Guidance on erosion resistance of inner slopes of dikes from 3 years of testing with the Wave Overtopping Simulator

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Introduction

The Wave Overtopping Simulator was developed in 2006 and destructive tests have been performed in February and March of 2007, 2008 and 2009 and will probably be continued in 2010. The tests show the behaviour of various inner slopes of dikes, embankments or levees under simulation of wave overtopping, up to a mean overtopping discharge of 125 l/s per m. In 2010 a Technical Report on strength of inner slopes of dikes against wave overtopping will be written, leading to new guidelines for the required five-yearly safety assessment of flood defence assets in the Netherlands.

This paper will give a mid-term review and first guidance, based on 3 years of destructive testing. Till summer 2009 15 sections of dikes at 5 different locations in the Netherlands have been tested. The paper will give guidance to practical engineers, based on observations and analysis from all the testing so far. It also discusses the possible modifications in safety assessment.

The Wave Overtopping Simulator

The process of wave overtopping on a dike, levee, seawall or embankment is well known, see the new Overtopping Manual, 2007. In contrast, the erosive impact of wave overtopping on these structures is not known well, mainly due to the fact that research on this topic can not be performed on a small scale, as it is practically impossible to scale clay and grass down properly. Therefore, the Wave Overtopping Simulator has been developed, see Van der Meer et al. (2006, 2007 and 2008) for more details.

The Simulator consists of a high-level mobile box to store water. The maximum capacity is 5.5 m^3 per m width (22 m^3 for a 4 m wide Simulator). This box is continuously filled with a predefined discharge and emptied at specific times through a butter-fly valve in such a way that it simulates the overtopping tongue of a wave at the crest and inner slope of a dike. The discharge of water is released in such a way that for each overtopping volume of water the flow velocity and thickness of the water tongue at the crest corresponds with the characteristics that can be expected. See Figure 1 for the principle of the Wave Overtopping Simulator. Various overtopping volumes are released randomly in time, see Figure 2.

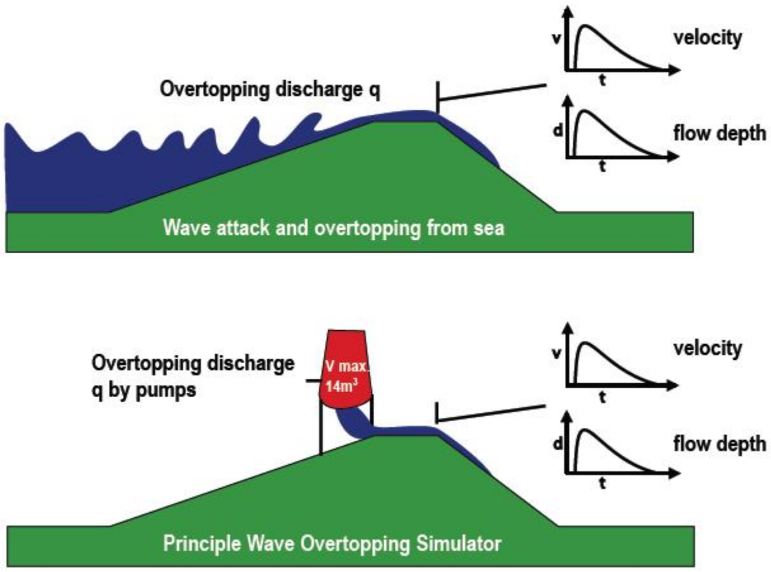

Figure 1. Principle of Wave Overtopping Simulator

Figure 2. Release of a wave

Figure 3. Set-up of Wave Overtopping Simulator close to a highway (February 2009)

Field tests on real dikes have been carried out in 2007, 2008 and 2009, all at the end of the winter when grass roots are in worst condition. Figure 3 shows the set-up of the simulator at the crest and seaward side of a dike and very close to a highway. The design and calibration of the Wave Overtopping Simulator has been described by Van der Meer (2007) and the test results of the resistance of the first tested dike have been described by Akkerman et al. (2007-1 and 2007-2). Part of the tests in 2008 have been described by Steendam et al. (2008). A summary report on all the testing in 2007 and 2008, with many pictures of damages and with preliminary conclusions, has been described by Van der Meer (2008). Evaluation of the tests in 2009 are underway and have partly been described in this paper.

