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**DOI**

[10.3850/IAHR-39WC2521711920221364](https://doi.org/10.3850/IAHR-39WC2521711920221364)

**Publication date**

2022

**Document Version**

Final published version

**Published in**

Proceedings of the 39th IAHR World Congress (Granada, 2022)

**Citation (APA)**

Marino, S., Scaravaglione, G., Francone, A., Valentini, N., Saponieri, A., Damiani, L., van Gent, M. R. A., & Tomasicchio, G. R. (2022). Laboratory investigation on armour stability for extremely shallow water conditions. In M. Ortega-Sánchez (Ed.), *Proceedings of the 39th IAHR World Congress (Granada, 2022)* (pp. 5973-5979). (Proceedings of the IAHR World Congress). <https://doi.org/10.3850/IAHR-39WC2521711920221364>

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## Laboratory investigation on armour stability for extremely shallow water conditions

Stefano Marino<sup>1</sup>, Giulio Scaravaglione<sup>1</sup>, Antonio Francone<sup>2</sup>, Nico Valentini<sup>3</sup>,  
Alessandra Saponieri<sup>2</sup>, Leonardo Damiani<sup>1</sup>, Marcel R.A. van Gent<sup>4</sup> and Giuseppe Roberto Tomasicchio<sup>2</sup>

<sup>1</sup> Dept. of Civil, Environmental, Building Engineering and Chemistry, Polytechnic University of Bari, Bari, Italy  
stefano.marino@poliba.it, giulio.scaravaglione@poliba.it, leonardo.damiani@poliba.it, ,

<sup>2</sup> Dept. of Engineering for Innovation, University of Salento, Lecce, Italy  
antonio.francone@poliba.it, alessandra.saponieri@unisalento.it, roberto.tomasicchio@unisalento.it

<sup>3</sup> BRGM Université de Montpellier, Montpellier, France  
n.valentini@brgm.fr

<sup>4</sup> Dept. of Hydraulic Engineering, TU Delft and Deltares, Delft, the Netherlands  
marcel.vangent@deltares.nl

### Abstract

The hydraulic stability of the armour layer of rubble mound breakwaters has been widely discussed in literature and several empirical formulas have been proposed for design purposes. Nevertheless, few studies focused on the armour stability in case of water depth limited water conditions. The aim of the present research is to provide more insights into the stability of the armour layer including extremely shallow water conditions. New 2D physical experiments have been carried out at the European Maritime and Environmental Research (EUMER) laboratory of the University of Salento (Lecce), in presence of a 1:30 foreshore. Tests aimed to investigate both the nearshore hydrodynamics and the hydraulic armour stability under different irregular wave loading.

In the present paper the experimental set-up is described together with the main preliminary results. Specifically, the measured damage  $S$  after each test in different shallowness conditions is reported and discussed. Experimental results are also compared with the damage evaluated from the empirical formula proposed by Van Gent et al. (2003), in order to investigate the predicted damage level in the shallowness range analysed.

**Keywords:** Rubble mound breakwater; Shallow waters; Wave-structure interaction; Hydraulic stability; Rock armour.

## 1 INTRODUCTION

The hydraulic stability of a rubble mound breakwater is a key topic in coastal engineering, widely studied over the years. The stability is expressed through the well-known Hudson formula (Hudson, 1959), which relates the stability number ( $N_s = H_{s,t}/\Delta D_{n50}$ ) to the wave height of the incident waves at the toe of the structure,  $H_{s,t}$ , the reduced relative density ( $\Delta$ ), the nominal diameter of the armour layer ( $D_{n50}$ ), the angle of the front slope of the structure and a dimensionless parameter that depends on the damage level of the armour layer. Van der Meer (1988) improved the Hudson's formula, through analysis of the tests by Thompson and Shutter (1975) and a large experimental campaign aimed at investigating the influence of further parameters (e.g., porosity, number of waves, wave steepness, type of armour layer) on the structure stability, mainly in deep water conditions.

Coastal structures such as rubble mound breakwater are often built close to the coast in shallow waters. In this context, the wave behavior is affected by shoaling and breaking, then, the interactions with a structure drastically change with respect to deep water conditions.

