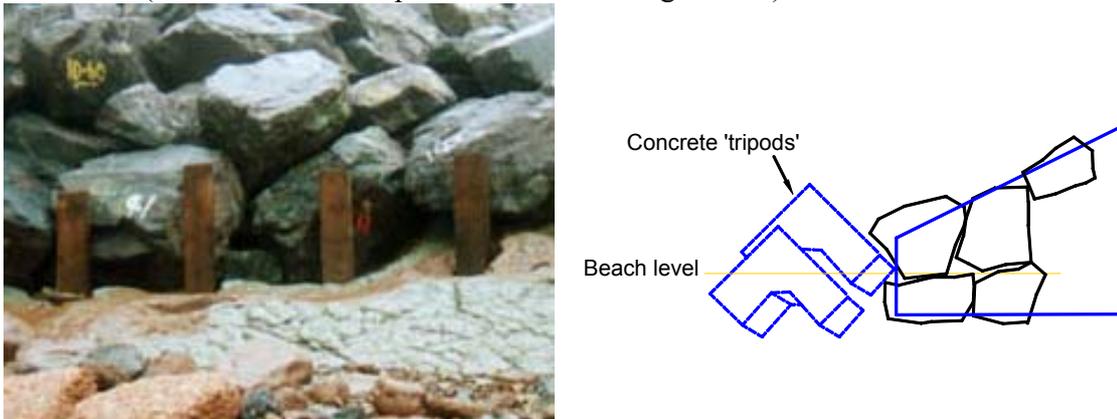


Figure 17 (a) Timber piles and (b) concrete units used to enhance toe stability at Barton and Highcliffe

6.2.2 Liquefaction

As waves propagate over the seabed they cause the pore pressures in the bed to vary inducing flow and hence variations in the effective stresses. In some cases, where the mean effective stress reaches zero, liquefaction can be achieved. If this takes place near a structure (eg breakwater) then failure may occur.

For sands, two different types of liquefaction have been identified in the literature. The first, momentary liquefaction, was first dealt with in detail by Madsen (1978) and Yamamoto et al. (1978). In this analysis the seabed is assumed to behave elastically and the pore fluid is allowed to be compressible. The pore pressure fluctuates at the wave frequency and induces reversing flow in the bed. It was shown that under the wave trough, the flow is directed upwards and out of the bed, causing the effective stresses to reduce. If the bed liquefied it only remained so for a small proportion of the wave period as it regained strength as the crest approached and the flow reversed. The extent of the liquefied zone depended on the compressibility of the pore fluid and seabed material, the wave characteristics (wave height and period) and the water depth. This type of liquefaction is prominent when gas is present in the pore fluid. The gas content only needs to be a little more than 1% to significantly increase the risk of liquefaction.

The second cause of liquefaction in a sand bed is more analogous with earthquake-induced liquefaction, and is termed residual liquefaction. Due to the oscillating bottom pressure under a wave, significant shear stresses are created and can be as large as the cyclic shear stresses induced by large earthquakes. Consequently, if the seabed is in a relatively loose initial state, excess pore pressures will be generated as the bed attempts to consolidate, as in the case of an earthquake. The rate of pore pressure build-up will depend on the magnitude of the forcing (ie wave conditions and water depth), the relative density of the sand and the coefficient of consolidation (ie drainage characteristics). This has been dealt with in detail by Rahman and Jaber (1986) and McDougal et al. (1989) and more recently by Cheng et al (2001). This type of liquefaction affects the whole seabed, irrespective of the position of the wave crest, and occurs over a longer period (typically in the order of minutes).

From cyclic testing of sands and knowledge gained from earthquake investigations, it is generally accepted that liquefaction is unlikely in a sand with a relative density greater than approximately 85%. On the other hand, if the relative density is less than 50%, the potential for liquefaction is very high.

In very soft clays, liquefaction (or fluidisation) can occur if the wave-induced shear stresses in the bed are greater than the undrained shear strength (de Wit, 1995). When this occurs, the bonds are broken and the structure tries to collapse, hence causing a rapid increase in the pore pressure. This mechanism is unlikely to occur in cases where the undrained shear strength is greater than 10kPa.