Each test condition was given by a mean discharge and lasted for 6 hours. Test conditions increased from 0,1 l/s per m to 1; 10; 30; 50 and 75 l/s per m. A full test on a dike section took about one week and often more than 14,000,000 litres of water flowed over the inner slope of 4 m width. Each test condition consisted of simulation of the required distribution of overtopping volumes (see the Overtopping Manual (2007)). Such a distribution depends on expected conditions at sea: a larger significant wave height (as at sea dikes) will show less overtopping waves, but the volume in the overtopping waves will be bigger than for a smaller wave height (as for example at river dikes). All tests till now have assumed a significant wave height of 2 m with a wave steepness of 0.04 (using the peak period). Distributions of overtopping volumes for this condition and for various mean discharges are given in Figure 4.

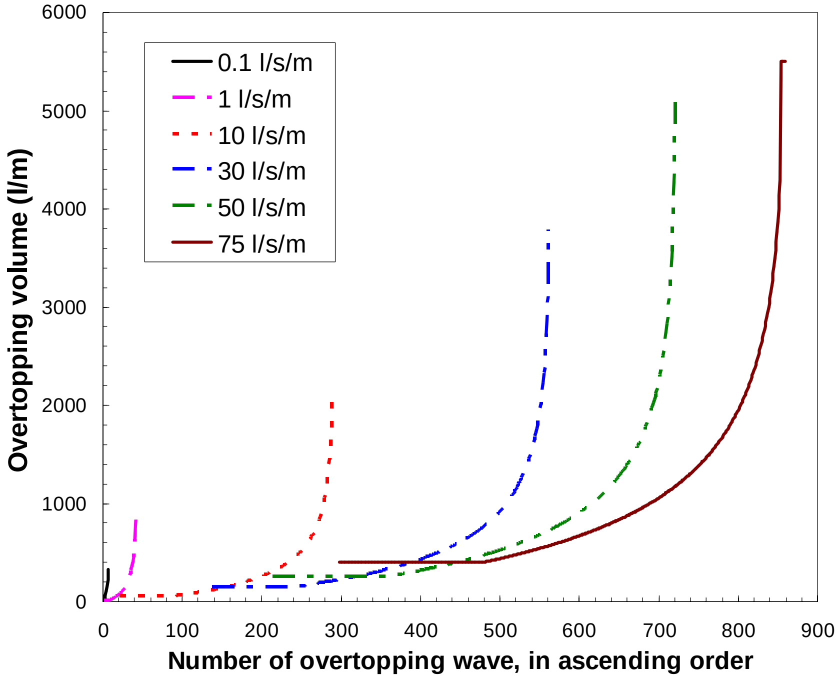

Figure 4. Distribution of overtopping volumes of waves for sea dikes and various mean overtopping discharges, as simulated by the Wave Overtopping Simulator.

Figure 4 clearly shows that for each mean discharge there are only a small number of waves that give large overtopping volumes. The general behaviour of wave overtopping can be described by a large number of fairly small overtopping waves and a few which are much bigger. These few but bigger waves often cause the damage to the inner slope.

In the first years of testing it appeared to be very difficult to measure any hydraulic parameter on the inner slope, like flow velocity or flow depth. The velocities can approach 10 m/s and the water is very turbulent with a lot of air entrainment, see also Figure 2. Laboratory instruments have not been designed for this kind of conditions. In 2009 a lot of attention was focussed on improving the measurements. Amongst them a floating device to measure the flow depth and a high speed camera to measure front velocities of an overtopping wave. Figure 5 shows the record of this floating device for three consecutive overtopping volumes of 3.0 m^3 per m width each. Recording started exactly when the signal was given to open the valve. The overtopping volumes and the records of flow depth reproduce very nicely. Maximum flow depth is about 0.25 m and it takes 1.6 s for the water to flow from the valve to the device, which gives over a distance of about 8-9 m a mean front velocity of about 5.0-5.6 m/s.

