

# Stability of overtopped and submerged rubble mound breakwaters

Arthur de Graauw (Director Port Revel, ARTELIA)

## Abstract

The present analysis can be seen as a follow-up of work done previously by Foster (1977), Ahrens (1987), Vidal (1995) and Burcharth (2003). It aims at finding some simple relation between the governing parameters (water depth, structure height, stone size) and the equilibrium position of the crest of rubble mound breakwaters subject to long term wave attack in breaking wave conditions.

Some of van der Meer's tests (1992) and Ota's test reported by Kobayashi (2013) are also taken into account in the present analysis.

A few scale model tests were performed confirming the general trend.

It is concluded that undersized emerging rubble mound breakwaters reduce to submerged breakwaters and that the crest can be located as follows:

$R_c/h = 3.45 D_n/h - 1$  valid for  $1 < h/\Delta D_n < 30$  and for  $\Delta = 1.6$ .

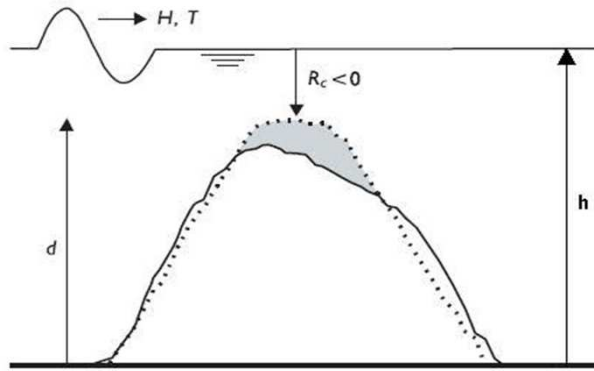
For a given stone size, submerged breakwaters stabilize to the predicted crest level after long term wave attack in breaking wave conditions.

---

The present analysis of long term stability concentrates on the worst possible wave conditions, considering that they will *eventually* occur in the long term. This means that *we consider only cases with waves breaking near the submerged structure*. Hence, the local wave climate must include waves large enough to break on the water depth in front of the submerged structure and breakwaters in very sheltered areas are not considered in this analysis. Similarly, breakwaters located in water depths larger than say 20 m are not likely to be subjected to breaking waves in the Mediterranean area and are therefore not considered in this study.

Let's first consider the processes involved. When a wave is breaking on a submerged structure, some of its energy is reflected back, some of its energy is found in the surf zone between the breaker line and the shore line, but a large part of its energy is "lost". This "lost" energy is converted into turbulence (heat) and into reshaping of the breakwater (hereafter called "BW"). If the BW was made of sand like the neighbouring sea bed, the obstacle would be eradicated in order to come back to the initial situation without any obstacle, as "castles made of sand slip into the sea, eventually" (J. Hendrix). But the BW being made of blocks of stone, the crest of the submerged structure is lowered until waves do not erode the crest anymore. The crest of this equilibrium profile results from a limited reduction of the crest level which obviously depends on the stone size.

**Definitions** (from the Rock Manual, 2007):



with:

$H_s$ : significant wave height in front of breakwater (BW) (m)

$h$ : water depth in front of BW (m)

$R_c$ : crest elevation of BW above water level ( $R_c < 0$  if under water) (m)

$d$ : height of BW above sea-bed (m)

$D_n$ : nominal diameter of rock (m) =  $(M50/\rho)^{1/3}$

$\rho$ : specific mass of rock ( $\text{kg}/\text{m}^3$ )

$M50$ : median mass of rock (kg)

$\Delta$ : relative buoyant density of rock =  $S_r - 1$  = around 1.58 for granite in sea water (-)

$S_r$ : specific mass of rock/specific mass of water = 2.65/1.025 for granite in sea water (-)

Different types of breakwater are usually distinguished (see Rock Manual, 2007):

>> **Emerging BW**, they are stable if:

a) they are not overtopped, i.e. they are high enough, say  $d > 2 h$  if waves are breaking at their toe,

b) they have a stable front armour layer, i.e. the stone size is large enough, say  $D_n > 0.2 h$  if waves are breaking at their toe.

