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# Measuring damage in physical model tests of rubble mounds

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## Abstract

This paper studies novel ways to evaluate armour damage in physical models of coastal structures. High-resolution damage data for reference rubble mound breakwaters obtained under the HYDRALAB+ joint-research project are analysed and discussed. These tests are used to analyse the way to describe damage, the influence of the sequence of testing, and touches on the possible influence of sea level rise. Results of two test programmes were used. Firstly, 3D physical model tests carried out at the University of Porto, in cooperation with Deltares, were used. Here a wide breakwater trunk was used for statistical reasons. Additionally, 2D test results from LNEC were analysed. Tests for a sea level rise scenario resulted in less damage to the seaside slope. In addition, clear differences between “cumulative damage” and “rebuild” test series were noticed. However, significant scatter was also observed in the result of tests carried out under identical conditions. It was also concluded that the damage to the trunk was lower in the tests with short-crested waves. The design values for the damage depth  $E_{2D}$  proposed by Hofland *et al.* (2011) were partly in line with the experimental results presented. Since the relation between  $S$  and the depth of damage  $E$  does not hold true for non-standard cases, it seems better to use a parameter based on the local damage depth when testing such a structure. The reliability of a damage number for a test on the stability of a trunk can be improved by either increasing the relative size (width) of the test section or repeating the test.

## Introduction

### Background

This paper presents conventional and innovative techniques to measure and evaluate armour damage in physical models of coastal structures, and is based on the results of tests from three European laboratories, carried out under the HYDRALAB+ joint-research project RECIPE (Representing Climate Change in Physical Experiments). This joint-research was motivated by the need to improve modelling, measurement and interpretation in coastal structure testing. The present paper is focussed on the results of stability tests. In two accompanying papers (Silva *et al.*, 2017; Gironella *et al.*, 2017), overtopping measurements and storm characterization are treated.

In order to cope with climate change and sea level rise induced issues, Burcharth *et al.* (2014) envisage that coastal structures can be adapted in a variety of ways, by adding extra layers, berms (Van Gent, 2013) or detached submerged breakwaters, or by increasing its crest level. These and other adapted structures will have to be evaluated for their main functions, which usually comprises damage measurements (stability). For these adapted structures that do not consist of a straight slope with a standard armour layer thickness, limited design guidance exists to evaluate the stability.

Sea level rise could also have a direct influence on the damage patterns. Due to the sea level rise, the wave attack will shift to higher elevations, and the question is whether the summation of incremental damage in terms of eroded area or depth (e.g. Melby & Kobayashi 1998) will still hold. How the damage progresses with a changing of the water level is not known.

To assess the total probability of failure, ideally many combinations of test parameters will have to be considered (wave height, water level, wave period, wave direction, etc.). In the current state-of-the-art all this variability is quantified in one or several safe so-called design conditions. The design condition is based on a response function (e.g. overtopping or damage). Because the response function is not known exactly per definition (otherwise the physical model tests were not necessary), this leads to an inherent uncertainty of this design point. In the future it would be preferable to test many more conditions in order to determine the probability of failure, or at least determine the sensitivity of the response to certain parameters. A prerequisite for enabling these kinds of analyses is to make testing more efficient. When the structure has to be rebuilt for new tests, experimental testing becomes inefficient and the total risk assessment of structures will remain out of reach. It would be preferable not to rebuild the structure after each test.

### Aim, approach and outline

The aim of this paper is to describe ways to assess damage in adapted coastal structures under variable conditions in a flexible and systematic manner. To this end, detailed scans of 2D and 3D tests on rock-armoured structures have been obtained. These are analyzed in depth to investigate the damage behaviour under varying conditions, and to obtain representative parameters to describe and qualify the stability of the adapted structures. An important point of attention is the accuracy of model testing, as the results of rubble mound damage tests are prone to a large uncertainty.

The paper starts with a brief description of existing literature on damage measurements, damage description, and damage to roundheads. Next, the measurement setups of the two test series that were used are described. This is followed by a presentation of the tests results, focussing on the damage to the trunk and the way to describe damage. The paper ends with a conclusions section.

### Literature

Damage measurements for new breakwater designs are usually still done as it has been done for decades (IAHR, 2011). However, several new measurement techniques have become available recently. With these techniques the surface elevation of rubble mound breakwaters can be obtained with millimeter resolution and sub-millimeter accuracy. The most commonly used high-resolution techniques are terrestrial laser scanning (Rigden & Steward, 2012; Molines *et al.*, 2013; Puente *et al.*, 2014), and stereo photogrammetry (Hofland *et al.*, 2011, Lemos & Santos, 2013).