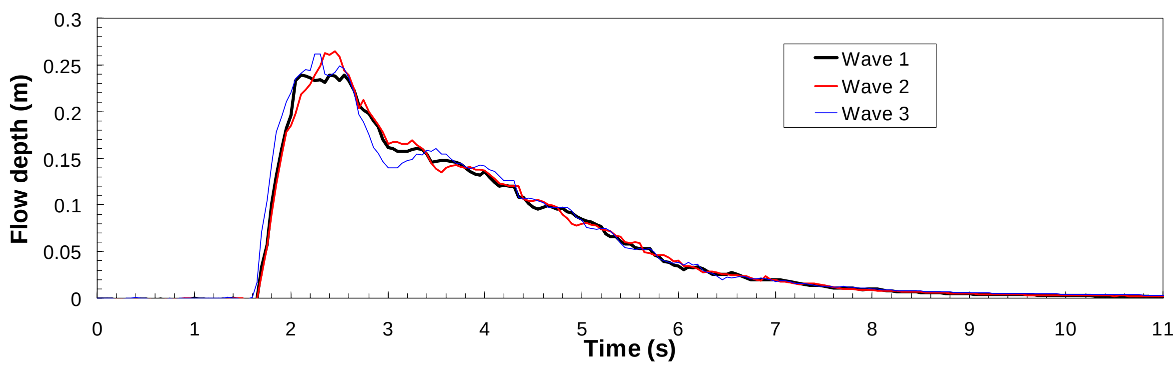

Figure 5. Record of flow depth with floating device for 3 overtopping waves of 3 m^3/m

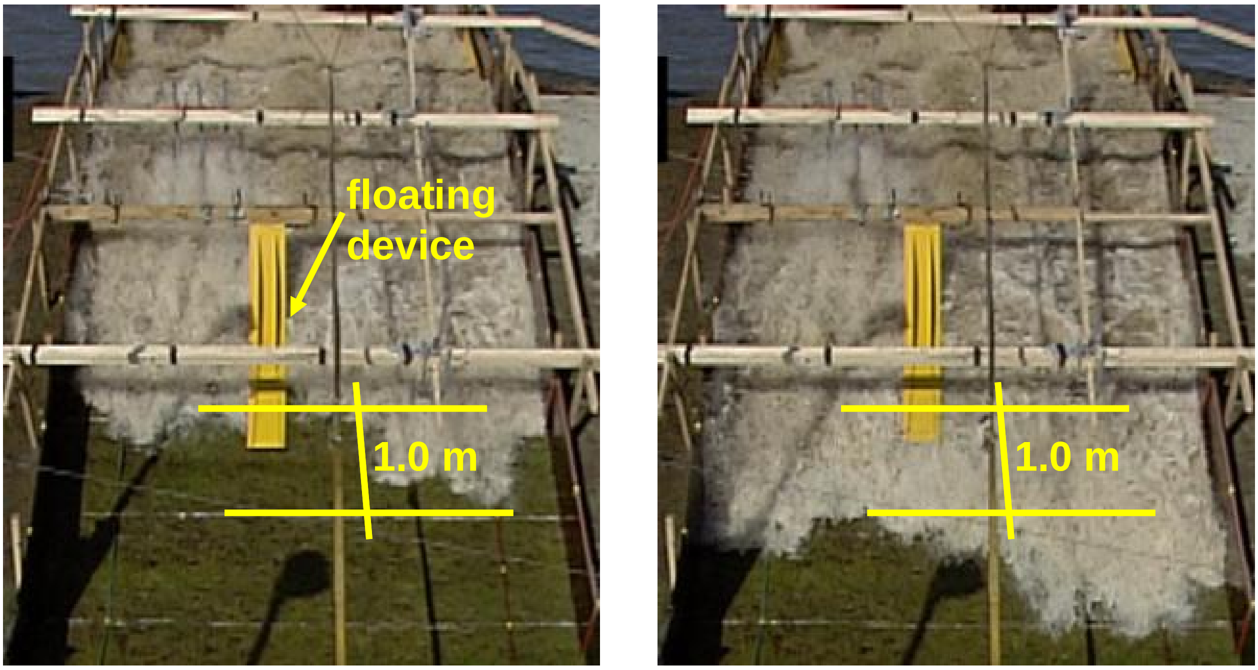

Figure 6. Two pictures of the high speed camera, where the front of the wave travelled 1.0 m in 0.18 s, giving a front velocity of 5.5 m/s

Figure 6 shows two pictures of the high speed camera, which took 50 frames per second. Contours with a distance of 1.0 m had been drawn on the grass of the inner slope. The time difference between the two pictures is 0.18 s, giving a front velocity of 5.5 m/s. This agrees well with the 5.0-5.6 m/s from the valve to the floating device, where the velocity was zero when the valve opened.

