

WAVE OVERTOPPING SIMULATOR TESTS ON VIETNAMESE SEA DIKES

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Little is known about the strength of the land-side slopes of the Vietnamese sea dikes under overtopping attack. Some grass-covered slopes were tested by a wave overtopping simulator. The paper describes the simulator and presents the test results. The tested grass covers could withstand overtopping rates varying from 20 to 100 l/s per metre of dike length in some hours. Damage usually started at bare spots, at the transition between the slope and the toe, at the transition between different materials, and around objects (e.g., big trees). These features reduce the strength of a grass cover and therefore should be avoided.

Keywords: grass cover; wave overtopping; simulator.

1. Introduction

In the north of Vietnam, the sea dike system is defined along the coastline of Quang Ninh, Hai Phong, Thai Binh, Nam Dinh and Ninh Binh provinces and has a total length of more than 700 km. The sea dike system was built with construction experience accumulated over centuries. The fundamental function of this system is to protect agricultural land against sea flooding and to prevent salt water intruding. In principle, a dike is constructed of a body of soil, an armoured sea-side slope and crest, and a grass-covered land-side slope. In general, dike crests are not high enough to prevent wave overtopping during design storm surges and high tide. Damage in-

duced by wave overtopping on the crests and land-side slopes has caused a large number of sea dike failures. For example, overtopping was estimated to contribute up to 46% of the total failure probability of a sea dike in Nam Dinh province [Cong, 2010].

To date, little is known about the strength and stability of the land-side slopes of the present Vietnamese sea dikes under the impact of wave overtopping. Overtopping discharges were estimated to vary roughly from 30 to 300 l/s per meter of dike length during storm surges [Trung, 2008]. According to the sea dike design guidelines currently applied in Vietnam, a discharge of 10 l/s per m is accepted on the land-side slope which is evenly covered with grass. In fact, the strength of a grass-covered slope can only be quantified by evaluating the post-storm condition of the structure after the actual storm surge event. Furthermore, due to difficulties in performing observation and measurement during storm surges, it is almost impossible to investigate the damage process of a grass-covered slope. The critical rate of wave overtopping at sea dikes remains an issue that needs to be further studied.

Basic processes in coastal engineering are usually studied by using numerical and/ or physical models depending on focusing process. There are two main concerns: first, the available theory describing the process and the corresponding solutions; and second, whether it is possible to physically model the process.

The phenomenon of wave overtopping on coastal defences has been studied intensively for decades using physical models. On the basis of a large number of physical experiments, formulae have been established to characterise the phenomenon such as mean discharge, probability of overtopping, volume of a single overtopping event and distribution of overtopping waves during a storm surge [Pullen, 2007]. However, the effects that overtopping waves have on grass-covered slopes are not comprehensively understood. That is mainly due to the fact that these effects cannot be studied in a small wave flume as it is impossible to scale down the properties of soil and grass. It is possible to build up a prototype dike cross-section in a large wave flume by taking large blocks from a real dike. Only one type of grass cover can be tested for each cross-section; and transitions and objects like trees are difficult to build. To address this issue, a device that can simulate wave overtopping tongues on real dikes was developed in the Netherlands [Van der Meer, 2006; Van der Meer, 2007; Van der Meer, 2008].

The wave overtopping simulator was used to test the strength of various grass-covered slopes of sea dikes and river dikes in the Netherlands and Belgium between 2007 and 2013. These tests revealed that a good grass cover could withstand overtopping rates in the order of 75 l/s per m, but bad grass covers (many open spots) failed even when overtopping discharges was as low as 1 - 10 l/s per m [Van der Meer, 2009]. Note that the quality of a grass cover was assessed according to categories given in the safety standard of flood defences in the Netherlands [VTV, 2006]. Damage was observed to start at weak areas of a slope such as the transition between the slope and horizontal dike toe, and around obstacles, such as a big tree or

a staircase [Steendam, 2010]. The erosional resistance against wave overtopping on a grass-covered slope is mainly determined by the root condition and rate of grass covering rather than the soil characteristics [Steendam, 2008].

In the north of Vietnam, the sea dikes are different in cross-sections, material components and grass species compared to those either in the Netherlands or in Belgium. It is not feasible to interpret and apply the findings obtained after the Dutch and Belgian tests to the Vietnamese sea dikes without validation testing. Therefore, testing sea dikes with the simulator is strongly encouraged in Vietnam. In late 2008, the original design of the simulator was adjusted and improved to make another machine. Since 2009, this new simulator has been utilised to test the strength of several grass-covered dike slopes in Hai Phong, Thai Binh and Nam Dinh provinces. The performance of these slopes under wave overtopping attack was observed and investigated in order to understand and improve the present sea dike structures.

The results of the Vietnamese tests are presented in the paper. After the introduction, the operation principle and design of the simulator are given. Test sections and experimental conditions are described in the third section. Main test results are introduced in the fourth section. The formation and development of erosion induced by overtopping flows are discussed in the fifth section. Finally, conclusions and recommendations are given.

2. Wave Overtopping Simulator

2.1. Failure mechanisms due to wave overtopping

Wave overtopping may cause damage to the crest and the land-side slope of a sea dike covered with grass. In principle, two different mechanisms can be distinguished as shown in Figure 1. The first one, i.e., sliding of landward slope, mainly occurs on steep slopes, especially between 1/1.5 and 1/2.0, due to water infiltration and sliding. This sliding may directly cause dike collapse. However, the mechanism is not likely to be due to wave overtopping, but rather due to large quantities of water infiltrating the dike body. This process may be aggravated by heavy rainfall.

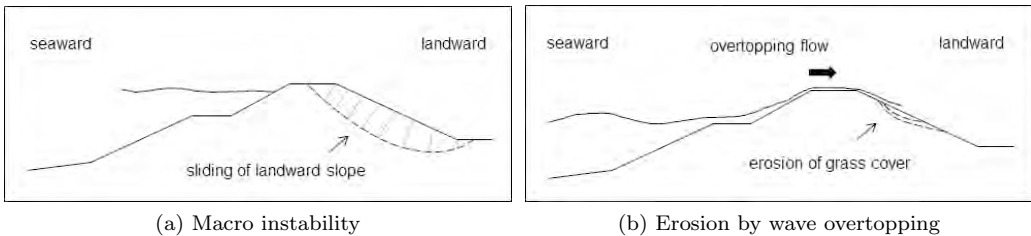


Fig. 1: Failure mechanisms of landward slopes.

The present study focuses on the second mechanism, i.e., erosion by wave overtopping, induced by fast overtopping flows. If initial damage due to erosion occurs, it can extend to the material layers underneath and may possibly lead to a dike breaching. The Wave Overtopping Simulator can be used to simulate the fast wave overtopping on real sea dikes. The simulated process is often limited to the initial stage in which grass cover is eroded and the lower material layer is exposed.

