Hydraulic simulators on real dikes and levees

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Abstract

The first part of this chapter gives a short description of wave processes on a dike, on what we know, including recent new knowledge. These wave processes are wave impacts, wave run-up and wave overtopping. The second part focuses on description of three Simulators, each of them simulating one of the wave processes and which have been and are being used to test the strength of grass covers on a dike under severe storm conditions. Sometimes they are also applied to measure wave impacts by overtopping wave volumes.

1 Introduction

When incident waves reach a coastal structure such as dike or levee, they will break if the slope is fairly gentle. This may cause impacts on the slope in zone 2, see Figure 1. When large waves attack such a dike the seaward side in this area will often be protected by a placed block revetment or asphalt. The reason is simple: grass covers cannot withstand large wave impacts, unless the slope is very mild.

Above the impact zone the wave runs up the slope and then rushes down the slope till it meets the next uprushing wave. This is the run-up and run-down zone on the seaward slope (zone 3 in Figure 1). Up-rushing waves that reach the crest will overtop the structure and the flow is only to one side: down the landward slope, see zone's 4 and 5 in Figure 1.

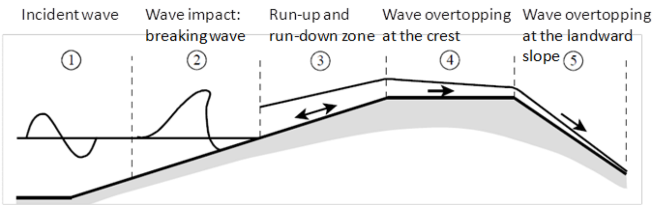

Figure 1. Process of wave breaking, run-up and overtopping at a dike (figure partly from Schüttrumpf (2001)).

Design of coastal structures is often focussed on design values for certain parameters, like the $p_{max,2\%}$ or p_{max} for a design impact pressure, $Ru_{2\%}$ for a wave run-up level and q as mean overtopping discharge or V_{max} as maximum overtopping wave volume. A structure can then be designed using the proper partial safety factors, or with a full probabilistic approach. For wave flumes and wave basins, the waves and the wave processes during wave-structure interaction are simulated correctly using a Froude scale and it are these facilities that have provided the design formulae for the parameters described above.

Whether the strength of coastal structures can also be modelled on small scale depends on the structure considered. The erosion of grass on clay cannot be modelled on a smaller scale and one can only perform resistance testing on real dikes, or on parts moved to a large-scale facility as the Delta Flume of Deltares, The Netherlands, or the GWK in Hannover, Germany. Resistance testing on real dikes can also be performed by the use of Simulators, which is the subject of this chapter. Each Simulator has been developed to simulate only one of the processes in Figure 1 and for this reason three different types of simulator are available today.

If one wants to simulate one of these processes at a real dike, without a wave flume or wave basin, one first has to describe and model the process that should be simulated. Description of the wave-structure-interaction process is, however, much more difficult than just the determination of a design value. The whole process for each wave should be described as good as possible.

2 Simulation of Wave Structure Interaction Processes

2.1 General aspects

Three different wave-structure-interaction processes are being recognized on a sloping dike, each with design parameters, but also with other parameters that have to be described for all individual waves. An overall view is given below.

Impacts: *Design parameters:* $p_{max,\,2\%}$; p_{max}
Description of process: distribution of impact pressures, rise times, impact durations, impact width ($B_{im\text{-}pact,50\%}$) and impact locations;

Wave run-up and run-down:
Design parameters: $Ru_{2\%}$; $Rd_{2\%}$
Description of process: distributions of run-up and run-down levels, velocities along the slope for each wave;

Wave overtopping: *Design parameters:* q; V_{max}
Description of process: distributions of indivi-dual overtopping wave volumes, flow velocities, thicknesses and overtopping durations.