The damage to a rubble mound structure can most easily be obtained from these survey techniques by regarding the difference of the structure elevation before ( $z_{\text{before}}$ ) and after ( $z_{\text{after}}$ ) a test run or test series, via:

$$e = (z_{\text{before}} - z_{\text{after}}) \cos \alpha \quad , \quad (1)$$

where  $\alpha$  is the structure slope. In this definition, erosion of the profile is positive. Damage to rubble mound structures is classically based on either the number of rocks that are displaced during a test (storm), or by the area of the cross-section of the damage hole (dimensionless erosion area  $S$ , Broderick, 1983; Van der Meer, 1988; see left panel in Figure 1):

$$S = \frac{\int_{\langle e \rangle_w > 0} \langle e \rangle_w dx}{D_{n50}^2} \quad , \quad (2)$$

where  $e$  is the erosion during a test (series),  $\langle e \rangle_w > 0$  indicates that only the positive erosion depths are taken into account,  $\langle \rangle_w$  indicates an average over the alongshore direction (flume width),  $x$  is the cross-shore coordinate along the slope, and  $D_{n50}$  is the nominal diameter of the armour rock for which 50% of the rock mass is smaller.

The modern scanning methods can be used to obtain the latter type of damage definition, but also to

obtain local damage estimates. Melby & Kobayashi (1998) treat various ways of describing damage in a trunk cross-section. The characteristic dimensions of a 2D (trunk) erosion hole are made dimensionless by the nominal rock diameter,  $D_{n50}$ . Besides  $S$ , they also treat the dimensionless erosion depth. This measure can only be applied for a 2D (flume) test, *i.e.* breakwater trunk (Hofland *et al.*, 2011):

$$E_{2D} = \frac{\max(\langle e \rangle_w)}{D_{n50}} \quad (3)$$

Hofland *et al.* (2014) additionally propose the local damage depth  $E_{3D,m}$ , which is different from the erosion depth of Melby & Kobayashi (1998) due to the inclusion of a spatial filter (circular moving average) of the erosion pattern, such that it is applicable to a variety of non-standard two and three-dimensional rubble mound structures:

$$E_{3D,m} = \frac{\max(\langle e \rangle_{mD_{n50}})}{D_{n50}} \quad (4)$$

where  $\langle \rangle_{mD_{n50}}$  indicates a spatial average over a circular area with  $mD_{n50}$  diameter. In the right panel of Figure 2 the influence of the filter size  $m$  on the determined damage depth at a roundhead is illustrated. The optimal size of the filter size, a compromise between accuracy and resolution, is not known yet, but is expected to be in the range of 3 to 5 times  $D_{n50}$ . As a flume test from which  $E_{2D}$  is determined is typically  $20D_{n50}$  wide, the filter size for  $E_{3D}$  would have to be about  $m=4$  or  $m=5$ , in order to obtain a similar accuracy (determined by the number of potentially moving rocks).

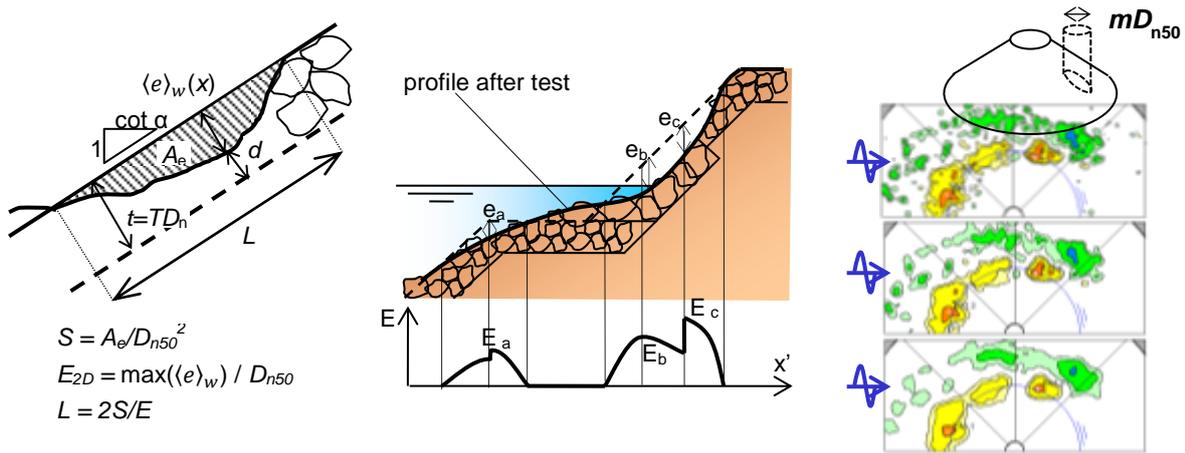


Figure 1. Left: definition of damage numbers  $S$  and  $E_{2D}$  on width-averaged 2D trunk profile. Middle: variation of  $E_{2D}$  on a non-standard cross-section. Right: pattern of  $E_{3D}$  on a roundhead smoothed with different filter size ( $2D_{n50}$ ,  $3D_{n50}$ ,  $5D_{n50}$  – from top to bottom). From Hofland *et al.* (2011, 2014).

For several of the existing damage definitions criteria are advised for certain structure types, and for certain damage levels, such as ‘initial damage’ and ‘failure’. For the dimensionless erosion area  $S$ , these design values are different for each (straight) slope angle, and they are unknown when the slope is non-standard. The design values of the damage depth  $E$  seem less dependent on the structure slope (Hofland *et al.*, 2011), and the design value for  $E$  is related to the notion that failure occurs when locally the entire armour layer is removed (Thompson & Shuttler, 1975). Hofland *et al.* (2011, 2014) suggested design values for  $E_{2D}$ . However, these values are based on a small number of tests and not yet acceptable for current design practice.

