# Recent Developments in the Design of Conventional Rubble Mound Structures

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# Recent Developments in the Design of Conventional Rubble Mound Structures

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**Abstract**
Conventional rubble mound structures such as breakwaters, seawalls, and revetments are the most common type of coastal structures around the world used to protect harbour basins and embankments from wave action. To have a safe and economic design, two aspects need to be considered. The first one is the structural stability where the required armor size (weight) must be determined. The second aspect is the safety, where the crest freeboard of the structure is usually determined based on the allowable mean wave overtopping rate. Several semi-empirical formulas have been developed for these purposes. These formulas, which have evolved over time, are generally semi-empirical and based on the small-scale laboratory experiments where both incident wave characteristics and the structure configuration are considered.

This paper aims to provide a comprehensive overview of the performance of existing formulas developed for the assessing the stability and mean overtopping rate of conventional rubble mound structures, while also introducing the recent ones. The Rock Manual formulas for the slope stability and EurOtop formula for estimating the mean overtopping rate will be discussed, and their performances will be compared with those of more recent and comprehensive ones using both lab and field data. It will be shown that the recent formulas that utilize the spectral energy mean period for stability analysis and run-up for the mean overtopping rate are more robust and physically sound. Finally, design formulas and uncertainty estimates are presented, along with guidance for practitioners.

*Keywords: mean overtopping discharge, armour stability, seawall, breakwaters, shallow water*

## 1. Introduction

Conventional rubble mound structures such as breakwaters, seawalls, and revetments are the most common type of coastal structures around the world used to protect harbour basins and embankments from wave action. Two aspects need to be considered to have a safe and economical design. The first aspect is structural stability, where the required armour size (weight) must be determined. The second aspect is safety, where the crest freeboard of the structure is usually determined based on the allowable mean wave overtopping rate. Several semi-empirical formulas have been developed for these purposes. These formulas, which have evolved in time, are generally semi-empirical and based on the small-scale laboratory experiments where both incident wave characteristics and structure configuration are considered.

This paper aims to provide an overview of the performance of formulas developed in the literature for the estimation of mean overtopping rate and stability of conventional (simple sloped) rubble mound structures under both head-on and oblique, multi-directional waves; and to discuss their performances.

## 2. Background

### 2.1 Mean Overtopping Rate of Rubble Mound Structures

The safety of rubble mound structures, such as breakwaters and seawalls, is mainly determined by the mean overtopping rate. Excessive overtopping may damage properties, hazard to people on the crest of structure, and erosion as well as instability of the rear and crest of structures. Hence, the crest level of coastal structures is usually determined based on the allowable mean overtopping rate. Design manuals such as EurOtop (2018) specify the allowable mean overtopping rates. These allowable overtopping rates depend on the function of the structure and wave height; and are determined based on the structural stability and property/operation aspects (EurOtop, 2018).

Different tools have been developed to predict the mean overtopping rate at rubble mound structures. These tools are primarily developed based on either traditional curve fitting (e.g., TAW, 2002; EurOtop, 2018) or data mining ones (e.g., Van Gent et al, 2007; Jafari and Etemad-Shahidi, 2012; Hosseinzadeh et al., 2021). The developed models are based on small-scale laboratory experiments and scaling arguments in both approaches. The pioneer study of mean overtopping rate was conducted by Owen (1980) on seawalls and the following formula was proposed:

$$Q^* = a \exp\left(-b \frac{R_c}{H_{m0}} \sqrt{\frac{s_{oz}}{2\pi}} \frac{1}{\gamma_f}\right) \quad (1)$$

where $Q^* = q/(gH_{m0} T_z)$, $q$ = the mean overtopping rate ($m^3/s/m$), $g$ is gravity acceleration, $T_z$ = mean zero crossing wave period, $s_{oz}$ = the fictious wave steepness, $H_{m0}$ = the (significant) spectral wave height, and $R_c$ = the crest freeboard.

The roughness (and permeability) reduction factor, $\gamma_f$ accounts for the roughness and permeability of different armour layers. Here, *a* and *b* are empirical coefficients which depend on the seaward slope of the structure.

De Waal and Van der Meer (1992) found that this formula performs poorly for all wave-breaking conditions. Hence, they suggested a more physical-based approach, i.e., a mean overtopping formula based on the difference between the max run-up and crest height ($R_{umax}$ - $R_c$). This parameter has been used for the estimation of the mean overtopping rate by some other researchers (e.g., Hedges and Reis, 2004; Ibrahim and Baldock, 2020).

