STABILITY OF PLACED BLOCK REVETMENTS IN THE WAVE RUN-UP ZONE

Mark Klein Breteler[1], Gijsbert Mourik[2] and Yvo Provoost[3]

Placed block revetments are constructed to withstand the wave forces on dikes, especially in regions where rip rap is not locally available, such as the Netherlands. The blocks are placed adjacent to each other on a filter layer to form a relatively closed and smooth surface, which is easy to walk on.

The present research is about the stability of block revetments under wave attack. It focusses on the stability in the run-up zone (above the stil water level, SWL) and compares this with the stability in the wave impact zone (below SWL). To obtain practical design formulae both small-scale and large-scale tests have been analysed, in combination with theoretical considerations based on the leakage length theory. Two types of hydraulic load are considered: a wave front that rushes up the slope (like a bore), and high velocity flow in the run-up and run-down. The later is especially an issue for revetments in which blocks are sticking out among adjacent blocks due to lack of maintenaince.

Keywords: Block revetment, levee, wave attack, stability

INTRODUCTION

Placed block revetments are constructed to withstand the wave forces on dikes, especially in regions where rip rap is not locally available, such as the Netherlands. The blocks are placed adjacent to each other on a filter layer to form a relatively closed and smooth surface, which is easy to walk on.

In the past years extensive research is conducted by Deltares, commissioned by Rijkswaterstaat (Dutch Ministry of Infrastructure and Environment). Recently large-scale model tests were performed to investigate the stability of block revetments in the run-up zone. Above the water line the hydraulic forces are much smaller than below the water line, allowing for a much smaller revetment thickness.

Conflicting scale rules make it impossible to investigate this in a small scale model. The wave motion is governed by the Froude scaling, while the pressure difference across the cover layer is also governed by the Reynolds scale. The pressure difference across the cover layer, for which the flow in the filter is very important, is the primary load that induces instability. This means that the stability can not be investigated on small-scale.

It was, however, possible to use the results of small-scale tests regarding the pressure distribution on the slope, because for this aspect only the Froude scale is important.

Figure 1, Block revetment during construction.

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THEORY OF HYDRAULIC LOAD

The stability of block revetments is especially jeopardized by pressure gradients on the slope. These are largest below the water level (wave impact zone), but not completely absent in the run-up zone. For practical reasons we focus on the pressure potential (piezometric head) ϕ:

$$\phi = \frac{p}{\rho g} + z \qquad (0)$$

with: ϕ = pressure potential (m); p = pressure (Pa), ρ = density of water (kg/m^3), g = gravity acceleration (m/s^2) and z = vertical coordinate (m)

The pressure potential in the filter underneath the cover layer is a damped representation of the pressure distribution on the slope. This is shown in figure 2 for the primary load in the impact zone. We assume the wave impact zone is below stilwaterlevel and the run-up zone is above it.

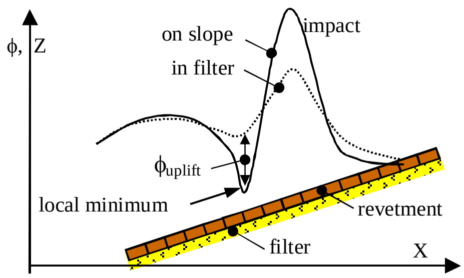

Figure 2, Pressure potential distribution on the slope and in the filter during wave impact (schematised).

A formula for the pressure potential in the filter can be derived on the basis of the mass balance in a small section of the revetment and filter with length Δy, as shown in Figure 3 (Bezuijen et al 1996). The pressure transmission through the filter is governed by the leakage length (Λ):

$$\Lambda = \sqrt{\frac{kDb}{k'}} \qquad (1)$$

where k and k' are the permeability of the filter and top layer respectively (m/s) and b and D the thickness of these layers (m). A long leakage length means quite some damping of the pressures in the filter layer compared to the wave pressure on the slope and therefore potentially high uplift pressures.

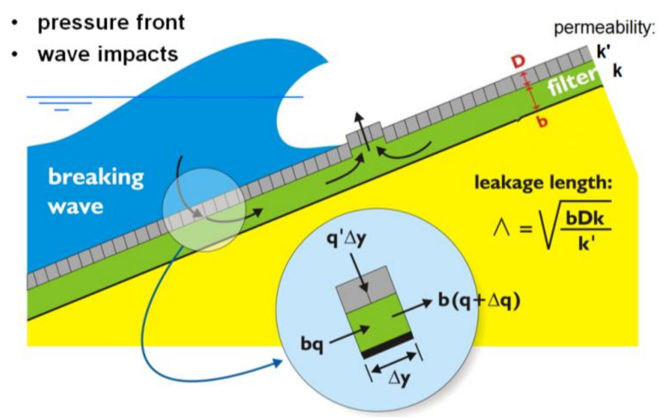

Figure 3, Principle of physical processes leading to uplift pressure (schematised).

