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Wave run-up on slender piles in design conditions — Model tests and design rules for offshore wind

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ABSTRACT

Wave run-up on foundations is a very important factor in the design of entrance platforms for offshore wind turbines. When the Horns Reef 1 wind turbine park in Denmark was designed the vertical wave run-up phenomenon was not well known in the industry, hence not sufficiently considered in the design of Horns Reef 1. As a consequence damage was observed on the platforms. This has been the situation for several sites and design tools for platform loads are lacking. As a consequence a physical model test study was initiated at Aalborg University to clarify wave run-up on cylindrical piles for different values of diameter to water depth ratios (D/h) and different wave heights to water depth ratios (H/h) for both regular and irregular waves. A calculation model is calibrated based on stream function theory for crest kinematics and velocity head stagnation theory. Due to increased velocities close to the pile an empirical factor is included on the velocity head. The evaluation of the calculation model shows that an accurate design rule can be established even in breaking wave conditions. However, calibration of a load model showed that it was necessary to increase the run-up factor on the velocity head by 40% to take into account the underestimation of run-up for breaking or nearly breaking waves given that they produce thin run-up wedges and air entrainment, two factors not coped with by the measurement system.

1. Introduction

To provide access to an offshore wind turbine from the sea a platform is needed. For safety reasons the platform height is limited in order to provide safe access from a service vessel. If the platform level is too high typically smaller platforms would be required further down to meet safety requirements. Therefore, run-up generated loads cannot be prevented and can be very significant in many cases.

Fig. 1 shows wave run-up on a foundation in the Horns Reef 1 wind turbine park in Denmark for significant wave height of approximately 2.5 m, i.e. approximately half of the design situation. In this case damage was observed on the platforms which at that time consisted of grates supported by beams. Therefore, a repair project was initiated with recalculation of loads and redesigning the platforms.

The present study was carried out not only as part of this project, but also for the Horns Reef 2 park which was under design. For that park much more focus was put into run-up generated loads due to the problems observed at Horns Reef 1. The present investigation was thus carried out in order to solve the problem of determining run-up generated loads on entrance platforms. The idea was to determine the impact pressures and loads in a three step procedure:

1) Calculate the expected maximum wave run-up height with no platform.
2) Use this run-up height to calculate the velocity at the level of the platform.
3) Use a slamming force model to get the maximum pressures and loads.

The present paper deals with model tests performed at Aalborg University to investigate steps 1 and 2. The report of Andersen and Brorsen (2006) deals with the model tests performed for the third step.

2. State of the art in wave run-up on circular cylinders

De Vos et al. (2007) gives a recent review on the previous investigations on wave run-up on cylinders including velocity stagnation models and diffraction models and will not be repeated here and consequently only the latest additions will be mentioned.

However, it might be useful to consider a vertical wall corresponding to an infinite large diameter (D), i.e. water depth to pile diameter ratio of zero ($h/D=0$). In such a situation a standing

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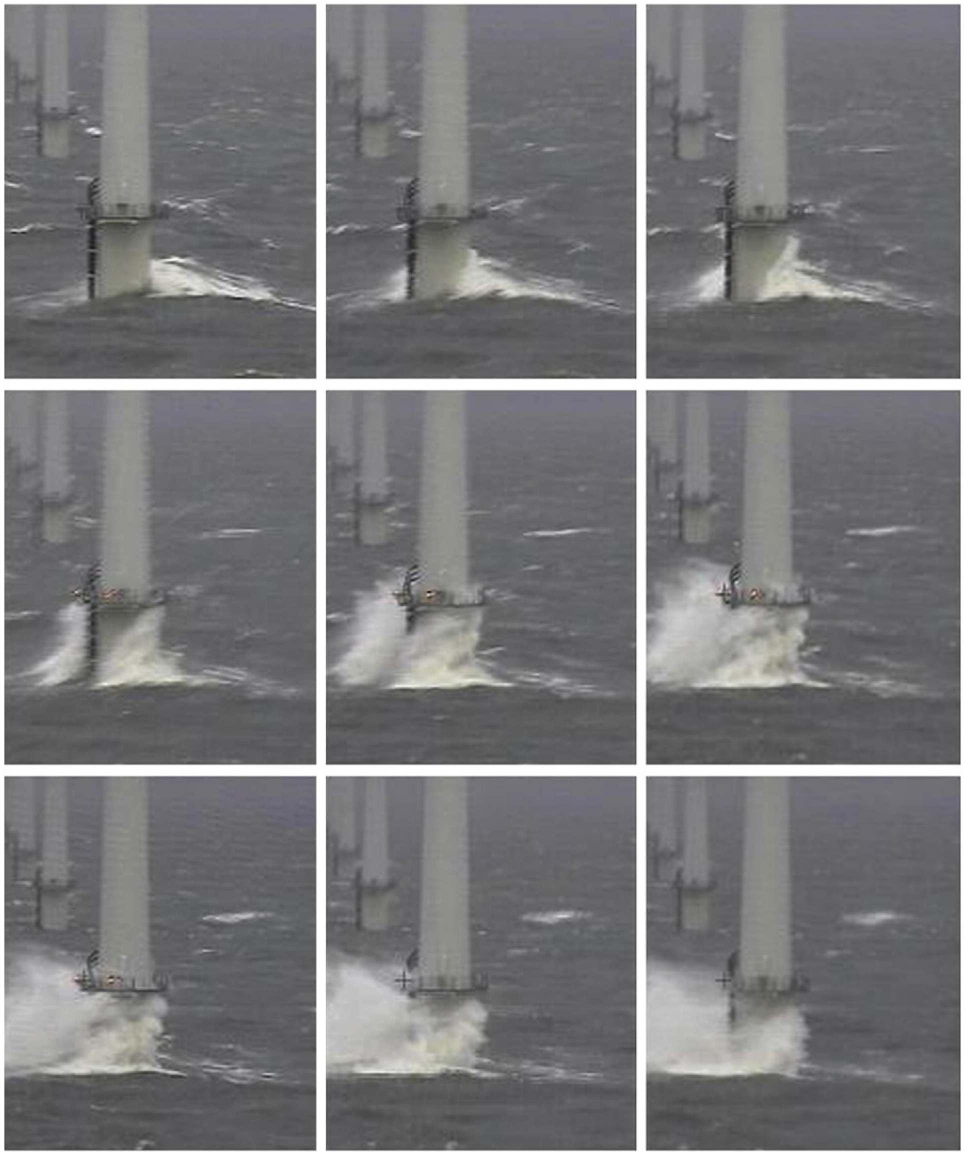