2.2. Operation principle and design

The Simulator is a water reservoir which can be transported from one place to another and assembled or dismantled in a reasonable time (one to three days). When working, the simulator is continuously filled by water with certain constant rate and is emptied at predefined timings through a butterfly valve at the bottom. The released water volumes simulate (generate) the overtopping wave tongues on the dike crest and then on the land-side slope. Figure 2 illustrates the principle of the simulator and shows how it releases water on a grass slope. As long as the velocity and the depth of the released flow are similar to that of actual wave overtopping flow on the crest, the flow behaviour on the land-side slope should be similar to each other. The theoretical background and design of the simulator were described in another report [Van der Meer, 2007].

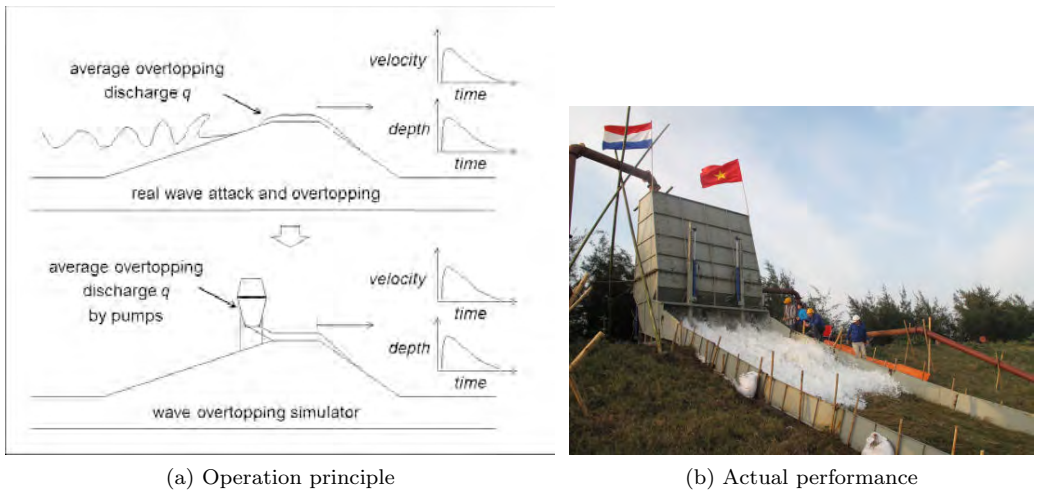


Fig. 2: Wave Overtopping Simulator.

The Vietnamese simulator was constructed in 2008 and 2009. Its maximum dimensions are 4 m-wide (along the dike stretch), 2 m-thick and 5.5 m-high giving a maximum volume of 5.5 m^3 per unit width (m) and therefore a total volume of $5.5 \times 4 = 22 \text{ m}^3$. With these dimensions, a mean wave overtopping discharge of 100 l/s per m can be produced (simulated). The simulator is divided into two parts

for convenience of transporting. The open width of the butterfly valve is 0.8 m giving a good simulation of the water overtopping on the dike crest. The hydraulic pump, which opens and closes the valve, is attached to the machine body. The vertical position of the simulator can be adjusted by six hydraulic cylinders.

3. Test sections and test scenarios

3.1. Description of test sections

In 2009 and 2010, tests with the simulator were performed on two real dike stretches. Positions were selected to satisfy the following operational and test requirements. Operation requires that heavy trucks (20 tons) and a 25-ton crane can approach the site safely. A crest with width of at least 4 m is required for positioning all equipment such as the simulator, the generator and the two pumps. The selected slope has to be covered partly or entirely with grass. A water source, either a canal or a pond close to the dike, is necessary to supply water for testing. A volume of roughly 9000 m³ is estimated sufficient for testing with maximum discharge of 100 l/s per m and for maximum duration of 6 hours.

In line with previous works in the Netherlands, the quality of a grass cover is evaluated using the categories proposed in the Dutch safety standard of flood defence [VTV, 2006]. A grass cover can be classified as 'good', 'moderate' or 'bad' depending upon the density and distribution of roots over depth (usually within 20 cm under the slope surface). Investigation of the tested grass covers is beyond the scope of this paper and is elaborated in another report [Trung, 2012]. On site, the covering rate of grass can be roughly estimated by bare eyes. The two test sites are described in the following sections.

The first site was in Nam Dinh province, where the Thinh Long dike was constructed of sand (core) and clay (outer layer). The dike crest level is at +5 m above Mean Sea Level (MSL), and the crest width is 4.5 m and is paved with 20 cm-thick concrete. The dike toe was about 10 m long and was covered with Ray grass (*Panicum Repens*). The 1:3-inclination slope was 10 m long, and was covered with mainly Bermuda grass (*Cynodon Dactylon*), sometimes in combination with Casuarina trees. Bermuda grass was about 4 years old when testing. Three sections were selected. TL1 was a regular slope covered with good grass; TL2 had a big Casuarina tree in the middle; and TL3 had some open spots (i.e. no grass). Water was pumped from a brackish water canal parallel to the dike toe. The Thinh Long dike cross-section is graphically illustrated in Figure 3.

The second site was on a estuary dike in the Thai Tho commune, Thai Thuy district, Thai Binh province. A combination of Bermuda grass of 5 years old and 6-month-old Vetiver grass (*Vetiver Zizanioides*) on the riverside slope was tested at three positions: TT1, TT2 and TT3. The dike body was constructed of clay and the 1:3-inclination slope was reinforced with concrete frames from crest to toe. These sections were chosen to have different concrete beam configurations and grass

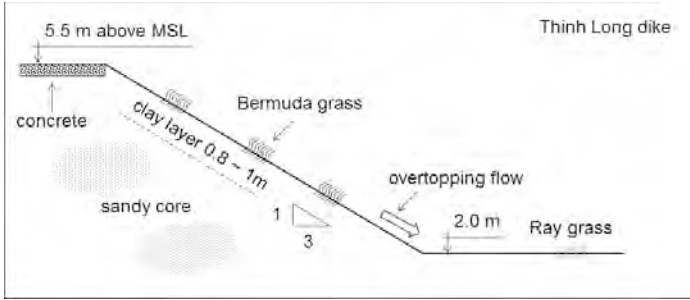


Fig. 3: Cross-section of the Thinh Long sea dike.

quality. The horizontal toe was regularly covered with a dense sward of Ray grass. Water was pumped from Tra Ly river. A schematization of the Thai Tho dike is given in Figure 4. A brief description of all slope sections is given in Table 1 such as the length of the slope and how evenly the slope is covered by grasses.

