Wave run-up on bermed coastal structures

Karthika Pillai[a], Amir Etemad-Shahidi[*b,c,d] and Charles Lemckert[e,d]

[a]Griffith School of Engineering and Built Environment, Griffith University, QLD, 4222, Australia

karthika.pillai@griffithuni.edu.au

[b]Griffith School of Engineering and Built Environment, Griffith University, QLD, 4222, Australia

[c]School of Engineering, Edith Cowan University, WA, 6027, Australia

[d]Griffith Centre for Coastal Management, Griffith University, QLD, 4222, Australia

E-mail: a.etemadshahidi@griffith.edu.au

[e]School of Design and the Built Environment, University of Canberra, Bruce, ACT, 2617, Australia

[d]Griffith Centre for Coastal Management, Griffith University, QLD, 4222, Australia

E-mail: Charles.Lemckert@canberra.edu.au

Abstract

Reliable estimation of wave run-up is required for the effective and efficient design of coastal structures when flooding or wave overtopping volumes are an important consideration in the design process. In this study, a unified formula for the wave run-up on bermed structures has been developed using collected and existing data. As data on berm breakwaters was highly limited, physical model tests were conducted and the run-up was measured. Conventional governing parameters and influencing factors were then used to predict the dimensionless run-up level with 2% exceedance probability. The developed formula includes the effect of water depth which is required in understanding the influence of sea level rise and consequent changes of wave height to water depth ratio on the future hydraulic performance of the structures. The accuracy measures such as *RMSE* and *Bias* indicated that the developed formula is more accurate than the existing formulas. Additionally, the new formula was validated using field measurements and its superiority was observed when compared to the existing prediction formulas. Finally, the new design formula incorporating the partial safety factor was introduced as a design tool for engineers.

Keywords: Wave run-up; Bermed structures; Laboratory experiments; Surf similarity; Prediction formula; Field measurements

1. Introduction

Wave run-up height (R_u) is an important hydraulic parameter in the design of coastal structures such as dikes, revetments and breakwaters [1]. R_u is defined as the maximum distance travelled by a wave up a slope and is measured vertically from the still water level (SWL) during the wave action (Fig. 1). Knowledge of R_u is required for the safety assessment of coastal structures and for estimating the number of waves that will reach the crest of the structure and overtop [2]. In addition, information on run-up is utilized to estimate the damage to the inner structure slope [3]. Thus, a reliable prediction of run-up is essential to develop a safe design of the coastal structure. Importantly, the prediction tool must be robust and should include the influence of water depth on wave run-up as this may change over time as the result of sea level rise or unpredicted coastal subsidence.

Estimation of run-up is more relevant to smooth sloped structures and structures situated on shore (such as seawalls) and is often less important for rubble mound breakwaters as their crest levels are often determined from overtopping rates. Recently Van der Meer et al. [2] has provided allowable values of mean overtopping rates as well as maximum overtopping volumes for different cases. Pillai et al. ([4], [5]) reviewed the existing approaches for the estimation of mean overtopping rates and developed a more accurate formula for it. To our knowledge, no study has been conducted on the estimation of max overtopping volume and runup at berm structures. The run-up prediction is essential to estimate the number of waves that may reach the crest and eventually lead to wave overtopping [2]. Additionally, it is required to determine the maximum overtopping volume of bermed structures. One of the earliest formulas to estimate the maximum run-up (R_{um}) on bermed revetments was suggested by Ahrens and Ward [6] as:

$$R_{*m} = \frac{R_{um}}{H_{m0}} = \exp\left(0.695 - 11.269\frac{H_{m0}}{L_{0p}} - 0.158B^{`}\right) \qquad (1)$$

where H_{m0} is the significant wave height, L_{0p} is the peak wave length and $B^{'}$ is a berm width factor [6] which is defined as:

$$B^{'}=\frac{B}{\sqrt{H_{m0}L_{0p}}}\frac{h_{bt}}{h} \tag{2}$$

Here, B is the berm width, h_{bt} is the height of the berm above the toe of the structure and h is the water depth at the toe of the structure. It should be mentioned that the berm levels tested by Ahrens and Ward [6] were all submerged and therefore the effect of the berm emergence was not investigated. Noting that most of recent formulas for runup are given for 2% exceedance probability and Ahrens and Ward [7] experiments were conducted on slopes with two layers of rock, Eq. (1) can be modified to:

$$R_*=\frac{R_{u2\%}}{H_{m0}}=0.79\,R_{*m}=1.97\,\gamma_f\,exp\left(0.695-11.269\frac{H_{m0}}{L_{0p}}-0.158B^{'}\right) \tag{3}$$

The conversion factor has been estimated by considering a roughness factor (γ_f) of 0.4 for experiments with two layers of armour [2] and a $R_{u2\%}$ to R_{um} ratio of 0.79 obtained by measurements of Van der Meer [8].

