CHAPTER 191

EROSION AND OVERTOPPING OF A GRASS DIKE LARGE SCALE MODEL TESTS

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ABSTRACT

A comprehensive test series at full-scale was performed on a grass dike to obtain quantitative information on the wave loading, erosion rate, residual strength and overtopping under design storm conditions. The purpose was to establish design and assessment criteria for this type of dike, which exists in many places in The Netherlands. The particular dike tested exhibited, in general, a considerable resistance to erosion. However, such a non-homogeneous surface is subject to weak spots which can erode very quickly. The most critical region, where the wave loading is the highest, has been identified. The residual strength of the underlying layer of clay is defined as the time it takes for erosion to progress through the entire layer and expose the sand core of the dike. For the dike tested, the residual strength of the 0.8 m thick clay layer is estimated to be 10 hours under design conditions. Measurements of the average overtopping rate and the volumes per overtopping wave were compared to existing formulae, developed from the results of small-scale tests. The comparison was favourable, indicating that scale effects are not substantial for these quantities.

INTRODUCTION

Along many rivers and some sections of sea coast in The Netherlands a grass dike protects the low-lying polder land. Interest in grass dikes is growing, as a result of increasing environmental, economic and aesthetic concerns. In spite of this, no method was available to assess the strength of a grass layer for design or repair

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purposes. In recent years the strength characteristics of a grass dike have been systematically studied in order to answer the following questions: Is the strength mostly determined by the grass itself, the root system or the underlying soil? How does the grass layer behave during long-term wave and current loading and how long does it take before the grass layer can be considered to have failed? What is the residual strength of the dike, ie. how long does it take, after failure of the grass layer, until the sand core of the dike becomes exposed?

In 1992 a large-scale testing programme was created in order to answer these questions. By also measuring the overtopping rate, these tests then also provided the opportunity to compare recently derived overtopping formulae with full scale measurements. The aim of the study was:

- 1. to study the behaviour of the water motion (velocity, pressure distribution, layer thickness and runup) over the grass slope.
- 2. to measure the erosion resistance of the grass layer.
- 3. to measure the average overtopping rate and the overtopping volume per wave, for a wide range of wave and water level conditions.
- 4. to measure the residual strength of the underlying layer of clay.

The project was commissioned by the road and hydraulic engineering division of the Dutch Ministry of Public Works and was performed by DELFT HYDRAULICS, Delft Geotechnics and the Agricultural University of Wageningen.

The tests were performed in de Delta Flume of DELFT HYDRAULICS. This flume is 250 m long, 5 m wide, 7 m deep. Waves can be generated up to 3 m in height. With this facility waves occurring under design storm conditions for the north coast of Holland could therefore be generated.

TESTING SETUP

For this project a section of an existing sea dike was excavated and transported to DELFT HYDRAULICS. This was achieved by cutting a total of 16 blocks out of the grass and clay covering layer out of the dike. Each block measured 2.5 m x 2.5 m x 1 m thick (Photo 1).

The dike section was reconstructed in de Delta Flume and consisted of an outer slope (sea side) of 1:4, a 2 m wide crest at at height of 7 m above the flume bottom and an inner slope (land side) of 1:2.5 (see Figure 1 and Photo 2). The grass section of the dike was located in the centre 2.5 m of the flume, with 1.25 m wide concrete sections on either side. The inner slope consisted of grass from the crest down to an elevation of 3.5 m and below that of concrete to a height of 2.5 m. Behind this slope was the overtopping container, which had a capacity of 17 m³. The grass was placed only in the middle of the flume for three reasons:



Photo 1 Excavation of sections from the dike



Photo 2 Dike in the Delta Flume prior to testing



Figure 1 Cross-section of the dike in the Delta Flume

- 1. Fewer sections were needed to be excavated from the dike and placement in the flume was facilitated due to access from the sides. This also reduced the possibility of damage during placement.
- 2. Instruments could be easily attached to the concrete sections of the slope.
- 3. The concrete section of the slope served as a smooth reference for wave runup measurements.

In order to prevent interaction of the water on the concrete and grass sections of the dike, 0.45 m high wooden partitions were placed in between and sealed water-tight.

