WAVE RUN-UP OF BREAKING AND NON-BREAKING WAVES WITH LONGSHORE CURRENT

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Abstract

In the frame of the EU HYDRALAB-III-Program investigations concerning the wave run-up and wave overtopping under special boundary conditions like onshore wind, longshore current and oblique incident waves were made. This paper presents the run-up tests and their results as well as their relations to former investigations.

1. Introduction

A reliable calculation of wave run-up on slopes is needed for the freeboard design at levees and embankment dams. Usually the wave run-up is calculated by means of two different formulas for breaking and non-breaking range. As both approaches run towards infinity this produces a discontinuity in the function of the normalized run-up versus the surf similarity parameter. To overcome this difficulty an integrated approach is needed.

2. Previous Run-up Investigations

The well-known surf parameter is used for the classification of the breaking behavior and breaker types. It is consisting of the quotient of the slope tan α and the root of the wave steepness with the deep water wave length $L_{m-1,0}$:

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{\frac{H_{m0}}{L_{m-1,0}}}} \quad [1]$$

For small surf parameters breaking waves (spilling, plunging, surging) can be expected whereas for $\xi_{m-1,0} > 2 \ldots 3$ non-breaking waves (reflection) are typical. As in a wave spectrum a wide spread of wave parameters may be included there are both breaking and non-breaking waves influencing the run-up process in the transition zone.

This was taken into account by Pohl, 1997 proposing the formula:

$$R = R_{nb} \cdot P + R_b \cdot (1 - P) \quad [2]$$

with the wave run-up height for non-breaking waves R_{nb} and the wave run-up height for breaking waves R_n, the occurrence probability of non-breaking waves P and the occurrence probability of breaking waves (1-P). Adapted to the 2%-run-up we get

$$R_{2\%} = R_{nb,2\%} \cdot P + R_{b,2\%} \cdot (1 - P) \quad [3]$$

This yields a smooth curve for the run-up starting from zero at very small slopes and surf similarity parameters respectively and running towards the run-up height at the vertical wall with a local maximum in between. Recent model tests (s. below) on wave run-up with a longshore current provided the opportunity to compare this approach with the new test results.

The data basis necessary for establishing the proposed run-up formula consists of both own model test data as well as available data sets of other authors (e.g. Ahrens, 1981; van der Meer & Janssen, 1994; Pohl, 1997). The hydraulic model tests by Heyer & Pohl, 2005 were conducted in a glass flume which was 30 m long, 0.8 m high and 0.8 m wide. The installed slopes had an inclination of 1:2, 1:1.3, 1:1 and 1:0.5. A sea state according to a JONSWAP-spectrum with H_{m0} = 0.077 m and a peak frequency of 1.14 Hz was used. As a result from these experiments the following modified probability distribution was derived

$$P = 1 - e^{-\left(\frac{\xi_0}{3.6}\right)^{2,25}} \quad [4]$$

that considers the statistical $R_{2\%}$ run-up height to consist of a fraction P of non-breaking waves and a fraction (1-P) of breaking waves. In other words P is assumed the probability that no breaking takes place and (1-P) is the breaking probability.

The run-up of breaking waves $R_{b,2\%}$ on smooth slopes may e.g. be calculated by means of the Hunt/Battjes formula:

$$R_{b,x\%} = k_r \cdot k_x \cdot \sqrt{H_m \cdot L_m} \cdot \tan\alpha \quad [5]$$

The coefficient k_r represents the roughness of the slope surface (s. a. Wagner, 1974). Using k_r = 1.0 on smooth slopes and k_x = 2.23 as a dimensionless parameter for the run-up exceedence probability of 2 % equation [5] yields with $H_m = 0.63 \cdot H_s$

$$R_{b,2\%} = 1.77 \cdot H_{m0} \cdot \xi_{m-1,0} \quad [6]$$

For non-breaking waves $R_{nb,2\%}$ approximately the approach

$$R_{nb,2\%} = 1.89 \cdot \sqrt{\frac{\pi}{2 \cdot \alpha}} \cdot H_{m0} \quad [7]$$

can be used. This yields almost identical results for breaking waves ($\xi_{m-1,0} < 2$). In the transition zone this gives a local maximum for the normalized run-up $R_{2\%}/H_{m0}$ at $\xi_{m-1,0} \approx 3$. The weakness of the most other approaches that the results either tend to infinity with growing (breaking) or dropping (non-breaking) $\xi_{m-1,0}$ or that different formulas must be used for different ranges of validity, could be overcome with this approach. For large $\xi_{m-1,0}$ (non-breaking, vertical wall) the $R_{2\%}/H_{m0}$-curve by Pohl & Heyer, 2005 goes asymptotically towards the value of $R_{2\%}/H_{m0} \rightarrow 2$, which stands for full reflection and is known as a standing wave (clapotis) from the theory.