Recently, Van der Meer et al. [2] suggested a power formula to estimate the dimensionless run-up as:

$$R_*=\frac{R_{u2\%}}{H_{m0}}=1.65\,\gamma_b\gamma_f\gamma_\beta\xi_{m-1,0} \tag{4}$$

With a maximum of

$$\begin{cases} 1.0\gamma_f\gamma_\beta\left(4-\dfrac{1.5}{\sqrt{\gamma_b\xi_{m-1,0}}}\right)<3 & \text{for dikes} \\[4mm] 1.0\gamma_{fsurging}\gamma_\beta\left(4-\dfrac{1.5}{\sqrt{\gamma_b\xi_{m-1,0}}}\right)<2 & \text{for rubble mounds} \end{cases} \tag{5}$$

where

$$\xi_{m\text{-}1,0} = \frac{\tan\alpha_{av}}{\sqrt{s_{m\text{-}1,0}}} \qquad (6)$$

$\tan\alpha_{av}$ is the average slope defined as:

$$\tan\alpha_{av} = \begin{cases} \dfrac{3H_{m0}}{L_s - B} & 1^{st}\text{estimate} \\[2mm] \dfrac{1.5H_{m0} + R_{u2\%(1st\ estimate)}}{L_s - B} & 2^{nd}\text{estimate} \end{cases} \qquad (7)$$

$$L_s = B + [(1.5H_{m0} + h_b)\cot\alpha_u] + [(1.5H_{m0} - h_b)\cot\alpha_d] \qquad (8)$$

Here, $\cot\alpha_u$ and $\cot\alpha_d$ represent the slopes above and below the berm, respectively (Fig. 1). h_b represents the depth of still water level above the berm and is positive for submerged berms. L_s is the horizontal length between two points on slope, $R_{u2\%}$ above and $1.5H_{m0}$ below the SWL. Van der Meer et al. [2] suggests an iterative solution to determine $\tan\alpha_{av}$ since $R_{u2\%}$ is initially unknown. The drawback of the iterative calculation is that the reliability of the estimated $R_{u2\%}$ depends on the accuracy of the first estimation of $R_{u2\%}$.

The wave steepness, $s_{m\text{-}1,0}$ is given as:

$$s_{m\text{-}1,0} = \frac{2\pi H_{m0}}{g T_{m\text{-}1,0}^2} \qquad (9)$$

where g is the acceleration due to gravity, $T_{m\text{-}1,0}$ is the mean energy wave period or spectral wave period. It should be mentioned that Van der Meer et al. [2] has provided a similar formula for estimating the $R_{u2\%}$ at dikes. However, the $\gamma_{f\,surging}$ in Eq. (5) has been replaced with γ_f.

Influence factors were proposed to take into account the effects of roughness (γ_f). A γ_f of 0.4 was suggested for rubble mound structures with two layers of rock over a permeable core. However, for maximum run-up of surging waves, $\gamma_{f\,surging}$ was suggested as:

$$\gamma_{fsurging} = \gamma_f + \frac{(\xi_{m\text{-}1,0} - 1.8) \times (1 - \gamma_f)}{8.2} \quad \text{for } 1.8 \leq \xi_{m\text{-}1,0} \leq 10 \qquad (10)$$

It is worth mentioning that the same formula with different maximum values is suggested by Van der Meer et al. [2] for dikes and rubble mounds. The main difference is introducing $\gamma_{f\,surging}$ in Eq. (4) for armoured rubble mounds and slopes which replaces γ_f when $\xi_{m-1,0} > 1.8$, and hence increases the maximum value of R_* for surging and collapsing waves.

The influence factor for oblique wave incidence (γ_β) was considered as:

$$\gamma_\beta = \begin{array}{ll} 1\text{-}0.0063|\beta| & 0 \leq \beta \leq 80° \\ 0.496 & |\beta| > 80° \end{array} \tag{11}$$

where β is the angle of wave incidence. For bermed structures, Van der Meer et al. [2] suggested the following influence factor:

$$\gamma_b = 1 - r_B(1 - r_{hb}) \geq 0.6 \tag{12}$$

$$r_B = \frac{B}{L_{berm}} \tag{13}$$

where L_{berm} is the horizontal length between two points on slope, 1.0 H_{m0} above and 1.0 H_{m0} below the middle of the berm.

$$r_{hb} = \begin{cases} 0.5\text{-}0.5\cos\left(\pi\frac{h_b}{R_{u2\%}}\right) & \text{berm above SWL} \\ 0.5\text{-}0.5\cos\left(\pi\frac{h_b}{2H_{m0}}\right) & \text{berm below SWL} \\ 1 & \text{berm lying outside the area of influence} \end{cases} \tag{14}$$

It should be noted that similar to the estimation of average slope, $R_{u2\%}$ needs to be estimated first for emerged berms.

