Stability of overtopped and submerged rubble mound breakwaters

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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.

Van der Meer's tests (1992), the near-bed structures mentioned in the Rock Manual (2007) 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:

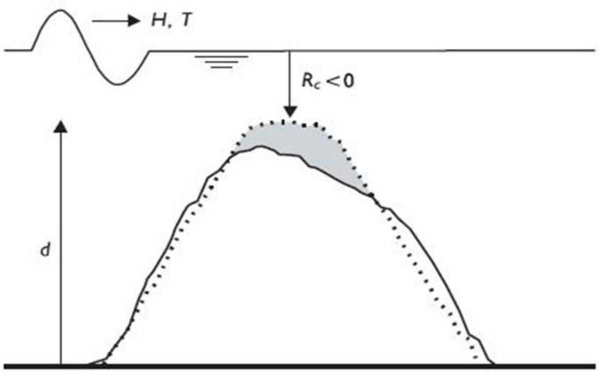
$Rc/h = 6\ Dn/h - 1$ valid for $4 < h/\Delta Dn < 30$ and for $\Delta = 1.6$.

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 Rock Manual, 2007):



with:
Hs: significant wave height in front of breakwater (BW) (m)
h: water depth in front of BW (m)
Rc: crest elevation of BW above water level (Rc < 0 if under water) (m)
d: height of BW above sea-bed (m)
Dn: nominal diameter of rock (m) = (M50/ρ)1/3
ρ: specific mass of rock (kg/m3)
M50: median mass of rock blocks (kg)
Δ: relative buoyant density of rock = Sr - 1 = around 1.58 for granite in sea water (-)
Sr: 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 Dn > 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 Dn); they are stable if made of large blocks (Burcharth's rule: Dn > 0.3 d) and they are eroded by offshore movement of front slope blocks combined with onshore movement of crest blocks that fall behind the BW, the result being a lowering of the crest.
If they have a wide crest (say 50 Dn and more) the eroded crest blocks 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; acoording 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 Dn below SWL. The stone size is smaller than for the stable types of BW, typically Dn = 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 heigth d is small compared to the water depth h; they are stable if made of medium size blocks (say Dn = 0.05 h, if waves are breaking over them).

The parameters Rc/Dn and Hs/ΔDn (the latter also called stability number Ns) are widely accepted as representative for breakwater stability under wave attack. This includes submerged breakwaters (Rc < 0).
It is widely accepted that random waves are breaking when their height Hs is around 0.6 h (NB: this is valid for mild offshore bed slopes, up to say 1:20, but this breaker index Hs/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/ΔDn.
Hence, we will try to find some relationships between Rc/Dn (or Rc/h) and h/ΔDn.

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 (Hs/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 (Hs/h = 0.63);
Burcharth's rule (2003) is for breaking waves and therefore very useful for this analysis;
One very interesting case is given by Foster (1977) for model and prototype.
The range of validity of model tests is usually quite limited: Rc/Dn > -4.3 (Ahrens); Rc/Dn > -3 (Burcharth). The real cases reach Rc/Dn = -5.2 (Rosslyn).
Available experimental data is shown in fig. 1 below.