The block motion is related to the product of the uplift pressure across the cover layer and the duration of this. This product equals the uplift impuls, which can be held responsible for the block motion. This approach has been checked thoroughly in previous research (e.g. Hofland et al 2005).

This kind of hydraulic load will also be present above the water line, where rare and small wave impacts occur. But there is also a load by the wave run-up and run-down. This will cause two types of hydraulic load (see Figure 4):

1. wave front that rushes up the slope (like a bore)
2. high velocity flow in the run-up and run-down.

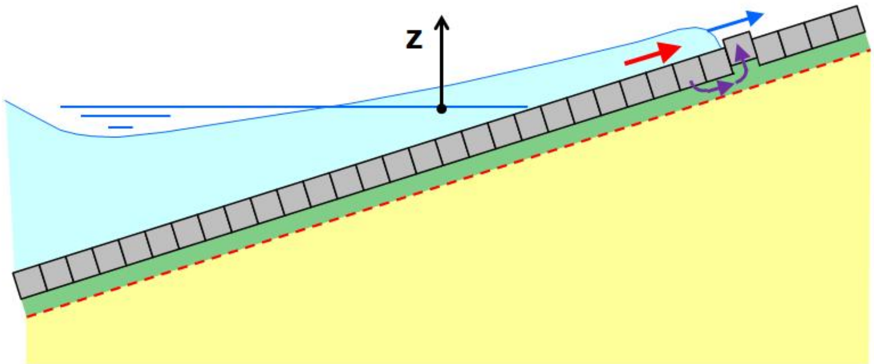

Figure 4, Hydraulic load during wave run-up.

The first hydraulic load can be calculated with the same formulae as derived for the wave impact zone (see section 'Uplift pressure due to run-up wave front').

The second hydraulic load looks not very effective at first sight. During the large-scale tests it appeared that this hydraulic load does not jeopardise the stability of the revetment as long as the revetment is in perfect shape. However, once there are blocks sticking out among the surrounding blocks, the hydraulic forces can be substantial. Although block revetments have a quite smooth surface, lack of maintenance can lead to exposed block edges (see Figure 5). Based on theoretical considerations the relation between flow velocity on the structure and uplift pressures has been derived. This is explained in the next section.

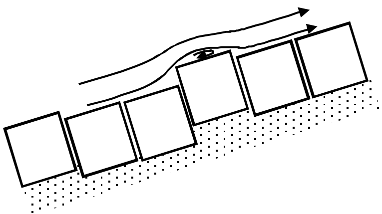

Figure 5, Uneven surface in the run-up zone (vertical cross section).

CONSEQUENCES OF UNEVEN SURFACE

Block revetments usually are very well capable to withstand high velocity flow, but problems can arise if the revetment is not in perfect state. During the years it is possible that the surface becomes uneven because of differences in settlement or other causes, see Figure 4.

The high velocity flow over a block which is not completely flush with the other blocks, will induce significant uplift pressures. This is quantified in this section.

One aspect of the uplift pressure comes from the flow which is blocked by the block, see Figure 6. The high pressure on the upstream side of the block is transmitted to the filter layer, contributing to the uplift pressure. The second aspect, which is usually dominant, comes from the curved streamlines over the block. This leads to a substantial decrease of the pressure on the block, which can be even lower than the atmospheric pressure.

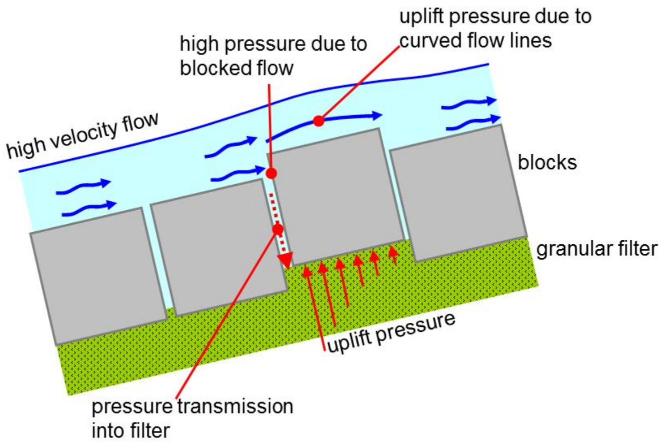

Figure 6. High pressure on upstream side of block is transmitted to filter.