A complicated three-dimensional part of many breakwaters is the roundhead. The waves that travel over the side of the roundhead can form water jets that give large loads on the rear side armour units. However, the largest damage is sometimes also reported to occur at the front section (*e.g.* Van Gent & Van der Werf, 2010). Hofland *et al.* (2014) performed detailed stereo-photogrammetry measurements of the average damage pattern to 8 identical roundheads. The results show that the radial length of the erosion hole is smaller at the rear side of the roundhead than at the front section.

Therefore, most damage numbers, except the dimensionless damage depth,  $E$ , do not have a direct link to exposure of the under layer. Therefore it appears that the question whether there is “more damage” to the front or rear side of the breakwater roundhead depends on the definition of damage. Also, the variation of damage (one test compared to the other) was so large that the answer to this question varied between identical repeated tests.

### Physical model tests

To obtain a larger dataset of high-resolution damage scans, two test campaigns were performed and combined with existing datasets. The new test campaigns consisted of:

- 2D tests at Laboratório Nacional de Engenharia Civil (LNEC), Portugal, where several kinds of cumulative damage were measured considering different wave height build-up and constant wave period;
- 3D tests at the Faculty of Engineering of the University of Porto (FEUP), Portugal, where the influence of wider test sections, a roundhead, and short-crested wave attack was investigated, for constant wave steepness.

The analysis of test results from several laboratories allows the comparison of the techniques available and to discuss the results repeatability.

#### 2D Tests at LNEC

A first test series was carried out to obtain damage scans for a standard wave flume case. These tests were done at the LNEC laboratory in a 50 m long, 1.2 m deep wave flume with an operating width of 0.8 m. The flume is equipped with a piston-type wave-maker that combines both irregular wave generation and dynamic absorption of reflected waves through the use of two wave gauges located in front of the paddle. Ten additional resistive-type wave gauges were deployed along the flume and an extra gauge was placed on the model armour layer slope to measure run-up levels (not analyzed presently). The breakwater model was built and operated according to Froude’s similarity law, with a geometrical scale of 1:30, to ensure reduced scale effects (the wave height-based Reynolds number was  $D_{n50}\sqrt{gH_s}/\nu > 3 \times 10^4$ ). The reference breakwater cross section was built on a 14 m long and 0.29 m high foreshore. The breakwater model was 0.8 m ( $18D_{n50}$ ) wide and composed of a main armour slope of 1:2,  $2D_{n50}$  thick, with  $D_{n50} = 0.0445$  m, with a single filter layer and permeable core below, as shown in Figure 2. The armour crest was 0.5 m above the structure toe. A concrete superstructure was also included, with its crest level 1.7 cm below the armour crest. The width of the armour layer crest seaward of the crest element was 19.7 cm.

Series	Test	depth at toe $d$ (m)	$T_p$ (s)	$H_s$ (m)
Series 1	Construct breakwater			
	1	0.30	1.83	0.107
	2	0.30	1.83	0.123
	3	0.30	1.83	0.140
	4	0.34	2.01	0.123
	5	0.34	2.01	0.140
	6	0.34	2.01	0.157
Series 2	Repair breakwater			
	10	0.37	2.20	0.123
	11	0.27	2.20	0.123
	12	0.37	2.20	0.140
	13	0.27	2.20	0.140
	14	0.37	2.20	0.157
	15	0.27	2.20	0.157
	16	0.37	2.20	0.173
	17	0.27	2.20	0.173

Table 1: Test programme of LNEC 2D tests (model values).

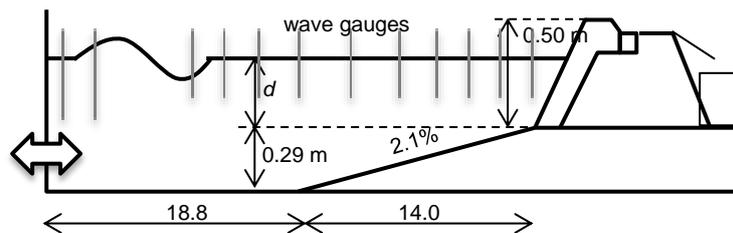


Figure 2. Model setup in LNEC flume.

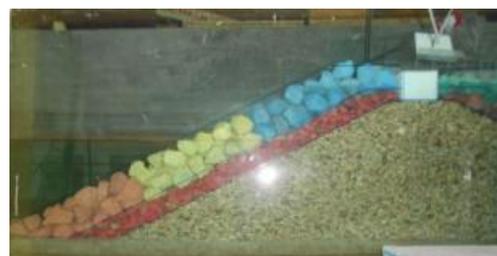


Figure 3. Model setup in LNEC flume.

