Run-up Reduction through Vetiver Grass

FINAL REPORT

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PREFACE

This report is my Master thesis at the Delft University of Technology, Faculty of Civil Engineering and Geosciences, Section of Hydraulic Engineering. The study and the tests were performed at the university.

I sincerely hope the study and this report will help creating better and stronger dikes. Then doing this study was not only a pleasant but also a very meaningful period.

I want to thank my thesis committee for being a great help during this study. Without their help and critical review I would not be able to write the report as it is now. I also owe thanks to the people from the Laboratory for Fluid Mechanics. During the tests they were always directly available for answering my questions and helping me.

Delft, 2006
Auke Algera

ABSTRACT

Vetiver grass is used in tropical regions to stabilize soil structures and arable land. Because of its stiff stems and firm roots, flow velocities are reduced and the soil retained. The application of Vetiver grass on the outer slope of a dike is investigated in this report.

The objective is to determine the effect of Vetiver grass hedges on the run-up. Also the effect of different planting configurations on the run-up has to be discussed.

Different research shows that the layer thicknesses and velocities on a location on the outer slope depend on the maximum run-up height and the height of the location. The overtopping volumes over a dike depend mainly on the fictitious run-up height and the crest height.

A dense hedge of Vetiver grass is able to pond water. A relationship can be found between the water depth behind the hedge and the specific discharge through the hedge. About the failure of Vetiver grass hedges little is known, however research shows that a Vetiver grass hedge is able to pond water up to 40 cm.

Tests are conducted on small-scale. Vetiver grass hedges is modelled as vertical plates with vertical slits. The blocking factor is determined by use of the relationship between the water depth and the specific discharge of a Vetiver hedge. A blocking factor of 75% corresponds with a Vetiver hedge. Different plates with different slit widths are used to determine the effect of the width of the slits on the results.

The measures in the tests are chosen so that the dependency of the results on the openings are is negligible. For tests it can be derived from theory that the relative reduction of the run-up height only depends on the blocking factor.

The theory that the reduction of the run-up for the tests is independent of the run-up height could not be rejected. The results of the tests show a constant reduction of the overtopping volumes. The breaker parameter is important for the reduction of the run-up height. It is assumed that the different amount of turbulence in the run-up tip with different breaker parameter, causes the dependency of run-up height on the breaker parameter.

With a blocking factor of 75% a reduction of the run-up volume of more than 55% is measured. A blocking factor of 60% causes a reduction of the volume of 40%.

The flow through the openings in the tests is drag dominant. For larger run-up the flow remains drag dominant. Thus modeling a Vetiver hedge by a plate with larger openings is allowed.

An example with the use of Vetiver grass on a dike in Vietnam is worked out. This example shows that with the use of two Vetiver grass hedges on a dike a reduction of 90 cm of the crest height is feasible. This corresponds with a reduction of 20% of the costs and material use in this example.

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1. INTRODUCTION

In this chapter some background is given on the subject. From this background a problem definition and objectives of this research are extracted. After that the outline is given for research on Vetiver grass and run-up.

1.1 Background

A large part of the world's human population lives along coasts and in river deltas. These people are often protected against flooding by dikes. The most well known failure mechanism of a dike is overflow when the water level exceeds the dikes crest level. An overview of failure mechanisms can be seen in figure 1.1

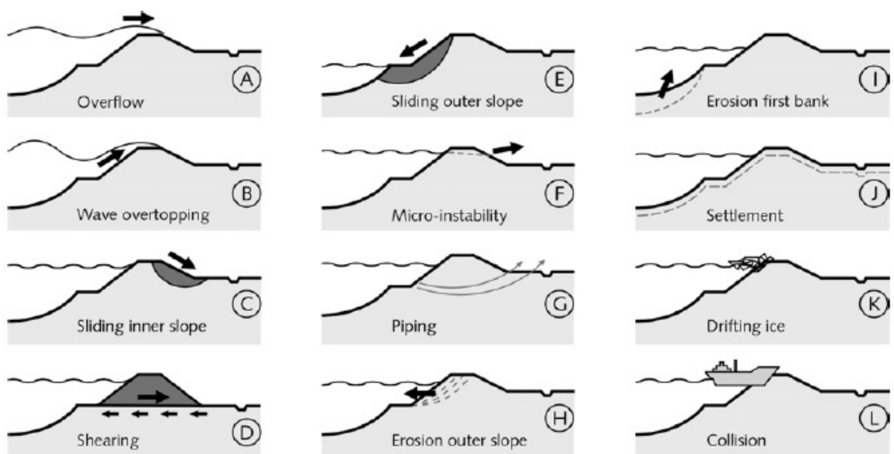


Fig. 1.1: Failure mechanisms of Dikes (TAW,1998)

