PART III

Coastal Structures

CHAPTER 67

Analysis of practical rubble mounds

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ABSTRACT

This paper describes results from an unusual research project completed under Topic 3R2 of the European Union's MAST research project G6-S Coastal Structures. In this project, data were collected from the major European hydraulics laboratories on the hydraulic and structural responses of example rubble mound breakwaters and sea walls that each laboratory had previously studied in wave flume or wave basin tests. The main responses considered here are:

a) Main armour stability, given by measurements of armour movement and/or displacement under wave action.

b) Wave overtopping, described by the number of waves passing over the structure crest, or by the mean overtopping discharge;

The paper describes some of the analysis of armour stability and hydraulic performance of these structures, and explores the potential to develop general conclusions from ad hoc studies. This paper develops some of the analysis described initially within the G6-S project by Allsop & Franco (1992), but also re-considers and revises some of the early analysis and initial conclusions.

1. OUTLINE OF THE STUDY

Within the MAST I project G6-S Coastal Structures, work under Topic 3R addressed the performance of rubble mound breakwaters. Such structures may be used to protect harbours, cooling water intakes or outfalls, and related areas of coastal development, against wave action. Rubble mound breakwaters are formed by constructing the inner part of the mound, termed the core, from quarried rock. The core is protected against erosion by armour layers, supported by filter or under-layers. The size of the armour is closely related to the height of the design waves. Such structures may include a crown wall, a number of armour and underlayers on the seaward and lee faces. They are usually designed with a number of different levels from foundation and toe layers to crest armouring (Fig 1).

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Rubble mounds are used to reduce levels of wave activity by limiting wave overtopping or transmission, and/or to protect against erosion. The degree of wave reduction needed, and hence the hydraulic responses required, depend on the requirements of the harbour or coastal development. The structural design of the breakwater must ensure that it can serve its stated purpose over its full design life, and that damage to the structure is therefore kept below accepted limits.

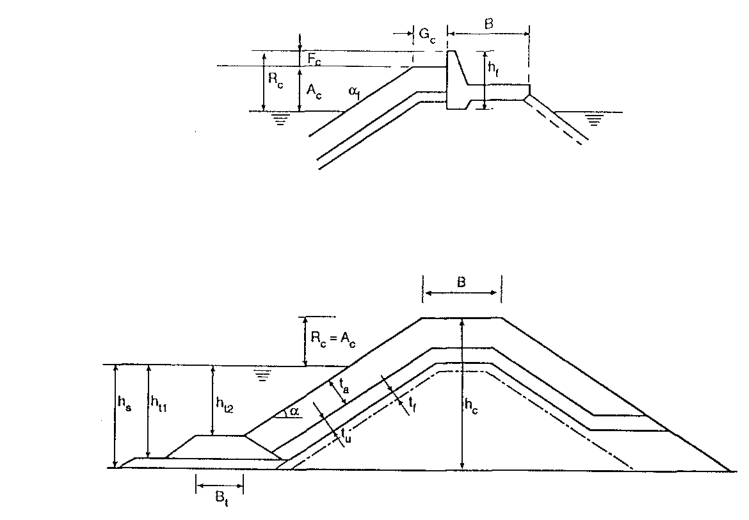

Figure 1 Rubble mound breakwaters, main geometrical parameters

The main methods used in the design of rubble mound breakwaters are based on empirical formulae, supported by results from hydraulic model tests. Such methods are therefore derived for simplified structure sections tested under normal wave attack. Very few methods address the stability of structures incorporating complex or "non-standard" details; under oblique wave attack; at and around the outer breakwater end or roundhead; or at junctions with dis-similar construction.

These types of structures are used worldwide wherever quarried rock is available in adequate sizes and quality for construction of coastal structures. Many such structures have however suffered significant damage, with an apparent peak between about 1977 and 1988. In analysing these failures, and difficulties with similar structures, it has become clear that analysis and design methods have been insufficiently reliable. A programme of studies were therefore proposed to the European Union. The programme that was contracted was somewhat restricted, but included key elements of the original proposal.

1.1 Work in G6-S Coastal Structures

The MAST I research project "G6-S Coastal Structures" addressed techniques available for the analysis and design of coastal and harbour structures such as sea walls, revetments, and breakwaters. The research covered three technical topic areas:

Topic 1. Wave action on and in coastal structures;

Topic 2. Wave impact loading on vertical structures;
Topic 3B/R. Berm and rubble mound breakwaters.