Later on, Van der Meer and Janssen (1994) considered the effects of structure slope explicitly and proposed different formulas for low and high Iribarren numbers as:

If $Ir_{op} < 2$ then $q^* =$

$$\sqrt{\frac{\tan\alpha}{s_{op}}}\, 0.06 \exp\left(-5.2 \frac{R_C}{H_{m0}} \frac{\sqrt{s_{op}}}{\tan\alpha} \frac{1}{\gamma_f \cdot \gamma_h \cdot \gamma_\beta}\right) \quad (2a)$$

If $Ir_{op} \geq 2$ then $q^* =$

$$0.2 \exp\left(-2.6 \frac{R_C}{H_{m0}} \frac{1}{\gamma_f \cdot \gamma_h \cdot \gamma_\beta}\right) \quad (2b)$$

Where $q^* = q/(gH_{m0}^3)^{1/2}$, $Ir_{op} = \tan\alpha / s_{op}^{1/2}$, $q^* = q/(g H_{m0}^3)^{1/2}$, $\gamma_h$ = shallow water reduction factor (unity in deep water) and $\gamma_\beta$ = wave obliquity reduction factor.

In a more recent study, EurOtop (2018) used another functional form and recommended the following formula for steep slopes ( 1:2 to 1:1.33):

$$q^* = 0.09 \cdot \exp\left[-\left(1.5 \frac{R_c}{H_{m0} \cdot \gamma_b \gamma_f \cdot \gamma_\beta}\right)^{1.3}\right] \quad (3)$$

$\gamma_\beta = 1 - 0.0063\beta \geq 0.496$ and surprisingly, the effects of the wave period, crest wall and structure slope (Figure 1) are omitted in this formula. However, the effect of crest width has been considered using the following reduction factor:

$C_r$= 3.06 exp (-1.5 $G_c/H_{m0}$) < 1 for $G_c$ > 3 $D_{n50}$ *(4)*

Where $D_{n50}$ = nominal rock diameter and $G_c$ = the crest width.

Koosheh et al. (2022), extended the existing seawall database by conducting 2D experiments and showed that the inclusion of wave period (in terms of wave steepness) improves the accuracy of the prediction formula.

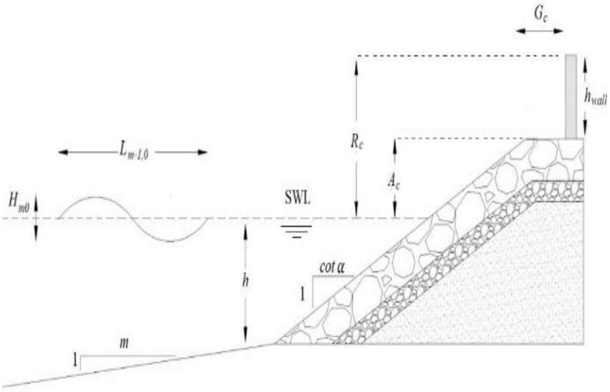


Figure 1 Typical cross-section of rubble mound structure ( after Etemad-Shahidi et al. 2022).

Finally, Etemad-Shahidi et al. (2022) scrutinised and extended the existing databases of EurOtop (2018) and provided a more comprehensive one for the rubble mound structures. The utilized dataset was the extended CLASH-database (also known as the EurOtop 2018 database), which was supplemented with the latest measurements from Koosheh et al. (2022). Initially, a thorough examination of the database's references was conducted to ensure accurate reporting or proper estimation of wave characteristics. Most of the references prior to 2000 provided values for $H_{1/3}$ (significant wave height based on time-domain analysis) and *Tp*. Therefore, in order to create a standardized database containing the required parameters, $H_{1/3}$ values were converted to $H_{m0}$ for shallow water tests that had not been corrected. Subsequently, $T_{m-1,0}$ was estimated using the method proposed by Hofland et al. (2017). Finally, the recent dataset from Koosheh et al. (2022) was incorporated into the database, consisting of approximately 140 small-scale records of relatively steep rock armoured seawalls with an impermeable core. They provide an improved prediction and a compact formula for the mean overtopping rate given below:

$$q^* = 1.2\times10^{-4} \exp\left[3.50\left(\frac{Ru_{2\%}-R_c}{H_{m0}}\right) - 0.64\left(\frac{G_C}{H_{m0}}\right)\right] \quad (5)$$