The main difficulty in trying to determine the liquefaction potential of the seabed is measuring the seabed properties such as the relative density, cyclic behaviour, gas content of the pore fluid, coefficient of consolidation etc. The standard practice for marine geotechnical investigations is to carry out a series of CPTs (Cone Penetration Tests) or SPTs (Standard Penetration Tests). CPTs can be used to give a continuous record of properties such as the relative density of a sand or the undrained shear strength of a clay. SPTs are carried out in conjunction with bore hole sampling and can also be used to determine the relative density and shear strength. Samples taken from the borehole can then be analysed in the laboratory and parameters such as the coefficient of consolidation, Atterburg limits etc. can be found. Ideally cyclic triaxial tests (or equivalent) would be conducted to determine the number of cycles to liquefaction. These tests however, may be prohibitively expensive for a low-cost rock structure. It is also understood that CPT and SPT tests are inaccurate over the first 1m or so of the seabed.

6.2.3 Underlayer and Filter criteria

Traditionally breakwaters have been constructed from cores of fine material (often quarry run) with various gradings of underlayers and filters preventing migration of the core through the outer layers. There are two reasons for including a core of fine material in a large breakwater; it allows a greater proportion of the quarry yield to be utilised (resulting in reduced cost) and reduces wave transmission through the breakwater.

The function of filter or under layers in rock structures for beach control and coast protection are not dissimilar, they are intended to prevent migration of the underlying beach material (normally sand or shingle) through the structure as this would result in settlement. Some filter or under layers also have secondary functions; providing a permeable core to the structure thus reducing the size of armour required (large value of van der Meer's P parameter) and / or enabling the desired finished level to be achieved more easily.

In some conditions the beach control structure may also be designed to restrain horizontal movement of beach material. Some rock sills retaining nourished beaches (hitherto more common in Italy or Spain) have been designed to include filters against such horizontal sediment movement, but inclusion of such a fully designed filter in rock groynes is very rare. This may reduce the groyne efficiency. Even if explicitly including a filter, however, a moderate/large rock groyne can sustain a sand bed level difference of at least 1-1.5m, as shown in Figure 18.



Figure 18 Permeable Rock groyne at Mappleton

The established design guidance presents a number of different criteria for filter or under layers as follows:

| Reference | Criteria |
|--------------------|---|
| CIRIA / CUR (1991) | $D_{15 \text{ armour}} / D_{85 \text{ filter}} < 4-5$ |
| SPM (1984) | $W_f = W_a/5 - W_a/20$ |
| CERC 93-19 | $D_{50(A)} / D_{50(B)} < 6.8$ |
| Bruun | $D_{50 \text{ filter}} > D_{50 \text{ armour}} / 3$ |

When converted to ratios between armour layer weight and filter layer weight the above values suggest that a ratio of approximately 1/10 W_{50} is appropriate. There is, however, evidence from the case studies which suggests that ratios of up to 40 have been used without any obvious problems.

6.2.4 Use of geotextiles

Use of geotextiles is desirable when there is potential for suffusion of fine material through a structure, leading to settlement. Geotextiles may also assist with distribution of loading of armour point loads across soft substrate conditions.

Placement of geotextile under water is extremely time consuming and may require special plant or development of novel techniques. If the geotextile is to be used, it must:

- be undamaged
- provide cover in a controlled manner

The use of geotextiles and constructability of toe details was investigated in a prototype trial in Hampshire described in Box 5.