Fig. 1. Wave run-up at Horns Reef 1 for $H_s \approx 2.5$ m, while the platform level is 9.0 m above MWL. $H_{s,design} = 5.3$ m.

wave is developed in front of the structure. This means that wave height and vertical particle velocity at the wall are amplified compared to incident values. In case of pulsating small amplitude waves we get that the run-up height is equal to the wave height. Sainflou (1928) used finite amplitude wave theory (trochoidal waves) and gave a formula to calculate the upper level for wave pressures on a vertical wall and valid for a pulsating finite amplitude shallow water waves. The Sainflou run-up formula reads:

$$R_u = H + \frac{\pi H^2}{L} \coth\left(\frac{2\pi h}{L}\right) \tag{1}$$

where R_u is the run-up height (vertical distance from SWL), H is the wave height and L the wave length. The first term is the linear solution and the last term is the second order contribution to take into account the asymmetry of the standing wave. Run-up on a vertical wall has also been investigated by Miche (1944) for finite water depths. The formula of Miche is identical to the Sainflou formula on deep water, while the run-up height is increasing with decreasing water depth. However, the Miche formula is not valid on really shallow waters as it predicts infinite run-up heights.

It is also well known that the run-up on a vertical wall structure is significantly different for pulsating waves as described earlier and for impacting waves where significantly larger run-ups are observed. For waves breaking onto the structure the run-up is characterised by a high run-up with the entrainment of air. For waves that are close to breaking but not directly breaking onto the structure (flip-through phenomenon) a high run-up is also observed but with much less air entrained than the breaking conditions. In these conditions a term depending on the crest velocities needs to be taken into account. This term can be based on the particle velocity in the top part of the crest (u). When dealing with plunging breakers it can alternatively be based on the wave celerity (C) as used by Wienke and Oumeraci (2005) and others for calculation of slamming loads on piles.

Run-up on circular cylinders has a lot in common with that for the vertical wall. However, the standing wave is much less significant for slender piles due to diffraction of reflected energy. Therefore, surface elevation amplifications are much smaller. However, close to the cylinder the velocities are increased just as for the reflective wall and the incident waves feel the back pressure generated by the cylinder. On the front of the cylinder large vertical velocities are generated due to the presence of the pile similar to the vertical wall but in pulsating waves less significant for the pile compared to the wall. Moreover, a potential flow calculation of a stationary flow around a circular cylinder gives that the horizontal velocity doubles on the side of the cylinder. These items form the basis for including a factor (m) on the velocity head in the stagnation theory. If the wave kinematic at the pile were correctly calculated including these effects then $m=1$ must be valid. However, if undisturbed wave kinematic are used then $m>1$ must be expected. This procedure was used by De Vos et al. (2007) and others using the following type of formula for prediction of run-up levels exceeded by 2% of the waves ($R_{u,2\%}$):

$$R_{u,2\%} = \eta_{max,2\%} + m \cdot \frac{u_{2\%}^2}{2g} \tag{2}$$

where $\eta_{max,2\%}$ is the crest level of the 2% highest wave, $u_{2\%}$ is the horizontal particle velocity in the top of the crest for the same wave and g is the gravity acceleration. Both are by De Vos et al. (2007) calculated from the 2nd order Stoke theory for which they give $m=2.71$ as the mean value for a monopole, but they do not take into account the actual scatter of the m values obtained. The m factor makes it also possible to describe run-up for waves with a steep, almost vertical, front as well as breaking waves, where run-ups have been observed to be much higher than for pulsating waves, just as in the vertical wall case (higher m value).

Gravesen (2006) performed a rough reanalysis of the data from De Vos et al. (2007) by considering one of the graphs in their article. These preliminary investigations indicated a lot of scatter on the m factor and a strong increase in m with significant wave height to water depth ratio (H_s/h) which might be explained by impulsive waves. The data of De Vos et al. (2007) correspond to $H_s/h<0.42$ in all cases and therefore the depth limited situation was not completely reached. The data of De Vos also indicated the influence of h/D which could not be further quantified due to limited data.

Myrhaug and Holmedal (2010) used the data from De Vos et al. (2007) to compare with the 2nd order theory for irregular waves including the sum–frequency components and stagnation head theory with $m=1$. They found a fair agreement with the data from De Vos, but the model is generally underestimating wave run-up and can thus not be used for the design of platforms.

3. Motivation for new study

In the present study the original data of De Vos et al. (2007) was available and reanalyzed. It is found that m varies between 1.9 and 4.2 when the procedure used by De Vos is used for the kinematics in the crest. The present study uses Dean's stream function theory as this method more accurately calculates the kinematics in the crest especially in case of shallow water. In the stream function theory it is assumed regular waves that are symmetric around the crest and with constant form. This type of stream function theory has the advantage that it is easily available and commonly used for predicting kinematics for extreme loads on the pile. An example of a freely available implementation is that which is included in the WaveLab software package (Aalborg University, 2010). In prototype waves are irregular where an approximate stream function theory is available under the assumption of constant form (Sobey, 1992). However, implementations of this theory are not easily available and thus more difficult to use for design. Moreover, the method needs a surface elevation time series as input which is generally not available for design situations.

