Clients: ComCoast
Rijkswaterstaat, Dienst Weg- en Waterbouwkunde
Rijkswaterstaat, Waterdienst
Projectbureau Zeeweringen

Erosion strength of inner slopes of dikes against wave overtopping

Preliminary conclusions after two years of testing with the Wave Overtopping Simulator

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Date: August 2008

Executive summary

The main purpose of this report is to give in English an overall view of results/observations, obtained with destructive testing with the Wave Overtopping Simulator in the past two years on real dikes, and based on that to give preliminary conclusions on strength of grassed inner slopes of dikes against wave overtopping.

The first chapter describes in summary what the Wave Overtopping Simulator is and how it works. The second chapter summarizes the tests performed, the observations and comes to preliminary conclusions and some recommendations. The third and final chapter contains a large set of photo's with results of the testing at various dike sections. Final conclusions will be made after further testing and when a Technical Report on strength of dikes against overtopping will be written in 2010/2011.

This summary report is based on various official reports for the Rijkswaterstaat (in Dutch) and for ComCoast (www.comcoast.org). The official reporting includes soil and vegetation investigations, predictions and modelling of failure modes, test procedures and analysis of all the testing. This summary report focuses only on observed results and preliminary conclusions, not on the scientific background and detailed analysis. It will give the reader a fair idea of what has been obtained so far. It gives also a good idea of what can be expected if similar tests will be performed elsewhere.

Various clients have contributed to make all the testing possible:

- ComCoast (tests in 2007, Delfzijl, Groningen);
- Rijkswaterstaat, Dienst Weg- en Waterbouwkunde (SBW-project, tests 2007 Delfzijl, Groningen);
- Rijkswaterstaat, Waterdienst (SBW-project, tests Boonweg, Friesland);
- Projectbureau Zeeweringen (tests St Philipsland and Kattendijke, Zeeland).

All of the testing has been performed by a project group SBW Wave Overtopping and Strength of Dikes. SBW is the overall research project of the Rijkswaterstaat – Waterdienst on "Strength of and Loads on Water Defence Assets". Members of the SBW project group were:

- André van Hoven, Deltares, chairman
- Henk Verheij, Deltares
- Dr Gijs Hoffmans, Deltares
- Dr Jentsje van der Meer, Van der Meer Consulting
- Gosse Jan Steendam, Infram
- Gert Jan Akkerman, Royal Haskoning
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This report has been written by Dr Jentsje van der Meer and has been reviewed by the SBW project group.

Contents

1 The wave overtopping simulator

The process of wave overtopping on a dike, levee, seawall or embankment has been subject of a huge amount of research, resulting in e.g. equations for maximum velocities and flow depth of overtopping waves at the crest of a dike, see the new Overtopping Manual, 2007. The overall conclusion is that the hydraulic part of wave overtopping on a dike is well-defined.

In contrast, the erosive impact of wave overtopping on dikes, embankments or levees 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. Hence, in order to establish the resistance or strength of a dike for wave overtopping, field tests are required. For the simulation of overtopping, it is actually sufficient to reproduce the overtopping flow only. Thus, generation of true waves in a large scale facility, such as Delta flume (Netherlands) or GWK (Germany), is not required.

Basically, the following starting-points underlay the idea of the simulator, also described in Van der Meer et al., 2006:

- knowledge on wave breaking on slopes and generating overtopping discharges is sufficient;
- knowledge on the pattern of overtopping waves, known as volumes, distributions, velocities and flow depth of overtopping water on the crest, is sufficient as well, except for some minor points;
- only the overtopping part of the waves needs to be simulated, see Figure 1.1.
- tests can be performed in-situ on each specific dike