Condition of the grass in the flume

Approximately 16 m of the dike was located inside the hall of the facility (only part of the flume is located within a hall). The inside section (the outer slope, from the toe to an elevation of 6 m) received therefore less light than would occur under natural conditions. This, in conjunction with relatively high temperatures (25 to 30° C), caused the condition of the grass to deteriorate. In order to minimize this, twenty special 400 Watt lamps were hung 1 m above the grass slope. The lamps burned about 16 hours per day, following the natural day-night cycle. The grass reacted well to this measure and had grown about 30 cm before preparations for the tests were completed. The Agricultural University of Wageningen performed a detailed inspection of the vegetation and root network before testing commenced. The vegetation cover was in good condition and had a high covering density.

GRASS DIKE EROSION

Instrumentation

A large number of instruments was placed on and in the dike. For measurements of the water motion on the slope 6 electro-magnetic velocity meters (EMV's), 2 runup meters (one on the grass and one on the concrete), 3 water layer thickness meters and 30 pressure transducers were used. In order to measure the pore water pressure response in the clay, Delft Geotechnics inserted 32 pore pressure transducers into the clay layer. These transducers were inserted at various depths below the surface via the sides of the grass/clay blocks. In order to measure the rate of erosion due to wave attack the elevation of a large number of points on the grass surface was regularly surveyed.

The overtopping rate was measured via a water level gauge located in the overtopping container. This container consisted of three separate sections to ensure that only the overtopping water from the grass section was accumulated and measured. The number of overtopping waves was determined from the runup meter, which extended to the crest elevation of the dike.

TESTING PROGRAMME

To achieve the three main aims of the study, three test series were performed: erosion, overtopping and residual strength.

Erosion tests

This series consisted of seven tests. The purpose of these tests was to obtain information about the behaviour of the water motion on the slope before and after breaking and about the erosion resistance of the grass layer itself.

The first five tests were of short duration and were performed with relatively small regular and irregular waves. These tests were intended to provide information about the water motion without damaging the grass layer. For the interpretation of the measurements of the erosion rate, differentiation was made between the average rate and hole development. On average, the erosion process progresses relatively slowly and is more or less evenly distributed over the surface. Hole development occurs after the erosion has locally progressed through the root layer. The sixth and seventh erosion tests provided information over both types of erosion.

The sixth test was performed with irregular waves having $H_S = 1,4$ m and $T_P = 4.7$ s and a water level of 4.8 m (Photo 3). These are the conditions for a design storm along the northern Dutch coast. During the test the waves were interrupted several times for inspections and surveying of the grass surface. After 9 hours of wave attack a hole 0,75 m in diameter and 0,12 m deep was noted 1 m under the still water level (SWL). After 1 more hour of wave attack the hole was measured again and then again after a second hour. The hole had in that time grown to a diameter of 1 m and a depth of 0.15 m (Photo 4).

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Photo 3 Erosion test 6: $H_S = 1.4 \text{ m}$, $T_P = 4.7 \text{ s}$



Photo 4 Erosion test 6: Damage after 11 hours of testing

In order to obtain more information about the average erosion rate, it was decided to continue the test for at least another several hours. To prevent the hole from becoming so large as to affect the other measurements and the subsequent test series, it was repaired and protected from further erosion. After a total of 17 hours of wave loading a second hole had appeared 0.5 m under the SWL. This hole was 0.8 m in diameter and had a depth of 0.11 m. This was the location of the heaviest wave loading, where the most damage was expected to occur. This was the end of the test; the surface was inspected in detail and then repaired and protected for the remaining tests.

The seventh erosion test was performed with small irregular waves ($H_S = 0.75 \text{ m}$; $T_P = 3.4 \text{ s}$) and a lower water level (3.5 m). This lower part of the slope had not been subjected to much wave loading and was considered to still be in the original condition. The purpose of this test was to obtain information over the influence of the wave height on the erosion rate. The test was carried out in the same manner as test number 6, except the inspections and surveys of the surface were made every 4 hours instead of every hour due to the slower erosion rate. After 20 hours of wave loading the test was stopped. No serious erosion had occurred (Photo 5).



