

Reviewer 2

This paper presents a new formula for the stability of coastal revetment rock armour under depth-limited breaking waves. The formula is well-calibrated using empirical data from physical model tests and offers a more coherent approach compared to existing methods. However, some limitations, such as the scope of the data and the consideration of seabed slope, need to be addressed. Overall, the paper makes a valuable contribution to coastal engineering by providing a practical tool for designing rock armour in shallow water conditions. The following suggestions will further improve the author's article.

- (1) *Does the method of calculating wave energy in the new formula adequately take into account the energy loss of waves during the breaking process?*

The formula for the total amount of wave energy is equation (5), being $E = f(H^2L)$. When the wave breaks the energy would be dissipated in a myriad of ways; in sound, moving rocks, wave runup, splashing, etc. How much is moving rocks we don't know. Nevertheless, that is the purpose of the scale model calibration, to relate the damage to the variables in the wave energy equation as follows:

The proposed wave energy formula was re-written as follows to make the calibration coefficient K' the dependent variable:

$$K' = \frac{H_s^2 T_p \sqrt{gh_{toe}}}{D_{n50}^3 \Delta^3 \cot \alpha^{1.5}} = f\left(\frac{D}{\sqrt{N}}\right) = a \left(\frac{D}{\sqrt{N}}\right)^n \quad (9)$$

Plotting $\frac{H_s^2 T_p \sqrt{gh_{toe}}}{D_{n50}^3 \Delta^3 \cot \alpha^{1.5}}$ against $\left(\frac{D}{\sqrt{N}}\right)$, as done in Figure 5, gave the values for the factor a and exponent n in equation (9) by fitting a power curve. Essentially, the relationship becomes:

$$k' = f\left(\frac{\text{Wave Energy}}{\text{bouyant mass} \times \text{revetment slope}}\right) \quad (10)$$

- (2) *Has an optimization analysis been performed on the effect of berm slope on the stability of the berm, and is there an optimal slope?*

This work was done originally by Hudson (1958) and modified by van der Meer (1988), the latter adopted in this article (as stated in the article). No optimal slope has been defined; the flatter the slope the more stable the rock.

- (3) *Does the method of quantifying revetment damage take into account the effect of localized damage on overall stability?*

No it doesn't and, often, geotechnical global stability is overlooked by coastal engineers. Nevertheless, the scale of armour damage is small compared with the scales considered for geotechnical global stability. This issue would not be pertinent in this article.

- (4) *Is the new formula effective in distinguishing the effects of different wave breaking types (e.g., plunging and surging) on berms?*

The new formula is applicable to waves shoaling on the relatively flat fronting seabed slope and breaking onto a steeper coastal revetment, a condition that the Shore Protection Manual describes as the most reasonable for revetment design. The new formula does not refer to waves shoaling and breaking on the steep revetment slope itself, as do the formulae of van der Meer that distinguish between plunging and surging. The formula has been based on both plunging and surging waves, although that did not become apparent in the calibration plots, so the new formula does not make that distinction, probably because only one seabed slope was modelled. The van der Meer formulae indicate that the latter are more critical for stability.

(5) *Is the number of wave actions mentioned in the paper sufficient to model the long-term wave erosion effects in real engineering?*

The number of waves used in the modelling of van Gent *et al.* (2003) was around 1,200, which could be representative only of a single storm. However, the number of waves as modelled by Gordon (1973) was far greater and could represent the impacts of several storms. Nevertheless, the damage has been normalised by the number of waves ($D/N^{0.5}$), which forms the basis of the calibration, so the formula can apply to however many waves the designer wishes to choose, although that would involve extrapolation as discussed in Experimental Constraints, which, with respect to the number of waves, includes:

- Much of the data is based on around 1,000 zero-crossing waves, which limits a small 1:50 Froude scale model to schematise a storm duration of up to 2 hours. For larger scale models the maximum prototype storm duration schematised would be shorter. Revetments in depth-limited situations are likely to be subjected to a much larger number of waves.

(6) *Are formulas calibrated solely on the basis of existing physical model data reliable enough to be applied to different geographic regions?*

Yes. The basic physics applies world-wide. However, as wave conditions vary over the globe there will be locations where the results from this modelling will have to be extrapolated rather than interpolated. Discussion around this has been greatly expanded with new sections on modelling constraints and scale effects:

Application of the proposed predictor should consider the following experimental constraints:

- Data are available for only one fronting seabed slope of $m = 0.033$. A likely range of nearshore seabed slopes could be $0.025 < m < 0.05$. Test data for these slopes are not available but are likely to result in a modified formula.
- The predictor is based on a paucity of data, with only 16 suitable test results available. A much larger data set is warranted.
- The shortest wave period used in the models was 1.3 s, which limits a small 1:50 Froude scale model to schematise only prototype peak wave periods shorter than 8s. For larger scale models the maximum prototype peak period schematised would be even shorter. Prototype peak wave periods may exceed 15 s.
- The smallest significant wave height modelled was 0.04 m, which limits a small 1:50 Froude scale model to schematise wave heights only up to 2 m. For larger scale models the maximum prototype significant wave height schematised would be smaller. Depth-limited prototype waves incident upon coastal revetments can exceed this.
- The lightest model armour rock was $M_{50} = 0.047$ kg, which limits a small 1:50 Froude scale model to schematise armour rock only up to 6 t. For larger scale models the maximum mass of prototype armour rock schematised would be smaller.
- Much of the data is based on around 1,000 zero-crossing waves, which limits a small 1:50 Froude scale model to schematise a storm duration of up to 2 hours. For larger scale models the maximum prototype storm duration schematised would be shorter. Revetments in depth-limited situations are likely to be subjected to a much larger number of waves.
- The predictor is based on results of 2-dimensional flume studies, which do not replicate any 3-dimensional processes that occur in the nearshore surf zone fronting revetments.
- The predictor is based on flume studies that did not consider wave overtopping.