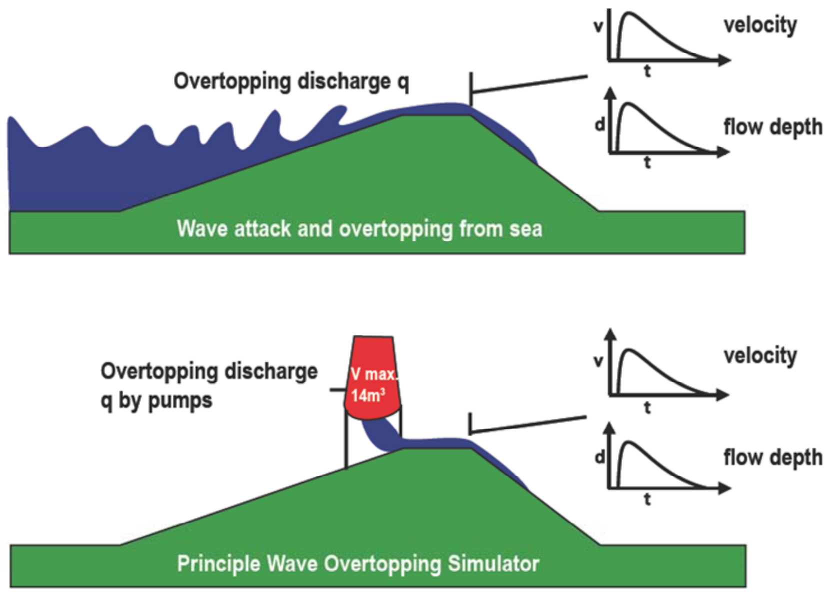

Figure 1.1 Principle of the wave overtopping simulator

The simulator was developed and designed early 2006 and constructed late 2006 within the ComCoast project (a European project between governments along the North Sea, see www.comcoast.org). At first a 1 m wide prototype was constructed and results of the calibration phase have been described by Van der Meer et al., 2006 and the full results on the wave overtopping simulator by Van der Meer, 2007 and Van der Meer et al., 2007.

The final simulator consists of a high-level mobile box (adjustable in height) to store water. The maximum capacity in first instance was 3.5 m^3 per m width (14 m^3 for the final, 4 m wide, simulator). This box is continuously filled with a predefined discharge q and emptied at specific times through a butterfly valve in such a way that it simulates the overtopping tongue of a wave at the crest and inner slope of a dike. As soon as the box is filled with a required volume, V, the valve is opened and the water is released on a transition section that leads to the crest of the dike, see Figure 1.2. The discharge of water is released in such a way that flow velocity, turbulence and thickness of the water tongue at the crest corresponds with the characteristics that can be expected.

Figure 1.2. Wave overtopping simulator at a dike, releasing 14 m^3 in a few seconds

In order to clarify erosion resistance of grass protection under wave overtopping, tests have been performed using the Wave Overtopping Simulator (Figures 1.2 and 1.3) in 2007 and 2008 on four sea dikes in the Netherlands. For the 2007 tests, the grass dike had a 1:3 inner slope of fairly good clay with sand content smaller than 30%. The overtopping simulator was used to test the erosion resistance of this inner slope for a simulated 6 hour storm for each overtopping condition. These started with overtopping equivalent to a mean discharge of 0.1 l/s per m and increased to 1; 10; 20; 30 and finally even 50 l/s per m. After all these simulated storms the slope was still in good condition and showed little erosion. Results have been described in Akkerman et al., 2007-1 and 2007-2.

In 2008 the wave overtopping simulator was enlarged to 22 m^3 to simulate mean discharges up to 75 l/s per m, see Figure 1.3. In total 9 different dike sections were tested, including dikes with a sand core and clay layer, less erosion resistant clay (larger sand content), steeper inner slope (1:2.5), bad coverage of grass, initial damages on the inner slope by farmer work, many mole holes, and finally overtopping resistant solutions with open asphalt concrete and a newly developed system called elastocoast (small gravel glued together by two-component glue). First results have been described by Steendam et al., 2008.

It should be noted that direct erosion of the slope is only one possible failure mechanism. 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 steep inner slopes, so should be taken into account in safety analysis.

