A COMPARATIVE STUDY ON THE STABILITY FORMULAS OF RUBBLE MOUND BREAKWATERS

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An example study showed that there may be 70% difference in armour stone weight when Van der Meer (1988) and Van Gent et al. (2004) formulas are applied as recommended by "The Rock Manual: The use of rock in hydraulic engineering" (2007) with specific design constraints. In this paper, questions arise in the application of these formulas and their dependence on certain design constraints given in literature as mentioned in "The Rock Manual: The use of rock in hydraulic engineering" (2007) are discussed. Based on the results of this study, a new design flowchart that uses Van der Meer (1988) and Van Gent et al. (2004) formulations is proposed and tested by physical model experiments. Furthermore, a real case study in Aliaga, Izmir, Turkey is presented in order to indicate the importance of the new design flowchart.

Keywords: rubble-mound breakwaters; Van der Meer (1988) formula; Van Gent et al. (2004) formula

INTRODUCTION

Rubble mound breakwaters are type of coastal defense structures that are widely preferred all around the world, especially along the coastlines of Europe. Design of rubble mound breakwaters is a challenging issue due to the uncertainties in the design parameters. Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) and Van Gent et al. (2004) give the major design formulations referred in various design manuals. Guler (2013) clarified that there are some discrepancies in the application of Van der Meer (1988) and Van Gent et al. (2004) design formulations that results in a relative difference of armour stone size up to 70% under the same design conditions.

In this study, discrepancies given by Guler (2013) are revisited and extended by a comparative study focused mainly on Van der Meer (1988) and Van Gent et al. (2004) design formulations considering design constraints given in literature. Discussions on Hudson (CERC, 1977; CERC, 1984) formula is included throughout this study in order to provide a well-known reference formulation. Using the results of comparative study, a new design flowchart that uses Van der Meer (1988) and Van Gent et al. (2004) formulations is proposed; furthermore, physical model experiments conducted to test new design flowchart proposition are presented. Finally, a real case study in Aliaga, Izmir, Turkey is given to indicate importance of this new design flowchart.

MAJOR STABILITY FORMULAS

Hudson (CERC, 1977; CERC, 1984) derived a formula using results of physical model experiments conducted using regular waves. A formula by Van der Meer (1988) was presented for relatively deep water and moderate shallow water conditions considering irregular wave state. Van Gent et al. (2004) proposed another formulation based on irregular wave conditions that is derived mainly for shallow water conditions. In this section, these formulations are given including a brief note on the nature of the formulas.

Hudson (CERC, 1977; CERC, 1984) Formula

Hudson formulation is one the most well-known equation that is used to find weight of armour layer of rubble mound breakwaters given by Equation 1.

$$W_{50} = \frac{\gamma_{stone}\ H^3_{design}}{K_D\ \Delta^3\ \cot\alpha} \tag{1}$$

In Hudson formula, γ_{stone} is the specific weight of the stone, K_D is the stability coefficient specifically determined for the type of armour unit and breaking condition, Δ is defined as relative buoyant density and α is the breakwater face slope. In this approach, the most important parameter is H_{design} defined as design wave height. It is determined according to wave breaking condition. If the waves are breaking, H_{design} is taken as breaking wave height, if not, it is taken as the wave height at the toe of the structure. Another important issue related to design wave height in Hudson formula is

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different recommendations for selecting deep water design wave height. According to CERC (1977), design wave height at deep water is recommended to be taken as significant wave height (H_s), on the other hand, CERC (1984) recommends to use the deep water design wave height ($H_{1/10}$) as the wave height exceeded by 10% of the waves in a certain storm. Both formulations are used in practice.

Van der Meer (1988) Formula

Formula proposed by Van der Meer (1988) is a well-developed approach that considers randomly generated waves, type of wave breaking, number of waves attack coastal structure (N, implicitly wave period) and notional permeability (P) in the design of rubble mound breakwaters. In this approach, surf similarity parameter (ξ_m) using mean wave period (T_m) given by Equation 2 should be first calculated and compared to a critical surf similarity parameter (ξ_{cri}) given by Equation 3. Type of wave breaking is determined as plunging or surging by this comparison. The design formulas to be used are presented in Equations 4 and 5 given by Van der Meer (1988) depending on type of breaking.

$$\zeta_m = \tan\alpha / \sqrt{(2\pi/g)\ H_{s,toe}\ /\ T_m^2} \quad (2)$$

$$\zeta_{cri} = \left(\frac{C_{pl}}{C_s}\ P^{0.31}\ \sqrt{\tan\alpha} \right)^{\frac{1}{P+0.5}} \quad (3)$$

Plunging waves ($\xi_m < \xi_{cri}$)

$$\frac{H_{s,toe}}{\Delta D_{n50}} = C_{pl}\ P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \ \zeta_m^{-0.5} \quad (4)$$

Surging Waves ($\xi_m \geq \xi_{cri}$)

$$\frac{H_{s,toe}}{\Delta D_{n50}} = C_s\ P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \ \sqrt{\cot\alpha}\ \ \zeta_m^{P} \quad (5)$$

In Equation 2, g is defined as acceleration of gravity. Furthermore, in Equations 4 and 5, $H_{s,toe}$ is defined as significant wave height at the toe of the structure, C_{pl} is plunging coefficient, S is damage level, D_{n50} is nominal armour stone diameter assuming a 50% cumulative distribution and C_s is surging coefficient. Plunging and surging coefficients are calibrated as 6.2 and 1.0, respectively, using the results of physical model experiments conducted by Van der Meer (1988).

