IMPROVING TRANSITIONS AT GRASS COVERED DIKE SLOPES USING THE WAVE OVERTOPPING SIMULATOR

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Most tests with the Wave Overtopping Simulator on grass covered dike slopes have been performed on operational dikes. They showed that transitions and obstacles reduce the strength of the grass cover. The tests in 2023 were performed on sections where innovative solutions were constructed in the dike before testing. The aim was to make transitions stronger than the grass cover itself. The governing transitions were a geometrical transition (from slope to berm) and a transition from smooth asphalt to grass on a horizontal plane. The tests showed that all innovations were successful under test conditions. Moreover, where the front velocity over a slope will increase due to gravity, a horizontal berm reduces the front velocity significantly, giving smaller loads to transitions on the berm. The present tests showed that the underlayer of boulder clay was in this case much more erosion resistant than normal clay. Finally, a cliff erosion test has been performed in order to determine whether an asphalt road on the berm may reduce or terminate cliff erosion. The road terminated cliff erosion.

Keywords: wave overtopping simulator, grass cover erosion, clay erosion, dike transitions, dike breach

INTRODUCTION

Strength of grass cover by the Cumulative Overload Method

The strength of grass covered dikes attacked by wave overtopping has been tested since 2007 using the Wave Overtopping Simulator (Van der Meer et al. 2007, Akkerman et al. 2007, Steendam et al. 2008, Van der Meer et al. 2009). The objective of the first years of testing was to develop a prediction tool for grass cover strength, which was achieved by the so-called Cumulative Overload Method. A preliminary version, based on erosion due to excess of shear stress, was developed by Van der Meer et al. 2010. This was later extended to include transitions in the dike geometry and possible obstacles (Steendam et al. 2014, Hoffmans et al. 2018). It appeared that transitions and obstacles are more vulnerable to damage of the grass cover than a plain grass cover itself.

The Cumulative Overload Method describes the grass cover with an approximate thickness of 0.2-0.3 m. This is the rooted zone, where most of the roots are. It does not describe continuing erosion of underlayer and core. The method is given by:

$$D = \sum_{i=0}^{N} \max[\alpha_M u_i^2 - \alpha_S u_c^2;\ 0] \qquad (1)$$

In which:

D	Cumulative damage number	[m²/s²]
N	Number of overtopping waves	[-]
u_i	Local front velocity at point of interest	[m/s]
u_c	Critical front velocity (= strength of the grass cover)	[m/s]
α_M	Load factor to include geometrical transitions and obstacles	[m/s]
α_S	Strength factor to include geometrical transitions and obstacles	[m/s]

The strength of a plane grass cover without geometrical transitions or obstacles is calculated with α_M = 1 and α_S = 1. Geometrical transitions and obstacles may lead to a larger load (α_M > 1) and/or a reduced strength (α_S < 1). The values of α_M and α_S depend on the type of transition or obstacle.

The cumulative damage number, D, is given by the state of the damage to the grass cover:

Initial damage (first erosion spot)	D = 1000 m²/s²
Damage at several locations	D = 4000 m²/s²
Failure of the grass cover (erosion depth > rooted zone)	D = 7000 m²/s²

The first two damage definitions and values above are not very reliable, as sometimes damage progresses gradually over time and sometimes fast in a final stage of the test. This is why more attention

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is focussed on when the grass cover actually fails: D = 7000 m^2/s^2. If failure of the grass cover is reached during a test, the strength of the grass cover by the critical front velocity, u_c, can be calculated. The description of the strength of grass covers in the Netherlands and surrounding countries (or countries with similar climate and grass vegetation) has been derived by the many tests with the wave overtopping simulator. Other climates and other grass types, like in the tropics, may give other values. For the Netherlands the following critical velocities have been found (with the standard deviation for a normal distribution):

Well maintained closed grass sod on clay	u_c = 8 m/s	σ = 1.0 m/s
Maintained grass, some open spots, on clay	u_c = 6 m/s	σ = 0.75 m/s
Well maintained closed grass sod on a sand core	u_c = 5.5 m/s	σ = 0.8 m/s

The Cumulative Overload Method requires the front velocity, u_i, of every wave at the location of damage. The front velocity of an overtopping wave increases from the crest downwards along the slope, due to gravity. After 6-10 m along the slope the velocity may have increased by 40%, depending on the slope angle. For this reason most damages and failures are found some distance from the crest and not close to the crest.

Transitions in the Cumulative Overload Method

Two main transitions can be determined in a dike section if this section has a berm and a road on top of the dike or on the berm, see Fig. 1. Geometrical transitions appear where a slope turns into a horizontal plane at a berm or at the toe of the dike (left graphs of Fig. 1). The load and strength factors found are α_M = 1.2-1.5 and α_S = 1 (no grass strength reduction), Hoffmans et al. 2018. It is also the reason why in many tests erosion and sometimes failure of the grass cover was found at the toe of the dike. Because of the strength of the remaining dike body, failure of the grass cover at the toe is less critical for the flood probability, than when grass failure occurs near the crest. This is different if a berm is much closer to the crest of the dike.

