The principal function of a rubble mound breakwater is to protect a coastal area from excessive wave action. The dissipation of wave energy through absorption rather than reflection distinguishes rubble mound breakwaters from other types of fixed breakwater. A principal design objective is to determine the size and layout of the components of the cross-section. Designing and constructing a stable structure with acceptable energy absorbing characteristics continues to rely heavily on past experience and physical modelling. This paper outlines key design and construction issues, with particular regard to armour stability.

Keywords: breakwaters, harbour structures, revetments, riprap, rubble, shore protection

1. Introduction

The principal function of a rubble mound breakwater is to protect a coastal area from excessive wave action. Incident wave energy is dissipated primarily through turbulent runup within and over the armour layer (Figure 1). If the wave is steep or the seaward slope of the breakwater is relatively flat then the wave will overturn and plunge onto the slope, dissipating further energy. Some of the remaining energy is converted to potential energy as the wave runs up the slope whilst the balance is reflected seaward and also transmitted to the leeward (sheltered) side. In most situations only limited overtopping is tolerated and so most transmission occurs internally. Some of the internally transmitted wave energy is dissipated during flow through the core, with the remainder appearing as a wave on the leeward side. The effectiveness of a breakwater is judged by its ability to limit the height of this transmitted wave.

The bulk of the cross-section comprises a relatively dense rockfill core. This is armoured with one or two layers of rock or one of the numerous types of precast concrete armour unit. The outer layer is referred to as the primary cover layer. The term “rubble” as used here includes rock, riprap and precast concrete armour units. Similarly, “armour unit” includes both rock and precast concrete units.

The dissipation of wave energy largely through absorption rather than reflection distinguishes rubble mound breakwaters from other types of fixed breakwater. The famous rubble mound breakwaters at Cherbourg and Plymouth constructed late last century apparently arose from difficulties dissipating wave energy through reflection (Townson 1973). Designing and constructing a stable structure with acceptable energy absorbing characteristics continues to rely heavily on past experience and physical modelling. This paper outlines key design and construction issues, with particular regard to armour stability. It is relevant to other coastal rubble mound structures, such as groynes, revetments and training walls.

2. Design approach

A principal design objective is to determine the size and layout of the components of the cross-section. This traditionally involves empirical formulae and other guidelines like those given by the Permanent International Association of Navigation Congresses (PIANC) (1980) and the United States Army Corps of Engineers (USACE) (1984).
Reviews of failures of full size structures like those presented by Bruun (1979), Baird et al. (1981), Sorensen and Jensen (1986), Silvester and Hsu (1989) and others are useful, along with the information presented by PIANC (1985). The latter present information on 161 breakwaters constructed in 27 different countries.

Following preliminary design, breakwater performance, in particular armour stability, maximum runup, wave reflection and wave transmission, is normally checked with a physical model in a wave flume. Physical models are constructed geometrically similar to the full size structure. It is assumed that as gravitational forces dominate, models are scaled with the Froude model law. Viscous forces are made negligible by selecting linear scales of sufficient size and by careful selection of core material for the model. Scale ratios usually depend on the maximum wave height and water depth that can be reproduced in the flume along with the available size of model armour units, but are typically within the range 20 to 50.

Consideration needs to be given to environmental effects associated with visual impact, occupation of the seabed, changes in wave patterns and the mobilisation of fines during core dumping. Wave reflection and diffraction may impact on navigability, and the breakwater itself may influence currents which could cause scour and alter bathymetry, particularly at the advancing end during construction (Hickson and Rodolf 1951, Fried 1965, Kaplan 1971, Thorpe 1984, Van Damme et al. 1985, Sorensen and Jensen 1986). This can increase the amount of material required or affect the stability of nearby structures.

As the bulk of a breakwater is underwater, construction and inspection is difficult, especially under rough seas. The practicalities of translating an idealised model into a full size structure should be borne in mind. Moore (1989) presents useful guidance on “constructability” in which design is planned for construction. A design that has worked well in shallow water should only be extrapolated to deep water if due consideration has been given to the practicalities of construction and the possibly significant differences in wave climate and structural behaviour. Breakwaters in deep water should be model tested with an appropriate sea state.