To consider further influences on the wave run-up the coefficients γ_b for the influence of a berm, γ_f for the slope roughness and γ_β for oblique incident waves are used.

There are many approaches existing for the estimation of γ_β for the oblique wave approach. The coefficient γ_β is defined as the quotient of normalized run-up height with incident wave angle $\beta \neq 0°$ and the normalized run-up height for straight approaching waves:

$$\gamma_\beta = \frac{(R_{2\%}/H_{m0})_\beta}{(R_{2\%}/H_{m0})_{\beta=0°}} \quad [8]$$

Here the equations by Wagner & Bürger, 1973:

$$\gamma_\beta = 0{,}35 + 0{,}65 \cdot \cos\beta \quad [9]$$

and by de Waal & van der Meer, 1992 for a short crested Sea should be mentioned:

$$\gamma_\beta = 1 - 0{,}0022 \cdot \beta \quad [10]$$

Especially in the range of very oblique wave approach (β → ± 90°) the limiting values are partly not plausible, wherefore the application of these equations should be limited to angles β < |±50°|.

3. Recent Experiment and Data Processing

To verify the above approaches and to study further influences on the run-up height two series of model tests were carried out in the frame of a HYDRALAB-III-project at the DHI shallow water basin at Hørsholm, Denmark in 2009 (see Figure 1). The focus was on oblique incident waves and longshore currents. Besides this the effect of onshore wind has been investigated.

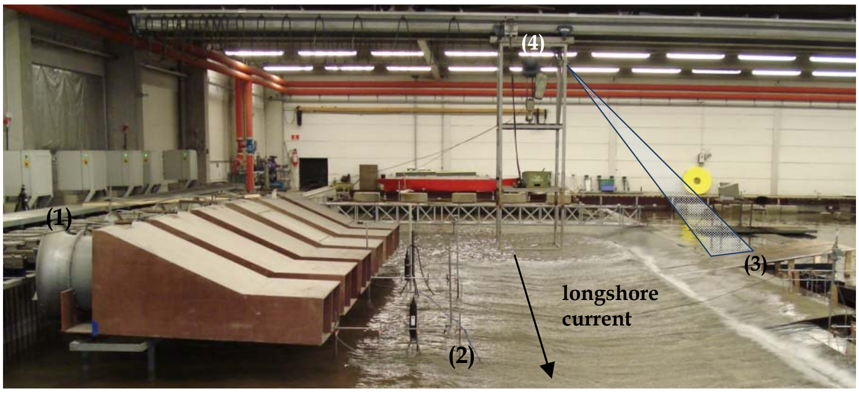

Figure 1. Model set-up in the large DHI shallow water basin at Hørsholm, Denmark; (1) wave-maker, (2) wave gauges, (3) run-up slope area with capacitive gauge, (4) video camera. In front of the wave-maker the large fans producing the onshore wind are visible.

For the model tests a 1:3 sloped levee and later a 1:6 sloped levee were constructed in the wave basin. The wave run-up on the smooth cement faced slope was measured by a capacitive gauge fitted to a 2 m wide run-up board and recorded in addition by a digital video camera.

Table 1. Wave heights (H_s) and peak periods (T_P) of the run-up tests.

		WAVE 1	WAVE 2	WAVE 3	WAVE 4	WAVE 5	WAVE 6
1:3 SLOPED LEVEE	H_s [m]	0.07	0.07	0.1	0.1	0.15	0.15
	T_P [s]	1.474	1.045	1.760	1.243	2.156	1.529
1:6 SLOPED LEVEE	H_s [m]	0.09	0.09	0.12	0.12	0.15	0.15
	T_P [s]	1.670	1.181	1.929	1.364	2.156	1.525

The model set-ups were constructed to investigate different sea states (JONSWAP-spectra) with significant wave heights between 0.05 and 0.15 m and peak periods between 1.04 and 2.16 s (see Table 1).

An oblique wave approach (β = 15°, 30°, 45°) was considered in order to compare the measured data with previous results. In addition the influence of a longshore (levee-parallel) current (1:3 sloped levee: v = 0.15 m/s, 0.3 m/s; 1:6 sloped levee: v = 0.15 m/s, 0.3 m/s, 0.4 m/s) was studied. The measurements included the current velocity, wave parameters (height, spectral period) and the wave run-up height as time series.

