A COMPARISON OF OVERTOPPING PERFORMANCE OF DIFFERENT RUBBLE MOUND BREAKWATER ARMOUR

Tom Bruce[1], Jentsje van der Meer[2], Leopoldo Franco[3] and Jonathan M. Pearson[4]

This paper describes a major programme of tests to establish better the influence of armour roughness and permeability on overtopping. Specifically, the tests determined the relative difference in overtopping behaviour for various types of armour units. Roughness factors γ_f for the database and for use in the neural network prediction of overtopping were determined for rock (two layers), cubes (single-layer and two-layers), Tetrapod, Antifer, Haro, Accropode, Core-Loc™ and Xbloc™.

INTRODUCTION

The overtopping database of the *CLASH* project (Steendam *et al.*, 2004) contains more than 10000 test results on wave overtopping at coastal structures worldwide and is therefore considerably larger than initially foreseen. The database was used as input for a neural network which resulted in a generic prediction method for overtopping at coastal structures (Pozueta *et al.*, 2004, Van der Meer *et al.*, 2005).

Interim analysis of the overtopping database indicated that there were some "white spots" where further model tests would be advantageous. Test programmes to fill these "white spots" were devised. This paper describes a major programme of 2D model tests to better establish the influence of roughness or permeability on overtopping. Specifically, the objective was to determine the relative difference in overtopping behaviour for various types of armour units leading to roughness factors γ_f for the database and for use in the neural network prediction of overtopping.

TECHNICAL BACKGROUND

Run-up on sloping structures

What is a good definition of relative overtopping? A complete smooth structure could be considered as a structure giving maximum overtopping (no friction, no porosity). If this situation is considered as a reference with $\gamma_f = 1$, which until now is the case in the database, roughness factors will always be equal or smaller than 1. Although we are interested in overtopping, it is also

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possible to determine the roughness factor by comparing wave run-up. Figure 1 gives the results for the 2% (non-dimensional) run-up level for smooth slopes (using peak period T_p excluding very shallow foreshore cases). The graph shows a large influence of the (offshore) surf similarity parameter or breaker parameter ξ_{op} for smaller values (plunging or breaking waves) and no influence for larger breaker parameters (surging or non-breaking waves). This behaviour is very typical for smooth slopes.

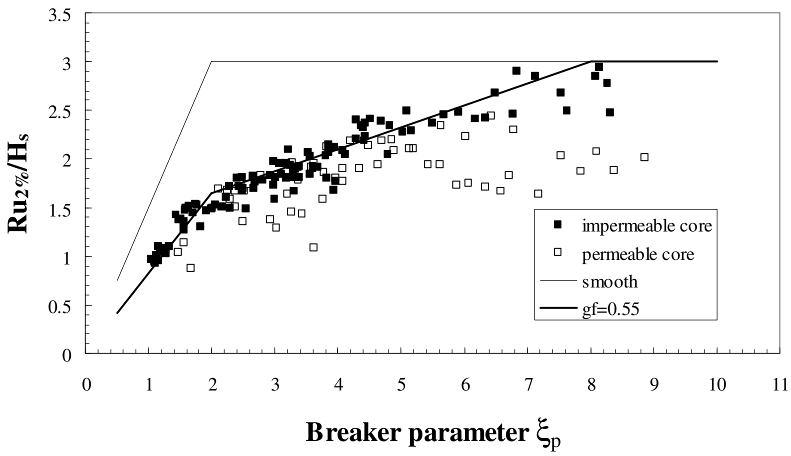

Figure 1. Wave run-up for smooth and rock slopes.

In the same figure, results of run-up tests on rock slopes have been given. Rock slopes with slopes of 1:2, 1:3 and 1:4 for impermeable structures (such as dike revetments with an impermeable core) and 1:1.5, 1:2 and 1:3 slopes with a permeable core, comparable to breakwaters, are given. Based on the structures with an impermeable core, a roughness factor was determined for the Dutch TAW guidelines for wave run-up and wave overtopping at dikes (TAW, 2002). For $\xi_{op} < 2$ the roughness factor was $\gamma_f = 0.55$.