3. Design and construction issues

3.1 Sea state

Wave direction, either normally incident or oblique, refers to the direction of wave travel with respect to the breakwater axis. In comparison, length of wave crest, either long or short, is a three-dimensional phenomenon. It is imperative that model tests represent the sea state correctly, particularly for large breakwaters and when a two-dimensional wave flume is used. The cost of refining the design wave height through additional data collection or analysis has to be weighed against the cost of over or under-designing the structure.

The significant wave height, \( H_s \), is often used to characterise an irregular sea and is usually defined as the average of the one-third highest waves. The USACE (1984) recommend that \( H_{1/10} \), defined as the average of the ten highest waves, be used for the design of rubble mound breakwaters. Vidal et al (1995) suggest a new wave parameter in stability formulae, \( H_n \), defined as the average height of the \( n \) largest waves in a sea state. They show that both the wave height distribution and the total number of waves to achieve a given damage level are taken into account with this parameter and suggest that \( n \) is
approximately equal to 100, but recommend further research. Jensen et al (1997) suggest that $H_{1/20}$ should be used for rock slopes and that $n$ should equal 250.

When there is a paucity of wave data and there are shallow water conditions, it may be appropriate to use the maximum height depth-limited wave for design. If so, then it is important to ensure that storm surge, setup or future changes in bathymetry through dredging or scour do not affect this assumption.

3.2 Wave-breakwater interaction

The wave climate along the entire length of the breakwater should be checked to see if the bathymetry concentrates wave energy anywhere. Where appropriate the breakwater can be aligned to minimise such concentrations. Concentrations of wave energy can result from a change in wind direction during a storm, with wind waves and swell arriving from different directions.

Although most energy is dissipated by the breakwater, the amount of wave reflection should be checked to assess effects on navigation and toe scour. Large scour depths are often associated with a high reflection coefficient (Sawaragi 1967, Eckert 1983). A layer of rock extending seaward from the toe can protect against scour and may cause waves to break before reaching the seaward slope, reducing the design wave height (Van Damme et al 1985).

The relationship between slope angle $\theta$, wave height $H$ and offshore wave length $L_o$ can be expressed in terms of the Surf Similarity Parameter ($\xi$) or Iribarren Number (Ir) given by:

$$\xi = \frac{\tan \theta}{(H/L_o)^{1/3}}$$

The usefulness of $\xi$ has been demonstrated in the widely referenced work of Battjes (1975). He gave a physical interpretation of $\xi$, showing that for waves that break on the slope, it is approximately proportional to the ratio of $\theta$ to the local steepness of the breaking wave. Resonance of uprush and downrush with wave period occurs when $\xi$ is within the range 2 to 3 and hence breakers are of the plunging or collapsing types (Bruun 1985).

Because of the extensive amount of data on maximum runup, $R_u$ for smooth slopes, the use of the relationship:

$$R_u(rough) = r.R_u(smooth)$$

is often advocated, where $r$ is a roughness and permeability correction factor. $R_u$ is measured relative to the still water level (SWL), defined by the USACE (1984) as the elevation that the surface of the water would assume if all wave action were absent. Stoa (1978) and Losada and Giménez-Curto (1981) present values of $R_u$ for armoured slopes and show that it is not possible to associate a single value of $r$ with each type of armour. Van der Meer and Stam (1992) present empirical design formulae for runup on smooth and rock slopes. Good results have been obtained for the numerical modelling of wave runup on mild slopes (Christian and Palmer 1997).

3.3 Cross-section configuration

3.3.1 Slope angle

Side slopes are generally as steep as possible to minimise the volume of core material and to reduce the reach of cranes working from the crest (Moore 1989). However it may be possible to develop a less steep slope if the cranes operate from a barge. Slopes are typically within the range 1V:1.5H to 1V:3H and influence the amount of interaction between armour units. As the angle increases, the contribution to stability from friction and interlocking also increases due to the squeezing or increase in slope-parallel forces applied by adjacent units. There is however a corresponding decrease in the slope-perpendicular component of self-weight (Price 1979). This implies optimum slope angles for maximum interaction and stability (Losada and Giménez-Curto 1982).

3.3.2 Layer thickness

Armour stability generally increases with an increase in armour layer thickness. All layers should be constructed thicker than designed to allow for settlement, provided the armour will tolerate settlement.
without breakage. Allowance should also be made for the initial settlement that occurs as the units nest into a more stable position under wave action.