If an emerging BW is not stable, it will be eroded and eventually become a submerged BW.

>> **Submerged BW**, have their crest at or below Still Water Level (SWL) and have a narrow crest (say 3 to 5  $D_n$ ); they are stable if made of large stones (Burcharth's rule:  $D_n > 0.3 d$ ) and they are eroded by offshore movement of front slope stones combined with onshore movement of crest stones that fall behind the BW, the result being a lowering of the crest.

If they have a wide crest (say 50  $D_n$  and more) the eroded crest stones remain on the crest, the result being a rise of the crest similarly to the reconstruction of an S-shaped beach.

>> **Reef BW**, are low crested BW that do not have the traditional multi-layer structure; according to Ahrens (1987) "this type of breakwater is little more than a homogeneous pile of stones with individual stone weights similar to those ordinarily used in the armor and first underlayer of conventional breakwaters."

>> **Berm BW**, they are voluntarily unstable and reshaping into an S-shaped profile; the front slope is locally getting milder, rotating around a pivot point located under water at a distance of:  $0.2 h + 0.5 D_n$  below SWL. The stone size is smaller than for the stable types of BW, typically  $D_n = 0.04 h$  to  $0.08 h$ . Hence, the pivot point will be located at  $0.22$  to  $0.24 h$  below SWL.

>> **Near-bed structures** are used for sea bed protection works and their height  $d$  is small compared to the water depth  $h$ ; they are stable if made of medium size stones (say  $D_n = 0.05 h$ , if waves are breaking over them).

The parameters  $R_c/D_n$  and  $H_s/\Delta D_n$  (the latter also called stability number  $N_s$ ) are widely accepted as representative parameters on which breakwater stability under wave attack depends. This includes submerged breakwaters ( $R_c < 0$ ).

It is widely accepted that random waves are breaking when their height  $H_s$  is around  $0.6 h$  (NB: this is valid for mild offshore bed slopes, up to say 1:20, but this breaker index  $H_s/h$  may increase to say 0.8 for steeper bed slopes and/or longer waves). See Goda (2010) for a detailed overview on this complex subject.

Anyway, this means that the stability number above can be written as  $h/\Delta D_n$ .

Hence, we will try to find some relationships between  $R_c/D_n$  (or  $R_c/h$ ) and  $h/\Delta D_n$ .

Let's first consider **narrow crested BW**.

Although several hundreds of scale model tests were carried out on submerged and slightly emerging breakwaters, only (very) few were pushed until waves were breaking at the toe of the structure:

Vidal's data (1995) is for non breaking waves ( $H_s/h =$  around 0.3) and therefore of limited interest for this analysis; the same holds for van der Meer's tests (1988);

From Ahrens tests (1987), only the 4 tests with highest waves are taken over here ( $H_s/h = 0.63$ );

Burcharth's rule (2003) is for breaking waves and therefore very useful for this analysis;

An interesting study on model and prototype of the Rosslyn Bay BW is given by Foster (1977).

The range of validity of model tests is usually quite limited:  $R_c/D_n > -4.3$  (Ahrens);  $R_c/D_n > -3$  (Burcharth). The real case reaches  $R_c/D_n = -5.2$  (Foster).

Available experimental data is shown in fig. 1 below.