The pressure head on the upstream side of the block (ϕ) is approximately $\phi = u^2/(2g)$, with u = water velocity (m/s) and g = acceleration of gravity (m/s^2). The contribution of this pressure head on the pressure in the filter layer (under the block) has been calculated based on the leakage length theory (based on the principles as explained in Figure 2 and 3).

The decrease of the pressure on top of the block was calculated with the formulas of Chen (1995), which were originally derived for partially buried pipelines. The flow velocity over the structure can be calculated with the formulae of Van der Meer et al (1990).

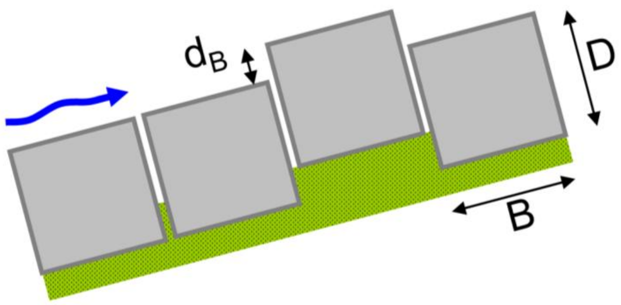

Figure 7. Explanation of parameters.

The derivations have led to the following formulae for assessing the stability of the revetmnet in the run-up zone (Mourik et al 2012):

- If $d_B \leq 0{,}2B$, then stable if: $$0{,}9\left(z_{2\%} - z\right) < \frac{2\Delta DB(\cos\alpha - 0{,}5\sin\alpha)}{4{,}5d_B - 0{,}5D} \qquad (2)$$
- If $d_B > 0{,}2B$, then stable if: $$0{,}9\left(z_{2\%} - z\right) < \frac{2\Delta DB(\cos\alpha - 0{,}5\sin\alpha)}{0{,}9B\left(2 - \dfrac{B}{5d_B}\right) - 0{,}5D} \qquad (3)$$

with $z_{2\%}$ = wave run-up exceeded by 2% of the waves, on slope of infinite height (m); α = steepness of slope (^{o}); d_B = maximum unevenness (distance a block is sticking out of the surface, see Figure 7) (m); B = width of the block (m);

This derivation is comparable to the one for block revetments on low crested breakwaters (Klein Breteler et al 2011).

These formulae are applicable in the range corresponding with the range of performed tests (with minor extrapolation): $2.7 < \cot\alpha < 4.2$; $1.5 < \xi_{op} < 3$; with $\xi_{op} = \tan\alpha/\sqrt{(H_s/L_{op})}$ = breaker parameter for equivalent deepwater wavelength (m); $L_{op} = T_p^2 \cdot g/(2\pi)$ = deepwater wave length (m).

Usually the first case will apply. If that is the case, the formulae can be rewritten to the following practical criterion for sufficient stability:

$$\Delta D > \frac{2\left(z_{2\%}-z\right)\left(d_B-0,11D\right)}{B(\cos\alpha-0,5\sin\alpha)} \tag{4}$$

From this formula it is clear that if $d_B < 0.11D$ there is no additional criterion for the stability of the revetment related to high velocity flow in the run-up zone (because the right term will be negative). This, however, doesn't mean that any revetment will be sufficient. In the next section it is explained that the run-up wave front also leads to uplift pressures.

UPLIFT PRESSURE DUE TO WAVE FRONT IN RUN-UP

During wave run-up the front of the wave rushes up to the slope. In that process each block will suffer for a short time an uplift pressure due to the presence of the wave front. This is a similar process as explained in Figure 2 and 3.

To quantify the uplift pressure we have reanalysed small-scale tests that have been carried out by De Waal (1992). The tests have been performed in the Schelde Flume of Deltares with waves in the range of 10 cm to 20 cm height. At the end of the flume a dike slope was constructed with 5 to 12 pressure cells in the run-up zone. The following geometries of the smooth dike slope were used for the tests:

- plain slope of 1:3 (V:H), without a berm
- 1:3 (V:H) slope with a 40 cm wide berm at 60 cm above the flume bottom. The slope of the berm was 1:15.
- 1:3 (V:H) slope with 100 cm wide berm at 60 cm above the flume bottom. The slope of the berm was 1:15
- 1:4 (V:H) slope with a 40 cm wide berm at 60 cm above the flume bottom. The slope of the berm was 1:15.
- 1:4 (V:H) slope with 100 cm wide berm at 60 cm above the flume bottom. The slope of the berm was 1:15 (Figure 8).

