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Influence of management and maintenance on erosive impact of wave overtopping on grass covered slopes of dikes; Tests

G.J. Steendam
Infram B.V., Marknesse, The Netherlands

W. de Vries
Infram B.V., Marknesse, The Netherlands
Formerly employed at Water Board Fryslân, Leeuwarden, The Netherlands

J.W. van der Meer
Van der Meer Consulting, Heerenveen, The Netherlands

A. van Hoven
Deltares, Delft, The Netherlands

G. de Raat
Rijkswaterstaat, Centre for Water Management, Lelystad, The Netherlands

J.Y. Frissel
Alterra, Wageningen, The Netherlands

ABSTRACT: In March 2008 wave overtopping tests were performed at a dike along the Waddensea in the Netherlands. At this location 4 different test sites were available. Over a period of over 15 years each of these sites has been managed and maintained in a different manner. This has resulted in 4 different grass covers. These differences could have an effect on the resistance against erosion by overtopping waves. Therefore were performed with the Wave Overtopping Simulator. The research is part of the Dutch SBW program. This paper describes the setup and the results of the tests.

1 INTRODUCTION

A huge amount of research is available on the hydraulics of wave overtopping on dikes, levees, seawalls and embankments. In contrast, little research has been performed on the erosive impact of waves overtopping dikes, embankments or levees. Therefore strength of grass covered inner slopes is not well known. With the development of the wave overtopping simulator (Van der Meer et al. 2006, 2007) it is now possible to perform field research on the resistance against erosion of grass covered slopes. First tests with this overtopping device were performed within the ComCoast project (Van der Meer et al. 2007, Akkerman et al. 2007, www.Comcoast.org). In February and March 2008 further comparative field tests on differently managed and maintained grass covers were performed at a Waddensea dike in the Netherlands. These tests were performed as part of the long term project Overtopping and Strength of Grass Covers within the program Strength and Loads on Flood Defences (SBW) from the Dutch Ministry of Transport, Public Works and Water Management.

2 REASONS TO PERFORM OVERTOPPING TESTS AND PREVIOUS TESTS

2.1 *Experiences in the past*

In 1953 parts of the Netherlands and UK were flooded. Also in 1954 and 1964 in the north of Germany and recently in the south of the US flooding occurred. One of the main reasons for the dike failures was that their crests were too low. These dikes were simply overflown. In other cases, water levels were just below the crest. Here the water did not flow over the dikes, but the breaking and uprunning waves did. This overtopping lead to erosion and sliding of parts of the inner slope. These mechanisms were the main reason for the dike failures. Often the combination with steep inner slopes lead to instable conditions.

2.2 *Previous overtopping tests*

In 1992 large scale tests were performed in the Delta flume (Smith 1994). In these tests a inner slope of 1:2.5 was built with grass on a layer of good clay. The grass was excavated from an existing dike and transported to the Delta flume. These tests were performed in summer. The grass was growing fast and had to be mowed in the flume. Thus the grass was in a good condition. However, in reality most of the storms in the Netherlands happen in winter. In this season the condition of the grass is worse than in summer. The grass is not growing at all below 7 degrees centigrade, which is the case in most winters in the Netherlands. The tests in the Delta flume had an overtopping discharge of up to 25 liters per second per one meter dike length (25 l/s per m). No damage occured after many hours of testing.

Also in the GWK flume in Hanover Germany tests were performed (Möller et al. 2002). In these tests a sandy slope of fresh clay was built, which showed gulleys and holes within half an hour of testing with only 2 l/s per m.

With the development of the wave overtopping simulator (Van der Meer et al. 2006 and 2007) tests can now be performed at the actual dike and during grow conditions. In this way it is possible to test the real behaviour of grass slopes in prototype conditions. First tests were performed in February an March 2006 within the European project Comcoast (Akkerman et al. 2007). These tests showed no damage of the grass cover up to 50 l/s per m. Bare clay (section with removed grass cover) could resist 5 and 10 l/s per m each for 6 hours, and was then eroded

3 DESCRIPTION OF TEST LOCATION

3.1 *Profile of the dike*

In the northern part of the Netherlands the dike along de Wadden Sea protects the provinces of Fryslân and Groningen against flooding. The dikes are managed and maintained by the regional Water Boards. At the location of Sint Jacobiparochie over 15 years a site exists at which the influence of management and maintenance on the grass cover on the inner side of the dike has been monitored. This part of the dikes along the Waddensea is managed by the Frisian waterboard called Wetterskip Fryslân.