2.2 Wave impacts

A lot of information on wave impacts has been gathered for the design of placed block revetments on sloping dikes. Klein Breteler [2012] gives a full description of wave impacts and a short summary of the most important parameters is given here. Wave impacts depend largely on the significant wave height. For grassed slopes on a dike the wave impact is often limited, say smaller than H_s = 1 m, otherwise the slope would not be able to resist the impacts. Tests from the Delta Flume with a wave height of about 0.75 m have been used to describe the process of wave impacts. The 2%-value of the maximum pressure can be described by [Klein Breteler, 2012]:

$$\left(\frac{p_{max,2\%}}{\gamma_{berm,p_{max}} H_s}\right)\left(\frac{\rho_w g H_s^2}{\sigma_w}\right)^{0.1} = 12 - 0.28\frac{\xi_{op}}{tan\alpha_T}$$

for $3 \le \frac{\xi_{op}}{tan\alpha_T} \le 24$ (1)

with $\gamma_{berm,p_{max}} = 0.17\left(\frac{h_b}{H_s} - 1.2\right)^2 + 1$

where:

g = acceleration of gravity [m/s²]
H_s = significant wave height [m]
h_b = vertical distance from swl to berm (positive if berm above swl) [m]
$p_{max,\,x\%}$ = value which is exceeded by x% of the number of wave impacts related to the number of waves [m water column]
α_T = slope angle [°]
$\gamma_{berm,\,pmax}$ = influence factor for the berm [-]
ρ_w = density of water [kg/m³]
σ_w = surface tension [0.073 N/m²]
ξ_{op} = breaker parameter using peak period T_p [-]

The tests in the Delta Flume clearly showed that the distribution of p is Rayleigh distributed, see Figure 2. The graph has the horizontal axis according to a Rayleigh distribution and a more or less straight line then indicates a Rayleigh distribution. This is indeed the case in Figure 2.

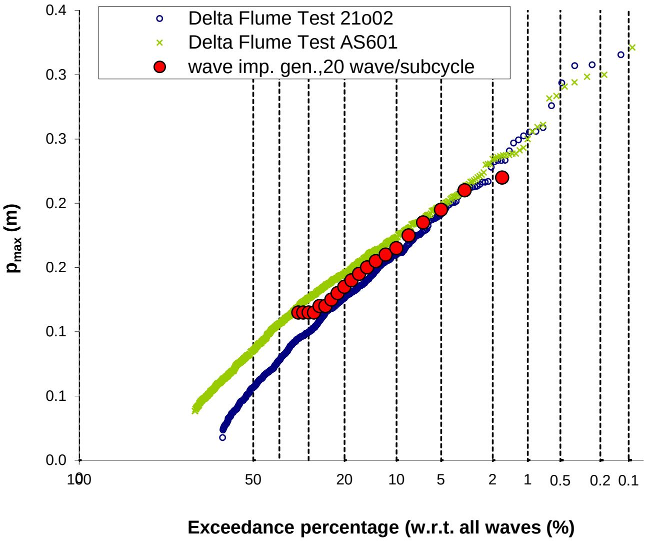

Figure 2. Peak pressures of impacts, measured in the Delta Flume and given on Rayleigh paper. Also simulated pressures are shown (described later in the chapter)

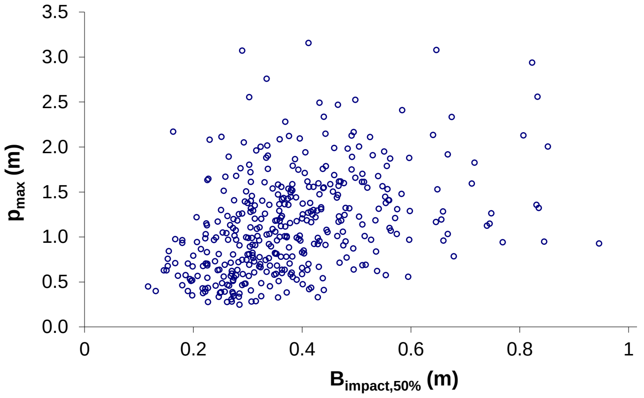

Figure 3. Peak pressures of impacts versus the width of the impacts (Delta flume measurements [Klein Breteler, 2012]).

Each parameter can be given as a distribution or exceedance curve, but often the relationship between two parameters is not so straight forward. Figure 3 shows the relationship between the peak pressure and the corresponding width of the impact, $B_{impact,\,50\%}$ for a wave field with $H_s \approx 0.75$ m. It shows that peak pressures may give values between 0.25 and 3 m water column, whereas the width of impact may be between 0.15 and

1 m, with an average value around 0.4 m. But there is hardly any correlation between both parameters.