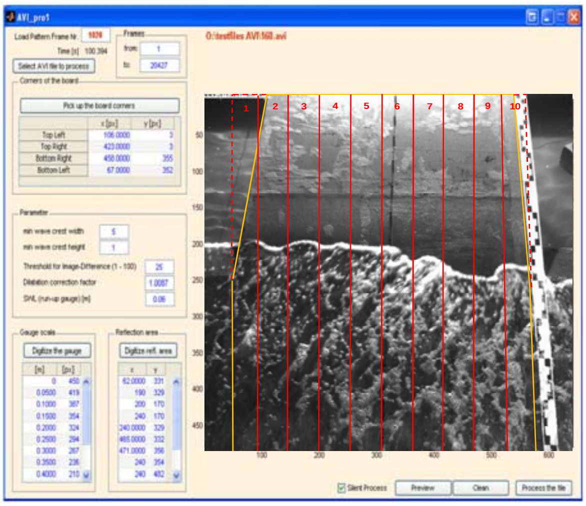

Figure 2. MATLAB user interface for image data processing.

A very detailed description of all data processing and data validation can be found in Lorke et al., 2012.

The wave run-up time series for each test were obtained from the voltage-values measured by the capacitive run-up gauge by means of a calibration curve. To create wave run-up time series from video films a MATLAB procedure has been used.

In a first step of the procedure it was detected in which parts (pixel) of the frame a movement has taken place which is visible by changes in pixel brightness. Therefore the difference between two frames in sequence was calculated. The difference is equal zero if there was no movement and unequal zero if there was a movement. A variable threshold (threshold for image difference, see "Parameter" in Figure 2) has been used to adjust the sensitivity in detection of pixels with significant brightness difference.

As a result a new black/white frame was created. Pixels with a significant change in pixel brightness were defined as white pixels and all other as black pixels. Figure 3 shows as an example a video frame in grey scale and an according frame in black and white which represents the change in pixel brightness between the two successive frames. The wave front is easily detectable but there are white pixels right above the up-rushing water front which are caused by water from the previous wave flowing down the run-up board. Furthermore there are white pixels above in the middle of the run-up board which indicates light reflected on the capacitive gauge.

The reflections are in general characterized by a size of only one pixel or very few pixels. Therefore it was necessary to define a so called “minimum region” by determining a “minimum wave crest width” and a “minimum wave crest height” to avoid false detection of reflections as upmost wave tongue.

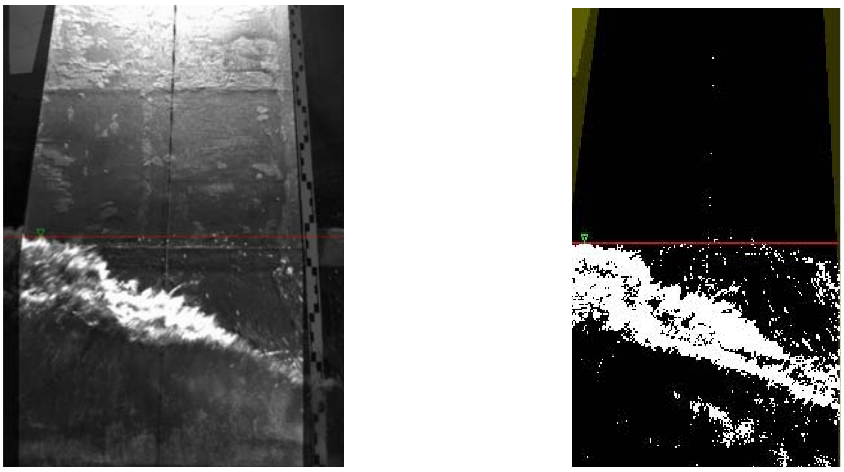

Figure 3. Left: original video frame Right: associated picture displaying the difference in pixel brightness between the frame at the left side and its successive frame in the video film (test s5_22_15_w6_00_30w).

The setting of these two parameters is possible within the left section “Parameter” of the designed MATLAB interface (see Figure 2). Different settings were necessary because different video cameras were used. A “minimum wave crest width” between 5 and 20 pixels was sufficient in most cases. The “minimum wave crest height” was set between 1 and 5 pixels respectively. In a next step every line of the black/white frame was checked beginning in the left above corner of the frame and continuing in right and downwards direction. If the routine find a minimum region (between 5 and 100 contiguous pixels) this was defined as the maximum run-up tongue (marked with a green triangle in Figure 3). At the end the recording time of each video frame has to be assigned to the detected run-up in it in order to get the run-up time series.