Within this project, studies under Topic 3R addressed the stability and hydraulic performance of rubble mound breakwaters and sea walls using analysis of previous model test data, described here, and by new model tests described by Galland (1994).

The main objective of the desk study described here was to provide information on the stability and performance of rubble mound breakwaters, where possible at singular points. These include: roundheads; junctions; bends; toe and rock berms. The study was based on the collection of data from study reports from the major hydraulic laboratories in Europe. It was agreed that HR Wallingford would design the study approach; that each laboratory would be responsible for the collation of their own test results; data; and that HR collect together and analyse the results. The objectives of the project were to:

a) Identify whether data on responses from practical studies could be used to draw general design guidance;
b) Collect data from many different institutes, and retain in consistent form for analysis;
c) Contrast data from ad hoc studies with predictions made using methods based on idealised structures;
d) Identify new or modified design methods based on these data;
e) Identify gaps in present design information or methods;
f) Provide justification for further research on rubble mound breakwaters.

The approach taken was firstly to identify the main breakwater failure modes and the principal parameters influencing failure, and to develop standardised parameter definitions and notation. These were used to design a database spreadsheet to hold model test data on breakwater structures and responses. Test results were then collected from each partner in the project. In parallel with this work, the prediction methods for the main design parameters were summarised, see Allsop (1993) and these were then used to devise the analysis strategy.

2 DATA COLLECTED

The data analysis was based on responses measured previously in site specific studies conducted by the institutes. The data used were therefore confined to those aspects of structure performance of concern to the designers of the particular structures, and were limited to those combinations of wave conditions and water levels for which the tests had been conducted.

The principal responses recorded were:

a) Toe armour stability, given by measurements of toe armour movement and/or displacement;
b) Main armour stability, again by movement and/or displacement, often using the damage parameters S or $N_{d\%}$.
c) Wave overtopping, described by the number of waves passing the crest, $N_{wo\%}$, or by the mean overtopping discharge, Q;

Each set of results were combined by response types, and then compared with the simple design methods to test their use, and/or to identify whether new prediction methods could be derived. The data collected in this study were too great to handle as a single spreadsheet. During the analysis process, the larger data sets were split into smaller and increasingly more specific sets, mirrored by the sections within this chapter. Typical

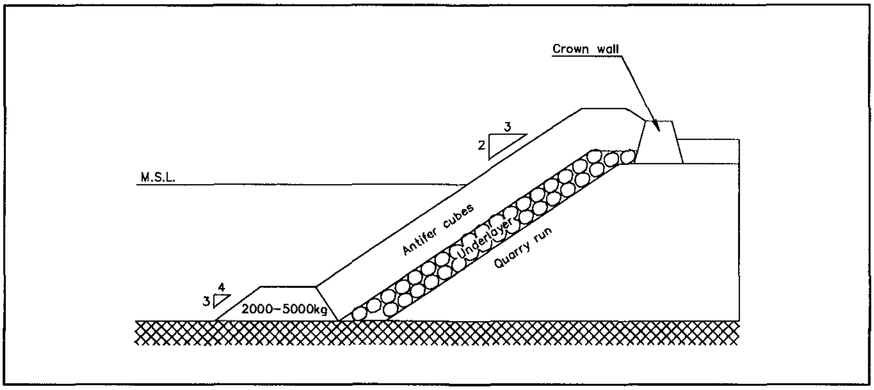

Figure 2 Rubble mound breakwater armoured by Antifer Cubes

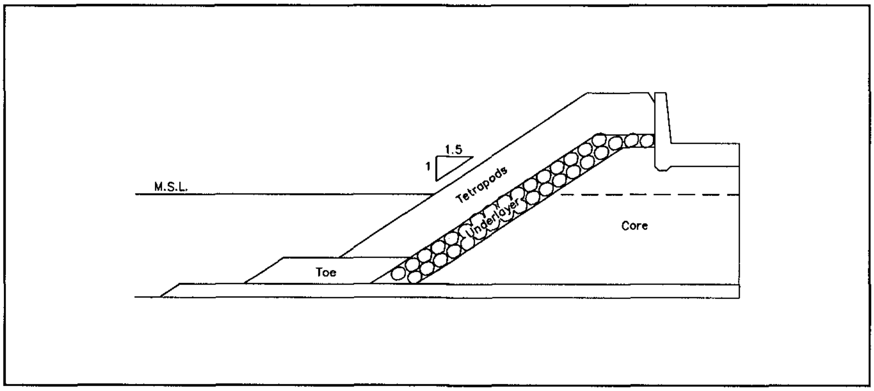

Figure 3 Rubble mound breakwater armoured by Tetrapods

cross-sections through the structure are shown in Figures 2-5. The influences on overtopping and armour damage of armour type, cross-section geometry, and plan configuration have been treated separately. This paper describes only the analysis of armour movement and wave overtopping.