A transition from a smooth to a rough surface on a horizontal plane results in extra stresses on the grass cover, because turbulence increases on a rougher surface. Such transitions are found at the sides of roads with a smooth surface (right graph of Fig. 1.). The load and strength factors found are α_M = 1.2-1.8 and α_S = 0.9, Hoffmans et al. 2018. The strength factor is smaller than one as the grass cover is likely not connected to the road. It may be clear that especially the large load factors have significantly effect on the strength of the entire grass cover of the dike and therefore on possible dike breaching and resulting flooding.

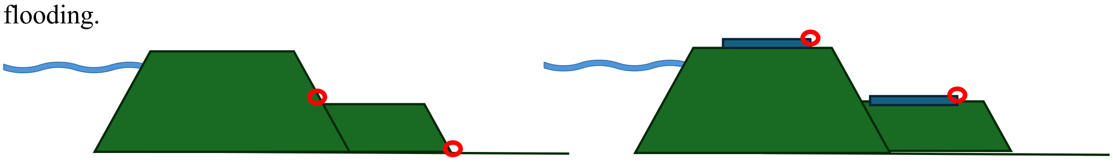

Figure 1. Geometrical transition at a dike (left) and transitions smooth to rough (grass) at the right.

Innovative research on the IJsselmeer dike

Transitions on the landward part of the dike have effect on design or improvement of dikes for wave overtopping, as such transitions and obstacles may require higher dike crests to limit the overtopping. This, in turn, can lead to significantly higher construction costs. This problem was observed in an early stage of a dike reinforcement along the IJsselmeer in the Netherlands and served as the primary motivation for this research project. Initial calculations, taking the transitions into account, led to a significant increase of crest level as well as to significant extra costs. A schematic cross-section of the dike is given in Fig. 2. Two transitions were seen as vulnerable to erosion by wave overtopping:

- The geometrical transition from a slope to a horizontal section. This transition is often present at the toe of the dike, but also at the horizontal berm 3 m below the dike crest. The geometrical transition causes extra loads on the horizontal section.
- The transition from a smooth surface to a rougher surface at a 6 m wide asphalt road on the berm to the adjacent grass cover. The change in turbulence at the transition causes extra loads on the grass cover, whereas the transition itself may also be weaker.

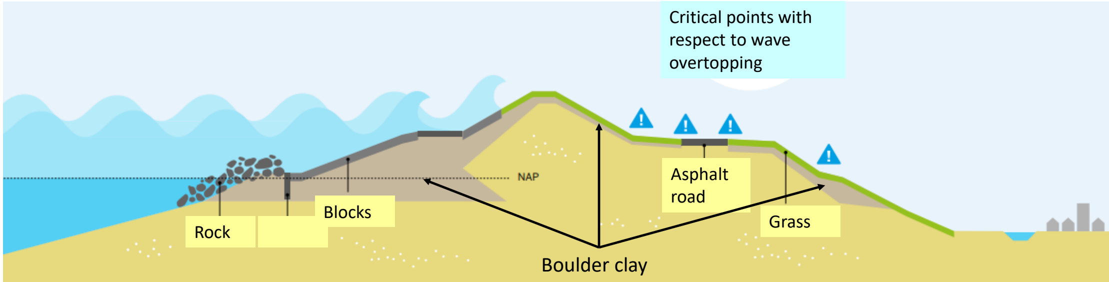

Figure 2. Schematic cross-section of the IJsselmeer dike (dike along the Lake IJssel) with a wide landward berm 3 m below the crest and a 6 m wide asphalt road on this berm. The underlayer of the grass cover consists of boulder clay.

The objective of testing this specific dike along the IJsselmeer near Lelystad in the Netherlands, was to design improved or reinforced transitions, construct these solutions at the dike as test sections and finally, test them with the Wave Overtopping Simulator. Note that the underlayer beneath the grass cover consisted of boulder clay. This boulder clay (deposited and heavily compacted during the ice ages) proved to be much more erosion resistant than normal clay during construction. Another objective was to test the erosion of this boulder clay, that had since then been in the dike, and compare it with clay.

IMPROVED TRANSITIONS AND TEST SECTIONS

The Cumulative Overload Method describes the strength of the grass cover only. If the grass cover fails a clay underlayer may have erosion resistance and may fail at a later stage, delaying dike failure. However, many tests with the wave overtopping simulator on clay underlayers up to 0.6 m thick on a sand core showed fairly quick failure of the clay layer after the grass cover failure, if the wave overtopping discharge was quite large (50 l/s per m or more). It is for this reason that in the Netherlands clay underlayers up to 0.6 m thick are not considered to have significant strength under large overtopping. More research is needed to determine the erodibility of clay for small overtopping (and failed grass cover).

Innovative solutions were introduced that should make the transitions strong and ideally stronger than the plane grass cover on the slope. Some of these innovations do not rely on a possible strength of the grass cover, but replace the underlayer and give enough erosion resistance to that underlayer. With such innovations one should not only consider the grass cover strength, but the strength of the whole system.

During construction of the innovations it was in some cases necessary to carefully remove the grass cover, implement the improvement or innovation and reinforced transitions and place the grass cover back. The same technique of grass transplantation was used as in Akkerman et al. 2007, where also a geogrid was constructed in the grass cover. The improved sections were constructed in June 2023, allowing several months for the grass cover to establish new roots in the clay.