To calculate $R_{2\%}$ based on the run-up time series for both measurement devices (capacitive gauge, video films) another MATLAB procedure has been programmed. The wave run-up height $R_{2\%}$ is determined with a crossing analysis using a threshold level different from zero. This was chosen due to practical reasons. Not all smaller events can be detected but it avoids losing higher run-up events when the down rushing water after a run-up event still remains above SWL until the next wave rushes up.

The crossing level was always chosen in such a way that at least n = 500 run-up events and their maximum run-up height could be detected. These n maximum values were than sorted in descending order. The number of incoming waves per test was approximately N = 1000. For N = 1000 the wave run-up height $R_{u2\%}$ has been defined as the minimum value of the highest $k = 0.02 \cdot N = 20$ run-up events. This is one reason why the relative wave run-up height is a very sensitive parameter.

To find appropriate formulas for the reduction coefficients γ_β considering oblique incident wave crests theoretical considerations were made to find boundary conditions and to allow an extension of the functions beyond the known ranges towards its tails at incident angles near 90 degrees (levee-parallel).

4. Results and discussion

For orthogonal incident waves (β = 0°) the $R_{2\%}$ run-up heights confirmed former results (Fig. 4) and provided a good basis for the further investigations with more complex boundary conditions. In Fig. 4 numerous results of several researchers are plotted. Among them are e.g. the formulae by Ahrens, 1981, from the EUROTOP-Manual, 2007.

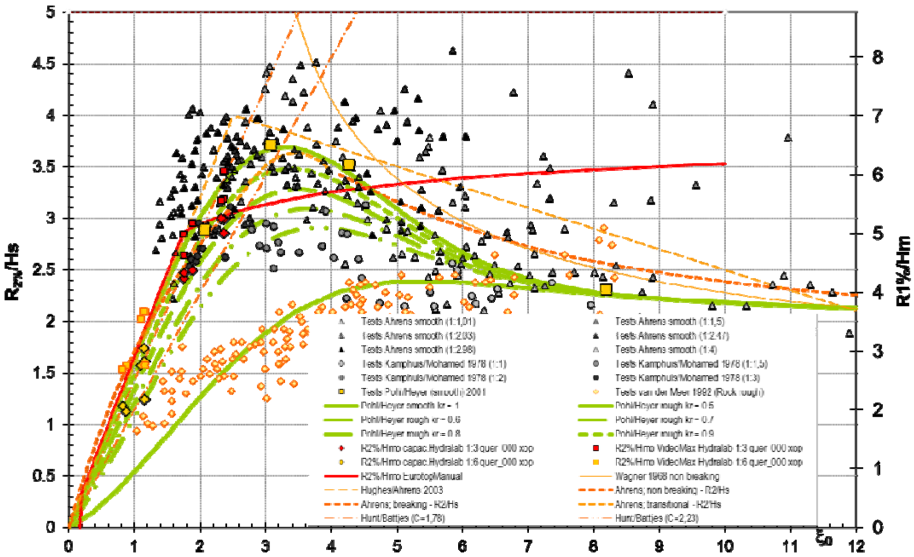

Figure 4. Test data and fitting curves for the normalized wave run-up height $R_{2\%}/H_{m0} \approx R_{2\%}/H_S$ compared with own results.

The longshore currents between the wave-maker and the model levee were of v = 0, 0.15, 0.3 and 0.4 m/s (only at the 1:6 levee). Although the plotted results in Figure 5 had a considerable scatter the appropriate coefficient γ_{cu} was in average about one which shows that currents with the tested (or up-scaled) celerities have no considerable influence on the run-up height.

This is important e.g. for freeboard design at levees in the presence of parallel currents. The currents could be generated by the tides, by the wind or river flow. They can be found at sea, river and estuary levees. The gained data seem to give some evidence that the existing freeboard design methods for lakes and reservoirs could also applied to flowing waters.

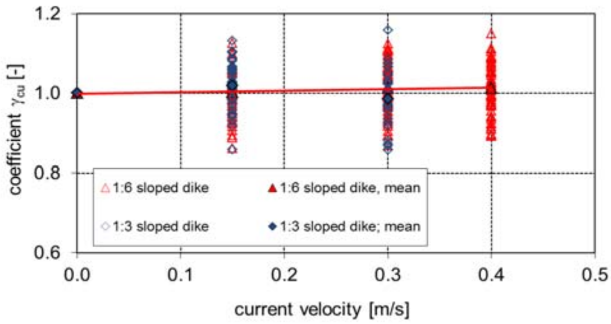

Figure 5. Coefficient γ_{cu} for longshore current effects.