It should be noted that significant wave height at the toe of the structure ($H_{s,toe}$) can be replaced by wave height exceeded by 2% of the waves in a certain storm ($H_{2\%}$) for moderate shallow water conditions (Van der Meer, 1988). For these conditions, surging and plunging coefficients should be multiplied by 1.4 since $(H_{2\%}/H_{s,toe})^{-1}$ is guaranteed to be 1.4 by Rayleigh distribution at deep water.

Van Gent et al. (2004) Formula

Van Gent et al. (2004) proposed a similar formula to Van der Meer (1988) that can be applied in shallow water conditions. In this formulation, spectral mean energy wave period ($T_{m-1,0}$) is taken into consideration instead of mean wave period (T_m) to include influence of spectral shape which is an important issue especially in shallow water. Similar to Van der Meer (1988) approach, type of breaking is determined by comparing surf similarity parameter ($\xi_{m-1,0}$) calculated using spectral mean energy wave period and critical surf similarity parameter (ξ_{cri}) given by Equations 6 and 7, respectively. According to type of breaking, formulas given by Equations 8 and 9 are selected and used to design rubble mound breakwaters.

$$\zeta_{m-1,0} = \tan\alpha / \sqrt{(2\pi/g)\ H_{s,toe}\ /\ T_{m-1,0}^2} \quad (6)$$

$$\zeta_{cri} = \left(\frac{C_{pl}}{C_s}\ P^{0.31}\ \sqrt{\tan\alpha} \right)^{\frac{1}{P+0.5}} \quad (7)$$

Plunging waves ($\xi_{m-1,0} < \xi_{cri}$)

$$\frac{H_{s,toe}}{\Delta D_{50}} = C_{pl}\ P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \left(\frac{H_{2\%}}{H_{s,toe}} \right)^{-1} \zeta_{m-1,0}^{-0.5} \quad (8)$$

Surging Waves ($\xi_{m-1,0} \geq \xi_{cri}$)

$$\frac{H_{s,toe}}{\Delta D_{50}} = C_s \ P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \left(\frac{H_{2\%}}{H_{s,toe}}\right)^{-1} \sqrt{\cot\alpha} \ \zeta_{m-1,0}^{P} \tag{9}$$

Plunging and surging coefficients in Van Gent et al. (2004) formulation are calibrated as 8.4 and 1.3, respectively, based solely on physical model experiments by Van Gent et al. (2004).

Note on the Nature of Van der Meer (1988) and Van Gent et al. (2004) Formulas

The Rock Manual (2007) recommends using Van der Meer (1988) approach for deep water and moderate shallow water conditions and Van Gent et al. (2004) approach for shallow water conditions as a general application procedure. In general, this recommendation is satisfied by the original references of these formulations such that Van der Meer (1988) equations are developed for deep water and moderate shallow water conditions whereas Van Gent et al. (2004) equations are developed for shallow water conditions. The Rock Manual (2007) gives rough definitions for deep and shallow water conditions (Table 5.29, Chapter 5, The Rock Manual, 2007) and refers Van Gent et al. (2004) approach as Van der Meer shallow water equations.

DEFINITION OF PROBLEM

It is stated previously in this study that The Rock Manual (2007) defines deep and shallow water conditions roughly using a dimensionless parameter which is water depth at the toe of the structure (h) over significant wave height at the toe of the structure ($H_{s,toe}$). If this parameter ($h/H_{s,toe}$) is bigger than 3, The Rock Manual (2007) recommends to use Van der Meer (1988) formulation; on the other hand, it recommends to use Van Gent et al. (2004) formulation if $h/H_{s,toe}$ is smaller than 3. Considering the nature of the formulations, this parameter provides a limit to define deep and shallow water limits. However, this recommendation has some discrepancies that results in problems in application of these formulas.

An example study was carried out to show the discrepancies in application of Van der Meer (1988) and Van Gent et al. (2004) formulations. Armour stone weight of a rubble mound breakwater was calculated using both formulations and Hudson (CERC, 1977; CERC, 1984) approaches were included to provide a comparison measure. In Table 1, design parameters are given for a realistic hypothetical case assuming these parameters are realistic.

