# WALOWA (WAVE LOADS ON WALLS)
# - LARGE-SCALE EXPERIMENTS IN THE DELTA FLUME -

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**Abstract:** Overtopping wave loads on vertical structures on top of a dike have been investigated in several small scale experiments in the past. A large-scale validation for a mild foreshore situation is still missing. Hence the WALOWA experimental campaign was carried out to address this topic. In the present paper the objectives of the WALOWA project are outlined in detail, the model and measurement set-up described and the test program presented. Furthermore, preliminary results featuring a single 1000 irregular waves test of the test program are highlighted. This includes the study of the mild and sandy foreshore evolution by comparing profiles before and after the test execution. The profile measurements are obtained with a mechanical profiler. The wave parameters offshore and at the dike toe are numerically simulated using a SWASH model. The numerical results are validated against the measurements. Finally, the force and pressure time series of the waves impacting against the wall are processed and filtered. The load cell measurements and the time series of integrated pressures are compared to each other and for each impact event the maximum force is derived.

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## INTRODUCTION

Commonly design guidance for overtopping and overtopped bore impacts on structures situated on top of a dike is developed in relatively deep water conditions. However, in countries like Belgium, The Netherlands, Germany and in the northern Adriatic sea in Italy the coast is characterized by shallow water depths at the dike toe, with gentle foreshore conditions, termed as mild foreshore. Typically, a dike is followed by a promenade and at the end of the promenade buildings or storm walls are constructed. This setting makes it possible for waves to overtop the dike and impact on the storm wall or building. Especially during the storm season the overtopping waves can induce large forces on these structures. New scenarios for climate change and sea level rise make it worthwhile to invest in research regarding overtopping wave impacts.

Design guidance for this set-up is scarce and often neglects the effects of the mild foreshore (dissipation of wave energy due to depth-limited wave breaking and release of low frequency waves). Recently, the reduction in overtopping (Altomare et al., 2016) and the increase in wave periods at the toe (Hofland et al., 2017) were studied for such conditions. Furthermore, numerical simulations were performed (Suzuki et al., 2017). Small-scale experiments of overtopped wave impacts on walls were conducted (Streicher et al., 2016) and design guidance for overtopped wave impacts on walls for mild foreshore conditions derived (Chen et al., 2016). A first large scale experiment to measure the overtopped bore impacts on a storm wall, situated on top of a dike, was described by Van Doorslaer (2017). Anyhow the effects of a mild foreshore on wave breaking processes and wave impacts were not modelled. Hence, a large-scale validation of overtopped wave impacts for this set-up is still missing.

## WALOWA OBJECTIVES

For the research project WALOWA (WAve LOads on WAlls), carried out within the EU programme Hydralab+, model tests in the Delta Flume in Delft, The Netherlands were conducted in March 2017. The project is a cooperation of Ghent University (Belgium), TU Delft (The Netherlands), RWTH Aachen (Germany), Polytechnic University of Bari, University of L'Aquila, University of Calabria, University of Florence (Italy) and Flanders Hydraulics Research (Belgium). The facility provider is Deltares. It is the aim to study overtopping wave impacts on storm walls and buildings situated on top of a dike and for mild foreshore conditions. More detailed the objectives are as followed:

(1) To study the impact force and pressure behavior of overtopped waves on vertical structures on top of a dike. Eventually to relate this to the incoming waves and geometrical parameters.
(2) To study scale effects by comparing the physical model results to small scale experiments conducted on a similar geometry.
(3) To analyze bed profile changes and suspended sediment concentration of the sandy foreshore.
(4) To validate numerical models in terms of wave evolution over the mild foreshore (SWASH), foreshore evolution itself (XBeach) and wave impact forces and pressures on the wall (DualSPHysics).
(5) To study the overtopped flow formation, layer thicknesses and velocities, on the promenade and the complex interaction between incoming and reflected bores.

(6) To apply new measurement techniques such as a laser scanner to monitor the whole chain of overtopping and wave impact parameters. Starting from the wave heights at the dike toe, the overtopping wave characteristics and overtopping flow formation until impact on the wall and run-up at the wall.

## MODEL SET-UP

The study of overtopped wave loads on vertical walls requires a large-scale wave flume facility to validate and extend the small-scale experiments conducted at Ghent University and Flanders Hydraulics Research. It is the objective of this study to design an experimental set-up comparable to these small-scale experiments, which again are based on typical coastal profiles at the Belgian coast.