Smith et al. (2003) addressed the study of the hydraulic stability of a rock slope in shallow waters through an experimental campaign conducted with a 1:100 foreshore and with different wave conditions (single and double peaked spectra and breaking wave). Experimental results showed that the empirical formulations used in Van der Meer (1988) are very sensitive to the choice of input parameters (statistical or spectral wave characteristics). Van Gent et al. (2003) extended the available dataset on the hydraulic stability in depth-limited water conditions by performing new laboratory experiments. Tests were aimed to improve the formulations proposed by Van der Meer (1988), including conditions with shallow foreshores. Furthermore, in addition to a modified version of the formula by Van der Meer (1988), a new stability formula was proposed,

not depending by the wave spectral period  $T_{m-1,0}$ , but simply by structure characteristics (the structure slope angle  $\alpha$ , and the nominal rocks diameter both of armour and core  $D_{n50}$  and  $D_{n50, \text{core}}$ ).

Recently, Herrera et al, (2017), proposed a new design formula (Eq.1), based on new tests conducted with a double armour layer with a 1:50 foreshore and for breaking waves.

$$S = 0.066 \left( \frac{H_{m0,t}}{\Delta D_{n50}} \right)^{1/6} \quad [1]$$

The authors found out that the best fit for the Eq. [1] is obtained for a spectral wave height ( $H_{m0,t}$ ) measured at a distance of  $3h_t$  (water depth at the slope toe) seaward from the structure toe. Eldrup et al. (2019) investigated the hydraulic stability of a conventional rubble mound breakwater in shallow waters, focusing on the effects of nonlinear waves. To this aim, new experimental tests have been performed with a foreshore slope of 1:30 and 1:100. They revisited the stability design formulae reported in Van der Meer (1988) and later modified in Van Gent et al. (2003), achieving a new reliable formulation based on the newly acquired data.

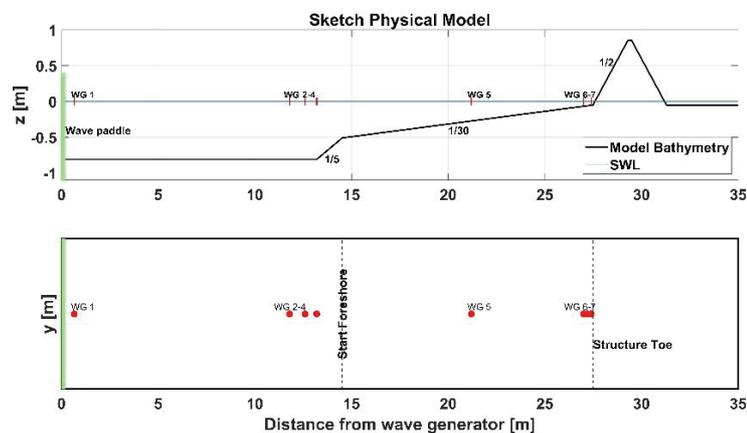
The present paper aims to explore the influence of depth-limited water conditions on the rock stability in presence of intermediate to extremely shallow waters. To this end, new tests have been performed on a 2D physical model in presence of a mild slope foreshore ( $\tan \theta=1:30$ ), in order to evaluate the damage of the structure with varying stone diameters and different water depths.

## 2 EXPERIMENTAL SET-UP

The experimental campaign has been carried out in a wave flume 45 m long, 1.4 wide and 2 m deep at the EUROpean Maritime and Environmental Research laboratory (EUMER) of the University of Salento (Lecce, Italy). The facility is equipped with a piston type wave paddle which movements are remotely controlled in order to perform tests with regular or irregular waves. An active absorption system is present to reduce the wave reflection during the tests.

### 2.1 Physical model

A concrete foreshore 14.3 m long with a slope  $\tan \theta=1:30$  has been constructed inside the flume; a 1:5 transition slope is present between the foreshore and the bottom of the flume. The foreshore starts at about 15 m away from the paddle such that the waves approaching the structure are fully adapted to the foreshore slope. The structure, placed on a horizontal bed, consist of natural rocks, arranged to have a front slope  $\alpha = 1:2$ . The slope is 0.9 m high and 1.8 m long (Figure 1).



**Figure 1.** The top panel is a sketch section of the physical model; the position of the wave probes is reported as well; The bottom panel is a plan view of the model.