These two physical model test programmes are done using a rather standard straight slope in order to “calibrate” the design values for  $E$ , by comparing the values to the established design values for  $S$ . Tests were performed to represent two approaches: A) a standard cumulative storm build-up (with

increasing wave heights) with increasing water level (Tests 1-7); B) a standard cumulative storm build-up with a constant water level (tests 1-3 and 4-7); C) a constant wave period (1-3, 4-7 and 10-17); and D) a standard storm build-up, with a constant water level and with rebuilding (tests 4-7). The damage after each test step was measured using a stereo photogrammetry setup (Lemos and Santos, 2013). Irregular wave tests conformed to a JONSWAP spectrum, with a peak enhancement factor of 3.3. The test duration was approximately 1000 waves for all tests. Tests 7 and 16 were repeated twice. The nominal test conditions are presented in Table 1, in which  $H_s$  represents the significant wave height at the toe of the structure and  $T_p$  the peak wave period.

### 3D Tests at FEUP (UPORTO)

This test series was intended to obtain high-resolution data on damage to rubble mound structures considering 3D effects, namely the impact of short-crested waves and damage to a curved roundhead section. In addition, a 4.0 m wide trunk section was added to obtain converged statistical values for the damage numbers.

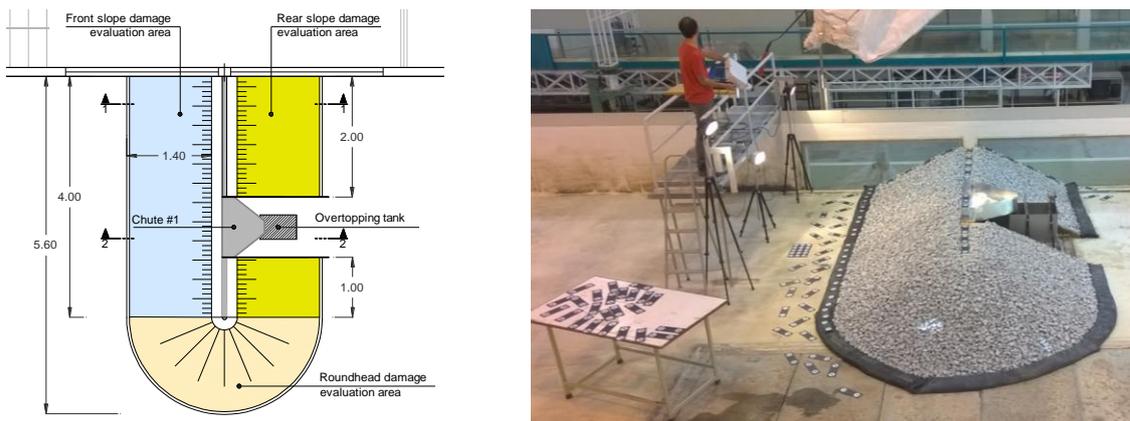


Figure 4. Model setup in FEUP basin. Left: plan view, right: damage measurement.

The tests were carried out in the multidirectional wave basin of the Hydraulics Laboratory of the Hydraulics, Water Resources and Environment Division of FEUP (28.0 m long, 12.0 m wide and 1.2 m deep), which is equipped with a 12.0 m wide segmented wavemaker with active reflection absorption. A reference rubble mound breakwater was reproduced on a geometric scale of 1:35. The breakwater model was 5.6 m long, 3.1 m wide and 0.68 m high, Figure 4. The armour layer was composed of a double layer of rock ( $2D_{n50}$  thickness), with  $D_{n50} = 32$  mm and  $D_{n85}/D_{n15} = 1.25$ , placed on a rubble mound core, with  $D_{n50} = 11.6$  mm. The slope angle was 1:2. The overall crest width was 0.42 m and a 0.10 m wide superstructure was placed at its centre. The armour was supported by a toe materialized by cylindrical sand bags. The breakwater core was enclosed in a mesh to avoid movement of the core materials, while maintaining the desired level of permeability (~40-50%). A 4.0 m wide trunk section was placed perpendicularly against the straight sidewall of the basin, and a semi-circular roundhead was placed at its other side. The slope angle and the armour layer thickness were equal for all trunk and roundhead sections. The armour layer on the rear side was interrupted to allow the installation of a reservoir to collect the overtopped water, allowing the measurement of the overtopping discharges and volumes, which are treated in a companion paper (Silva *et al.*, 2017).

The water free surface elevation was measured using standard resistance-type wave gauges. A set of four aligned gauges was placed in front of the model in order to determine the characteristics of the incoming waves. For the tests with short-crested waves, a setup with six gauges in a CERC6 array was applied (Davis & Regier, 1977). The damage in the structure was measured using a stereo-photography technique (Raaijmakers *et al.* 2011). The structure was photographed with a set of hand-held double-cameras in a drained wave basin. Using a typical number of 50 photo pairs, point clouds of measurements were created before and after each test run. These point clouds were filtered and interpolated on a 1x1 mm grid. The post-processing steps are described in Hofland *et al.* (2011).

The test programme is depicted in Table 2. All tests, except the ones indicated with SLR, were conducted for a water depth of 0.566 m. The tests with the indication SLR were conducted with a 0.023 m larger water level in order to simulate the effect of sea level rise after a few initial storms (the 60% and 80% test runs).