Discussion of Reynolds number scale effects has been included as follows:

Reynolds number scale effects are possible. For example, if the armour rock was assumed to be spherical, to ensure similitude for drag and lift coefficients on a spherical object, the Reynolds number $2 \times 10^3 < Re < 2 \times 10^5$ for both model and prototype (Figure 10). For $Re < 2 \times 10^3$ the drag and lift coefficient for spheres increases whereas for $Re > 2 \times 10^5$ the coefficient decreases.

Reynolds Number is $Re = \rho VL/\mu$ or VL/u where ρ is fluid density, μ is dynamic viscosity, u is kinematic viscosity, the wave breaking velocity jet being $V = \sqrt{gh_{toe}}$ and the characteristic length $L = D_{n50}$. Should the model data be applied at normal scales for prototype concept design, the likely prototype ranges of Reynolds number would be $Re > 2 \times 10^5$ as indicated in Table 4. Values of $Re > 2 \times 10^5$ in a prototype with spherical armour units, such as armour rock, may cause the drag and lift forces of the flow being relatively lower than that in the model, leading to conservative estimates for requisite armour mass. However, Figure 10 applies to spheres that are submerged in flow, such as under non-breaking waves, but its applicability to armour rock in aerated flows of breaking waves is not known. Nevertheless, similar scale effects occur with certain types of concrete armour units [23,24].

Table 4. The range of model Reynolds numbers for the flume data presented herein and the range of likely prototype Reynolds numbers for the model data applied at two different scales

	D_{n50}		h_{toe}		Re	
	Min. (m)	Max. (m)	Min. (m)	Max. (m)	Min. (-)	Max. (-)
Model	0.026	0.029	0.060	0.200	2.0E+04	4.1E+04
Prototype for Scale 1:10	0.26	0.29	0.60	2.00	6.3E+05	1.3E+06
Prototype for Scale 1:50	1.3	1.5	3.0	10.0	7.1E+06	1.5E+07

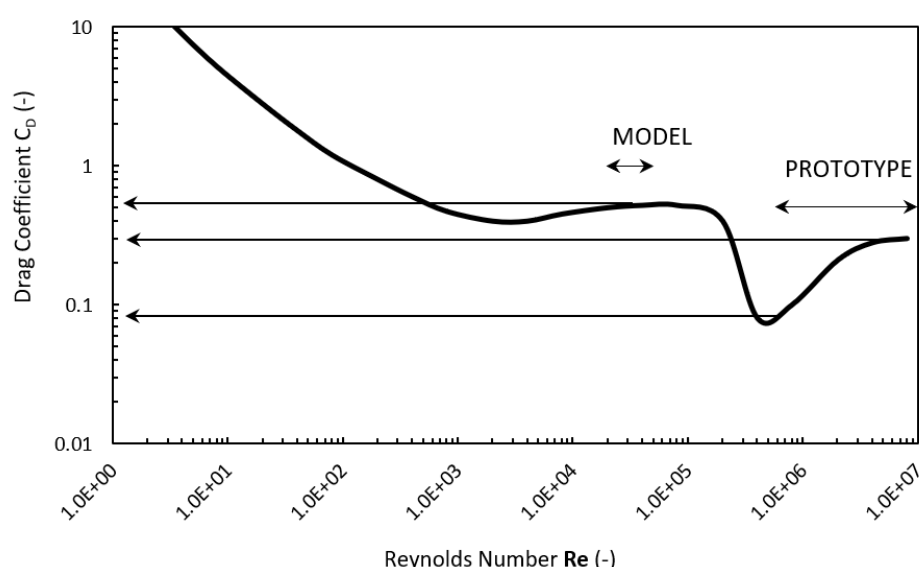


Figure 10. The variation with Reynolds number of the drag coefficient for spheres submerged in uniform flow [25]. The range of model values and likely prototype values is indicated.

(7) *Please clearly define the scope of “shallow water depth” in the text.*

The following (italicised) has been included at line 120:

In shallow water, where the depth is less than $0.04L$ ($0.016T^2$) [15 p2-9],

(8) *Is the effect of wave period on slope stability overemphasized? Are there other factors whose effects have been underestimated?*

The requisite stable armour mass is directly proportional to wave period, as indicated in Figure 7, which shows that both the wave energy formula and the van der Meer formulae indicate that waves with period $T \approx 13$ s require twice the mass than do $T \approx 7$ s waves in the same depth. However, the very

popular Hudson formula and that of van Gent have no dependence whatsoever on wave period. That the new wave energy formula supports the van der Meer formulae in this respect adds weight to the importance of considering wave period in design, which requires emphasis. Factors that have not been estimated at all include seabed slope, which is highlighted in the article and for which recommendations have been made. Further, the article emphasises the importance of the number of waves in design.

(9) Under what conditions is the new formula more advantageous than the Hudson and van Gent formulas?

The new formula has advantages over the Hudson and van Gent formulae in that it gives consideration not only to wave period but requires the designer to think also about water depths for design and, with respect to the Hudson formula, the number of waves, that is, storm duration and wave period.