The influence factor for shallow foreshores was not included explicitly in Van der Meer et al. [2] mainly because $T_{m-1,0}$ was used in the formulations. However, Van der Meer and Janssen [9] suggested the following formulation for γ_h.

$$\gamma_h = \begin{cases} 1\text{-}0.03\left(4\text{-}\dfrac{h}{H_{m0}}\right)^2 & \text{for } \dfrac{h}{H_{m0}} < 4 \\[2ex] 1 & \text{for } \dfrac{h}{H_{m0}} \geq 4 \end{cases} \tag{15}$$

The formula to estimate run-up for berm structures has been developed mostly based on data from smooth dike studies [10]. Hence, the approach used for composite dikes is used for these structures [2]. However, the response of a rubble mound berm could be different from that of a smooth bermed slope ([11], [2]).

The main objectives of this study are to investigate run-up on bermed structures and develop a unified prediction formula based on a database which includes bermed dikes and berm breakwaters. Hence, first, small-scale experiments were conducted to record the run-up on berm breakwaters. Then, using the M5` algorithm (e.g. [12]), a design formula was developed for bermed structures. The performance of the developed formula was compared with those of the existing prediction formulas for the combined data sets of bermed breakwaters and dikes. Moreover, the developed formula was validated using measurements at prototype berm structure.

2. Lab Experiments and existing data

As discussed above, hardly any measurement of run-up on bermed rubble mound structures exists. Hence, laboratory experiments were undertaken in this study on berm breakwaters as the first part of this research. The experiments were conducted in the 18m long × 0.80m deep × 0.45m wide wave flume in the hydraulics laboratory of Griffith University, Australia. The piston wave maker installed in the flume is able to produce both regular and irregular waves and is equipped with an active wave absorption system. Irregular waves with a JONSWAP spectrum and a peak enhancement factor (γ) of 3.3 were used for all

tests. The tests were carried out on different rubble mound berm breakwater cross sections with a *cotα* of 1.50 above and below the berm (Fig. 2a). The model had a core with a nominal diameter (D_{n50}) of 0.021 m and D_{n85}/D_{n15} of 2, while the primary armour had a D_{n50} of 0.032 m.

The free surface of the water in the flume was measured and the wave parameters were estimated using three capacitance wave gauges placed near the toe of the structure (Fig. 2b). The gauges were placed at distances of $X_{12} = 0.50$ m and $X_{13} = 1.30$ m where X_{12} and X_{13} are the distances of the second and third gauges from the first, respectively. Incident and reflected waves were separated using the method of three fixed probes [13]. Experiments were performed by varying the SWL, h_b and the peak wave steepness ($s_{m-1,0}$). Wave run-ups were measured using images collected from digital video cameras for approximately 1000 waves for each test condition. A scaled ruler was fixed on the front slope above the berm to increase the accuracy of the run-up estimates. The video of each test case was individually analysed frame by frame and the top 2% value of R_u was reported as $R_{u2\%}$. The ranges of tested parameters are given in Table 1.

In addition, to have a more comprehensive data base, the smooth berm dataset was obtained from Van der Meer and De Waal [10]. The ranges of their parameters are summarised in Table 1. It can be seen from the table that rather wide berms with gentle smooth slopes were tested in their experimental program. However, rubble mound breakwaters in general have steeper slopes [2].

3. Results and discussion

Fig. 3a shows the scatter diagram of measured (R_{*meas}) and estimated run-ups (R_{*est}) using the modified Ahrens and Ward [6] formula. The runup data collected in this study is slightly underestimated by this formula, specially at high values of R_{*meas}, which could be due to the subjectivity in the run-up variable from visual observations ([6], [7]). However, it is interesting to note that all Van der Meer and De Waal [10] data points are overestimated by the modified Ahrens and Ward [6] formula. Fig. 3b presents the performance of the Van der Meer et al. [2] formula. It shows that there is an overestimation for the $R_{*meas} > 2.0$ for the Van der Meer and De Waal [10] data. The run-up measurements of this study are also overestimated. This could be due to the linear relationship between R_* and $\xi_{m-1,0}$. Interestingly, this formula performs better for R_{*meas} between 1.5 and 2.0.

To develop a new prediction model, seventy percent of the data, detailed in Table 1, was used as the training data and the remaining ones were used for testing the model. Dimensional analysis and M5` model tree technique [14] of WEKA software (see details in Appendix A) was used to develop a prediction formula to estimate the R_* (output). The governing variables such as $\xi_{m-1,0}$, γ_b were used as inputs. γ_b was calculated using Eq. (12). However, the r_{hb} was obtained from Eq. (14) by replacing $R_{u2\%}$ with 1.5 H_{m0}. Also, $\xi_{m-1,0}$ was obtained from Eq. (6), using the first estimate of $\tan\alpha_{av}$ in Eq. (7). Hence it does not need any iteration or second estimation. γ_f and γ_β were assumed as 0.4 and 1.0 (head on waves), respectively [2]. Table 2 summarises the ranges of the used parameters.