Table 2: Test programme of FEUP 3D tests (an X indicates a performed test). Model values.

Series	Condition (%)	60%	80%	100%	120%
	$H_s$ (cm) $T_p$ (s)	7.1 1.29	9.4 1.58	11.8 1.87	14.2 2.18
1: classic approach		X	X	X	X
2: non-cumulative		-	X	XX	X
3: sea level rise		X	X	X <sup>+SLR</sup>	X <sup>+SLR</sup>
4: short-crested waves		X	X	X	X

The tests were carried out with irregular waves, either long or short crested. Each test consisted of 1000 waves, generated according to a standard Jonswap spectrum (peak enhancement factor of 3.3), using the filtered white noise technique. The same temporal sequence of waves was used in the tests having the same peak wave period. In Test Series 4, short crested waves were used ( $\cos^2 \theta$  spreading function). The local wave steepness,  $s_p = H_s/L_p$ , was set constant for all tests at 0.03. Hence the fictitious offshore wave steepness,  $s_{op} = 2\pi H_s/(gT_p^2)$ , varied between 0.027 and 0.019 for the lowest and highest wave height, respectively. The surf-similarity parameter ranged between  $3.0 < \xi_{op} < 3.6$ . Mainly surging breakers can be expected.

## Results

### Overview

The trunk section of the breakwater model tested in FEUP wave basin was 4.0 m wide in order to get a large section where statistical convergence can be obtained for damage results. The damage after the 100% condition for the three test series and one repetition is depicted in Figure 5.

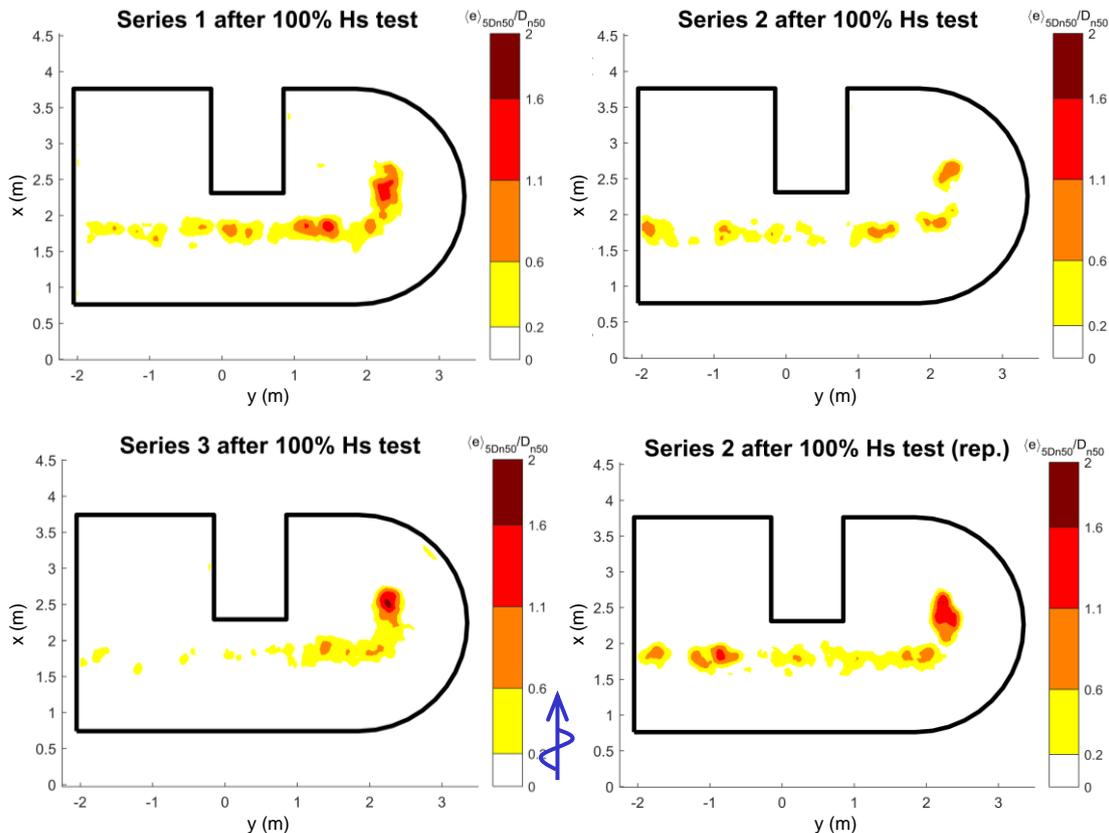


Figure 5. Erosion depth after the 100% test condition for Series 1, 2 (including repetition) and 3.

Some observations can be made based on Figure 5 results. On the seaward trunk slope adjacent to the roundhead, the damage is consistently larger for all test series. Therefore, this area is not taken into account for the analysis of the trunk section. Moreover, the two identical repetitions of the 100% condition of the second test series still give rather different results, so the scatter seems to be large. The results for Test Series 2 were obtained without the damage of two preceding tests (for 60% and 80% test conditions), while Test Series 4 was carried out with short-crested waves and in Test Series 3 the effect of SLR was already included.