Table 1: Design Parameters for Example Study		
Parameters for all Approaches		
Deep Water Significant Wave Height	H_{s0} (m)	5.3
Significant Wave Period	T_s (sec)	8.5
Depth of Construction or Depth at the Toe of the Structure	h (m)	14
Structure Slope	cot(α)	2
Foreshore Slope	m	0.03
Specific Weight of Armour Stone	γ_{stone} (t/m^3)	2.7
Specific Weight of Sea Water	γ_{water} (t/m^3)	1.02
Deep Water Wave Approach Angle	α_0 (˚)	0
Deep Water Significant Wave Steepness	H_{s0}/L_0	0.047
Deep Water Peak Wave Steepness	S_{op}	0.04
Wave Height at the Toe of the Structure	$H_{s,toe}$ (m)	4.87
Parameters for Hudson (CERC, 1977; CERC, 1984) Approach		
Deep Water Wave Height Exceeded by 10% of the Waves	$H_{1/10,0}$ (m)	6.73
Stability Parameter for Non-Breaking Case	$K_{D,non-breaking}$	4
Stability Parameter for Breaking Case	$K_{D,breaking}$	2
Parameters for Van der Meer (1988) and Van Gent et al. (2004) Approaches		
Notional Permeability	P	0.4
Damage Level	S	2
Number of Waves	N	1000
Mean Wave Period	T_m (sec)	6.89
Wave Height Exceeded by 2% of the Waves at the Toe	$H_{2\%}$ (m)	7.18
Spectral Mean Energy Wave Period	$T_{m-1,0}$ (sec)	8.33
Peak Wave Period	T_p	9.18
Surf Similarity Parameter, Van der Meer Approach	ξ_m	1.95
Critical Surf Similarity Parameter, Van der Meer Approach	$\xi_{c,vdm}$	3.77
Surf Similarity Parameter, Van Gent et al. Approach	$\xi_{m-1,0}$	2.36
Critical Surf Similarity Parameter, Van Gent et al. Approach	$\xi_{c,vg}$	3.95

Computational results obtained from all approaches are tabulated in Table 2. Under the same design conditions, Hudson approaches (CERC, 1977; CERC, 1984), Van der Meer (1988) approach and Van Gent et al. (2004) approach results in significantly different armour stone size. In comparison, Van der Meer (1988) approach gives minimum armour stone size whereas Hudson (CERC, 1984) approach gives maximum armour stone size. There is a 19% difference in calculated armour stone diameter between Van der Meer (1988) and Van Gent et al. (2004) approaches which results in a 70% difference in armour stone weight for the same design parameters. This difference has to be questioned since it results in drastic cost and application problems in practice.

Table 2: Results of Example Study

Approach	Condition	Armour Stone Diameter, D_{n50} (m)	Weight of Armour Stone, W (tons)
Hudson (CERC, 1977)	Non-Breaking	1.49	8.9
Hudson (CERC, 1984)	Non-Breaking	1.89	18.2
Van der Meer (1988)	Plunging	1.38	7.0
Van Gent et al. (2004)	Plunging	1.65	12.0

There are a number of parameters that affect results of armour stone size related to definition of shallow water. These parameters are regarded as "design constraints" in this study including the parameter ($h/H_{s,toe}$) recommended by The Rock Manual (2007). Definitions of design constraints are given as follows and presented in Table 3 for this example study:

<u>Constraint 1:</u> Depth at the toe of the structure over significant wave height at the toe of the structure should be less than 3, i.e. $h/H_{s,toe} < 3$.

<u>Constraint 2:</u> Wave height exceeded by 2% of the waves at the toe over significant wave height at the toe of the structure should be less than 1.4, i.e. $H_{2\%}/H_{s,toe} < 1.4$.

<u>Constraint 3:</u> Significant wave height at the toe of the structure over deep water significant wave height should be less than 0.9, i.e. $R_H = H_{s,toe}/H_{s,0} < 0.9$.

Table 3: Design Constraints for Example Study

	Parameter	Value
Constraint 1	$h / H_{s,toe}$	2.8750
Constraint 2	$H_{s,toe} / H_{s0}$	0.9188
Constraint 3	$H_{2\%} / H_{s,toe}$	1.4744

According to The Rock Manual (2007), Van Gent et al. (2004) formula is recommended to be used since Constraint 1 is smaller than 3. However, even this parameter is so close to the limiting value 3 for the hypothetical example study, armour stone weight calculated by Van Gent et al. (2004) approach is much bigger than armour stone weight calculated by Van der Meer (1988) approach. On the other hand, Constraints 2 and 3 are not satisfied for this hypothetical example study. Therefore, this difference in calculated armour stone sizes is questioned considering Constraints 2 and 3 in addition to Constraint 1.

COMPUTATIONAL TOOL

To investigate effect of design constraints in finding armour stone sizes, a computational tool is developed in MATLAB environment. This tool consists of two parts, namely, Wave Transformation and Regular Wave Breaking (WT) and Design Armour Stone (DAS).