2.3 Wave run-up and run-down

The engineering design parameter for wave run-up is the level on the slope that is exceeded by 2% of the up-rushing waves ($Ru_{2\%}$). The EurOtop Manual [2007] gives methods to calculate the overtopping discharge as well as the 2% run-up level for all kinds of wave conditions and for many types of coastal structures. Knowing the 2% run-up level for a certain condition is the starting point to describe the wave run-up process. Assuming a Rayleigh distribution of the run-up levels and knowing $Ru_{2\%}$ gives all the required run-up levels. As the EurOtop Manual [2007] is readily available, formulae for wave run-up have not been repeated here.

The wave run-up level is a start, but also run-up velocities and flow thicknesses are required. From the wave overtopping tests it is known that the *front velocity* is the governing parameter in initiating damage to a grassed slope. Focus should therefore be on describing this front velocity along the upper slope. By only considering random waves and the 2%-values, the equations for run-up velocity and flow thickness become:

$$u_{2\%} = c_{u2\%}(g(Ru_{2\%} - z_A))^{0.5} \qquad (2)$$

$$h_{2\%} = c_{h2\%}(Ru_{2\%} - z_A) \qquad (3)$$

where:

- $u_{2\%}$ = run-up velocity exceeded by 2% of the up-rushing waves
- $c_{u2\%}$ = coefficient
- g = acceleration of gravity
- $Ru_{2\%}$ = maximum level of wave run-up related to the still water level swl
- z_A = location on the seaward slope, in the run-up zone, related to swl
- $h_{2\%}$ = flow thickness exceeded by 2% of the up-rushing waves
- $c_{h2\%}$ = coefficient

The main issue is to find the correct values of $c_{u2\%}$ and $c_{h2\%}$. But comparing the results of various research studies [Van der Meer *et al.*, 2012] gives the conclusion that they are not consistent. The best conclusion at this moment is to take $c_{h2\%}$ = 0.20 for slopes of 1:3 and 1:4 and $c_{h2\%}$ = 0.30 for a slope of 1:6. Consequently, a slope of 1:5 would then by interpolation give $c_{h2\%}$ = 0.25. This procedure is better than to use a formula like $c_{h2\%}$ = 0.055 cotα, as given in EurOtop [2007]. One can take $c_{u2\%}$ = 1.4-1.5 for slopes between 1:3 and 1:6.

Moreover, the general form of Equation 2 for the maximum velocity somewhere on a slope, may differ from the front velocity of the up-rushing wave. Van der Meer [2011] analyzed individual waves rushing up the slope. Based on this analysis the following conclusion on the location of maximum or large velocities and front velocities in the run-up of waves on the seaward slope of a smooth dike can be drawn, which is also shown graphically in Figure 4.

In average the run-up starts at a level of 15% of the maximum run-up level, with a front velocity close to the maximum front velocity and this velocity is more or less constant until a level of 75% of the maximum run-up level. The real maximum front velocity in average is reached between 30%-40% of the maximum run-up level. Figure 4 also shows that a square root function as assumed in Eq. 2, which is valid for a maximum velocity at a certain location (not the front velocity) is different from the front velocity. The process of a breaking and impacting wave on the slope has influence on the run-up, it gives a kind of acceleration to the up-rushing water. This is the reason why the front velocity is quite constant over a large part of the run-up area.

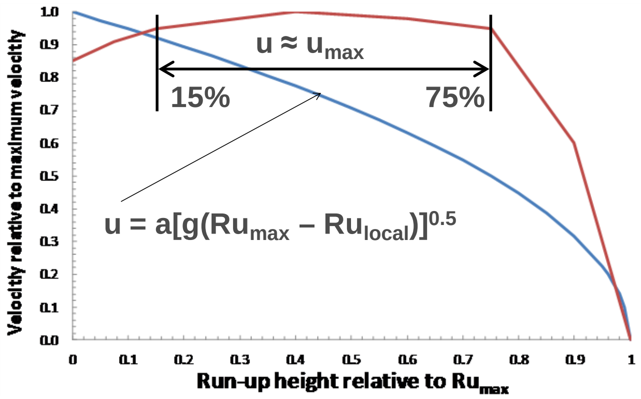

Figure 4. General trend of front velocity over the slope during up-rush, compared to the theoretical maximum velocity at a certain location.