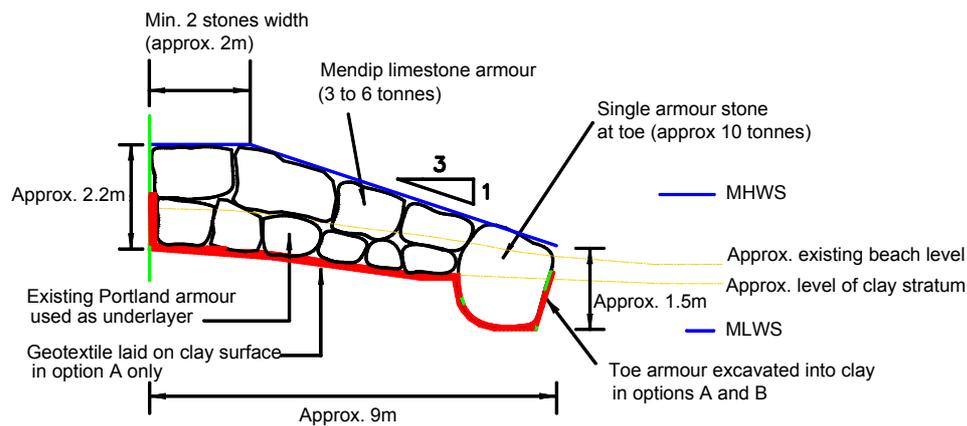
Box 7 Paddys Gap Rock Revetment Works

Maintenance works carried out to coastal defences at Milford-on-Sea, Hampshire provided an opportunity to compare a number of different foundation details. The works comprised the reconstruction of a 60m rock revetment which protects against the undermining of a concrete seawall within a groyne bay.

An existing revetment of 1-2 tonne Portland stone armour had been placed as emergency works in 1995, but performance had deteriorated and it was judged to have reached the end of its useful life by March 2003.

A new revetment was designed using existing rock as the underlayer and 3-6 tonne Mendip limestone for armour with the toe comprising larger stones up to 10 tonnes. Three different designs, representing varying degrees of complexity were built with the time taken and ease of construction monitored, and the performance of the different designs will be assessed during future monitoring.

The most complex design (A) is illustrated below and includes both geotextile and excavation for the toe rock. The intermediate design (B) omitted the geotextile and the least complex (C) did not include any geotextile or toe excavation.



Potential savings afforded by the designs were discussed with the contractor prior to the works and observed on site. The toe of the structure was found to be critical since this is most affected by weather and the 'tidal window', construction of the slope could occur on a rising tide providing the toe had been placed properly. The time saving resulting from the omission of geotextile was relatively small (approximately 30 minutes for the 20m section of revetment) and savings were found to be minimal since the same plant (one tracked excavator with a bucket and one with a rock grab) were required on standby for the whole of the work. The omission of an excavated toe provided a further saving of 10 minutes, however, again both excavators were needed as the mobile beach was removed even though the toe was not excavated into the clay.

The least complex section did, however, provide significantly more flexibility in construction which enabled work to be carried out during neap tides and in poor weather conditions. This provided minimal savings in the example, but could have had a significant benefit, enabling programme to be maintained, during more extensive works.



Construction continuing on a rising tide following placement of the ‘toe stones’

6.3 Monitoring and maintaining low cost rock structures

Efficient and effective monitoring and maintenance is particularly important for the successful performance of lower cost rock structures. Whilst many conventional structures are only monitored sporadically and seldom maintained, ongoing assessments and works are an integral element of the low cost concept and should be thoroughly considered at the design stage of the scheme.

6.3.1 Monitoring

In order to monitor performance it is necessary to define indicators that can be measured. These indicators are related to both the required performance and failure modes of the structure or scheme. The Rock Manual sets out four levels for the assessment of the state of rock structures and performance indicators relating to the processes at each level may be developed:

Table 10 Performance levels for rock structures

| Level of structure state | Typical processes taking place at this level |
|---------------------------------|--|
| Level I: Location | <ul style="list-style-type: none">• Settlement of foundation• Change of alignment |
| Level II: Geometry | <ul style="list-style-type: none">• Consolidation of structure (and relation to overtopping)• Change of slope profile due to wave action• Scour damage |
| Level III: Composition | <ul style="list-style-type: none">• Loss or movement of armour rocks• Overall sliding of armour layers• Voids requiring emergency/planned repair |
| Level IV: Element composition | <ul style="list-style-type: none">• Rounding of rocks• Loss of material by breakage |