Different functional forms and governing dimensionless parameters were utilised and the optimum formula was obtained as:

$$R_* = \frac{R_{u2\%}}{H_{m0}} = 7.7\gamma_b\gamma_h\gamma_f\gamma_\beta[1\text{-}\exp(-0.78\xi_{m-1,0})] \qquad (16)$$

With a maximum of

$$\begin{cases} 1.0\gamma_f\gamma_\beta \left(3.15 - \dfrac{1.5}{\gamma_b\xi_{m-1,0}}\right) & \text{for impermeable dikes} \\ 1.0\gamma_{fsurging}\gamma_\beta \left(3.15 - \dfrac{1.5}{\gamma_b\xi_{m-1,0}}\right) & \text{for rubble mounds} \end{cases} \qquad (17)$$

As seen, compared to Van der Meer et al. [2] formulas, a unified exponential formula which includes the water depth effect has been developed. As before, the surf similarity parameter is the most influential parameter in the estimation of run-up. This parameter includes the influence of $\tan\alpha$, H_{m0} and $T_{m-1,0}$ and its physical significance is well known, and the exponential functional form of the developed formula is justifiable since the sensitivity of R_* to $\xi_{m-1,0}$ decreases for surging waves ([15], [2]). The effects of other parameters, such as berm width and height, water depth and friction, are also considered in the developed formula. Eq. (16) includes the effect of water depth which is required in predicting the influence of sea level rise and consequent changes of wave height to water depth ratio on the future hydraulic performance.

Fig. 3c compares the R_{*meas} against the R_{*est} using the Eq. (16). It can be seen from the figure that the scatter is reduced, and the data points are closer to the optimal line, when compared to Figs. 3a and 3b. In addition, higher values of R_{*meas} (>2) are better estimated by the developed formula. Furthermore, the performances of different formulas were quantitatively compared using the following accuracy measures:

$$RMSE = \sqrt{\frac{1}{n}\sum_{i=1}^{i=n}\left(R_{*\,est} - R_{*\,meas}\right)^2} \qquad (18)$$

$$DR = \frac{1}{n}\sum_{i=1}^{i=n}\left(\frac{R_{*\,est}}{R_{*\,meas}}\right) \qquad (19)$$

$$Bias = \frac{1}{n}\sum_{i=1}^{i=n}\left(R_{*\,est} - R_{*\,meas}\right) \qquad (20)$$

where n denotes the number of measurements. Table 3 summarises the performances of the different prediction formulas. The DR value of the modified Ahrens and Ward [6] formula is greater than 1, indicating overestimation (as is presented in Fig.3a). $RMSE$, DR and $Bias$ of the new formula are improved, even for the test data, when compared to the Van der Meer et al. [2] formula.

Due to the complex nature of the run-up process, the estimated run-up using the developed formula has uncertainties. In order to consider these uncertainties, the semi-probabilistic approach, with a partial safety factor [2], was considered and the formula was modified as follows:

$$R_* = \frac{R_{u2\%}}{H_{m0}} = 7.7\ (1+0.24N)\gamma_b\gamma_h\gamma_f\gamma_\beta[1-\exp(-0.78\xi_{m-1,0})] \tag{21}$$

With a maximum of

$$\begin{cases} (1+0.24N)\gamma_f\gamma_\beta\left(3.15-\dfrac{1.5}{\gamma_b\xi_{m-1,0}}\right) & \text{for impermeable dikes} \\[3mm] (1+0.24N)\gamma_{fsurging}\gamma_\beta\left(3.15-\dfrac{1.5}{\gamma_b\xi_{m-1,0}}\right) & \text{for rubble mounds} \end{cases} \tag{22}$$

Assuming a Gaussian residual distribution, values of N can be obtained from Table 4. For example, when the acceptable risk is 2%, an N value of 2.05 needs to be used. One of the limitations of Eq. (16) is that it is developed from experimental data with head on waves. Hence, the estimation of γ_β suggested by Van der Meer et al. [2] is suggested. The effects of wind and currents are not considered, and they can be modelled as influencing factors as suitable data becomes available.

4. Application to the field data

Results from applying different formulas are compared with the field measurements on Petten Sea defence in The Netherlands [16]. It should be mentioned that the local depth contours are nearly parallel to the coast and therefore the wave obliquity is negligible. The dike has a 14 m wide berm and upper and lower smooth slopes of 1:3 and 1:4.5, respectively. In the field, run-up measurements were obtained in the upper slope using step gauges. The average wave height was 2.8 m and $\xi_{m-1,0}$ varied from 1.91 to 2.56 (Table 5). Fig. 4 compares the estimated dimensionless run-up rates with those measured in the field. Even though 3 D field measurements and 2 D lab experiments could produce different results, this comparison can give some insight about the performance of different formulas, especially for head on wave condition. It can be seen that the measurements are mostly overestimated by the Van der Meer et al. [2] and modified Ahrens and Ward [6] formulas. Results obtained from the small scale experiments (Fig. 3) also show that for cases with high values of R_* ($1.8 < R_*$) the formula of Van der Meer et al. [2] generally overestimates the observed run up levels by about 25%. The overestimation is not likely due to the wave directionality since the reported waves were normal to the coast [16]. It is plausible that the overestimation is due to the inherent monotonic relationship between R^* and $\xi_{m-1,0}$ in their formula which does not exist in other formulas.