3 ANALYSIS OF ARMOUR MOVEMENTS

Design methods for rubble mound armour layers focus principally on the calculation of the median armour unit mass, M_{50}, or the nominal median stone diameter, D_{n50}, for given levels of armour damage. In most instances damage is defined in terms of erosion area A_e or number (%) of armour units displaced, $N_{d\%}$. Damage may also be described by N_{od} referring to the number of unit displaced related to a width along the breakwater of $1.0D_n$. For a Tetrapod, D_n is 0.65 D where D is the height of the Tetrapod; and for Accropode D_n is 0.7 D. The definition of units displaced N_{od} may be compared with the damage parameter S. Generally S is about 2 N_{od}, but the relationship differs for different armour units laid at different porosities:

For Cubes $S = 1.8 N_{od} + 0.4$ (1a)

Tetrapods and Accropode $S = 2 N_{od} + 1$ (1b)

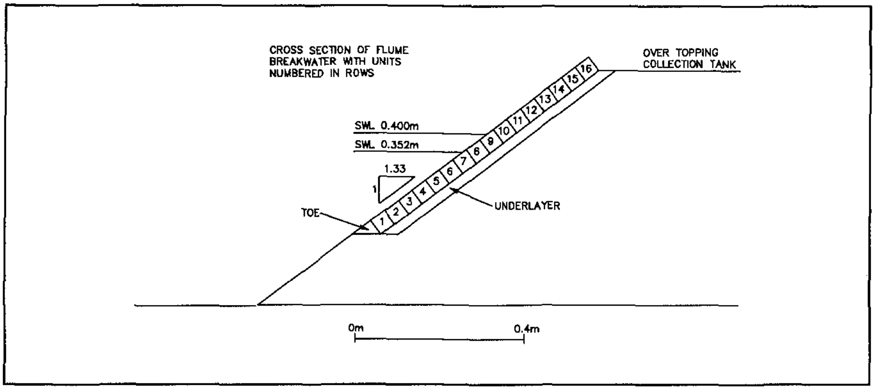

Figure 4 **Rubble mound slope armoured by Hollow Cubes**

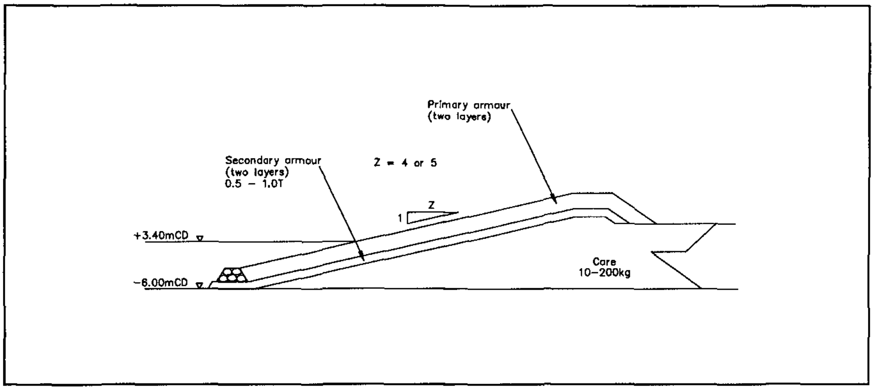

Figure 5 **Rubble mound slope armoured by rock**

The initial analysis of armour response presented by Allsop & Franco (1992) sought to identify the effects of wave obliquity and trunk versus roundhead on armour damage. Data from tests on structures armoured with Rocks, Tetrapods and Cubes formed the main body of the analysis. The data was collected from diverse sources, and much effort was expended to try to harmonise values of the input parameters for this analysis. Damage was presented in the database as displacement in % of units related to a certain area, or as the damage parameter $S=A_e/D_{n50}^2$ in which A_e is the area eroded around SWL. When the level of displacement was given, this was often divided into classes in relation to the nominal diameter D_{n50} of the armour unit:

N_{D1}: units displaced less than $0.5D_{n50}$
N_{D2}: units displaced more than $0.5D_{n50}$ and less than $1D_{n50}$
N_{D3}: units displaced more than $1D_{n50}$

Sometimes the number of rocking units were also given. Comparisons with other data sets and with predictions methods demanded that damage be presented in a consistent way, and in this analysis, damage was always defined by S. Unfortunately, values of damage have not often been expressed as S in the past, so an alternative approach was needed. In the initial analysis, a simple method was suggested to relate damage in these various classes to S using:

$$S = 0.8\ (0.25N_{D1} + 0.75N_{D2} + 1.0N_{D3}) \qquad (2)$$

Some of the data sets contain information on very small movement, perhaps as small as $0.1D_{n50}$, but these were not included in this anlysis. In many cases the collaborating laboratories themselves combined categories N_{D2} and N_{D3}.

A damage formula developed by Van der Meer (1988) from the Hudson formula was used to compare damage data with values of Hudson's stability coefficient, K_D:

$$H_s/\Delta D_{n50} = a\ (K_D \cot\alpha)^{1/3}\ S^b \qquad (3)$$

where for rock armour a=0.67, b=0.16
and for Tetrapods and Cubes a=0.69, b=0.14

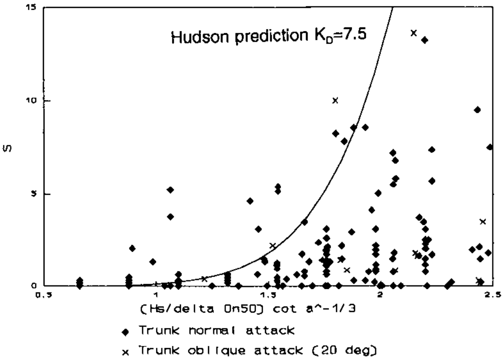

Figure 6 Stability of cube armour, analysis includes movement in categories N_{D2} and N_{D3}

The stability of cube armour is illustrated in Figure 6, which shows the damage S against $H_s/\Delta D_{n50} \cot\alpha^{-0.333}$. The plot shows a great deal of scatter with many data points above the prediction line using K_D=7.5. Analysing measurements in category N_{D3} only, it was clear that relatively few small movements had been recorded, and the damage vaules calculated for N_{D3} only lie in very similar positions.

Designers of breakwaters using concrete armour units have been more rigorous in requiring more information on small movements in recent years, particularly for slender units. The use of these small categories in calculating an equivalent value of S less certain than suggested in eqn (2). It is probable that the use of N_d% alone to calculate S will under-estimate the damage, but the simple method used by Allsop & Franco (1992) to include the influence of small movements appears to lead to significant over-estimates of damage compared to existing prediction methods. This is particularly so where cumulative damage is estimated by summing damage in individual tests. This problem was illustrated when considering damage to Tetrapod armoured slopes.

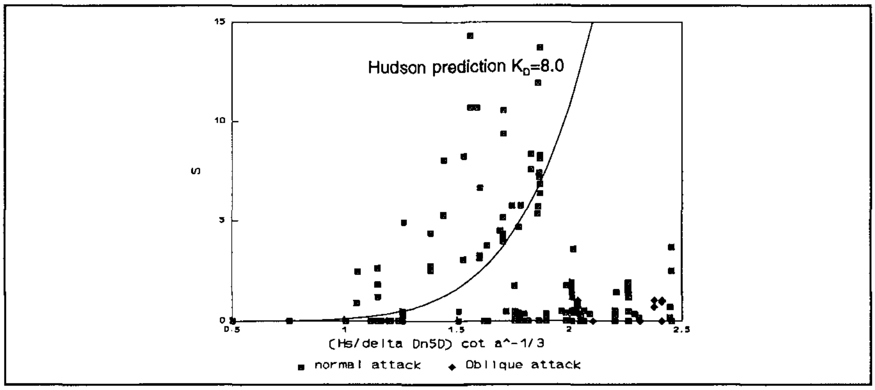

Figure 7 Stability of Tetrapod armour, analysis includes movement in Categories N_{D2} and N_{D3}