The influence of the angle of wave approach on run-up can be described using the function (cos β) because dike slope (tan α = 1/m) for perpendicular wave approach and the according dike slope (tan(α′) = 1/m′) considering a wave approach under the angle β can be expressed by:

$$\frac{\tan\alpha'}{\tan\alpha} = \cos\beta \qquad [11]$$

Because the run-up is proportional to the levee slope the ratio γ_β is proportional to (cos β) too. To estimate the boundary value for a function $\gamma_\beta = f(\beta)$ wave run-up on a very flat shore as well as at a vertical wall should be discussed. On a rather flat shore (α → 0°) a total refraction is possible. Wave direction in case of shore parallel waves (β = 90°) would be changed and resulted in an almost perpendicular wave approach and the run-up would approach more or less that in case of β = 0° (see Figure 6, left side). From this a ratio $\gamma_{\beta = 90° (\alpha \to 0°)} \approx 1$ could assumed. Of course there would be no refraction and no run-up on a horizontal bottom.

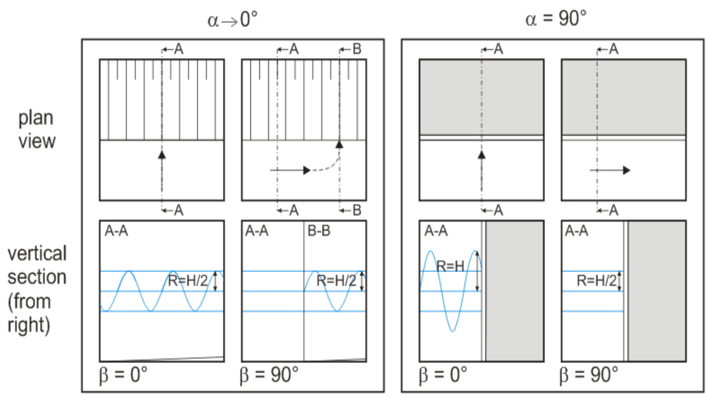

Figure 6. Wave run-up height: boundary values for perpendicular or parallel "run-up" und a very flat shore (left) and at a vertical wall (right).

Waves propagating in the perpendicular direction (β = 90°) of a vertical wall (α = 90°) creates a run-up R = H (see Figure 6, right side). If one considers a vertical wall and a wall parallel wave approach (β = 0°) the waves would move alongside the wall and the "run-up" is equal to the wave height above the still water level of R = H/2. From this it follows that $\gamma_{\beta = 90°(\alpha \approx 0°)} = 0.5$.

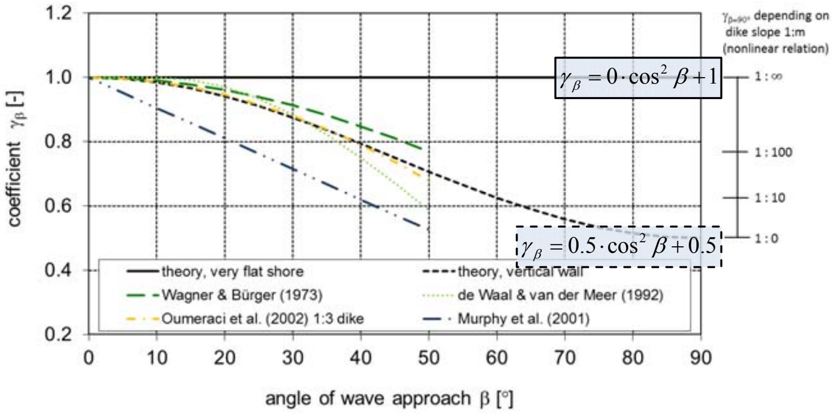

Figure 7. Empirical function for the influence factor γ_β in dependence on the angle of wave approach.

A function capturing all these considerations could be

$$\gamma_\beta = a \cdot \cos^2 \beta + b \quad [12]$$

The variables a and b (with a + b = 1) depend at least on the levee slope. The coefficient b represents the boundary value γ_β = 90°. It has to be lower in the case of a steeper slope and higher in the case of a flatter slope (see Figure 7).