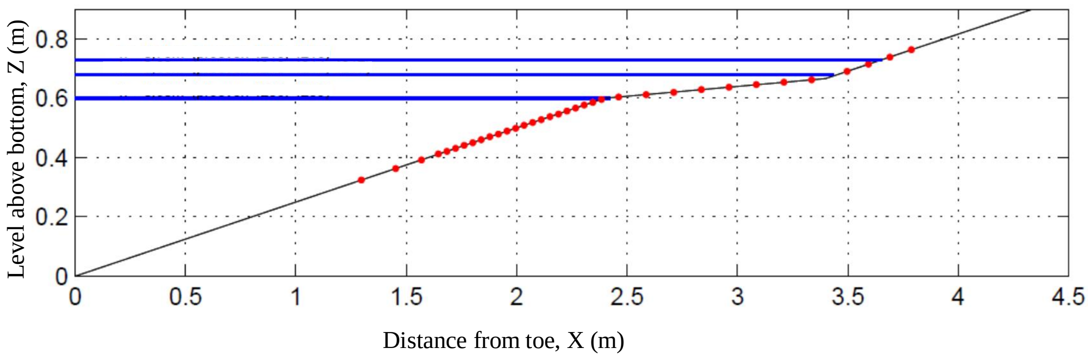

Figure 8. Example of test setup with pressure cells (red dots) on a smooth slope.

In total 80 tests were carried out with each approximately 1000 waves. The significant wave height was varied in the range of $0.10 < H_s < 0.20$ m, with a wave steepness in the range of $0.010 < s_{op} < 0.040$, with $s_{op} = H_s/L_{op}$. The level of the berm, relative to the water level, was varied by changing the water level in the range of $0.52 \leq h \leq 0.73$ m.

Only the pressure on top of the slope could be measured during these small-scale tests. To measure the uplift forces on a block revetment it is necessary to perform large-scale tests. Only on large-scale both the waves and the flow in the filter under the blocks are represented without scale effects.

The enormous amount of data of measured pressures as a function of time and location on the slope is not easy to analyse. In a few steps these data have been schematised in a few representative numbers. The first step was to calculate the uplift pressure for an imaginary block revetment and filter layer combination, within the range of interest of this research. The calculated uplift pressures are translated to a standard wave front, which is supposed to be determining the stability of the revetment in the run-up zone. This is explained in more detail below.

For the present analysis the uplift pressure was calculated with the Deltares wave analysis program 'AnalyseWave'. This program calculates the uplift pressure for a given leakage length with the theory given in Figure 3. The output of the programme is a list of the maximum uplift pressures for each of the thousand waves per test and each location on the slope. From this list the 2% exceedence value was selected for each location on the slope, since it is believed that this value is determining the stability.

The next step in the analysis is to calculate a schematised wave front, as shown in Figure 9, with such a height that the uplift pressure is exactly the same as the measured 2% exceedence value. The front angle was chosen at 75°. This leads to the equivalent pressure front height for a specific location on the slope. With this procedure we were able to schematise the pressure front to a single number per location. By using this later in the design formula, together with the assumed front angle of 75°, a simple and sufficiently accurate result can be obtained. The choice of leakage length and front angle hardly influences the results of this analysis.

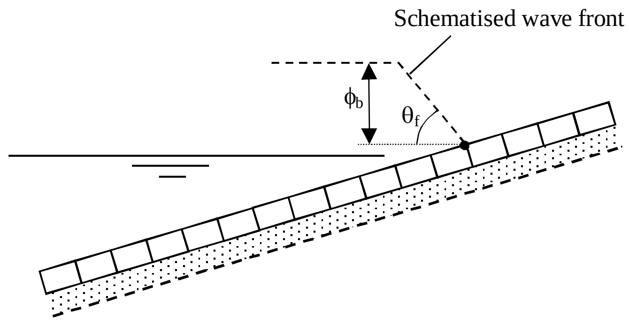

Figure 9. Schematised wave front with such a height that the uplift pressure is the same as the calculated one from the measured wave front.