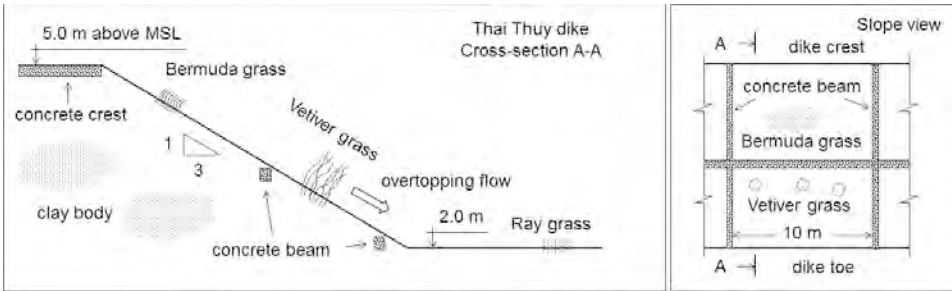


Fig. 4: Schematization of the Thai Tho sea dike.

3.2. Test scenarios

A wave condition was determined to represent the storm characteristics of the northern coast of Vietnam. All tests were conducted with a wave height, H_{m0} , of 1.5 m and a peak period, T_p , of 6 s. The sea-side slope of every dike was assumed to have a $\tan \alpha$ of 1/4. Various overtopping discharges were simulated varying from 10 to 120 l/s per m at the six sections. Each mean discharge characterised the duration of the storm, e.g., 4 or 6 hours. The test was terminated if the grass cover was seriously damaged. Note that the product of the mean overtopping discharge and the estimated duration of the storm corresponds to the total volume of water applied to the slope. The total volume of water depends on both the volume of overtopping water due to each wave and the number of waves overtopping during the storm.

Each simulated storm consists of a number of waves, whose overtopping volumes

Table 1: Description of the tested slope sections.

Section	Length of the slope	Regularity	Grass	Structures
TL1	10.0 m	regular	good Bermuda	no
TL2	10.0 m	irregular with some small holes	poor mixture of Bermuda + Carpet grass, a small Casuarina tree	no
TL3	9.5 m	relatively regular	very poor Bermuda + Ray grass around dike toe	no
TT1	9.0 m	regular	good Bermuda + Vetiver grass	concrete frames
TT2			poor Bermuda + Vetiver grass	
TT3				

are computed with a given condition of wave and discharge. The procedure and formulas of the computation follow the theory presented in the Overtopping Manual [Pullen, 2007]. Figure 5 shows the non-exceedance curve of overtopping water volume per each wave per unit width. In reality, wave overtopping is a random process and for that reason, the volume order depicted in Figure 5 is randomized. This random series is used to to steer (open/ close) the butterfly valve of the simulator to release water flows on a real dike crest. More explanation is given in Appendix A.

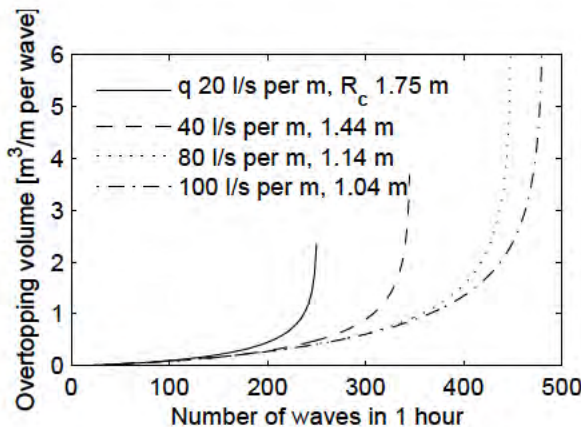


Fig. 5: Non-exceedance curve of overtopping wave volumes (m^3/m per wave) in 1 hour with different values of mean discharge using $H_{m0} = 1.5$ m, $T_p = 6$ s and the theory of the Overtopping Manual EurOtop 2007. The dike crest freeboard is R_c .

Overtopping parameters derived from calculation with various discharges are given in Table 2 including crest freeboard, percentage of overtopping waves, number of overtopping waves and maximum overtopping volume. Besides, each discharge is associated with corresponding durations applied at every sections. For example, section Thinh Long TL1 was tested with 4 hours of 10 l/s per m, 4 hours of 20 l/s per m, 4 hours of 40 l/s per m and 3 hours of 70 l/s per m. Note that at section TT1, the dike toe was always submerged in water 10 to 20 cm deep. This water buffer was predicted to reduce the flow impact on the dike toe. At the other sections, water was free to flow over the dike toe.

Table 2: Wave overtopping parameters for various values of mean discharge using $H_{m0} = 1.5$ m, $T_p = 6$ s and the theory of the Overtopping Manual EurOtop 2007; and applied duration of these discharges at each test section.

Overtopping papameters	Mean overtopping discharge (l/s per m)						
	10	20	40	70	80	100	120
Crest freeboard R_c (m)	2.27	1.93	1.60	1.32	1.26	1.15	1.06
Percentage overtopping waves P_{ov} (%)	22	33	47	60	63	68	72
Number overtopping waves N_{ow}	152	231	327	412	433	468	492
Maximum overtopping volume V_{max} (l/m)	1330	1900	2860	4120	4520	5300	6060
Thinh Long TL1	4 h	4 h	4 h	3h			
Thinh Long TL2	4 h	1h49m					
Thinh Long TL3	4 h	4 h	2h20m				
Thai Tho TT1		4 h	4 h		4 h	4 h	
Thai Tho TT2		4 h	4 h		4 h		1 h
Thai Tho TT3		4 h	4 h		4 h	2 h	
Test section	Duration (hours : minutes) of each discharge						

3.3. Experimental procedure

The simulator was positioned on the dike crest to release water to the grass-covered slope. Two 50-cm-high side walls were erected running from the simulator bottom gate to the dike toe in order to guide the overtopping flow (see Figure 2). A test with a certain discharge was interrupted after every hour to investigate the condition of the slope section by measuring cross-profiles and taking pictures. Slope profiles were measured along lines which are parallel to the left side walls (facing the simulator gate) with increasing distance 1.0, 1.5, 2.0, 2.5 and 3.0 m. The experimental results at the dike stretches Thinh Long (TL) and Thai Tho (TT) are briefly described in the next section and more details can be found in another report [Trung, 2011].