WT is a basic wave transformation and regular wave breaking code that is used to transform design wave properties from deep water to the depth at the toe of the structure. WT uses outputs of Van der Meer 1D Energy Decay Model (Van der Meer, 1990) for wave transformation. If the case is out of limits of Van der Meer 1D Energy Decay Model, wave height at the toe of the structure is computed by multiplying deep water design wave height by shoaling coefficient (K_s) and refraction coefficient (K_r) that are given for regular waves. Furthermore, WT computes wave breaking depth and breaking wave height for regular waves to check the breaking condition of the design wave for Hudson approach (CERC, 1977).

DAS computes armour stone size using Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) and Van Gent et al. (2004) approaches in addition to design constraints.

In both WT and DAS, relations between significant wave period, peak wave period and mean wave period are taken from Goda (2000) for a JONSWAP P-Type Spectrum with a shape coefficient of 3.3 whereas relation between peak wave period and spectral mean energy period is taken from Dingemans

(1987). Furthermore, wave height exceeded by 2% of the waves in a certain storm is calculated using the methodology given by Battjes and Groenendijk (2000).

A COMPARATIVE STUDY ANALYZING MAJOR STABILITY FORMULAS

A comparative study analyzing major stability formulas is carried out to observe the effect of design constraints in a wide range of application with selected deep water design wave characteristics. This comparative study mainly aims to visualize differences in application of Van der Meer (1988) and Van Gent et al. (2004) approaches by extending previously presented example study. The parameters used in this comparative study were selected covering the wide range of cases that can be encountered in practice to show the trend of differences obtained from Van der Meer (1988) and Van Gent et al. (2004) approaches effectively.

In order to compute armour stone sizes in a wide range of application, deep water significant wave steepness (H_{s0}/L_0) was fixed and deep water significant wave height (H_{s0}) was increased by 10 cm increments across a logically applicable range. Significant wave period (T_s) was computed for each deep water significant wave height (H_{s0}). Using deep water significant wave properties and conventional design parameters given in Table 4, armour stone sizes were computed for each set of parameters using computational tools WT and DAS.

Table 4: Design Parameters for Comparative Study		
Parameters for all Approaches		
Deep Water Significant Wave Steepness	H_{s0}/L_0	0.04
Range of Deep Water Significant Wave Height	H_{s0} (m)	2.5-8
Depth of Construction or Depth at the Toe of the Structure	h (m)	8
Structure Slope	cot(α)	2
Foreshore Slope	m	0.03
Specific Weight of Armour Stone	γ_{stone} (t/m^3)	2.7
Specific Weight of Sea Water	γ_{water} (t/m^3)	1.02
Deep Water Wave Approach Angle	α_0 (˚)	0
Parameters for Hudson (CERC, 1977; CERC, 1984) Approach		
Stability Parameter for Non-Breaking Case	$K_{D,non\text{-}breaking}$	4
Stability Parameter for Breaking Case	$K_{D,breaking}$	2
Parameters for Van der Meer (1988) and Van Gent et al. (2004) Approaches		
Notional Permeability	P	0.4
Damage Level	S	2
Number of Waves	N	1000

In Figures 1 and 2, the results obtained from comparative study were presented. In Figure 1, horizontal axis was selected as first design constraint ($h/H_{s,toe}$) and vertical axis was armour stone diameter (D_{n50}). Satisfied design constraints were indicated by the use of arrows at the top of the figure and different major stability equations were indicated by colors. In Figure 1, armour stone diameters bigger than 2.5 m were results obtained for breaking case of Hudson (CERC, 1977; CERC, 1984) approaches. It is seen in Figure 1 that difference between Van der Meer (1988) and Van Gent et al. (2004) approaches decreases when the design constraints are applied. To view these differences more effectively, percent relative differences in armour stone diameter between Van der Meer (1988) and Van Gent et al. (2004) approaches versus $h/H_{s,toe}$ is plotted as Figure 2. Percent relative difference in armour stone is calculated by dividing absolute value of the difference in armour stone diameter obtained from both formulas by armour stone diameter computed by Van der Meer (1988) formula multiplied by 100. In Figure 2, design constraints were indicated by different colors.

It is clearly seen from Figure 2 that relative difference in armour stone diameter is about 20% without using any design constraint in calculations. However, when the design constraints are used, relative difference decreases around 5% percent which can be regarded as very shallow water.