Where $Ru_{2\%}/H_{m0}$ = the dimensionless wave run-up exceeded by 2% of waves. As seen, the formula is very compact and simpler than those proposed by other researchers. In addition, it is more physically sound compared to previous ones as it relates the mean overtopping rate to the difference between the run-up and crest levels, i.e., excess run-up. To optimise the accuracy of the formula, different run-up formulas, proposed in the literature were tested, and finally, the following form suggested by EurOtop (2018) was selected:

$$R_{u2\%}/H_{m0}=1.65\gamma_f\gamma_\beta\gamma_w\, Ir_{m-1,0}\leq 1.0\, \gamma_{f surging}\, \gamma_\beta\, \gamma_w\, (4.0 - 1.5/\sqrt{Ir_{m-1,0}}) \qquad (6)$$

Where $\gamma_{f\ surging} = \gamma_f + (Ir_{m-1,0} - 1.8)\times(1-\gamma_f)/8.2$ with a maximum of $R_{u2\%}/H_{m0}$= 2 for structures with a permeable core (such as breakwaters) and a maximum of 3 for impermeable structures (such as seawalls). Here $\gamma_\beta = \cos^2(\beta - 0.6\, S)$ for $S < \beta$. The effect of wave wall (see Figure1) was also considered as a reduction factor, i.e., $\gamma_w$= exp (0.10 $h_{wall}/R_c$) with a minimum of $\gamma_w$= 1.0.

## 2.2 Stability of Rock Armoured Structures

In a pioneering study of the stability of armoured rock structures, Hudson (1959) conducted tests using regular waves. The following formula was proposed:

$$N_s = H_s / (\Delta D_{n50}) = (K_D \cot \alpha)^{1/3} \qquad (1)$$

Where $N_s$ = stability number, $H_s$ = significant wave height, $D_{n50}$ = nominal rock median size, $\alpha$ =the structure front angle, $\Delta = \rho_a/\rho_w - 1$ is relative buoyant density, $\rho_a$ =rock density, and $\rho_w$ = water density. The effects of armour type and breaking/non-breaking wave condition were considered in the stability coefficient, $K_D$. The effects of the permeability, wave period, damage level, and number of waves (storm duration) were not considered in the abovementioned formula. Later, Van der Meer (1988) conducted tests using irregular waves but mostly in deep water ($h/H_s$ > 3) where $h$ = water depth at the toe. By introducing the (nominal) permeability, $P$, two formulas were developed for plunging and surging wave conditions:

$$N_s = 6.2 S_d^{\ 0.2} P^{\ 0.18} N_w^{\ -0.1} Ir_m^{-0.5} \quad \text{for } Ir_m < Ir_{mc} \qquad (2a)$$

$$N_s = S_d^{\ 0.2} P^{\ -0.13} N_w^{\ -0.1} Ir_m^{p} \cot \alpha^{0.5} \quad \text{for } Ir_m > Ir_{mc} \qquad (2b)$$

Where $S_d$ = the damage level, $Ir_m$ =the Iribarren parameter using the mean wave period ($T_m$) and $Ir_{mc}$= (6.2 $P^{0.31}$ tan $\alpha^{0.5})^{1/(P+0.5)}$. For shallow water conditions, the stability number in these formulas was corrected using $H_{2\%}/H_s$=1.4. More tests in shallow water were conducted and analysed by Van Gent et al. (2003). They recalibrated Van der Meer (1988) formula using $T_{m-1,0}$, the spectral mean energy period as:

$$N_s=8.4 S_d^{\ 0.2} P^{0.18} N_w^{\ -0.1} Ir_m^{-0.5} (H_s/H_{2\%}) \text{ for } Ir_m<Ir_{mc} \qquad (3a)$$

$$N_s=1.3 S_d^{0.2} P^{-0.13} N_w^{-0.1} Ir_m^{p} \cot\alpha^{0.5} (H_s/H_{2\%}) \text{ for } Ir_m>Ir_{mc} \qquad (3b)$$