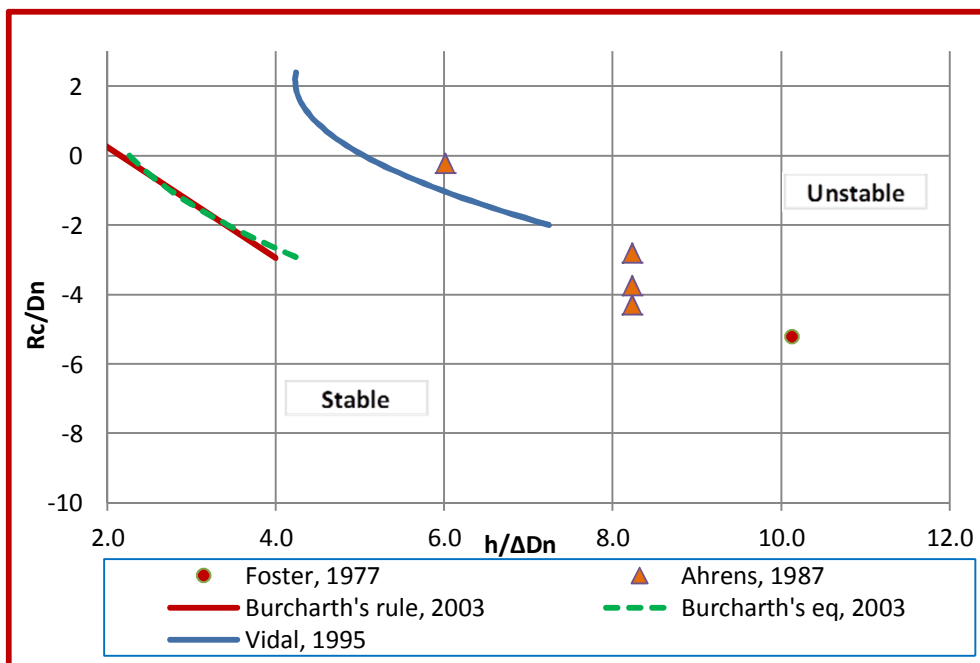


Figure 1. Stability of submerged breakwaters with breaking waves.

Burcharth's rule for stable submerged BW is (Burcharth, 2003):

$$D_n = 0.29 d \quad \text{valid for } -3 < R_c/D_n < 2 \quad (1)$$

with  $d = h + R_c$ , it can be written as:

$$R_c/h = +3.45 D_n/h - 1 \quad (2)$$

This rule assumes  $H_s/h = 0.6$  and  $\Delta = 1.6$ .

It can be noted that  $R_c/h = -1$  for very large  $h$  or very small  $D_n$ , Burcharth's rule is shown on fig. 1 as a straight line in upper left side of the figure.

Burcharth deduced his rule above from an analysis of his equation defining the worst conditions for stability:

$$H_s/\Delta D_n = 0.06(R_c/D_n)^2 - 0.23(R_c/D_n) + 1.36 \quad (3)$$

This "Burcharth's equation" is shown in fig. 1 as a curved dotted line. It is obviously very close to his "Burcharth's rule" for  $R_c/D_n > -3$ .

Let's now turn to **wide crested BW**.

An interesting comparison is provided by van der Meer (1992) in his fig. 12. A low crested BW ( $d = 850$  mm high in  $h = 800$  mm water depth, with a crest width of around 1200 mm consisting of stones with  $D_n = 11$  mm, hence a crest width of over  $100 D_n$ ) was submitted to waves with  $H_s/h = 0.24$  (pretty far from breaking wave conditions). The initial crest level was  $R_c = +4.5 D_n$  (that is  $R_c/h = +0.06$ ) and the final crest level rose to nearly  $+10 D_n$  during the test, generating an S-shaped profile with a slope around 1:3 to 1:5 near SWL.

He superimposed the final profiles of a non overtopped berm profile and this low crested BW, and it appeared that "a large part of the profile is the same". This shows that, at least for non breaking waves, *a rubble mound can behave in a similar way to a berm or a gravel beach* .