This system worked quite well for a geogrid (Akkerman et al. 2007) as the roots could easily grow into the grid, but it did not work for the present situation. The roots had to grow into the clay and time was by far not sufficient to develop a strong grass cover on top of the solutions. The transplanted grass cover was easily removed by the wave overtopping and was therefore not considered further in testing.

The tests were conducted from late September to early November 2023 just before the winter period, when the grass roots are supposed to become weaker as it will go into a dormant state in countries around the North Sea.

The dike as presented in Fig. 2 has a landward slope of about 1:3 with at a vertical distance of 3 m from the crest and a 14 m wide horizontal berm. This berm included a 6 m wide asphalt road. Three test sections were constructed with each a modified and improved section for the geometrical transition and for the transition from asphalt to grass. The following test sections were prepared and tested:

1. The actual dike, as is, with sharp geometrical transition of slope-berm and the transition from asphalt to grass.
2. The geometrical transition was improved by 0.8 m of good compacted clay from the asphalt road up to a few meters on the upper slope. It replaced theoretically an underlayer of normal clay. The asphalt-grass transition was improved by concrete tiles with a roughened surface, in order to cancel the difference in turbulence at the transition location to the grass, see Fig. 3 left picture.
3. The geometrical transition was improved with a soil stabilization (a mineral cement additive was mixed through the clay), called Mixed-in-place. The layer thickness was 0.3 m and was constructed

from the asphalt road to about 4 m on the upper slope, see Fig 3 right picture. This solution replaced a possible underlayer of clay. The grass cover of 0.2 m was not replaced and the structure was tested directly. In order to stop any head cut erosion at the asphalt-grass transition, this transition was improved by a block - made of the same material as the improved geometrical transition – of about 0.6 m wide and 0.8 m deep, see Fig. 4 left picture.

4. The geometrical transition was made as a gentle curve and not as a sharp transition from slope to berm, see Fig. 5, in order to spread the change in flow direction over a larger area. The asphalt-grass transition was improved by a concrete border with a poured in geogrid extending in the zone of the grass roots, to strengthen the grass cover and connection at the transition, see Fig. 3 middle and right picture. An overall view of the sections on the dike is shown in Fig. 6.

Figure 3. Concrete tiles with roughness in order to cancel the roughness effect at the location of the grass transition (left). Mixed-in-place solution at the geometrical transition (right).

Figure 4. A block of Mixed-in-place of 0.6 m by 0.8 m to prevent failure at this transition (left). A concrete block with geogrid connected to give the grass a stronger connection (middle and right).

Figure 5. A rounded geometrical transition.

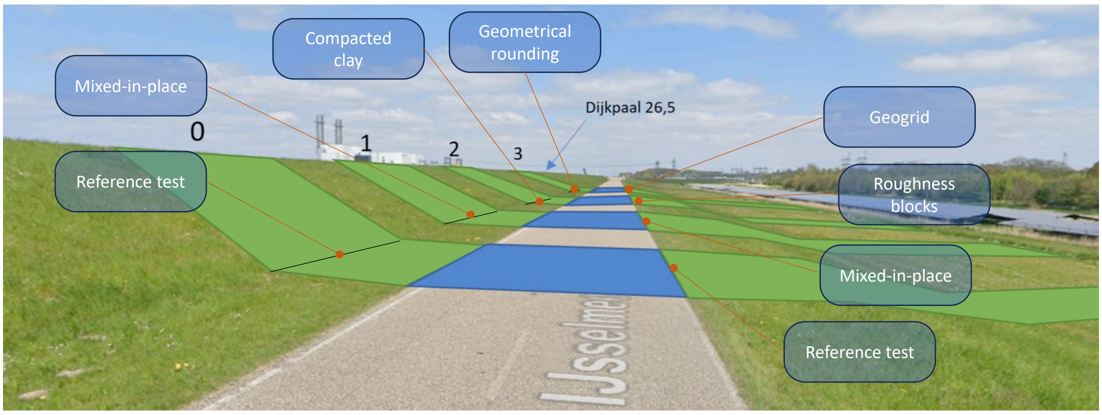

Figure 6. The actual dike and various options with improved solutions for the transitions.

TESTING WITH THE OVERTOPPING SIMULATOR

Test set-up

The Wave Overtopping Simulator that had been developed for tests in Singapore (Van der Meer et al. 2020) was used to perform the wave overtopping tests at a dike along the IJsselmeer. The simulator was placed just on the seaward side of the crest, see Fig. 8. Fresh water from the IJsselmeer (a large lake) was pumped into the simulator with a preset discharge. Guide walls 4 m apart kept the water flow within a constant width.


Figure 7. Test set-up at the dike with a wide berm and an asphalt road 3 m below the crest.

Wave overtopping conditions at the crest of a dike can be calculated by EurOtop 2018, Eqs. 5.10 to 5.13 and Eq. 5.56. Distributions of overtopping wave volumes can be calculated with EurOtop Eqs. 5.52 to 5.54. Although the tested dike itself is a dike along a large lake with quite large design waves in the order of 3 m, test results should also be confirmed for river dike situations with much smaller waves. Therefore two wave overtopping regimes were simulated, starting with a river regime with a significant wave height of 1 m, followed by a sea (and lake) regime with a significant wave height of 3.2 m. Distributions of overtopping wave volumes were calculated for 1, 10 and 100 l/s per m for the river regime and 50 and 100 l/s per m for the sea regime. Fig. 8 gives the five distributions. The various volumes were released in random order during testing.