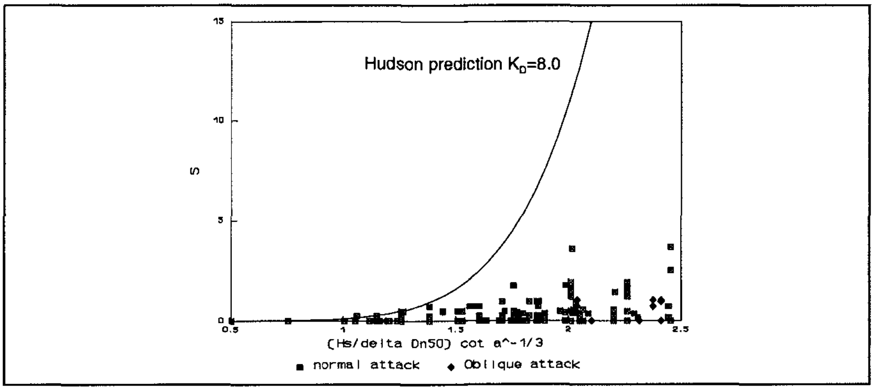

Figure 8 Stability of Tetrapod armour, analysis only includes movement in category N_{D3}

Damage to Tetrapods was also analysed, with damage categories N_{D2} and N_{D3} used to calculate S, plotted in Figure 7. Again the damage appears to be much greater than predicted by K_D=8.0. The affect of plotting points in category N_{D3} only is shown in Figure 8, which shows that the contribution of the smaller movements had a substantial influence on the comparison. Now all the data point lie below the prediction curve.

This exercise leads to a rather disappointing, but not altogether surprising conclusion. Unless the damage definitions used in design formulae precisely reflect those used by design engineers in site specific studies, a correlation of data from these studies with simple formulae may lead to considerable uncertainty, if not confusion. This is however to be expected, as the sophisticated designer and experienced modeller should be expected to use more sophisticated descriptions of structure response than those appropriate for simple design formulae.

4 ANALYSIS OF OVERTOPPING

Wave overtopping may be described by the number or percentage of waves passing over the crest expressed as N_{wo}; or by the mean overtopping discharge per unit length, Q. The data returned seldom identified both responses, so analysis had to concentrate on the two sets of data separately. Most recent research has been concentrated on the prediction of the mean overtopping discharge Q, so most of this section will address this response. Some initial work was however completed on analysing the data returns that only gave the number of waves overtopping, expressed as N_{wo}.

4.1 Number of waves overtopping, N_{wo}.

The test data examined in this study were limited to structures under normal wave attack. Structures were constructed with crown wall elevations equal to or below the front armour crest level. Four sets of data for which the number of waves overtopping had been recorded were analysed.

Example results for slopes armoured with Cubes and Tetrapods under waves of constant steepness of s_m=0.030 were analysed by plotting $\ln(N_{wo\%}/100)$ against R^{*2} as derived by Owen (1980, 1982). The scatter of the data on $N_{wo\%}$ was wide, even when restricted to a single sea steepness. Agreement between measured and predicted values were not good for the Cube structure. A better agreement was found for the Tetrapod armoured structure. The methods used to predict the number of overtopping waves is described by Allsop and Franco (1992). It was concluded that there was little to be gained by extending the analysis. These uncertainties confirmed that the mean overtopping discharge Q gives a more reliable description of overtopping of such structures.

4.2 Mean overtopping discharge, $\bar{Q}$.

The main aim of this analysis was to examine the influence on overtopping discharges of singular points such as crown wall element geometries, armour crest levels, and slope configurations. The overtopping performance of structures armoured with rock, Antifer cubes, Tetrapods, and high-porosity Hollow Cubes, were examined.

The mean wave overtopping discharge depends on freeboard R_c, H_s and T_m. The prediction method developed by Owen (1980) relates dimensionless parameters Q* and R* by an exponential equation with a roughness coefficient, r, and coefficients A and B for each slope angle:

$$Q^* = A \exp(-B\, R^* / r) \tag{4}$$

Where $Q^*=\bar{Q}/(gT_mH_s)$ and $R^*=R_c/T_m(gH_s)^{0.5}$, and for smooth slopes, r = 1.0, and values of A & B have been derived for slopes from 1:1.0 to 1:4.0:

Slope	A	B
1:1.0	0.0079	20.1
1:1.5	0.0102	20.1
1:2.0	0.0125	22.1
1:3.0	0.0163	31.9
1:4.0	0.0192	47.0

