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AN EXPERIMENTAL STUDY ON WAVE OVERTOPPING OF RUBBLE MOUND BREAKWATER ARMOURED WITH A NEW CONCRETE ARMOUR UNIT

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ABSTRACT

Overtopping of waves is considered as one of the major concern to design a coastal structure like sea defence and breakwater. Nevertheless, in this research only the rubble mound breakwater is considered. Rubble mound breakwaters using single layer concrete armour systems are being widely used nowadays in the design of coastal structures compared to double layer system. There are several types of artificial concrete armour blocks which can be placed as single layer armour system. In recent, a new concrete armour unit called crablock has been invented and applied as single layer system in one damaged breakwater at Al Fujeirah, UAE. There is no design guidance exists yet for crablock as it is still under development. However, the preliminary design guidance on wave overtopping is required in order to use crablock as monolayer system in the design of rubble mound breakwater. Therefore, the present research is developed to investigate the wave overtopping over crablock slope to come up with first design guidance for the application of crablock. For the determination of wave overtopping, altogether 14 independent test series comprised of 87 tests were performed in a wave flume at the Fluid Mechanics Laboratory of the Faculty of Civil Engineering and Geosciences at Delft University of Technology, Netherlands. This paper describes the test results of 2D wave flume tests in line with wave overtopping over crablock armour slope.

Keywords: *Breakwater, crablock, single layer armour and wave overtopping*

1. INTRODUCTION

In the design of coastal structures like sea defences to protect coastal flooding, coastal protections to minimize coastal erosion and breakwaters at harbours to ensure safe navigation and mooring of vessels; overtopping of waves is considered as one of the prime concern (EurOtop, 2007). Overtopping of waves mainly occur due to the low crest height in comparison to wave run-up levels of the utmost waves (TAW, 2002). In that case crest freeboard or free crest height (R_c) is determined by the difference in elevation between height of the crest and the still water level. In general, wave overtopping is expressed by the term mean discharge per linear metre of width, q in terms of m^3/s per m or in l/s per m (EurOtop, 2007).

Rubble mound breakwaters have been mostly applied by designers among several types of breakwaters. A rubble mound breakwater is usually made with the use of rock armour or concrete armour in double layer systems or in single layer systems. In the design of rubble mound breakwaters, nowadays one layer systems using concrete armour units have become more common practice compared to conventional two layer systems. In recent, a new concrete armour unit called crablock has been invented and applied as single layer system in one damaged breakwater at Al Fujeirah, UAE. There is no design guidance exists yet for crablock as it is still under development. However, the preliminary design guidance on wave overtopping is required in order to use crablock as monolayer system in the design of rubble mound breakwater. Therefore, the present research is developed to investigate the wave overtopping over crablock slope to come up with first design guidance for the application of crablock. For the determination of wave overtopping, altogether 14 independent test series comprised of 87 tests were performed in a wave flume. In this research, two constant spectral wave steepnesses ($s_{m-1,0}$) of 0.02 and 0.04 were tested together with two different orientations of units, two different placing grids and four different packing densities. This paper describes the test results of 2D wave flume tests in line with wave overtopping over crablock armour slope.

2. PREDICTION OF WAVE OVERTOPPING

Based on the available wave conditions and water levels, various methods have been prescribed in EurOtop (2007) to predict the overtopping of waves; Analytical method, Empirical methods, PC-Overtopping and Neural network tools from CLASH database, Numerical methods and finally Physical models. In this research, empirical methods have been used to estimate wave overtopping over one layer crablock slopes, which have been checked with small scale 2D flume tests.

2.1 Empirical Methods

For the simplicity in determination, mean overtopping discharge (q) is very often used and expressed in terms of basic empirical equations of overtopping (EurOtop, 2007). EurOtop (2007) describes empirical equations in details for the approximation of overtopping over rubble mound slopes. For the prediction of wave overtopping of dikes, Van der Meer and Janssen (1995) introduced new conceptual design formulae for both breaking and non-breaking waves. In that research, estimation of overtopping of waves is expressed in terms of mean overtopping discharge, crest freeboard, slope angle, breaker parameter and the influence factors. These formulas are being widely used in the determination of wave overtopping and also explained further in TAW (2002) and in EurOtop (2007).