Using the stream function theory on the data of De Vos et al. (2007) it gives less scatter and on average an increase in the m values, as they lie between 2.7 and 4.9 compared to the above mentioned values. This increase in m factor is due to the 2nd order stoke theory used in the crest ($z>0$) which leads to overpredictions of velocities. Furthermore, in some conditions the 2nd order Stoke theory is not valid due to steep waves on shallow water and thus predicting the crest to be too high due to a bump in the wave trough. Fig. 2 shows the m values found by reanalyzing the data of De Vos et al. (2007) as a function of h/D and H_s/h. An increase in m can be seen with increasing wave height to water depth ratio just as found by Gravesen (2006). Therefore, it was decided to systematically investigate the influence of these two parameters up to depth limited conditions by performing additional physical model tests.

4. Model test set-up

The shallow water wave flume at the Dept. of Civil Engineering, Aalborg University has been used for the present tests. The flume configuration is shown in Fig. 3 and explained in the following.

The bottom was horizontal on the first 6.5 m then a 3.5 cm step followed by a 1:98 slope with a length of 9 m. The last part of the flume was horizontal and the model was placed 1.5 m into this horizontal part. The water depth at the wave maker was 12.5 cm larger than at the model.

The bases for the model tests are the Horns Reef 1 and 2 locations, where a relatively flat bed and a wide spectrum are found which is the reason for the above mentioned bottom configuration. Reproduction of the actual sea bed slope is important as a steeper bed might generate higher extreme waves and more plunging waves both leading to higher run-ups. In the tests it was observed that even for this relatively flat bed some waves were plunging and also some waves were hitting the pile with a very steep front even though the Irribarren number seems to indicate spilling breakers.

An absorbing rubble mound beach with a slope of 1:4 to 1:5 was created in the end of the flume to absorb the main part of the incident energy. The waves were measured both at the location of the model and 1.7 m from the paddle. For the wave calibration tests the model was removed and the wave gauges were placed at the location of the pile (in the center of the flume) with the middle wave gauge placed at the center of the pile. For the run-up tests the wave gauges were moved next to the model, but still with the middle wave gauge at the centerline of the pile, cf. Fig. 3.

Wave run-up was measured using a run-up model similar to that used by De Vos et al. (2007). Resistance type water surface gauges were attached to the model. These gauges consist of 2 wires with a diameter of 1 mm, placed approximately 2 mm from the surface of the cylinder and 7 mm between the centers of the two wires. Five pairs of wires were placed for measuring the run-up height at 0, 22.5, 45, 67.5 and 90° from the front of the pile, cf. Figs. 4 and 5. It was chosen to focus on the front part of the pile as the highest run-up occurs here

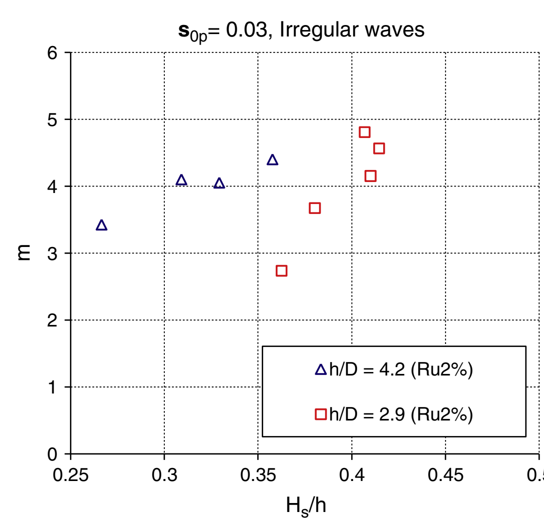

Fig. 2. Run-up velocity head factor (m) for the results from De Vos et al. (2007). Data reanalyzed to use stream function theory. JONSWAP spectrum with $\gamma \approx 3.3$.

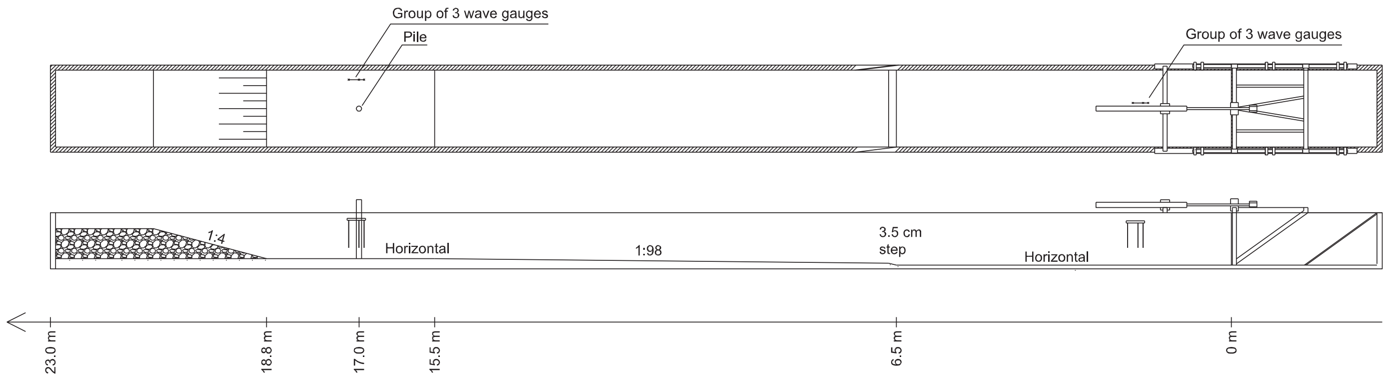

Fig. 3. Layout in flume.

and limited influence of H_s/h is believed on the distribution on the rear side, i.e. results from De Vos et al. (2007) are assumed trustworthy. The wires were pre-stressed using the system shown on the picture in the right hand side of Fig. 4. Even though the gauges placed in 0 and 22.5° are very close to each other it was found that the interaction between the two was small. Between the other gauges there was no interaction. Both the wave gauges and the run-up gauges were calibrated by filling the flume with water. This procedure was chosen due to non-linearities of the very long gauges. Because the conductivity depends on the water temperature the flume was filled with cold water each day and the gauges were recalibrated if necessary.