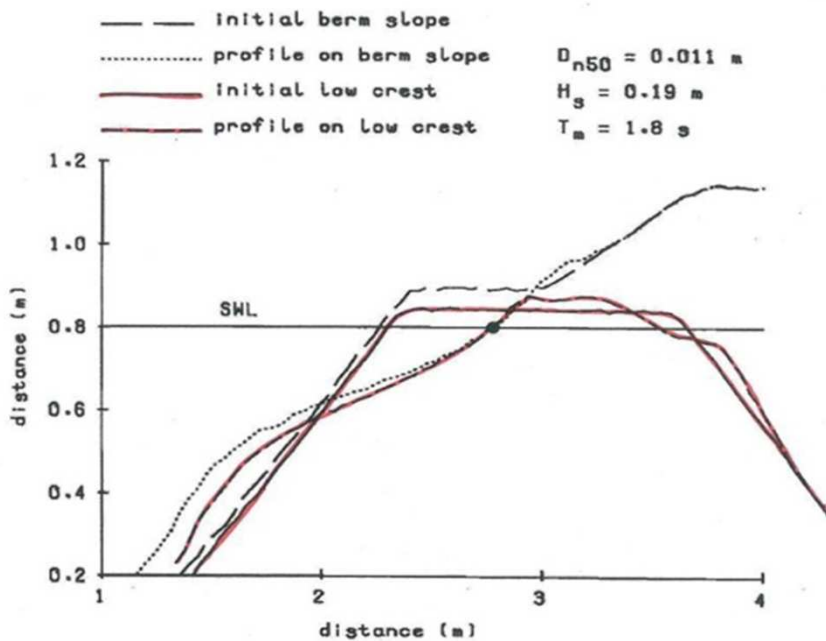


Figure 2. van der Meer's Figure 12 (1992)

A similar model test performed by Ota is reported by Kobayashi (2013). A reef BW ( $d = 167$  mm high on  $h = 222$  mm water depth, with a crest width of 1100 mm consisting of stones with  $D_n = 25.2$  mm, hence a crest width of  $44 D_n$ ) was submitted to around 36 000 waves with  $H_s/h = 0.52$  (which may be considered as breaking wave conditions). The initial crest level was  $R_c/D_n = -2.18$  (that is  $R_c/h = -0.25$ ) and the final crest level was around SWL: like in the van der Meer test above, the crest rose during the test. Stones were taken from the seaward side of the crest towards the landward side of the crest and heaped up there without falling down on the back slope of the BW. An S-shaped beach profile was building up similar to that of gravel or sand beaches. It is also worth noting that the rising of the crest was linear in time and still ongoing after the very long testing time, unlike narrow crested BW that are known for their logarithmic damage progression, i.e. most damage occurs in the early stages of the storm.

The large width of the crest might give an explanation as waves seem not to have been able to move the stones as far as the rear slope where they would have fallen down without rising the crest level.

Waves cannot transport stones very far landwards on the crest of the submerged BW, hence a narrow BW will initially be lowered and flattened, until the crest reaches a certain width that does not allow stones to proceed further landwards under wave action. Hence, stones start to heap up and may reach SWL again like in van der Meer's and Ota's tests.

The example below illustrates this. All figures 3a, 3b and 3c are undistorted and at the same scale. The initial BW height of 2.5 m above SWL is reduced to 0.5 m above SWL. As no material is supposed to be lost (i.e. the cross-section of the BW remains around  $120 \text{ m}^2$ ), the length of the submerged reshaped BW on the sea bed increased from 27.5 m up to nearly 33 m during the reshaping process (Fig. 3b: in this example, the choice of a front slope of 1:5 is arbitrary).

The question that remains to be answered at this stage is: will the BW further lower until it becomes submerged and further flatten out? If so, to which level under water (Fig. 3c with an arbitrary 1:10 front slope)?