Figure 1 shows that for large breaker parameters the run-up comes closer to the values for smooth structures. It appears that long waves on steep slopes (large breaker parameters) do not feel any roughness, as long the core is impermeable. It is on the basis of this conclusion that the TAW guidelines linearly increase the roughness factor from 0.55 to 1.0 between ξ_{op} = 2 and 8. This relationship is graphically presented in Figure 1, which shows that the data is represented well for rock on an impermeable core. This method leads to the conclusion that the influence of roughness decreases for larger breaker parameters.

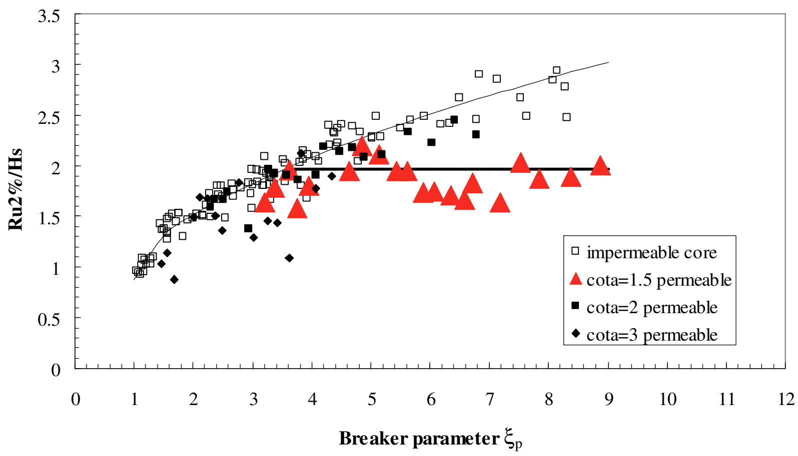

Figure 2. Wave run-up on rock slopes.

Another way to look at the influence of roughness and permeability is to take the results for rock structures as a reference. Figure 2 shows the run-up results in more detail. Run-up formulae for rock are (van der Meer, 1998):

$$R_{u2\%} / H_s = 0.88\ \xi_{op} \quad \text{for } \xi_{op} < 1.5$$
$$R_{u2\%} / H_s = 1.1\ \xi_{op}^{0.46} \quad \text{for } \xi_{op} > 1.5$$

with a maximum of $R_{u2\%} / H_s = 1.97\ \xi_{op}$ for permeable structures.

These formulae are graphically represented as solid lines. In this graph the impermeable core structures are shown as one group. Distinction has been made for the structures with a permeable core, comparable to breakwaters. A slope of 1:1.5 is very common for rubble mound breakwaters and this slope for a rock structure is given more attention in Figure 2. The maximum run-up predictors were clearly based on the results for this slope. Although there is quite some scatter, for breaker parameters larger than 5 there is no other clear trend than a horizontal line. For ξ_{op} values between 3 and 5 there still could be an influence of increasing run-up with increasing breaker parameter. This range is very important as it includes a slope of 1:1.5 with wave steepnesses between 0.02 and 0.04.

Standard test situation

Large wave overtopping is often related to situations quite close to the design wave conditions for structure stability. The stability of the structure itself is not an issue. In order to compare different units of different sizes, a standard test situation and standard cross-section is required. Such a standard test situation can best be based on design conditions for the structures.

Very often breakwaters with a steep slope are designed for a fixed stability number $H_s/\Delta D_n$ where Δ is the relative buoyant density of the unit ($\rho_{unit}/\rho_{water}-1$) and D_n is the unit nominal diameter ($\{mass_{unit}/\rho_{unit}\}^{1/3}$). A stability number is defined for each unit and is then the basis for both the test set-up and the cross-section. Table 1 gives various units with their stability number for design, proposed to be used for setting up and scaling of the experiments. It was originally anticipated that the model tests would include Dolosse and Sheds, but unfortunately at the time of testing suitably-sized units could not be sourced.