The height of dikes is often determined by the permitted wave run-up and wave overtopping discharge. Since one of the failure mechanisms of dikes is the erosion of the inner slope, caused by wave overtopping, this should be controlled (mode B in fig. 1.1). In the Netherlands research has been done on overtopping and run-up in order to determine the right crest levels (van der Meer, 2001). The crest level is an important parameter in dike design since it has considerable influence on the materials and space required. However, to prevent the inner slope from erosion also other measures besides raising the dikes crest can be implemented.

Designing a berm or making the slope of the dike less steep can be effective as wave run-up reduction. These measures are quite costly since they require a lot of extra building material and space. Especially in densely populated areas these measures will be very expensive.

Another way to reduce the run-up is increasing the roughness of the outer slope. With a rough outer slope more energy will be dissipated and less water will overtop the dike. An outer layer of rubble mound rocks can reduce the wave run-up by 45 % compared to a smooth asphalt layer (van der Meer, 2002). An armour layer on the inner slope of the dike will increase the permitted wave overtopping discharge since the erosion will be limited. If no rock is available nearby this will be expensive too.

A relatively cheap armour layer can be a vegetative layer. The roots of the vegetation will hold the soil of the dike and the part above the ground will be able to reduce the wave run-up. The reduction of wave run-up by a field of low grass, is small compared to a smooth slope (van der Meer, 2002). A low grass cover on the outer and inner slope of a dike is used to control erosion, since it can create a good closed sod. Higher vegetation like bushes and trees may give more reduction in run-up however, uprooting or falling of a tree or bush during a storm leaves the slope exposed, so the use of trees and bushes is often discouraged.

Tall stiff grasses cannot get uprooted; roots of over a meter tall are common and when loaded by a horizontal force the stems will bend before the roots break. The stiff stems can reduce the wave run-up flow. Another advantage of these grasses is that they may grow fast and any damage can be repaired relatively fast compared to trees and bushes.

1.1.1 The use of Vetiver grass

Vetiver grass hedges have been used in agriculture for centuries in India and South-east Asia. The hedges are used to protect slopes from erosion and retain the soil and water. They are planted along the contour of the slopes and the Vetiver roots reinforce the soil and the leaves and stems slow down the flow and thus allow sediment to settle an example can be seen in figure 1.2. The most used type of Vetiver grass is Vetiver Zizanioides. This is a large type of Vetiver grass which hardly produces any seeds.

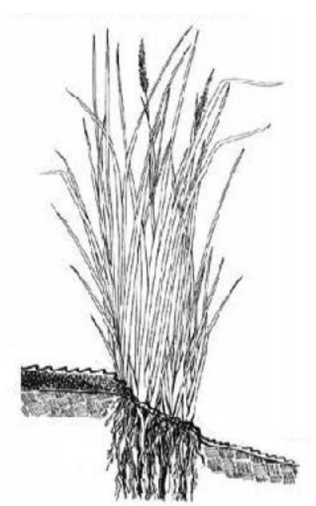


Fig. 1.2: A clump of Vetiver grass ponding water (Maaskant, 2005)

Vetiver grass is not the only stiff stemmed grass; some others are switch grass (*Panicum virgatum L.*), miscanthus (*Miscanthus sinensis*) and tall fescue (*Festuca arundinacae*). Below in table 1.1 some mechanical properties of the different grasses are described.