It can be observed that the damage hole is deepest around the water line, see Figure 6. The size of the hole increases, as expected, with the wave height. For the last test (120% condition) the damage hole extends up to the crest (vertical black line). It can be noticed that for the wide measurement section that is applied, with a width of more than  $100D_{n50}$ , the confidence band of the results is narrow enough to clearly distinguish the damage stages for the various tests. For the test with the smallest wave height also the damage around the water line is determined with confidence, while at lower elevations (where rocks are both deposited and removed) the confidence band is wider.

The influence of a limited sea level rise (SLR) is demonstrated in the left panel of Figure 7 where Series 1 and 3 are compared. The first two tests (60% and 80%) are repetition tests with the same conditions. After the first two tests with a constant water level, the 100% and 120% tests are performed for a 2.3 cm higher water level in Series 3 (0.80 m SLR at prototype scale). The water level difference is relatively small (about 20% of the design  $H_s$ ). Still a rather large difference might be observed in the results of Test Series 1 and 3: less damage in the tests carried out for the higher water level, while for the same water level very comparable damage levels were obtained.

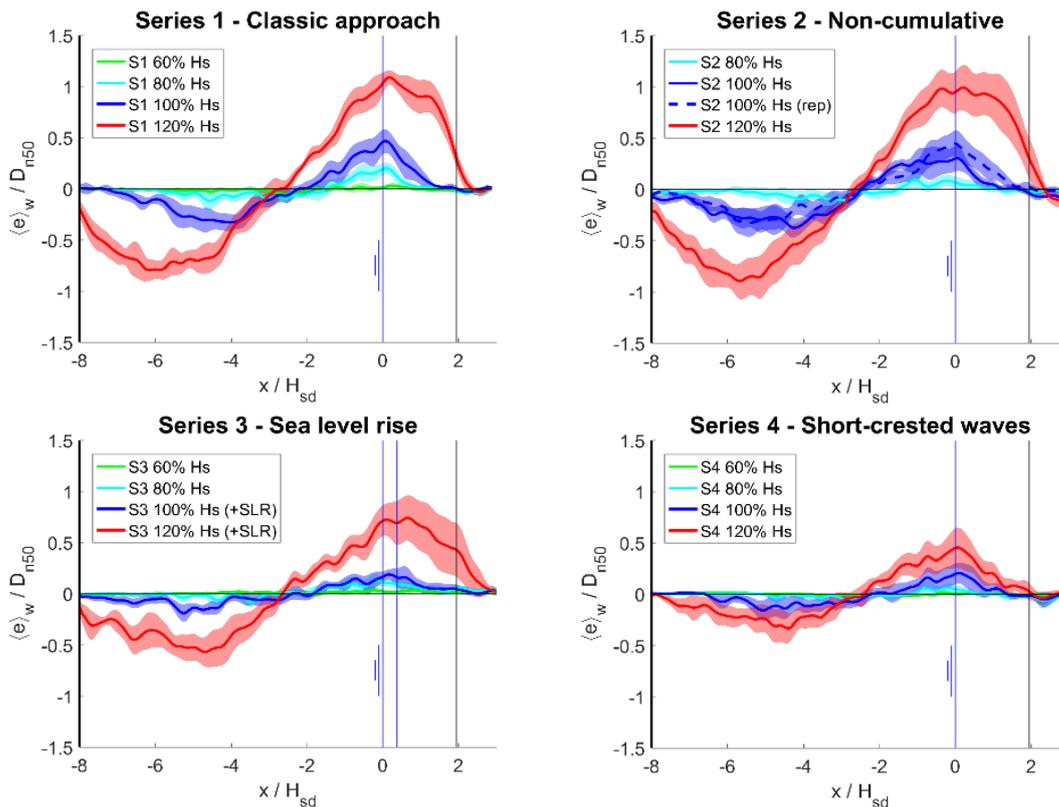


Figure 6. Width-averaged erosion profiles for the four test series with confidence band (shaded areas). Here  $x$  is the horizontal cross-shore coordinate, with its origin at the waterline.

The comparison of the cumulative damage case (no rebuilding after each test run) with rebuilding (a newly constructed slope for each test run) is shown in the right panel of Figure 7. As the damage in the model is not repaired before each test run in Test Series 1, the new inflicted damage is added to an already damaged slope. In this right panel only the additional damage that occurred during the specific test run is shown. There are clearly visible differences between the two test series. However, the test run that was repeated in Test Series 2 shows a much larger damage level.