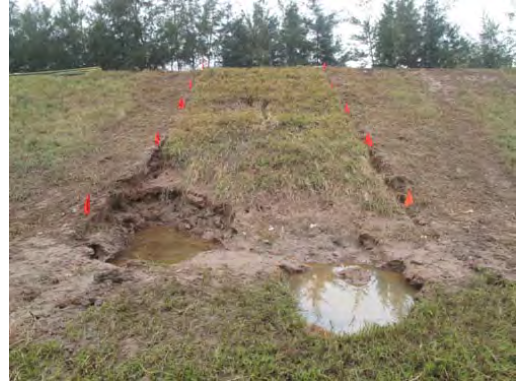
4. Main test results

4.1. *Thinh Long Dike*

At section TL1, the slope started to be eroded after three 4 hour discharges of 10, 20 and then 40 l/s per m. After that, 1 hour of 70 l/s per m was applied and damage was clearly recognised at various points on the slope and at the toe. Testing two more hours with 70 l/s per m seriously damaged the grass cover, and the test was stopped. Figure 6 compares the condition of grass cover before and after testing at the TL1 section. Figure 7 illustrates the slope profiles measured along a line parallel to the left side wall with a distance of $a = 2.5$ m. Changes of these profiles give an impression of the deepest eroded hole at the dike toe. The vertical axis gives the elevation above the Sea Water Level (SWL) and the horizontal axis gives the horizontal distance from the crest edge where the grass cover starts.



(a) Before testing



(b) After testing

Fig. 6: Grass cover at Thinh Long TL1 section, maximum applied discharge was $q_{max} = 70$ l/s per m.

At section TL2, there was a *Casuarina* tree with a trunk diameter of 7 cm in the middle of the slope. After a couple of hours testing with 10 l/s per m, the first damage was extended from an existing hole towards the tree. After 2 more hours of 10 l/s per m and 30 minutes of 20 l/s per m, the tree was swept away, thus resulting in a large hole with a length of 3 m (in the crest-to-toe direction) and a depth of about 1 m, as can be seen in Figure 8. The changes of the slope profiles measured along a line parallel to the left side wall with a distance of $a = 2.0$ m is depicted in Figure 9.

The third section TL3, which was unevenly covered with a combination of Bermuda grass and some small *Casuarina* trees (trunk diameter of 1 cm), was first eroded after 4 hours discharge of 10 and 20 l/s per m each. The slope was damaged at several points after 2 more hours discharge of 40 l/s per m. The test finished

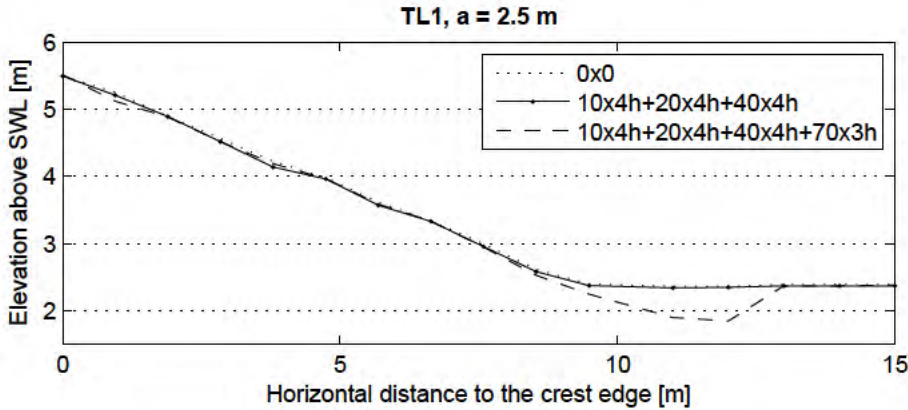


Fig. 7: Changes of the slope profiles measured parallel to the left side wall (facing the simulator gate) with a distance of $a = 2.5$ m at section Thinh Long TL1. 0x0 is the initial profile; $10 \times 4h + 20 \times 4h + 40 \times 4h$ is the profile measured after 4 hours of 10, 20 and 40 l/s per m each.



Fig. 8: Grass cover at Thinh Long TL2 section, maximum applied discharge was $q_{max} = 20$ l/s per m.

after another hour of 40 l/s per m discharge when one of the eroded holes became significantly large with a length of 5 m (in the crest-to-toe direction) and a depth of 1 m. The initial and final condition of the grass cover are shown in Figure 10. Changes of slope profiles measured along a line parallel to the left side wall with a distance of $a = 1.5$ m are given in Figure 11.

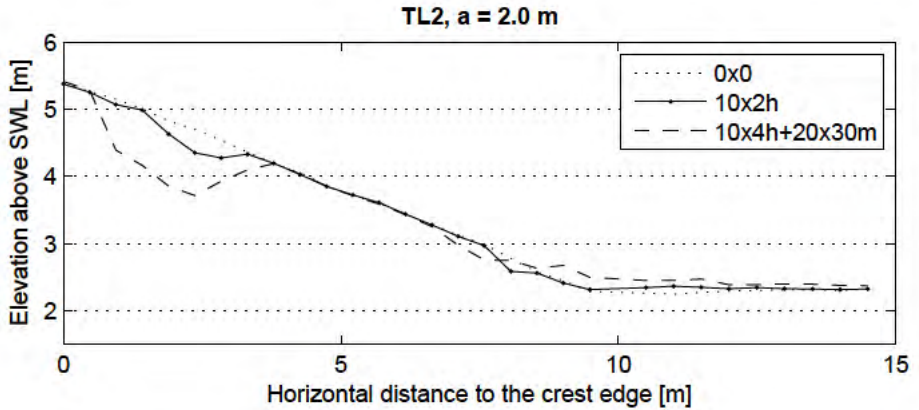


Fig. 9: Changes of the slope profiles measured parallel to the left side wall (facing the simulator gate) with a distance of $a = 2.0$ m at section Tinh Long TL2. 0×0 is the initial profile; $10 \times 4h + 20 \times 30min$ is the profile measured after 4 hours of 10 l/s per m and 30 minutes of 20 l/s per m.



(a) Before testing



(b) After testing

Fig. 10: Grass cover at Tinh Long TL3 section, maximum applied discharge was $q_{max} = 40$ l/s per m.

4.2. Thai Tho Dike

At section TT1, Bermuda grass was dominant while Vetiver clumps were scattered. The first damage took place above the horizontal concrete beam after three 4 hour tests of 20, 40 and 80 l/s per m. The slope failed after 4 hours discharge of 100 l/s per m. The initial and final condition of the grass cover are shown in Figure 12. Figure 13 illustrates the changes of the slope profiles measured along a line parallel to the left side wall at $a = 3.0$ m.