The released volume in the example above was 3.0 m^3 per m. A recalculation of the released volume can also be derived by integrating the flow depth record over time and multiplying it with the velocity. By assuming that the front velocity is similar to the mean velocity for about 4 or 5 seconds, the integrated signal of the flow depth can simply be multiplied by the front velocity. The flow depth of wave 1 was integrated over time from 1.6 s to 8 s. After this time it is assumed that the flow velocity for this small flow depth will be much smaller than the measured flow velocity. The integration gave a released volume of 3.35 m^3 per m. This matches very well with the actual released volume as air entrainment is measured by the floating device and a larger volume than 3 m^3 per m can be expected. If, however, it is assumed that flow velocity and flow depth have the same *shape of record in time*, which might not be true, than the calculated volume becomes 2.1 m^3/m. More analysis is underway.

Destructive field tests

Failure mechanisms

Wave overtopping may lead to failure of the crest and inner slope of a dike. In principle there are two different failure mechanisms. Fast overtopping water may damage the surface of the crest and inner slope and, if initial damage or erosion has occurred, this may continue to the layer underneath the grass cover and may lead to an initial breach. This is actually the process which is simulated by the Wave Overtopping Simulator: erosion of the slope.

A major failure mechanism on steep inner faces (typically 1:1.5 and 1:2) in the past was slip failure of the (rear) slope. Such slip failures may lead directly to a breach. For this reason most dike designs in the Netherlands in the past fifty years have used a 1:3 inner slope, where it is unlikely that slip failures will occur due to overtopping. This mechanism might however occur for inner slopes steeper than 1:3 and should then be taken into account in safety analysis. This failure mechanism is NOT simulated by the overtopping tests, as a slip failure needs more width to develop than the 4 m wide test section. Another test method should be used to investigate this failure mechanism, which is not described in this paper.

Description of test locations

Tests have been performed at the end of the winters in 2007-2009. Tests were performed in the winter season, when the grass condition is worst. A short description of the tested locations with some characteristics per location and per test is given below. The maximum mean discharge performed for each section is in italics.

Delfzijl, Groningen (homogeneous clay dike) – *maximum discharge limited to 50 l/s per m*
Normal grass cover; *50 l/s per m*
Reinforced grass cover (geotextile; Smart Grass Reinforcement - SGR); *50 l/s per m*
Bare clay (20 cm of grass cover removed). *10 l/s per m*

Boonweg, Friesland (60 cm clay at inner slope on top of a sand core)
Normal way of maintenance (grazing sheep, 4x per year); *75 l/s per m*
2x grazing sheep, no fertilizer; *75 l/s per m*
1x grazing sheep, 1x mowing/hay, no fertilizer; *75 l/s per m*
2x mowing/hay, no fertilizer, no grazing/sheep. *75 l/s per m*

St Philipsland, Zeeland (60 cm clay on sand core)
1x mowing/hay; steep inner slope (1:2,5), bad grass coverage. *50 l/s per m*

Kattendijke, Zeeland (60 cm clay on sand)
1x mowing/hay; bad grass coverage, many moles; *75 l/s per m*
Similar, damage by manure injector; 2 poles in the slope; *50 l/s per m*
Elastocoast (gravel and two-component glue); *125 l/s per m*
Open asphalt concrete. *125 l/s per m*

Afsluitdijk, connecting dike Friesland-Holland (40 cm clay on 1 m boulder clay)
Normal dike section with transition to toe of grass on clay; *75 l/s per m*
Normal dike section with transition to parking place with brick revetment; *30 l/s per m*
Normal dike section with concrete stair case and fence on the inner slope. *75 l/s per m*

Limitations with respect to observations and conclusions

Observations of the tests have led to preliminary conclusions, which are valid only within the given boundary conditions for the tests. These can be summarized as:

- only the failure mechanism of erosion of inner slopes by wave overtopping is considered (not sliding);
- the significant wave height considered in front of the dike should be around 2 m;
- inner slopes should be between 1:2.5 and 1:3;
- the duration for an overtopping event (specific overtopping discharge) is 6 hours or less;
- grass cover to be comparable with Dutch situations (type of grass, winter season).