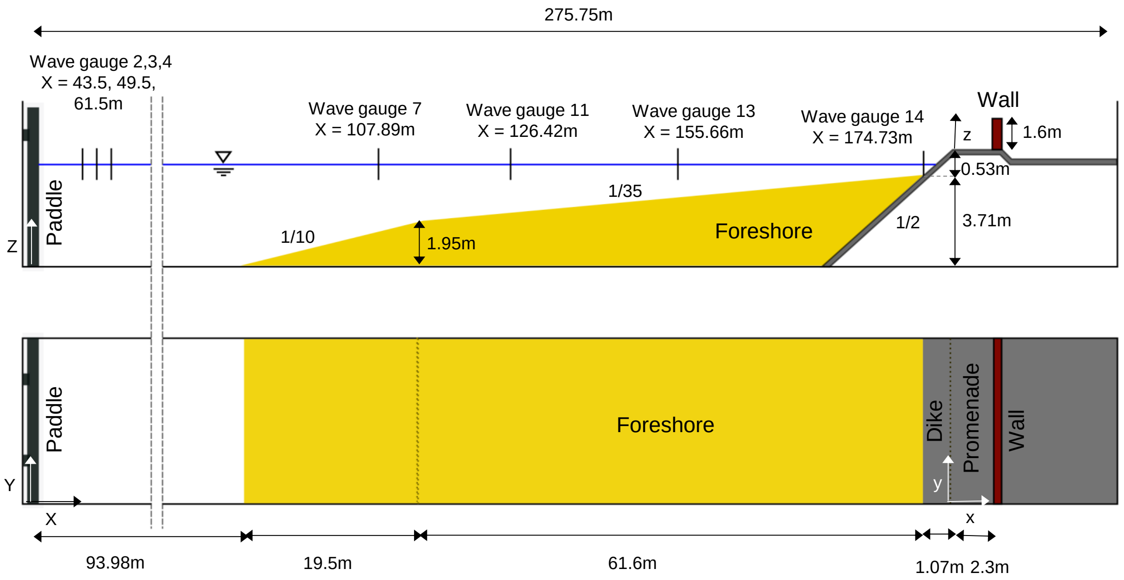


Figure 1. Model geometry of the WALOWA project

As shown in Figure 1, the model geometry is divided into 4 parts. (1) A sandy foreshore with a combined slope $\theta_1 = 1/10$ at the beginning and $\theta_2 = 1/35$ until the toe of the dike, along 19.5 m and 61.6 m, respectively. The total foreshore volume is comprised of ~1000 $m^3$ of sand. Sand with a grain size $D_{50} = 320$ μm was installed in a 0.4 m deep top layer over the entire foreshore. Below the top layer sand with $D_{50} = 230$ μm was installed. The erosion depth over the entire foreshore never exceeded 0.4 m during the tests, hence a uniform sand distribution of $D_{50} = 320$ μm is assumed. The sand profile was levelled up to 2 cm accuracy before the start of the tests. (2) Attached to the foreshore a concrete dike with a 1/2 slope and (3) a 2.3 m wide promenade with an offshore slope of 1/100 to drain the water. (4) At the end of the promenade a vertical 1.6 m high steel wall. The steel wall comprises 3 horizontal IPE500 beams and 9 vertical IPE160 beams serving as a support structure for the 12 mm thick steel plate (Figure 2). The steel plate is prefabricated and openings for the measurement sections are foreseen. The total weight of the wall is approximately 2.3 ton. Simplifying the wall to a clamped single beam structure for the lowest horizontal IPE500 beam and adding the mass of one IPE500 and 9 IPE160 beams, steel plate and added water in front of the wall over 0.66m height, results in a $1^{st}$ natural frequency of 77 Hz.