Table 1 Values of A and B for smooth slopes, r=1

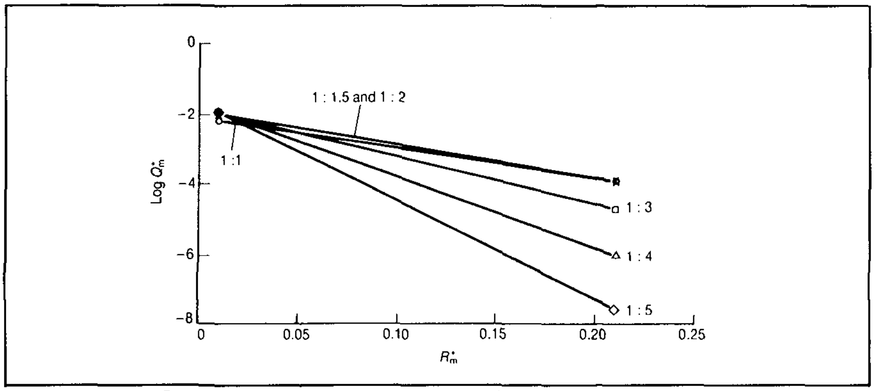

Figure 9 **Overtopping of smooth straight slopes (Owen 1980)**

A design graph such as that shown in Figure 9 can be compiled by plotting Q* as a function of R* and using the constants A and B described in Table 1. For structures with small relative freeboards and/or large wave heights, the regression lines come together at one point, indicating that the slope angle, and relative roughness are no longer effective in controlling the overtopping discharge. The discharge characteristics for slopes 1:1, 1:1.15 and 1:2 are very similar, but overtopping reduces significantly for slope angles less than 1:2.

Owen's method was developed initially from results for smooth slopes only, but the use of the roughness factor, r, allowed its extrapolation to study the overtopping performance of rough, and even armoured slopes. Since 1980, various researchers have explored alternative prediction methods for armoured slopes, see Bradbury & Allsop (1988) and Aminti & Franco (1988), but no new method has proved any more reliable. The advantage of Owen's method is its simplicity, and the ready availability of data to support particular coefficient values. Three alternative approaches have therefore been developed:

a) Use Owen's method and coefficients A and B with r derived from tests with the correct slope geometry;
b) Use Owen's general equation, but with new values of A and B derived for similar cross section, and r = 1.0;
c) Develop alternative equation, with new coefficients for that section.

The overtopping performance of armoured breakwater structures with and without crown walls have been studied. Owen (1980, 1982) showed that in relatively shallow water berms or beaches in front of a structure will reduce overtopping. The toe design of the structures vary somewhat, but it is likely that these differences will have little bearing on the overtopping discharge during the deep water test analysed here. Although the crest detail of the various structures was not identical, the crown wall level was generally equal or below the armour crest, so it might be expected that the variations of crest detail would have had little affect on the overtopping discharge.

Measurements show that there is a good relationship between Q* and R* for all the structures studied. Data for Tetrapod and Antifer cube armoured structures are shown in Figures 10 and 11 respectively. The data presented here show that for the rough armour