These performance indicators are useful, but must be used in conjunction with further analysis. It is important to understand not only the processes taking place, but also the extent to which these impact on the performance of both the structure and the scheme as a whole. Three examples may be considered. An armour slope may fail if its armour units break. This may lead to a sudden and catastrophic failure of the system, viz. the breakwater failure at Diablo Canyon or Sines. Alternatively the armour breakage may simply increase the probability of armour movement in future storms, a more gradual process. But this may not yet reduce the performance of the overall scheme. Where armour damage extends to crest level reduction, then the increased overtopping / wave transmission may directly degrade the performance of the overall scheme, but the change may be small.

Only when each step can be described, can ceilings on the acceptable probabilities of failure of structural elements or the entire structural system be set with reference to the appropriate fragility curves.

Any examination of performance should also include an appreciation of construction tolerances, possible remedial actions, likely changes in loading / function and cost implications. Thus a judgement may be made as to the most appropriate design or configuration. The extent to which the exposure or function of the structure may change during its life is particularly important and performance assessments, as well as design activities, will necessarily involve some prediction of future exposure and likely response.

When assessing Level I performance indicators it is important that the relative location of the structure with reference to both the shore profile (which may be changing due to erosion or accretion) and water levels (changing due to climate change / sea level rise) should be assessed in addition to any more distant datum. For example, a rock revetment may be designed as a backstop to a beach / groyne system, in which location it is afforded a degree of protection by high beach levels. Indeed if the design of the revetment does not take account of the high beach levels it could be argued that the structure is overly 'conservative'. However, if the beach rolls back (if it is open) or any groyne system is neglected and the beach levels drop, the revetment will become increasingly exposed with time. Thus the revetment may fail, not because the design was inadequate, but because the original performance requirements have changed.

6.3.2 Maintenance and adaptation

Works will be initiated by the monitoring programme to either maintain the structure in the design state or adapt it in response to changing requirements or circumstances. In some cases maintenance may involve replacement of small numbers of armour stones dislodged from the structure, filling voids and reconstruction of small areas that have suffered damage. This work will normally be carried out by similar plant as was used during the original construction, although in some situations accessibility of the site may change with time (due to development of the hinterland or the effect of the structure). Maintenance may sometimes require small quantities of additional rock armour (to compensate for settlement or replace damaged or broken stones) which may be stockpiled at the site during the original construction or imported specifically for the maintenance (usually by road as quantities will be small).

Once constructed, an armoured rubble structure acts as a whole unit and patching may not be possible without creating a weakness. The local area needs to be removed, mixed with the new stones (carefully graded) and the reworked material re-laid. Only isolated plugging of holes for safety is satisfactory –otherwise a local reconstruction approach is needed.

Adaptation of structures will often involve more substantial activity, which might include the removal or extension of parts of structures and / or the raising / lowering of crest levels. Raising or lowering of the crest level may require substantial reconstruction of the structure if the change in level is anything other than a multiple of a single layer of armour. Changes will in any case require some reconstruction to ensure adequate interlock, otherwise a plane of weakness will be produced. Such changes can be more easily effected if a relatively wide grading is used although care must be taken to ensure that the grading in different parts of the structure is not distorted.

7. CONCLUSIONS

There is considerable potential for the wider use of lower cost rock structures for beach control and coast protection. Several examples of these structures have been reviewed and shown to offer acceptable performance as well as real financial savings. The performance of such structures does, however, require careful consideration and assessment.

Recent research into the packing of rock armour and the distribution of wave heights on shallow foreshores, together with the ongoing requirement to reduce the risk to the public, is likely to lead to the wider use of structures with smaller, tightly packed armour layers. The result of this and appropriate performance requirements will also contribute to a reduction in whole life costs of rock structures.