Fig. 8 shows the large difference between the river and sea regime. When the wave height is only 1 m a large number of waves overtop the dike with relatively small overtopping volumes. With the sea regime, however, much less waves overtop, but the overtopping volume per overtopping wave is much larger. The dotted lines in the graph show the theoretical distributions, the solid lines indicate the released volumes. Very small volumes cannot be released as it takes time to open and close the valve and the simulator has a maximum content of 5300 l/m, which limits the maximum volume that can be released.

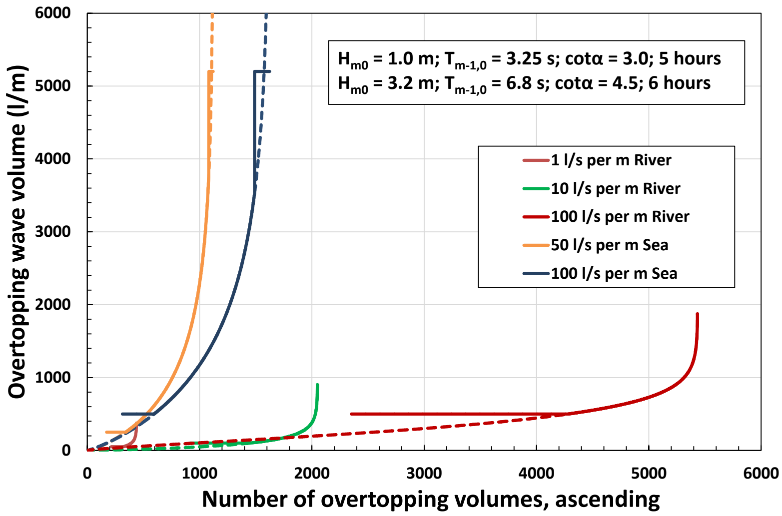

Figure 8. Distribution of overtopping wave volumes in ascending order for tests with a river and a sea regime.

Each discharge event (simulated storm) within the river regime lasted for 5 hours and for the sea regime this was 6 hours. The total simulated storm duration over a full test was 27 hours, during which a total volume of overtopped water of 5.2 million l/m was discharged. During 11 hours an overtopping discharge of 100 l/s per m was released. This test sequence was repeated for every test section.

Front velocities over slopes and berm

The front velocity of an overtopping wave at the crest increases rapidly and significantly when the wave moves down the slope. After 6-10 m from the crest it will find more or less an equilibrium between gravity and friction forces and will then remain more or less constant. For small overtopping wave volumes the front velocity will reduce again due to energy loss by friction forces. In the past 5-10 years the front velocities were measured by a drone above the test section or by a camera on a high pole. The velocities can be calculated by counting the number of pictures in the (high speed) movie between certain distances over the slope. Latest techniques are digital tracking of the wave front.

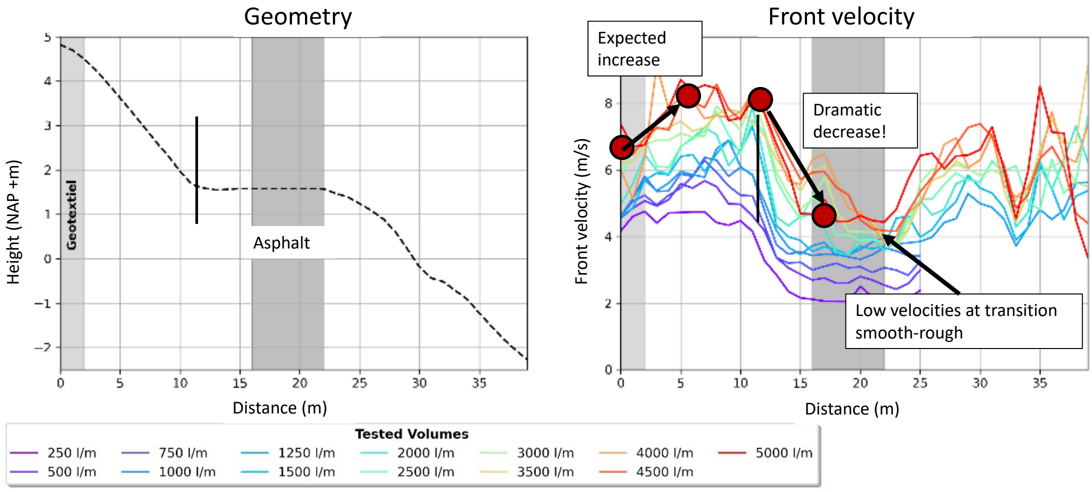

Figure 9. Geometry of the surface of the dike with the asphalt road in grey (left). Measured front velocities over the dike for specific wave volumes (right). The velocity increases over the upper slope, but reduces significantly after the transition to the horizontal berm.