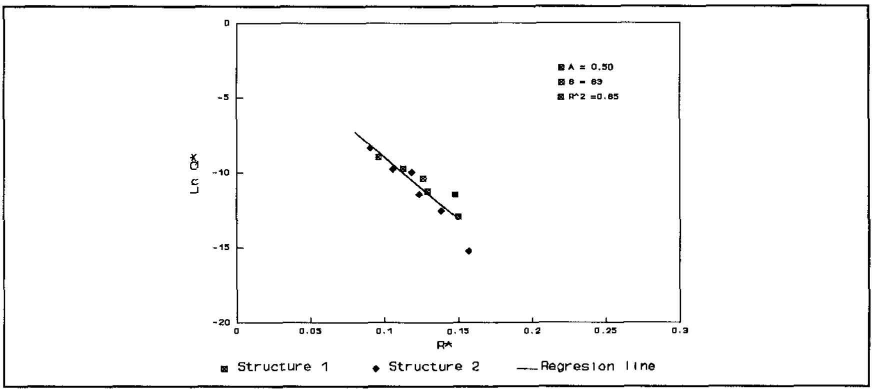

Figure 10 Overtopping of Antifer Cube armoured structure

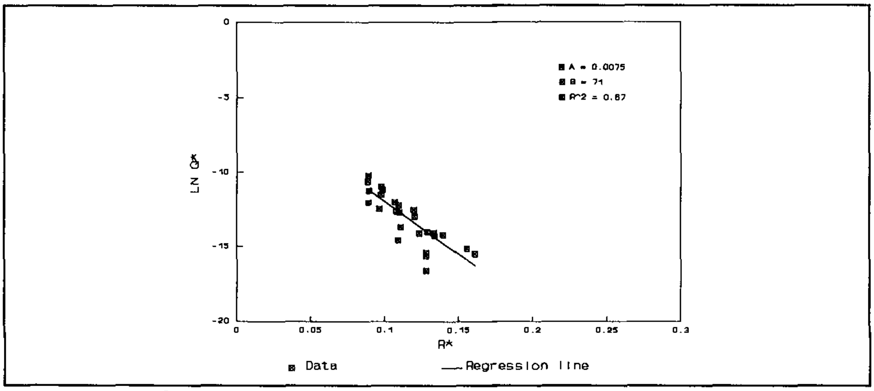

Figure 11 Overtopping of Tetrapod armoured structure

structures analysed the overtopping performance is better described by modifying the coefficients A and B. The overtopping data for rough structures shows that regression lines passing through data points have varying steepness. This change in steepness is due to the increased turbulence and friction caused by the 'rough armour'. The regression lines cross the R* axis at different points depending on the armour type. This is not taken into account in the earlier theory as the coefficient A remains constant for a constant slope angle. This suggests that the hydraulic responses cannot be represented correctly by the roughness coefficient alone.

For armoured slopes, it is therefore suggested that the original Owen formula equation (4) should be used for overtopping, but that the coefficients A and B should be changed depending on the armour type and structure slope. The original Owen method using values of the roughness coefficient is not as accurate as using regression lines for site specific data. The simple Owen method is however very quick and easy to use where little site specific data is available.

The values of the coefficients A and B for rough slopes analysed during the study are tabulated below. The regression lines of the Tetrapod and Antifer cube structures are shown in Figures 10 and 11.

Structure	Slope	A	B
Cob units	1:1.33	0.00839	46.5
Shed units	1:1.33	0.00268	29.9
Antifer Cubes	1:1.5	0.496	82.7
Efficient units	1:1.5	0.016	60.0
Tetrapod	1:1.5	0.0075	71.0

Table 2 Values of A and B for armoured structures, r=1

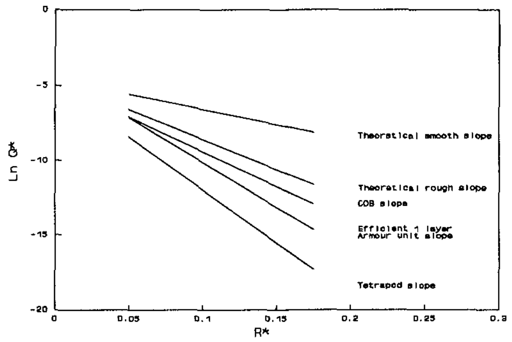

Figure 12 Variation of overtopping with armour type

The overtopping performance of 1:1.5 sloping structures armoured with Tetrapod, Antifer cube and efficient single layer armour units are compared in Figure 12. For similar armour units and structure designs it may be expected that for a given R* the overtopping discharge would be equal. However, Figure 12 shows that as the porosity of the armour units increases, overtopping discharge decreases. The two layer Tetrapod system performs significantly better than the single layer structures. The armour efficiency increases as the relative freeboard increases.

The effect of armour layout is shown in Figure 13. This figure describes the overtopping performance of a single layer and a double layer hollow cube armour system. The armour units had a porosity of about 60%, and were placed to a tight patter on a slope of 1:1.3. The overtopping is reduced for the two layer structure, but not as effectively as on the much thicker Tetrapod armour.

So far only simple structures have been considered. A similar method can be used to assess structures with berms or crest detail. Two test series have been carried out on two layer rock structures and the overtopping performance is shown in Figure 14. The data at the crest of the 1:4 slope shows a slightly lower overtopping discharge compared with the Owen theoretical rough slope. The 1:5 slope crest data shows a small increase