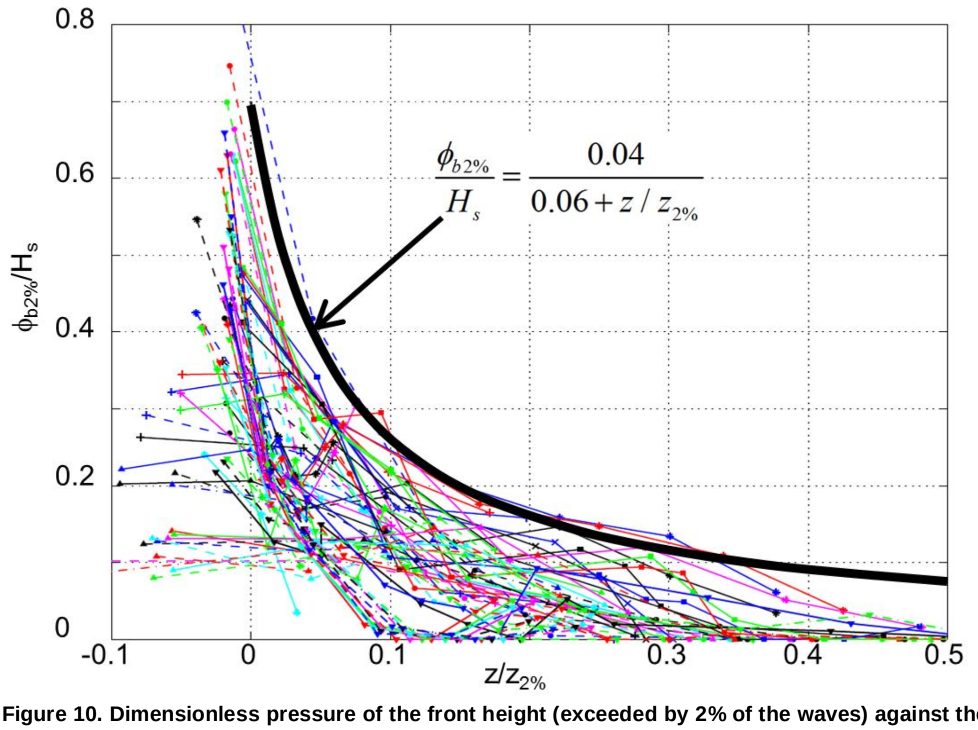

Figure 10. Dimensionless pressure of the front height (exceeded by 2% of the waves) against the location on the slope (height above the water level divided by the 2% wave run-up level).

The result of the analysis is summarised in Figure 10. On the vertical axis the height of the pressure front is given, exceeded by 2% of the waves, which is made dimensionless with the significant wave height. On the horizontal axis the location on the slope is given. This is the height above the water level, which is made dimensionless by dividing it by the wave run-up, exceeded by 2% of the waves.

All measurements, obtained from the tests with berm and without berm, with the 1:3 slope and the 1:4 slope, are plotted together in this figure. Further research might reveal a specific influence of the berm level and berm width, but for now only the upper limit of the measured front height is used to derive a practical formula. The following formula for the upper limit of the measurements is also given in the figure:

$$\frac{\phi_{b2\%}}{H_s} = \frac{0.04}{0.06 + z / z_{2\%}} \qquad (5)$$

With: $\phi_{b2\%}$ = height of the pressure front, exceeded by 2% of the waves (m); H_s = significant wave height at the toe of the dike (m); z = height of the location on the slope, relative to the still water level (m); $z_{2\%}$ = height of the wave run-up above the still water level, exceeded by 2% of the waves (m).

Note that this formula is only applicable in combination with the wave front steepness of 75°. Furthermore, it is assumed that the filter layer is completely filled with water. Higher up the slope it is unlikely that this will be the case at the moment that the run-up front rushes up the slope. If the filter is not filled with water, there will be no uplift pressures possible. This means that the formula is only applicable up to a certain level above the still water level. It is assumed that this formula is applicable for $z < 0.4z_{2\%}$ (safe assumption). At a higher level the hydraulic load due the wave run-up will be negligible.