3.3.3 Crest elevation
The elevation of the crest should be the minimum at which acceptable overtopping occurs. This should be based on maximum wave runup, with an allowance for freeboard and post-construction settlement.

3.3.4 Equilibrium profiles
Equilibrium, S-shaped or “reshaped” profiles develop with the redistribution of material under wave action. Extensive damage to a breakwater with a plane seaward slope usually results in such a profile, with material removed from a zone centred near the SWL being deposited at the toe. The profile continues to develop until equilibrium between erosion and accretion is reached. To ensure equilibrium is achieved they are usually restricted to sites with small tidal ranges. The similarities between beach step profiles and equilibrium profiles has been demonstrated by Van der Meer and Pilarczyk (1987), who also present empirically derived data on the geometry of the reshaped profile.

3.3.5 Berm breakwaters
Berm breakwaters are constructed with a horizontal berm at or near the SWL, with the berm occupying the full width of the armour layer. This is thought to give lower internal velocities than a conventional armour layer and so smaller armour, either rock or riprap, can be used. The seaward profile retains its original overall shape, but the berm compacts under wave action into a more stable form (MacIntosh and Baird 1987). The development of the seaward profile has been physically and numerically modelled by Van Gent (1995).

3.4 Armour layer
3.4.1 Armour type
Armour units can be classified as compact, interlocking or hollow according to their shape and means of obtaining stability. Compact armour units which include rock and riprap use their weight, and to a lesser extent friction, to resist wave action. In comparison, interlocking armour units, such as the Dolos (Figure 2) rely principally on interlocking with adjacent units. A more porous armour layer is created enabling a higher proportion of wave energy to be dissipated within the voids between armour units. This however tends to force the armour units apart. The breakage of several adjacent interlocking units can lead to failure of the whole armour layer.

FIGURE 2: Dolos armour unit.

Hollow armour units such as the SHED (Shephard Hill Energy Dissipator) (Figure 3) and Accropode are placed in a regular pattern forming a single layer (Kobayashi and Kaihatsu 1995). The Accropode was developed by Sogreah Consultants (France) in the early 1980s. The internal void ensures that satisfactory space is always available for energy dissipation, unlike compact and
interlocking armour units where porosity is dependent on their relative positions. Like interlocking units, hollow units are susceptible to differential settlement of the underlayer.

The USACE (1984) list some of the many types of armour unit available. They present a logic diagram for preliminary selection which includes royalty costs and the availability of forms as criteria. Bruun (1985) presents a summary and evaluation of various armour types.

FIGURE 3: SHED armour unit.

Precast concrete armour units are used when suitable rock is unavailable or, according to the survey results of PIANC (1985), when $H_s$ exceeds approximately 5m. Their results also show that they generally weigh between 6t and 50t. The choice between rock and concrete may be influenced by a desire to blend in with existing structures or natural features.

The Tetrapod, the first of the “engineered” precast concrete armour units, was developed by the French company Neypric in 1953. This followed a film viewed by PIANC in 1949 showing the lifting of armour rock during downrush. Rather than resist this through self-weight, Neypric sought to improve permeability and interlock between units (Dock and Harbour Authority 1957). Armouring of the extension to Wellington airport runway in 1955 was one of the earliest projects to use Tetrapods.

The Dolos (plural Dolosse) was developed in 1966 for rehabilitating the damaged breakwater at the Port of East London, South Africa (Merrifield and Zwamborn 1967) (Figure 2). It was decided to develop a new unit because of patents and costs of existing types. Wooden models were used to refine the shape with the criteria of achieving high void to solid ratio and interlocking. Dolosse have been the subject of much research and insitu monitoring, because of the large number of failures and the need to provide information to support ongoing management and maintenance of the many existing structures armoured with them.

For rock it is normal to specify a tolerance on armour weight, such as 25% of the nominal weight (Van Oorschot 1983), and a limit on the maximum percentage of armour weighing less than the nominal weight (typically 50-70%). Riprap differs from rock in that a wide range of particle sizes is present and the USACE (1984) recommend that it be used only where the design wave height (average of the highest 10% of all waves) is less than about 1.5m. Construction specifications developed from model testing must recognise that grading can not be checked in the field by sieving.