The dike has a sand core which is covered with a 1 m layer of clay on the outer side and a 0.6 m layer of clay on the inner slope. The outer slope is then covered by a pitched layer of basalt blocks at the toe and from the toe up covered in asphalt at a slope angle of 1:5. Also the outer berm is covered in asphalt. The berm is situated at 7 m above MSL. The outer slope above the berm is covered with grass. The crest of the dike has an elevation of 9.3 m above MSL and a width of 3 m. The crest and the inner slope are completely covered with grass. A road for maintenance is situated a few meters from the toe of the inner slope. The total width of the inner berm is approximately 30 m. A 10 m wide canal is present some 30 m from the dike.

Figure 1. Test sites.

Table 1. Treatment of the test fields.

Treatment	Regular section 1	section 2	section 3	section 4
Grazed by sheep (times a year)	4	2	–	1
Hayed (times a year)	–	1	2	2
Fertilizer (kg N/ha)	70	–	–	–

3.2 *Description of test sites*

Three test sites are maintained next to the adjacent dike sections which are maintained by the water boards in a ‘regular’ way.

The test sites are all located on the inner side of the dikes. The length of each site is 50 m. The slope of the dike is 1 in 3. The sites are separated from each other by a small fence.

Over 15 years the test locations were maintained and managed in different ways. The differences involve whether or not the field is grazed by sheep, whether the field is hayed and differences in the amount of fertilizer applied, if any. This has resulted in 3 different grass covers. Together with the regular managed field these are the four test sites which were tested for resistance against wave overtopping. In Table 1 the differences in treatment of the test fields is shown.

Different treatments give different vegetation types. Section 1, 2 and 4 with grazing, can be identified as moderate species rich grasslands. Section 3, with haymaking management is a species rich grassland.

3.3 *Condition of grass cover*

De condition of the grass cover was determined before the overtopping tests started. This determination consisted of several tests. First the coverage and the root structure was determined by some standard tests. Measuring of the percentage of coverage is done with a standardized plate with prescribed holes. The second measurements concerned the root structure. With a gouge bit approximately 0.3 m clay with roots is retrieved from the surface layer of grass and clay, see Figure 2. This 0.3 m is then divided in partitions of 2.5 cm. Of each of the partitions the number of roots is counted. The number of roots in each section is matched with standard classification scoring in a class from 0 to 5. In this classification 5 means that a lot of roots are present and a 0 that no roots are present. The classification is not linear.

To get an idea of how the grass cover was developing in the winter season 2007/2008 the same tests were repeated every month from November 2007 to February 2008 till the overtopping tests started.

In Figure 3 and 4 examples are shown of the root structure in November 2007. Figure 3 shows by black dots the root structure section 1. The gray dots in figure 3 show test section 2. In figure 4 the lightest dots represent section 3 and the darker dots section 4.

The figures show that at the surface a high number of roots are required to get a condition score “good”. At the deeper sections less roots are needed to get to this score. This is consistent with the general way for ground covering vegetations; roots decrease with depth.

Figures 3 and 4 also show that the condition of the grass cover in February 2008 of the test locations was good at every test site.

It was noted that the grass cover at the end of the winter of 2007/2008 was in a slightly better condition than in fall. It has been a mild winter with average temperatures over 5°C. In February even temperatures over 13°C occurred.

3.4 *Identification of soil*

Next to tests of the grass cover, also the soil in and just underneath the cover layer was identified. Amples were collected near every test section, which were tested in the laboratory. From each sample the grain distributions and fines were determined as well as the salinity, plasticity and other geotechnical parameters.

The thickness of the top layer, the clay layer underneath and the sand core of the dike were determined with a hand driven eager to a depth of 2 m below the surface. Each surfaced stretch was described thoroughly.