Recently, Etemad-Shahidi et al. (2020) provided a unified design formula for the stability of rock armour structures. Using datasets of Van der Meer (1988), Van Gent et al. (2003), Thompson and Shuttler (1975) and Vidal et al. (2006), they developed an extensive database with about 800 records within the design range, i.e., $2 \leq S_d \leq 12$ which covers a wide range of parameters. For example, it covered tests in deep ($h/Hs$ >~3), shallow (~1.75 < $h/Hs$ < ~3) and very shallow ($h/Hs$ < ~1.75) conditions where $h$= local water depth. It should be mentioned that the data base includes both small-scale and large-scale tests. The majority of tests conducted under depth-limited conditions were from the Van Gent et al. (2003) study. These tests were carried out with foreshore slopes of 1:30 and 1:100. The utilized dataset encompasses a wide range of wave spectra. The Thompson and Shuttler (1975) employed the Moskowitz spectrum, Van der Meer (1988) predominantly utilized peaked Pearson-Moskowitz spectra, while Van Gent et al. (2003) employed JONSWAP, TMA, and double-peaked spectra. In certain test programs, $T_{m-1,0}$ values were not directly measured and were instead estimated using standard relationships, specifically $T_p$=1.1 $T_{m-1,0}$ for standard single-peaked spectra in deep water. The wave steepness values indicate that the database covers both sea and swell conditions. Approximately 75% of the tests in the database were conducted with surging waves ($Ir_{m-1,0}$ > 1.8).

They resolved the issue with the physical justification of the functional form and role of the nominal permeability in Van der Meer (1988) type formulas as:

$$N_s=4.5 C_p N_w^{-1/10} S_d^{1/6} Ir_{m-1,0}^{-7/12} \text{ for } Ir_{m-1,0}<1.8 \qquad (4a)$$

$$N_s=3.9 C_p N_w^{-1/10} S_d^{1/6} Ir_{m-1,0}^{-1/3} \text{ for } Ir_m>1.8 \qquad (4b)$$

Interestingly, their splitting criteria for breaking type is much simpler than previous one and is in line with the recently suggested value in EurOtop (2018). Here $C_p$ = the coefficient of permeability defined as:

$$C_p = [1+ (D_{n50c}/D_{n50})^{3/10}]^{3/5} \qquad (5)$$

Where $D_{n50c}$ = the median nominal size of the core material. As seen, the permeability of the conventional layer composite was quantified using the relative core size, which is physically sound. Etemad-Shahidi et al. (2020) also showed that for shallow water conditions where the water depth is less than three times of significant wave height, the stability number could be multiplied by (1-3 $m$), where $m$ is the foreshore slope.

In many real cases, incident waves are neither head-on (normal to the structure) nor unidirectional. Hence, several studies have been conducted to resolve these effects on the stability of rubble mound structures. These effects are usually considered as a reduction factor that depends on the wave angle. Table 1 summarises the wave reduction factor suggested in different references. Within them, Galland (1995) indicates the lowest influence of wave obliquity, while others yield more or less similar results for wave angles less than 45$^0$.

Table 1 Wave obliquity reduction factor for rock armour diameter, different studies.

| **Reference** | ***Bias*** |
|---|---|
| Galland (1995) | $\cos^{0.25}\beta$ |
| Yu et al. (2002) | $\cos^{1.16}\beta$ |
| Wolters and Van Gent (2011) | $\cos^{1.1}\beta$ |
| Van Gent (2014) | $(1-c_\beta)\cos^2\beta + c_\beta$ |

Recently, to consider both the effects of incident wave obliquity and spreading on the stability of rock slopes, Bali et al. (2023) used lab wave basin data sets of Yu et al. (2002) and Van Gent (2014). They examined the existing formulas mentioned above and suggested the following one to quantify both the effects of wave angle as well as directionality:

$$\gamma_{\beta s} = (1-c_\beta)\cos^2\beta + c_\beta\ ,\quad c_\beta = 0.44 + 0.004S \qquad (6)$$

where, $\gamma_{\beta s}$ = reduction in the nominal rock diameter, and $S$ is the spreading (in degrees). Simply saying, the formula which is compatible with Etemad-Shahidi et al. (2020), implies that the more the obliqueness of the incident wave and /or the more its spreading, the smaller the required rock size.

# 3. Results and Discussions

## 3.1 Mean overtopping rate

The performance of the developed formula and others were evaluated using an extensive head-on wave database both qualitatively and quantitatively. Table 2 displays the accuracy metrics of the most well-known formulas. As shown, the formula developed by Etemad-Shahidi et al. (2022) has the minimum *bias* and *RMSE.* Interestingly, the formulas suggested in EurOtop different versions (EurOtop 2018, Van der Meer and Janssen, 1994) underestimate the mean overtopping rate, resulting in an unsafe design.