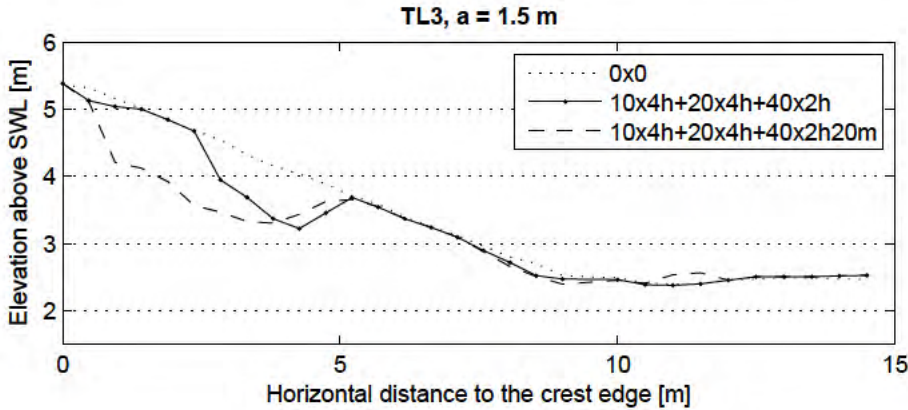


Fig. 11: Changes of the slope profiles measured parallel to the left side wall with a distance of $a = 1.5$ m at section Thinh Long TL3. 0x0 is the initial profile; 10x4h+20x4h+40x2h20min is the profile measured after 4 hours of 10 and 20 l/s per m each, and 2 hours and 30 minutes of 40 l/s per m.

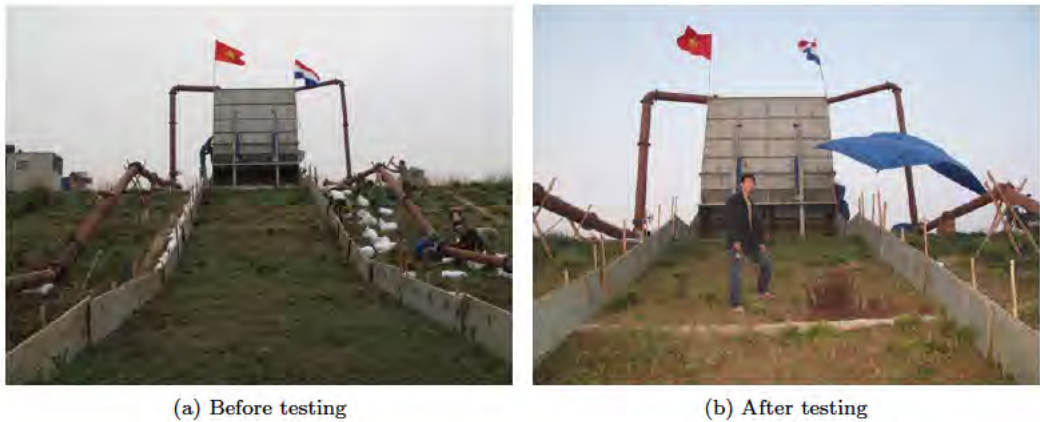


Fig. 12: Grass cover at Thai Tho TT1 section, maximum applied discharge was $q_{max} = 100$ l/s per m.

Section TT2 showed a similar strength to TT1, as it was first damaged after 4 hours discharge of 20, 4 hours of 40 and 4 hours of 80 l/s per m. The slope failure occurred when one hour discharge of 120 l/s per m was applied. As shown in Figure 14, there was a hole under the horizontal concrete beam after testing. Repair to this kind of erosion is technically not easy, because it is difficult to compact soil under the beam. The slope profiles with the deepest hole after testing is presented in Figure 15.

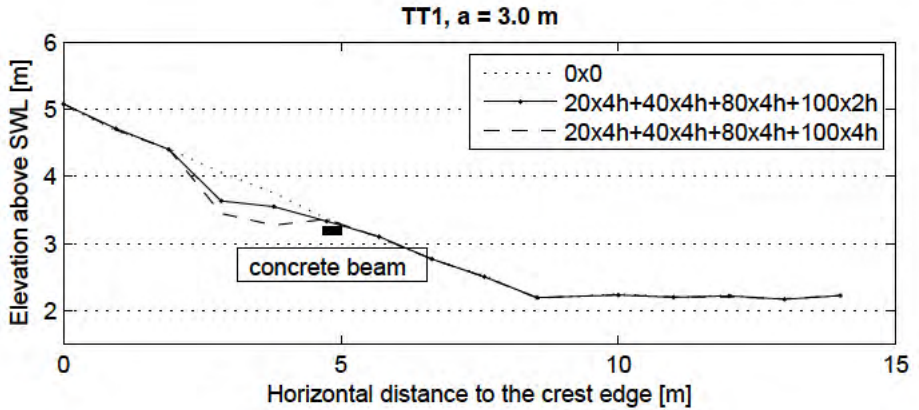


Fig. 13: Changes of the slope profiles measured parallel to the left side wall with a distance of $a = 3.0$ m at section Thai Tho TT1. 0x0 is the initial profile; $20 \times 4h + 40 \times 4h + 80 \times 4h + 100 \times 4h$ is the profile measured after 4 hours of 20, 40, 80 and 100 l/s per m each.



(a) Before testing



(b) After testing

Fig. 14: Grass cover at Thai Tho TT2 section, maximum applied discharge was $q_{max} = 120$ l/s per m.

The third section TT3 showed first erosion after 4 hours of 20 and then 40 l/s per m, and 2 hours of 80 l/s per m. Erosion was observed at various points after 2 more hours discharge of 80 l/s per m. The section failed after 2 more hours testing with 100 l/s per m. Damage extended in the flow direction creating a hole 3 m long by 1 m wide (see Figure 16). Figure 17 compares the slope profiles on both sides of the concrete beam.

There was always a water layer of 10 to 20 cm thick at the TT1 dike toe while

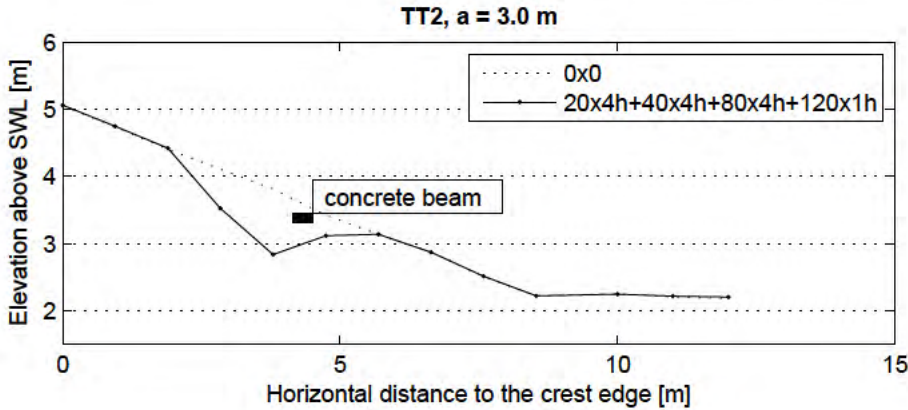


Fig. 15: Changes of the slope profiles measured parallel to the left side wall with a distance of $a = 3.0$ m at section Thai Tho TT2. 0x0 is the initial profile; 20x4h+40x4h+80x4h+120x1h is the profile measured after 4 hours of 20, 40 and 80 l/s per m each and 1 hour of 120 l/s per m.