5. Test programme

The purpose of the wave calibration tests was to match the sea state at the location of the pile to the prespecified sea states. The wave heights to water depth ratios (H_{m0}/h) were from 0.35 to 0.50 which was considered the most relevant range for the present tests. However, $H_{m0}/h = 0.50$ was impossible to generate in the flume for this flat bottom configuration. This was due to waves breaking just in front of the paddle caused by limited water depth and due to wave breaking on the foreshore. Therefore, the initial test programme was modified so the four tested values of H_{m0}/h were 0.35, 0.40, 0.43 and 0.46. Three water depths $h = 0.20$ m, 0.30 m and 0.40 m were tested giving 12 combinations of H_{m0}/h and h/D, cf. Table 1. This was done for two peak wave steepnesses giving a total of 24 tests. The wave steepnesses chosen were $s_{0p} = H_{m0}/(g/2\pi \cdot T_p^2) = 0.02$ and 0.035, i.e. calculated using deep water equation but using parameters at the pile. The wave spectrum generated was JONSWAP (ISO19901) with a peak enhancement factor (γ) of 1.5. The length of each test corresponded to 1000 waves.

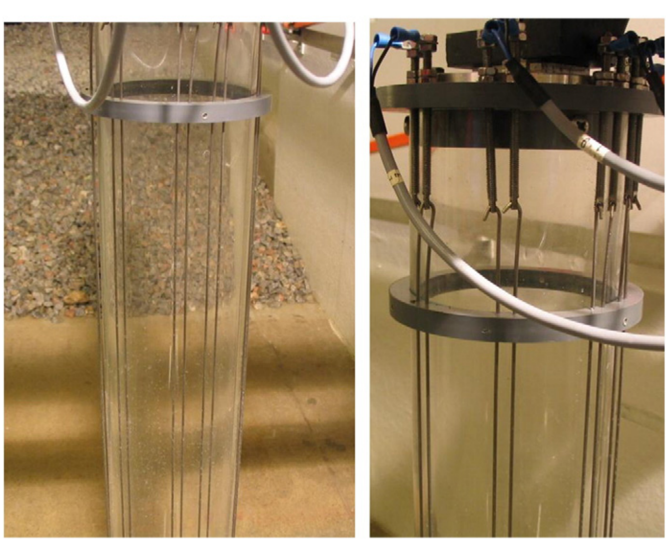

Fig. 4. Pictures of the run-up model.

The target sea states were reproduced at the pile verifying that both incident H_{m0} wave height and peak period (T_p) were correct. The entire wave spectrum shape was not reproduced due to shoaling and breaking on the foreshore. The same wave train could then be reproduced as the steering signal sent to the paddle was stored. In case of non-breaking waves ($H_{m0}/h = 0.35$) the peak period and the entire spectrum shape were both close to unchanged. However, in case of breaking waves the spectrum becomes wider and in the present case corresponds approximately to $\gamma = 1.0$ at the pile instead of the generated $\gamma = 1.5$.

The test with $H_{m0} = 0.184$ m ($H_{m0}/h = 0.46$ and $h = 0.4$ m) was impossible to generate due to heavy breaking both on the paddle and on the foreshore. In this case the wave height at the structure does not increase for a larger generated wave height. It was possible to generate $H_{m0}/h = 0.46$ for the water depths $h = 0.20$ m and 0.30 m.

In addition to the irregular tests the same number of regular wave sea states was tested, but the focus in the present paper lies on the irregular waves. The results from the regular wave tests are available in the study of Lykke Andersen and Frigaard (2006).

6. Data analysis

To minimize the influence of high frequent noise, an analog low-pass filter with a cut-off frequency of 8 Hz was applied to the wave data. The sample frequency was chosen to 20 Hz.

The incident wave spectrum and wave trains were determined by the WaveLab2 software package by Aalborg University (2010) utilizing the Mansard and Funke (1980) method. This linear method was used even though the generated waves in most cases are very non-linear and even breaking in some cases. The lower frequency boundary for the reflection analysis was set to the maximum of 0.1 Hz in model scale and 1/3 times the peak frequency (f_p). The upper boundary was $3 \times f_p$. The number of data points in each FFT block was 512 with 20% tapering in each end and 20% overlap of the subseries. Wave reflection coefficients between 9% and 33% have been calculated for the beach.

First order wave generation was applied. Some low frequency energy was observed to be present in the wave spectra which are expected to be due to bounded and free long waves that trigger the eigenmode of the flume (Sand, 1982). This is not taken into account in the wave analysis as this is mainly outside the band from 1/3 to 3 times f_p. Instead the low frequent energy could be treated as mean water level fluctuations. This was done for some few tests, but did not change the 2% run-up values significantly. Therefore, the values given in this paper do not take this into account.