Photo 5 Condition after erosion test 7

The results of the erosion measurements of these two tests led to the definition of 3 erosion zones over the slope. The most erosion (both hole development and the highest average erosion rate) was concentrated to a depth of $(0.3H_S)$ to $(0.6H_S)$ under the SWL. This is defined as zone 1 (see Figure 2). Between the SWL and a depth of $0.3H_S$ (zone 2), the average erosion rate was half of that in the first zone.

No hole development occurred in this second zone. In the third zone, above the SWL, practically no erosion was noted.



Figure 2 Defined erosion zones

The results showed clearly that the strength of this particular grass dike lay in the root network which was between 5 and 10 cm thick. It was also noted that the average erosion rate was proportional to about the square of the significant wave height (see Figure 3).



Figure 3 Erosion rate (E) as function of H_S

Based on these measurements a preliminary model has been developed which allows one to estimate the maximum allowable duration of continuous wave loading before a certain depth of erosion is achieved (Meijer and Verheij, 1994). An example is given in Figure 4 wherein lines of 5 mm and 50 mm erosion are shown. Of course, based on only two tests, this model is very limited. However, more tests have recently been performed with other wave heights on a wide range of different vegetation and subsoil characteristics and it is meant to expand the model with the new information.



Figure 4 Loading duration for erosion of d = 5 mm and d = 50 mm

Overtopping tests

In this series twelve tests were performed with irregular waves. The purpose was twofold:

1. to give a visual impression of average overtopping rates from 0.1 to 25 l/s/m, under typical conditions for sea dikes (large freeboard and large waves) as well as for river dikes (low freeboard and small waves). A video presentation of the differing conditions was made. During some of the tests an observer stood on the crest of the dike to experience the force of the overtopping water under the various conditions. This was performed for the practical purpose of setting a criterion for the safe performance of dike inspections and repairs during a storm.

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2. to compare the predictions of overtopping formulae, developed from smallscale tests, with large scale measurements. Formulae for both the average overtopping discharge and the overtopping volumes per wave and maximum overtopping volume were tested.

The observer on the crest noted that for an average overtopping discharge of 25 l/s/m (H_S = 1.8 m, T_P = 4.7 s and freeboard R_C = 1.8 m) the force of most of the overtopping waves (green water, acting mostly on the observer's legs) was not large enough to cause a loss of balance. However, twice in 15 minutes the spray from the waves (acting over the full height of the observer) did have sufficient force to cause him to lose his balance. If it is also considered that during real storm conditions a high wind speed is also present, it must be concluded that performance of inspection or repair work during these conditions is not possible. This conclusion is also extended to conditions leading to an average discharge of 10 l/s/m.

Comparison of measurements to existing formulae

Average overtopping rate:

The most widely applied formula for the average overtopping discharge is (Van der Meer and Janssen, 1994):

$$Q = A \exp(-BR) \tag{1}$$

For this formula the parameters Q, R, A, B are dependent on the breaker parameter as follows:

for $\xi_{op} < 2$ (breaking waves):

$$Q_b = \frac{q}{\sqrt{gH_s^3}} \sqrt{\frac{s_{op}}{\tan \alpha}}; \quad R_b = \frac{R_c}{\gamma_f H_s} \frac{\sqrt{s_{op}}}{\tan \alpha}; \quad A = 0.06; \quad B = 5.2$$
(2a)

for $\xi_{op} > 2$ (non-breaking waves):

$$Q_n = \frac{q}{\sqrt{gH_s^3}}; \quad R_n = \frac{R_c}{\gamma_f H_s}; \quad A = 0.2; \quad B = 2.6$$
 (2b)

with

- Q_b, Q_n = dimensionless overtopping discharge [-] = subscript for breaking waves b
- = subscript for non-breaking waves n [-]

$$A,B$$
 = regression coefficients

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| q | = | average overtopping discharge, per metre length of dike | [m ³ /s/m] |
|-----------------------|---|---------------------------------------------------------|-----------------------|
| g | = | acceleration due to gravity | [m/s/s] |
| R _c | | crest freeboard | [m] |
| H _s | = | significant wave height | [m] |
| R_{b}, R_{n} | = | dimensionless crest freeboard | [-] |
| ξ _{op} | = | breaker parameter = $\tan \alpha / \sqrt{s_{op}}$ | |
| sop | = | wave steepness = $2\pi H_S / (gT_P^2)^r$ | [-] |
| γ_{f} | = | reduction factor for roughness | [-] |

Analysis of the runup measurements for the grass slope indicate that the γ_f value is, on average, between 0.8 and 0.9. It is dependent on the length of the grass, etc.