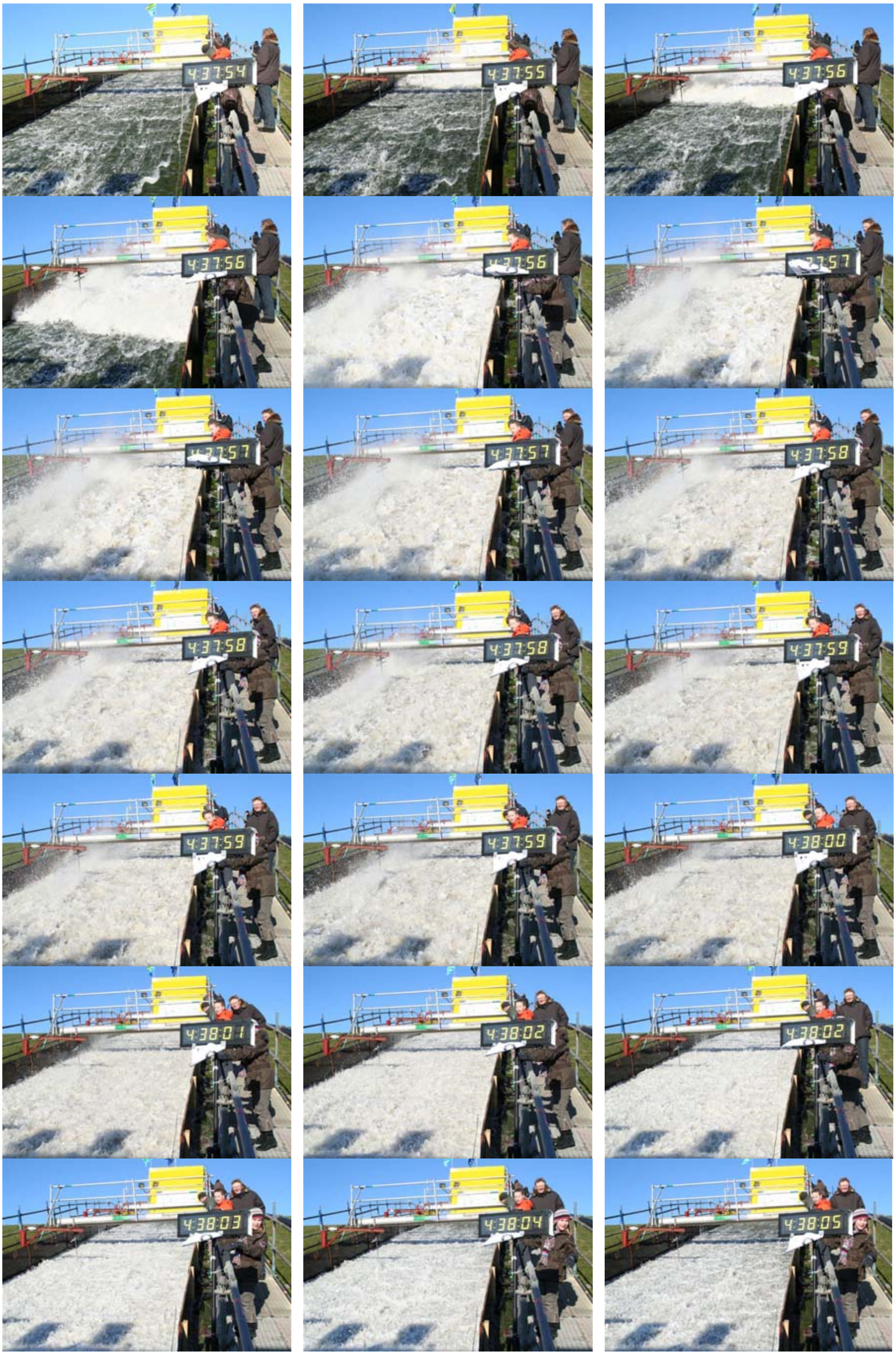

Figure 1.3. The Wave Overtopping Simulator releases 22 m^3 of water over 4 m width in about 5 s. It simulates a large overtopping wave with a mean discharge of 75 l/s per m.

Mean overtopping discharges and distributions of overtopping waves can be calculated by methods given in the Overtopping Manual, 2007. The mean overtopping discharge is an input parameter for the overtopping tests: the required overtopping discharge is pumped into the Wave Overtopping Simulator. It depends, however, on actual storm conditions and dike geometry how many waves will overtop the dike and how large the overtopping volumes will be. In order to make a correct simulation, it is necessary to set these boundary conditions.

For the time being it was assumed that dikes have a 1:4 outer slope with a smooth revetment (grass or other protection). The significant wave height at the toe of the structure was assumed to be 2 m, with a wave steepness (with peak period) of 0.04. The final assumption was a storm duration of 2 hours. With these data it is possible to calculate distributions of overtopping volumes for each required mean overtopping discharge. Some characteristic values have been summarized in Table 1.1 and distributions of overtopping volumes are shown in Figure 1.4.