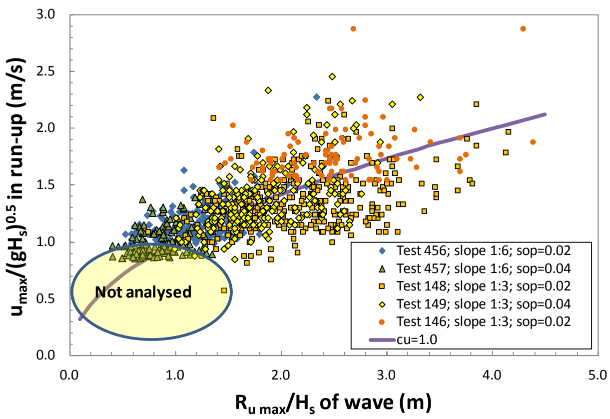

Figure 5. Relative maximum front velocity versus relative run-up on the slope; all tests.

Further analysis showed that there is a clear trend between the maximum front velocity in each up-rushing wave and the (maximum) run-up level itself, although there is considerable scatter. Figure 5 shows the final overall figure (detailed analysis in Van der Meer, [2011]), where front velocity and maximum run-up level of each wave were made dimensionless. Note that only the largest front velocities have been analysed and that the lower left corner of the graph in reality has a lot of data, but less significant with respect to effect on a grassed slope.

The trend and conclusion in Figure 4 explains for a part why the relationship between the maximum front velocity and the maximum run-up in Figure 5 gives a lot of scatter. A front velocity close to the maximum velocity is present over a large part of the slope and the actual location of the maximum velocity may be more or less "by accident". The trend given in Figure 5 can be described by:

$$u_{max}/\sqrt{(gH_s}=c_u\sqrt{Ru_{max}/H_s} \quad (4)$$

with c_u as stochastic variable ($\mu(c_u)$ = 1.0, a normal distribution with coefficient of variation CoV = 0.25).

2.4 Wave overtopping

Like for wave run-up the EurOtop Manual [2007] gives the formulae for also for mean wave overtopping. This is the governing design parameter, which will not be repeated here. In reality there is no mean discharge, but several individual waves overtopping the structure, each with a certain overtopping volume, V. Recent improvements in describing wave overtopping processes have been described by Hughes *et al.*, [2012] and Zanuttigh *et al.*, [2013]. The distribution of individual overtopping wave volumes can well be represented by the two parameter Weibull probability distribution, given by the percent exceedance distribution in Equation 5.

$$P_{V\%} = P(V_i \geq V) = \exp\left[-\left(\frac{V}{a}\right)^b\right] \cdot (100\%) \quad (5)$$

where P_V is the probability that an individual wave volume (V_i) will be less than a specified volume (V), and $P_{V\%}$ is the percentage of wave volumes that will exceed the specified volume (V). The two parameters of the Weibull distribution are the non-dimensional shape factor, b, that helps define the extreme tail of the distribution and the dimensional scale factor, a, that normalizes the distribution.

$$a = \left(\frac{1}{\Gamma(1+\frac{1}{b})}\right)\left(\frac{qT_m}{P_{ov}}\right) \quad (6)$$

where Γ is the mathematical gamma function.

Zanuttigh *et al.*, [2012] give for b the following relationship (Fig. 6):

$$b=0.73+55\left(\frac{q}{gH_{m0}T_{m-1,0}}\right)^{0.8} \quad (7)$$

Figure 6 shows that for a relative discharge of $q/(gH_{m0}T_{m-1,0}) = 5.10^{-3}$ the average value of b is about 0.75 and this value has long been used to describe overtopping of individual wave volumes (as given in EurOtop, [2007]). But the graph shows that with larger relative discharge the b-value may increase significantly, leading to a gentler distribution of overtopping wave volumes. This new knowledge may have effect on design and usage of wave overtopping simulators.