Table 1. Proposed stability numbers for different armour units to be used to scale the experiments

Type of armour	$H_s/\Delta D_n$	no. of layers	Layer thickness coefficient k_t	Porosity (%)	Proposed packing density φ
Rock	1.5	2	1.15	± 40	1.38
Cube	2.2	2	1.1	47	1.17
Antifer	2.2	2	1.1	47	1.17
Accropode	2.5	1	1.51	59	0.62
Tetrapod	2.2	2	1.04	50	1.04
Core-Loc™	2.8	1	1.51	63	0.56
Xbloc™	2.8	1	1.49	61	0.58
One layer cubes	2.2	1	1.0	30	0.70
Haro*		2		51	

* Haro: for two layer armouring, the number of blocks per $m^2 = 1 / 0.89b^2$, where b is the characteristic width

Given the stability number, the wave height under design conditions (H_0) can be calculated. This should be a wave height which can be generated in the flume. This design wave height is given as

$$H_o = stability\ number * \Delta D_n$$

Tests with $H_{m0} = H_0$ would be the maximum significant wave height for testing. Other tests should be carried out with $H_{m0} = 0.5\ H_0$ and $H_{m0} = 0.75\ H_0$. Each wave height would then be repeated for three wave steepnesses, s_{op} = 0.02, 0.035 and 0.05, where s_{op} is the wave steepness offshore. Two water levels would be tested, giving (for most tests) R_c/H_0 = 1.3 and 0.8. This leads to a total of at least 18 tests for one structure, covering small to large overtopping. The actual number of tests however varied for each configuration.

Standard cross-section

Most structures are built in fairly shallow water. But in order to make the comparison between tests for the roughness factor easier, it is better not to include a foreshore. On the other hand, the water depth should not be too large, because the structure becomes much larger with a larger water depth. A water depth of $2.5H_0$ is chosen for the tests. For such a water depth the waves do not break and they can be generated without a foreshore. Also a horizontal foreshore is acceptable, as long as the water depth above this foreshore is $2.5H_0$.

The crest freeboard under design conditions was 1.3 H_0. The total structure height is thus 3.8 H_0. Figure 3 gives the proposed standard cross-section. For the actual layer thickness the values in Table 1 have to be used.

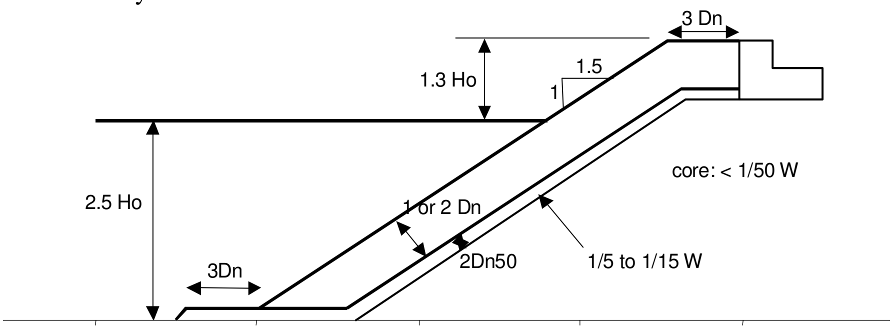

Figure 3. Proposal for a standard cross-section.

The slope of the structure is 1:1.5. All dimensions are related to H_0 or to the nominal diameter of the unit. The crest and toe are $3D_n$ wide. The under-layer was chosen to be in the range 1/5 to 1/15 of the weight of the armour unit.

EXPERIMENTAL SET-UP AND PROCEDURE

The 2-d experimental investigations at small-scale were all completed in the wave channel in the School of Engineering and Electronics at University of Edinburgh, UK (Figure 4). The channel is 20m long, 0.4m wide and has an operating water depth of 0.7m. The side walls and the bottom of the flume are made of glass. Waves are generated by a flap type wave paddle that is capable of generating regular and irregular waves with significant wave heights up to 0.11m and wave periods up to 2.0s for a fixed water depth of 0.70m at the paddle. The paddle is equipped with active absorption which significantly reduces reflected waves returning from the structure.

Overtopping discharges were directed via a centrally placed chute (width either 0.1 or 0.2m), which discharged into a measuring container suspended from a load cell. Individual overtopping events were detected by two parallel strips of metal tape run along the structure crest which acted as a switch closed by the water. For higher discharge conditions, water was removed from the collection container using an electric pump during data collection periods. At the end of the test the loadcell voltage trace was passed through an algorithm which determined that total volume of water which overtopped the structure during the test. Similarly wave-by-wave overtopping volumes were measured by determining the increment in the mass of water in the collection tank after each overtopping event following the general approach first used by Franco *et al.* (1994).