Table 1.1 Characteristic values depending on mean overtopping discharge. Background: H_s = 2 m; s_{op} = 0.04, outer slope 1:4 and storm duration 2 hours

Mean overtopping discharge (l/s per m)	**0,1**	**1**	**10**	**30**	**50**	**75**	**100**	**125**
Crest freeboard (m)	5,06	3,84	2,61	2,03	1,76	1,54	1,39	1,27
Percentage of overtopping waves (%)	0,2	2,7	18,9	36,6	47	56	62	68
Number of overtopping waves	3	42	289	561	720	858	956	1034
Maximum overtopping volume (l/m)	400	835	2110	3790	5180	6750	8250	9680

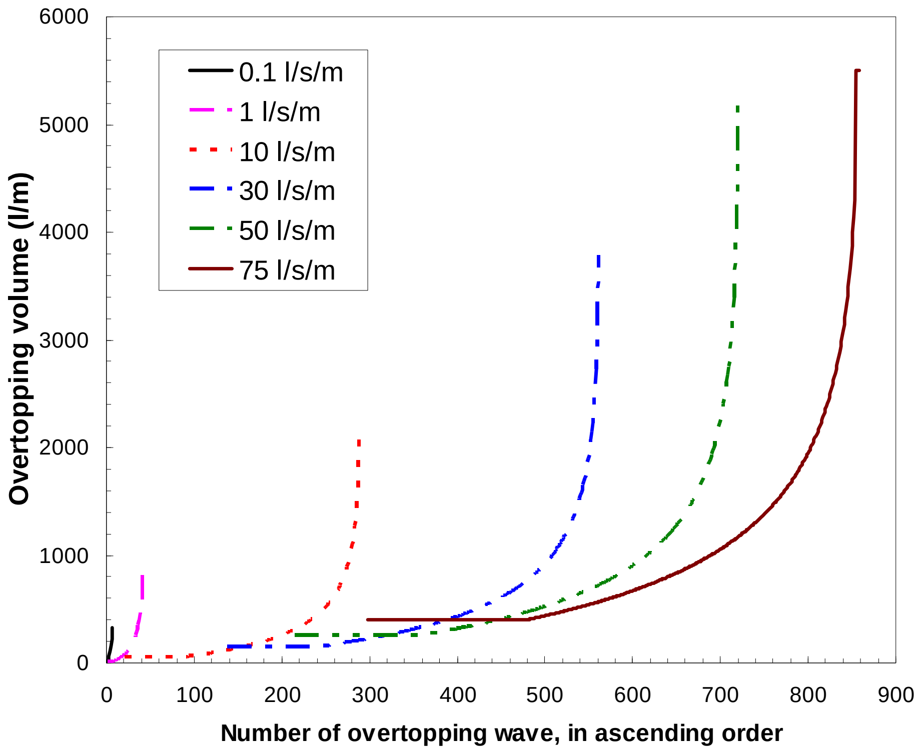

Figure 1.4. Distribution of overtopping waves for various mean overtopping discharges, as simulated by the Wave Overtopping Simulator. Background: H_s = 2 m; s_{op} = 0.04, outer slope 1:4 and storm duration 2 hours

Most dikes in the Netherlands have been designed for 1 l/s per m. The water level has to raise 2.3 m more to come to a mean overtopping discharge of 75 l/s per m, see Table 1.1. The number of overtopping waves increases then from 42 to 858 on a total of about 1500 incident waves. The maximum overtopping volume increases from 835 l/m to 6750 l/m (but the Wave Overtopping Simulator is limited to 5500 l/m).

A 6-hour storm duration has been taken as a representative (conservative) value for producing damage for each simulated wave overtopping distribution. Tests at the dike sections in 2007 consisted of the following mean overtopping discharges:

- 6 hours storm with overtopping rate of 0.1 l/s per m width;
- 6 hours storm with overtopping rate of 1 l/s per m width;
- 6 hours storm with overtopping rate of 10 l/s per m width;
- 6 hours storm with overtopping rate of 20 l/s per m width;
- 6 hours storm with overtopping rate of 30 l/s per m width,
- 6 hours storm with overtopping rate of 50 l/s per m width,

For the 0.1 l/s per m, the number of overtopping waves is very limited: hence, this test was speeded up to 36 minutes (by accelerating the intermediate periods 10 times). The other tests were carried out in real-time. After each 2 hours the tests were stopped for a detailed survey of the erosion.

As the Wave Overtopping Simulator was enlarged in 2007, the test series in 2008 were adapted:

- 6 hours storm with overtopping rate of 0.1 l/s per m width;
- 6 hours storm with overtopping rate of 1 l/s per m width;
- 6 hours storm with overtopping rate of 10 l/s per m width;
- 6 hours storm with overtopping rate of 30 l/s per m width;
- 6 hours storm with overtopping rate of 50 l/s per m width,
- 6 hours storm with overtopping rate of 75 l/s per m width,

Both the 0.1 and 1 l/s per m conditions were speeded up (by 10, respectively 5 times). Figure 1.5 gives an example where indeed damage was created by the largest overtopping discharge of 75 l/s per m.