The performance of different prediction formulas in the field are quantitatively compared in Table 5. As seen, the developed formula performs well for the field data with improved *RMSE* and *DR* compared to those of the existing prediction formulas. It is not surprising that the performances of all formulas are inferior in the field mainly due to modelling and scaling effects ([17], [18]).

In order to help practicing engineers, an example using design condition of Sirevag berm breakwater is given in Appendix B and the results are compared with those of other formulas.

5. Summary and Conclusion

In this study, a unified prediction formula for the bermed slope structures was developed for bermed dikes and berm breakwaters. It was found that the existing measured run-up data on berm breakwaters, for model development and validation purposes, was highly limited and therefore physical model tests were conducted on different berm breakwater cross sections to help correct this deficiency.

The dominant input parameter in the developed model is the surf similarity parameter while the effects of berm, roughness, and water depth were influencing factors. The formula was validated using the measured run-up rates from prototype dike structure at Petten Sea defence in The Netherlands. It was shown that the introduced formula is more precise than the previous ones in estimating the full-scale run-up. The model also considers uncertainties to make it an appropriate design tool for engineers.

List of symbols

α_{av}	Average slope excluding the berm (°)
α_u	Structure slope above the berm (°)
α_d	Structure slope below the berm (°)
β	Angle of wave incidence (°)
$\xi_{m-1,0}$	Surf similarity parameter based on $T_{m-1,0}$
γ	Peak enhancement factor
γ_β	Influence factor for oblique wave incidence
γ_b	Influence factor for a berm
γ_f	Influence factor for roughness
$\gamma_{fsurging}$	Influence factor for roughness, surging waves
γ_h	Influence factor for shallow foreshore
B	Berm width (m)
$B^{'}$	Dimensionless berm width
D_{n15}	Equivalent cube length exceeded by 15% of a sample by weight (m)
D_{n50}	Nominal diameter (m)
D_{n85}	Equivalent cube length exceeded by 85% of a sample by weight (m)
DR	Discrepancy Ratio
g	Acceleration due to gravity = 9.8 m/s^2

h	Water depth (m)
h_b	Berm level above the still water level ($-h_b$ for emerged berm) (m)
h_{bt}	Height of the berm above the toe of the structure (m)
H_{m0}	Significant wave height from spectral analysis (m)
L_{berm}	Horizontal length between two points on slope, 1.0 H_{m0} above and 1.0 H_{m0} below middle of the berm (m)
$L_{m-1,0}$	Deep water peak wave length using $T_{m-1,0}$ (m)
L_{0p}	Deep water peak wave length (m)
L_s	Horizontal length between two points on slope, $R_{u2\%}$ above and 1.5 H_{m0} below SWL (m)
n	Number of measurements
N	Coefficient for various levels of acceptable risk
R_*	Dimensionless $R_{u2\%}$ ($R_{u2\%}/H_{m0}$)
R_{*est}	Estimated dimensionless $R_{u2\%}$
R_{*m}	Maximum dimensionless runup (R_{max}/H_{m0})
R_{*meas}	Measured dimensionless $R_{u2\%}$
r_B	Reduction factor for size of berm
R_c	Crest freeboard of structure (m)
r_{hb}	Reduction factor for level of berm with respect to SWL

R_u Run-up height, measured vertical with respect to the SWL (m)

$R_{u2\%}$ Run-up height exceeded by 2% of the incident waves (m)

R_{um} Maximum run-up height of all waves in a sea state (m)

$R_{u2\%\_meas}$ Measured run-up height exceeded by 2% of the incident waves (m)

$s_{m-1,0}$ Wave steepness based on $T_{m-1,0}$, $= H_{m0}/L_{m-1,0} = 2\pi H_{m0}/(gT_{m-1,0}^2)$

SWL Still Water level (m)

$T_{m-1,0}$ Spectral wave period defined by m_{-1}/m_0 (s)

X_{12} Distance between first and second gauge (m)

X_{13} Distance between first and third gauge (m)

List of tables

Table 1. Ranges of parameters in the existing and new datasets of bermed slopes

Parameter	Van der Meer and De Waal [10]	This study	Total
R_c (m)	-	0.08 – 0.18	0.08 – 0.18
B (m)	0.40 – 1.00	0.25 – 0.45	0.25 – 1.00
h_b (m)	-0.08 – 0.16	-0.08 – 0.05	-0.08 – 0.16
h (m)	0.52 – 0.76	0.38 – 0.48	0.38 – 0.76
H_{m0} (m)	0.10 – 0.20	0.06 – 0.16	0.06 – 0.20
$T_{m-1,0}$ (s)	1.49 – 2.56	1.00 – 1.76	1.00 – 2.56
cotα	3.00 – 4.00	1.50	1.50 – 4.00
$s_{m-1,0}$	0.01 – 0.05	0.02 – 0.07	0.01- 0.07
$\xi_{m-1,0}$	0.99 – 4.46	2.15 – 5.73	0.99 – 5.73
γ_b	0.60 – 0.82	0.61 – 0.78	0.60 – 0.82
γ_h	1.00	0.77 – 1.00	0.77 – 1.00
$R_{u2\%}$ (m)	0.15 – 0.39	0.06 – 0.17	0.06 – 0.39
$R_{u2\%}/H_{m0}$	1.21 – 2.96	0.73 – 1.82	0.73 – 2.96
Structure type	Bermed dike	Berm breakwater	Both
Number of data	75	59	134