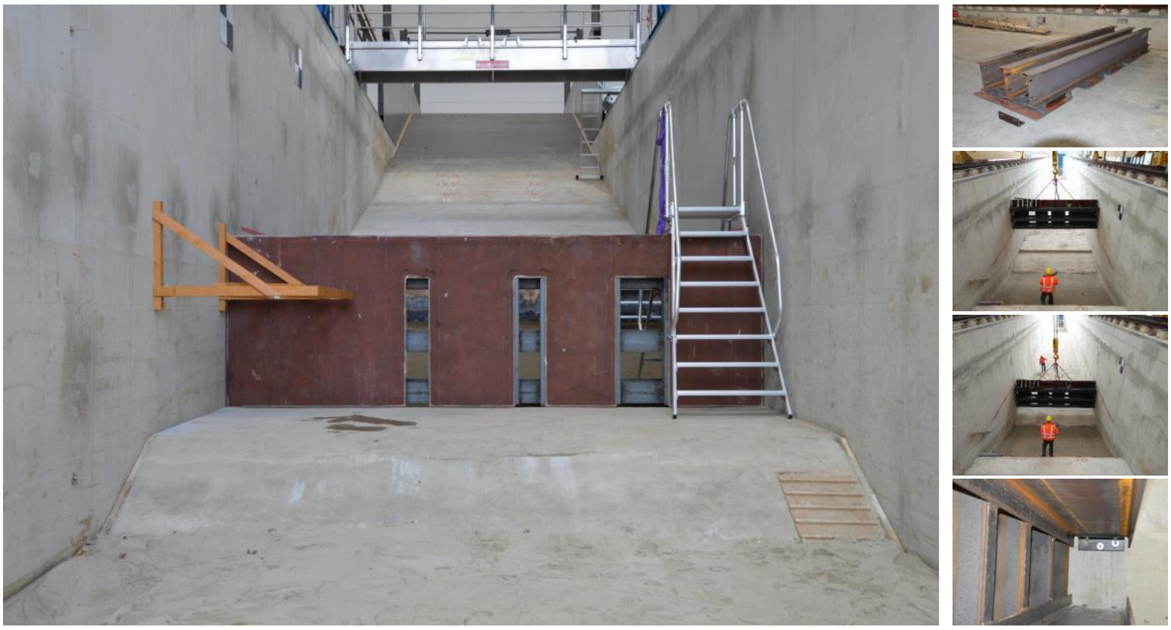

Figure 2. Steel wall as constructed in the Delta Flume with openings for the force and pressure measurement sections and an observation window. Images to the right show the installation of the steel wall.

The global coordinate system for the Delta Flume is originated at the lower right corner of the wave paddle when standing with the back to the paddle (Figure 1). The positive X-direction is defined in the main flume axis pointing towards the model. The positive Y-direction is defined in cross flume direction pointing to the left and the positive Z-direction is pointing upwards from the flume bottom. X, Y and Z are written in capital letters. Since most measurements are located close to the wall a second, local coordinate system is defined. It is originated at the dike crest location on the same side of the flume as the global coordinate system with positive x-direction in the main flume direction pointing towards the wall, positive y-direction in cross flume direction pointing to the left and positive z-direction pointing upwards. x, y and z are written in small letters. The local coordinate system origin corresponds to $X = 176.15$ m, $Y = 0$ m and $Z = 4.24$ m of the global coordinate system. The model dimensions are given in model scale and are comparable to a real case situation at the Belgian coast (using Froude similarity 1/4.3).

## MEASUREMENT SET-UP

The applied measurement devices are divided into groups based on the measured parameters. In total 5 different groups of measured parameters can be distinguished. (1) The water surface elevation is measured by resistance type wave gauges installed at 3 locations offshore for reflection analysis and 4 locations along the foreshore until the dike toe (Figure 1). The metal rods are deployed at the left wall of the flume ($Y = 5$ m). Special care is taken to submerge the reference electrodes (and to not expose them to sand or air) at all times. Additionally, a WaveGuide wave radar is deployed 7-8 m above the dike toe to double check the water surface measurement at this sensitive location. The wave radar has a beam divergence angle of 5° and a stated vertical accuracy of 1 cm. (2) The overtopping flow parameters layer thickness and velocity are measured by instruments attached to a wooden frame (Figure 2, left side) installed 1 m above of the promenade. The layer thickness is recorded by resistance type wave gauges (with submerged reference electrode), 3 ultrasonic distance sensors of type MaxSonar HRXL and 1 ultrasonic distance sensor of type