Figure 1.5. Damage to a dike section during a test with 75 l/s per m wave overtopping.

2 Results and preliminary conclusions

Wave overtopping tests have been performed in March 2007 at Delfzijl, Groningen, in February 2008 at Boonweg, Friesland and in March/April 2008 at two locations, St Philipsland and Kattendijke, Zeeland. Each of these three test series has been reported separately. The tests at Delfzijl have fully been reported in ComCoast and the reports can be downloaded (www.comcoast.org). Factual reports (in Dutch) have been made for the tests at Boonweg and at Zeeland. Full analysis is in progress.

In May 2008 a (Dutch) report was made, which summarizes briefly all the tests, the observations from the tests and preliminary conclusions based on the observations. This chapter is a free translation of that report.

The tests with the Wave Overtopping Simulator have been performed for certain boundary conditions, partly described in Chapter 1. Preliminary conclusions, therefore, are only applicable within these boundary conditions, which can be described as follows:

- The only failure mechanism described is *erosion of inner slopes of dikes by wave overtopping.* The mechanism of infiltration of water by overtopping waves and (superficial) sliding of the inner slope has not been subject of investigation. Research focused on this mechanism will be performed in future. This mechanism is very important for steep inner slopes. In the Netherlands most sea and lake dikes have been designed with fairly gentle slopes of 1:3 and it is generally assumed that these kind of slopes will be stable for this mechanism. For the application of the preliminary conclusions it is assumed that sliding of the inner slope by infiltration will not occur.
- The overtopping simulation was based on the assumption of a wave height of about 2 m in front of the dike. This is more or less a mean value for safety assessment in the Netherlands for sea and lake dikes. Dikes directly fronting the North Sea may have a much more severe wave attack, say around 4 m, and dikes with much lower wave attack (1 m or smaller, many river dikes) are too far from the value of a wave height of 2 m. If the wave height is much larger than 2 m, less waves will overtop for the same mean discharge, but the volume per wave will be larger (giving a more severe loading). A smaller wave height, on the contrary, will give more overtopping waves, but the volumes per wave will be smaller. If the waves are very small, the situation will be quite close to overflow. Preliminary conclusions, therefore, yield mainly for *sea and lake dikes with average wave attack around 2 m.*
- The tested dikes had inner slopes between 1:2.5 and 1:3. Steeper inner slopes, possibly with a berm as for many river dikes, do not fall within the tested boundary conditions.
- All tested sections had a width of 4 m (the width of the Wave Overtopping Simulator). Results and observations do not indicate that this was too narrow, although infiltrated water could disappear sideways. Almost all damage was initiated in the section of 4 m wide and not against one of the side boards.
- Each test with a constant predefined mean discharge had a duration of 6 hours. Each test series started with a small mean discharge and this was increased after every 6 hours, still on the same section. A section which was tested with 75 l/s per m, for example, withstood already 6 hours of 10, 30 and 50 l/s per m. If the duration of the peak of a storm lasts longer than 6 hours and the mean discharge will exceed 30 l/s per m, the preliminary conclusions will not be valid.

A short description of the tested locations with some characteristics per location and per test is given below.

Delfzijl, Groningen (homogeneous clay dike)

- Normal grass cover
- Reinforced grass cover (geotextile; Smart Grass Reinforcement - SGR)
- Bare clay (20 cm of grass cover removed)

Boonweg, Friesland (60 cm clay at inner slope on top of a sand core)

- Normal way of maintenance (grazing sheep)
- 2x grazing sheep, no fertilizer
- 1x grazing sheep, 1x mowing/hay, no fertilizer
- 2x mowing/hay, no fertilizer, no grazing/sheep

St Philipsland, Zeeland (60 cm clay on sand core – at damage location it appeared to be 40 cm)

- 1x mowing/hay; steep inner slope (1:2,5), bad grass coverage

Kattendijke, Zeeland (60 cm clay on sand core – at damage location it appeared to be 75 cm)

- 1x mowing/hay; bad grass coverage, many moles
- similar, damage by manure injector; 2 poles in the slope
- elastocoast
- open asphalt concrete
- 20 minutes of demonstration for visiting people from Florida

Chapter 3 gives a large number of photo's with results of all tested sections. The legend at each photo gives extra information. Reading this chapter gives a fair idea of what has happened during all the testing at various locations.