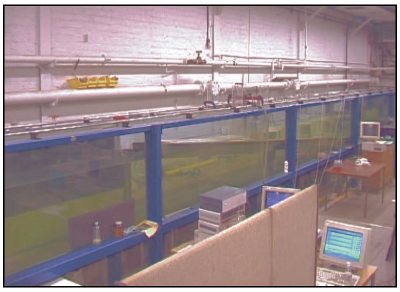

Figure 4. General view of wave flume.

Figure 5. The model core and filter layer.

All tests were recorded on video tape for later analysis as necessary. The camera was positioned to the side of the flume to record the wave breaking regime and the overtopping characteristics. In addition still photographs were taken of the armour slope to investigate the stability of the armour layer as a record of unit placements.

To determine the incident wave characteristics, three resistance type wave gauges were used. Three gauges were positioned seaward of the structure separated by 0.75 and 0.3m. Incident and reflected conditions were separated using the Aalborg University *WaveLab*™ software (which uses the methodology described by Mansard and Funke). The wave gauges were calibrated each morning.

Core Material

The core and filter material was graded into the correct size by sieving the material through the appropriate mesh size. For the core layer, the requirement was that the weight was less than 1/50*W*, where *W* is the weight of the armour unit. As all the armour units differed in weight, careful consideration was given such that the filter layer fitted all criteria. Figure 5 shows the installation of the core layer.

Table 2. Filter and core layer characteristics.

Grading	Core	Filter A	Filter B
W_{50} (g)	0.86	3.37	7.42
W_{85}/W_{15}	2.18	2.12	1.70

The weight difference between the different types of armour units was too large to allow a single grading of filter layer so two filter layer gradings were made. The smaller grading (Filter Layer A) was used for the Haro, whilst the larger grading (Filter layer B) was used for the Cubes, Antifer, Tetrapod, Rock, Core-Loc™, Accropode, and Xbloc™. Table 2 shows the core and filter layer characteristics.

Testing procedure

Prior to each test, the water level was adjusted to the required freeboard (R_c). For each armour unit type, two freeboards were investigated. The size of armour unit determined the design offshore significant wave height, H_{m0}. As the units were of different size, H_{m0} also varied accordingly.

For the smooth structure R_c /H_{m0} = 1.0 and 1.7. For the natural rock R_c /H_{m0} = 0.9 and 1.3, and for all the other armour units R_c /H_{m0} = 0.8 and 1.3. Overtopping measurements were made at the crest wall edge, although repetitions were made with measurements at the seaward corner of the crest berm (at distance of $3D_n$ seaward of the wall) to study the berm width influence. The unit properties of the armour units are summarised in Table 3.

Table 3. Unit properties and model / wave parameters

	actual unit weight	actual nominal diameter	actual mass density	no. of units per sq m per layer	approx no. units required	actual relative mass density	actual wave height	water depth	crest freeboard	toe width = crest width	underlayer weight, max	underlayer weight, min	core material weight, max
	W	D_n	ρ	N_a	N	D	H_s	h	R_c				
	[kg]	[m]	[kg/m^3]	[-]	[-]	[-]	[m]	[m]	[m]	[m]	[kg]	[kg]	[kg]
Rock - large	0.1910	0.042	2650	398	159	1.65	0.103	0.26	0.103	0.125	0.038	0.013	0.0038
Rock - small	0.0720	0.030	2650	763	305	1.65	0.074	0.19	0.074	0.090	0.014	0.005	0.0014
Cube	0.0620	0.030	2361	660	264	1.36	0.089	0.22	0.089	0.089	0.012	0.004	0.0012
Cube (single layer)	0.0620	0.030	2361	792	317	1.36	0.089	0.22	0.089	0.089	0.012	0.004	0.0012
Antifer	0.0850	0.033	2361	535	214	1.36	0.099	0.25	0.099	0.099	0.017	0.006	0.0017
Accropod	0.0742	0.032	2361	622	249	1.36	0.107	0.27	0.107	0.095	0.015	0.005	0.0015
Tetrapod	0.1000	0.035	2350	427	171	1.35	0.104	0.26	0.104	0.105	0.020	0.007	0.0020
Core-Loc™	0.0605	0.030	2300	632	253	1.30	0.108	0.27	0.108	0.089	0.012	0.004	0.0012
Xbloc™	0.0620	0.030	2300	607	243	1.30	0.109	0.27	0.109	0.090	0.012	0.004	0.0012
Haro	0.0420	0.026	2361			1.36	0.092	0.23	0.092	0.078	0.008	0.003	0.0008

The tests had a fixed duration of 1024s and hence, depending upon the wave period, gave between 700 and 1300 waves. For all conditions a JONSWAP (γ=3.3) pseudo-random wave spectrum was used.