For each test section on overtopping 2 probes were driven in the sub soil to a depth of 20 m beneath the surface. One was driven into the crest of the dike and one at the toe of the dike. The probes deliver information

Figure 2. Gouge bit with 0.25 m sample of top layer.

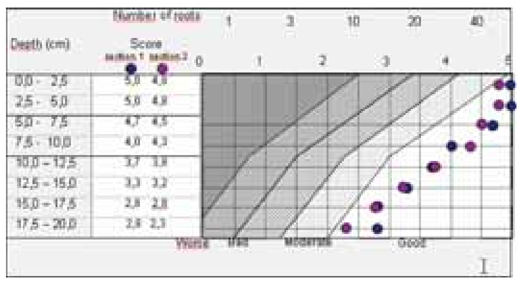

Figure 3. Root density February 2008, sections 1 and 2 (diagram VTV, 2006).

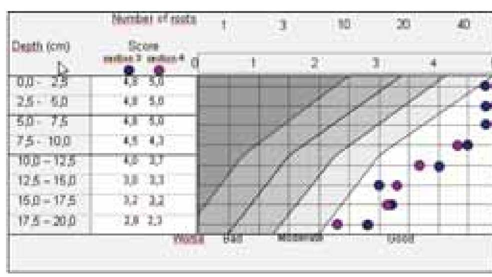

Figure 4. Root density February 2008 sections 3 and 4 (diagram VTV 2006).

Table 2. Quality of Clay.

Index	Section Regular	2	3	4
Yield value Wi (%)	29	29	29	29
Plasticity index Ip (%)	6	6	6	6
Sand content Zk (%)	60	60	60	60
luthum (%)	13	13	13	13
Clay category	c3	c3	c3	c3

on the stratification of the sub soil. Also the ground water pressure was measured. Table 2 shows some values of clay indicators. The category is an indicator of resistance against erosion. The categories are described in VTV, 2006.

3.5 *Description of sections*

Before the overtopping tests started the section was visually observed. All irregularities were recorded. The test section was divided in squares of 1 m^2.

In the description matters such as the presence, depth and orientation of holes of mice or moles was stated. Also local bold spots or superficial damage were indicated. Occasionally, tracks of tractors were present on the slopes. The mowing of the grass on the slopes is done by tractors which leave there tracks on the slope. These tracks cause some disturbance of the fairly flat slopes. The descriptions also mention the presence of higher or shorter grass and local differences in vegetation.

4 OVERTOPPING SIMULATOR

The overtopping simulator is a device which can simulate overtopping at the crest and inner slopes of dikes, levees or embankments. The device is mobile so the tests can be performed in situ at any location which meet a few conditions. These conditions mainly concern the availability of water in the quantities needed and the accessibility for the truck and crane displacing the device.

After design and construction of the device in 2006 and performing a series of tests at Delfzijl within the European project of Comcoast (Akkerman et al., 2007) some adjustments were made, resulting in a new capacity of 22 m^3.

Next to the enhancement of the capacity, also the foundation of the simulator was changed. With hydraulic cylinders the simulator now can be placed exact vertically on the inner slope of the dike or levee.

The simulator is operated manually with a joystick. The valves in the simulator are opened hydraulically.

Figure 5. Enhanced wave overtopping simulator.

5 SIMULATION OF WAVE OVERTOPPING

Wave overtopping is the phenomenon of waves passing over the sea defence in extreme conditions. The water level is situated below the crest of the water defence. The waves travelling to the dike or levee run up on the outer slope and pass the crest. Then the water continues down the inner slope.

Wave overtopping is usually described by a mean overtopping discharge. This discharge is the mean over time and passing a certain stretch of the dike or levee. This discharge is thus presented as a quantity per second per stretch of 1 meter dike or levee, often in l/s per m.

The amount of water passing the crest of the dike is influenced by the wave height in front of the dike in combination with the water level. Also the wave period has influence on the quantities. The water has to travel up the outer slope. If this slope is smooth the water will experience less resistance than more rough slopes, resulting in more overtopping water.