The general formula (see Equation 2.1) used for the estimation of wave overtopping discharge over coastal structure is (EurOtop, 2007),

$$\frac{q}{\sqrt{gH_{m0}^{3}}} = a\exp\left(-b\frac{R_c}{H_{m0}}\right) \tag{2.1}$$

Recently, Van der Meer and Bruce (2014) concluded that empirical formulas provided by EurOtop (2007), for breaking waves as well as for non-breaking waves over-estimate wave overtopping for slopping structures with very low or zero crest height. Furthermore, Van der Meer and Bruce (2014) recommended following formulas (Equation 2.2 & 2.3) to predict wave overtopping on slopping structures with zero and positive crest height.

— for breaking waves

$$\frac{q}{\sqrt{gH_{m0}^{3}}} = \frac{0.023}{\sqrt{\tan\alpha}}\cdot\gamma_b\cdot\xi_{m-1,0}\cdot\exp\left[-\left(2.7\frac{R_c}{\xi_{m-1,0}\cdot H_{m0}\cdot\gamma_b\cdot\gamma_f\cdot\gamma_\beta\cdot\gamma_v}\right)^{1.3}\right] \tag{2.2}$$

— and for non-breaking waves maximum value of

$$\frac{q}{\sqrt{gH_{m0}^{3}}} = 0.09\cdot\exp\left[-\left(1.5\frac{R_c}{H_{m0}\cdot\gamma_f\cdot\gamma_\beta}\right)^{1.3}\right] \tag{2.3}$$

Furthermore, Van der Meer and Bruce (2014) illustrated new formula for the design of wave overtopping over smooth slopping structures of slope angles steeper than 1:2 with non-breaking conditions. The formula (Equation 2.4) prescribed in that research is as follows,

$$\frac{q}{\sqrt{gH_{m0}^{3}}} = a\cdot\exp\left[-\left(b\frac{R_c}{H_{m0}}\right)^{1.3}\right] \tag{2.4}$$

Where, coefficients a and b are mentioned by researchers as following,

$a = 0.09 - 0.01\,(2-\cot\alpha)^{2.1}$, for $\cot\alpha \le 2$ and: $a = 0.09$ for $\cot\alpha > 2$
$b = 1.5 + 0.42\,(2-\cot\alpha)^{1.5}$, with a maximum of b = 2.35 and: $b = 1.5$ for $\cot\alpha > 2$

2.2 CLASH Database

To approximate wave overtopping for an extensive variety of coastal structures, a standard design tool has been generated in the European research project CLASH (Van Der Meer, et al., 2005). CLASH database is an international database freely available on internet for wave overtopping over coastal structures. The database is comprised of more than 10,000 tests from 163 independent test series which is formatted in Excel containing a matrix of 31 columns for 31 parameters and more than 10,000 rows (Steendam, et al., 2004). The overtopping discharges for all kind of coastal structures are available in the database.

Physical Modelling

Analytical or theoretical methods to predict overtopping of waves have not been generated very well. Therefore, EurOtop (2007) recommended small scale wave flume tests to generate empirical equations of overtopping estimation. Prototype situation can be scaled to a physical model by the use of small scale model testing (Van Buchem, 2009). Furthermore, experimental model testing is often applied where the coastal structures are designed using single layer concrete armour units and overtopping is an important criterion (Wolters, et al., 2009). Small scale physical model tests were performed in this experimental research in order to determine wave overtopping over crablock slope.

3. LABORATORY SET-UP

Small scale model tests were performed in a 2D wave flume (see Figure 1) at the hydraulics laboratory of the Delft University and Technology, Delft, Netherlands. The set-up of cross-section to perform flume tests has been done by considering the small scale set up of accropode (Van der Meer, 1987), set up of xbloc (DMC, 2003) and set up of Bruce, et al. (2009) for rubble mound breakwaters with various types of armour units.

Figure 1: Picture of wave flume

3.1 Model Set-Up

To conduct small scale hydraulic tests, three cross-sections were tested. The designed breakwater was comprised of one layer crablock armour, under layer, core, stone protection at toe and a crest wall, see Figure 2. The slope of crablock armour was kept as 1:4/3 as similar as accropode, core-loc and xbloc. The ratio between freeboard and design significant wave height was fixed as 1.2 allowing some waves overtopping. This design significant wave height has a stability number around 2.8. This clearly indicates that wave overtopping over the crest of breakwater will be lot more for the significant wave height beyond the design significant wave height. Nevertheless, in this research significant wave heights higher than the design significant wave heights were also tested to observe the failure of armour layer.