Careful consideration must be given to the elevation at which the primary cover layer is terminated. According to the survey of PIANC (1985) it is typically equal to $1.5H_s$ below the SWL. Armour below this level usually consists of rock, even when concrete units are used above. Losada and Giménez-Curto (1981) present useful data on maximum rundown, the lowest elevation of the runup tip, which is not widely reported, and can be used to determine the depth of termination.

It is unusual to use different types of precast concrete armour unit within the same cover layer. Foster (1985a) describes problems interfacing Dolos and Tribar units during construction, especially near the SWL where most wave action is concentrated. Armour weights may vary along the length of a breakwater in accordance with a smooth variation in water depth and hence wave height (Depuy and Stickland 1976), but it may be more economic and practical to adopt a constant weight.
Transitions are required at changes in slope, typically on the leeward side where the flatter slope near the head is steepened to match that of the trunk (Pope and Clark 1983). Care must be taken with transitions between new and existing armour, particularly for oblique wave attack and where the new armour differs in weight from the existing. Baumgartner et al (1985) and Markle and Dubose (1985) describe the use of rock buttresses to support Dolos placed to rehabilitate damaged breakwaters. For cubes there can be difficulties nesting the outer and inner layers (Groeneveld et al 1983) and also units in the same layer (Van Damme et al 1985).

### 3.4.2 Armour size and stability

It was not until 1933 that the first formula was presented, by De Castro and Briones, to aid the designer in selecting a rock armour weight for a given wave height. This was followed in 1938 by the semi-theoretically derived Iribarren Formula. Model tests performed at the Waterways Experiment Station (WES) of the USACE in the 1950s using cubes and Tetrapods showed that the inter-unit friction coefficient, an independent variable in the formula, varied widely with armour unit shape and method of placement, as well as between similar tests (Hudson 1959). Further use of the Iribarren Formula at the WES was therefore abandoned, although it is still used elsewhere today. R.Y. Hudson, Chief Engineer of the WES Wave Action Section, then developed the well known Hudson formula:

\[
W = \frac{w_r H^3}{K_D (S_w - 1) \cot \theta}
\]  
(3)

where \( W \) is the dry weight of an individual armour unit in the primary cover layer, \( K_D \) is an empirical stability coefficient which is intended to account for all of the factors affecting armour stability that are not included as variables, \( w_r \) is the unit weight of the armour material, \( H \) is the design wave height, \( S_w \) is the specific gravity of the armour unit and \( \theta \) is the slope of the structure. Equation (3) is applied to riprap by substituting the median weight, \( W_{50} \) for \( W \) and using the appropriate value of \( K_D \).

Hudson derived (3) by evaluating the fluid drag and inertial forces acting on an individual armour unit in the primary cover layer, and equating these to the weight of the armour unit. The friction between units was ignored but its effect was assumed to be included in \( K_D \). Values of \( K_D \) are determined from model tests by measuring the value of \( H \) at which some threshold level of damage is exceeded. Values are presented by the USACE (1984) and others and are periodically revised.

More than 20 other formulae exist, some including inter-unit friction and wave period, \( T \) as variables (PIANC 1976). Most, including (3), are for the condition of long crested waves moving perpendicular to the axis of a straight breakwater. Losada and Giménez-Curto (1979) present design (interaction) curves based on model tests showing the relationship between \( H \), \( T \) and \( \theta \) for various armour types. Holtzhausen and Zwamborn (1993) present statistically derived design formulae for Dolos on a 1:1.5 slope in deep water and Burcharth and Liu (1993) present design charts on the stability and strength of Dolos.

Van der Meer (1995) presents the following empirically derived formulae for sizing rock armour:

\[
H_s / \Delta D_{s50} \times \sqrt{\xi} = 6.2P^{\alpha_{18}} (S / \sqrt{N})^{0.2}
\]  
(4)

for plunging waves \( (\xi < 2 - 4) \), and

\[
H_s / \Delta D_{s50} = 1.0P^{\alpha_{11}} (S / \sqrt{N})^{0.2} \sqrt{\cot \alpha_s \xi} P
\]  
(5)

for surging waves, where \( \Delta \) is the relative mass density, \( D_{s50} \) is the nominal armour diameter, \( P \) is a dimensionless permeability factor and \( N \) is the number of waves to achieve the damage level, \( S \) given by:

\[
S = A_s / D_{s50}^2
\]  
(6)

where \( A_s \) is the cross-sectional area of erosion.