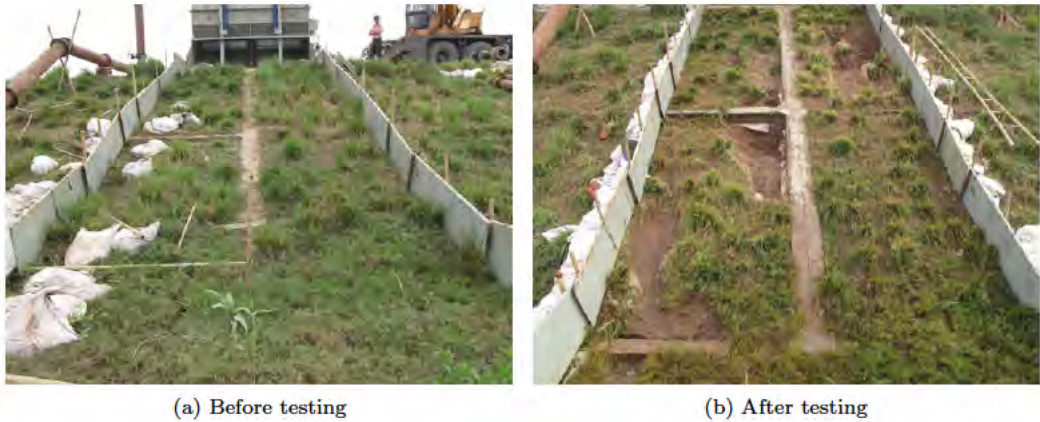


Fig. 16: Grass cover at Thai Tho TT3 section, maximum applied discharge was $q_{max} = 100$ l/s per m.

water was free to flow over the TT2 and TT3 toe. Minor damage were found at all three toes without any considerable difference. Configuration of the concrete frames was similar at TT2 and TT3 with both vertical and horizontal beams, but different from TT1 where there was only the horizontal beam. In general, damage concentrated in the areas around the horizontal beam.

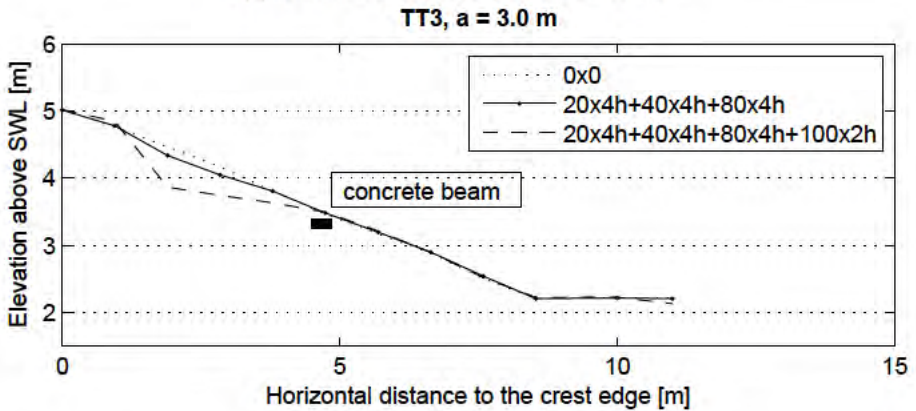
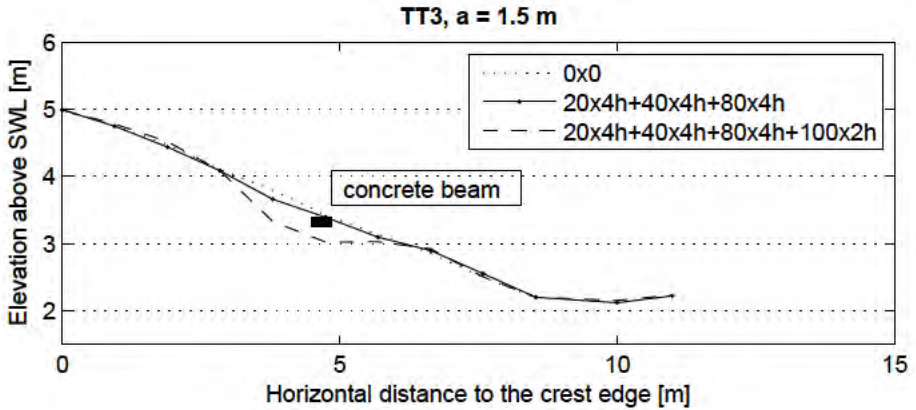


Fig. 17: Changes of the slope profiles measured parallel to the side walls at section Thai Tho TT3. 0×0 is the initial profile; $20 \times 4h + 40 \times 4h + 80 \times 4h + 100 \times 2h$ is the profile measured after 4 hours of 20, 40 and 80 l/s per m each and 2 hour of 100 l/s per m.

4.3. Critical wave overtopping discharge which initiates the failure

Depending on the specifications of each section, erosion did rapidly extend under a certain discharge; before that, little change was observed. The eroded hole at Thinh Long TL1 section was enlarged suddenly during the second hour of 70 l/s per m. An eroded hole gained 60% of its final size during 30 minutes applying 20 l/s per m at TL2 section. Similarly, erosion of TL3 was extended dramatically within the second hour of 40 (l/s per m)-discharge. Discharges less than 80 l/s per m did not erode the slopes at sections Thai Tho TT1, TT2 and TT3. Testing further with 80 l/s per m and higher rates, damage started and then developed rapidly.

Thai Tho dike stretch was homogeneously constructed of clay. Even though

the grass cover was different from section to section, the maximum discharge was relatively consistent, 100 l/s per m at TT1 and TT3, and 120 l/s per m at TT2. On the contrary, tests at Thinh Long revealed that the strength of a grass cover varied considerably along the stretch. The applied discharge was up to 70 l/s per m at TL1, which was constructed of clay with a thickness of 100 cm and covered with four-year-old Bermuda grass. At TL2, the combination between the poor cover of Bermuda and Crab grasses and a 7-cm-diameter Casuarina tree on a sandy clay layer of 80 cm thick could withstand 20 l/s per m. The outer layer, which was 80 to 85 cm thick of clay and protected with a mix of grasses and some small Casuarina trees, failed after more than 2 hours of 40 l/s per m at TL3. At Thinh Long, variation in the applied discharge was up to 3.5 times along a stretch of about 50 m long but the dike itself also differed enormously from section to section. Theoretically, we can presume that a greater degree of uncertainty might be expected for a longer stretch, e.g., some hundreds metres. And the weakest section will fail first, therefore a dike stretch is very much to fail under a lower discharge, e.g., less than 10 l/s per m.