For the cubes, it was decided to investigate if the placement orientation of the cube influenced the overtopping characteristics, hence the cubes were tested in a ‘flat’ orientation whereby the cubes were placed relatively flat to each other and the second case was when the cubes were placed in a more rough random pattern. For the rock tests, two stone sizes were used; *small rock* (W_{50} = 72g; W_{85}:W_{15} = 2.62) and *large rock* (W_{50} = 190g; W_{85}:W_{15} = 3.64).

RESULTS – MEAN OVERTOPPING DISCHARGE

The tests carried out are summarised in Table 4. Figure 6 shows the different armoured structures in the flume at the beginning of testing. Only in the case of the irregular cubes was any significant movement of the armour noticeable.

The tests carried out with a smooth slope formed the reference case (Figure 7). The data agree quite well with the van der Meer equation, though with a 5% over-estimation of γ_f. It is assumed that $\gamma_f = 1$ is the true value, and thus all subsequent values of γ_f obtained from the experiments are corrected by this 5%.

Table 4. Summary of test conditions for the various armour types.

Armour	no. of tests	R_c/H_{m0}	R_c (mm)	actual packing density	notes
Smooth slope	18	1.7 & 1.0	187 & 110		
"Large" Rock	20	1.3 & 0.9	134 & 95	1.38	1
"Small" Rock	20	1.3 & 0.9	134 & 95	1.38	1
Cubes (Flat)	30	1.3 & 0.8	118 & 71	1.19	
Cubes (Irregular)	31	1.3 & 0.8	118 & 71	1.17	2
Single layer cubes (flat)	29	1.3 & 0.8	116 & 71	0.65	
Antifer	31	1.3 & 0.8	128.7 & 79	1.17	
Accropode	23	1.3 & 0.8	135 & 83	0.62	
Tetrapod	29	1.3 & 0.8	135 & 83	0.99	3
Core-Loc™	23	1.3 & 0.8	140 & 86.4	0.56	
Xbloc™	23	1.3 & 0.8	142 & 90	0.58	
Haro	30	1.3 & 0.8	118 & 74		4

Note 1. natural rock

Note 2. Towards the end of the tests it was noted that the cubes moved slightly from a "random" pattern to a more regular "flat" pattern. No readjustment of the units was made during the testing.

Note 3. arranged in a "T" pattern (SOTRAMER)