Four different rock gradings have been collected and sieved (Figure 2a), each one used in tests with different water depths. The shape of rocks has been characterized through two parameters, the aspect ratio (LT, length to thickness) and the Blockness coefficient ( $BLc = 100\% \text{ rock volume} * 1/XYZ$ , with X, Y, Z the dimensions of the smallest box that can enclose a rock), which have been determined following the procedures reported in the Rock Manual (2007). These parameters have been retrieved by averaging the values of 20 samples taken randomly from the grain population of three different rocks (Figure 2b). The results of this procedure are reported in Table 1.

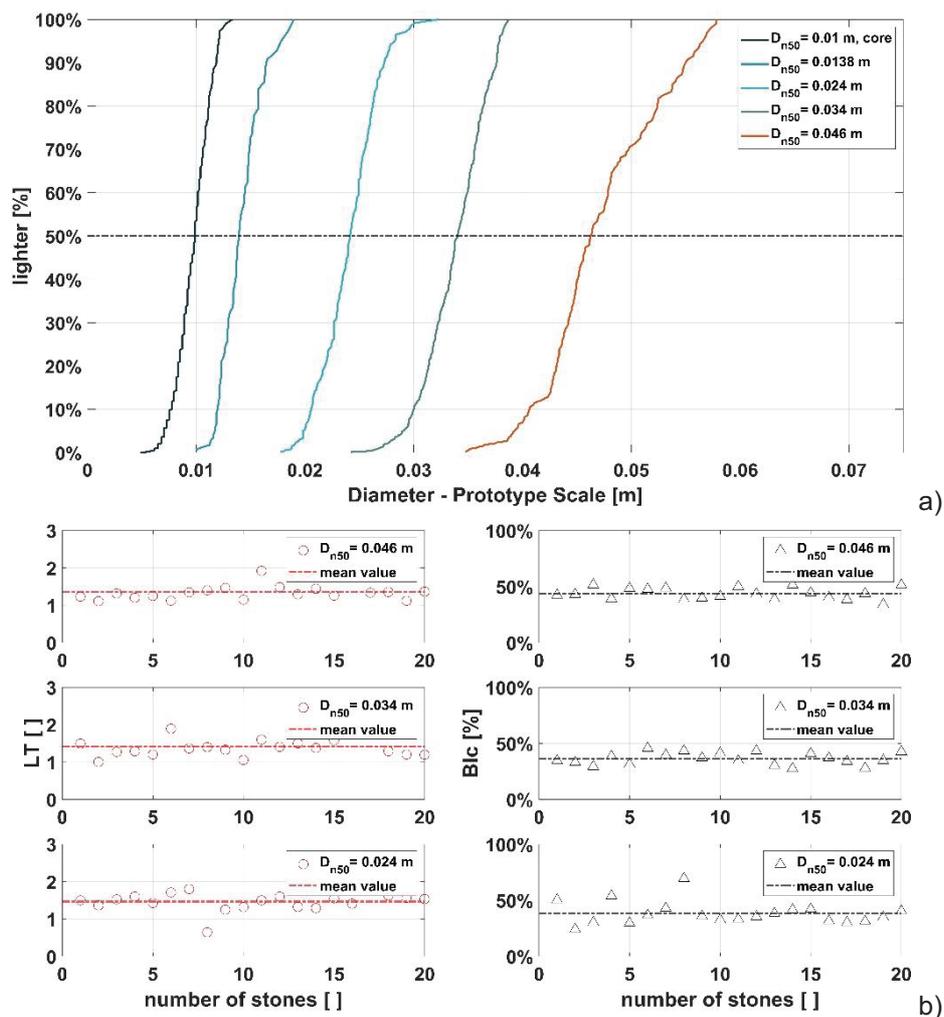


Figure 2. a) Rocks gradings; b) rocks shape factors.