According to Van der Meer (1995), the value of \( P \) varies from a minimum of 0.1 for an armour layer with a thickness equal to \( 2D_{s50} \) on an impermeable core, to a maximum of 0.6 for a homogeneous structure consisting only of rockfill. He recommends that a sensitivity analysis be performed on all parameters as part of the design process.

Increasing the waist thickness of the Dolos, defined as the ratio of the width of the octagonal central stem (measured between flats) to the longest dimension of the unit, reduces stresses within the unit but decreases stability (Scholtz et al 1983, Zwamborn and Scholtz 1987, Burcharth and Liu...
Burcharth and Brejnegaard-Nielsen (1987) report the same result, but only for damage greater than 5% within the zone SWL±Hs. This is apparently due to less interlocking and lower armour layer porosity. The results of limited model tests comparing Accropode and Dolos are presented by HoltzhAUSEN and Zwamborn (1991). They conclude that Accropode are more stable than Dolos for long period waves (ξ>3), but have similar stability for shorter waves.

Users of stability formulae must ensure that their definition of “acceptable damage” is consistent with that for which each formula and its respective empirical coefficients were derived. For example Iribarren accepted that wave action would form the equilibrium slope angle if it was initially too steep.

3.4.3 Definition and assessment of damage

A pre-determined, acceptable level of damage is tolerated under design conditions. Damage is generally defined as the removal or breakage of individual armour units, or sliding of the armour layer en-masse. For precast concrete units, rocking is classed as damage too since this can lead to breakage and progressive failure of the armour layer. For breakwaters subjected to heavy overtopping or internal wave transmission, the stability of armour near the top of the leeward armour layer may be relevant (Walker et al 1975, Foster 1985b, Anderson et al 1993).

When using a model to determine threshold damage conditions, a small amount of damage must be observed which is generally referred to as the “no damage” condition. It is important to assess how damage observed in the model will relate to performance of the full size structure and to interpret model results accordingly.

One approach is to express the number of units damaged as a percentage of the total number of units on the slope. One drawback is that the value calculated depends on the number of units in the armour layer, making comparison of different cross sections and armour types difficult. A better approach is to express damage as the percentage of armour units displaced from the zone of active armour removal (Ouellet 1973). This zone extends from the middle of the crest down the seaward side to a depth of one characteristic wave height below the SWL. Ouellet (1973) defines the characteristic wave height as that corresponding with 1% damage. The “zero damage” or “no damage” condition is typically within the range 0-5%. For riprap slopes, including equilibrium profiles, it is more appropriate to describe movement of a volume of material using (6) or similar.

Because precast units can break as the result of rocking, a more relevant approach is to use combinations of rotation and displacement to classify the movement of individual units. A damage distribution can then be determined by comparing percentage damage to damage class, where percentage damage is expressed in terms of the total number of units within the zone of active armour removal. Based on the susceptibility of each type of unit to breakage, allowable damage distributions can be used to evaluate model test results (Partenscky et al. 1987).

3.4.4 Armour unit strength

Because of the complex time-dependent loading it is difficult to accurately design a reinforcing scheme for interlocking concrete units. Disadvantages of reinforcement are the substantial increase in cost and the possibility of cracking resulting from corrosion. The latter can be eliminated if synthetic reinforcing material is used.

Armour unit strength can be assessed with full scale drop tests to simulate inter-unit impacts and forces. Groeneveld et al (1983) suggest that large concrete units are more susceptible to breakage than small units, due to reduced reserve strength and higher temperature gradients during curing. However PIANC (1985) report that their survey shows that breakage of larger precast concrete units is no more frequent than that of smaller units.

Alternative methods of measuring stress in model units are described by Burcharth et al (1991). One approach is to observe the number of rocking and displaced units and then estimate the number that would break in the full size structure. A second approach is to measure forces at several locations within selected units which are then extrapolated elsewhere within the unit using a numerical model. A further approach is to scale the strength of the model armour unit material. This avoids subjective damage interpretation, but means that broken units can not be re-used.