Almost all wave overtopping tests on a dike so far have been performed with a constant and often long slope up to the toe of the dike. The tested dike here has a horizontal berm 3 m below the crest, followed by a second lower slope. This situation resulted in unexpected fast front velocity decreases over the dike geometry! Fig. 9 shows the measurements for distinct overtopping wave volumes from 250 l/m up to 5000 l/m. The left graph gives the geometry of the dike with the berm and asphalt road (indicated in grey). The vertical line gives the transition from slope to horizontal berm and is also given in the right graph of Fig. 9. This right graph shows the development of the front velocities.

From the crest to about 6-10 m from the crest the front velocities increase with approximately 40 % to a kind of maximum, as also expected from earlier tests. But in the 2-5 m after the transition to horizontal (starting from 12 m in the graphs), the front velocity decreases sharply to values that are lower than at the crest! They remain more or less constant over the horizontal part and increase again on the lower slope. There is again a decrease in front velocity at the small berm half way down the slope (around 30-33 m).

The decrease in front velocity over the horizontal berm has large influence on the results for the improved transitions from asphalt road to grass. First of all, the tests did not show significant damage to any of these transitions, which makes it difficult to validate any of the load and strength factors given earlier. The main conclusion is, however, that due to the large decrease in front velocity over the geometrical transition and the berm, the transitions from asphalt to grass are no longer governing for the actual strength of the dike. The high velocities at the grass covered upper slope are normative, not the transitions. this means that improvements in strength at these transitions from asphalt (smooth) to grass are not necessary with these kinds of dike geometries.

RESULTS ON GRASS COVER AND GEOMETRICAL TRANSITIONS

The grass cover on the upper slope of the actual dike section showed sudden initial damage caused by one wave in the 100 l/s per m river regime. The left picture of Fig. 10 shows the situation after this wave and before the next wave reached the hole. The damage occurred in the middle of the slope. The picture was taken from a drone video and shows the hole as well as the remains of the grass cover on the berm and downslope and some clay sediment. This was all removed by the next wave.

Figure 10. Instant development of initial damage half way the upper slope by one wave. The remains of the grass cover and clay sediment is visible on berm and down slope (left). Situation of failed grass cover (a hole in de middle of the test section with a depth of about 0.3 m) after 4 hours of 50 l/s per m with the sea/lake regime (middle). Failure of the geometrical transition (lower part of the picture) after the full test with the sea/lake regime and gully development in the boulder clay underlayer on the slope (right).

The hole gradually widened and deepened and after 4 hours of the next test with 50 l/s per m of the sea/lake regime the depth was almost 0.3 m. This is presented in the middle picture of Fig. 10 and is regarded as the moment of failure of the cover layer. There is also damage on the picture even more upwards on the slope and next to the guide wall. This was caused by a specific accident and was not seen as regular damage to the slope. The hole was covered by a geotextile in order not to disturb further testing, see the right picture in Fig. 10.

Given the location of failure of the grass cover and the time in testing, a critical velocity (strength of the grass cover) could be calculated of u_c = 5.6 m/s, which is just a little smaller than the average of 6 m/s for maintained grass with some open spots on clay. For this calculation the front velocities at the failure location were taken and used in Eq.1 with D = 7000 m^2/s^2, α_M = 1.0 and α_S = 1.0.

The middle picture of Fig. 10 shows that damage occurred over a wider area at the geometrical transition from slope to berm (the down part of the picture). Damage/failure of the grass cover at this transition occurred earlier than at the slope. From this a load factor of α_M = 1.2 could be established.

The rounded geometrical transition (Fig. 5) was covered with transplanted grass, which did not have the full strength of a grass cover. It was eroded quite easily (Fig. 11), but still the test was useful. When the 5 cm thick transplanted grass cover had been removed by overtopping waves, the underlaying clay eroded more or less similar over the full width and length of the rounded geometrical transition. From this it might be concluded that the hydraulic loads on the rounded transition were not concentrated any longer, but spread over the full transition. This also appeared from the front velocities over the transition: the front velocities decreased as much as for the sharp geometrical transition (Fig. 9), but it took two times more distance along the berm to reduce.

The overall conclusion was that the measure of rounding the geometrical transition at the berm, and at the toe of the dike, leads to similar hydraulic loads on the grass cover as for the slope. This means that a load factor of α_M that tends to 1.0 can be used for a sufficiently rounded geometrical transition.

Figure 11. Almost complete erosion of the transplanted gras sod (5 cm thick), but then similar erosion of the underlaying clay of the rounded geometrical transition, showing similar loads over the transition.

The test section with Mixed-in-place for the geometrical transition showed no damage during the full test and may be regarded as a solution that is stronger than the grass cover on the slope. A disadvantage of this solution is that it is impermeable to water. Holes have to be drilled in real application to release rain water from the grass cover above.

The solution with compacted clay showed some erosion, but withstood the whole test and may also be regarded as a strong solution. The question remains whether good compacted clay will remain in the situation as it is, or will become unstructured over many years.