Table 2 Accuracy metrics of different mean overtopping rate formulas, head-on tests

| **Reference** | ***Bias*** | ***RMSE*** |
|---|---|---|
| Owen (1980) | 0.26 | 0.74 |
| Van der Meer and Janssen (1994) | -0.30 | 0.80 |
| EurOtop (2018) | -0.60 | 0.98 |
| Etemad-Shahidi et al. (2022) | 0.0 | 0.54 |

The more recent formula is unbiased and has the minimum *RMSE*, i.e., better agreement with measurements. The scatter diagram of their formula is also shown in the following figure:

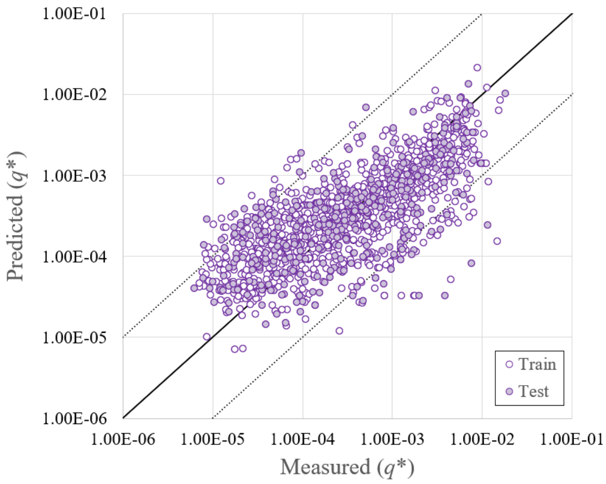


Figure 2. Scatter plot of mean overtopping rate, small-scale head-on tests (after Etemad-Shahidi et al. 2022). Dashed lines indicate 10 times under/overestimation.

They also showed that their formula outperforms existing ones in oblique waves, prototypes and model scaled cases with headwall.

## 3.2 Stability of Rock Armoured Structures

### 3.2.1 Head-on waves

Visual comparison of measured and predicted stability numbers using Etemad-Shahidi et al (2020) formula set is shown in Figure 3. As seen, the formula performs well for a wide range of stability number with no bias regarding any specific data set. It should be mentioned that the 90% confidence band is ±0.19 for Eq. 4a and ±0.15 for Eq. 4b, respectively.
The following accuracy metrics, i.e. scatter index and discrepancy ratio were used to evaluate the performances of formulas qualitatively:

$$DR = \frac{1}{n}\sum_{i=1}^{n}(p_i/m_i) \qquad (7)$$

$$SI = \frac{\sqrt{\frac{1}{n}\sum_{i=1}^{n}(p_i-m_i)^2}}{\bar{m}_i} \times 100 \qquad (8)$$

where $p_i$ = precited value, $m_i$ =the measured value, and $n$ = number of measurements, and the bar denotes the mean value. $DR$ = discrepancy ratio and $SI$ = scatter index. Results of different formulas shown in Table 3, also imply the skilfulness of Etemad-Shahidi et al. (2020) approach with a minimum Scatter Index and no bias.