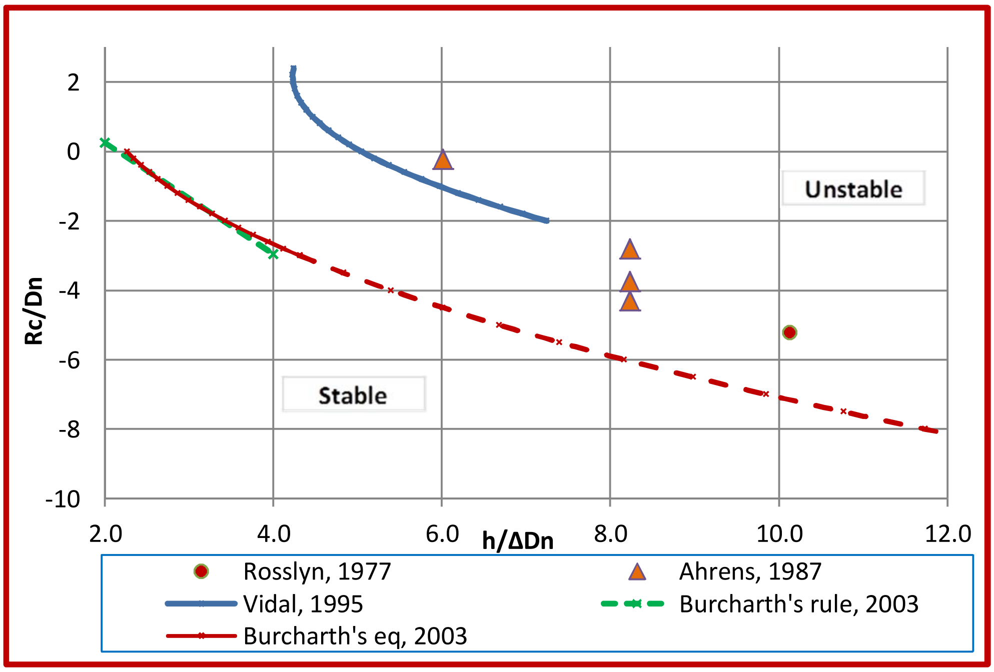


Figure 1. Stability of submerged breakwaters with breaking waves.

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

$Dn = 0.29\ d$ with $d = h + Rc$ (1)

It can be written as:

$Rc/h = +3.45\ Dn/h - 1$ (2)

This rule assumes $Hs/h = 0.6$ and $\Delta = 1.6$. For submerged BW, it is valid for $2 < h/\Delta Dn < 4$, that is $Rc/h > -0.46$.
It can be noted that $Rc/h = -1$ for very large h or very small Dn. Burcharth's rule is shown on fig. 1 as a straight dotted 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:

$Hs/\Delta Dn = 0.06(Rc/Dn)2 - 0.23\ (Rc/Dn) + 1.36$

This "Burcharth's equation" is also shown in fig. 1. It is obviously very close to his "Burcharth's rule" for $h/\Delta Dn <$ 4, and a "wild" extrapolation up to $h/\Delta Dn = 12$ is shown in fig. 1 as a curved dotted line.

Some further information can be derived from "near-bed structures" which are described in the Rock Manual (2007). These structures are obviously low compared to the water depth and rather long in wave propagation direction. They consist typically of bed protection works protecting against local scour or protecting a sealine.
Stability is defined on the base of currents: orbital bottom velocity u compared to the Shields mobility parameter (see Rock Manual, 2007, pp 607-608):
eq 5.173 reduces to the following in shallow water where sinh kh = kh and $c = \sqrt{gh}$:

$u2 = g\ Hs2 / 4\ hc$ (3)

where hc: water depth above the near-bed structure (-Rc in this paper).
For $Sd = 10$ and $N = 3000$ (Sd: damage, N: number of waves), eq 5.175 yields:

$u2/g\ \Delta\ Dn = 1$ (4)

Combining both, and with $Hs = 0.6\ hc$ and $\Delta = 1.6$, the following is found:

$hc/Dn = 18$ or: $Rc/Dn = -18$ (5)

It may be noted that if hc is close to h (small structure height), then Dn = around 0.05 h.
However, equation (5) is independent of h and would yield a horizontal line in fig. 1 ... which seems odd as we are supposed to be in shallow water: this approach is perhaps still a bit imperfect.

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 Dn = 11 mm, hence a crest width of over 100 Dn) was submitted to waves with $Hs/h = 0.24$ (pretty far from breaking wave conditions). The initial crest level was $Rc = +4.5\ Dn$ (that is $Rc/h = +0.06$) and the final crest level rose to nearly +10 Dn 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* .