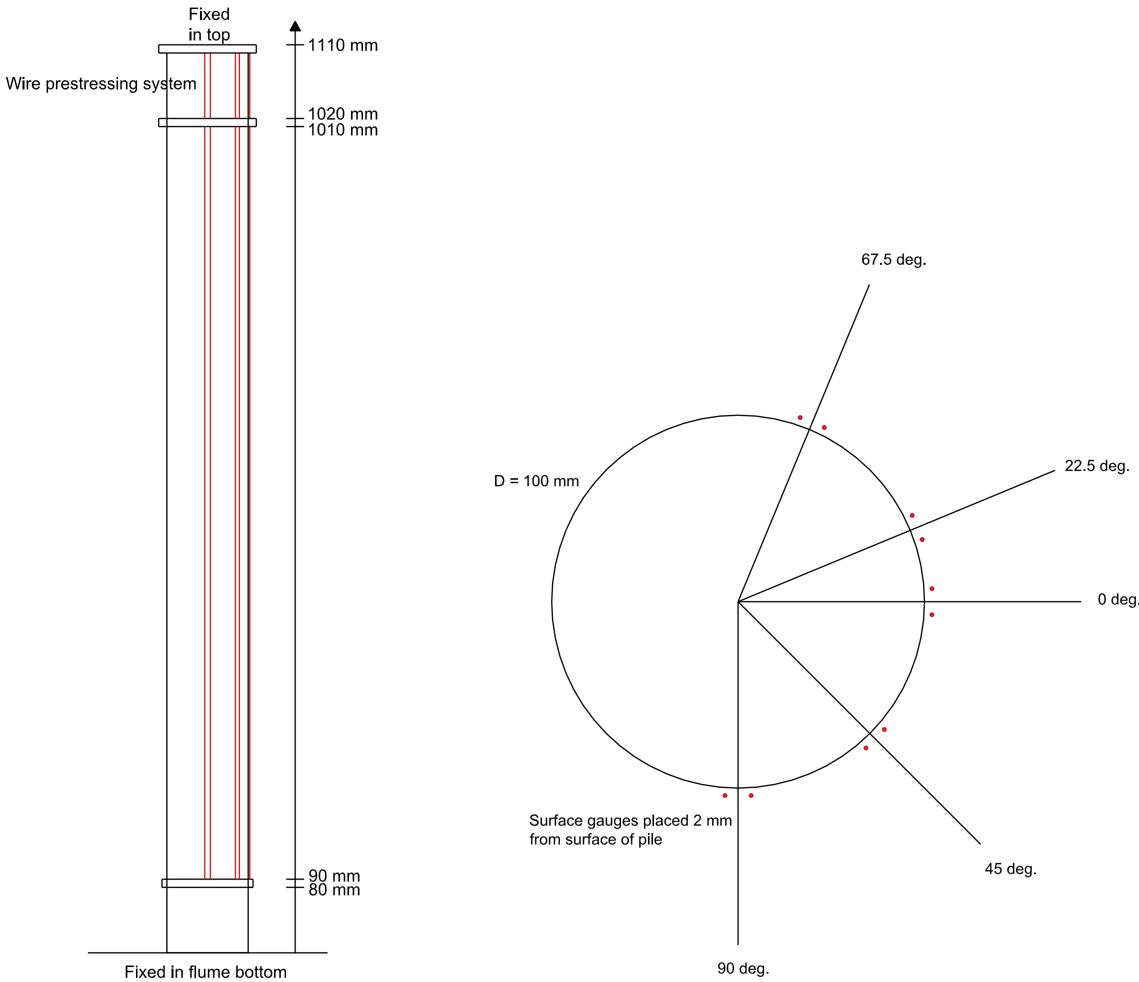

Fig. 5. Run-up model.

From the initial analysis of the run-up data it was found that the run-up data contains no "real" energy above 8 Hz — only noise. Therefore, an 8 Hz analog low-pass filter was applied for the run-up signals as well. Cross-correlation has been calculated with WaveLab2 in order to find the time delay between paddle displacement signals for calibration test and the run-up test. Hereby reproduction of wave trains was checked and run-up could be related to a specific wave in the wave train.

To derive the m factor in Eq. (2) it is necessary to estimate the crest elevation and the particle velocity in the top of the crest. Here the stream function theory for regular waves on a horizontal bed was utilized to perform these calculations. The number of terms in the Fourier series was set to $N=30$. The $H_{2\%}$, T_p values for the 2% run-up values and H_{max}, T_p for the maximum run-up values are used as wave height and period in these calculations.

7. Run-up height results

The m factor for each test can be derived from Eq. (2) for 2% run-ups and for maximum run-ups by replacing 2% values with maximum values. In both cases wave crest height and crest velocity are calculated using the above mentioned methodology. These results are presented in Fig. 6 for 2% and 3.5% peak wave steepnesses showing that low steepness waves give higher m values. It can also be concluded that the obtained m values are not significantly different for 2% and maximum run-up levels, but more scatter is observed for the maximum values.

The range of the m values is in pretty good agreement with the data of De Vos et al. (2007). An m value of four indicates that the initial run-up velocity is the double of incident crest particle velocity. Here the reference is made to the incident crest particle velocity as the structure interaction is included in the m value.

However, a difference with respect to the data of De Vos is that the big influence of H_{m0}/h was not identified in the present case. The influence of h/D seems also to be smaller than the one estimated from the De Vos et al. (2007) data. This might partly be a consequence of the larger γ-value applied by De Vos et al. (2007).

Fig. 7 shows an evaluation of Eq. (2) for 2% and maximum run-up using $m=4$ for 2% peak wave steepness and $m=3$ for 3.5% peak wave

Table 1
Irregular test conditions.

	$D=0.10$ m; $h=0.20$ m $(h/D=2)$	$D=0.10$ m; $h=0.30$ m $(h/D=3)$	$D=0.10$ m; $h=0.40$ m $(h/D=4)$
$H_{m0}/h=0.35$	$H_{m0}=0.070$ m	$H_{m0}=0.105$ m	$H_{m0}=0.140$ m
$H_{m0}/h=0.40$	$H_{m0}=0.080$ m	$H_{m0}=0.120$ m	$H_{m0}=0.160$ m
$H_{m0}/h=0.43$	$H_{m0}=0.086$ m	$H_{m0}=0.129$ m	$H_{m0}=0.172$ m
$H_{m0}/h=0.46$	$H_{m0}=0.092$ m	$H_{m0}=0.138$ m	$H_{m0}=0.184$ m