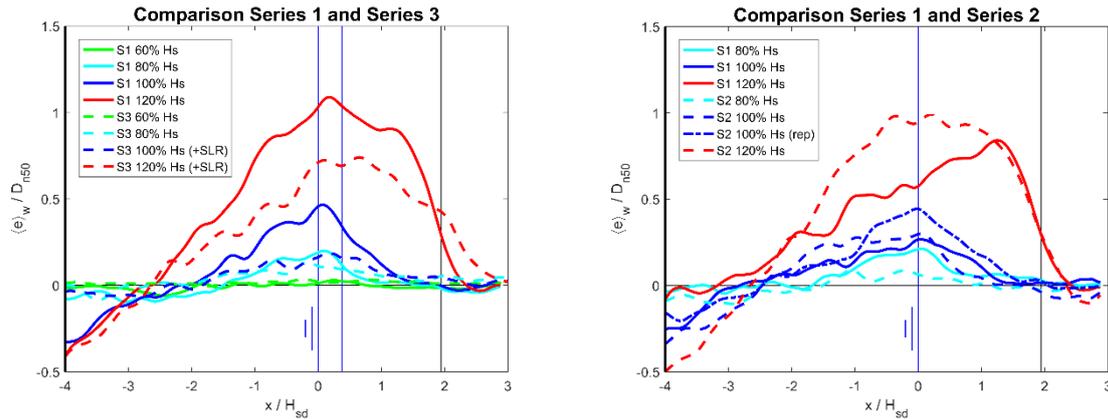


Figure 7. Left: width-averaged erosion profiles for Series 1 (constant WL) and 3 (SLR). Right: width-averaged erosion profiles for Series 1 (cumulative) and 2 (incremental).

The damage to the structure for long-crested and short-crested waves is compared in Figure 8. In the tests with short-crested waves, extra damage occurred in the vicinity of the sidewall, due to the energy concentration resulting from the imperfect reflection on the sidewall of waves travelling with a direction not perpendicular to the breakwater. However, in the remainder part of the trunk, the wave conditions were similar and the results obtained can be considered valid. It can be concluded that the damage to the trunk is lower for the tests with short-crested waves. Also, the size of the damage hole created at the roundhead is smaller. The damage does seem to extend further around the roundhead.

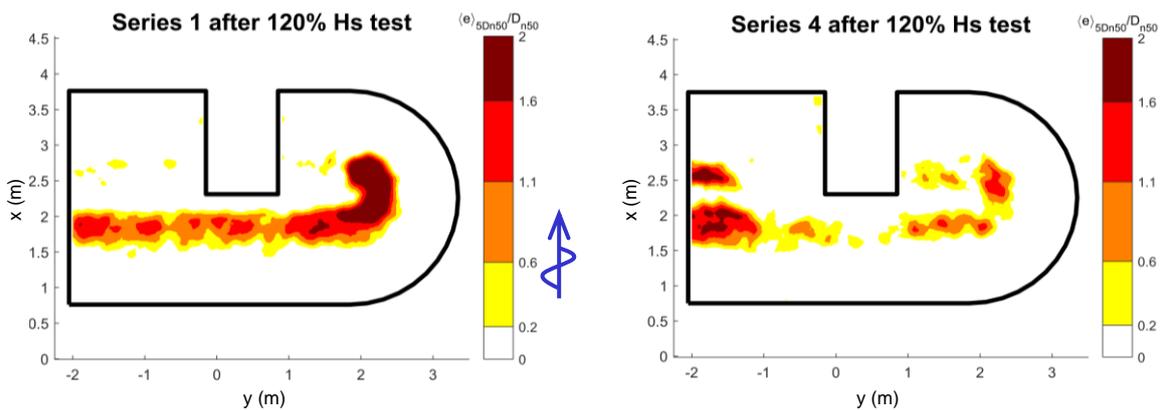


Figure 8. Erosion depth on breakwater trunk and roundhead after Test 4. Left: Series 1 (long crested waves). Right: Series 4 (short crested waves).

### Relationship between E and S

In this section the relationship between the damage parameters  $S$  and  $E_{2D}$  is examined for a number of test series. The design values of the  $S$  parameter for the various stages of damage and a 1:2 slope are given in the Rock Manual (CIRIA *et al.*, 2007). Hofland *et al.* (2011) presented, as a first estimate, possible design values for  $E_{2D}$  summarized in Table 3. Using the present results, and comparing the values of  $S$  and  $E$  for a straight slope, the design values for  $E$  can be calibrated.

The results for FEUP and LNEC tests are depicted in Figure 9. The LNEC results, which were based on depth-limited conditions and varying wave steepness collapse with the deep water FEUP results. The scatter in the LNEC results seems somewhat larger. This might be due to the smaller width of the LNEC test section. Note that the parameter  $E_{2D}$  can be used for any slope angle, or combination of slope angles (*e.g.* berms). It can be seen that those tests give comparable values for  $E_{2D}$  as given in Hofland *et al.* (2011) for the initial and intermediate damage levels. Only the  $E_{2D}$  value for “failure” does not seem to match the definition of 2011. As the background of this definition is most objective, *i.e.* the exposure of the filter layer, this can be checked. Indeed for Test run 4 some locations were visible on the trunk where the erosion depth is  $2D_{n50}$ . Hence it seems that the design value for the mean erosion depth  $E_{2D}$  should be much stricter, around 0.8. It seems better to use this definition for

(possible) failure complementary to the direct observation of under layer exposure, as it can be seen that there are locations of  $20D_{n50}$  wide (the standard order of magnitude of the width of a test section) where no exposure of the filter layer occurs. For the situation where a considerable amount of damage occurs to the crest, the allowed value of  $S$  becomes unclear, as it was not intended for this case.