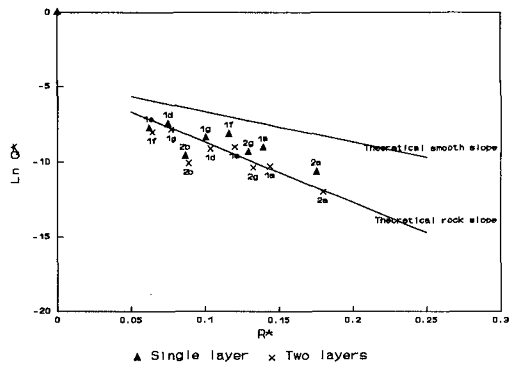

Figure 13 Overtopping of hollow cube armoured structure

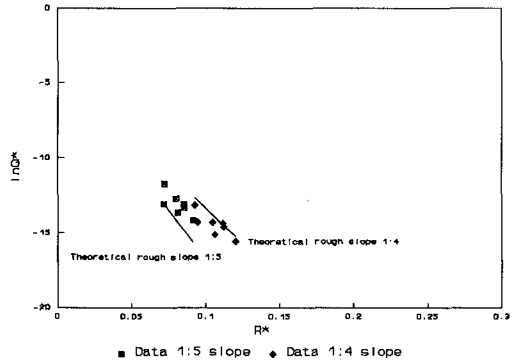

Figure 14 Overtopping of rock armoured structure

in overtopping discharge compared with the Owen theoretical rock slope r=0.5. Within the accuracy of the measurement, the prediction lines show a relatively good fit for the rock armour slopes with shallow slope angles.

The data shown in Figure 14 suggests that the roughness coefficient may be affected by the slope of the structure and the wave conditions. A value of r may be calculated for the measured discharges using Owen's value of A and B for a given slope angle. When the r value is plotted against the Iribarren number I_{rm}, the value of r decreases as I_{rm} increases, this relationship is shown in Figure 15. The result is consistent with the conclusions described earlier noting that both A and B values need to be modified when investigating the overtopping of permeable seawalls when using the Owen formula for structures armoured with 'rough' material.

For the same wave conditions overtopping discharges 10m behind the crest were about one-tenth of those measured at the crest. The crest detail of the 1:4 slope structure was

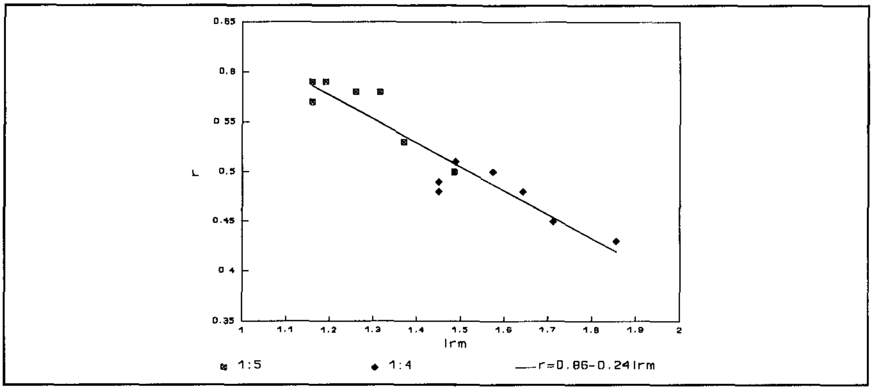

Figure 15 **Variation of the roughness coefficient with Iribarren number for rock armoured structures**

altered to include a wave return wall. The armour crest freeboard A_c = 6.8m and the wave wall freeboard R_c = 8.8m. The performance of this structure can be compared with the structure with no wave wall, R_c = A_c = 8.8m. Although the inclusion of a wave wall shows a higher overtopping discharge compared with the full length armour slope, the wave wall design does have it's advantages. The number of units and the volume of underlayer material required to build the structure is reduced and the structure may therefore be cheaper to build.

Where values of A and B cannot be calculated using site specific data, the analysis has shown that the original Owen formula with values of A and B for various slopes can be used with a roughness coefficient appropriate for the armour concerned. Values of the roughness coefficient for various armour units are given in Table 3.

Armour type	r
Rock	0.5-0.6
Hollow cubes	0.5
Dolos	0.4
Stabits	0.35
Tetrapods	0.3

Table 3 **Recommended values of r for armoured structures using A and B values given in Table 1**

5 CONCLUSIONS

Overall The spreadsheet database worked adequately, but needed plans and sections to convey important information.

Armour on trunks and roundheads Analysis of damage was complex. Initial comparisons show wide scatter, with many tests showing little damage when prediction methods suggest severe damage. Cumulative damage is not given by design methods.

Overtopping Few studies recorded both $N_{wo\%}$ and Q. Selected studies gave data on overtopping allowing new coefficients to be derived. The original Owen formula should be used for rough slopes, but both the coefficients A and B must be changed depending on the armour type and structure slope. The original Owen method is not as accurate as using regression lines for site specific data, but is simple to use where little site specific data is available.

ACKNOWLEDGEMENTS

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