Honeywell 943 M18 (Figure 4). The velocity is recorded by a Valeport 802 electro-magnetic current meter and 4 Airmar flow meter paddle wheels installed in one line in the flow direction (Figure 4). (3) The impact force is measured by 2 HBM U9 load cells connecting a hollow steel profile to the support structure of the wall (Figure 3, right). The load cells are spaced vertically above each other. The measurement range is 20 kN. The impact pressures are measured with 15 Kulite HKM-379 (M) pressure sensors spaced vertically and horizontally over a metal plate (Figures 3, left). The measurement range is 1 bar. Both hollow steel profile and metal plate fit into prefabricated openings in the steel wall. The hollow steel profile is hanging in the opening and attached towards the side to avoid rotational movements. The metal plate with the pressure sensors is screwed into the opening. Both hollow steel profile and metal plate are flash mounted with the steel wall as a result. (4) The bottom profile is measured with a mechanical profiler after each test. The profiler wheel is pulled over the sandy foreshore and the elevation differences are recorded. One center profile and 4 lateral profiles in 0.2 m and 0.4 m distance from the centerline are recorded. Additionally, the suspended sand concentration is measured with an Argus Surface Meter (ASM) IV-N, which is installed 1.4 m apart from the dike toe location and 1 m apart from the right side wall (x = -2.47 m, y = 1 m). (5) Synoptic measurements are taken at different locations in the model. 3 GoPro Hero5 cameras are mounted to monitor the entire promenade in a top view, the side view of the promenade and the back view of the promenade from behind the observation window in the steel wall. The spatial resolution never exceeds 2 mm on the promenade and the devices record in Line mode, to reduce fish-eye effects. Additionally, a high speed camera is placed behind the observation window in the steel wall to monitor the impacting flow. A void-fraction meter is installed in a small opening of the steel wall (x = 2.35 m, y = 1.57 m , z = 0.31m), to measure the air-water fraction in the impacting flow. A SICK LMS511 laser scanner is deployed approximately above the dike toe location (x = -1.4 m , y = 5 m, z = 5 m), measuring a profile line starting from 10 m offshore along the dike, middle section of the promenade and steel wall. The angular resolution is set to 0.25°. A combination of distance and intensity measurement from the laser scanner is used to detect the water surface along the monitored profile. Except for the wave radar, profiler and ASM all instruments are synchronized using a sync pulse from the main data acquisition system and the cameras via a LED light triggered by the sync pulse. Wave radar and ASM are synchronized using the internal computer clocks. The wave radar measurement is sampled at 5 Hz, ASM at 1 Hz, laser scanner at 35 Hz, profiler at 100 Hz, void fraction meter at 10000 Hz, high-speed camera with 100 fps, GoPros with 60 fps and all other devices at 1000 Hz.

Figure 3. Pressure array (left) and hollow force measurement section (right) before implementation into steel wall

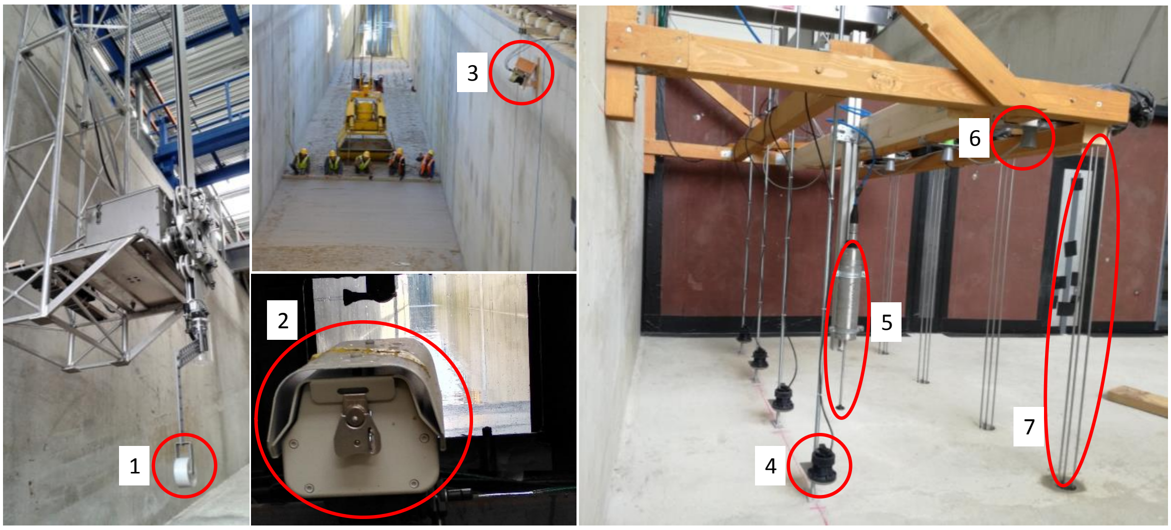


Figure 4. Highlighted in red circles the following measurement devices: (1) mechanical profiler wheel, (2) high speed camera behind observation window, (3) laser scanner, (4) paddle wheel, (5) electro-magnetic current meter, (6) ultrasonic distance sensor, (7) wave gauge on promenade

