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Short communication

# On front slope stability of berm breakwaters

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ABSTRACT

The short communication presents application of the conventional Van der Meer stability formula for low-crested breakwaters for the prediction of front slope erosion of statically stable berm breakwaters with relatively high berms. The method is verified (Burcharth, 2008) by comparison with the reshaping of a large Norwegian breakwater exposed to the North Sea waves. As a motivation for applying the Van der Meer formula a discussion of design parameters related to berm breakwater stability formulae is given. Comparisons of front erosion predicted by the use of the Van der Meer formula with model test results including tests presented in Sigurdarson and Van der Meer (2011) are discussed. A proposal is presented for performance of new model tests with the purpose of developing more accurate formulae for the prediction of front slope erosion as a function of front slope, relative berm height, relative berm width, method of armour stone placement, and hydraulic parameters. The formulae should cover the structure range from statically stable berm breakwaters to conventional double layer armoured breakwaters.



## 1. Introduction

The main contents of this short communication is an analysis performed by the author in 2008 with the objective of predicting the observed and recorded front erosion of the Sirevåg berm breakwater in Norway. Sirevåg breakwater is designed as a statically stable breakwater, i.e. designed for very small deformations and in that sense not very different from conventional rock armoured breakwaters. On that background the author explored the possibility of predicting the observed deformations of the Sirevåg breakwater by the use of the Van der Meer (1988) rock armour stability formula which contains all relevant hydraulic and geometrical parameters (front slope, and relative berm height as used for low crest structures). Only berm width is not included in the formula. The needed information of the performance of the Sirevåg breakwater was given in Tørum et al. (2003b). The analysis of the author was presented solely orally in COAST 2008, Trondheim, Norway, a conference dedicated to Professor Alf Tørum, (Burcharth, 2008).

### 1.1. Reshaping berm breakwaters

The original concept of berm breakwater is to let the waves shape the front of the structure as nature – for the given size of stone material – will ensure that the most resistant profile is obtained. The berm is initially unstable but will reshape during normal and more severe conditions into more stable gentle S-curved slopes which change/adjust to the various sea states. The smaller the stone sizes, the flatter the S-profile thus demanding more materials initially placed in the berm. Moreover, the smaller the stone, the more the stone moves including transport along the structure in the case of oblique waves. The minimum stone size is thus determined by the risk of stone degradation and in some sections the lack of stones to feed the transport along the structure (PIANC MarCom WG 40, 2003). The structures are designed for maximum reshaping/recession of the berm in the design storm.

As a single simple parameter to characterize the deformation of the reshaping type of berm breakwaters was introduced the recession $R_{ec}$ of the berm shoulder (Burcharth and Frigaard, 1987), see Fig. 1. $R_{ec}$ has since then been used in a number of formulae fitted to model test results for multilayer berm breakwaters, i.e. Tørum et al. (1999, 2003a,b), Tørum (2007), Lykke Andersen (2006), Sigurdarson et al. (2008), Lykke Andersen and Burcharth (2010), Moghim (2009), Moghim et al. (2011), see Tørum et al. (2012) for more discussions.

### 1.2. Non-reshaping berm breakwaters

The non-reshaping type is designed for no erosion of the berm under more severe wave actions. Only for design storm conditions is some limited recession of the berm allowed. The advantage of this design philosophy which for some years has been developed and applied to a number of berm breakwaters in Iceland, (Sigurdarson et al., 2008), is that problems related to stone degradation by abrasion and to transport of stones along the structure are omitted. The design damage parameter has – besides overtopping – been solely the recession $R_{ec}$. However, for this type of breakwater it is most likely – as was the case for the Sirevåg breakwater – that the front slope is eroded before recession takes place, at least if the berm level (freeboard) is more than approximately half a significant wave height over SWL, i.e. $h_b/H_s >$ app. 0.5.

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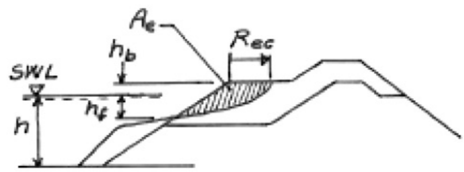


**Fig. 1.** Illustration of reshaping type of berm breakwater and definition of recession $R_{ec}$.

**Table 1**
Classification of berm breakwaters according to PIANC MarCom Report of WG 40.

| Type | $N_s = H_o$ | $H_oT_{om}$ |
|---|---|---|
| Statically stable, no reshaping of berm, negligible erosion of front | <1.5–2 | <20–40 |
| Statically stable, some reshaping of berm in design sea states | 1.5–2.7 | 40–70 |
| Dynamically stable, larger reshaping, movements of stones | >2.7 | >70 |

The relative berm height is of importance in optimization of the cross section. In Sigurdarson et al. (2005), relating to the Hammerfest breakwater, the authors states that: "the berm was heightened from elevation +5 m to +8.2 m, but at the same time the total width of the structure at elevation +5 m was narrowed by 7.4 m. The width reduction resulted in significant saving in rock volume".

The damage development for berm breakwaters with larger relative berm heights should be very much the same as for conventional low crested rubble mound breakwaters for which the main parameter in the stability formulae (e.g. Hudson and Van der Meer) is $H_o = N_s = H_s/\Delta D_{n50}$ in which $H_s$ is the significant wave height, $\Delta = \rho_s/\rho_w - 1$ where $\rho_s$ and $\rho_w$ are the mass density of rock and water respectively, and $D_{n50} = (M_{50}/\rho_s)^{1/3}$ where $M_{50}$ is the median mass of the rocks.