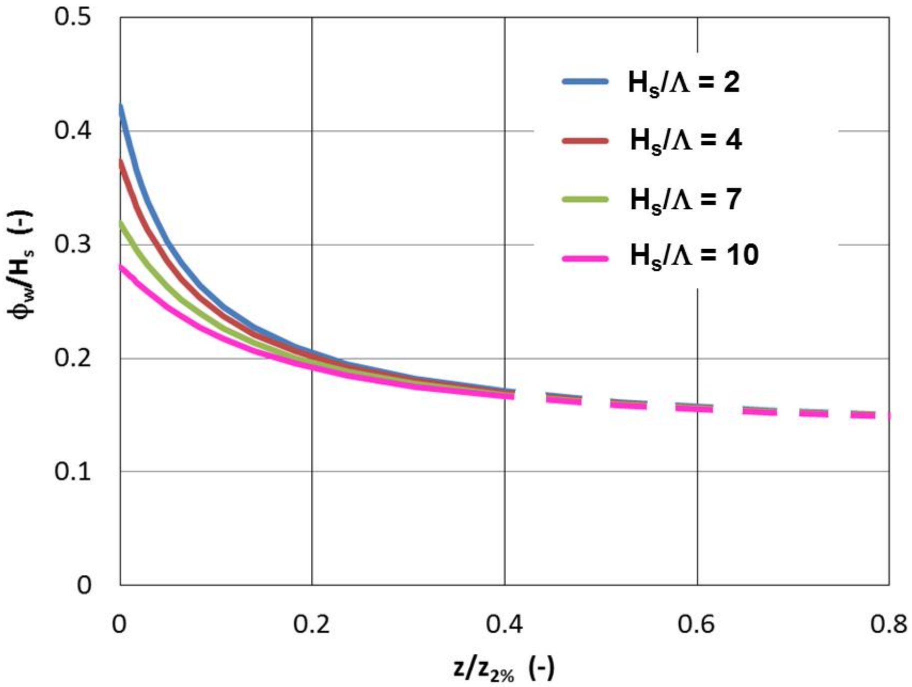

Figure 11. Dimensionless uplift pressure potential of the block revetment (exceeded by 2% of the waves) against the location on the slope (height above the water level divided by the 2% wave run-up).

The uplift pressure due to the wave run-up front can be calculated with the leakage length theory (see Figure 3). The following general formula is applicable for a wave front with shape as in figure 9 (Bezuijen et al 1996):

$$\phi_w = \frac{\Lambda}{2}\left(\cos\alpha \tan\theta_f \left(1 - \exp\left(\frac{-\phi_b}{\Lambda \cos\alpha \tan\theta_f}\right)\right) + \sin\alpha\right) \qquad (6)$$

With: ϕ_w = maximum uplift pressure potential over the block revetment (m); Λ = leakage length (m); α = slope angle (°); θ_f = angle of the pressure potential front (°); ϕ_b = height of the pressure potential front (m).

Combining equation (5) and (6) and substituting θ_f = 75° yields the following formula to calculate the uplift pressure potential as a function of the leakage length, location on the slope, wave conditions and slope angle:

$$\frac{\phi_{w2\%}}{\Lambda} = 1.8\left(1 - \exp\left(\frac{-0.012}{0.06 + z / z_{2\%}} \cdot \frac{H_s}{\Lambda}\right)\right) + \frac{\sin\alpha}{2} \tag{7}$$

This formula is assumed to be valid in the range of $0 < z/z_{2\%} < 0.4$. The formula is shown in Figure 11 for various values of H_s/Λ.

LARGE-SCALE TESTS

The theoretical considerations, and small scale test results has lead to 2 criteria for the stability of the block revetments in the run-up zone. These results were verified with large-scale tests in the Delta flume of Deltares. This 220 m long flume is capable of generating waves of up to H_s = 1.6 m.

Figure 12. Large-scale tests on a block revetment in the Delta Flume of Deltares.

The large-scale tests have been carried out in the Delta Flume to measure the uplift pressures on the revetment and to measure the wave conditions at which instability occurs. The tests were carried out on scale, because otherwise it was not possible to achieve damage to the revetment within the range of possible wave conditions in the Delta Flume. The scale compared to revetments on real dikes was in the range of 1:2 to 1:3. All dimensions given in this paper are the actual dimensions in the flume.

The cross section of the test setup is given in Figure 13. The tested dike had a slope of 1:3 (V:H). A dummy revetment with a high stability was constructed up to +4.80 m above the flume bottom. From there the test section was constructed, which ran up to +5.80 m above the flume bottom. The crest of the dike was at a level of +8.5 m.

The test section in the flume was split up in a left section and a right section with two different revetment types (see Figure 14):

- concrete rectangular blocks (20x20x10 cm^3), placed close together on a 5 cm thick granular filter (D_{f15} = 5 mm).
- concrete columns (∅ 8 cm) with 12% open area between them and a revetment thickness of 8 cm on a 5 cm thick granular filter (D_{f15} = 13 mm).