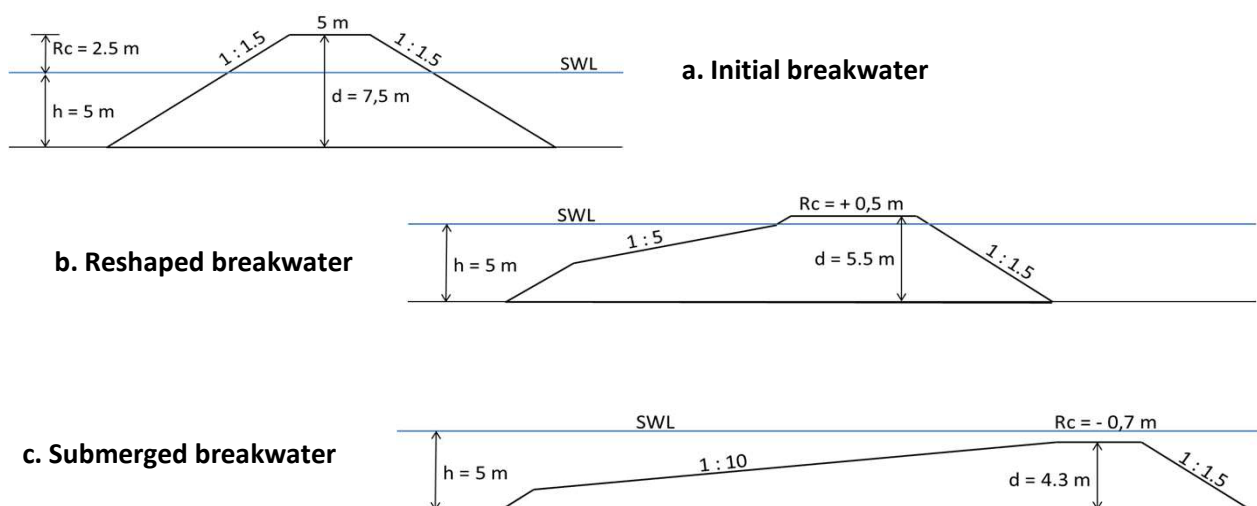


Figure 3. Destruction and reshaping of a breakwater.

It may be expected that the highest waves will break near the toe of the BW. Probably plunging heavily in that area. With the milder front slopes of Fig. 3b and 3c, the broken wave may further propagate as a translation wave at a speed around  $\sqrt{gh}$ , in the order of say 3 to 6 m/s (for resp.  $h = 1$  m and 3.5 m).

If we consider a flow speed of 3 m/s on a water depth  $h = 1$  m, a stone size of  $D_n = 0.20$  m would be stable; for a flow speed of 6 m/s on a water depth  $h = 3.5$  m, the stable stone size would increase to  $D_n = 0.60$  m.

Some unpublished **scale model tests** were performed in a wave flume at SOGREAH's Laboratory in April 1993. The flume was 40 m long, 1 m wide and included 5 glass wall sections for observation. The flume was equipped with a wave generator capable of producing either regular or random waves. The wave maker was controlled by an integrated system for wave generation, data acquisition and analysis. An active wave absorption technology was used for a real-time absorption of the reflected waves at the paddle, allowing control of the incident wave field over the course of an experiment.

The seabed was represented as a non-erodible, concrete surface and was built with a 1.5% slope (1:66). A parabolic section with a mean slope of approximately 6% connected the flume bottom at the wave maker to the sea bed. The parabolic section was built with a mean slope lower than 10% in order to prevent spurious wave reflections.

The submerged rubble mound was given a very simple trapezoidal shape with 1:1.5 slopes, 40 mm high, and 100 mm long on the crest. The water depth  $h$  was 250 mm for all tests, except the last one when  $h$  was reduced to 200 mm. The wave height was increased step by step during the test until full wave breaking occurred and no further increase of significant wave height could be obtained. The wave period was set at  $T_p = 1.75$  s for all tests, except the last one when  $T_p$  was increased to 2.5 s. Wave breaking was of the "spilling" type for all tests. The rubble mound was built with one single type of stone defined by its nominal  $D_n$ . Three different stone sizes were tested:

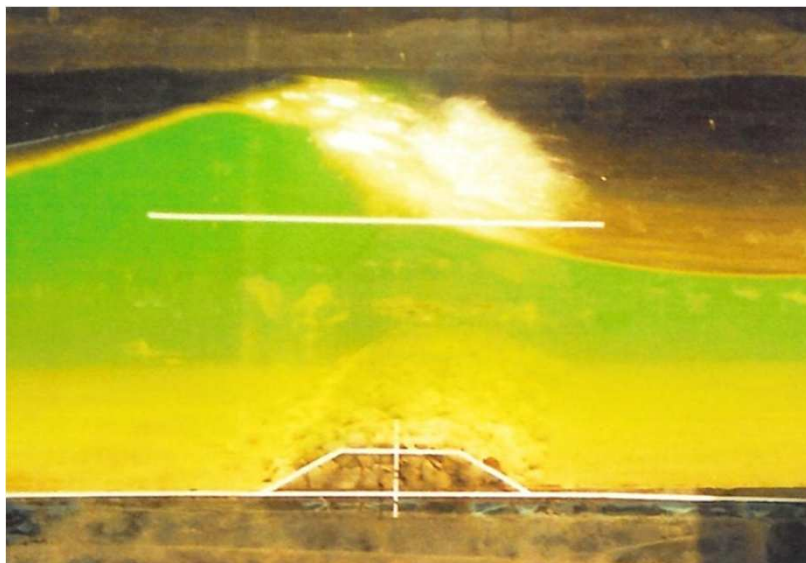
>>  $D = 12 - 18$  mm with average mass = 4.77 g and  $D_n = 12.2$  mm (Test 1)

>>  $D = 8 - 12$  mm with average mass = 1.46 g and  $D_n = 8.2$  mm (Test 2)

>>  $D = 4 - 8$  mm with average mass = 0.34 g and  $D_n = 5.0$  mm (Tests 3, 3a, 3b)

For all stones  $\Delta$  was 1.65.

The trapezoidal mound structure was rebuilt after Test 1 and Test 2, but no repair was done during Test 3.



*Figure 4. Model test on submerged rubble mound with breaking wave attack*

Waves were increased in steps, starting from 45 mm. For  $T_p = 1.75$  s, the highest waves that could be obtained on this water depth were measured as follows:  $H_s = 133$  mm, with  $H_1/10 = 157$  mm and  $H_{max} = 178$  mm (note  $H_s/h = 0.53$  and  $H_{max}/H_s = 1.33$ ). For  $T_p = 2.5$  s, the measured maximum was  $H_s = 150$  mm, and for  $h = 200$  mm, the measured maximum was  $H_s = 138$  mm ( $H_s/h = 0.69$ ).

Each step of wave height was maintained for about 4 minutes (around 150 waves) until reshaping of the structure (if any) would stabilise. The last step of each test was maintained for about 45 minutes (around 1500 waves).

Test 1 with  $D_n = 12.2$  mm: the structure was reshaped into a rounded mound that was globally stable with waves  $H_s = 133$  mm.

Test 2 with  $D_n = 8.2$  mm: the structure was reshaped into a rounded mound that was dynamically stable with waves  $H_s = 133$  mm, i.e. some stones were continuously moving back and forth but the overall mound shape was maintained.

Test 3 with  $D_n = 5.0$  mm: the structure was reshaped into an asymmetric rounded mound that was dynamically stable with waves  $H_s = 133$  mm, i.e. the front slope was steeper than the rear slope so that the whole mound was moving slightly backwards.

Test 3a with  $D_n = 5.0$  mm and  $T_p = 2.5$  s: the same processes were going on with  $H_s = 150$  mm as the structure had now lost 5 to 8 mm of its initial crest height and the rear toe had moved 50 mm backwards before stabilising.