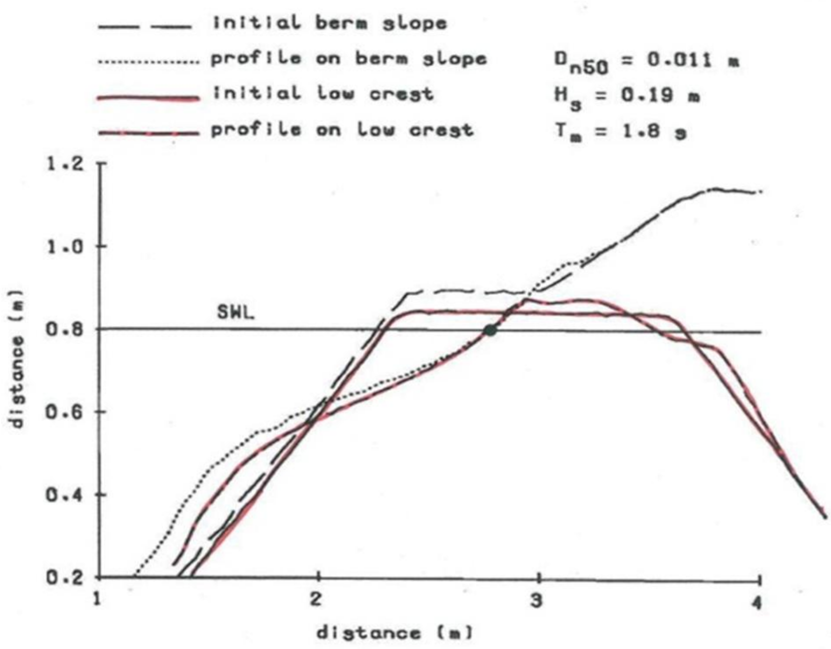


Figure 2. van der Meer's Figure 12 (1992)

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 Dn = 25.2 mm, hence a crest width of 44 Dn) was submitted to around 36 000 waves with Hs/h = 0.52 (which may be considered as breaking wave conditions). The initial crest level was Rc/Dn = -2.18 (that is Rc/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. Blocks 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 blocks as far as the rear slope where they would have fallen down without rising the crest level.
Waves cannot transport blocks 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 blocks to proceed further landwards under wave action. Hence, blocks 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 undistorded 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 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)?

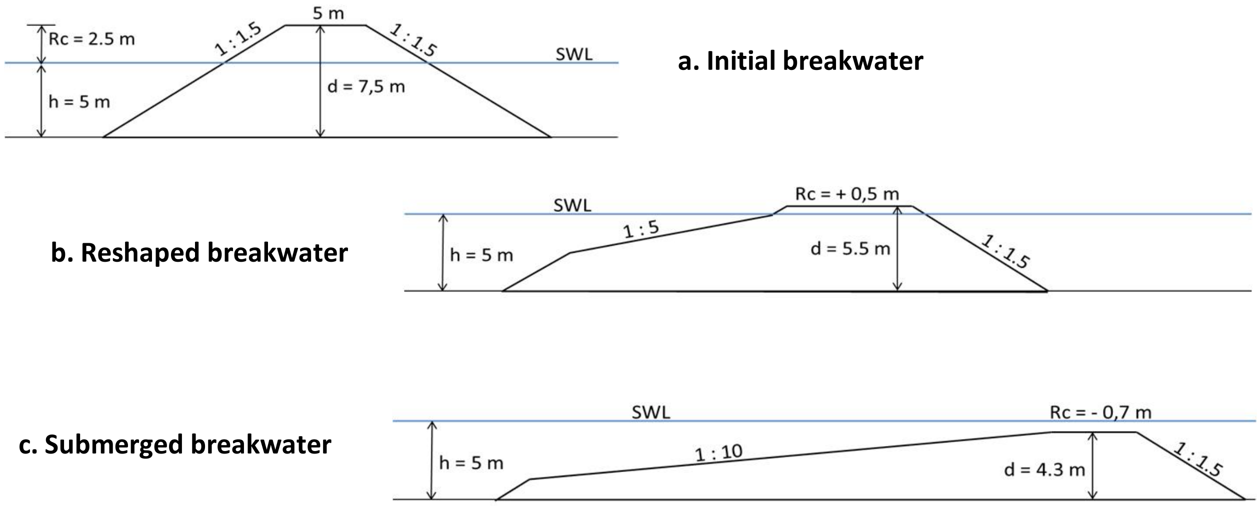


Figure 3. Destruction and reshaping of a breakwater.

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 √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 Dn = 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 Dn = 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 Tp = 1.75 s for all tests, except the last one when Tp 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 Dn. Three different stone sizes were tested:
>> D = 12 - 18 mm with average mass = 4.77 g and Dn = 12.2 mm (Test 1)
>> D = 8 - 12 mm with average mass = 1.46 g and Dn = 8.2 mm (Test 2)
>> D = 4 - 8 mm with average mass = 0.34 g and Dn = 5.0 mm (Tests 3, 3a, 3b)
For all stones Δ was 1.65.
The trapezoidal mound structure was rebuilt after Test 1 and Test 2, but no repair was done during Test 3.