STRENGTH OF BOULDER CLAY UNDERLAYER

The test on the actual dike, as is, section showed that the grass cover failed after 4 hours of testing with 50 l/s per m with the sea/lake regime, giving a critical strength of u_c = 5.6 m/s for the grass cover. Underneath the grass cover a layer of erosion resistant boulder clay appeared to be present. This material was used in the Netherlands in most dikes around Lake IJssel from 1928 to about 1976 and was retrieved from the bottom of the lake. Although clay underlayers up to 0.6 m do not have large strength under severe overtopping conditions (about 50 l/s per m or more), the underlayer of boulder clay, about 1 m thick, appeared to be much stronger or more erosion resistant.

After failure of the grass cover testing was continued to the end with another two hours of 50 l/s per m and six hours 100 l/s per m with the sea/lake regime. The hole in the grass cover became a gully that extended downwards to the geometrical transition and then widened and deepened, see the middle part of the right picture in Fig. 10. The width became more than 1 m with a depth up to 0.6 m. The sand core was not reached during testing, although it was not far from this situation.

The Cumulative Overload Method is applicable for grass covers, not for underlayers. But it is possible to use the method to describe more quantitatively the strength of the boulder clay underlayer compared to the grass cover. The starting point is the moment of failure of the grass cover layer (D = 7000 m^2/s^2 and u_c = 5.6 m/s). Given a critical velocity for the boulder clay underlayer, it is possible to calculate a D-value till the end of the test. In this way D-values of respectively 22,172 m^2/s^2; 11,268 m^2/s^2; 4,175 m^2/s^2 and 1,018 m^2/s^2 were calculated for assumed critical velocities of u_c of 5 m/s; 6 m/s; 7 m/s and 8 m/s.

To give an example for comparison: after failure of the grass cover with u_c = 5.6 m/s it would take two times the same wave overtopping conditions to come to failure of the 1 m thick boulder clay underlayer. A D-value of 11,268 m^2/s^2 with a u_c of 6 m/s is almost similar to a D-value of 14,000 m^2/s^2 with a u_c of 5.6 m/s. And that is double the D-value of 7,000 m^2/s^2 for failure of the grass cover.

STRENGTH AND FUNCTION OF AN ASPHALT ROAD ON DIKE BREACH BY CLIFF EROSION

A final and extra test was performed after testing the improved transitions. It is a very specific test that had never been performed before. Although an asphalt road gives extra transitions and from this point of view may weaken the dike, it is also a strong element on the surface (crest or berm). It will not be eroded by overtopping water, but it may have a function in the failure path to a final breach in the dike. Failure of the grass cover may lead to failure of the underlayer and then the sand core will be attacked by overtopping waves. This will create a larger hole and fast erosion of sand, followed by more erosion of above laying underlayer and grass cover which fall into the hole (a vertical cliff will be reached). The progression of this failure mechanism is upwards the slope and will finally reach the crest with a dike breach as result. The mechanism is called head-cut erosion and has been observed during some wave overtopping tests, although always to limited erosion before the test was terminated.

The assumption is that the grass cover may fail at the down slope, followed by failure of the underlayer and then cliff erosion, or head-cut erosion towards the crest of the dike. This process may then find a strong element like an asphalt (or concrete) road on top of the dike or on a berm. The question is: what will happen to the cliff erosion? Will it give some or very much resistance, or will it easily fall into the hole?

Figure 12. Creation of a cliff behind the asphalt road onto the sand core (left). Top view of the cliff erosion hole with the remains of 1 m grass cover with boulder clay underlayer and the 0.7 m wide Mixed-in-place block against the asphalt road (right).

A hole of 4 m wide and 1.3 m deep was made behind the asphalt road, see Figure 12. At a depth of 1.3 m the sand core was found that was also present underneath the road. From this depth a slope of 1:8 was made towards the downslope, through the boulder clay layer. Side slopes of about 45 degrees were made in order to avoid these side slopes falling into the hole. In this way a vertical cliff was created onto the sand core.

The test section with the Mixed-in-place block behind the asphalt road was used for this test. This block with a width of 0.7 m was left in place (see right picture of Fig. 12) as well as 1 m of grass cover with boulder clay underlayer. So, the test did not start directly at the asphalt road, but with real cliff erosion of the slope. Also the asphalt road was cut to a 4 m wide section (along the guide walls), that had no contact anymore with the asphalt on both sides of the section.

Roads may be present on river dikes as well as on sea or lake dikes. The test started with a discharge of 100 l/s per m for the river regime (the most severe test for river dikes) for five hours. In a second stage the test was continued with 100 l/s per m for the sea/lake regime, also for five hours.

Sand eroded from underneath the boulder clay and after 45 minutes with the 100 l/s per m river regime, the 1 m wide layer was fully eroded and fell into the hole (left picture of Fig. 13). Another 30 minutes later the Mixed-in-place block was undermined, broke and fell into the hole (right picture of Fig.13). Both cases give “cliff erosion speeds” of about 1.3-1.4 m/hour.

Figure 13. Cliff erosion test. The 1 m wide (boulder clay with grass cover) fails after 45 minutes (left). The 0.7 m wide Mixed-in-place block fails 30 minutes later (right).

Figure 14. Start of cliff erosion at the asphalt road (left). Undermining 1.5 m after 5 hours of testing (right) and the erosion was almost terminated.