Test 3b with  $D_n = 5.0$  mm and  $T_p = 2.5$  s and  $h = 200$  mm: the same processes were going on with  $H_s = 138$  mm as the structure had now lost 10 to 15 mm of its initial crest height and the rear toe had moved 90 mm backwards before stabilising.

The test results are summarised in the table below.

N°	h (mm)	$D_n$ (mm)	$H_s$ (mm)	$T_p$ (s)	$R_c$ (mm)	$R_c/D_n$	$h/\Delta D_n$
1	250	12.2	133	1.75	-210	-17	12.4
2	250	8.2	133	1.75	-210	-26	18.5
3	250	5.0	133	1.75	-210	-42	30.3
3a	250	5.0	150	2.50	-215	-43	30.3
3b	200	5.0	138	2.50	-170	-34	24.2

Results of Tests 3a and 3b provide two new experimental points which are shown in the figure below.

These tests are of course very limited and modest, but they yield most important results enabling a much wider perspective on the processes involved.

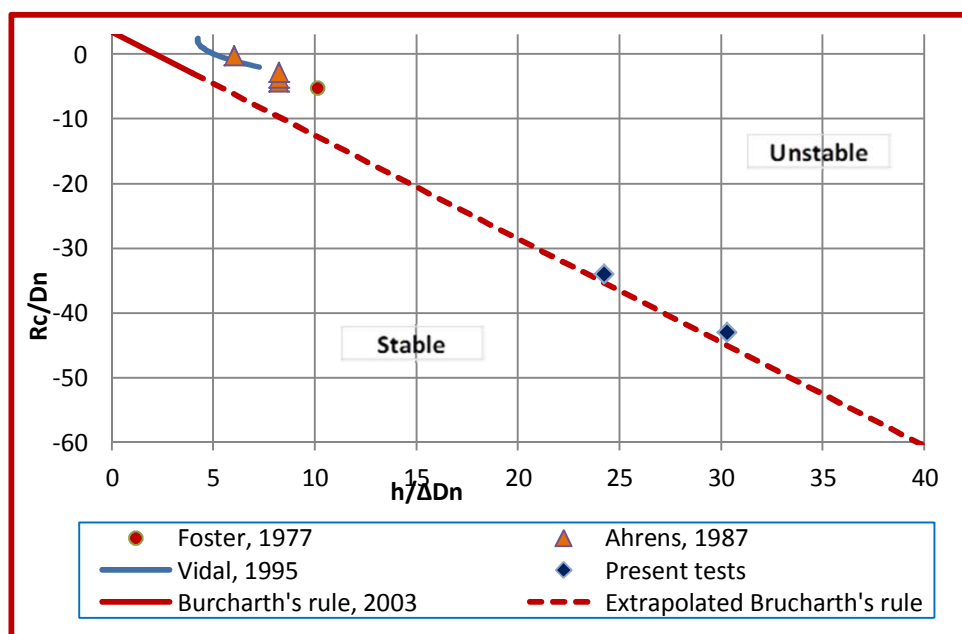


Figure 5. Stability of submerged breakwaters with breaking waves.

Figure 5 is no more than an out-zooming of fig. 1. Existing data shown in fig. 1 is now gathered into the upper left corner of the graph.

The new data from the model tests very well fits Burcharth's rule (2).

This is the simple relation which was sought at the start of this study. It is shown in fig. 6 below for a few values of Dn.