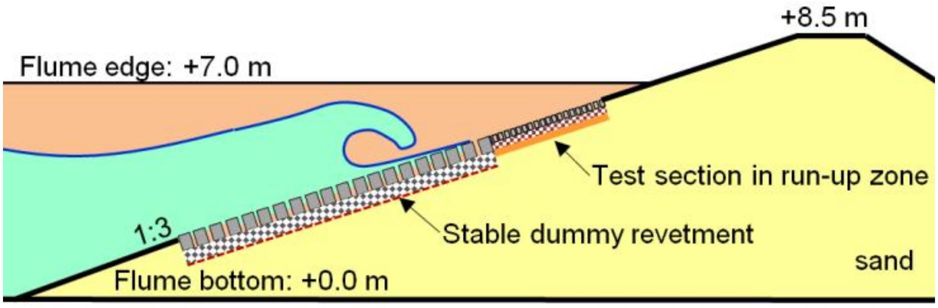

Figure 13, Cross section of test set-up in Delta Flume

Figure 14, The two applied types of revetment (left: columns; right: rectangular blocks)

The columns are a scaled representation of frequently used types of block revetments in the Netherlands, see Figure 15. These were made from concrete palisade piles as commonly sold at garden shops. With a stone saw they were cut at the right length.

The relative open area in the revetment with columns was in the same range as the revetments used on dikes in the Netherlands: 12%.

Figure 15, frequently applied types of block revetments in the Netherlands (left: Basalton; right: Hydroblocks; bottom: rectangular blocks)

The filter layer under the blocks was broken rock gravel with a steep seeve curve. The determining grain size regarding the permeability is the D_{f15}, which is the grain size of which 15% by weight is smaller. Under the rectangular blocks a filter with D_{f15} = 5 mm was applied. Under the columns a coarser filter was necessary, to avoid washing out of the filter: D_{f15} = 13 mm.

The test section was equipped with 8 pressure cells to measure the pressure difference across the revetment during the tests, see Figure 17. For each of the two revetments two blocks were instrumented with a pressure cell on top of the block and one facing downward to the filter. The uplift pressure potential was obtained by calculating the difference between the measured pressure on the block and under the block.

Figure 16, Wave attack on test section in Delta flume

The tests were carried out with three water levels: 6 tests with h = +4.54 m, 5 tests with h = +4.84 m and 3 tests with h = +5.34 m. The wave conditions for these tests ranged from H_s = 0.40 m up to H_s = 1.46 m (all model dimensions), with a wave steepness s_{op} = 0.03. The low wave heights (H_s < 0.8 m) were only used for the highest water level because with that water level there are direct wave impacts on the revetment, leading to a high load and low stabilitity. During the tests with a low water level the hydraulic load only was because of the wave run-up, which is a lower hydraulic load than wave impacts.

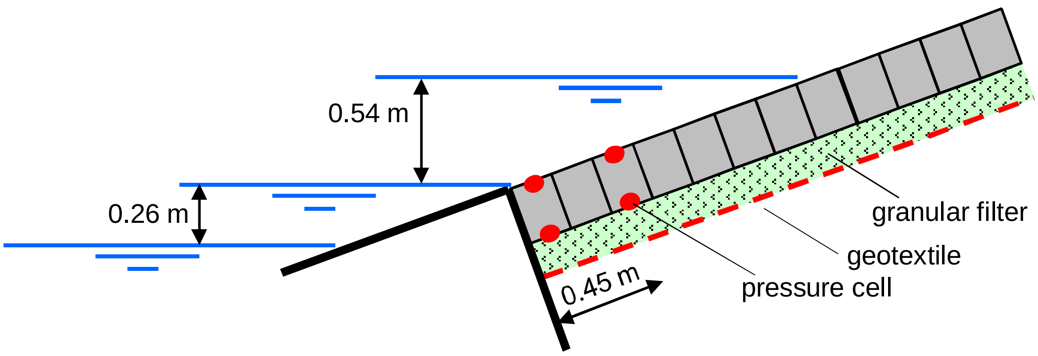

Figure 17, Cross section of the test setup, applied water levels and location of pressure cells.

During the tests it became clear that especially blocks that stick out among the adjacent blocks are very vulnerable.

ANALYSIS OF THE LARGE-SCALE TEST RESULTS

One of the results of the tests was that the uplift pressure decreases if the distance to the water level increases. This is understandable, because of the reducing water velocity and pressure front height.

During the tests with a high water level, when direct wave impacts occured on the test section because it was under the still water level, the uplift pressure was clearly larger at the side of the rectangular blocks then at the columns. This can be explained from the leakage length theory. The leakage length of the blocks was Λ = 0.22 m, while that of the columns was only Λ = 0.12 m. Since a smaller leakage length leads to smaller uplift pressures, this is clearly the reason for the measured differences.