After removal of the Mixed-in-place block from the erosion hole, the cliff erosion test for the asphalt road could start (left picture in Fig. 14). Erosion of the core underneath the asphalt road started and revealed that the asphalt had been placed on top of the old brick road that was constructed in the seventies during construction of the dike (right picture of Fig. 14).

This picture also shows the vertical cliff of sand underneath the road and an undermining of 1.5 m that was reached after full testing with the river regime. At the foot of the cliff a sand slope was formed that went into the hole where the overtopping waves were falling. Overtopping wave volumes came with large velocities into the hole, plunging on the water that remained in this hole. Only small waves generated by the plunging volumes reflecting backwards reached the sand slope and sometimes also the vertical cliff underneath the asphalt road. In this way the cliff erosion decreased and in the second stage of the test with 100 l/s per m with the sea/lake regime, the total erosion increased by only 0.2 m to a final undermining of 1.7 m. After this the cliff erosion was terminated.

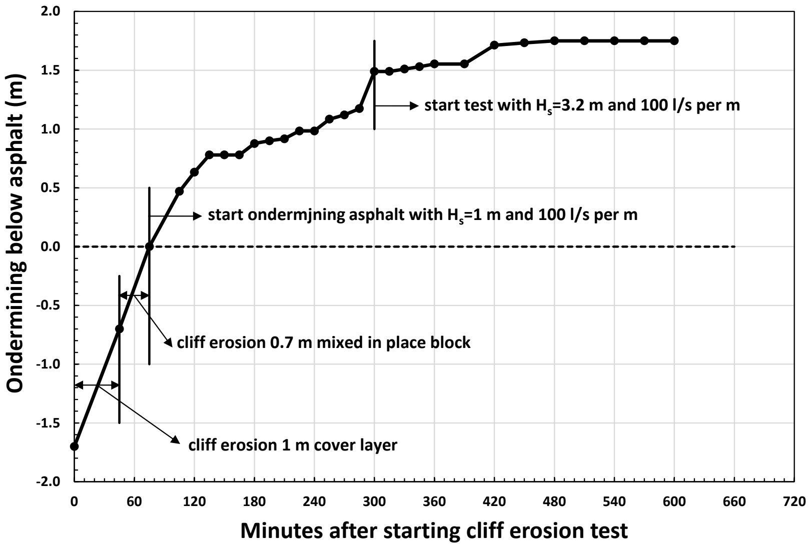

Figure 15. Development of cliff erosion (undermining) at the asphalt road in time.

Figure 15 shows the cliff erosion of the boulder clay cover layer and Mixed-in-place block in time, together with the undermining of the asphalt road. The actual cliff erosion went faster than the process of undermining the road. It is also clear that the undermining already terminated after half of the second stage of the test.

The conclusion is that this 6 m wide and 0.25 m thick asphalt road was indeed a strong element in the dike as it terminated cliff erosion even during the 100 l/s per m sea/lake regime. But when and how would the asphalt road fail if undermining would have continued?

To find out, undermining was forced manually by a jet pipe. Sand was slowly eroded over the full width till the road collapsed, see Fig. 16. The asphalt slab broke over the full width by overturning due to its own weight when the undermining reached a depth of 2.9 m, almost half of the width of the road.

It is a classical mechanical failure and can calculation wise be reconstructed, using the properties of the asphalt material. It means that the failure of other roads that are less wide (cycle path) or less thick, or with another type of asphalt, can also be calculated. Of course, if the undermining becomes more than half of the width of the road, it will not mechanically fail, but can just overturn as a whole into the cliff erosion hole.

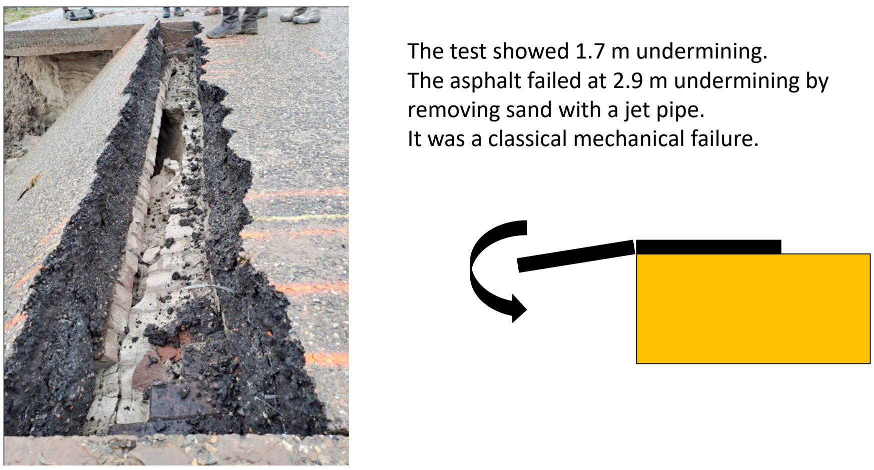

Figure 16. Classical mechanical failure after forced undermining up to 2.9 m.