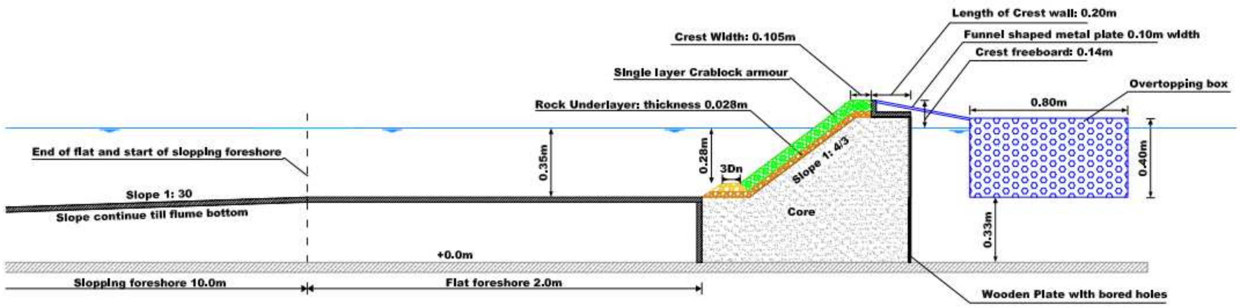


Figure 2: Cross-section of breakwater with crablock armour slope (R_c= 1.2 X Design wave height); tests 1-8

A sloping foreshore was considered in front of a horizontal foreshore with a uniform slope of 1:30. The length of sloping foreshore was 10 m starting from the bottom of the flume up to depth 0.33 m above the bottom. Besides slopping foreshore, a horizontal length of 2 m before toe structure was provided in order to put wave gauges to measure wave heights. The design stability number for crablock was selected as 2.8 equal to xbloc, core-loc and accropode II in order to define the design significant wave height. The design wave height can be estimated from the known stability number following the approach used by Bruce, et al. (2009). In this research, for the crablock armour unit the design wave height was estimated as following (see Equation 3.1),

$$H_0 = \text{Design stability no.} \times (\text{relative density} \times \text{nominal diameter}) \tag{3.1}$$

Thus, design wave height, $H_0 = 2.8 \times (1.36 \times 0.03) = 0.114 \text{ m}$

The water depth at structure was considered as 0.35 m that means around 3.0 times of design wave height. In order to have water depth 0.35 m at the toe, the water depth at deep water was kept 0.68 m. For most of the tests (test series 1 to 8), the ratio of water depth before toe and water depth upon toe was fixed to 0.80 resulting water depth of 0.28 m upon toe of the breakwater.

3.2 Test Programme

The most important parameters that governs the geometrical design of breakwater are placement pattern, packing density, crest height and wave steepness in terms of wave height and wave length (Bonfantini, 2014). The placement grid, orientation of units and packing density were selected mainly based on the results of dry placement tests, see Salauddin, 2015. Ten test series were performed for the determination of wave overtopping over crablock armour slope. Furthermore, to examine the accuracy of the measured wave heights and wave overtopping, two test series were also executed using a smooth slope of 1 in 4/3. Also, two test series (test 13 and 14) were conducted without the presence of a structure to determine the actual incident wave heights. Test programme followed in this research is presented in Table 1. It should be noted that each test was comprised of seven sub tests for different wave conditions.

Table 1 Test Programme for flume tests

Test Series No.	Placement Grid	Orientation	Hor. Vs Upslope distance	Packing Density	Crest Freeboard (m)	Under Layer (mm)	Deep water Wave Steepness $s_{m-1,0}$	Water depth at structure (m)
1	Rectangular	Uniform	0.65Dx0.64D	$0.69/D_n^2$	0.140	7 - 11	0.04	0.35
2	Rectangular	Uniform	0.65Dx0.64D	$0.69/D_n^2$	0.140	7 - 11	0.02	0.35
3	Diamond	Random	0.75Dx0.61D	$0.63/D_n^2$	0.140	11- 16	0.04	0.35
4	Diamond	Random	0.75Dx0.61D	$0.63/D_n^2$	0.140	11 -16	0.02	0.35
5	Rectangular	Uniform	0.68Dx0.64D	$0.66/D_n^2$	0.140	7 - 11	0.04	0.35
6	Rectangular	Uniform	0.68Dx0.64D	$0.66/D_n^2$	0.140	7 - 11	0.02	0.35
7	Rectangular	Uniform	0.71Dx0.64D	$0.63/D_n^2$	0.140	7 -11	0.04	0.35
8	Rectangular	Uniform	0.71Dx0.64D	$0.63/D_n^2$	0.140	7 -11	0.02	0.35
9	Rectangular	Uniform	0.68Dx0.64D	$0.66/D_n^2$	0.185	7- 11	0.04	0.35
10	Rectangular	Uniform	0.68Dx0.64D	$0.66/D_n^2$	0.185	7 - 11	0.02	0.35
11	Smooth 1 : 4/3 slope				0.185	-----	0.04	0.35
12	Smooth 1 : 4/3 slope				0.185	-----	0.02	0.35
13	Without structure				-----	-----	0.04	-----
14	Without structure				-----	-----	0.02	-----