Table 1. Rocks characteristics

	WL1	WL2	WL3	WL4
$D_{n50}$ – armour [m]	0.046	0.034	0.024	0.0138
$D_{n50}$ – under layer [m]	0.024	-	-	-
$D_{n50}$ – core [m]	0.01	0.01	0.01	0.01
B1c [%]	43.79	36.28	38.18	-

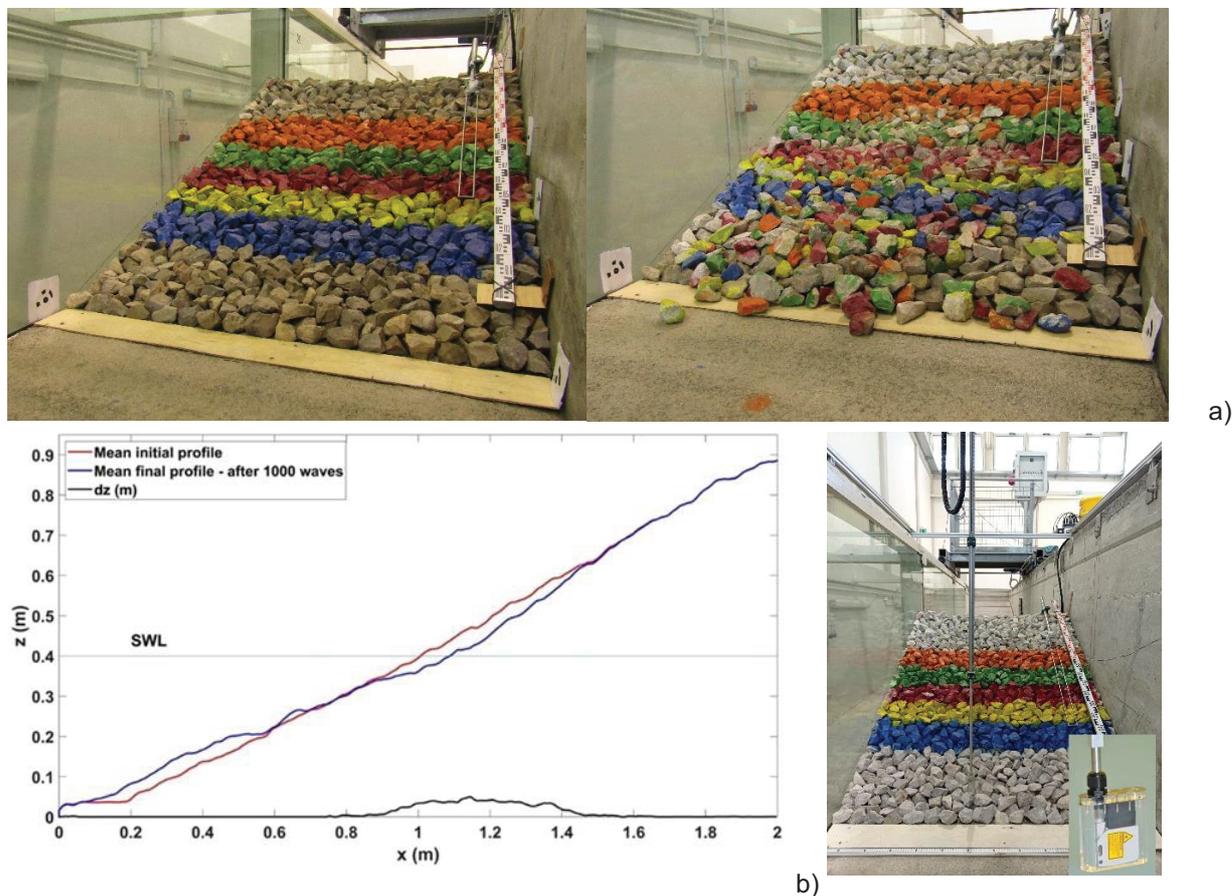
## 2.2 Damage measurement

The damage level of the structure is defined by Eq [2] as the ratio between the cross sectional eroded area ( $A_e$ ) to the nominal diameter of the armour layer ( $D_{n50}$ ) as also used by Van der Meer (1988).

$$S = \frac{A_e}{D_{n50}^2} \quad [2]$$

The eroded area has been retrieved by measuring the cross-shore profile of the structure before and after each test. The measurements have been made with a laser bed profiling system (Figure 3c) along 10 cm equally spaced 10 transversal sections. The laser probe is able to move with a horizontal and vertical velocity, respectively up to 50 mm/s and 25 mm/s with a resolution in both x and z directions equal to +/- 0.5 mm.