The wave conditions used to calculate overtopping distributions are a wave height H_s of 2 m and a wave period T_p of 5.7 s. These are more or less average conditions along the Dutch Waddensea and estuarine coasts. The duration of the peak of the storm was set to be 6 hours. Background of this period is that for the safety assessments duration of the storm is considered to be 45 hours. In this stretch of time the setup starts from zero to its maximum. Also the tide is going up and down in this period of time. The peak of the storm is supposed to coincide with a peak of the astronomical tide. The overtopping occurs often over 6 hours or less, being the period of testing.

Each overtopping wave has a certain volume of water. The distribution of these overtopping volumes can be described by a two-parameter Weibull distribution with a shape factor of 0.75. The scale factor depends on the mean overtopping discharge and the probability of a wave overtopping (TAW, 2002).

The overtopping simulator has been calibrated to meet this probability density function. The volume overtops the crest of the dike with a certain velocity and a flow depth, varying in time. These parameters have been investigated by Schütrumpf (2002) and Van Gent (2002).

As an indication in Table 4 some overtopping parameters are given for this case.

Figure 6 gives the distribution of overtopping waves at different mean overtopping discharges. In this figure the storm duration is 2 hours with a mean wave period of 4.7 s. In 2 hours there are 1,500 incoming waves. Not every wave will overtop the dike. In the figure only the overtopping waves are shown. This is a percentage of the total incoming waves. If the mean overtopping discharge is larger, more waves will overtop. The largest overtopping volume of the Wave overtopping simulator is 5,500 liters per meter width. This discharge is the largest volume with a mean discharge of 75 l/s per m. This means that if even larger mean discharges are needed, the biggest volumes can not simulated anymore with the current device. Under the same conditions mean discharges of 100 l/s per m have 6 waves with a volume larger than 5,500 l/m. A mean discharge of 125 l/s per m has 11 larger volumes.

The smallest overtopping waves are not simulated. This is because the valves in the simulator need some time to open and close. The minimum time needed before the next wave can be simulated is about 5 s. With a mean discharge (actually the pumping capacity) of 50 l/s per m it can be calculated that in 5 s the smallest wave to be simulated is 250 l per m. The total volume of the smaller overtopping waves in the theoretical overtopping distribution is simulated with a fixed number of waves with the minimum volume, giving the same total volume.

The overtopping simulator works with an average discharge over a certain period of time. This means that in this time no variation occurs. In reality sometimes groups of large waves occur. These wave groups can not be simulated, as the will lead to varying discharge. The simulated overtopping waves in the simulated storm are randomly distributed over time.

Table 3. Overtopping parameters for $H_s = 2$ m; $T_p = 5.75$ s, $\cot\alpha = 4$ and a peak of the storm of 2 hours.

	Mean discharge (l/s per m)					
	0,1	1	10	30	50	75
Crest freeboard (m)	5,06	3,84	2,61	2,03	1,76	1,54
% overt. waves	0,2	2,7	18,9	36,6	47	56
No. overt. waves	3	42	289	561	720	858
Max. volume (l/m)	400	835	2110	3790	5180	6750

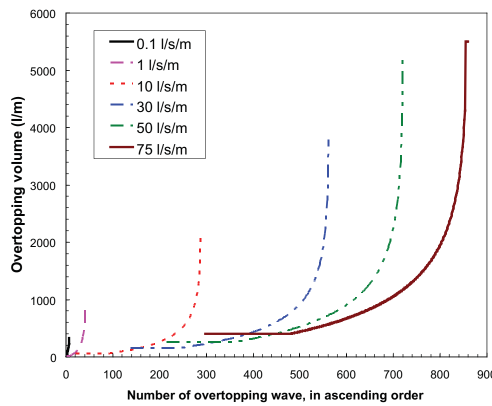

Figure 6. Overtopping distributions for various mean discharges.