According to the present standards of safety applied in some developed countries, tolerable discharge on a grass slope is in order of litres per second per metre of dike length. For example, the Overtopping Manual suggests a mean discharge varying between 1 and 10 l/s per m in case of no damage to crest and rear face of grass-covered embankment of clay [Pullen, 2007]. Experimental results in Vietnam show that grass covers are able to withstand certain overtopping rates that are considerably higher than the currently recommended values. This finding is comparable to the results in the Netherlands and in Belgium [Steendam, 2010; Van der Meer, 2009]. The simulator tests were limited to only two dike stretches therefore it is not reasonable to conclude or recommend actual tolerable discharges on a grass-covered slope. More tests will evaluate more properly the potential strength of a grass cover in protecting slopes, thus resulting in better guidelines on sea dike design in Vietnam.

5. Process of damage

5.1. *Initiation of damage*

Section TL1 was covered by only one species of grass, Bermuda. A mix of different grass species such as Bermuda and Carpet grass (*Axonopus compressus*) was found at TL2 and TL3. Additionally, there were some Casuarina trees with different trunk diameters at these sections. Sections TT1, TT2 and TT3 were protected with a mix of Bermuda grass and Vetiver grass. Bermuda grass and Carpet grass form a continuous mat covering the slope surface, while Vetiver grass grows in separate clumps scattered across the dike slope.

A grass-covered slope is considered to be damaged when one or some aggregates of soil particles including sward and root are torn out of the slope surface and then moved away by overtopping flows. In other words, damage takes place when the

grass cover is eroded at any point, even with small area and depth. Slope damage mostly started around Casuarina trees (obstacles) or at existing bare spots on slopes covered by Bermuda and Carpet grass such as TL1, TL2 and TL3. The areas among Vetiver clumps were usually eroded first at sections TT1, TT2 and TT3. These three sections were eroded more seriously around the concrete beams. In general, damage often started at vulnerable positions as discussed further in the coming sections.

5.2. *Damage transition from the slope to the horizontal dike toe*

Section Thinh Long TL1 was eroded at the transition between the slope and the horizontal toe under a discharge of 40 l/s per m. At the transition area, Bermuda and Ray grass was poorer than on the higher part of the slope and the lower part of the toe (see Figure 18, right panel). Due to the thin cover of grass, soil was more directly exposed to wave overtopping flows. Furthermore, the load induced by these flows changed direction acting on the toe like water jet. Similarly, damage also took place at the sharp transitions from steep part to berm or toe in previous tests at Afsluitdijk and Vechtdijk in the Netherlands [Steendam, 2010].

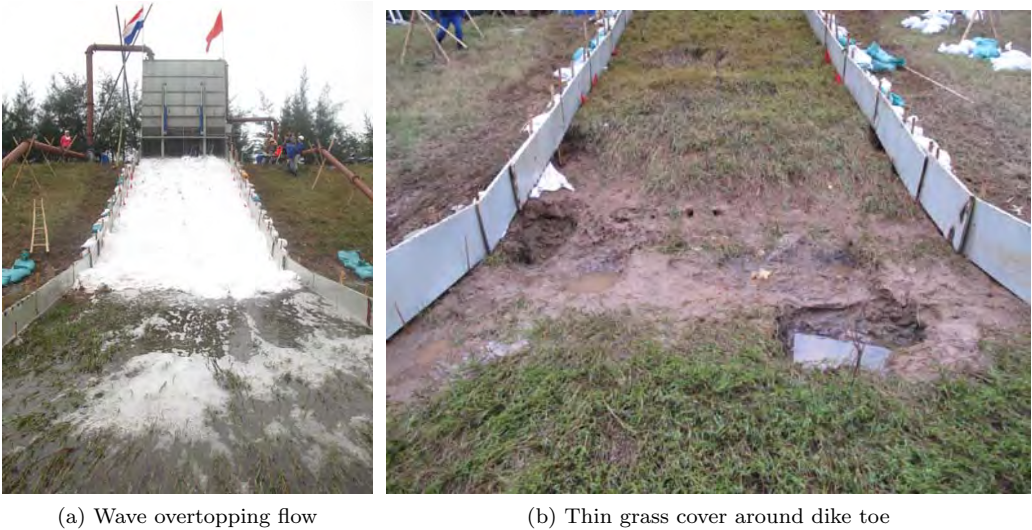


Fig. 18: Damage around the transition between the slope and the toe at TL1.

The damage at TT3 dike toe is shown in Figure 19. When the top layer of grass sod (about 10 cm thick) was eroded, a concrete beam and a thin layer of cement mortar were found preventing grass roots from penetrating deeply into the dike body. The grass sod and the mortar layer were easily eroded to reveal the body of soil underneath. In short, the damage at the transition between slope and horizontal toe was possibly facilitated by the existence of different material layers.

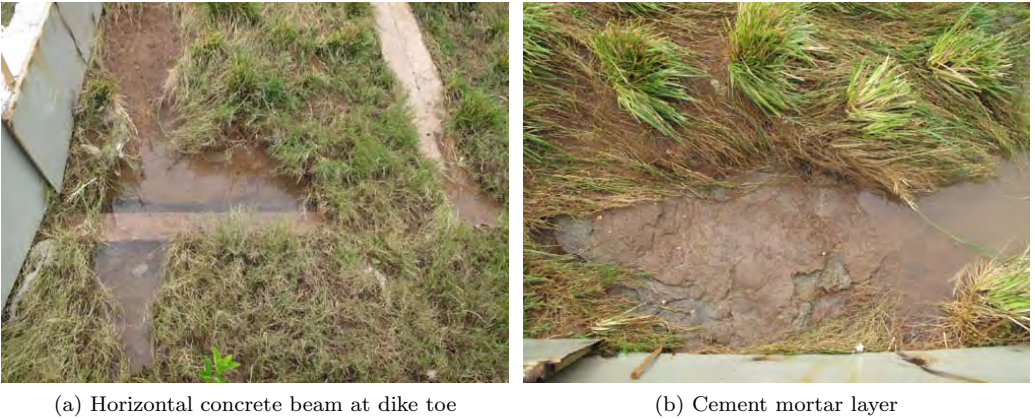


Fig. 19: Damage around the transition between the slope and the toe at TT3.