Impact forces and the temporal and spatial distribution of acceleration have been the subject of experimental research by Van der Meer and Heydra (1991), Bürger et al (1993) and others. D’Angremond et al (1995) present empirical design curves for the structural design of Tetrapods,
based on the measurement of stresses in model units. Turk and Melby (1995) present the results of model tests with Dolos and conclude, amongst other things, that rocking and the associated inter unit impact produces impact stresses that, when combined with static and wave induced stresses, can often be high enough to exceed the concrete strengths typically found in full size units.

Pope and Clark (1983) and Phelp et al. (1995) describe techniques used to monitor full size Dolos armoured breakwaters. Kendall and Melby (1993) present the results of monitoring Dolos on the Crescent City breakwater over a six year period. They conclude that static stress is the most significant structural design parameter for these large (38 tonne weight, approximately 4.5 m overall length) Dolos units and report that static stress increases over time as a result of subtle movements and inter unit wedging. Luger et al (1995) surveyed 357 broken Dolos units on seven breakwaters and classified each unit according to one of six breakage modes. They report that 89% of breakages occurred near the fluke-shank intersection and recommend incorporating a large fillet extending to mid-shank.

3.4.5 Handling and placing armour units

Rock armour should be placed with the longest dimension perpendicular to the slope. Although interlocking concrete units are usually oriented randomly, the desired packing density (number of units per unit surface area) is ensured by specifying insitu coordinates and placing each unit individually. Consideration should be given to the number of crane positions and hence movements required when determining armour unit positions. In some cases it may be economic to use a smaller number of larger units. This would involve less handling, with a commensurate reduction in both construction time and breakage.

Placing from a barge may be more time consuming than placing from land because of loading and positioning and the prerequisite of a suitable sea state. It may also result in a higher rate of breakage during placement and limit the range of permissible working conditions. Despite this, breakage generally occurs irrespective of the particular placement method.

If an armour layer is designed assuming regular maintenance then consideration should be given during design to the lifting and placing equipment required and the methodology. Lifting slings and hooks may be difficult to attach to units insitu, especially underwater. Cast-insitu lifting eyes must be maintained in good order since retrofitting is difficult.

3.4.6 Toe support

The critical sea state for toe design, and hence model tests, will usually be at low tide, provided wave heights are not depth-limited. Scour of the leeward toe can result from overtopping (Walker et al 1975). A toe structure of placed armour units, often rock, provides direct support of the armour layer and also surcharges the seabed against a slip circle failure. The literature contains numerous reports on armour layer failures initiated by loss of toe support or leaching of a sandy seabed, along with rehabilitation methods (Danel and Greslou 1963, Kjelstrup and Bruun 1983, Smith and Gordon 1983, Silvester and Hsu 1984, Thorpe 1984, Read 1986, Sorensen and Jensen 1986). Exposure of the core can result, leaving it vulnerable to wave attack. Fredsøe and Sumer (1997) describe scour mechanisms near the head and present empirical formulae for sizing protective rock armour.

A toe structure formed from cast insitu concrete is essential with hollow units, and is why they tend to be restricted to shallow water. In some cases the stability of interlocking units is improved by modifying the shape of a unit such as the Dolos (Van Dijk et al 1983) or by special orientation. From a construction point of view, the latter is only practical in shallow water. If a geotextile is used, consideration must be given to the practicalities of placing it underwater. Care must also be taken to prevent it being undermined.

3.4.7 Special considerations near the head

The seaward end of a shore connected breakwater is termed the “head” and is circular in plan. Difficulties with construction in deep water and high exposure to storms need consideration, along with concentrations of wave energy due to diffraction. A wave trough on the leeward side coincident with maximum runup on the seaward side may create a head for internal flow, dislodging leeward armour. Vidal et al. (1991) provide guidance on required armour weights at breakwater heads.

Because the head is wider than the trunk it is possible to deflect it either inwards or outwards. The choice should consider the effect on the redirection of wave energy, and hence navigation, harbour
circulation patterns and sediment transport. Consideration should be given to whether or not the completed breakwater can be easily extended in the future.

3.5 Underlayer

The underlayer acts as a foundation for the armour layer and as a filter to prevent the core being eroded. It also protects the core during construction. The size of rock to be used in the underlayer is expressed as some proportion of W. This ensures adequate interlock between adjacent layers and also that the gradation reduces the potential for internal erosion. Armour stability generally increases with an increase in underlayer permeability.