First of all ***observations*** of the tests will be described. This means that descriptions will be given of what occurred or was observed, without giving interpretations of what could be the cause of the observations. These observations are not preliminary, although small differences may appear in the final reporting of the analysis of the tests.

The following ***observations*** were noticed.

1. None of the test sections gave significant damage at the inner slope at 30 l/s per m during 6 hours of simulation.

2. One test section gave significant damage on the inner slope at 50 l/s per m (St Philipsland). The inner slope was 1:2.5 with a sandy clay on a sand core. The grass cover was fairly open.

3. Some test sections gave extensive damage at 75 l/s per m at the inner slope, but a significant part of the test sections withstood these loads very well.

4. The transition from crest to inner slope never showed any damage.

5. Elastocoast and open asphalt concrete did not show any damage at 125 l/s per m. A small repair with cold asphalt disappeared immediately.

6. Transitions from slope to horizontal gave significant damage in 6 out of 9 test sections; sometimes already at 30 l/s per m, but especially at 50 l/s per m or more. A maintenance road of gravel in dry circumstances is hard and stable for

driving, but erosion and severe damage occurs for wave overtopping of 30 l/s per s and more.

7. A hole in the clay with a vertical slope gives the mechanism of head cut erosion. the vertical slope erodes by lumps of clay falling down and the vertical slope moves upwards on the inner slope of the dike. This mechanism was observed in 4 test sections (the right hole in Figure 2 of Chapter 3 gives an example).

8. Damage (a hole) through the layer of clay onto the sand, at 50 l/s per m or more overtopping, gives very quickly a severe increase of damage. The sand from the sand core erodes very quickly for this large overtopping, again according to the head cut erosion mechanism.

9. Good erosion resistant bare clay, as at the dike at Delfzijl, can resist 6 hours with 1 l/s per m overtopping, followed by 6 hours with 5 l/s per m and subsequently 6 hours with 10 l/s per m. During each large overtopping wave erosion was noticed over the inner slope of bare clay. The erosion consisted of small particles of clay. The final damage at this bare clay inner slope, therefore, was influenced by the other two conditions (the final result is shown in Figure 2 of Chapter 3). Results are valid for a homogeneous clay dike. If a clay layer covers a sand core, the critical moment will be when the erosion will reach this sand core.

10. Damage was initiated in the grass cover on this erosion resistant clay (Delfzijl), by digging holes of 15 x 15 cm, 15 cm deep. A discharge of 50 l/s per m did not increase this initiated damage. At initiated holes of 40 x 40 cm (15 cm deep) and 1 x 1 m (5 cm deep) gulleys were created with the same discharge of 50 l/s per m. These gulleys extended from the down side of the holes to the toe of the dike, see Figure 1 of Chapter 3.

11. The reinforced gras cover at Delfzijl (SGR – Smart Grass Reinforcement) did not show the creation of gulleys after creating initial damage as described under observation 10, see Figure 1 of Chapter 3. This system appeared to be stronger than a normal grass cover.

12. Some test sections showed the presents of (many) moles and sometimes mice. The tracks became clearly visible after first wave overtopping and sometimes the entrances eroded a little, but none of these natural damages gave initiation of damage to the inner slope.

13. At two test sections, both (almost) without grazing with sheep and a strong grass sod, the “bulge out” mechanism was observed for 75 l/s per m overtopping. This mechanism was unknown, see Figures 5, 7 and 9 of Chapter 3.

14. Damage initiated by a farmer’s manure injector (see Figure 22 of Chapter 3) gave no further increase of damage during wave overtopping.

15. Recently placed poles with a small diameter in general do not give initiation of damage. One pole (at Kattendijke) disappeared from the slope, mainly because it had been placed only at a depth of 30 cm and in or close to a number of tunnels by moles. Although damage was initiated, the damage did not increase (up to the maximum discharge of 50 l/s per m at this section). Poles with a larger diameter or other larger obstacles have not been investigated.

The observations as given above have led to ***preliminary conclusions***, which are valid within the given boundary conditions, 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 (constant overtopping discharge) is 6 hours or less.

The ***preliminary conclusions*** are:

1. It seems credible that (in Dutch situations, so far) an inner slope of a dike, covered with grass will never fail by erosion due to overtopping for a mean overtopping discharge of 30 l/s per m or less. Only further research can confirm this and give a more final conclusion.