4. RESULTS AND DISCUSSIONS

4.1 Measured Wave Conditions

In general, the wave height at the structure differs from the wave height at deep water due to complex phenomenon like shoaling and wave breaking at depth limited conditions (Van der Meer, 1987). In this research, to determine the actual incident significant wave height at the structure, wave heights were also measured without the presence of the breakwater in the flume (test series 13 and 14). The test results showed that for high wave steepness (except very high wave height) and also for lower wave heights in case of low wave steepness, the incident wave heights at the structure without the presence of breakwater are almost as same as the wave heights with breakwater.

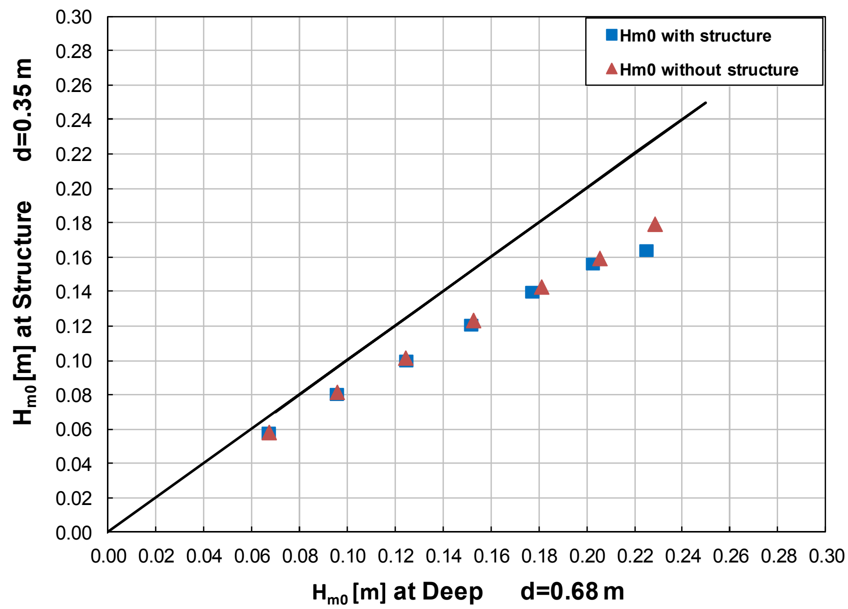


Figure 3: Comparison of measured wave heights (H_{m0}) with and without the presence of structure (short period)