## TEST PROGRAM

The range of tested wave parameters is similar to the 1/1000 and 1/17000 design storm conditions for the Belgian coast (Veale et al., 2012). The values are downscaled using scale factor 4.3 and Froude scaling. For selected tests the water level is increased by 0.1 m or the wave parameters are increased by 20% to account for sea level rise and increased storminess respectively. A distinction is made between irregular waves (Irr) in first (F) and second order (S) form, to simulate a standard Jonswap based sea spectrum and bichromatic waves (Bi), to enable the study of wave-wave interaction. Four tests of the test program are repeated (R). The letters between brackets are also part of the unique testID for each test. The tests are given in chronological order in Table 1 together with the measured (water level and freeboard) and numerically modelled (wave height and period) test parameters.

Figure 5. Image of broken wave on foreshore, dry promenade, wet promenade + wave impact, sandy foreshore after approximately 20000 waves (left to right)

The index 'toe' and 'off' for the wave parameters refers to dike toe and offshore measurement location respectively.

Table 1. Test program WALOWA with measured parameters in chronological order

| testID | Waves | $h_{paddle}$ | $h_{toe}$ | $A_c$ | $H_{m0,off}$ | $H_{m0,toe}$ | $T_{m-1,0,off}$ | $T_{m-1,0,toe}$ | $h_{toe}/H_{m0,off}$ |
|---|---|---|---|---|---|---|---|---|---|
| - | - | m | m | m | m | m | s | s | - |
| Bi_1_4 | ~18 | 3.99 | 0.28 | 0.25 | 1.11 | 0.36 | 6.76 | 19.89 | 0.25 |
| Bi_1_5 | ~18 | 4.00 | 0.29 | 0.24 | 1.29 | 0.42 | 6.99 | 21.55 | 0.22 |
| Bi_1_6 | ~18 | 4.01 | 0.30 | 0.23 | 1.23 | 0.40 | 7.40 | 21.44 | 0.24 |
| Bi_2_4 | ~18 | 4.13 | 0.42 | 0.11 | 1.17 | 0.44 | 6.10 | 19.36 | 0.36 |
| Irr_1_F | ~1000 | 3.99 | 0.28 | 0.25 | 1.05 | 0.30 | 5.80 | 12.30 | 0.27 |
| Irr_2_F | ~3000 | 4.00 | 0.29 | 0.24 | 0.92 | 0.29 | 5.36 | 10.39 | 0.32 |
| Irr_2_S | ~3000 | 3.99 | 0.28 | 0.25 | 0.92 | 0.29 | 5.38 | 9.35 | 0.30 |
| Irr_3_F | ~3000 | 4.12 | 0.41 | 0.12 | 0.92 | 0.36 | 5.36 | 7.98 | 0.45 |
| Bi_2_5 | ~18 | 4.14 | 0.43 | 0.10 | 1.27 | 0.49 | 6.16 | 17.31 | 0.34 |
| Bi_2_6 | ~18 | 4.14 | 0.43 | 0.10 | 1.30 | 0.51 | 6.24 | 17.14 | 0.33 |
| Bi_2_6_R | ~18 | 4.14 | 0.43 | 0.10 | 1.31 | 0.50 | 6.19 | 17.26 | 0.33 |
| Irr_8_F | ~1000 | 4.13 | 0.42 | 0.11 | 0.49 | 0.35 | 3.83 | 4.85 | 0.86 |
| Irr_4_F | ~1000 | 3.79 | 0.08 | 0.45 | 0.87 | 0.22 | 5.41 | 12.05 | 0.09 |
| Irr_5_F | ~1000 | 3.78 | 0.07 | 0.46 | 1.05 | 0.26 | 5.82 | 13.55 | 0.07 |
| Irr_1_F_R | ~1000 | 4.01 | 0.30 | 0.23 | 1.06 | 0.35 | 5.80 | 10.43 | 0.28 |
| Irr_7_F | ~1000 | 4.00 | 0.29 | 0.24 | 0.65 | 0.29 | 4.65 | 7.00 | 0.45 |
| Irr_2_F_R | ~3000 | 4.01 | 0.30 | 0.23 | 0.92 | 0.32 | 5.36 | 8.55 | 0.33 |
| Bi_1_6_R | ~18 | 4.01 | 0.30 | 0.23 | 1.34 | 0.48 | 6.07 | 17.50 | 0.22 |
| Bi_3_6 | ~18 | 3.77 | 0.06 | 0.47 | 1.05 | 0.31 | 6.52 | 22.79 | 0.05 |
| Bi_3_6_1 | ~18 | 3.77 | 0.06 | 0.47 | 1.16 | 0.34 | 6.64 | 21.71 | 0.05 |
| Bi_3_6_2 | ~18 | 3.76 | 0.05 | 0.48 | 1.28 | 0.35 | 6.36 | 19.59 | 0.04 |
| Irr_6_F | ~1000 | 3.77 | 0.06 | 0.47 | 0.65 | 0.19 | 4.68 | 10.05 | 0.09 |