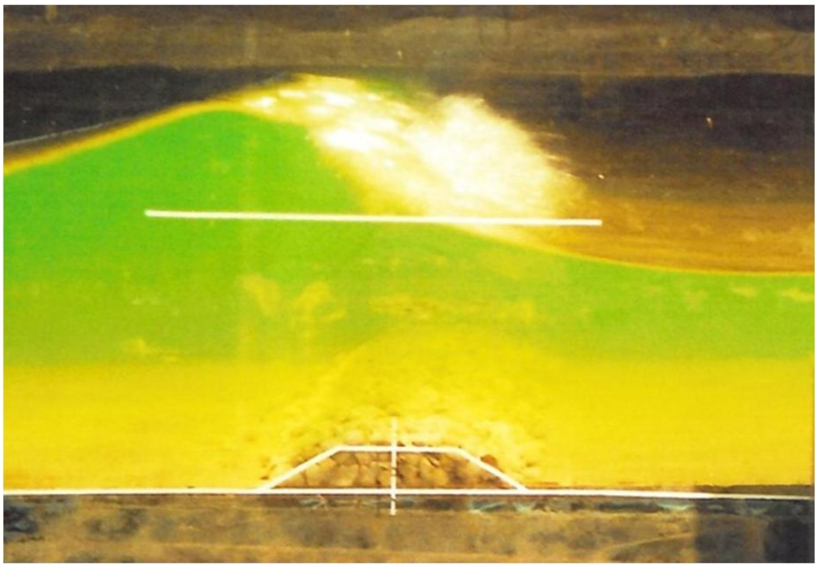


Figure 4. Model test on submerged rubble mound with breaking wave attack

Waves were increased in steps, starting from 45 mm. For Tp = 1.75 s, the highest waves that could be obtained on this water depth were measured as follows: Hs = 133 mm, with H1/10 = 157 mm and Hmax = 178 mm (note Hs/h = 0.53 and Hmax/Hs = 1.33). For Tp = 2.5 s, the measured maximum was Hs = 150 mm, and for h = 200 mm, the measured maximum was Hs = 138 mm (Hs/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 stalibise. The last step of each test was maintained for about 45 minutes (around 1500 waves).

Test 1 with Dn = 12.2 mm: the structure is reshaped into a rounded mound that is globally stable with waves Hs = 133 mm.

Test 2 with Dn = 8.2 mm: the structure is reshaped into a rounded mound that is dynamically stable with waves Hs = 133 mm, i.e. some stones are continuously moving back and forth but the overall mound shape is maintained.

Test 3 with Dn = 5.0 mm: the structure is reshaped into an asymetric rounded mound that is dynamically stable with waves Hs = 133 mm, i.e. the front slope is steeper than the rear slope so that the whole mound is moving slightly backwards.

Test 3a with Dn = 5.0 mm and Tp = 2.5 s: the same processes are going on with Hs = 150 mm as the structure has now lost 5 to 8 mm of its initial crest height and the rear toe has moved 50 mm backwards before stabilising.

Test 3b with Dn = 5.0 mm and Tp = 2.5 s and h = 200 mm: the same processes are going on with Hs = 138 mm as the structure has now lost 10 to 15 mm of its initial crest height and the rear toe has moved 90 mm backwards before stabilising.

The test results are summarised in the table below.

N°	h (mm)	Dn (mm)	Hs (mm)	Tp (s)	Rc (mm)	Rc/Dn	h/ΔDn
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.