5.3. *Transition between different materials*

The Thai Tho dike slope was divided into separate cells by a system of concrete beams. The development of Bermuda grass was not in line with Vetiver grass to cover these cell surfaces. As can be seen in Figure 20, Vetiver clumps were shortened to expose a thin cover of Bermuda grass. Vetiver grass overwhelmed Bermuda grass; in addition, there was less grass and soil surface was lower (due to soil settlement) around the concrete beams than in the centre of the cells. Flow might have concentrated among separate clumps due to the high shape of Vetiver, and around the concrete beams due to the uneven transition from soil to concrete. As a result, trenches were formed as depicted in Figures 16 and 20. Using different grass species and concrete frames might cause slope discontinuity, making it vulnerable to wave overtopping flow attack. However, more tests are required to provide sufficient evidence before drawing conclusion about the effects of Vetiver grass and concrete beams on the strength of the dike slopes as well as the combination of Vetiver and Bermuda grass.

5.4. *Existing damage*

It is hardly possible to prevent a grass cover from being damaged naturally or manually. At section TT3, erosion was first recognised at a mouse-hole as shown in Figure 21. Especially, small holes had been dug and then filled again to plant Casuarina trees at Thinh Long dike. These holes were quickly eroded by an overtopping rate of 10 l/s per m (see Figure 21) and then extended by greater values of discharges. Accordingly, damage easily starts or extends from potentially weak points already existing on a grass-covered dike slope.



(a) Thin cover of Bermuda among Vetiver clumps



(b) Erosion of Bermuda among Vetiver clumps

Fig. 20: The grass-covered slope of the Thai Tho dike.



(a) Damage developed from a mouse-hole



(b) Erosion around a Casuarina tree

Fig. 21: Damage starts at eroded holes on grass-covered slopes.

5.5. Obstacles

At section TL2, damage started from an existing hole and then extended toward the 7-cm-trunk Casuarina tree. Overtopping flow eroded and carried away soil aggregates held by roots, gradually reducing the connection between the tree and slope. The anchoring force became weaker and weaker so that the tree collapsed. The amount of surrounding soil associated with this collapse was more considerable than what could be swept away by a grass sod erosion because the system of thicker and longer root threads of the Casuarina tree. Figure 22 gives impressions of the flow and damage around the Casuarina tree at section TL2.



(a) Flow around the Casuarina tree



(b) Damage extended to the Casuarina tree

Fig. 22: Flow and damage around the 7-cm-trunk Casuarina tree, Thinh Long TL2 section.

6. Conclusions and recommendations

The paper described some overtopping tests at two dike stretches in the north of Vietnam. For the first time, the strength of six grass-covered slopes was evaluated *in situ* using the wave overtopping simulator. The simulator is a device that is able to generate wave tongues on a real sea dike.

Experimental results have revealed that a grass cover at least could withstand a discharge in the order of 10 l/s per m in some hours. However, more tests are required to confirm this finding. Further studies, including simulator tests, are recommended to better evaluate the strength of a grass-covered slope. High sea dikes without any wave overtopping (i.e., zero discharge) are hardly achievable in developing countries like Vietnam due to budgetary constraints. Dikes could be lower and cheaper if certain rates of overtopping are allowed corresponding to the strength of grass covers.

The simulator tests also emphasize that damage often takes place at the transitions between slope and dike toe, the transitions between different materials, positions with existing damage, and around obstacles. Bermuda grass, Carpet grass and Ray grass with rhizomes and stolons usually cover a slope surface evenly and continuously, thus resulting in fewer exposed spots. Obstacles like trees, stones and staircases should be avoided because these can interrupt the continuity of a grass cover and partly block the flow or they should be designed/ made overtopping resistant. Besides, a gentle connection between the slope and the toe will apparently create smoother flow. In addition, regular and careful dike monitoring will help to recognise initial damage which needs to be repaired immediately to eliminate further enlargement.

To conclude, tests with the simulator provided new insight into the performance

and strength of a grass-covered dike slope and reveal its vulnerable positions under attack of wave overtopping flows.

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Appendix A. Overtopping parameters

Wave overtopping parameters are calculated using formulas given in the Overtopping Manual EurOtop [Pullen, 2007]. In a storm with duration D , the distribution of overtopping volumes is based on a given value of the mean discharge q associated with particular wave conditions of $H_{m,0}$ and T_p . This distribution will be tabulated to control the simulator. Main steps are explained as follows:

- The freeboard R_c required for the discharge q and wave height H_{m0} is derived from

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan \alpha}} \xi_0 \exp \left(-4.75 \frac{R_c}{H_{m0}} \frac{1}{\xi_0} \right) \quad (\text{A.1})$$

in which, the surf similarity $\xi_0 = \tan \alpha / \sqrt{s_0}$ and the wave steepness $s_0 = (2\pi/g)(H_{m0}/T_{m-1,0}^2)$ with the wave period $T_{m-1,0} = T_p/1.1$.

- The wave run-up levels are assumed to follow a Rayleigh distribution, see also Walton Jr [1992] and Hedges [2004]; its 2% value is

$$\frac{R_{u,2\%}}{H_{m0}} = 1.65 \gamma_b \gamma_f \gamma_\beta \xi_0 \quad (\text{A.2})$$

where γ_b , γ_f and γ_β are correction factors taking into account the influence of berm, permeability and roughness, and oblique wave attack, respectively.

- Overtopping probability P_{ov} associated with the crest freeboard R_c is

$$P_{ov} = \exp \left[- \left(\sqrt{-\ln 0.02} \frac{R_c}{R_{u,2\%}} \right)^2 \right] \quad (\text{A.3})$$

- Therefore, the number of overtopping waves N_{ov} in a storm duration is derived from P_{ov} and the total number of incoming waves N_w

$$P_{ov} = \frac{N_{ov}}{N_w} \quad (\text{A.4})$$

with $N_w = D/T_m$ and $T_m = T_p/1.15$.

- If overtopping volumes is described by a Weibull distribution, the probability P_V that an overtopping volume \underline{V} is smaller than V is

$$P_V = P(\underline{V} \leq V) = 1 - \exp \left[- \left(\frac{V}{a} \right)^{0.75} \right] \quad (\text{A.5})$$

in which, the scale factor is related to wave period, mean discharge and overtopping probability

$$a = 0.84 T_m \frac{q}{P_{ov}} \quad (\text{A.6})$$

- Create a table of N_{ov} values of overtopping volume V , in which the order i of each wave decreases from N_{ov} to 1. Probability that a wave has overtopping volume greater than V is

$$1 - P_V = \frac{i}{N_{ov}} \quad (\text{A.7})$$

- Overtopping volume of a wave with probability of exceedance $(1 - P_V)$ is therefore

$$V = a[-\ln(1 - P_V)]^{4/3} \quad (\text{A.8})$$

Note that the obtained value of V will increase from small to large in the table while i decreases. To simulate the actual behaviour of overtopping, the V -values are sorted randomly. With the given discharge q supplying water into the simulator, time to fill a volume V_i is $\Delta t_i = V_i/q$. The cumulation sum of all Δt_i results in a series of moments to open/ close the butterfly valve generating derised overtopping flows at a sea dike.