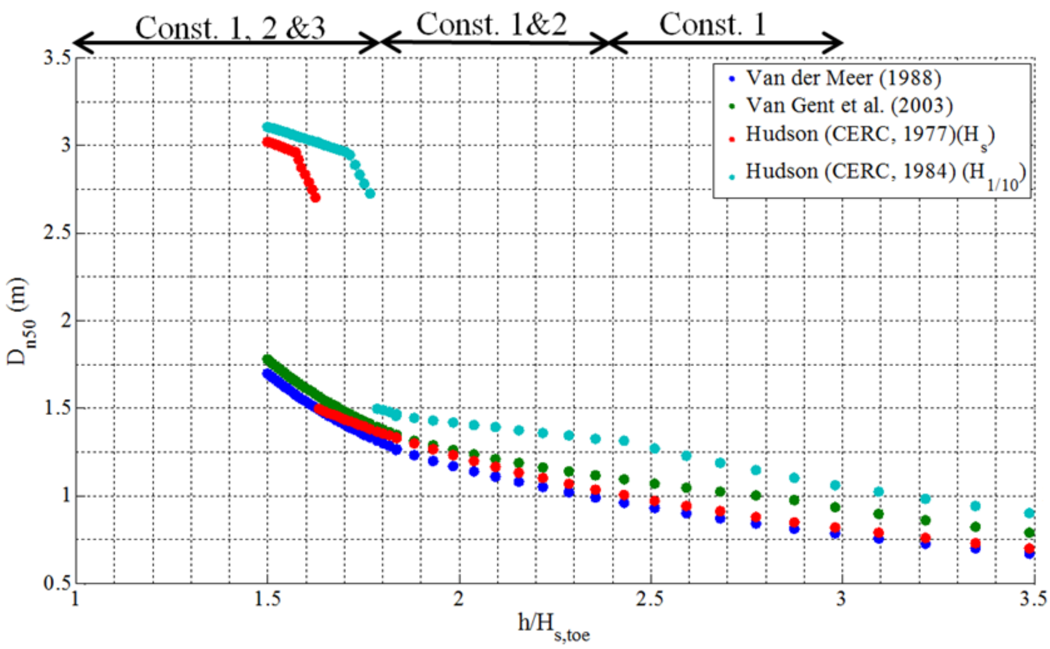

Figure 1: $h/H_{s,toe}$ vs D_{n50} for Comparative Study

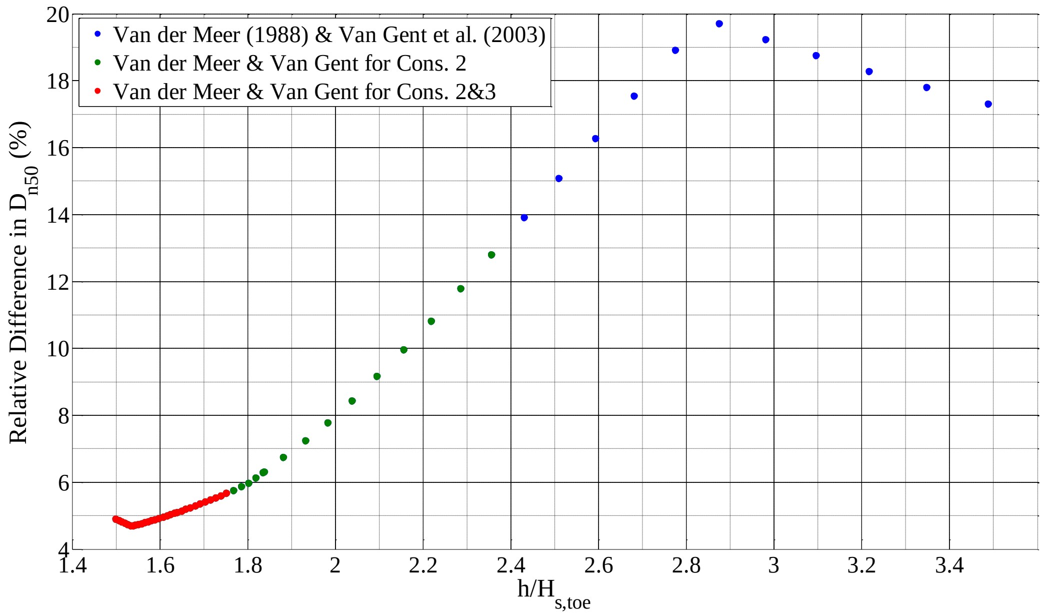

Figure 2: $h/H_{s,toe}$ vs Percent Relative Differences in D_{n50} for Comparative Study

Figures 1 and 2 shows that difference in armour stone size between Van der Meer (1988) and Van Gent et al. (2004) approaches decreases to about 5% relatively when all the design constraints are satisfied. Moreover, an early version of this study performed by Guler (2013) with deep water significant wave steepness of 0.04, deep water significant wave height range of 3-8 m and construction depth of 10 m showed same trend as in this study. Van Gent et al. (2004) approach is more conservative and may be more appropriate to use at very shallow water satisfied by design constraints due to complexities in the shallow water regions. Furthermore, spectral mean energy wave period describes shallow water processes more efficiently since it takes the influence of spectral shape into consideration. Beyond very shallow water, it is seen that there can be up to 70% relative difference between Van der Meer (1988) and Van Gent et al. (2004) approaches both by this study and the previous comparative study given by Guler (2013). Since Van der Meer (1988) approach is tested in practice widely at deep and moderate shallow water, it seems more appropriate to use this formulation in this range. These discussions are evaluated as proposition of a new design flowchart that uses Van der Meer (1988) and Van Gent et al. (2004) approaches. According to this new design flowchart (Figure 3), when all the design constraints are satisfied, Van Gent et al. (2004) approach; otherwise, Van der Meer (1988) approach is recommended.