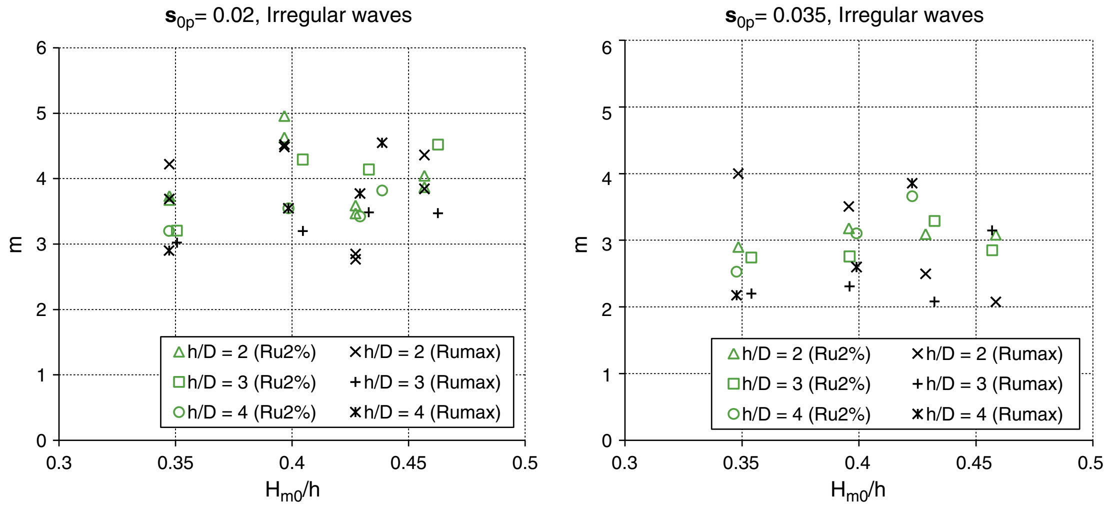

Fig. 6. The m values derived from the measurements for 2% and maximum run-up heights for the two wave steepnesses tested. Wave kinematics are calculated by stream function theory using $H = H_{2\%}$ for $R_{u,2\%}$, $H = H_{max}$ for $R_{u,max}$ and $T = T_p$ in both cases.

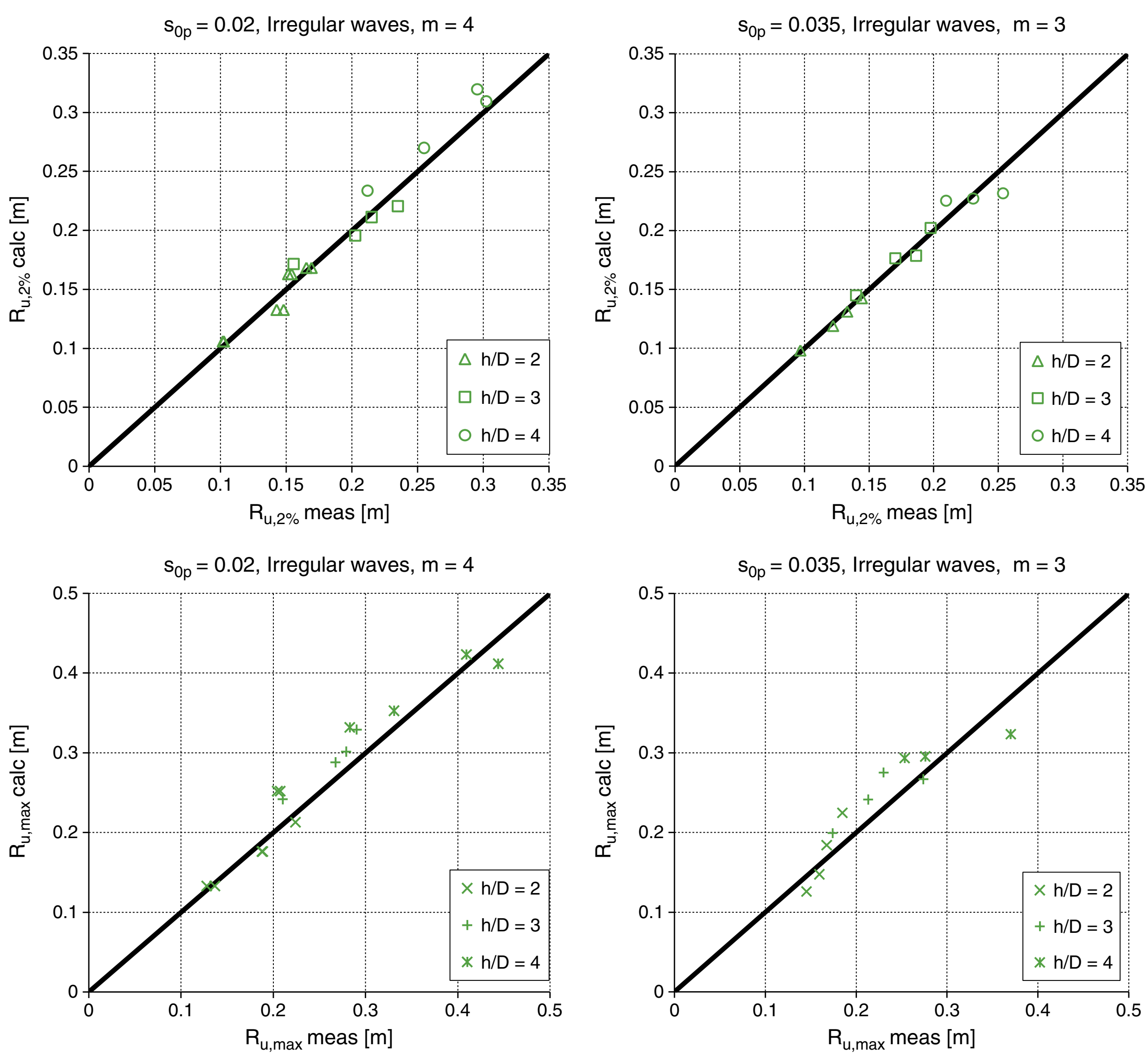

Fig. 7. Evaluation of Eq. (2) for 2% and maximum run-up heights using $m = 4$ for $s_{0p} = 0.02$ and $m = 3$ for $s_{0p} = 0.035$. Stream function theory is used with $H = H_{2\%}$ for $R_{u,2\%}$, $H = H_{max}$ for $R_{u,max}$ and $T = T_p$ in both cases.