Note 4. Haro arranged with the faces in flat manner; see also note to Table 1

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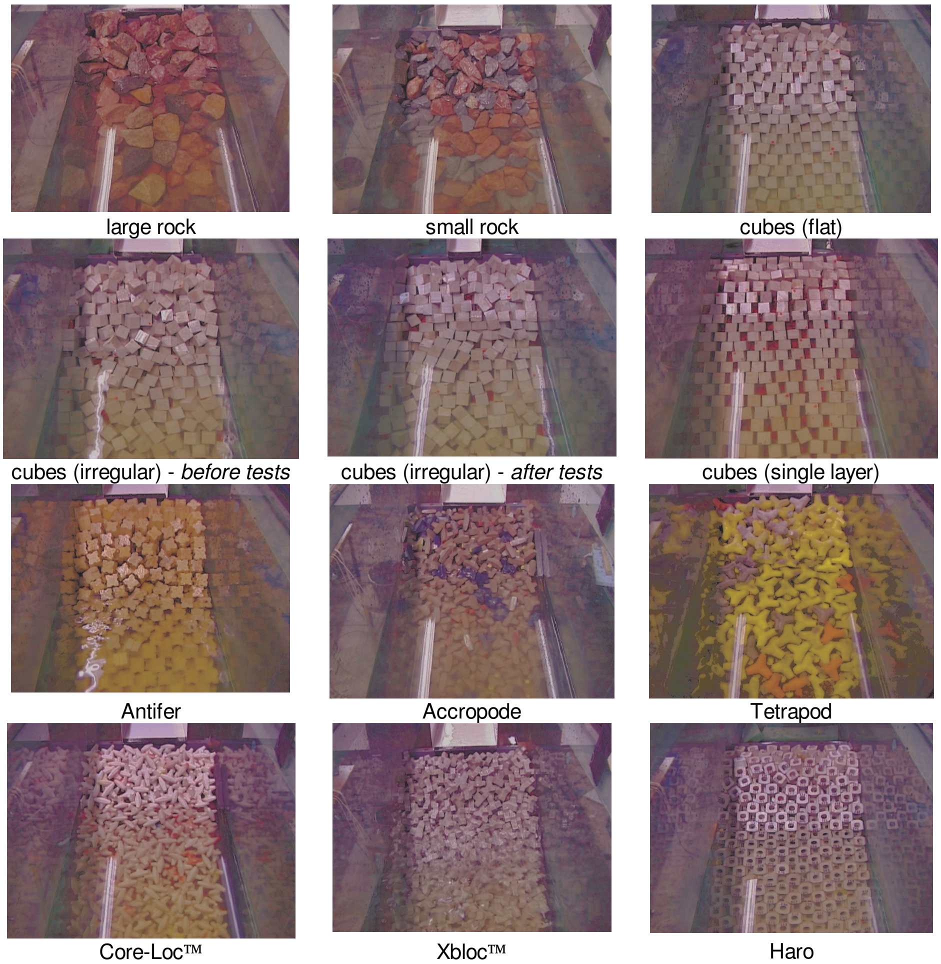

Figure 6. Photographs taken at the start of testing with the various armour. The condition of the irregular cubes at the end of testing is also shown (these being the only case which saw significant movement).

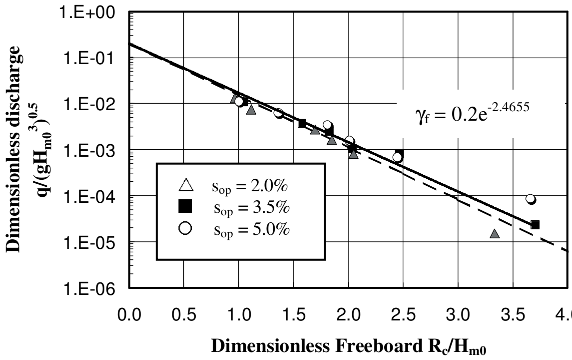

Figure 7. Graph of non-dimensional mean overtopping *v.* dimensionless freeboard for smooth 1:1.5 slope. Van der Meer formula (Equation 1) shown as dashed line.

Examples of the data from the main series of tests with the 1:1.5 structures are shown in Figure 8 (for irregular cubes and for Tetrapod). All data will be presented in a longer article currently in preparation (Bruce *et al.*, 2007). For comparison, also tested were 1:2 structures armoured with rock and with cubes.

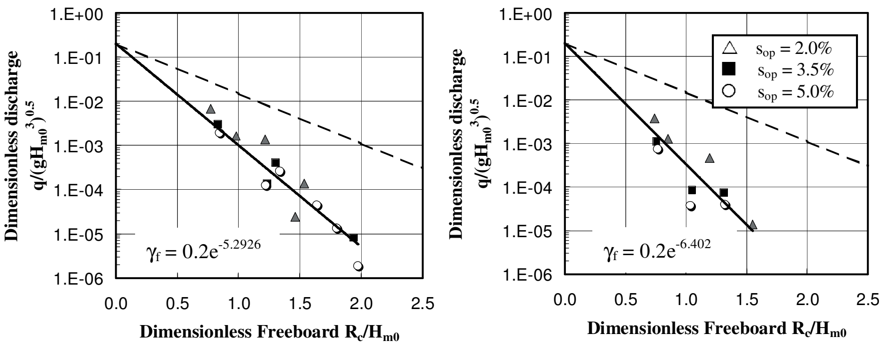

Figure 8. Example data from tests with irregular pattern Cubes (left) and Tetrapod (right). Graphs are of the conventional dimensionless mean overtopping discharge *vs.* dimensionless freeboard. Van der Meer formula *for smooth* slope (Equation 1) shown as dashed line.