## DISCUSSION OF PRELIMINARY RESULTS

In this section the results for testID Irr_1_F from the test program (Table 1) are depicted as an example of ongoing and further research of the dataset. For the selected test, approximately 1000 irregular waves are generated at a water depth of $h_{paddle}$ = 3.99 m at the paddle and $h_{toe}$ = 0.28 m at the dike toe. According to the geometry (Figure 1) this results in a freeboard of $A_c$ = 0.25 m between SWL and the dike crest. Profile measurements from before and after test execution obtained with the mechanical profiler are compared (Figure 6). The displayed profile is measured at Y = 2.5 m, the centerline in the Delta Flume. Ripple and sand dune formation can be notified for the after test profile. Also, an erosion depth of ~0.15 m at the dike toe and sand accretion at X = 120 m are observed. During the entire test program the extend of the erosion at the dike toe, accretion and formation of sand dunes further offshore and generation of ripples continuously changed depending on the imposed water level and wave conditions. Nevertheless, the erosion depth at the toe is always less than 0.35 m. The changing foreshore profile influences the wave transformation over the foreshore and consequently the wave parameters at the dike toe. When analyzing the overtopping, overtopping flow formation and impact measurements this has to be accounted for.

For the computation of the incident wave parameters with SWASH, the measured bottom profile before each test is used. The comparison of measured and computed wave parameters ($H_{m0}$, $T_{m-1,0}$, set-up) shows good agreement (Figure 7). Only the wave set-up at wave gauge 4 and wave gauge 7 show larger differences. When analyzing the time series, it was found that both wave gauges show significant drift over time. Hence, the computed values are used in the further research. In a next step, the dike was removed in the numerical model simulation to calculate the incident wave parameters, without reflecting dike and wall structure, relevant for the design of coastal structures. The results are given in Table 1. For the offshore wave parameters reflection analysis of the measured signal from wave gauge 1-3 is done and the results are given in Table 1.