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NOTATION

| | |
|------------|--|
| A_e | Cross-section area of erosion around the still-water level |
| A_t | Cross section area of the structure |
| B | Crest width |
| C_t | Coefficient of transmission |
| c' | Cohesion |
| D | Particle size or typical diameter |
| D_n | Equivalent-volume cube |
| D_s | Equivalent-volume sphere |
| D_{n50} | Nominal particle diameter calculated from the median particle mass M_{50} |
| F_s | Shape factor |
| g | Gravitational acceleration; also used as a failure function |
| H | Wave height, from trough to crest |
| H_b | Breaking wave height |
| H_s | Significant wave height, average of highest one-third of wave heights |
| $H_{2\%}$ | Wave height exceeded by 2% of waves in a record |
| $H_{1/10}$ | Mean height of highest 1/10 of waves in a record |
| H_{tr} | Transitional wave height of the Composite Weibull Distribution (CWD) |
| H_{m0} | Spectral significant wave height, $H_{m0} = 4\sqrt{m_0}$ |
| H_{rms} | rms wave height |
| h | Water depth in front of the structure |
| h_t | Depth of the toe below still-water level |
| h_m | Local water depth |
| K_D | Empirical damage coefficient used in Hudson equation |
| k_t | Layer thickness coefficient |
| L | Wave length, in the direction of propagation ($gT^2/2\pi$ for deepwater conditions) |
| L_p | Local peak wave length |
| M_{50} | Median mass or armour unit derived from the mass distribution curve |
| N_z | Number of waves |
| N_s | Stability number, $H_s/\Delta D_n = (K_D \cot\alpha)^{1/3}$ |
| N_{ow} | Number of waves overtopping |
| N_w | Number of waves in a sequence |
| n_f | Filter porosity |
| P | Notional permeability factor used in calculation of armour stability |

| | |
|------------------|---|
| Q | Mean overtopping discharge rate per metre run of seawall |
| Q* | Dimensionless overtopping discharge by Owen (1980) |
| R_c | Crest freeboard, level of crest less static water level |
| R^* | Dimensionless freeboard parameter |
| R_u | Run-up level, relative to static water level |
| $R_{u2\%}$ | Run-up level exceeded by 2% of run-up crests |
| R_{us} | Significant run-up level |
| R_{ux} | Run-up level exceeded by x of run-up crests |
| r | Roughness or run-up reduction coefficient, usually relative to smooth slopes |
| S_d | Damage number for (rock) armoured slopes = A_c/D_{n50}^2 ; also used as a general load or surcharge on the system in reliability analysis |
| S_m | Steepness of mean wave period = $2\pi H/gT_m^2$ |
| S_{op} | Offshore wave steepness |
| S_p | Steepness of peak wave period = $2\pi H/gT_p^2$ |
| T | Structural, economic, or design lifetime (in years), also used as a (regular) wave period |
| T_m | Mean wave period |
| T_p | Peak wave period |
| t | Thickness of a layer |
| U_c | Uniformity coefficient = D_{60}/D_{10} |
| V_{bar} | Average overtopping volume per overtopping wave |
| V_{max} | Maximum volume likely to overtop |
| W | Armour unit weight |
| W_x | Block weight for which x% of the total sample weight is of lighter blocks |
| W_{50} | Median armour unit weight |
| α (Alpha) | Structure front slope angle to horizontal |
| β (Beta) | Angle of wave attack to breakwater alignment |
| Δ (Delta) | Reduced relative density, e.g. $(\rho_r/\rho_w)-1$ |
| ϕ | Internal friction angle |
| ρ (Rho) | Mass density, usually of fresh water; also used as correlation coefficient |
| ρ_w | Mass density of sea water |
| ρ_r, ρ_a | Mass density of rock, armour units |
| ξ | Iribarren number or surf similarity parameter, = $\tan\alpha/s^{1/2}$ |
| ξ_m, ξ_p | Iribarren number calculated in terms of s_m or s_p |
| ξ_{cr} | Critical value of Iribarren number distinguishing plunging from surging waves |
| σ (Sigma) | Stress |
| ψ (Upsilon) | Factor related to the importance of the structure and its function |