Observations

The easiest way to describe observations of the testing is by photographs. The next section gives an overall view of observed damages for each tested location. Each photograph has a legend describing the observation.

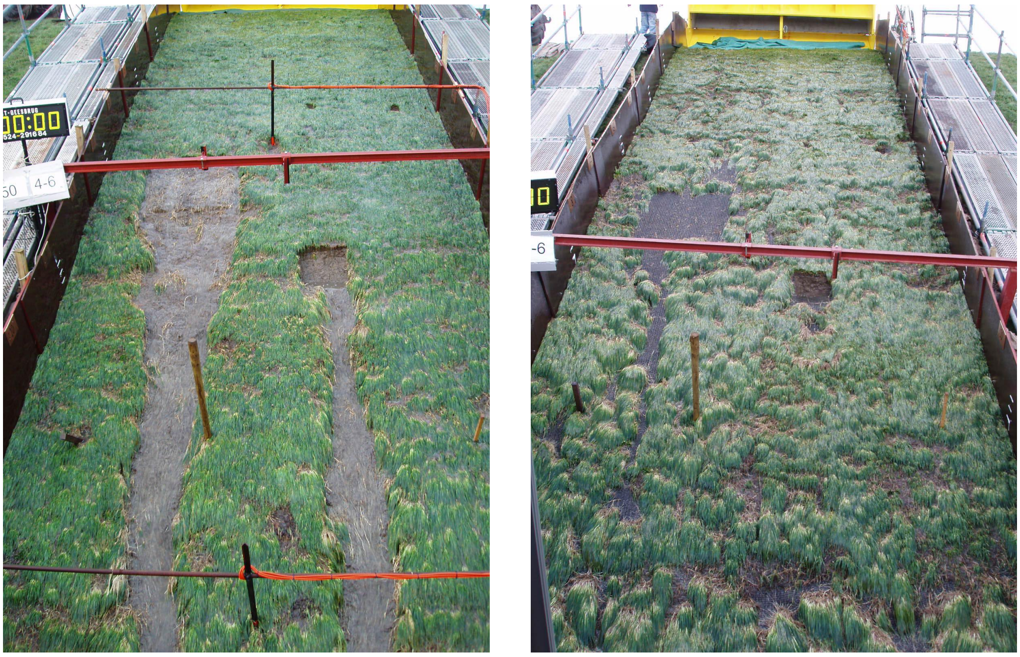

Figure 7. Final result Delfzijl, Groningen

No damage after 50 l/s per m. Left: the test section of the present dike after manual initiation of damage (1x1x0.05 m; 0.4x0.4x0.15 m; in the upper part two holes 0.15x0.15x0.15 m; three holes in the slope) and after 6 hours with 50 l/s per m. Gulley development for the two largest holes, none for the smaller. Right: a reinforced section with geotextile (SGR = Smart Grass Reinforcement), again after manual initiation of damage, where no gulleys were developed.

Figure 8. Final result Delfzijl, Groningen.

Bare clay (0.2 m grass cover was removed). Mean discharges of 1; 5; en 10 l/s per m, each during 6 hours. Ongoing erosion during each condition, which resulted in head cut erosion: horizontal part with vertical slope; vertical slope erodes by lumps of clay and the hole increases upwards.

Figure 9. Final result Boonweg, Friesland, two sections.

Maintenance: grazing with sheep, two times mowing of hay and no nitrogen for 17 years. No damage after 75 l/s per m. In last hour of 75 l/s per m damage to the toe (hidden path of brick stone, see photo right).

Figure 10. Final result Boonweg, Friesland, section 3

Initial damage during 75 l/s per m and gulley development in the clay layer. The damage extended to the toe where the hidden path with brick stone eroded too. The damage did not (yet) extent through the clay layer and did not reach the sand core. Final result after 75 l/s per m.