DISCUSSION AND CONCLUSIONS

Detailed measurements have been made in University of Edinburgh flume to parameterise the mean overtopping rates q for a range of different armour units on a 1:1.5 sloping structure. Additional investigations have been carried out on a 1:2 slope for rock, and for cube armour units. Within experimental limitations, the results demonstrate that the overtopping characteristics follow the general trend of the Van der Meer formula (TAW, 2002), (Equation 1).

$$q/\sqrt{gH_{m0}^3} = 0.2\exp(-2.6\frac{R_c}{H_{m0}}\frac{1}{\gamma_f}) \quad (1)$$

All lines start at $R_c/H_{m0} = 0$ and $q/(gH_{m0}^3)^{0.5} = 0.2$. For smooth slopes $\gamma_f = 1$ and the value for each unit is derived by fitting a line through the data points.

It is noticeable that in almost all cases the wave period has an influence on the overtopping, a larger period gives more overtopping, though the influence is not very significant.

Table 5. Final γ_f values arrived at from synthesis of new data and other comparable tests. Values in italics are estimated / extrapolated

Type of armour	No Layers	Final γ_f
Smooth	-	1.00
Rock (two layers; permeable core)	2	0.40
Rock (two layers; impermeable core)	*2*	*0.55*
Rock (one layer; permeable core)	*1*	*0.45*
Rock (one layer; impermeable core)	*1*	*0.60*
Cube	2	0.47
One layer of cubes	1	0.50
Antifer	2	0.47
Accropode	1	0.46
Tetrapod	2	0.38
Core-Loc™	1	0.44
Xbloc™	1	0.45
Haro	2	0.47
Dolosse	*2*	*0.43*
Berm Breakwater	*2*	*0.40*
Icelandic Bermbreakwater	*2*	*0.35*

Moreover available data from similar previous studies were reanalysed in the same way in order to optimize the final selection after joint discussions among *CLASH* partners. Specifically, Aminti &Franco (1988) (for rock, cubes and tetrapods); Franco &Cavani (1999) (for antifer, tetrapods and Core-Loc™) ; and the results of parallel *CLASH* tests undertaken at Aalborg University and at University of Ghent were reviewed and discussed. This allowed a final selection of γ_f values as given in Table 5, which are valid for a rubble mound structure (a breakwater but not a revetment) with slope 1:1.5, with crest berm width $G_c = 3D_n$ with a permeable core / under-layer. Additional geometries/unit types were tentatively assigned a corresponding γ_f, based on experience from other available model tests identified within the *CLASH* overtopping database.

It is observed that the comparison with previous / parallel studies showed that for the Rock case (permeable core), γ_f varies with slope angle

1:1.3 $\gamma_f = 0.52$
1:1.5 $\gamma_f = 0.42$
1:2.0 $\gamma_f = 0.38$
1:3.5 $\gamma_f = 0.33$

The results of this study also showed a dependency of γ_f with the slope angle for rock. In the database, and also for use of the Neural Network, only one value for γ_f is given. Thus it was noted that the Neural Network must be able to include the slope influence in its prediction. Therefore, in the database only structures with cotα=1.5 and $G_c = 3D_n$ should use γ_f from Table 5.

ACKNOWLEDGEMENTS

The investigations in this study have only been possible by the very kind support and assistance given by the following organisations, who supplied some of the model units:

Sogreah, France (Accropode & Core-Loc™); Delta Marine Consultants bv, The Netherlands (Xbloc™); Protecno, Italy (Tetrapods); Flanders Hydraulics, Belgium (Antifer).

The use of WaveLab 2.6, supplied by Thomas Lykke Andersen of the University of Aalborg, is gratefully acknowledged for additional analysis of the wave conditions.

The support of the European Community Fifth Framework under Project Code EVK3-CT-2001-00058 is also gratefully acknowledged.

Marco Falzacappa and Roberto Molino, recent graduates from the University of Roma 3, ably supported model testing at Edinburgh and subsequent analysis in Rome. Their most helpful contribution is gratefully acknowledged.

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KEYWORDS – ICCE 2006

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Authors **Bruce, T., van der Meer, J.W., Franco, L. & Pearson, J.M.**

Abstract number **1705**

Breakwaters
Rubble Mound Breakwaters
Coastal structures
Overtopping
Armour units
Model tests