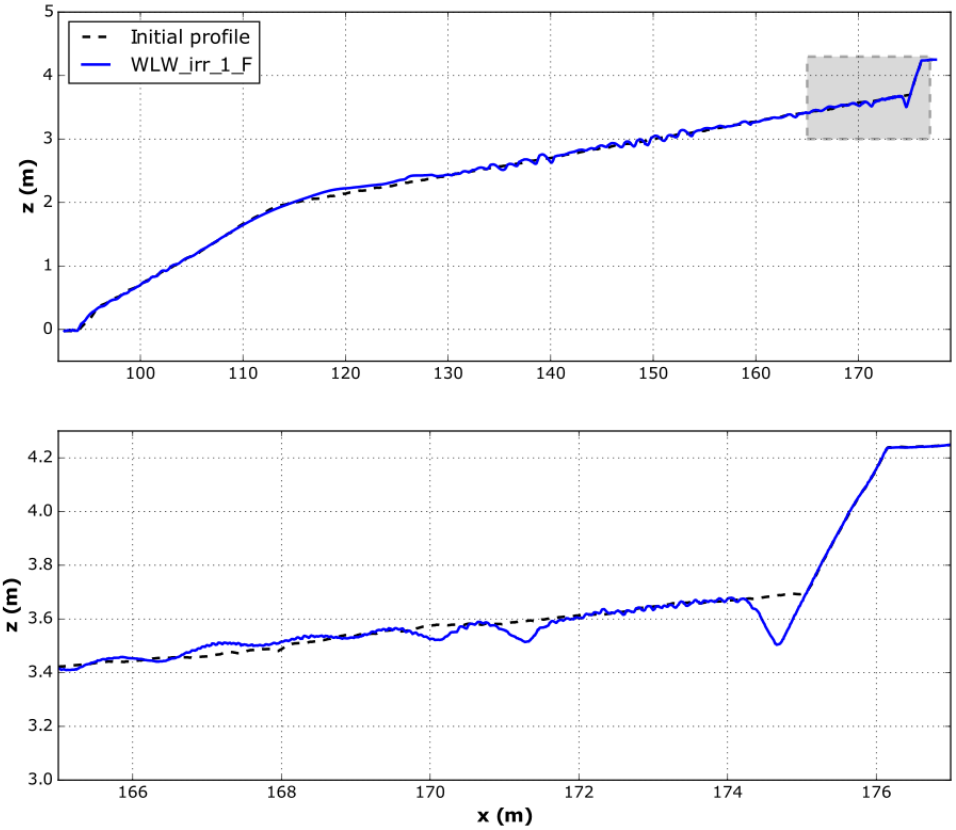

Figure 6: Measured initial bottom profile and profile after the first 1000 waves testID Irr_1_F. The entire profile (top pannel) and zoom on the dike toe area (bottom pannel)

To enable the study of impact loads the signals of load cells and pressure sensors are analyzed. The analysis is carried out in three steps: (1) Post processing of the signal involves removing trends and applying a zero offset correction to the signal. Additionally, a spectral filter with low pass frequency of 100 Hz is applied for the load cell measurement. The bandwidth of the noise in the load cell measurement is generally higher than the pressure sensor measurement and also showed noise peaks at 1.4 Hz and 4.1 Hz, which are removed using a Butterworth type band stop filter. (2) The sum of the two load cells is calculated in order to have the total force on the force measurement section and the result given in kN/m. The pressure sensor signals are integrated over the height of the pressure array using rectangular integration method. The result is given in kN/m as well and can be compared to the force obtained by the load cells. Only the noise around 50 Hz is removed in the pressure sensor signal but no low pass filter is applied to not filter out the short dynamic impacts. (3) The last step involves selecting the key events from the time series, namely indicating the maximum force for each impact event (Figure 8). The minimum time between consecutive impacts is set to 2 s, which is roughly similar to standard impact duration. The smallest impacts which can be determined visually from the time series are checked against the video

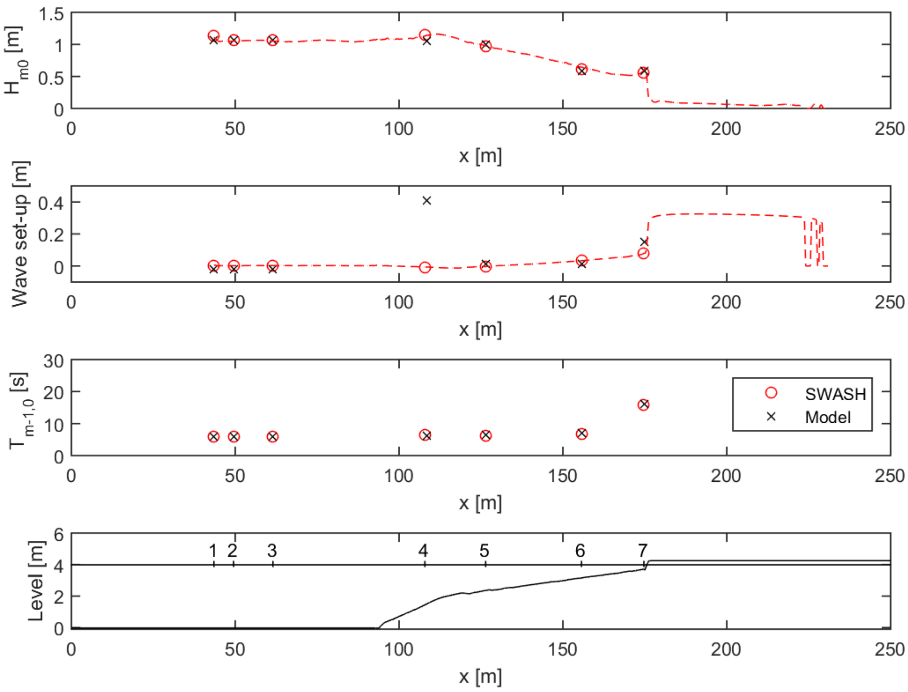


Figure 7: Measured (black crosses) and with SWASH calculated (red circles) wave parameters for the test Irr_1_F (Table 1)