In (Figure 3.b)  $d_z$  (black line) is the eroded profile which has been obtained as the absolute difference between the mean final and initial profiles. The  $d_z$  negative values have been set to zero. Finally, the eroded area  $A_e$  has been calculated as the integral of the black line ( $d_z$ ) graph. The failure of the slope is assumed with the exposure of the underlayer. As an example, in Figure 3 the observed initial and final profiles for an irregular test after 1000 waves are reported.



c) **Figure 3.** a) Rock slope before and after wave attack; b) mean initial and final slope profile after wave attack and the eroded profile (black line); c) laser bed profiling system.

### 2.3 Test programme

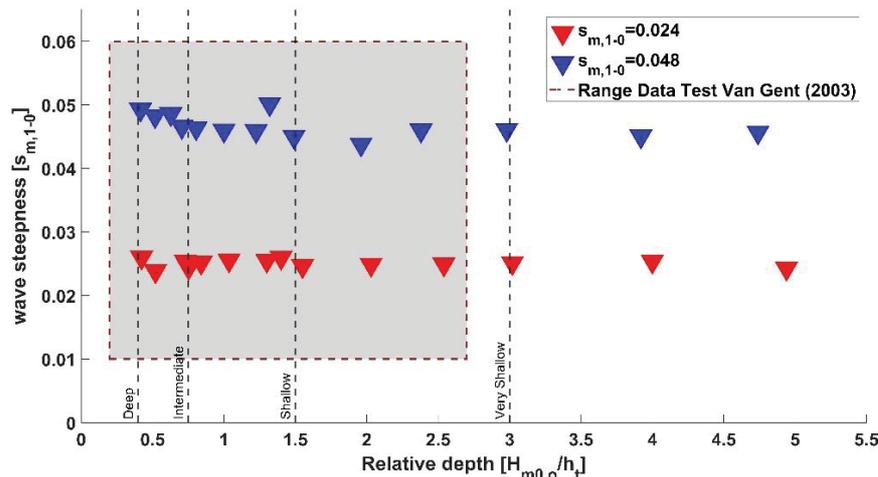
Four different layouts of the structure (WL1-WL4) have been tested. In Table 2 the main bulk wave parameters (offshore wave steepness  $s_{m-1,0} = 2\pi H_{m0,o} / (gT_{m-1,0,o}^2)$ , where  $H_{m0}$  and  $T_{m-1,0}$  represents the wave spectral characteristics in deep water), surf-similarity parameter measured at the toe  $\xi_{m0} = \tan\alpha / (s_{m-1,0,t})^{0.5}$ , the slope damage (S), stability number ( $N_s$ ) and the rocks characteristics ( $D_{85}$ ,  $D_{15}$ ,  $D_{n50}$ ,  $D_{n50,core}$ , the aspect ratio, LT) of the present tests have been reported along with the dataset of Van Gent et al. (2003). The relative depth, corresponding to the ratio between the deep spectral wave height ( $H_{m0,o}$ ) to the water depth at toe of the structure ( $h_t$ ), has been used as the criterion to rank the shallowness conditions according to Van Gent (1999). A similar classification is described in Hofland et al. (2017).

**Table 2.** Main wave and structure parameters.

Main parameters	WL1	WL2	WL3	WL4	Van Gent, 2003
Relative depth - $H_{m0,o}/h_t$ (-)	0.43-0.78	0.75-1.48	1.46-2.56	2.94-5.08	0.2-2.7
Wave steepness - $s_{m-1,0}$ (-)	0.024-0.048	0.024-0.048	0.024-0.048	0.024-0.048	0.01-0.06
Breaker parameter - $\xi_{m0}$ (-)	2.58-3.15	2.45-5.4	5.6-11.4	11.1-19.6	1.3-15
Slope damage - $S$ (-)	0-12	0-12	0-12	0-12	<62
Stability number - $H_{m0,t}/\Delta D_{n50}$ (-)	2.21-3.8	2.67-2.9	2.36-3.87	2.76-4.1	0.5-4.5
Grading slope material - $D_{85}/D_{15}$	1.26	1.27	1.20-1.28	1.28	1.4-2.0
Core material - $D_{n50,core}/D_{n50}$	0.21	0.29	0.41-0.71	0.71	0-0.3
Aspect ratio - $LT$ (-)	1.35	1.41	1.46	-	2.1

In total 31 tests of 1000 waves have been carried out. For 15 tests the damage has been measured also after 3000 waves. For each water depth, the tests have been grouped in two series of constant wave steepness ( $s_{m-1,0}$ ), respectively 0.024 and 0.048 as shown in Figure 4. Note that for WL4 data, not present in literature yet, have been acquired in the field of the extremely shallow waters (for values of relative depth greater than 3).