Table 1.1 Stem density, stem diameter, moment of inertia, modulus of elasticity, MEI product (Dunn, 1996)

Grass	M (m^{-2})	d (mm)	I (mm^4)	E (Gpa)	MEI (N)
Switchgrass old	3700	3.15	4.8	8.5	152
Switchgrass young	7400	3.85	10.8	2.9	231
Vetiver	3500	9.10	337	2.6	3060
Miscanthus	10400	2.25	1.3	3.5	46
Fescue	6870	1.75	0.2	0.2	1

Vetiver has some advantages compared to other tall grasses. The stems of Vetiver have a larger diameter than other types and so a complete clump is stiffer than the others (MEI), as can be seen in table 1.1. Vetiver also has a vigorous root system. A biological advantage of this type of Vetiver grass is that it is not invasive, which means that when you plant it, it will not become a weed because it is sterile. Vetiver is fast growing compared to the other grasses, under ideal circumstances Vetiver stems can grow over 1 cm per day (Maaskant, 2005). Vetiver grows under a wide variety of site conditions. Vetiver is hardly eaten by cattle so grazing on an area will not affect the hedges. The grass cannot stand severe frost so its use is limited to tropical and sub-tropical regions. Vetiver is currently used in many countries of the world for the retention of soils in agricultural areas and the stabilization of soil structures (see figure 1.3)

Fig. 1.3: Use of vetiver in the world (www.vetiver.org)

Vetiver is for instance currently used for bank stabilization in Vietnam (figure 1.4), to protect roadsides from erosion in China and stabilize hill slopes in South America. Since it has a lot of engineering applications some research has already been done on the characteristics of the stems and the roots of the grass. Because of its mechanical and biological characteristics and the fact that knowledge about maintenance and planting of Vetiver is available in a lot of countries, Vetiver grass is the prime candidate for the reduction of run-up.

Figure 1.4: Use of Vetiver grass for bank stabilisation

1.2 Problem Definition

Overtopping of waves may cause erosion of the inner slope and therefore weaken the dike. Vetiver hedges on the outer slope may decrease the run-up that is causing the overtopping. Using Vetiver may reduce material costs, construction costs and the required space to build a dike. Further research is necessary, before Vetiver grass hedges can be used to decrease the wave overtopping. The hedges on the dikes may protect large areas against flooding and therefore have to be reliable during storm events. Research has to be done in the field of performance, maintenance and construction before Vetiver can be relied on. One of the most important things that has to be investigated is the amount of reduction of wave run-up due to Vetiver grass hedges. The wave run-up levels can be used to determine the overtopping.

1.3 Objective of this research

The main objective of this research is:

"Determine the effect of Vetiver grass hedges on wave run-up"

A relation has to be found between the wave height, hedge characteristics and run-up level. The following questions related to this objective will be answered:

- What is the hydraulic resistance of Vetiver grass hedges?
- What is the effect of different planting configurations on the reduction of the wave run-up?

Related problems like failure of the grass due to overloading will be briefly mentioned in this report.

1.4 Outline

In this report first attention is paid to what is already known about Vetiver grass and run-up. This is described in chapter 2 "State of the art". In this chapter finally conclusions are drawn, about what subjects need further investigation. In chapter 3 "Small scale tests" tests are described to test the hydraulic resistance of Vetiver grass in wave run-up. These tests are conducted and the results are presented in chapter 4 "Results". The conclusions from chapter 4 are used to work out an example of real Vetiver on a dike. The effect of Vetiver on the construction costs and material use can be seen in chapter 5 "Vetiver on a dike". Conclusions and recommendations for further research are finishing this report in chapter 6.

2 STATE OF THE ART

In this chapter characteristics of wave run-up and Vetiver grass are discussed as far as they are useful for determining the reduction of the run-up. From the known characteristics of the wave run-up and Vetiver conclusions will be drawn on what needs to be tested furthermore.

2.1 Wave Run-up

A breaking wave on a slope of the dike causes a layer of water to run-up on the dike. For calculations on the reduction of the wave run-up the height, the layer thicknesses and velocities have to be known for a smooth slope.