Test 50 l/s per m; 2 hours		
Wave	Volume (liters)	Open hour.min.s
1	1556	0.00.27
2	395	0.00.36
3	250	0.00.41
4	1259	0.01.05
5	250	0.01.11
6	250	0.01.16
7	1816	0.01.51
8	301	0.01.58
9	583	0.02.10
10	1042	0.02.30
11	1429	0.02.58
12	679	0.03.13
13	1184	0.03.35
14	277	0.03.42
15	250	0.03.47
16	1355	0.04.13
17	513	0.04.24
18	3565	0.05.34
19	346	0.05.43
20	1403	0.06.10

Figure 7. Part of table with opening times simulator.

The above theory is used to set up a table with opening and closure times of the valve of the wave overtopping simulator. Also the volume of the individual waves is shown, see Figure 7.

6 SET UP OF THE TEST SITE

The width of the test section was 4 m in accordance with the width of the simulator. At each side of the section a boarding was placed with a height of 0.6 m. The panels of the boarding had a length of 2.5 m and were placed just overlapping each other leaving a 'flume' of 4 m wide and 30 m long.

A site office was placed on a flat trailer on the maintenance road. The spacing between the axes of the trailer was over 5 m so the water coming down the slope could pass under the site office. From this site office a good view was possible on the test section. As the office was mobile, transportation to the next section was easy.

In the site office equipment was installed to monitor the tests. This equipment consisted of camera's pointed at the test section, computers to monitor the measurements, storage of data and backup facilities. Also the operating joystick of the overtopping simulator was placed inside the site office. On the roof lights were placed.

After the overtopping simulator was placed the side panels were fitted to the outflow opening of the simulator in order to close the section. Also the boarding was extended under the site office. Behind the side office the water could flow freely, see figure 8.

Next to the boarding a plateau was placed on both sides to facilitate staff to monitor the tests and for visitors. This plateau was equipped with railings.

6.1 *Equipment for measuring*

Several measurements were performed during the tests. Equipment was installed to measure infiltration, front velocities and flow depths.

Infiltration was measured by two sets of tension meters. Each set consisted of 4 instruments. The first set was placed at a distance of about 2/3 of the slope from the toe. The individual instruments were placed from outside the test section. The instruments were placed after a hole was drilled with a long gouge bit. The instruments were placed above each other. The first was placed just 0.2 m below the surface in the top layer. The next was placed 0.4 m below the surface in the clay. The third instrument was placed 0.6 m below the surface at the interface between the cover layer of clay and the sand core. The last instrument was placed in the core itself about 1 m below the surface of the slope. The second set was placed in a similar way, but now on the slope at a distance of about 1/3 from the toe. The instruments were installed one week before the tests started. The sampling frequency was once every hour when no testing was done. As soon as the overtopping tests started the frequency was raised to 0.1 Hz. The data were collected by a logger with a on-line interface to an external location. From this location the data could be collected and stored. Also the frequency could be monitored and adjusted if necessary. In the site office also the data could be monitored to notice possible failures.

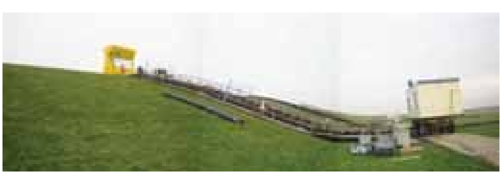

Figure 8. Overview of test site.

To measure the front velocity and the flow depth of the waves overtopping the dike and running down the slope, 5 flow depth instruments were placed on the slope. The distance between each instrument was about 4.5 m. Each instrument consisted of 2 sets of 2 parallel wires perpendicular to the slope. The wires were attached to a steel frame which was secured by a steel portal. The distance between the sets was 0.5 m

With the flow depth instruments the electric resistance is measured between the wires. When the flow depth becomes larger, the resistance between the wires becomes smaller. The instruments are sensitive to air inclusion in the water. If there is a lot of air in the water, the resistance will not be in accordance with the true flow depth. The frequency of measurement was 1000 Hz.

The two sets of wires were placed in line with the slope. During the tests it was observed that the downward set of wires were seriously influenced by the steel frame holding the wires. The flow depths measured by these wires were far from accurate and can not be used in analysis of the flow depths. However, the recorded signal can be used to determine water front velocities on the slope. In figure 10 an example of the flow depth measurements is shown. In this figure also a comparison with visual measurements is made.