2. It seems that the erosion resistance of the inner slope of a dike is determined most by the presence of grass and less by the erosion resistance (quality) of the clay. The variability of the grass sod has influence, but may be less 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.

3. Transitions from slope to horizontal are probably the most important locations for initial and increasing damage. During the tests this was often the transition from the inner slope to the toe of the dike, with or without a maintenance road. Damage was initiated by a mean discharge of 30 l/s per m or more. As the damage occurred at the lowest part of inner slope it will need a lot of time in real situations to cause a breach in the dike. Other transitions from slope to horizontal have not been investigated, but probably give the same picture. One may think of cycle paths, more elevated maintenance roads on a berm, inner berms, tracks of tractors, paths by sheep, roads crossing the dike, stairs, etc. Further investigation may give more confirmative conclusions.

4. 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.

5. Small obstacles like poles did not show any erosion. Larger obstacles, like bigger poles, or a fence which exists already for a long time (with possibly many tracks of moles or mice in the long grass and different texture of the soil) have not been investigated, and may give rise to initiation of damage. Also here, further research may give more final conclusions.

Safety assessment and design

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.

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.

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 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. After further research it should be discussed if tolerable overtopping discharges for safety assessment should be increased and by how much.

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.

3 Photo's of damages and results

Figure 1. Final result Delfzijl, Groningen. Left: the test section of the present dike after 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: the reinforced section with geotextile (SGR = Smart Grass Reinforcement), where no gulleys were developed.

Figur 2. 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 slop erodes by lumps of clay and the hole increases upwards).

Figure 3. Final result Boonweg, Friesland, first section. Clay layer of 0.6 m on sand core. Normal maintenance (whole summer season grazing with sheep and 70 kg/hectare nitrogen). No damage to inner slope after 75 l/s per m; After 4 hours with 50 l/s per m damage to horizontal part (toe), where a hidden path with brick stone appeared to be present. This damage increased during 75 l/s per m (photo right).

Figure 4. Final result Boonweg, Friesland, second section. Maintenance: two times 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 5. Initial damage Boonweg, Friesland; section 3. Two times mowing of hay, no nitrogen en in 17 years no grazing with sheep. Damage in the second hour with 75 l/s per m. At that moment the grass sod felt “spungy”. It might be that loose clay in the upper few centimetres of the sod was eroded. The force on the upper part of the grass could then become so large that the roots of the sod broke (this is still speculation, more analysis is required). The grass with clay layer “bulged out” and was filled with water. A large overtopping wave destroyed the bulge and a part of the grass cover with clay layer disappeared.

Figure 6. Final result Boonweg, Friesland, section 3 (see also figure 5). The “bulge out” mechanism appeared again in the last hour of the 75 l/s per m and finally, the two damages coincided to form one large damage area where the grass cover was not present anymore. 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 7. Initial damage Boonweg, Friesland, section 4. One time grazing with sheep, one time mowing of hay, no nitrogen for 17 years. Development of “bulge out” mechanism in last hour with 75 l/s per m. In this occasion it took about 5 minutes before a large overtopping wave destroyed the bulge. Water flows from left to right on the picture.

Figure 8. Boonweg, Friesland, section 4. Directly after the bulge in figure 7 was destroyed. Picture taken from above.

Figure 9. Boonweg, Friesland; section 4. After the first hole in figure 8, a second bulge was formed, directly below the first one. This one was “leaking” water.

Figure 10. Boonweg, Friesland; section 4. Destruction of the second bulge by a large overtopping wave, a few minutes after the bulge appeared. The plaquette of grass sod still has a small layer of clay. This might be a proof that the roots were broken beneath this clay layer (further analysis required), where less roots are present. The presence of this clay layer may be the reason why the bulge seems to be water tight, as the grass cover itself is of course not water tight at all. In the picture water flows from right to left.

Figure 11. Boonweg, Friesland; section 4. Situation about 5 minutes after the second bulge (Figures 9 and 10) was destroyed. Left in the middle, a hole in the clay layer becomes present with a vertical slope. Here the hole will increase and the sand core will be reached.

Figure 12. 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 after about 45 minutes after creation of the first bulge out (as in figure 7).

Figure 13. Initial damage at St Philipsland, Zeeland, at the start of the 50 l/s per m test. A steep 1:2,5 inner slope with a poor grass cover and some damages by moles. The two small holes (one left above, between two front velocity gauges and one right down on the picture) were created by the same large overtopping wave at the start of the test. The hole left above did not increase during further testing, whilst the hole on the right increased to “failure”.