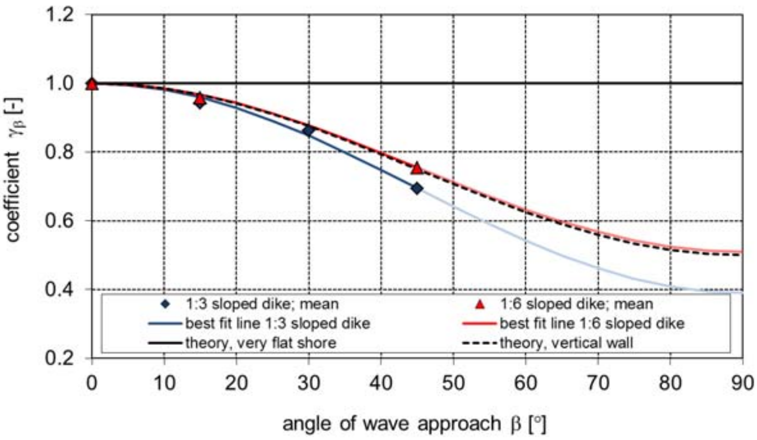

Figure 8. Influence factor γ_β in dependence on the angle of wave approach β.

The results in Fig. 8 show good agreement with existing empirical functions. In general it could be stated that the results fit in former investigations and could be an additional prove that the hydraulic model set-up was appropriate chosen. Two new equations were fitted to the results according to the equation type derived above:

$$\gamma_\beta = 0.49 \cdot \cos^2 \beta + 0.51 \text{ (1:6 sloped levee)} \quad [13]$$

$$\gamma_\beta = 0.61 \cdot \cos^2 \beta + 0.39 \text{ (1:3 sloped levee)} \quad [14]$$

Further data analysis is presented in Lorke et al., 2012. The combined effect of oblique wave approach and a longshore current was investigated too. The results show non obvious dependencies but it has to be considered that the relative wave run-up height is a very sensitive parameter. The data analysis included the comparison between measured and calculated relative wave run-up. Calculation was done using the formula of EurOtop, 2007 together with the estimated influence factors γ_β, γ_{cu}, and γ_w for the wind (not discussed here). The comparison shows a good agreement between the measured and the calculated values. All pairs of values are in a range of ± 20 %.

5. Conclusions

The study provides new test results for wave run-up under oblique wave approach and with longshore current. The data analysis on wave run-up was based on an advanced data extraction from video films by means of a MATLAB procedure.

In a first step the data were used to verify a formula for breaking and non-breaking waves. In a second step the influence parameter γ for each single influence variable were obtained by analysing tests either with only oblique waves or longshore current.

Results considering oblique wave approach confirm former empirical investigations for smaller angels of wave approach. New formulas for each levee slope investigated within the project were derived. Furthermore no significant effect on wave run-up in case of a longshore current velocity v < 0.4 m/s and a perpendicular wave approach was obtained.

6. Outlook

The longshore current had no significant influence on the wave run-up within the range of investigated velocities. It might be of some interest if this would change at higher velocities or if other combinations of influencing parameters would modify this result.

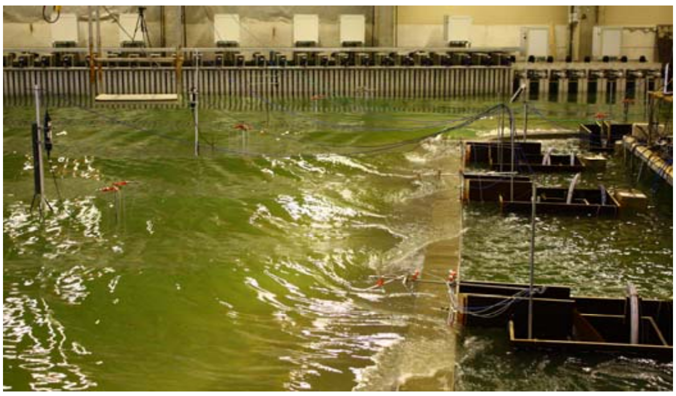

Figure 9. Model-Set-Up in the HYDRALAB-IV-project CornerDike 2012.

On the other hand it would also be of interest, how the very oblique wave approach for incident angles > 45° and for Lee situations (β > 90°) would influence the run-up and overtopping of waves. Additional experiments were made at the DHI in September and October 2012 (Fig. 9) to investigate this question. The analysis has not been finished yet and is being carried out by a user group consisting of researchers from the Universities Aachen (D), Brno (CZ), Dresden (D), Gent (B) and from Van der Meer Consult (NL).

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