Figure 11. Final result Boonweg, Friesland; section 4.

After 5 hours and 51 minutes with 75 l/s per m. The sand core has been eroded to at least 1 m depth and the right side wall is about to collapse. Final result about 45 minutes after first damage was observed.

Figure 12. Final result St Philipsland, Zeeland.

After the final 50 l/s per m test. Slope 1:1.5, bad grass coverage. Also damage at the horizontal part (the toe) was created, which is visible at the lower part of the picture. This hole became so large and deep that a large overtopping wave pushed the remaining layer below the hole in the inner slope into this hole at the toe. The transition from clay layer to sand core is clearly visible at the hole in the inner slope and a deep hole in the sand core was the final result.

Figure 13. Kattendijke, Zeeland, section 1.

Damage to the rear side of the maintenance road, which started at 30 l/s per m. Removal of the whole maintenance road. Half way the 75 l/s per m test. The hole became at the end 15 m wide and about 1 m deep.

Figure 14. Final result Kattendijke, Zeeland, section 1.

The test with 75 l/s per m was stopped after 5 hours and 40 minutes. The hole in the maintenance road reached the start of the inner slope and increased upwards (head cut erosion). The sand core was clearly reached and the erosion at this stage was very fast. The inner slope itself did not show any significant damage.

Figure 15. Kattendijke, Zeeland. Final test on elastocoast.

With two extra pumps a mean discharge of 125 l/s per m was realized. It must be noted that overtopping volumes larger than 5.5 m^3 per m could not be simulated (maximum content of the simulator). With 125 l/s per m about 10 overtopping waves in 2 hours will occur which are larger than 5.5 m^3 per m and 4 of them will be even larger than 7 m^3 per m. Both the elastocoast as well as the open asphalt concrete did not show any damage.

Figure 16. Final result Kattendijke, Zeeland; open asphalt concrete.

After 125 l/s per m. A large erosion hole was created beyond the maintenance road, about 1.5 m deep. The maintenance road itself was protected by plates and could not be eroded.

Figure 17. Afsluitdijk. Toe with grass on clay.

Grass ripped off on many locations on the slope and completely at the horizontal part, where a section of 4x4 m^2 was created without grass. The good clay (still with roots) showed hardly any erosion and resisted without problems 75 l/s per m. Erosion holes near toe about 0.4 m deep.

Figure 18. Afsluitdijk. Transition to parking area

Some ripping off of grass near the toe. Damage to bricks and fast development of damage as the bricks were placed on sand. The only test that had to stop after 30 l/s per m.

Figure 19. Afsluitdijk. Concrete staircase with fence.

Left before testing and right after 2 hours of 75 l/s per m. The staircase is near failure. Grass ripped off the slope, but gulley development occurred only along the staircase where concentration of flow was observed. Hardly damage to the clay layer.

Figure 20. Afsluitdijk. Concrete staircase.

Brick path to the staircase completely destroyed, as well as the fence gate. Two erosion holes developed at the toe, due to the concentrated flow along the staircase. Holes about 1 m deep, but not reaching the sand core (cover by 0.4 m clay and 1 m boulder clay). Situation after 2 hours of 75 l/s per m.

Summary of observations and preliminary conclusions

A tested inner slope of a dike, covered with grass on clay, never failed by erosion due to overtopping for a mean overtopping discharge of 30 l/s per m or less. Only one section failed at 50 l/s per m; some at 75 l/s per m, but part of the sections did not fail, even not for 75 l/s per m.

It seems that the large erosion resistance of the inner slope of a dike is determined by the *combination* of grass and clay. The grass cover or mattress seems stronger (Boonweg, Figure 9) if it grows on a sandy clay. Such a grass cover may resist even up to 75 l/s per m, but if significant damage occurs, the clay layer is not very erosion resistant (Figures 10 and 11). On the other hand, a good quality clay does not produce a very strong grass cover (it is difficult for roots to penetrate into the clay) and the grass may rip off for overtopping discharges around 30 l/s per m (Figures 17 and 19). But in that case the remaining good quality clay layer, still reinforced with some roots, has a large erosion resistance against overtopping waves (Figure 17).