Table 2. Ranges of input-output parameters in the berm structures training and test data

Parameter	Train data	Test data
$\tan\alpha_{average}$	0.22 – 0.78	0.26 – 0.66
$s_{m-1,0}$	0.01 – 0.07	0.02 – 0.05
γ_b	0.60 – 0.90	0.61 – 0.81
γ_h	0.75 – 1.00	0.92 – 1.00
γ_f	0.40, 1.00	0.40, 1.00
$\gamma_{f\,surging}$	0.43 – 0.69	0.49 – 0.54
$\xi_{m-1,0}$	0.99 – 5.73	1.32 – 3.72
R_*	0.73 – 2.96	0.81 – 1.81
Total	95	39

Table 3. Accuracy measures of different prediction models

Data	Modified Ahrens and Ward [6]			Van der Meer et al. [2]			This study		
	RMSE	DR	Bias	RMSE	DR	Bias	RMSE	DR	Bias
Test	0.63	1.25	0.41	0.28	1.19	0.22	0.20	1.15	0.15
All	0.62	1.20	0.68	0.36	1.16	0.24	0.21	1.06	0.03

Table 4. *N* values for various levels of acceptable risk [19]

Acceptable risk (%)	*N*
2	2.05
5	1.65
10	1.28
33	0.44
50	0.00

Table 5. Comparison of R_{*meas} with R_{*est} using different prediction formulas, field data

Storm number	$\xi_{m-1,0}$	R_{*meas}	Estimations		
			Modified Ahrens and Ward [6]	Van der Meer et al. [2]	This study
1.01	1.91	2.12	2.59	2.59	1.91
1.02	1.94	1.99	2.58	2.61	1.94
1.03	2.44	2.17	3.03	2.70	2.04
1.04	2.56	1.99	2.50	2.75	2.11
1.05	2.28	1.86	2.63	2.70	2.04
1.06	2.06	2.04	2.14	2.64	1.97
	RMSE		0.60	0.65	0.14
	DR		1.27	1.32	0.99
	Bias		0.55	0.64	-0.03

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Fig 1. Run-up on bermed slopes

$B = 0.25$ to 0.45

$h_b = -0.08$ to $+0.05$

$s_{m-1,0} = 0.02$ to 0.07

SWL

Core

Armour

$cot\alpha$ 1:1.5

$h = 0.38$ to 0.48

(a)

Wave gauges

X3   X2   X1

Wave paddle
arrangement

Wave absorber

Model

0.80   0.50   9.00

(c)

(All dimensions in metres)