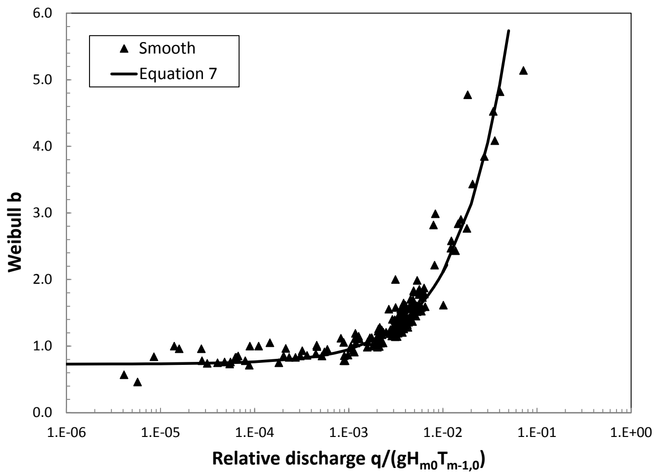

Figure 6. New Weibull shape factor, b, spanning a large range of relative freeboards (Zanuttigh et al., [2013].

3 Simulators as hydraulic test facilities

In total three types of Simulators have been developed, on impacts, run-up and on overtopping. The principle is similar for all three types: a box with a certain geometry is constantly filled with water by a (large) pump. The box is equipped with one or more valves to hold and release the water and has a specifically designed outflow device to guide the water in a correct way to the slope of the dike. By changing the released volume of water from the box one can vary the wave-structure-interaction properties.

3.1 Wave impacts

The Wave Impact Generator is a development under the WTI 2017-program of the Dutch Rijkswaterstaat and Deltares, see Figure 7. This tool is called a generator and not a simulator. It has been developed late 2011 and in 2012 and testing has been performed the first and second half of 2012. It is a box of 0.4 m wide, 2 m long and can be up to 2 m high (modular system). It has a very advanced system of two flap valves of only

0.2 m wide, which open in a split second and which enables the water to reach the slope at almost the same moment over the full width of 0.4 m and thus creating a nice impact. Measured impacts are given in Figure 2 and compared with impacts measured in the Delta Flume.

Figure 7. Test with Wave Impact Generator.

As the location of impacts varies on the slope, the Wave Impact Generator has been attached to a tractor or excavator, which moves the simulator a little up and down the slope. In this way the impacts do not occur all at the same location. Development and description of first tests have been described by Van Steeg [2012a, 2012b and 2013]. The main application is simulation of wave impacts on grassed slopes of dikes, like for river dikes, where the wave heights are limited to H_s = 0.5 - 1 m. The impact pressures to be simulated are given by Eq. 1, but within the range of wave heights given here. The impact pressure can be regulated by the empirically determined formula:

$$p_{max} = 1.10h_w + 0.87 \qquad (8)$$

where h_w is the water column in the box, with p_{max} measured in m water column. This relation has been calibrated for 0.25 m < h_w < 1.0 m. In fact only the largest 30% of the wave impacts is simulated, see also Figure 2.

Figure 8. Failed road crossing by under-mining due to simulated wave impacts.

Slopes with various quality of grass as well as soil (clay and sand) have been tested as well as a number of transitions, which are often found in dikes and which in many cases fail faster than a grassed slope. Figure 8 gives an impression of a road crossing of open tiles, which failed by undermining due to simulated wave impacts.

3.2 Wave run-up and run-down

The process of run-up was explored, see Section 2.3, as well as a procedure for testing was developed [Van der Meer, 2011] and [Van der Meer *et al.*, [2012]. Then in 2012 a pilot test was performed on wave run-up simulation, but using the existing Wave Overtopping Simulator as an existing tool (description in the next section). The Simulator was placed on a seaward berm and run-up levels were calibrated with released wave volumes and these were used for steering the process. In this way the largest run-up levels of a hypothetical storm and storm surge, which would reach the upper slope above the seaward berm, were simulated. Figure 9 gives the set-up of the pilot test and shows a wave run-up that even reached the crest, more than 3 m higher than the level of the Simulator. An example on damage developed by simulating wave run-up is shown in Figure 10. The up-rushing waves meet the upper slope of the dike and "eat" into it.

Figure 9. Set-up of the pilot wave run-up test at Tholen, using the existing Wave Overtopping Simulator.

Figure 10. Final damage after the pilot run-up test.