Figure 5 shows that for the higher overtopping rates the measured data points compare favourably to the predicted values.



Figure 5 Measured and calculated average overtopping discharges

Overtopping rates per wave:

The average overtopping discharge does not give an indication of how much water can be expected to overtop the crest by a particular wave. The maximum volume per wave can be more than 10 times higher than the average discharge. In Van der Meer and Janssen (1994) a formula is presented to calculate the overtopping volumes per wave, based on the probability distribution of the overtopping volumes of individual waves (a Weibull distribution). The volume per wave is based on the

average overtopping discharge. From this approach the maximum overtopping volume from a group of N waves can be predicted with

$$V_{\max} = a [\ln(N_{ov})]^{4/3}$$
(3)

where

The probability of overtopping P_{ov} can be calculated with

$$P_{ov} = \exp\left[-\left(\frac{R_c/H_s}{c}\right)^2\right]$$
(4)

The coefficient c follows from the assumption that the runup distribution follows a Rayleigh distribution. The value of c can be calculated with

$$c = 0.81 \gamma_f \xi_{op}$$
 with a maximum of $c = 1.62 \gamma_f$ (5)

The relation between the maximum volume in one wave and the average overtopping discharge is shown in Figure 6. In this figure the calculated line is based on a wave height of 1 m. The data points are for differing wave conditions. For average discharges of less than 10 l/s/m the wave heights were all approximately 1 m. For the higher rates the wave heights were approximately 1.5 m, which is why the data points fall above the calculated line.

It can therefore be generally concluded that the formulae developed on the basis of small scale tests are representative of prototype conditions.

Residual strength test

The residual strength of a dike is defined as the capacity of the underlayers to withstand wave and current loading after the top protection layer has failed. For a grass dike the top protection layer is the top 5 to 10 cm layer where the grass and root network is concentrated. The underlayer is the approximately 1 m thick layer of clay which covers the sand core of the dike. The residual strength is expressed in terms of the time it takes, after failure of the grass layer (the development of a hole), for the sand core to become visible.



Figure 6 Maximum overtopping volume in a single wave as a function of the average overtopping discharge

This last test was performed after all erosion and overtopping tests were completed. The wave conditions for this test were identical to those for the sixth erosion test (design wave conditions). At the location of the wave impact, the grass and root layer had already been fully eroded away during the first two test series. Therefore, this test was considered to give a measurement of the residual strength of the clay layer alone. At the start of the test the clay layer was about 0.8 m thick.

After 4 hours of wave loading an under water inspection of the slope was carried out. It was noted that a large hole 0.4 m deep had appeared just under the still water line, along the edge of the grass/clay section of the dike. After 1 more hour of loading this hole had expanded to the full depth of the clay layer and the test was stopped. Photo 6 shows the results.

The growth of this hole must most likely be attributed to edge-effect and these results cannot be interpreted as the behaviour of a pure clay surface. This can be considered representative, however, for a dike with a transition from grass to a "hard" cover layer such as asphalt or a block revetment. Based on these measurements, the residual strength would be (conservatively) estimated at 5 hours.

Measurements of other sections of the surface that were not disturbed by this hole can be used to get an indication of the erosion rate of the clay. These measurements showed 0.25 m erosion over the 5 hours of testing or 0.05 m/hr This compares extremely well with the results from erosion tests 6 and 7: the erosion rate, which can be compared to that for hole development, as well as the location of the erosion agreed with the values for zone 1. This value leads to an estimation of the residual strength as being 16 hours. Based on these two results the residual strength of this 0.8 m thick clay layer is estimated to be about 10 hours.



Photo 6 Condtion after residual strength test

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