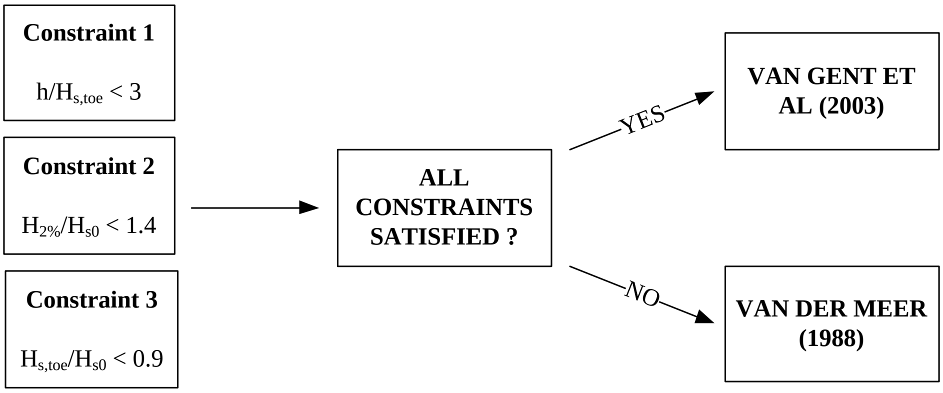

Figure 3: New Design Flowchart

PHYSICAL MODEL EXPERIMENTS

In order to test the validity of proposed design flowchart, physical model experiments were carried out in the wave flume of METU Department of Civil Engineering Ocean Engineering Research Center. Physical model experiments were conducted under the action of irregular wind waves using a conventional breakwater cross-section for two selected cases.

Details of cases selected to test validity of proposed design flowchart are given in Table 4. All design constraints were satisfied and the same conventional rubble mound breakwater cross-section was used in the experiments for both cases; however, design wave properties and depth at the toe of the structure were different for each case. Thus, Van der Meer (1988) formula and Van Gent et al. (2004) formulas are recommended for Cases 1 and 2, respectively, according to proposed design flowchart. Therefore, it is expected to observe considerable damage in Case 1 and no damage in Case 2.

Table 4: Design Parameters for Case 1 and Case 2 *(in prototype scale)*

Parameters		**Case 1**	**Case 2**
Deep Water Significant Wave Height (m)	H_{s0}	5.20	4.50
Significant Wave Period (sec)	T_s	8.10	7.60
Deep Water Wave Steepness	H_{s0}/L_0	0.050	0.049
Foreshore Slope	m	0.033	0.033
Water Depth at the toe of the structure (m)	h	8.00	7
Significant Wave Height at the toe of the structure (m)	$H_{s,toe}$	4.60	4.03
Unit Weight of Stone (tons/m^3)	γ_s	2.7	2.7
Unit weight of water (tons/m^3)	γ_w	1.025	1.025
Structure Face Slope	$\cot(\alpha)$	2	2
Notional Permeability	P	0.4	0.4
Number of waves	N	1000	1000
Damage Level	S	2	2
Stone Weight according to VdM*	$W_{50,VdM}$	4-6 tons	2-4 tons
Stone Weight according to VG**	$W_{50,VG}$	6-8 tons	4-6 tons
Constraint 1	$h/H_{s,toe}$	1.739	1.738
Constraint 2	$H_{2\%}/H_{s,toe}$	1.301	1.301
Constraint 3	$H_{s,toe}/H_{s0}$	0.880	0.895
Design Formula used		VdM*	VG **
Design Formula that should be used according to New Design Flowchart		VG**	VG **
** VdM: Van der Meer (1988) formula* *** VG: Van Gent et al. (2004) formula*			

Dimensions of cross-section were determined considering 4-6 tons armour stone size and conventional design recommendations for no damage case (CEM, 2003; The Rock Manual, 2007). Froude type scaling was used to scale cross-section with a length scale of 1/20. Scaled cross-section is given in Figure 4.