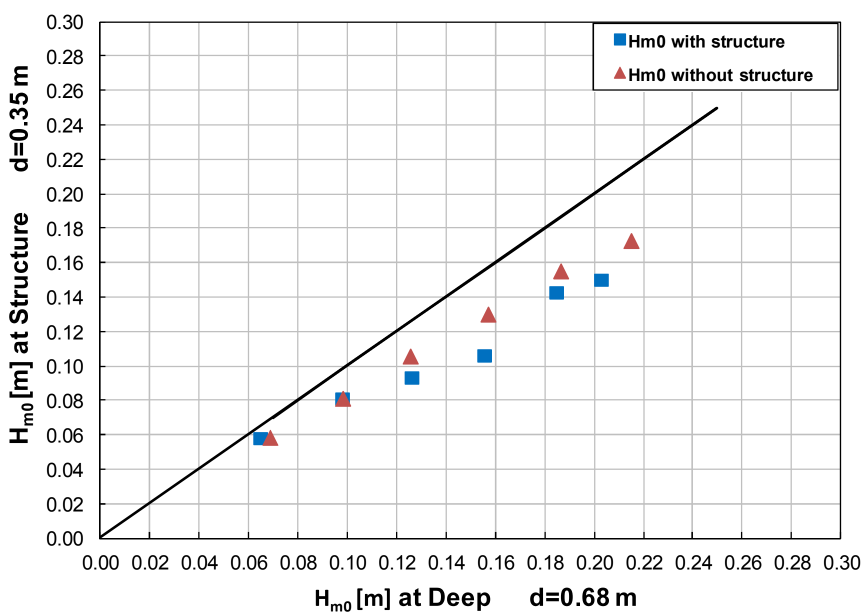
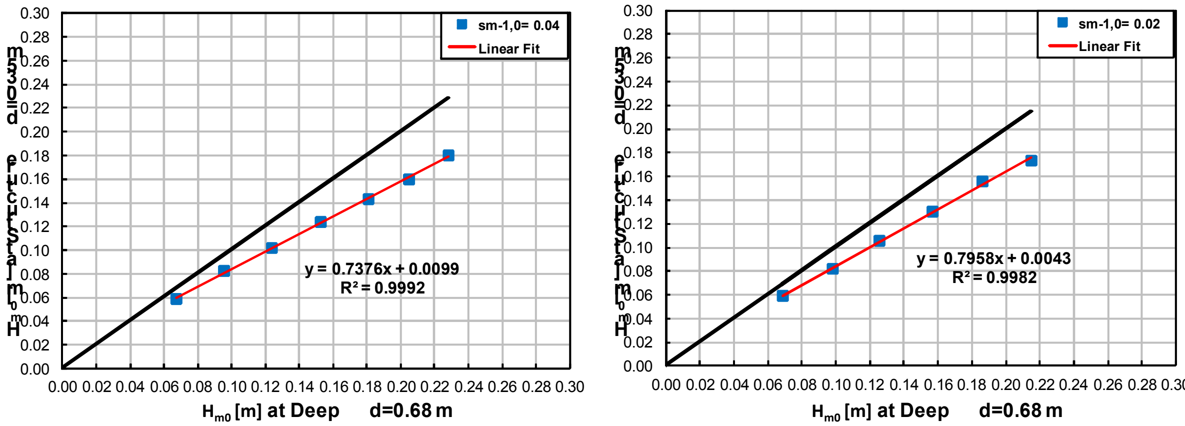


Figure 4: Comparison of measured wave heights (H_{m0}) with and without the presence of structure (long period)



a) b)

Figure 1: Relation between wave height (H_{m0}) at deep water and at structure for a) High wave steepness (short period) and a) Low wave steepness (long period)

In order to have a better understanding about the variation in wave heights, the measured wave heights at deep water and at structure for both with and without structure is compared in Figure 3 and Figure 4. Regarding to Figure 3 and Figure 4, it is visible that measured wave heights without the structure were some cases slightly higher both in case of low wave steepness and high wave steepness. This might be happening due to high reflection caused by the presence of the structure, as compared to without the structure. It may also be that the method to separate incident and reflected waves does not work properly in wave breaking conditions (no linear waves). However, to avoid possible errors in wave heights measurements in further overtopping analysis and to determine the real incident wave heights at toe with the presence of structure, for each individual tests wave heights were calibrated from the established relationship between deep water and structure; see Figure 2. It is worth mentioning that this relation was established without the presence of structure in the flume. It should be noted that the calibrated incident wave heights from the developed relationship of wave heights without structure were used in all the analysis.

4.2 Measured Wave Overtopping

The mean wave overtopping rate and overtopping percentages over a crab lock armour slope were measured for each test series. In all cases the incident wave height at the toe of the structure is considered, where the wave height is based on the spectrum (H_{m0}), as this is the wave height that is used in overtopping estimations (EurOtop, 2007).

Table presents an overview of measured wave overtopping for test series one and two performed in this experimental research, for the test results of all the test series see Salauddin, 2015. As shown in Table, test results showed that for the same wave height input (generator) with only different wave periods mean overtopping rate q (m^3/s per m) as well as percentage of overtopping (%) was a little higher for test series two (long period) compared to test series one (short period).