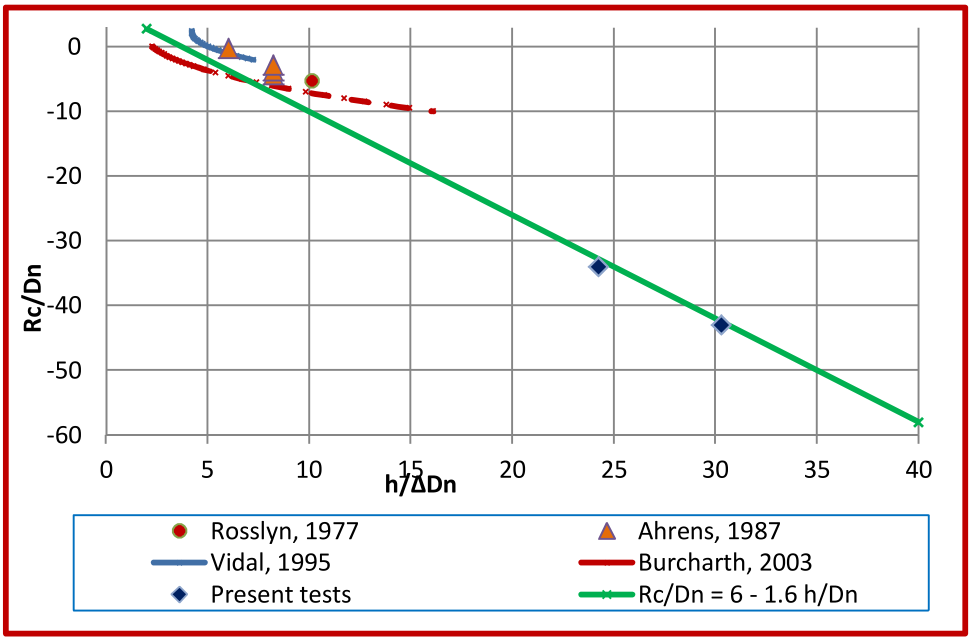


Figure 5. Stability of submerged breakwaters with breaking waves.

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. It can be seen that all extrapolations of existing data are incorrect (as the authors of these data have allways claimed).

The new data from the model tests show a new trend which is described by:

Rc/Dn = 6 -1.6 h/ΔDn (6)

or, with Δ = 1.6:

Rc/h = 6 Dn/h - 1 (7)

This 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.

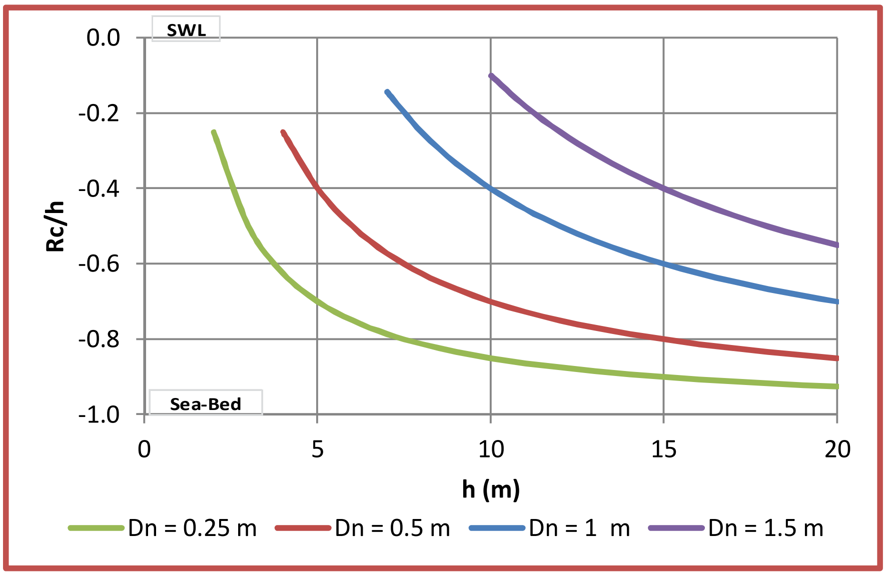


Figure 6. Stable submerged breakwater with breaking waves.

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 90% of the water depth, Dn = 1 m will be stable up to 60% of the water depth, Dn = 0.50 m up to 30% of the water depthand Dn = 0.25 m up to 15% of the water depth.

Note also that eq (7), which is valid for 4 < h/ΔDn < 30, can be written as: Dn = 0.17 d; similarly to Burcharth's rule eq (1), Dn = 0.29 d, which is valid for 2 < h/ΔDn < 4.

Conclusion:

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

$Rc/h = 6\ Dn/h - 1$ valid for $4 < h/\Delta Dn < 30$ 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.

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.

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