This leads to the conclusion that a good grass cover on a sandy clay and a worse grass cover on good clay show different failure mechanisms, but they show more or less similar strength against wave overtopping. The variability of the grass sod may, therefore, have less influence on the total strength than previously anticipated. This could lead to the conclusion that the way of maintenance of the grass has only minor effect on the strength of the inner slope. The test at St Philipsland may show that the bad grass coverage (small open areas without grass) on sandy clay may show less resistance (Figure 12).

Transitions from slope to horizontal are probably the most critical locations for initial and increasing damage (Figures 9, 10, 12, 13, 16, 18 and 20). During the tests this was often the transition from the inner slope to the toe of the dike, with or without a maintenance road. The tests in 2009 were focused on these kind of transitions. Damage was initiated by a mean discharge of 10 l/s per m or more. As the damage occurred at the lowest part of the inner slope it will take time for damage to extend to crest level and subsequent dike breach. Transitions higher on the inner slope (cycle paths, stability or piping berms with or without maintenance road, tracks of tractors, roads crossing the dike), which have not yet been investigated, might be more critical. Further investigation may give more confirmative conclusions.

A hole in the layer of clay, which reaches the under laying sand core and created at a large mean overtopping discharge of 50 l/s per m or more, will give a very quick ongoing erosion. This has not been observed for smaller overtopping discharges, for the simple reason that these smaller discharges never created significant damage to the inner slope. But the test with the parking place of bricks (Figure 18) showed that sand erosion with 30 l/s per m, and even with 10 l/s per m, goes fairly quickly. It must be noted that although the test was stopped for 30 l/s per m due to fast ongoing damage to the parking area, the dike itself was not in danger at all.

Small obstacles like poles did not show any erosion. Small holes from mice and moles did not initiate damage to the grass cover layer. Also a fence (Figure 19) and a little bigger pole (0.15 m by 0.15 m) showed no initiation of erosion. The grass around a fence at the toe of the dike had some influence on initiation of erosion, probably due to larger forces in this area. An obstacle like a concrete staircase on the inner slope was totally destroyed at a stage with 75 l/s per m overtopping (Figures 19 and 20). It should be noted, however, that also here the dike itself was not in danger, due to the large erosion resistance of the clay. Still, further research may give more final conclusions on other large obstacles.

Implementation of results in practice

Results of the testing show how strong the inner slope of a dike is for wave overtopping, what kind of failure mechanisms can be expected, what the week points are during overtopping and where further research should be focussed on.

It does, however, not describe how the results should be used in safety assessment of flood defence assets, nor in design of these assets. This elaboration should be done after more research, but a few points can already be mentioned.

Modelling of failure mechanisms

For calculation of flood risk or probability of flooding it is required to have a good description of all failure mechanisms, up to the moment where a breach in the dike is initiated. The present research gives a good basis to develop these failure mechanisms for erosion of the inner slope of a dike by wave overtopping.

Recent theoretical modelling (theoretical because there was no validation yet) has been described by Young (2005) en Van den Bos (2006) and this kind of modelling will be developed further. Work on overtopping velocities and flow depths is given by Bosman (2007). Young (2005) describes the ripping off of grass and considers the strength of the root system as an important parameter. Van den Bos (2006) uses scour theory for an open spot on the slope to calculate erosion on this spot. Similar theory can be used to model scour holes at the toe of a dike.

From the overtopping tests it is clear that a certain “amount of energy” above a certain threshold leads to damage initiation and ongoing damage. For overtopping discharges smaller than 10 l/s per m and often smaller than 30 l/s per m no damage was observed. For larger overtopping discharges the largest waves initiate the damage and also cause further developing of damage. It means that modelling should be focussed on these larger overtopping waves. The correct parameter to consider could be the velocity of the overtopping wave, described by front velocity, maximum velocity or a kind of mean velocity, U. But not every wave gives damage, which may lead to a critical velocity, U_c, where for lower velocities no damage has to be expected.

A kind of energy measure would then be $(U - U_c)^2$ and for all waves in a certain time this leads to $\Sigma(U - U_c)^2$. This energy measure should then be related to the strength model. It is important to get a good value for U_c from the overtopping tests and the energy measure may become a better predictor for damage than the overtopping discharge only (even if it is coupled to the incident wave height, which was assumed to be 2 m for the simulations). Research is ongoing.