The measurements and the visual observations seem to be in line with theory.

The average velocity of the front of the overtopping wave on the inner slope between two instruments can be determined as the distance between the instruments is known as well as the time difference between recordings of each instrument. With 5 sets of

Figure 9. Flow depth instrument.

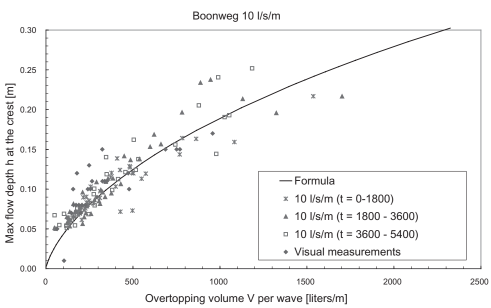

Figure 10. Flow depth measurements.

instruments 4 average velocities over the slope can be determined. An example is shown in Figure 11.

The instruments were calibrated in a big water tank filled with the same water which was used in the overtopping tests.

6.2 *Supporting equipment*

To meet the maximum mean discharge of 75 l/s per m pumps generating the flow needed a capacity of 300 l/s or 1080 m^3/hour. The distance between the water reservoir and the top of the simulator was about 65 m horizontal and 11 m vertical. To meet these criteria an electric pump was installed with a capacity

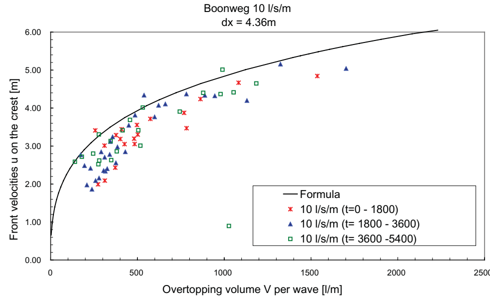

Figure 11. Calculated front velocities between to sets of instruments.

of 1,200 m^3/hour. The flow pipes had a diameter of 0.3 m. In the pipeline a valve was present to fine-tune the required discharge. The flow was set by adjusting the frequency of the pump. The electricity was generated by a 100 kW diesel generator. This generator also powered the hydraulic generator and the site office.

The valves in the overtopping simulator to release waves are operated hydraulically. An electrical driven hydraulic generator was placed next to the overtopping simulator, to give the hydraulic pressure.

7 OVERTOPPING TESTS

The overtopping tests at the Boonweg at Sint Jacobiparochie started with a testing day after a day of setting up the equipment on February 5th. In these tests all equipment was tested to meet the established requirements. The capacity of the pumping system was tested, as well as the instruments for measuring velocities and flow depths. Also the cameras and recording systems were tested.

On February 7th the overtopping tests started at the first test section. This section has a grass cover which is treated like all other dikes in Fryslân (regular maintenance, see Table 1).

Before starting the tests the grass cover was photographed with a hand held digital camera. To identify the location where the photographs were taken, every meter was indicated at the side panels, starting from the outer crest line (outflow opening of overtopping simulator). With a frame of 4 by 2 m every square meter was captured. Whenever changes of the grass cover were observed the frame was placed at the same location as in the photograph before testing. In this way the development of damage could be determined afterwards (together with the digital film from the video camera).

A full test series consisted of 6 tests: 0.1 l/s, 1 l/s, 10 l/s, 30 l/s, 50 l/s and 75 l/s per stretch of 1 meter

Figure 12. Overtopping wave.

dike. All tests had a duration of 6 hours. The 0.1 l/s per m test only has 9 overtopping waves in 6 hours. Also the maximum volume in a single wave is just 770 l/m. It was decided to carry out the test 10 times faster than in reality. This means that in average one wave in every 4 minutes occurred.

Also the 1 l/s per m test was simulated faster. The test was performed 5 times faster than in reality. In this test 126 waves were generated in 1 hour and 10 minutes (in average 1 wave every 33 s.). The largest wave contained 1177 l/m.