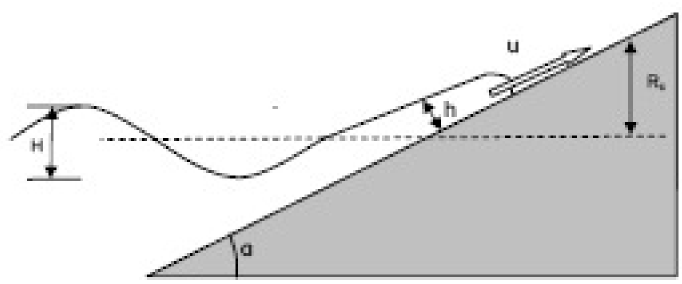


Figure 2.1 Definition sketch for wave run-up

2.1.1 Wave run-up height

The wave run-up height has been a subject to a lot of research, since it is an important parameter for the design of dikes and breakwaters. For the calculation of wave run-up heights of regular breaking waves on smooth slopes research has been done by Hunt and Schüttrumpf.

Regular waves

Hunt's Formula

Hunt's formula gives for regular breaking waves ($\xi \leq 2,5$):

Equation 2.1

$$R_u = \xi H$$

$$\xi = \frac{\tan\alpha}{\sqrt{\frac{H}{L_0}}}$$

The run-up is maximum at $\xi \approx 3$ just at the transition between plunging and collapsing.

Schüttrumpf, 2001

Schüttrumpf proposes the following formula for breaking and non-breaking regular waves:

Equation 2.2

$$R_u = H \cdot c_1 \cdot \tanh(c_1^* \cdot \xi)$$
$$c_1 = 2,25$$
$$c_1^* = 0,5$$

The advantage of this formula is the smooth transition between breaking and non-breaking waves in one formula.

Random waves

Van der Meer, 2001

For wave spectra van der Meer gives for waves on a dike or breakwater the following formula. It is used for the design of dikes in the Netherlands:

Equation 2.3

$$R_{u2\%} / H_{m0} = 1,65 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_0$$

With a maximum for larger ξ_0

$$R_{u2\%} / H_{m0} = \gamma_f \cdot \gamma_\beta \cdot (4,0 - 1,5 / \sqrt{\xi_0})$$

Where:

$R_{u2\%}$	=	2% wave run-up level above the still water line	(m)
H_{m0}	=	wave height $H_{m0} = 4 \cdot \sqrt{m_0}$	(m)
γ_b	=	the influence of a berm	(-)
γ_f	=	the influence of the roughness of the slope	(-)
γ_β	=	the influence of oblique wave attack	(-)
ξ_0	=	the surf similarity parameter $\xi_0 = \tan\alpha / \sqrt{s_0}$	(-)
s_0	=	wave steepness $s_0 = 2 \cdot \pi \cdot H_{m0} / (g T_{m-1,0}{}^2)$	(-)
$T_{m-1,0}$	=	spectral wave period $T_{m-1,0} = m_{-1} / m_0$	(s)
m_0	=	zero moment of spectrum	(m^2)
m_{-1}	=	first negative moment of spectrum	(m^2s)

This formula represents the average wave run-up level and is different from the design rule. This formula includes the influences of a berm and the roughness of the slope. The reduction of the Vetiver grass on the wave run-up could to be implemented in one of these factors. The wave spectrum that has to be used to calculate the wave run-up is the wave spectrum at the toe of the dike.

Van Gent, 2002

Van Gent proposes the following formula for the wave run-up:

Equation 2.4

$$R_{u2\%}/H_{m0} = c_2 \cdot \xi_0 \qquad \xi_0 \le p$$
$$R_{u2\%}/H_{m0} = c_3 - \frac{c_4}{\xi_0} \qquad \xi_0 \ge p$$
$$c_2 = 1.35 \qquad c_3 = 4.0$$
$$c_4 = 0.25 \cdot \frac{{c_3}^2}{c_2} \qquad p = 0.5 \cdot \frac{c_3}{c_2}$$

The advantage of this formula is that the transition of breaking to non-breaking is fluent, still two equations are used.

Schüttrumpf, 2001
Schüttrumpf uses one equation to describe the relationship between the run-up height and the ξ_d for wave spectra. This is another ξ than used by van Gent and van der Meer. To calculate L_0 Schüttrumpf used the average period instead of the $T_{m-1,0}$. ξ_0 is about 5% smaller than ξ_d in general.