Safety assessment and design

The design practice in the Netherlands over the past fifty years and more is that the required crest height is determined by almost no or little wave overtopping. First this was taken as the 2%-run-up level, which was later translated to tolerable overtopping discharges around 0.1 – 1 l/s per m. The results of the tests show that there might be quite some extra strength or safety in the mechanism of erosion by wave overtopping. But it does not mean that the design philosophy should be changed, as infiltration and sliding may give failure for fairly low wave overtopping and it is also required to have a safety margin in a proper design.

The design practice of allowing hardly any overtopping under very extreme storms (return periods up to 10,000 years) has in the Netherlands led to strong and high dikes. There is pressure to become even more safe for flooding and the assumed faster sea level rise may give more severe storm conditions. Continuation of the design policy will mean that almost all dikes have to be raised and improved again in the next 50 years.

The safety assessment procedure in the Netherlands also considers a tolerable overtopping discharge of 1 l/s per m or a little more. For assessment of erosion of inner slopes by wave overtopping, there might be a good reason to decide not to improve the height of the dike, if overtopping discharges are found to be a little larger than 1 l/s per m. But what would be the effect of a larger tolerable overtopping discharge on the crest height?

The answer is of course dependent on the actual situation, like wave conditions considered and geometry of the dike. Assuming similar situations as assumed for the wave overtopping tests (significant wave height of 2 m, wave steepness of 0.04 and an outer slope of 1:4) gives the result in Figure 21.

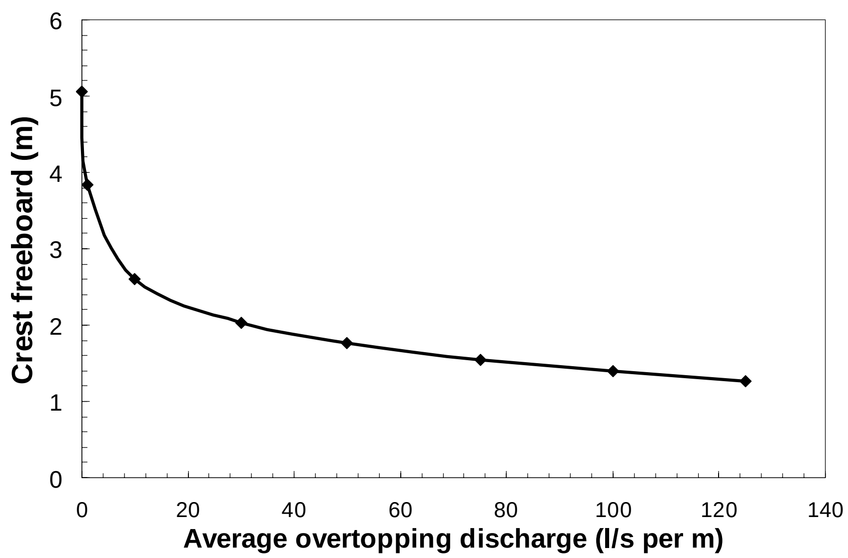

Figure 21. Relationship between overtopping discharge and required crest freeboard (H_s=2 m; s_{op}=0.04 and cotα=4)

A tolerable overtopping discharge of 1 l/s per m leads to a required crest height of 3.84 m. If this discharge is increased to 10 l/s per m, the required crest height reduces by 1.23 m, which is about 30% reduction! If we want another 30% or 1.23 m reduction (a required crest height close to 1.4 m) means that the tolerable overtopping discharge becomes 100 l/s per m. That would be totally unacceptable for a grass slope. Due to the shape of the curve the effect is largest if small allowable overtopping discharges are increased. The effect is relatively smaller for increasing even more.

With other words, it seems attractive for the situation in the Netherlands, to increase the tolerable overtopping for safety assessment to 10 l/s per m; it becomes less attractive to increase further.

Above are only remarks on discussions that still have to be held and which eventually will lead to new guidance in design procedures, safety assessment procedures and flood risk assessments.

Acknowledgements

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