All other tests had a duration of 6 hours. After all test series the overtopping simulator and all other equipment was moved to the next test section. A displacement of all equipment took 1 day. After 4 weeks all 4 test sections were finished. All tests have been performed to the end. Only in one case the last test of the series was not completed due to on-going damage of the grass cover and clay layer.

The weather conditions during testing varied. In the first week of the tests temperatures up to 14° C occurred, whereas in the 3rd week average day temperatures dropped to 5° and average night temperatures were below zero. The 3rd and 4th week the temperatures were mild (about 10° C). Precipitation was moderate. In the first 2 weeks there was no rain. In the last 2 weeks in total about 80 mm rain was measured. The first of March there were heavy winds up to 11 Beaufort. At that time the overtopping simulator was secured by 2 4-tons concrete blocks and a mobile crane.

7.1 *Results section 1: Regular treatment; grazed and slightly fertilized*

Most of the overtopping waves in the first test (0.1 l/s per m) did not reach the toe of the dike. All water dissipated into the slope. Only a few waves reached the toe. After the test the grass layed flat on the slope. No damage or start of damage occurred. Larger mean overtopping rates, 1.0 and 10 l/s per m, did not give damage to the grass cover. Only grass blades which were not attached well to the roots came off. In several places some bare spots seemed to occur. At first instance they were believed to be an indicator for start of damage. However after mean overtopping rates of 30, 50 and 75 l/s per m these bare spots still existed and just became a little more bare, not resulting in further damage. In the slope some mice holes were present. Even with the largest overtopping rates these holes did not initiate damage.

During the 30 l/s per m test and following tests damage occurred at the horizontal toe just in front of the slope. This is where the water with high veloc-

Figure 13. Damage test section 1 after 75 l/s per m.

Figure 14. Result section 2 after 75 l/s per m.

ity has to make a transition from the 1:3 slope to horizontal. This leads to forces on the horizontal part resulting in initial damage. The damage which occurred first showed a few clumbs of grass coming out of the lawn. The holes became slowly wider and deeper after each wave. The damage expanded on the horizontal part and did not travel up the slope. At a rate of 50 l/s per m it appeared that underneath the last part of the slope and the first part of the horizontal part at a depth of 0.1 m stones were pitched. These were remains of an old discharge gutter which was filled during the last dike improvement. Once the stones were reached the hole widened faster than before. At the rate of 75 l/s per m even the stones disappeared from the hole. The stones could be found back at the far end of the horizontal area, close to the canal.

7.2 *Results test section 2: Grazed2 times and hayed before winter, no fertilizer*

In this test section superficial tracks of tractors, used to mow the grass, were present parallel to the length axis of the dike. The tracks were all about 2 m apart from each other. Also in this section some holes of mice were present. Again they did not initiate any damage. At the upper part of the slope some bare spots were present before testing started.

These initial spots expanded at little without causing initial damage.

Normaly a fence, to keep the sheep in the section, is present at the toe of the test section. This fence was removed, just before testing, leaving a few holes of the poles of the fence. At these holes a little damage occurred during the 10 l/s per m test. Some lumbs of grass loosened a bit but did not come out.

Contrary to the first test section, the toe of this dike section did not erode yet. Just around the holes of the pole some erosion was seen at 30 l/s per m. At 50 l/s per m still no erosion worth mentioning was seen neither on the slope nor at the toe. Only some minor traces of erosion at the toe were visible. At 75 l/s per m damage occurred. After 2 hours of testing with this overtopping rate the pitched stones 0.15 m below the toe of the dike became visible again. After 6 hours a part of the stones came out en were transported to the far end near the canal. The damage did not proceed downward.

7.3 *Results section 3: Hayed two times, no sheep, no fertilizer*

The third test section has been hayed twice a year. The top layer felt spongy during the tests. In the assessment of the grass quality of this section it was found that the bare spot area was slightly larger than at the other sections. The coverage with grasses is lower than in sections 1 and 2. The vegetation of this section has the highest biodiversity, and thereby from nature conservation point of view the most species rich grassland.

The root density is the largest for this section, compared to the other test sections.

This section showed much more holes of mice. Tracks of vehicles were present in the lower part of the slope and were less visible than in the second section.