Table 3: Design values for damage.

Description:	$S$ (cota = 2)	$E_{2D}$ (2011)	$E_{2D}$ (present)
initial	2	0.2 to 0.3	0.3
intermediate	4 - 6	0.5 to 0.6	0.5
failure	8	1.5 to 1.6	0.8

In addition to the  $E_{2D}$  values,  $E_{3D,5}$  values for the trunk section are presented for all the test runs, obtained considering a circular spatial moving average of  $5D_{n50}$  diameter. These values can also be used as design limits in 3D situations. It can be seen that these values are about two times larger.

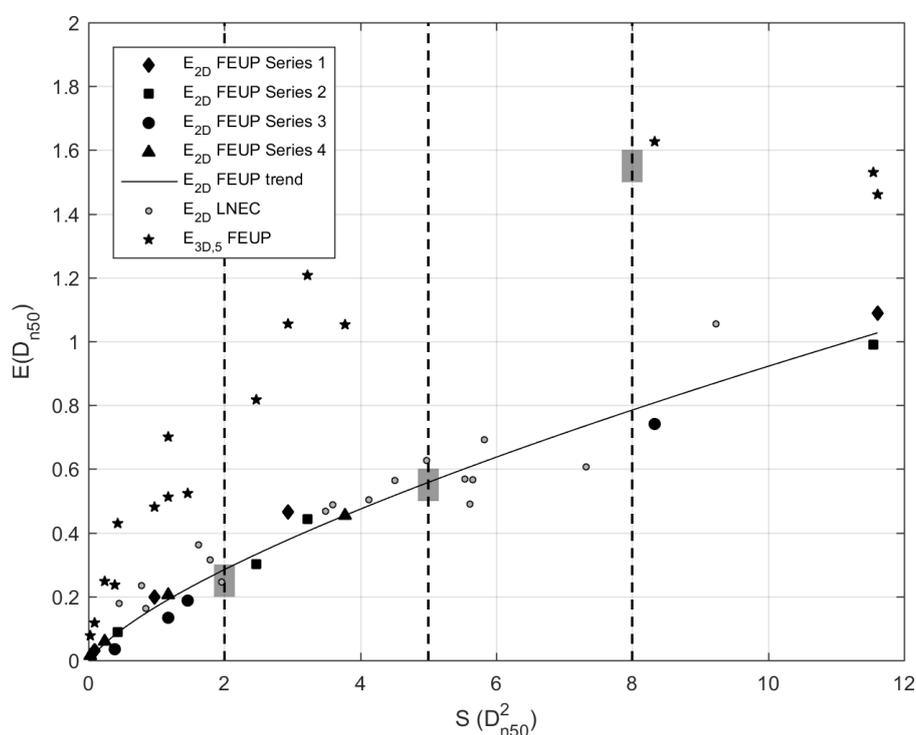


Figure 9.  $E_{2D}$  and  $E_{3D,5}$  versus  $S$ . Dashed lines and grey areas: design values for  $S$  and previous design values for  $E_{2D}$  (Hofland *et al.*, 2011), respectively.

Broderick stated that  $S = 2$  (equivalent to  $E_{2D} \approx 0.3$ ) was applied as start of damage, as that was the lowest discernible damage that could be measured. It can be seen that much lower damage quantities (i.e.  $S \approx 0.5$  or  $E_{2D} \approx 0.1$ ) can be measured using the presently used high-resolution techniques and wide test section.

## Conclusions

High-resolution damage data for a rubble mound breakwater, resulting from 3D physical model tests at FEUP, and 2D tests at LNEC, was presented and analysed. A sufficiently wide breakwater cross section was then considered in the 3D tests to obtain statistical convergence for the damage results. Erosion patterns and average erosion profiles with their 90% confidence bands were presented.

It was observed that the damage hole was deeper around the water line and that its size increased, as expected, with the significant wave height, extending up to the crest for the 120% wave condition. For the higher water level (with SLR) and equal wave height, less damage was observed in the structure. Moreover, clear differences between “cumulative damage” and “rebuilding” test series were observed. Nevertheless, it is important to mention that significant scatter was also observed in the results from

tests carried out under identical conditions. It was also concluded that the damage to the trunk was lower for short-crested waves.

The design values for  $E_{2D}$  proposed by Hofland *et al.* (2011) were partly in line with the experimental results presented and were partly adjusted. The relation between  $S$  and  $E_{2D}$  does not hold true for non-standard cases, namely: short-crested waves, strongly varying water levels, curved sections like roundheads, etc. It seems better to use a local parameter like  $E_{2D}$  when testing such a structure. The design values for  $E_{3D,5}$  appear to be roughly twice the values as those for  $E_{2D}$ . It has to be verified whether these values can be used on configurations like roundheads. The reliability of any damage number for a test on the stability of a trunk is rather limited. This can be improved by either increasing the relative width of the test section (width /  $D_{n50}$ ), or repeating the test.

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