The resulting relative wave overtopping discharge $q/\sqrt{gH_{m0}^3}$ as a function of the relative crest freeboard (R_c/H_{m0}) is presented in Figure 6. The graph shows that test series with irregular placement of crablock result in almost the same overtopping as the other test series with regular placement of crablock units, for the same wave steepness. To give an example, the comparison of measured wave overtopping in test series 1, 3, 5 and 7 (same wave period) demonstrates that regular placement (test 3) hardly has any influence on overtopping; see Figure 6. Furthermore, for the tests with same wave steepness overtopping results did not vary much between the different test series, with the change in packing density, see Figure 6. For instance, test series 1, 5 and 7 performed with uniform placement pattern with the same configuration, except a different packing density of armour layer. Based on the test results it can be concluded that the change in packing density did not really change the overtopping behaviour of these test series.

Table: Overview of measured wave overtopping in test series 1 and 2

Test Series	**Sub-Test No.**	**Crest Freeboard (m)**	**Wave Generator Input**		**H_{mo} at Deep [m]**	**H_{mo} at Toe [m]**	**Mean Overtopping Rate [l/s per m]**	**Percentage of overtopping waves [%]**
			H_{mo} at Generator [m]	**T_p at Generator [sec]**				
1	1a	0.140	0.070	1.24	0.067	0.059	0	0
	1b	0.140	0.100	1.43	0.096	0.080	0.000261053	0.42
	1c	0.140	0.130	1.60	0.125	0.102	0.020100172	2.49
	1d	0.140	0.160	1.75	0.152	0.122	0.165504372	11.22
	1e	0.140	0.190	1.89	0.177	0.141	0.525394328	27.74
	1f	0.140	0.220	2.02	0.203	0.159	1.33340678	45.04
	1g	0.140	0.250	2.15	0.225	0.176	2.233468874	50.80
2	2a	0.140	0.070	1.73	0.065	0.056	0	0
	2b	0.140	0.100	2.07	0.098	0.082	0.005966724	1.06
	2c	0.140	0.130	2.36	0.126	0.105	0.229268118	11.41
	2d	0.140	0.160	2.61	0.156	0.128	0.874001901	28.33
	2e	0.140	0.190	2.85	0.185	0.151	1.956913249	46.63
	2f	0.140	0.220	3.06	0.203	0.166	2.853398334	60.23

Figure 7 shows the measured percentage of overtopping waves with respect to a dimensionless crest height. In this research the nominal diameter (D_n) of the crab lock was constant thus the percentage of overtopping waves varied with significant wave height (H_{m0}) at the toe and the armour freeboard (A_c). The resulting graph clearly shows that the percentage of overtopping waves increases with the increase of significant wave height at the toe of breakwater, while it decreases with the increase of crest freeboard. Furthermore, the test results showed that in general the percentage of waves overtopping the structure were a bit higher for longer wave periods than for high wave steepness. For example, from Figure 7 it is seen that tests with wave steepness of $s_{m-1,0} = 0.02$ gave high percentages of waves overtopping compared to the tests with wave steepness of $s_{m-1,0} = 0.04$.