Hedges (1984) has suggested the use of “binders” (shear keys) between the armour layer and underlayer to prevent sliding of the former, but this is not common practice. The use of this technique is described by De Carvalho (1964) in which specially shaped precast concrete “cast through stones” were used to link two layers of rock armour.

3.6 Core

Core permeability affects wave runup and armour stability, with low permeability causing higher runup and lower stability (Timco et al 1985, Van der Meer and Pilarczyk 1987). Although a dense core reduces wave transmission, a minimum of fines should be used so as to avoid internal erosion. A graded filter prevents removal of fines from the core but does not prevent the internal redistribution of material which may cause differential settlement. The selection of core material generally uses empirical guidelines based on past experience.

Penetration into the seabed may result in greater quantities being required than estimated during design. It may also lower the crest, increasing the risk of overtopping. Significant consolidation of the seabed can occur during construction (Fang 1982) but can be managed by staging construction such that consolidation of the seabed occurs as construction proceeds. Because the core is deposited in a loose state, shakedown and settlement occurs initially under wave action, which may damage armour units. Earthquake or wave induced liquefaction may be relevant design considerations.

Techniques used to analyse equilibrium profiles can be used to assess the behaviour of the exposed core during a storm. The risk of damage is usually limited by specifying the maximum length of core which can be left exposed at any time during construction.

3.7 Superstructure

A superstructure consisting of a concrete cap block or wave wall (crown wall) provides access for maintenance or cargo handling, and is essential when hollow units are used. It can also reduce the amount of overtopping and strengthen the crest. It is usually poured after differential settlement of the core has reduced to an acceptable rate.

Dynamic loadings result from wave impact on the seaward face and uplift pressure on the base. The latter can be relieved with vent holes, but this requires a commitment to keep them clean. Extra vertical load may also result from wave overtopping. A parapet intended to prevent overtopping may trap waves causing additional downrush and hence loading on armour units.

Sliding is resisted by friction between the base and the underlayer or core. Difficulties arise in trying to reproduce this with physical models, especially when the relative size of the material has been altered to satisfy hydraulic scaling requirements.

Jensen (1983) presents the following guidelines:

- the wave wall should not extend above the level of the seaward armour (to minimise the forces on the superstructure)
- key the cap block into the core with a heel
- extend the core up to the underside of the cap block, and
- extend the rear of the cap block past the leeward slope to direct overtopping jets of water past the leeward armour to fall directly on the water surface.

The material beneath the base should either have very high permeability to relieve uplift pressures, or be of very low permeability to prevent pore water entering. The former may however dislodge armour on the leeward side. Other useful comments are made by Baird et al. (1981). Hamilton and
Hall (1993) present results of model tests with crown (wave) walls showing that short, stabilising legs at the seaward and leeward ends of the superstructure substantially increase stability and that stability also increases with decreasing wall height. Like Jensen (1983), they recommend direct placement on the core with armour extending up the front of the wall.
4. Conclusions

The principal function of a rubble mound breakwater is to protect a coastal area from excessive wave action. Despite more than six decades of applied research, design continues to be based largely on experience and physical modelling of the proposed structure. Interaction between armour units is poorly understood and, as a consequence, the design of armour layers remains largely empirical. Armour stability is affected by armour weight and shape, underlayer size and shape, underlayer and core permeability, toe support and the detailing of the superstructure. Armour layer damage must be clearly defined, interpreted and reported. Model tests and empirical formulae should be applied and interpreted with care. It is imperative that model tests represent the sea state correctly. The practicalities of translating an idealised scale model into a full size structure should be assessed, with due consideration given to the construction and maintenance phases.

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6. References


7. Notation

A_e Cross-sectional area of erosion
D_{50} Nominal armour diameter
H Wave height
H_n Average height of the n largest waves
H_s Significant wave height
Ir Iribarren Number
K_s Stability coefficient (Hudson Formula)
L_o Offshore wavelength
N Number of waves
P Permeability factor
R_u Maximum runup
r Roughness and permeability correction factor
S Damage level
S_s Specific gravity of the armour unit
SWL Still water level
T Wave period

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$W$  Dry weight of individual armour unit in the primary cover layer  
$W_{50}$  Median weight of graded riprap  
$w_i$  Unit weight of armour unit material  
$\Delta$  Relative mass density  
$\xi$  Surf Similarity Parameter  
$\theta$  Slope of the structure.