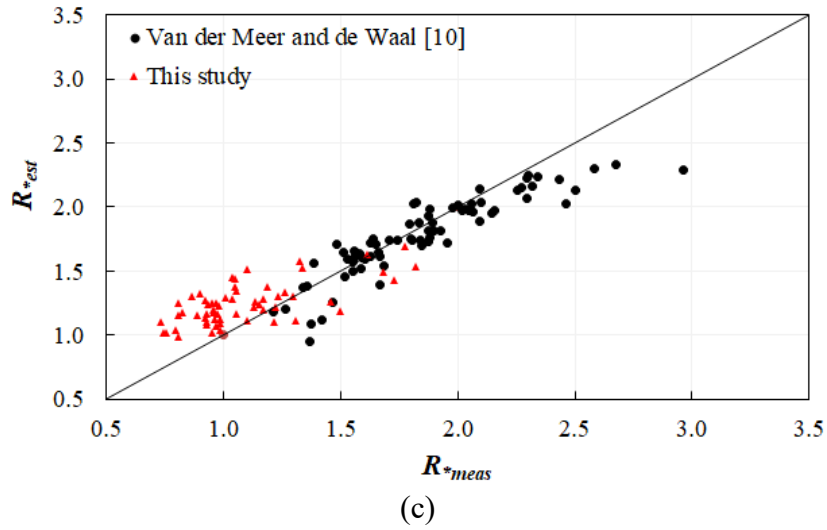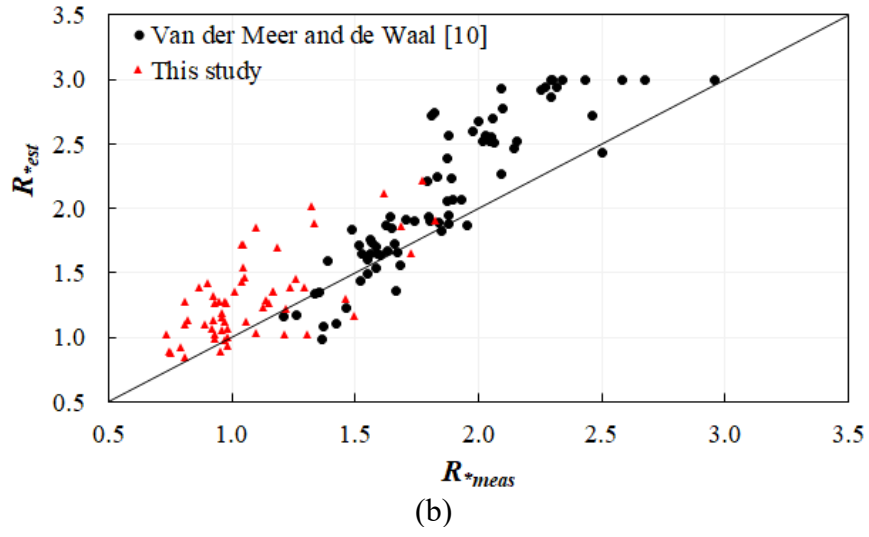Fig.2 (a) Schematic cross section of the tested model and b) Experimental set-up
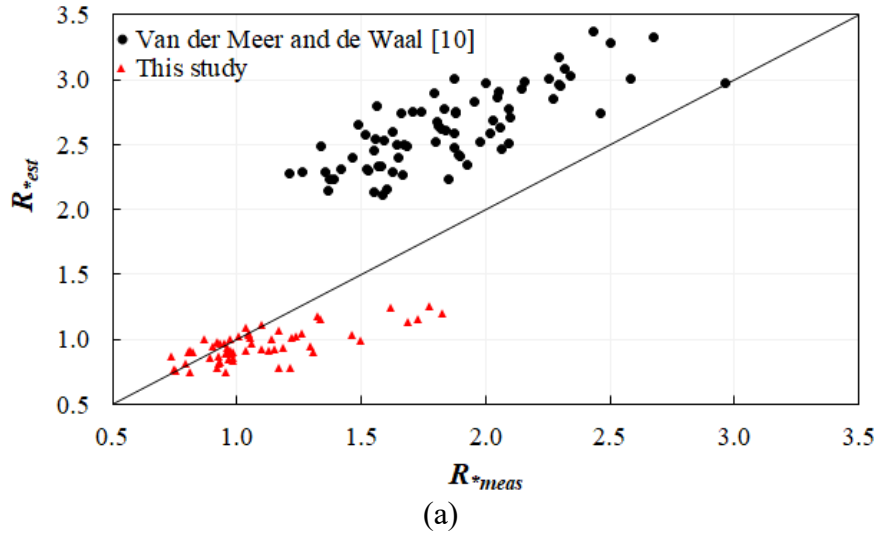
Fig 3. Comparison between R_{*meas} and R_{*est} using the formula of (a) Modified Ahrens and Ward [6] (b) Van der Meer et al. [2] and c) This study. The solid lines are the optimal line (45 degree).
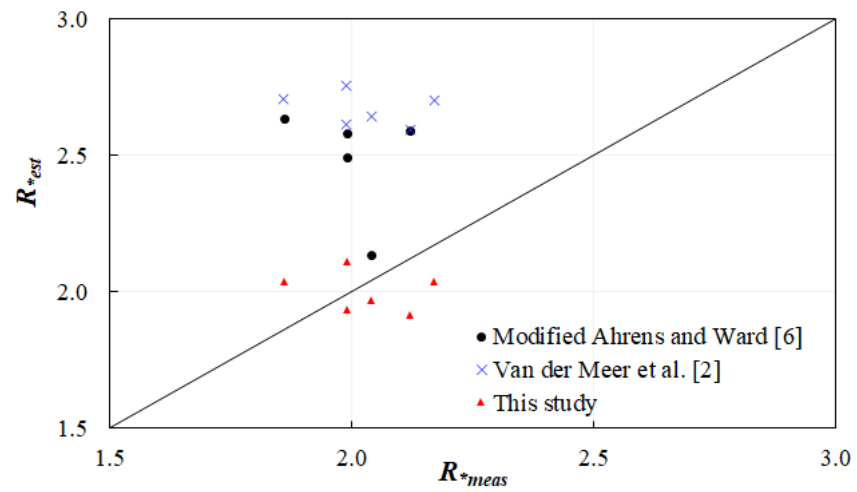
Fig 4. Comparison between R_{*meas} and R_{*est} using different formulas, field data. The solid line is the optimal line (45 degree).