Since no reflection analysis is suitable in depth-limited conditions (Van Gent, 1999; Altomare et al., 2016), the same tests have been repeated without the structure in place, to accurately measure the incident wave characteristics at the slope toe. The data analysis will refer only to these quantities.



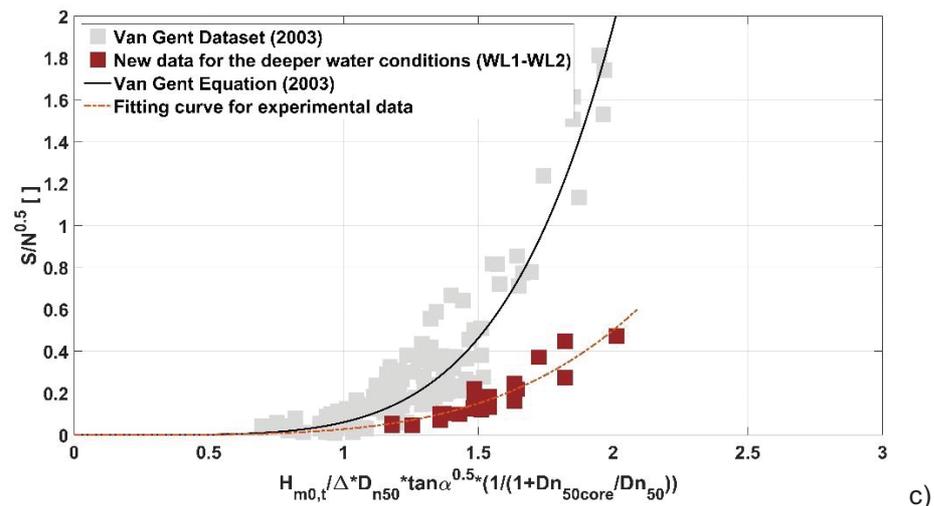
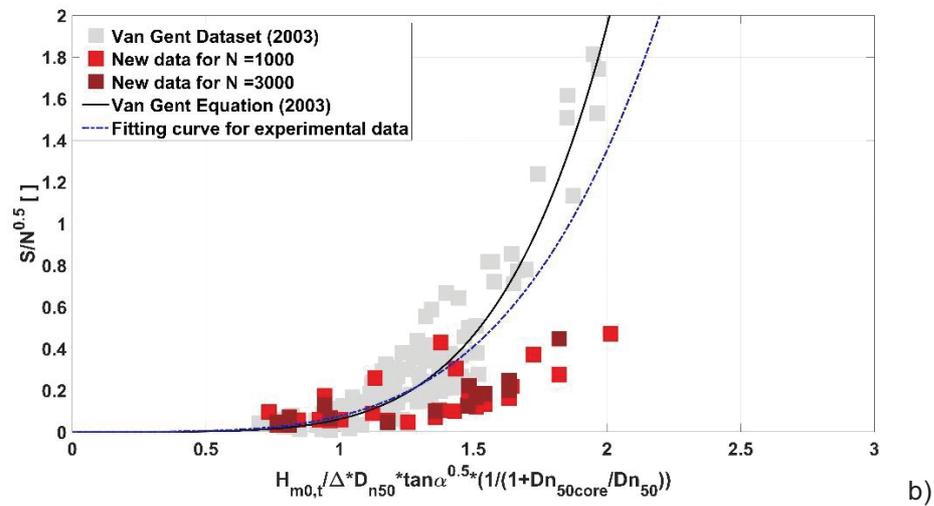
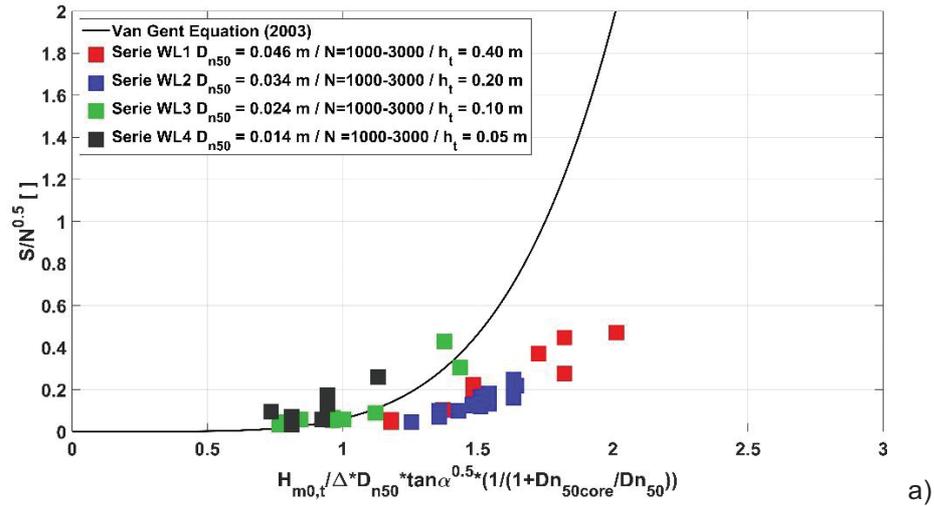
**Figure 4.** Classification of the performed test as a function of the relative depth and wave steepness; the gray area represents the range of Van Gent et al. (2003) dataset respect of these parameters.