Fig. 2 illustrates the front erosion given by the eroded area $A_e$. The front erosion develops into a recession if $H_o$ increases sufficiently.

Berm breakwaters are according to the PIANC MarCom Report of WG 40 classified as given in Table 1.

The Sirevåg breakwater belongs to the statically stable breakwaters. The following presentation is related solely to this class of structures, i.e. $H_o < 2.7$, and is moreover restricted to structures with relative high berms, i.e. values of $h_b/H_{s,design} >$ approximately 0.5. For such structures, it is expected that front erosion takes place before the recession of the berm shoulder starts.

A new damage parameter $Rec_{av}$ for front slope erosion is presented in Sigurdarson and Van der Meer (2011). The parameter is defined as the horizontal recession of the averaged profile averaged between the low water level and the top of the berm. Based on model tests with an Icelandic type berm breakwater was developed the following formula, fitted to the measured $A_e$-values:

$$Rec_{av}/D_{n50} = 2.5\,(H_s/\Delta D_{n50} - Sc)^2, \quad \mu(Sc) = 1.1 \text{ and } \sigma(Sc) = 0.22, \quad \text{valid for } H_o = H_s/\Delta D_{n50} < 2.5.$$

The validity ranges for the cross sectional geometrical parameters are not given in the paper nor is the definition of "low water level". In the same paper, the model test S-values are compared with predictions based on the Van der Meer formula (by use of the BREAKWAT program). The results are shortly discussed in Section 5.2 of this short communication together with comparison of predictions by the Van der Meer formula and model test results by Lykke Andersen (2006).

The next section discusses the relevance of the design parameters used so far in the formulae for prediction of recession and front erosion.

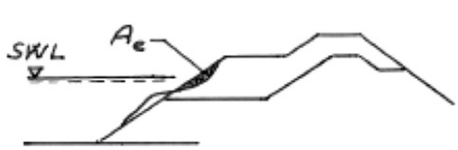


**Fig. 2.** Illustration of front slope erosion in non-reshaping berm.

## 2. Relevant parameters for characterization of deformations

The main stability parameter in the design of rubble mound structures is $N_s = H_o = H_s/(\Delta D_{n50})$ as applied in many stability formulae, e.g. by Hudson, Iribarren and Van der Meer.

The wave length/period influences the stability of rubble mound structures, especially for structures designed for larger values of $H_o$. In the armour stability formula of Van der Meer (1988) for prediction of the dimensionless eroded area $S = A_e/D_{n50}^2$, the wave period is taken into account through the surf similarity parameter $\xi = \tan\alpha/s_{om}^{0.5}$ where $s_{om}$ is the wave steepness which is inversely proportional to the square of the wave period. For plunging waves $H_o$ is proportional to $\xi^{-0.5}$. For surging waves $H_o$ is proportional to $\xi^P$, P being the notional permeability factor (Van der Meer, 1988). This reflects in both cases a rather week influence of the wave period.

Van der Meer (1988) introduced for berm breakwaters the dimensionless wave height–wave period parameter

$$H_oT_{om} = H_oT_m(g/D_{n50})^{0.5}$$

which implies that wave height and wave period have equal effects. The parameter is used in several formulae for $R_{ec}$, e.g. Vrijling et al. (1991), Van der Meer and Veldman (1992), Tørum (1998, 2007), Tørum et al. (2003a), Alikhani et al. (1996), Menze (2000), Lykke Andersen (2006), Sigurdarson et al. (2008).

However, Kao and Hall (1990) and Lykke Andersen (2006) found that the influence of the wave period on the profile deformation is very small for $H_o < 3.5$. The same was observed by Sveinbjørnson (2008) who concluded that $H_o$ probably would be a better parameter than $H_oT_{om}$. Moghim et al. (2011) derived by fitting to a number of model test results a formula based on the parameter $H_o(T_o)^{0.5}$, i.e. a relatively weak influence of the wave period and similar to that inherent in the Van der Meer, 1988 stability formula for conventional rubble mound breakwaters. Actually, depending on the region of wave steepness an increase in wave period (decrease in wave steepness) can lead to increased stability, i.e. less profile deformation, for conventional armour layers.

This indicates that the stability parameters $H_o$, or $H_o(T_o)^{0.5}$ are the more relevant parameters for statically stable berm breakwaters. This supports the assumption that the conventional formula for rock armour stability might be applicable.

## 3. Front erosion of statically stable berm breakwaters

It is obvious that the height of the berm over SWL, $h_b$ shown in Fig. 2 is of importance for the development of the erosion/deformation of the profile. If the berm is so high that significant over-wash of the shoulder does not occur, then it should be possible to predict closely the stability/erosion of the front by means of a stability formula for conventional rubble mound breakwaters, if valid for the steep slopes used in the berm breakwaters, and if adjusted for the high permeability of the berm. If significant over-wash occurs then the freeboard must be taken into account as is done for low crested structures.

Despite its importance the berm height $h_b$ is only included in the berm breakwater formulae by Lykke Andersen (2006), Lykke Andersen and Burcharth (2010), Moghim (2009) and Moghim et al. (2011). The