The pilot test gave valuable information on how testing in future could be improved, but also how a real Wave Run-up Simulator should look like. A Wave Run-up Simulator should have a slender shape, different from the present Wave Overtopping Simulator, in order to release less water, but with a higher velocity. At the end of 2013 such a new device was designed, constructed and tested. And before spring 2014 the first tests on the upper slope of the seaward part of a sea dike was tested. The box had a cross-section at the lower part of 0.4 m by 2 m, giving a test section of 2 m wide, see Figure 11. The upper part had a cross-section of 0.8 m by 1.0 m and this change was designed in order to have less wind forces on the Simulator. The cross-sectional area was the same over the full height of the Simulator in order not to have dissipation of energy during release of water. The overall height is more than 8 m.

Figure 11. The new Wave Run-up Simulator, designed in 2013.

A new type of valve was designed to cope with the very high water pressures (more than 7 m water column). A drawer type valve mechanism was designed with two valves moving horizontally over girders. In this way leakage by high water pressures was diminished as higher pressures gave a higher closing pressure on the valves. This new Wave Run-up Simulator was calibrated against a 1:2.7 slope. The largest run-up was about 13.5 m along the slope, this is about 4.7 m measured vertically. Besides transitions from down slope to berm and berm to upper grassed slope, also a stair case was tested by wave run-up, see Figure 2. As in many other tests, with as well impact or overtopping waves, a stair case is always a weak point in a dike.

Figure 12. Testing a stair case with the new Wave Run-up Simulator in 2014.

3.3 Wave overtopping

The Wave Overtopping Simulator has been designed and constructed in 2006 and has been used since then for destructive tests on dike crest and landward slopes of dikes or levees under loading of overtopping waves. References are Van der Meer *et al.*, [2006, 2007, 2008, 2009, 2010, 2011, 2012], Akkerman *et al.*, [2007], Steendam *et al*,. [2008, 2010, 2011] and Hoffmans *et al*,. [2008], including development of Overtopping Simulators in Vietnam [Le Hai Trung *et al*,. 2010] and in the USA [Van der Meer *et al*., 2011] and [Thornton *et al*., 2011].

The setup of the Overtopping Simulator on a dike or levee is given in Figure 13, where the Simulator itself has been placed on the seaward slope and it releases the overtopping wave volume on the crest, which is then guided down the landward side of the dike. Water is pumped into a box and released now and then through a butterfly valve, simulating an overtopping wave volume. Electrical and hydraulic power packs enable pumping and opening and closing of the valve. A measuring cabin has been placed close to the test section. The Simulator is 4 m wide and has a maximum capacity of 22 m^2, or 5.5 m^3 per m width. The Simulator in Vietnam has the same capacity, but the Simulator in the US has a capacity of 16 m^3 per m width (although over a width of 1.8 m instead of 4 m). Released volumes in a certain time are according to theoretical distributions of overtopping wave volumes, as described in this chapter, depending on assumed wave conditions at the sea side and assumed crest freeboard.

Figure 13. Set-up of the Wave Overtopping Simulator close to a highway.

Figure 14 shows the release of a large overtopping wave volume and Figure 15 shows one of the many examples of a failed dike section, here a sand dike covered with good quality grass.

Figure 14. Release of a large wave volume.

Figure 15. Failure of a sand dike.

3.4 Wave impacts by wave overtopping

One application of a Hydraulic Simulator is to test the resistance of a grass dike by destructive testing, as described in Sections 3.1-3.3. Another application came up different from destructive testing and that is the simulation of wave impacts and measurement of pressures and forces on a structure. The impacts were not generated by wave breaking, like for the Wave Impact Simulator, but by overtopping wave volumes. Two examples will be given here.

The relatively short Belgian coast has a sandy foreshore, protected by a sloping seawall, a promenade and then apartments. In order to increase safety against flooding, vertical storm walls were designed on the promenade. Under design conditions waves would break on the sloping revetment. giving large overtopping waves that travelled some distance over the promenade before hitting the storm wall. Impacts in small scale model investigations may differ significantly from the real situation with larger waves and often salt water (different behaviour of air bubbles compared with fresh water). A full scale test was set-up for the Belgian situation, see Figure 16, with the Wave Overtopping Simulator releasing the flow of overtopping wave volumes over a horizontal distance on to vertical plates where forces as well as pressures could be measured. The tests and results have been described in Van Doorslaer *et al.*, [2012].