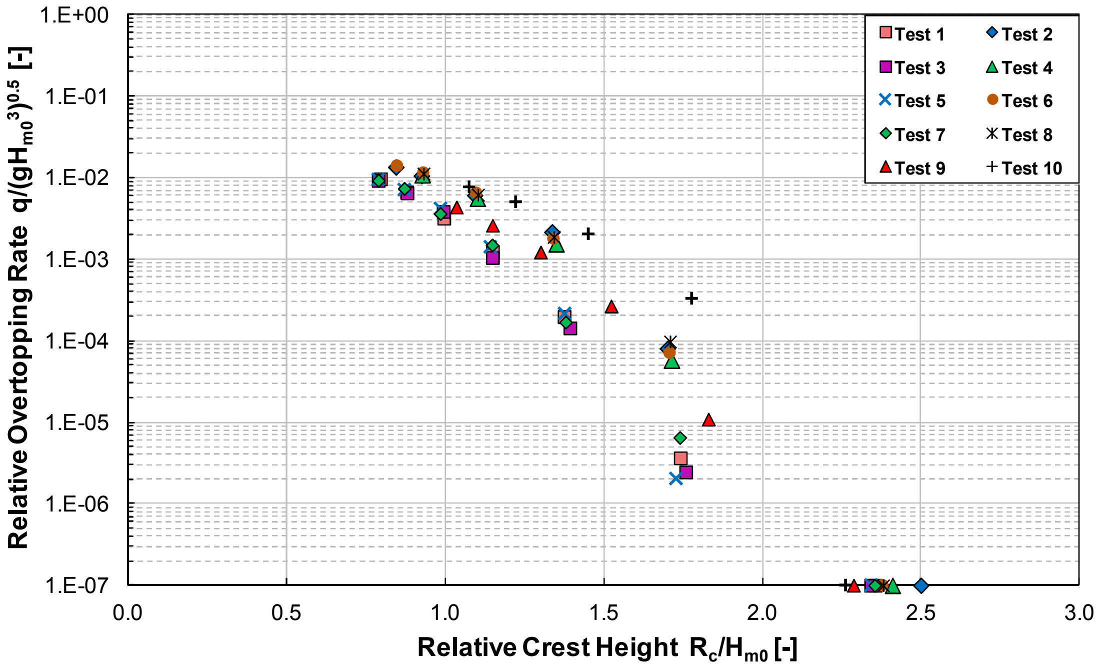


Figure 6: Relative overtopping discharge as a function of relative freeboard

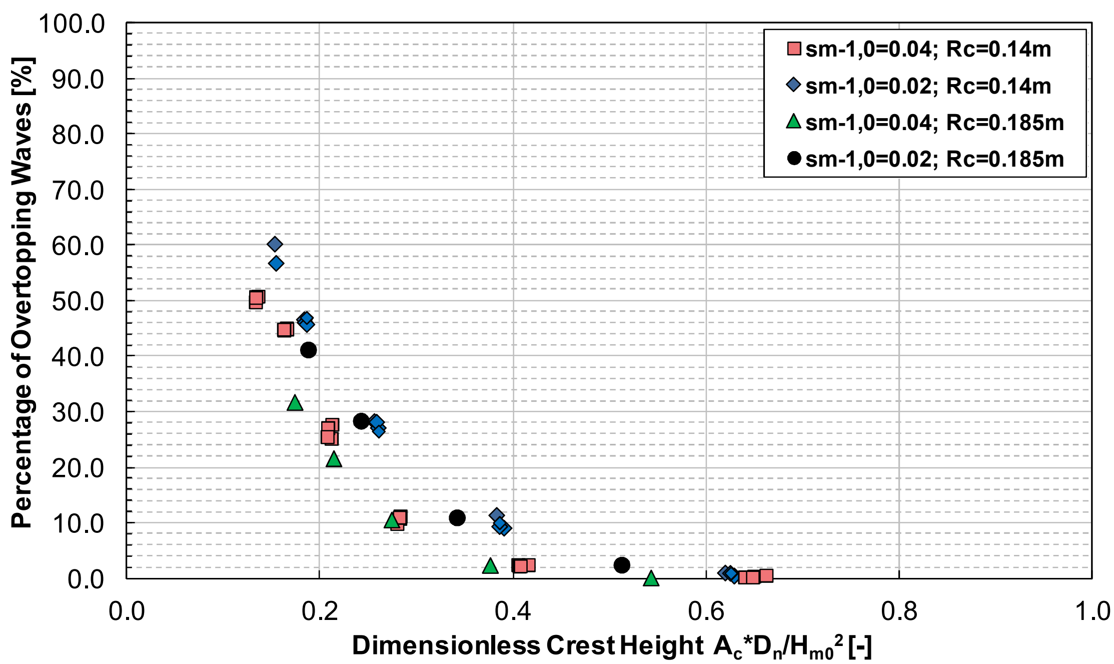


Figure 7: Percentage of wave overtopping as a function of dimensionless crest freeboard

5. CONCLUSIONS

Regarding to the test results, analysis and observations, the conclusions of these small scale physical tests can be pointed out as following:

- In this experimental investigation two different wave steepnesses were tested. Regarding to the test results, it was clear that low wave steepness (long wave period) gave higher overtopping compared to high wave steepness (short wave period). This might be due to the 1:30 foreshore slope that had large influence on the wave attenuation at the toe of the structure.
- Both uniformly placed crab lock armour and randomly placed crab lock armour were tested to observe the overtopping over slope. Overtopping results showed that there is no influence of placement pattern on wave overtopping.
- The test results with same configuration except different packing density proved that overtopping behaviour does not really change with change in packing density.
- Most of the test series were performed with the use of a crest freeboard 1.2 times the design wave height. Only two test series were conducted with a much higher crest freeboard, 1.6 times the design wave height. However, based on the test results it was monitored that different crest heights give unexpectedly deviation in dimensionless results.

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