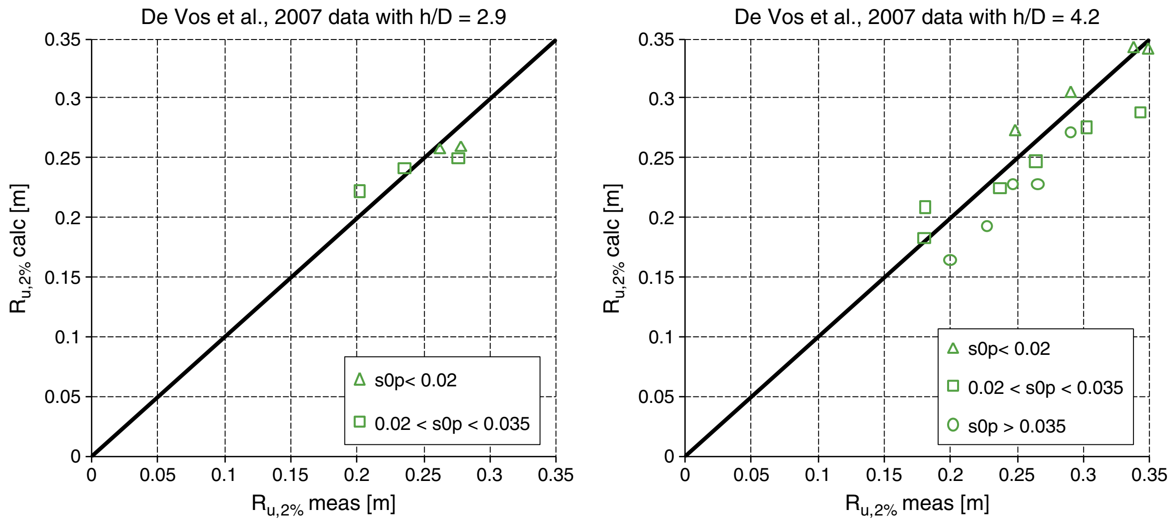

Fig. 8. Evaluation of Eq. (2) against De Vos et al. (2007) data for 2% run-up heights. Stream function theory is used with $H = H_{2\%}$ and $T = T_p$.

steepness. Even though there was some scatter on the m values the scatter on the predicted run-up heights is much less as the velocity is only one of the two terms involved. As expected it also appears that the scatter associated with 2% run-up levels is less than for maximum run-up levels due to higher statistical reliability.

Fig. 8 shows the application of the same method to De Vos data. In this case $m = 4$ for $s_{0p} \leq 0.02$ and $m = 3$ for $s_{0p} \geq 0.035$ are used and linear interpolation in between. The results show that the agreement is quite good but with more scatter than for present results and with a slightly conservative bias. This conservative bias might be due to the fact that De Vos positioned the gauges further away from the pile surface leading to a slightly larger underprediction of actual run-up heights compared to present measurements.

Fig. 9 presents the maximum measured run-up for each test as function of water depth. It can be seen that in the depth limited case the maximum run-up heights are slightly larger than the water depth for this gentle sloping bed and irregular waves. According to Damsgaard et al. (2007) results from DHI using 3D focused waves led to maximum run-up levels up to 1.75 times the water depth. This difference is expected partly to be due to higher possible waves when using 3-D focused waves with a focus point at the cylinder leading to a freak wave and partly due to using a high speed camera to measure run-up heights instead of a surface elevation gauge. From the DHI results Damsgaard et al. (2007) concluded that the use of a surface elevation gauge leads to an underprediction of the highest run-ups due to breaking waves and a thin run-up layer and with air entrainment. The curve they present shows that underprediction of run-up can be higher than 50% for the most extreme situations. However, here it is also important to separate between spray and green water run-up which is very difficult but important for load calculations.

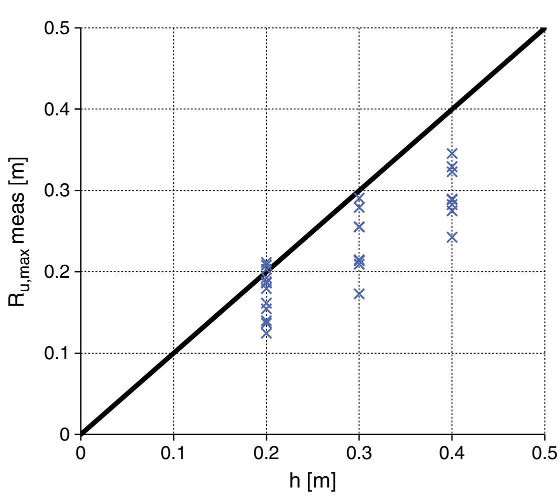

Fig. 9. Maximum run-up heights as function of water depth. Line shows $R_{u,max} = h$.

8. Distribution of run-up along pile

Fig. 10 presents the measured run-up distribution on the front part of the pile. A special note should be given to the result for 75° as much more scatter is observed here, which is expected to be due to insufficient pre-stressing of that gauge. Otherwise the results are in good agreement with the distribution presented by De Vos et al. (2007).

9. Run-up velocity results

Run-up velocities are needed for load calculations on platforms. The proposal is to use the assumption of no energy loss in run-up flow leading to:

$$v(z) = \sqrt{2g \cdot (R_u - z)} \qquad (3)$$

where z is the distance from SWL to the point of interest, i.e. the platform level.

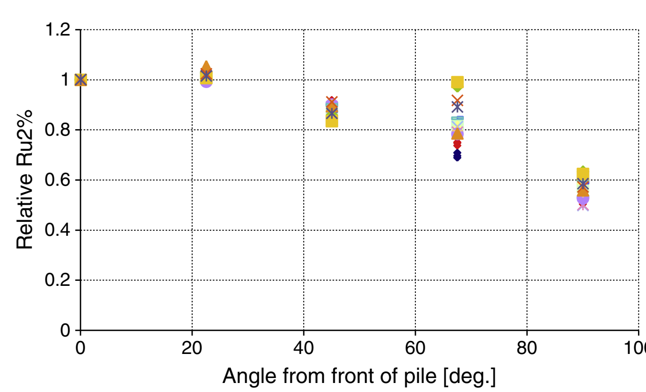

Fig. 10. Run-up distribution of 2% values along pile normalised with run-up measured at the front of the pile (0°).