Appendix A – M5

M5 model tree is a data mining technique implemented through the WEKA software [20]. M5 has been used in coastal engineering to predict the armour stability [21] and overtopping discharge [22] at rubble mound breakwaters, wave height prediction in Lake Superior [23], to develop the wave spectrum at the Gulf of Maine: USA and Bay of Bengal: India [24], estimation of wind speed from wave measurements [25], to develop an improved overtopping prediction model for inclined coastal structures with smooth impermeable surface [26] and vertical structures [27]. The details of M5 technique can be obtained from Quinlan [28] and Wang and Witten [14], the excerpt of which are enumerated below.

M5 is the algorithm used to develop the model tree and it has three stages 1) building the tree 2) pruning and 3) smoothing which leads to prediction of introduced data. The MT developed by M5 algorithm is similar to an inverted tree where the data is fed into the root and the related prediction of the introduced data is obtained from leaves. M5 algorithm splits the input data recursively into subsets. The splitting is monitored in such a way that the Standard Deviation Reduction (SDR) factor (Eq. A.1) is minimum between subsets. The SDR is estimated for every sub-space obtained as a result of the splitting process and it is compared with the data space before splitting. The splitting process is stopped when the SDR is less than 5%, or only a few data remain. A linear regression model is then developed for the data associated with each leaf.

$$\text{SDR} = \frac{M}{|T|} \, \text{sd}(T) - \sum \frac{|T_i|}{|T|} \times \text{sd}(T_i) \tag{A.1}$$

In the pruning stage, the developed linear model is multiplied with a factor $(n + v) / (n - v)$ to minimise the errors due to overfitting due to growth of the model tree. Here, n is the number of training data points that reach the node and v is the number of input parameters in the model that represent the output value at that node. The discontinuities between adjacent

linear models (pruned leaves) are regularised in the smoothing stage. Thus, the M5 algorithm

produces a set of linear multi-variable equations.

Appendix B – Worked example for estimation of wave run up

In this section, an example for the estimation of wave run up using the developed formula (Eq. 17) for the design conditions of Sirevag berm breakwater [29] is presented. The design conditions and calculation details for the estimation of $R_{u2\%}/H_{m0}$ using the formula developed in this study are listed below.

Design conditions: $H_{m0} = 7.0$ m, $T_{m-1,0} = 9.64$ s, $h = 17.5$ m, $\cot\alpha = 1.5$, $B = 19.7$ m and $h_b = -6.4$ m.

$s_{m-1,0} = \dfrac{2\pi H_{m0}}{gT_{m-1,0}^2}$: $s_{0m-1,0} = 0.048$ (Eq. 9)

$L_{berm} = B+(1H_{m0}+h_b)\cot\alpha_u+(1H_{m0}-h_b)\cot\alpha_d = 40.7$ m

$r_B = \dfrac{B}{L_{berm}}$: $r_B = 0.48$ (Eq. 13)

$r_{hb} = 0.5-0.5\cos\left(\pi\dfrac{h_b}{R_{u2\%}}\right)$: $r_{hb} = 0.67$ (Eq. 14, Here $R_{u2\%}$ is assumed as 1.5 H_{m0})

$\gamma_b = 1- r_B(1-r_{hb}) \geq 0.6$: $\gamma_b = 0.84$ (Eq. 12)

$\gamma_h = 1-0.03\left(4-\dfrac{h}{H_{m0}}\right)^2$: $\gamma_h = 0.93$ (Eq. 15)

Berm breakwater and head on waves: $\gamma_f = 0.40$ and $\gamma_\beta = 1.0$

$L_s = B+[(1.5H_{m0}+h_b)\cot\alpha_u]+[(1.5H_{m0}-h_b)\cot\alpha_d]$: $L_s = 51.2$ m (Eq. 8)

$\tan\alpha_{av} = \dfrac{3H_{m0}}{L_s-B}$: $\tan_{\alpha av} = 0.67$ (Eq. 7)

$\xi_{m-1,0} = \dfrac{\tan\alpha_{av}}{\sqrt{s_{m-1,0}}}$: $\xi_{m-1,0} = 3.033$ (Eq. 6)

$$\gamma_{fsurging} = \gamma_f + \frac{(\xi_{m-1,0}-1.8)\times(1-\gamma_f)}{8.2} \quad \text{for } 1.8 \leq \xi_{m-1,0} \leq 10: \; \gamma_{f\,surging} = 0.49 \; \text{(Eq. 10)}$$

$$1.0\gamma_{fsurging}\gamma_\beta\left(3.15 - \frac{1.5}{\gamma_b\xi_{m-1,0}}\right): \text{Maximum } R_* = 1.26 \; \text{(Eq. 17)}$$

$$R_* = \frac{R_{u2\%}}{H_{m0}} = 7.7\gamma_b\gamma_h\gamma_f\gamma_\beta\left[1-\exp\left(-0.78\xi_{m-1,0}\right)\right] < 1.26: \; R_* = 1.26 \; \text{(Eq. 16)}$$

$R_{u2\%} = 1.26\times7 = 8.82$ m

(Using the Van der Meer et al. [2] formula (Eqs. 4 and 5) and the modified Ahrens and Ward [6] formula (Eq. 3), the R_* values are 1.68 and 1.13, respectively.)

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