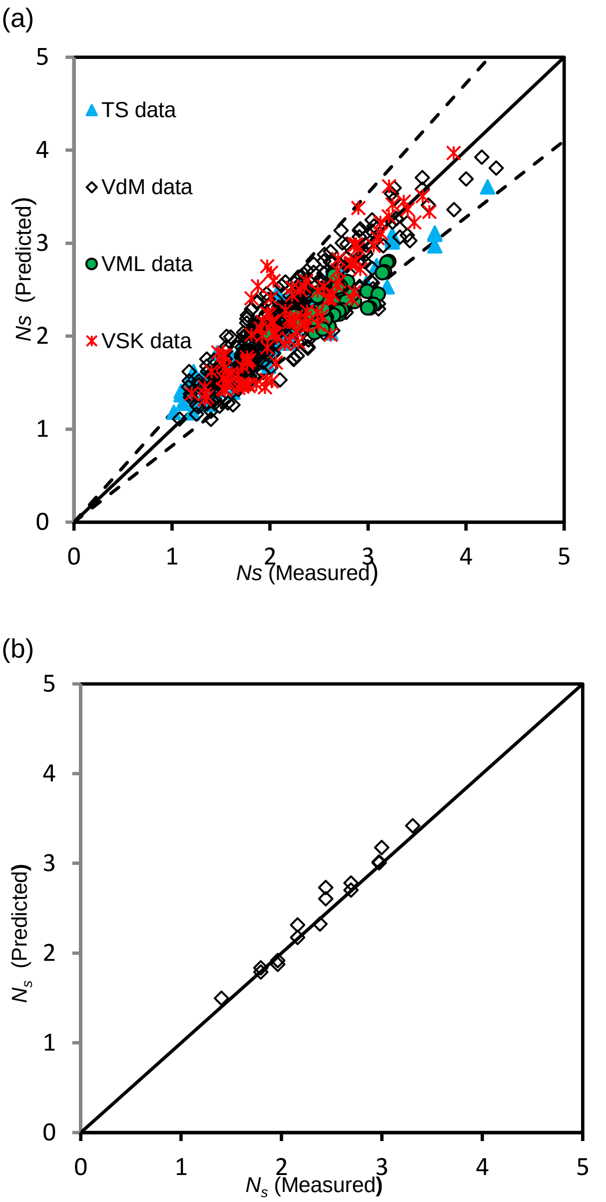


Figure 3 Scatter plot of stability numbers, (A) small scale test, ( b) large scale tests of Van der Meer (1988). Dashed line indicates 90% confidence band.

Table 3 Accuracy metrics of different stability formulas, small-scale tests

| **Reference** | ***SI* (%)** | ***DR*** |
|---|---|---|
| Van Der Meer (1988) | 13.4 | 1.02 |
| Van Gent et al. (2003) | 12.8 | 0.96 |
| Etemad-Shahidi et al. (2020) | 11 | 1.00 |

### 3.2.2 Oblique and multi-directional waves

The comparison between the measured and predicted stability numbers, using Bali et al. (2023) obliquity reduction factor, is displayed in Figure 4.

As seen, the data collapse well. The performance of the developed reduction factor was also evaluated quantitatively using accuracy metrics such the scatter index and correlation coefficient, for different data sets in Table 4. As seen the results are consistent for different data sets.

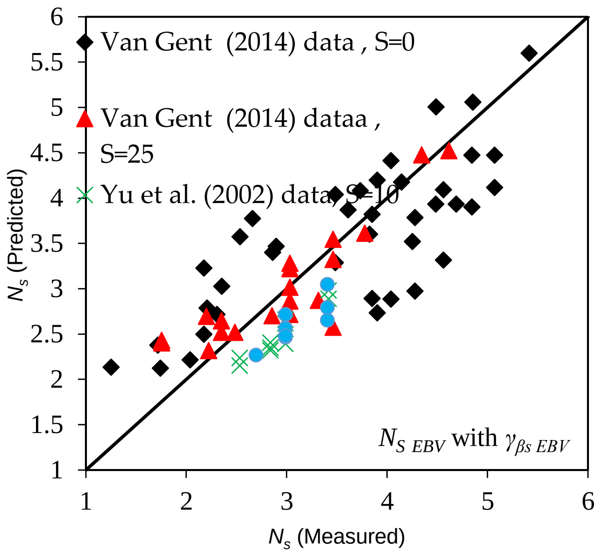


Figure 4 comparison between observed and calculated stability numbers using the new reduction factor.

Table 4 Accuracy metrics of Bali et al. (2023) formula, different stability formulas, small scale tests

| **Data base** | ***SI* (%)** | ***CC*** |
|---|---|---|
| Van Gent (2014) | 17.4 | 0.80 |
| Yu et al. (2002) | 16.6 | 0.81 |

## 4. Summary

This paper reviews and summarises recent findings regarding conventional armoured coastal structures' safety and structural design. It was shown that the newly developed formulas which are developed based on the extended databases, are more accurate both in small-scale and large-scale tests; and outperform existing ones. In addition, they are more physically sound as they include effects of all governing parameters. These formulas are well-suited for both the (conceptual) design of uncomplicated conventional rubble mound structures and the evaluation of existing structures regarding damage and wave overtopping. They are valuable tools for engineers and practitioners, enabling them to effectively address the design requirements and assess the performance of such structures, ensuring their resilience against damage and minimizing wave overtopping risks. It should be noted that practitioners need to consider the uncertainty in the design equations used. All the abovementioned formulas were obtained by best fitting and are based on the mean approach (see EurOtop 2018 for details). The adopted design approach should be semi-probabilistic with a partial safety factor, especially if physical modeling is not conducted. Otherwise, the chance of failure (say by using Eq. 4) is about 50%, which is too high.

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