In accordance with the previous sections the 0.1, 1.0 and 10 l/s per m tests showed no damage at all. The loose clay material of the first centimeter eroded, just like the first 2 sections and the bended grass blades covered most of the surface. The initial bare spots became a little more evident than before testing.

Some minor damage appeared also in this section after 10 l/s per m, just around the holes of the removed fence.

In the 30 l/s per m test some damage started. A bare spot became even more bare and started to deepen slowly. But after 2 hours this stabilized and did not expand any more.

Figure 15. Local lifting of the grass cover.

Figure 16. Head cut erosion section 4.

At 50 l/s per m damage did not increase. Some little spots at the toe showed initial damage but did not yet lead to ongoing damage.

During the first 2 hours of testing with 75 l/s per m a spot of roughly 0.5 m^2 lifted suddenly approximately 0.2 m. The location of this spot was identified as just below a potential weak spot before testing. At this spot several holes of mice were present.

The lifted spot was washed away by a large wave, leaving a hole of around 1 m^2. In the next 3½ hours of the test, the hole deepened to approximately 0.4 m and widened to 1.4 m. The sand core was not reached after 6 hours of testing with 75 l/s per m.

After the regular testing with 75 l/s per m some visual flow depth measurements were performed. At the 4th wave for these measurements again lifting of the grass cover occurred about 4 m higher on the slope than the first lifting. It was decided to continue regular testing. After 30 minutes the first and the second damage merged. The testing was stopped then.

7.4 *Results section 4: Hayed and grazed once, no fertilizer*

At this section the most weak sections were identified visually before testing started. Also many holes of mice were present. Again tractor tracks were visible at the upper part of the slope.

On the crest of the dike a relatively deep track was present. This track manifested in a bare spot which became even more bare during testing without initiating ongoing damage.

At the 75 l/s per m test in accordance with the 3th section the grass cover lifted 0.1 to 0.2 m over a surface of about a square meter. The location of this lifting was just below a relatively deep tractor track which was identified before testing. After the grass cover of this lifted spot was washed away by a large wave, a second location just below the first lifted and was washed away after a few large waves.

At the second damage the surface deepened to a vertical front, see figure 16.

This hole expanded. The deepening was an ongoing event. After 5 hours and 50 minutes, 45 minutes after the first damage, the test was stopped because the sand core of the dike was reached and was eroding rapidly. The experienced upward head cut erosion went on long after the test was stopped.

8 CONCLUSIONS

Overtopping tests on inner slopes of dikes with differently treated grass covers were performed to investigate their erosion behaviour. It has to be noted that erosion is one of the failure mechanisms which can be initiated by overtopping waves. Sliding of the inner slope or the clay layer is not considered, as the test section was to narrow to initiate this mechanism. Separate research on this failure mechanism will be performed at the end of 2008.

After the tests described in this paper also tests were performed at 2 locations in Zeeland in the Netherlands. Together with the findings in Delfzijl (Akkerman et. al, 2007) preliminary conclusions can be drawn. Together with next tests in different parts of the Netherlands definitive conclusions will be drawn and will be used to develop new standards for safety assessments and design.

The preliminary conclusions of the testing at Delfzijl, Sint Jacobiparochie and Zeeland are based on overtopping conditions with wave heights of about 2 m. situations with much lower of higher waves do not fall under the preliminary conclusions.

– It seems likely that, with a mean overtopping discharge of 30 l/s per m or less, no inner slope of a (Dutch) dike with a clay cover, topped with a grass cover, will fail due to erosion by overtopping waves. Further research may result in a definitive conclusion;
– The resistance against erosion of the inner slope is provided mainly by the grass cover and less by

the quality of the clay layer. The variability of the grass cover has an impact, but it may be less than expected before. This may mean that the treatment of the grass cover has little impact on the resistance against erosion of the inner slope;

– The intersection between slope and horizontal parts is most vulnerable and are possible spots for start and ongoing damage. Further research is needed;
– If damage reaches the sand core, it progresses rapidly by means of head cut erosion. This is only observed at high overtopping rates of 75 l/s per m. With lower rates no ongoing damage occurred.

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