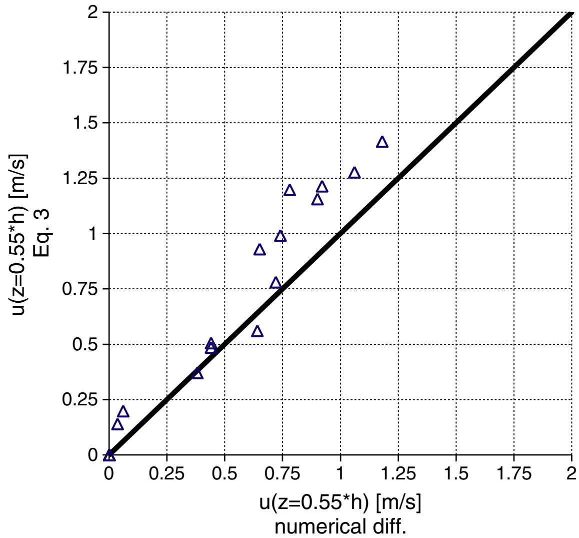

Fig. 11. Comparison of run-up velocities predicted by two methods. The run-up velocities are compared at $z = 0.55$ h, corresponding to the middle platform level tested by Lykke Andersen and Brorsen (2006). The run-up velocity on the abscissa axis is the maximum value of the five individual gauges.

This is compared in Fig. 11 to what is obtained by numerical differentiation of run-up signals. Reasonable agreement between the two methods is found, but with some bias for larger velocities.

10. Design rules

It turns out that when comparing identical events for measured run-ups and measured loads on the platform the run-up has been underestimated with the present set-up for run-up measurements. This can be explained by waves with steep front and breaking waves giving a thin run-up layer and with air entrainment and spray which are not correctly dealt with by the surface elevation gauge. Lykke Andersen and Brorsen (2007) found that in order to give good correlation between predicted run-up velocities and impact pressures on the platform a 40% increase of the calibrated m values is needed. Therefore, the following formulae should be applied for design situations where waves with steep front or breaking waves can occur.

$$R_{u,2\%} = \eta_{max,2\%} + 1.4 \cdot m \cdot \frac{u_{2\%}^2}{2g} \quad (4)$$

$$R_{u,\,max} = \eta_{max} + 1.4 \cdot m \cdot \frac{u_{max}^2}{2g} \quad (5)$$

An alternative to these formulae is to increase the run-up by 20% which leads to approximately the same results as the above given formulae, i.e.:

$$R_{u,2\%} = 1.2 \cdot \left(\eta_{max,2\%} + m \cdot \frac{u_{2\%}^2}{2g} \right) \quad (6)$$

$$R_{u,\,max} = 1.2 \cdot \left(\eta_{max} + m \cdot \frac{u_{max}^2}{2g} \right) \quad (7)$$

In Eqs. (4)–(7) $m = 4$ for $s_{0p} = 0.02$ and $m = 3$ for $s_{0p} = 0.035$. It should be noted that even for waves that are not depth limited some waves are still breaking or close to breaking with a steep front and thus leading to high and thin run-ups. Therefore, it is recommended to apply factor 1.4 (Eqs. (4) and (5)) and 1.2 (Eqs. (6) and (7)) in all typical applications of offshore wind turbines. To clarify this in more detail it is needed to consider another measurement system, for example a step gauge or video recordings. Also it should be remembered that factor 1.2 also applies to the data in Fig. 9 and thus a rule of thumb would be that for gently sloping beds the maximum run-ups would be approximately 1.2 times the water depth in depth limited conditions. For this rule of thumb it should be noted that the extreme wave heights in the model were smaller than predicted by Battjes and Groenendijk (2000).

From above it is therefore also obvious that factors 1.4 and 1.2 do not serve as safety factors, but are included to take into account underestimation of actual run-up in the tests for breaking and nearly breaking waves due to a thin run-up wedge with air entrainment not correctly measured by the measurement system.

Some guidance on slamming coefficients can be found in the study of Lykke Andersen and Brorsen (2007) to be applied for stiff structures with little mass (no dynamic amplification or dampening). In case of concrete platforms time duration of loads becomes important and a dynamical calculation is needed.

Here it is important to mention that safety on platform loads could not be included by a standard safety factor applied on the load as the run-up generated loads are very non-linearly dependent on run-up height. Therefore, extreme water levels and wave conditions are very important and safety could be included by considering extreme value combinations of these parameters for example by using a very high return period leading to the proper safety level.

If the safety requirements allow a narrower platform in a sector it is recommended to use the narrow platform side up against the dominant wave direction as this is expected to significantly reduce the loads. The same consideration should be made regarding the position of boatlandings as run-up is significantly smaller on the rear side of the pile, cf. De Vos et al. (2007).

11. Application of design rules to Horns Reef 1 case

The design procedure including the third step with the load model was applied to the storm giving damage to platforms at Horns Reef 1 wind turbine park. A fair agreement between damages and predicted loads was found. However, due to confidentiality reasons no further documentation can be presented at the moment.

12. Conclusion

Physical model tests were carried out in order to develop a design procedure for run-up and run-up generated loads on entrance platforms. The present article presents the run-up results. Typical ranges of water depth to pile diameter ratios (h/D), wave height to water depth ratios (H_{m0}/h) and wave steepnesses, relevant for application to offshore wind have been investigated.

An accurate design procedure has been established based on stream function theory for wave kinematics and velocity stagnation head theory with inclusion of an empirical factor on the velocity head. The method has further been evaluated against also data from other researchers and is reliable and accurate. Measurements of run-ups from impacting waves are difficult to measure accurately and visual observations indicate that the measurements in these cases underestimate actual run-up levels. Therefore, another empirical factor was needed in order to deal with these cases. Further research with an improved run-up measurement system (i.e. high speed camera) is needed in order to further decrease scatter on run-up and hence run-up generated loads for such situations.

The run-up results showed that the empirical run-up factor is highest for waves with low steepness. Together with the slightly higher crest velocities in the long waves this means that there is a quite significant influence of this parameter on run-up height. The influence of h/D and H_{m0}/h on the run-up factor seems rather small.

In conclusion the paper presents an easy to use design equation for run-ups related to typical offshore wind applications.

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