### 3 MAIN RESULTS AND CONCLUSIONS

The tests have been carried out to investigate the rock slope stability in different water depth-limited conditions, from intermediate until extremely shallow. The slope damage has been quantified for each test by comparing, the mean initial and final slope profile (Figure 3.b).

In Figure 5 the dimensionless damage ( $S/N^{0.5}$ ), where  $N$  is the number of waves, has been plotted against the stability number, which has been retrieved by the Eq. [3]. In the analysis, the spectral wave height has been used instead of the statistical one, as suggested in Eldrup et al. (2019). Particularly, Figure 5.a shows the data classified in terms of both water depth at the slope toe and rock armour diameter. In Figure 5.b the present data have been plotted together with the design formula of Van Gent et al. (2003), which tends to overestimate the new acquired data. One of the reasons could be that the rock's characteristics (e.g., aspect ratio) are different from the ones used to calibrate the formula.

$$\frac{H_s}{\Delta D_{n50}} = 1.75 \cot \alpha^{0.5} \left( 1 + \frac{D_{n50,core}}{D_{n50}} \right) \left( \frac{S}{\sqrt{N}} \right)^{\frac{1}{5}} \quad [3]$$



**Figure 5.**a) New data classified in terms of water depth and rock armour diameter; b) Present data compared to design formula and data by Van Gent et al. (2003); c) Data in the deeper water conditions tested (Test Serie WL1 and WL2), compared to design formula.

The black line in Figure 5.b represents the Van Gent et al (2003) equation. The shape of the curve is governed by both an exponential ( $a = 0.2$ ) and multiplying parameter ( $b = 1.75$ ) (Eq.3). Instead, the blue line shows the fitting of the present data with the dataset of Van Gent et al. (2003), for which the coefficient of

determination (Eq.4),  $R^2$ , is 0.806 and the root mean square error (RMSE) is 0.198. The curve parameters, a and b, have been retrieved by means a nonlinear regression analysis and they are respectively equal to 0.24 and 1.86, close to the ones reported in Eq.3.

$$R^2 = 1 - \frac{\sum_{i=1}^n (y_i - \tilde{y}_i)}{\sum_{i=1}^n (y_i - \bar{y})} \quad \text{RMSE} = \sqrt{\frac{\sum_{i=1}^n (\tilde{y}_i - y_i)^2}{n}} \quad [4]$$

In conclusion, a new experimental campaign aimed at studying the hydraulic stability of rock slopes in shallow waters has been performed. In accordance with the actual shallowness classification, new data have been acquired in extremely shallow waters. The damage level, which has come up from each test, has been compared with the formula defined in Van Gent et al. (2003). The results have been overestimated by the Eq. [3] and it is worth saying that the present data seem to be clustered in two different groups, one with a rather reasonable match with the formula (shallowest conditions) and one showing less damage than the formula predicts (deeper-water conditions Figure 5.c). This behaviour needs to be further investigated. A slight modification of Eq. [3] has been applied to fit the present data for the deep water and intermediate conditions. However, the Eq. [3] should be calibrated by combining this dataset and the available bibliographical data on stability in shallow waters.

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