The cliff erosion test showed that the tested dike with a boulder clay underlayer under the grass cover and a wide and strong asphalt road terminated cliff erosion. The boulder clay on the downward part of the slope had a positive effect on the depth of the cliff and erosion hole. In that sense the undermining could have been more if just a clay layer had been present underneath the grass cover. The dike core underneath the road was also dry (unsaturated). In the situation of a river dike with very high water levels for weeks, it may be that the core would have been fully saturated. The effect of this is not known.

Overall, the cliff erosion test gave a very positive result, but it is too early to apply this knowledge much wider with confidence and it is proposed to continue research in this area.

DETERMINING THE ROUNDING FOR THE GEOMETRICAL TRANSITION

During the field tests only the existing sharp geometrical transition and a very well-rounded geometrical transition with a radius of 24 m were tested. To determine more optimised radii for the geometrical transition, the Computational Fluid Dynamics (CFD) model OpenFoam was used after the performance of the field tests with the Wave Overtopping Simulator. In this way more dike geometries with rounded geometrical transitions could be analysed.

The tests using CFD were performed in two stages. During the first stage, a model was created using a LiDAR survey of the tested (reference) dike section. This model was calibrated and validated by comparing the wave front velocities over the dike geometry as tested. By use of this methodology the correct roughness factors were calibrated for the OpenFoam model.

During the second stage of testing, the dike geometry was altered to contain geometrical transitions with radii of 0, 6, 12, 20, 25 and 30 m, see Fig. 17.

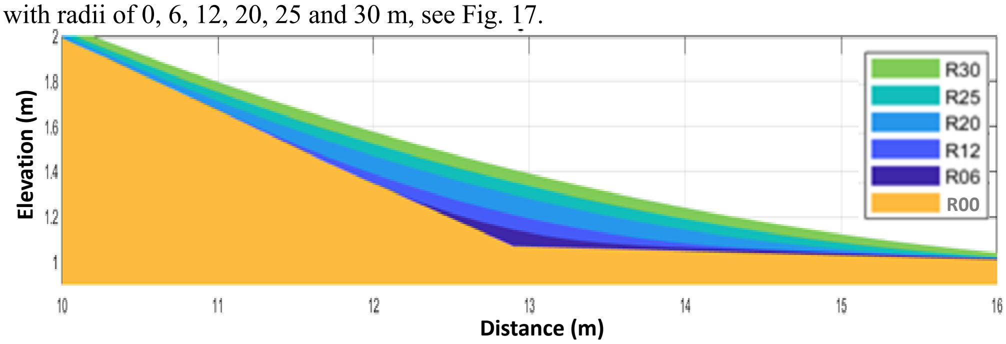

Figure 17. Modelled geometrical transitions with radii of 0, 6, 12, 20, 25 and 30 m.

For overtopping volumes of 2000 l/m (large volume in the river regime), 3500 l/m and 5000 l/m (large volumes in the sea/lake regime), the behaviour of the wave front velocities was studied and compared to the front velocities which were measured during the field tests.

During this study it was concluded that application of (large) radii for the transition from the slope to a flat surface will cause a smaller gradient in the loss of wave front velocity. This smaller gradient results in a smaller loss in velocity per meter of transition, which results in a smaller load on the grass cover of the dike.

Simulations with a radius of 6 m gave front velocities which were equal to the velocities as measured in the field during the reference tests. In the LiDAR survey it turned out that the 6-meter radius is observed as a sharp transition from the slope to flat surface.

For waves up to 2000 l/m (the river regime), rounded transitions with radii larger than 12 m cause a significantly lower front velocity and smaller gradient in the loss of velocity. For waves of 3500 and 5000 l/m (the sea/lake regime) the same image was presented for radii equal to and larger than 20 m.

CONCLUSIONS

The present tests were conducted on a dike with a wide horizontal berm 3 m below the crest of the dike. It appeared that the front velocity of the overtopping waves increased from the crest over the slope, as expected and always seen before, but it reduced significantly at the geometrical transition to a horizontal area. Front velocities became even smaller than on the crest. The consequence of this is that transitions at the horizontal berm, like from smooth asphalt to grass, are not governing anymore if a sufficiently wide berm is present of at least 4 m. The innovative improvements at this asphalt-grass transition showed hardly any damage, but are also not necessary as the strength of the grass slope is governing in this specific situation.

Mixed-in-place solutions or a thick layer of compacted clay at the geometrical transition from slope to horizontal, are strong solutions to increase the strength of the underlayer of the dike. More study is required for application in reality though, as there might be practical problems that have to be solved.

A relatively easy and efficient way to improve a geometrical transition from slope to horizontal (at a berm or at the toe of the dike) is to round the edge. A radius of 12 m is proposed for river dikes (smaller wave heights, smaller front velocities) and a radius of 20 m for sea and lake dikes with larger wave heights. This implies a width of the gradual transition of roughly 4 to 6 m given a slope of 1:3.

The erosion resistant boulder clay as underlayer in this test is maybe a typical Dutch phenomenon. If it is expected that high erosion resistant underlayers are present in an existing dike, the strength could be taken into account. First figures of the resistance of the tested boulder clay are given by combinations of critical strength and damage numbers that can directly be compared with the strength of the grass cover.

Cliff erosion may be terminated by a strong and wide asphalt road. And as such an asphalt road can be a really strong element in the dike geometry. More research is required to make this knowledge generally applicable.

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