Figure 16. Measuring impacts by overtopping waves on a promenade towards a vertical storm wall.

Figure 17. Measuring impacts by overtopping waves on a quay area towards a vertical step of a stair case type structure.

In Den Oever, The Netherlands, a 300 m long dike improvement was designed with the shape of a stair case. The steps were 0.46 m high (sitting height) and 2 m wide and the total structure had four steps. The design wave height was about 1.35 m, which broke over a 6.5 m quay area with the crest at the design water level.

The overtopping wave front hit the front side of the stair case type structure, giving very high impacts in a small scale model investigation. The new Wave Run-up Simulator was used to simulate similar impacts, but now on full scale and with salt water. Figure 17 shows the impact of a wave on the lower step of the stair case (the other steps were not modelled).

4 Summary and discussion

Erosion of grassed slopes by wave attack is not easy to investigate as one has to work at real scale, due to the fact that the strength of clay with grass roots cannot be scaled down. There are two ways to perform tests on real scale: bring (pieces of) the dike to a large scale facility that can produce significant wave heights of at least 1 m, or bring (simulated) wave attack to a real dike. For investigation in a large scale facility the main advantage will be that the waves are generated well and consequently also the wave-structure-interaction processes are generated well. The disadvantage is that the modelled dike has to be taken from a real dike in undisturbed pieces. This is difficult and expensive and real situations on a dike, like staircases, fences and trees are almost impossible to replicate. This type of research is often focussed on the grass cover with under laying clay layer only.

The second alternative of Simulators at a dike has the significant advantage that real and undisturbed situations can be investigated. The research on wave overtopping has already given the main conclusion that *it is not the grass cover itself that will lead to failure of a dike by overtopping*, but an obstacle (tree; pole; staircase) or transition (dike crossing; from slope to toe or berm). The main disadvantage of using Simulators is that only a part of the wave-structure-interaction can be simulated and the quality of this simulation depends on the knowledge of the process to simulate and the capabilities of the device. The experience of testing with the three Simulators, on wave impacts, wave run-up and wave overtopping, gave in only seven years a tremendous increase in knowledge of dike strength and resulted in predictive models for safety assessment or design.

By simulating overtopping waves it is also possible to measure impact pressures and forces on structures that are hit by these overtopping waves. Such a test will be at full scale and if necessary with salt water, giving realistic wave impacts without significant scale or model effects.

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References

Akkerman, G.J., P. Bernardini, J.W. van der Meer, H. Verheij and A. van Hoven (2007). *Field tests on sea defences subject to wave overtopping.* Proc. Coastal Structures, Venice, Italy.

EurOtop (2007). *European Manual for the Assessment of Wave Overtopping.* Pullen, T. Allsop, N.W.H. Bruce, T., Kortenhaus, A., Schüttrumpf, H. and Van der Meer, J.W. www.overtopping-manual.com.

Hoffmans, G., G.J. Akkerman, H. Verheij, A. van Hoven and J.W. van der Meer (2008). *The erodibility of grassed inner dike slopes against wave overtopping.* ASCE, Proc. ICCE 2008, Hamburg, 3224-3236.

Hughes, S, C. Thornton, J.W. van der Meer and B. Scholl (2012). *Improvements in describing wave overtopping processes*. ASCE, Proc. ICCE 2012, Santander, Spain.

Klein Breteler, M., van der Werf, I., Wenneker, I., (2012), *Kwantificering golfbelasting en invloed lange golven. In Dutch. (Quantification of wave loads and influence of long waves),* Deltares report H4421, 1204727, March 2012

Le Hai Trung, J.W. van der Meer, G.J. Schiereck, Vu Minh Cath and G. van der Meer. 2010. *Wave Overtopping Simulator Tests in Vietnam.* ASCE, Proc. ICCE 2010, Shanghai.

Schüttrumpf, H.F.R. 2001. *Wellenüberlaufströmung bei See-deichen*, Ph.D.-th. Techn. Un. Braunschweig.