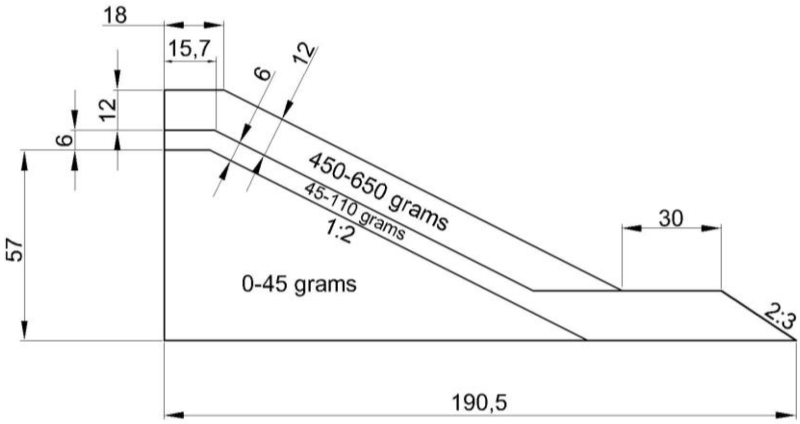

Figure 4: Scaled Cross-Section
(Dimensions are in centimeters and figure is not to scale.)

Experimental setup was built using appropriate size of stones, a 1/30 slope and 6 wave measurement gauges. Dimensions of wave channel are 28.8m X 6.2m X 1.0m. Side view of the experimental setup including wave measurement gauges are given in Figure 5. Measurement gauges were placed appropriately to consider reflections from ends of wave channel by using methodology given by Goda and Suzuki (1976).

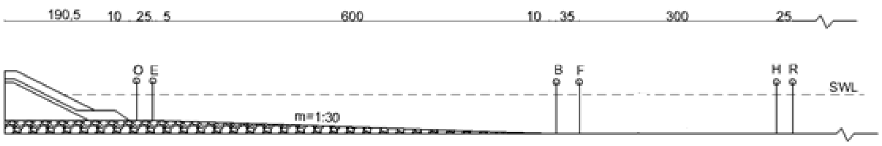

Figure 5: Side View of the Experimental Setup
(Dimensions are given in centimeters and figure is not to scale. O, E, B, F, H and R are wave measurement gauges.)

Cross-section in the wave channel is given in Figure 6. Experiments were conducted three times (sets) for each case. For each set of experiments, profile of cross-section was measured before and after each experiment along two different lines with 5 cm intervals. Measurement lines are indicated in Figure 6. Damage was calculated for each set of experiment. To define damage level (S), average of eroded area (A_e) measured along two lines was divided to square of diameter of armour stone given by Equation 10.

$$S = \frac{A_e}{D_{n50}^2} \qquad (10)$$

Figure 6: Cross-Section in the Wave Channel

Results of all sets of experiments are given in Table 5 presenting both inputs and measurements of physical model experiments. It is seen from the results that all design constraints were satisfied in each set of experiments. Furthermore, Case 1 resulted in intermediate damage level since damage parameter for each set of experiment was between 4-6 (The Rock Manual, 2007) and Case 2 resulted in no damage since damage parameter for each set of experiment was around 2 (The Rock Manual, 2007) which meets with expectations prior to experiments. Thus, validity of the proposed flowchart was tested and approved within these limited number of physical model experiments.

Table 5: Results of Physical Model Experiments

			Case 1			**Case 2**		
	Parameters		**Set 1**	**Set 2**	**Set 3**	**Set 1**	**Set 2**	**Set 3**
INPUT	Deep Water Significant Wave Height (m)	H_{s0}	5.20	5.20	5.20	4.50	4.50	4.50
	Significant Wave Period (sec)	T_s	8.10	8.10	8.10	7.60	7.60	7.60
	Water Depth at the toe of the structure (m)	h	8.00	8.00	8.00	7.00	7.00	7.00
	Significant Wave Height at the toe of the structure (m)	$H_{s,toe}$	4.60	4.60	4.60	4.03	4.03	4.03
MEASURED	Significant Wave Height at the toe of the structure (m)	$H_{s,toe}$	4.59	4.62	4.64	3.99	4.02	4.04
	Significant Wave Period (sec)	T_s	8.07	8.11	8.12	7.57	7.64	7.68
	Constraint 1	$h/H_{s,toe}$	1.74	1.73	1.72	1.75	1.72	1.72
	Constraint 2	$H_{2\%}/H_{s,toe}$	1.31	1.30	1.32	1.31	1.31	1.32
	Constraint 3	$H_{s,toe}/H_{s0}$	0.88	0.89	0.89	0.89	0.89	0.89
	Number of Waves	N	1061	1082	1048	1012	1059	1067
	Van der Meer Damage Parameter	S	4.79	6.25	4.99	1.34	2.22	1.85
	Damage Level		Intermediate Damage			No Damage		

A CASE STUDY: ALIAGA, IZMIR, TURKEY

A case study in Aliaga, Izmir, Turkey was carried out in order to show importance of the proposed design flowchart. Site specific wind, wave and bathymetry data were used for the case study. Wind data was taken from European Centre for Medium-Range Wave Forecasts (ECMWF) at point 38.8N-26.5E for the case study region between 1983 and 2010. In Figure 7, case study region and wind data point were given.