Figure 14. (Left). Final result St Philipsland, Zeeland, after 50 l/s per m. The entrance of the mole track was visible from the start of testing, but never increased to significant or further damage.

Figure 15. (Right). Result half way testing at St Philipsland, Zeeland, with 50 l/s per m. The hole got a vertical slope and the sand core has been reached. It appeared that the clay layer had a layer thickness of only 0.38 m.

Figure 16. Final result St Philipsland, Zeeland, after the 50 l/s per m test. 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 17. Kattendijke, Zeeland, section 1. At the start of the test with 10 l/s per m. Poor grass coverage. In the whole test section about 80 mole-holes were present. The mole-heaps disappeared with the first overtopping waves, and the holes/entrances became clearly visible. Also the tracks at the surface are clearly visible. At the start of the 10 l/s per m testing, it was expected that the test section would not survive this 10 l/s per m situation. The actual result was that the test section withstood 75 l/s per m with only small enlargements of some entrances, but no initiation of damage at all.

Figure 18. Kattendijke, Zeeland. Putting a mole trap (left) with the final result right.

Figure 19. Kattendijke, Zeeland, section 1. Damage to the rear side of the maintenance road, which started at 30 l/s per m. The inner slope of the test section is at the other side of the measuring cabin. This cabin had to be removed after 1 hour with 75 l/s per m in order not to fall in the hole. The picture has been taken just before the cabin was removed.

Figure 20. Kattendijke, Zeeland, section 1. Enlargement of the hole in Figure 19, including 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 21. 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 22. Kattendijke, Zeeland, section 2. Before the test. The cuts with the farmer's manure injector were present at the crest, the toe and the lowest part of the inner slope, not on the middle part of the inner slope.

Figure 23. Final result Kattendijke, Zeeland; section 2. After 50 l/s per m. The damage by the manure injector did not initiate further damage. No damage to the inner slope.

Figure 24. Final results Kattendijke, Zeeland; section 2. After 50 l/s per m. Two poles were placed, one on the crest and one in the middle of the slope. Very light erosion occurred around the pole on the crest, but no ongoing erosion.

Figure 25. Kattendijke, Zeeland; section 2. After 2 hours with 50 l/s per m. The pole in the middle of the slope was swept away. This pole was standing vertically (not perpendicular to the slope) and 30 cm deep. A plaquette of clay and grass was pushed upwards and a little down the slope when to pole was swept away.

Figure 26. Kattendijke, Zeeland; section 2. Continuation of the 50 l/s per m test after initial damage by the pole that was swept away (Figure 25). It was expected that the plaquette would be removed by a large overtopping wave.

Figure 27. Final result Kattendijke, Zeeland; section 2. Afer 50 l/s per m. During further testing the clay underneath the plaquette eroded, but the plaquette itself was still fastened enough not to be swept away. Finally, the plaquette subsided after this erosion underneath it and it became in line again with the slope (left). Right: after digging out the plaquette after the test (hole not created by testing!). It appeared that the pole had been placed in a mole track, some of them were more than 50 cm deep.

Figure 28. Final result Kattendijke, Zeeland; section 2. After 50 l/s per m. Damage to the toe of the inner slope (the filter structure existing of gravel) started already at 30 l/s per m. It was expected that this damage would increase drastically for 75 l/s per m, as was the case for section 1. It was decided not to create such a significant damage again. It was expected that further testing would not show significant damage at the inner slope and the main objective (effect of a manure injector) had already been reached.

Figurer 29. Kattendijke, Zeeland. Construction of test section with elastocoast and open asphalt concrete. A 5 cm grass cover was removed, an open geotextile was placed and then covered with 10-15 cm elastocoast or open asphalt concrete.

Figure 30. Kattendijke, Zeeland. Final test on elastocoast. Wiht 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 and 4 of them will be larger than 7 m^3 per m. Both the elastocoast as well as the open asphalt concrete did not show any damage.

Figure 31. Final result Kattendijke, Zeeland; section with 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 32. Kattendijke, Zeeland. Demonstration after all the tests were finished. On 16 April 17 water managers from Florida visited the site. On a new location a test section was set-up and during 20 minutes a demonstration was given with 75 l/s per m. In this short time a hole was created in the maintenance road with a depth of about 0.5 m.

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