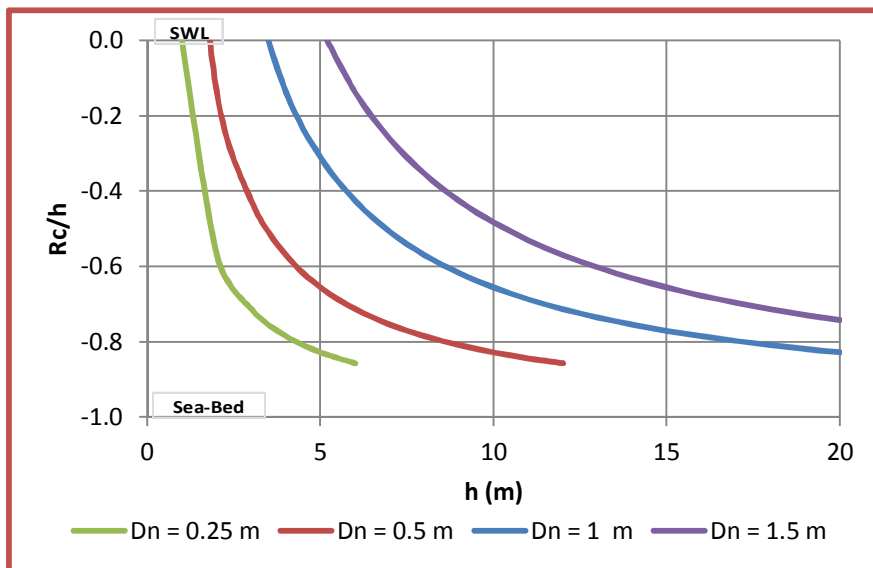


Figure 6. Stable submerged breakwater with breaking waves.

Note that this graph shows a typical "1/h effect" as we have set out a function of 1/h against h. It can nevertheless be seen that for e.g. a water depth of h = 10 m, stones with Dn = 1.5 m will be stable up to 50% of the water depth, Dn = 1 m will be stable up to 35% of the water depth, and Dn = 0.50 m up to nearly 20% of the water depth.

#### Conclusion:

It is concluded that undersized emerging rubble mound breakwaters reduce to submerged breakwaters and that the crest can be located as follows:

$$Rc/h = 3.45 Dn/h - 1 \quad \text{valid for } \Delta = 1.6 \text{ and for } 1 < h/\Delta Dn < 30 \text{ (that is } -0.86 < Rc/h < 0)$$

For a given stone size, submerged breakwaters stabilize to the predicted crest level after long term wave attack in breaking wave conditions. In other words, and considering eq (1) again:

*in breaking wave conditions,*

*a stable submerged breakwater cannot consist of more than 3 layers of stone .*

This result is obviously very useful for the design of the construction phases of breakwaters, when the core of the structure may be exposed to storms inducing waves breaking on the structure.

It is also useful to determine the long term equilibrium level of the crest of undersized breakwaters and near-bed rubble mounds protecting pipes.



**References:**

- Ahrens, J.** (1987) "Characteristics of reef breakwaters", Technical Report CERC 87-17, Vicksburg, MS.
- Foster, D.** (1977) "Model simulation of damage to Rosslyn Bay breakwater during cyclone "David"", 6th Australian Hydraulics and Fluid Mechanics Conference, Adelaide.
- Goda, Y.** (2010) "Reanalysis of regular and random breaking wave statistics", Coastal Engineering Journal, Vol. 52, No. 1, pp 71–106, Japan.
- Kramer, M. & Burcharth, H.** (2003) "Stability of low-crested breakwaters in shallow water short crested waves", ASCE, 4th Int. Coastal Structures Conf., Portland, OR.
- Kobayashi, N.** (2013) "Deformation of reef breakwaters and wave transmission", ASCE J. Waterway, Port, Coastal, Ocean Eng., 139, 336-340.
- Rock Manual** (2007) "The use of rock in hydraulic engineering" (2nd edition), CIRIA, CUR, CETMEF, Published by C683, CIRIA, London.
- van der Meer, J.** (1992) "Stability of the seaward slope of berm breakwaters", Coastal Engineering, 16, pp 205-234.
- Vidal, C., Losada, M., and Mansard, E.** (1995) "Stability of Low-Crested Rubble-Mound Breakwater Heads", ASCE J. Waterway, Port, Coastal, Ocean Eng., 121(2), 114–122.