However, the tests with a low water level, for which the test section is in the run-up zone, showed a much smaller difference. Apparently there is also a different physical process occurring: high velocity flow over an uneven revetment. In equation (4) the leakage length is absent, which means that a combination of hydraulic load by high velocity flow and pressure front shows a smaller influence of the leakage length, then the case with only a pressure front.

The recordings in the pressure cells showed the different aspects of the hydraulic load on the blocks that stick out the adjacent blocks. On top of such blocks the curved flow lines lead to a substantially lower pressure. The flow that hits the upstream edge of the block, leads to a very high pressure locally. This high pressure is transmitted through the joints to the underside of the block. These two processes induced a substantial uplift pressure that can cause damage.

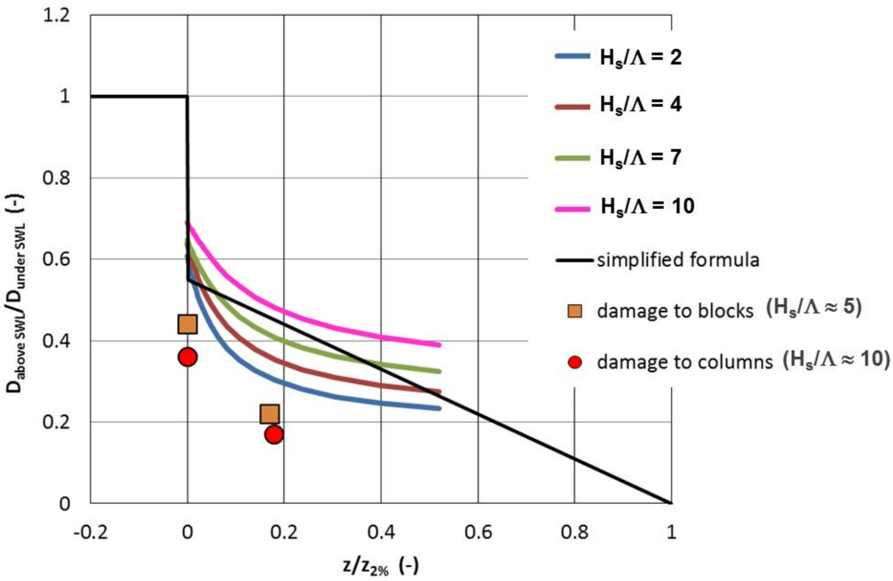

Figure 18, Calculated ratio between the required block thickness underwater and above water

Equation (5) should lead to a safe estimate of the uplift pressure, because we assume a fully saturated filter layer and the upper limit of all measurements have been used to derive the formula, see Figure 10. To verify this with the results of the Delta Flume tests, the equation is used to calculate the required thickness of the block revetment above the water level compared to below the water level. This ratio is approximately equal to the ratio between the uplift pressure under the water level and above the water level. The uplift pressure under the water level has been calculated with the formulae following from the leakage length theory (Bezuijen et al 1996). The uplift pressure above the water level is then calculated with equation (5) and the ratio has been put in Figure 18.

Since we have carried out Delta Flume tests with a high water level (with wave impacts directly on the test section under the still water level) and tests with a low water level (with the test section in the run-up zone), it was possible to estimate the required block thickness above the water level compared to that below the water level. This ratio is approximately the same as the ratio between the wave height at start of damage. In this way also the present test results could be plotted in Figure 18.

From Figure 18 it is concluded that the results of equation (5) give a safe overestimation of the required block thickness compared to the Delta Flume tests.

On the same way as Figure 18 has been made, equation (5) has been translated to a more simple formula for practical applications. This simplified formula is also given in Figure 18 and is as follows:

$$\frac{D_{above\ SWL}}{D_{under\ SWL}} = 0.55 \cdot \left(1 - \frac{z}{z_{2\%}}\right) \quad (8)$$

With: $D_{above\ SWL}$ = required block thickness in the run-up zone (m); $D_{under\ SWL}$ = required block thickness in the wave impact zone below the still water level (m).

CONCLUSIONS

The research has resulted in an easy to use set of formulae to calculate the required revetment thickness in the run-up zone. Furthermore, a more theoretically based calculation method is available, which is much more complicated. This is only recommended in more rare and complicated cases, such as for dikes with a berm.

The calculation methods are well supported by the test results. It shows that the revetment thickness in the run-up zone can be constructed much thinner than in the wave impact zone. But it also shows that the revetment should be well maintained. If blocks are sticking out among adjacent blocks, the revetment should be repaired.

The formulae are included in an easy to use calculation model (Steentoets), which can also be used to design the revetment in the wave impact zone.

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