recording and in case a real impact occurs the detection method is tweaked in a way which allows detecting this, very low impacts. This method is preferred over a fixed threshold for both load cell and pressure sensor signals to account for the fact that in the pressure signals smaller impacts are distinguished because of the smaller noise band (Figure 8). It is also visible that the maximum force peak per impact event can occur at different times for pressure sensor and load cell measurement (e.g. impact at 793 s in Figure 8). Also, it is observed that the maximum impact force can be higher for both the load cell (at time 759 s in Figure 8) or the pressure sensor (at time 787 s in Figure 8) measurement. The maximum impact force over the entire test length for load cell measurement and pressure sensor measurement is $F_{max,lc}$ = 4.77 kN/m and $F_{max,ps}$ = 4.54 kN/m respectively. It is assumed that the differences result from non-uniform bore characteristics and impact behavior over the promenade width and the different measurement principles (load cells measuring the total structural response and pressure sensors measuring the local pressure).

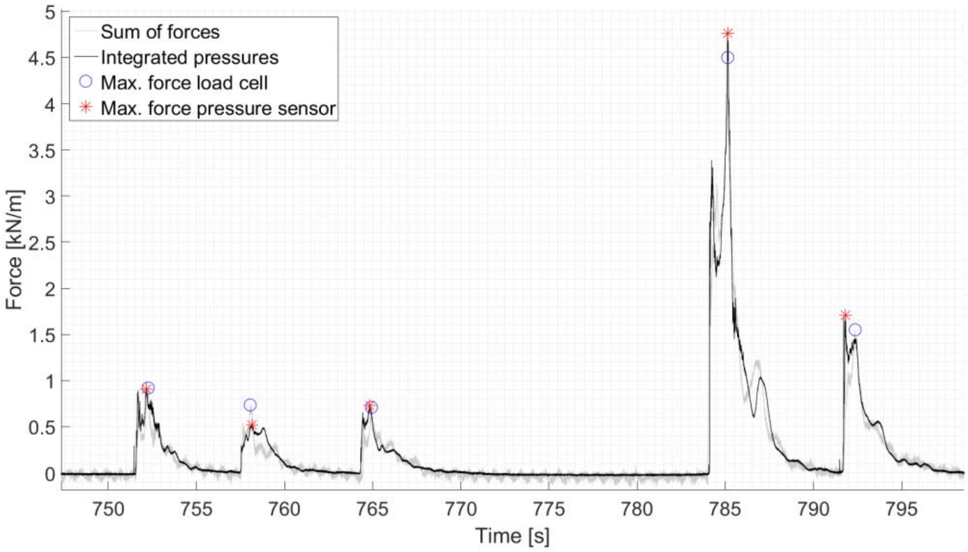

Figure 8: Force time series measured by load cells (grey line) and pressure sensors (black line) and the according maximum impact force (blue and red dots respectively) for testID Irr_1_F (Table 1)

CONCLUSION AND FURTHER RESEARCH

The WALOWA experiments as described above aim for a better understanding of wave impacts on vertical structures on top of a dike, in the specific case of mild foreshore conditions. A model geometry similar to the Belgian coastal profile is built in a 4.3 Froude scale model in the Delta Flume. Wave parameters and water levels for design storm conditions are generated in irregular wave tests with 1000 or 3000 waves. Additionally, bichromatic waves are generated. Measurements of the wave parameters along the foreshore, overtopping flow parameters and impacts of the flow at the wall are taken. The incident wave conditions at the dike toe are computed with SWASH based on the measured bottom profile after the last test and the measured water surface elevation at the offshore location as boundary condition. The computed wave parameters are validated against the measurements. The bottom profile shows erosion at the dike toe, accretion at a location 55 m offshore and several small ripples along the entire profile. The bottom profile varies based on the imposed waves and water levels. Nevertheless, the erosion depth at the dike toe never exceeds a value of 0.35 m. The filtered load cell and integrated pressure sensor signals show qualitatively good comparability. Still the pressure sensor signal shows a smaller noise band and generally is better able to capture fast pressure variations during impact. Hence, they are preferred in further analysis. Future research will focus on the detailed objectives outlined in section 2.

KEY WORDS

physical experiment, large scale, wave impact, overtopped wave load, storm wall, force, pressure, sandy foreshore

ACKNOWLEDGEMENT

The work described in this publication was supported by the European Community's Horizon 2020 Research and Innovation Programme through the grant to HYDRALAB-PLUS, Contract no. 654110.

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