Figure 7: Case Study Region and Wind Data Point

Wave hindcasting studies (Guler, 2014) were done using "Deep Water Wave Hindcasting Mathematical Model, W61" developed by METU Ocean Engineering Research Center (Ergin and Ozhan, 1986). Using hindcasted wave data, extreme term wave statistics studies were performed (Goda, 2000). Extreme term wave statistic studies resulted with a deep water significant wave height of 4.05 m and significant wave period of 7.72 sec for a return period of 100 years and this wave was selected as design wave. Design wave properties were transformed to the toe of the structure (Guler, 2014) using a bathymetry obtained for project site. Inputs of Design Armour Stone (DAS) code are presented in Table 6.

Table 6: Inputs of Design Armour Stone (DAS) Code		
Parameters		**Value**
Deep Water Significant Wave Height	H_{s0} (m)	4.05
Significant Wave Period	T_s (sec)	7.72
Deep Water Wave Approach Angle	α_0 (°)	0
Water Depth at the Toe of the Structure	h_{toe} (m)	8
Significant Wave Height at the Toe of the Structure	$H_{s,toe}$ (m)	3.96
Van der Meer Damage Parameter	S	2
Notional Permeability	P	0.4
Unit Weight of Stones	γ_{stone} (t/m^3)	2.7
Unit Weight of Water	γ_{water} (t/m^3)	1.025
Structure Face Slope	cot(α)	2

Armour stone size and design constraints were calculated by DAS and the results were presented in Table 7. It is seen that Van der Meer (1988) approach gave an armour stone weight of 3.8 tons whereas Van Gent et al. (2004) approach gave 7.3 tons. According to The Rock Manual (2007), Van Gent et al. (2004) approach should be used since first design constraint ($h/H_{s,toe}$) is less than 3. However, there is a 92% relative difference between both approaches for this case. On the other hand, Van der Meer (1988) approach should be used since only first design constraint is satisfied according to proposed design flowchart. Thus, it is shown that design flowchart might have great importance in practice considering drastic cost and application problems.

Table 7: Results obtained by Design Armour Stone (DAS) Code		
Parameter		**Value**
Design Constraint 1	$h/H_{s,toe}$	2.02
Design Constraint 2	$H_{2\%}/H_{s,toe}$	1.53
Design Constraint 3	$H_{s,toe}/H_{s0}$	0.98
Armour Stone Weight: Van der Meer (1988) Approach	W_{VdM} (tons)	3.8
Armour Stone Weight: Van Gent et al. (2004) Approach	W_{VG} (tons)	7.3

SUMMARY AND CONCLUSION

Rubble mound breakwaters are important coastal defense structures that are widely used all around the world. Major stability equations used to design these type of breakwaters are Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) and Van Gent et al. (2004) formulations. In this study, discrepancies in application of Van der Meer (1988) and Van Gent et al. (2004) formulations were clarified by conducting an example study. Furthermore, a comparative study that covers a wide range of application was performed to visualize trend of differences between both formulations. Hudson (CERC, 1977; CERC, 1984) approaches were provided as a well-known measure.

Example study and comparative study showed that there can be up to 70% relative difference in armour stone weight between Van der Meer (1988) and Van Gent et al. (2004) approaches. This relative difference was investigated considering three design constraints. It is seen that relative difference decreases to 4-6 % when all design constraints are applied. Due to complexity of shallow water regions, this difference is meaningful in application. Furthermore, spectral mean energy wave period used in Van Gent et al. (2004) approach defines shallow water regions more effectively since it takes influence of spectral shape. In the light of these discussions, a new design flowchart that uses Van der Meer (1988) and Van Gent et al. (2004) approach is proposed. This design flowchart recommends to use Van Gent et al. (2004) approach if all the design constraints are satisfied; on the other hand, Van der Meer approach should be used if even one of the design constraints is not satisfied.

In order to show validity of this design flowchart, physical model experiments were carried out in the wave flume of METU Department of Civil Engineering, Ocean Engineering Research Center. Two cases were selected to test proposed design flowchart. Design constraints were satisfied in both cases; however, rubble mound breakwater cross-section is designed using Van der Meer (1988) approach in the first case and Van Gent et al. (2004) approach in the second case. Therefore, considerable damage and no damage condition were expected for the first case and second case, respectively. Three sets of experiments were conducted for both cases and results of experiments met with expectations before physical model experiments. Hence, validity of proposed design flowchart is approved within the limited number of physical model experiments.

In the final part of this study, a case study in Aliaga, Izmir, Turkey was conducted to show the importance proposed design flowchart. It is shown that application of Van der Meer (1988) and Van Gent et al. (2004) may result in 92% relative difference which means drastic cost and application problems in practice for a real case. Therefore, it is important to use design constraints to determine which formulation is applicable for the case.

In view of this study, design of rubble mound breakwaters should be carefully carried out considering design constraints. In the future, number of physical model experiments should be extended to increase reliability of newly proposed design flowchart in this study.

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