Steendam, G.J., W. de Vries, J.W. van der Meer, A. van Hoven, G. de Raat and J.Y. Frissel. 2008. *Influence of management and maintenance on erosive impact of wave overtopping on grass covered slopes of dikes; Tests.* Proc. FloodRisk, Oxford, UK. Flood Risk Management: Research and Practice – Samuels et al. (eds.) ISBN 978-0-415-48507-4; pp 523-533.

Steendam, G.J., J.W. van der Meer, B. Hardeman and A. van Hoven. 2010. *Destructive wave overtopping tests on grass covered landward slopes of dikes and transitions to berms*. ASCE, Proc. ICCE 2010.

Steendam, G.J., P. Peeters., J.W. van der Meer, K. Van Doorslaer, and K. Trouw. 2011. *Destructive wave overtop-*

ping tests on Flemish dikes. ASCE, Proc. Coastal Structures 2011, Yokohama, Japan.

Thornton, C., J.W. van der Meer and S.A. Hughes. 2011. *Testing levee slope resiliency at the new Colorado State University Wave Overtopping Test Facility*. Proc. Coastal Structures 2011, Japan.

Van der Meer, J.W., P. Bernardini, W. Snijders and H.J. Regeling. 2006. *The wave overtopping simulator*. ASCE, ICCE 2006, San Diego, pp. 4654 - 4666.

Van der Meer, J.W., P. Bernardini, G.J. Steendam, G.J. Akkerman and G.J.C.M. Hoffmans. 2007. *The wave overtopping simulator in action*. Proc. Coastal Structures, Venice, Italy.

Van der Meer, J.W., G.J. Steendam, G. de Raat and P. Bernardini. 2008. *Further developments on the wave overtopping simulator*. ASCE, Proc. ICCE 2008, Hamburg, 2957-2969.

Van der Meer, J.W., R. Schrijver, B. Hardeman, A. van Hoven, H. Verheij and G.J. Steendam. 2009. *Guidance on erosion resistance of inner slopes of dikes from three years of testing with the Wave Overtopping Simulator*. Proc. ICE, Coasts, Marine Structures and Breakwaters 2009, Edinburgh, UK.

Van der Meer, J.W., B. Hardeman, G.J. Steendam, H. Schttrumpf and H. Verheij. 2010. *Flow depths and velocities at crest and inner slope of a dike, in theory and with the Wave Overtopping Simulator.* ASCE, Proc. ICCE 2010, Shanghai.

Van der Meer, J.W., C. Thornton and S. Hughes. 2011. *Design and operation of the US Wave Overtopping Simulator.* ASCE, Proc. Coastal Structures 2011, Yokohama, Japan.

Van der Meer, J.W. 2011. *The Wave Run-up Simulator. Idea, necessity, theoretical background and design.* Van der Meer Consulting Report vdm11355.

Van der Meer, J.W., Y. Provoost and G.J. Steendam (2012). *The wave run-up simulator, theory and first pilot test.* ASCE, Proc. ICCE 2012, Santander, Spain.

Van Doorslaer, K., J. De Rouck, K. Trouw, J.W. van der Meer and S. Schimmels. *Wave forces on storm walls, small and large scale experiments.* Proc. COPEDEC 2012, Chennai, India

Van Steeg, P., (2012a). *Reststerkte van gras op rivierdijken bij golfbelasting. SBW onderzoek. Fase 1a: Ontwikkeling golfklapgenerator. In Dutch (Residual strength of grass on river dikes under wave attack. SBW research Phase 1a: Development wave impact generator).* Deltares report 1206012-012-012-HYE-0002, April 2012

Van Steeg, P., (2012b). *Residual strength of grass on river dikes under wave attack. SBW research Phase 1b: Development of improved wave impact generator.* Deltares report 1206012-012-012-HYE-0015 August 2012

Van Steeg, P., (2013) *Residual strength of grass on river dikes under wave attack. SBW research Phase 1c: Evaluation of wave impact generator and measurements based on prototype testing on the Sedyk near Oosterbierum.* Deltares report 1207811-008.

Zanuttigh, B., J.W. van der Meer and T. Bruce, 2013. *Statistical characterisation of extreme overtopping wave volumes*. Proc. ICE, Coasts, Marine Structures and Breakwaters 2013, Edinburgh, UK.