Equation 2.5

$$R_{u2\%} = H_s \cdot c_1 \cdot \tanh(c_1^* \cdot \xi_d)$$
$$c_1 = 3,0$$
$$c_1^* = 0,65$$

2.1.2 Layer Thicknesses and Velocities of Wave run-up

Research done on flow on dike slopes was done to determine the wave overtopping rates. No significant difference in the layer thickness and run-up velocity between model tests with and without wave overtopping could be found (Schüttrumpf, 2001). Most investigations on velocities and layer thicknesses find the following relationships between run-up and thickness or velocities.

Equation 2.6

$$u = c_u \cdot \sqrt{g \cdot (R_u - z)}$$

Equation 2.7

$$h = c_h \cdot \left(R_u - z\right)$$

In these equations z is the height on the slope. and c_h and c_u are constants

Regular Waves
For regular waves Schüttrumpf, 2001 finds c_h = 0.284. This author conducted tests with slope angles of 1:3, 1:4 and 1:6. According to Schüttrumpf, 2001 this value is in good accordance with observations by Tautenhain (c_h = 0.246). For c_u a value of 0.94 is proposed. R_u was calculated by using equation 2.2

Random Waves
The following formulae were proposed by Schüttrumpf and van Gent, 2003 for the layer thicknesses and velocities for wave spectra:

Equation 2.8

$$\frac{u_{2\%}}{\sqrt{g \cdot H_s}} = c_{u2\%} \cdot \sqrt{\frac{R_{u2\%} - z}{H_s}}$$

Equation 2.9

$$\frac{h_{2\%}}{H_s} = c_{h2\%} \cdot \left(\frac{R_{u2\%} - z}{H_s} \right)$$

Equation 2.10

$$q_{2\%} = c_{q2\%} \cdot \sqrt{g} \cdot \left(R_{u2\%} - z \right)^{1.5}$$

Where:

$u_{2\%}$ = wave run-up velocity exceeded by 2% of the incoming waves
H_s = significant wave height
$c_{u2\%}$ = empirical coefficient
$h_{2\%}$ = layer thickness exceeded by 2% of the incoming waves
$c_{h2\%}$ = empirical coefficient
$q_{2\%}$ = wave run-up discharge exceeded by 2% of the incoming waves
$c_{q2\%}$ = empirical coefficient

Schüttrumpf (2002) and van Gent (2002) independently found different values for the empirical coefficients:

Table 2.1 Coefficients for run-up velocities, layer thicknesses and discharge

Coefficient	Schüttrumpf (2002)	Van Gent (2002)
$c_{u2\%}$	1.37	1.30
$c_{h2\%}$	0.33	0.15
$c_{q2\%}$	-	0.2

In Van Gent, 2002 the formulae presented are compared to data from other small-scale tests. In these scale model tests only measurements were done at the seaward site of the crest. In the same paper data from other tests is used to compare with the formulae of van Gent. Data from these tests give a better fit with $c_{h2\%}$ = 0.21. Schüttrumpf found this value for $c_{h2\%}$ in prototype tests with an outer slope of 1:6. In scale tests Schüttrumpf, 2001 finds a value of 0.216 for $c_{h2\%}$. Data and formulae from other research are hard to compare since different formulae to calculate $R_{u2\%}$ are used. The value of $c_{q2\%}$ was found by van Gent by multiplying $c_{h2\%}$ by $c_{u2\%}$.

The equations can be rewritten as:

$$u_{2\%} = c_u \cdot \sqrt{g \cdot (R_{u2\%} - z)}$$

$$h_{2\%} = c_h \cdot \left(R_{u2\%} - z \right)$$

$$q_{2\%} = c_{q2\%} \cdot \left(R_{u2\%} - z \right)^{1.5}$$

$u_{2\%}$, $c_{2\%,}$ $q_{2\%}$ are dependent on $R_{u2\%}$ and therefore the values for regular waves and wave spectra can be compared. It may not be necessarily true that for every wave in a wave spectrum that has a run-up height of $R_{u2\%}$ also the velocities and layer thicknesses measured are of the value $u_{2\%}$ and $h_{2\%.}$ However, these values are strongly correlated. Therefore the values of $c_{h2\%}$ and $c_{u2\%}$ can be compared to c_h and c_u.

2.1.3 Summary and Evaluation

For the run-up levels the following graphs compare the different researches about this subject.

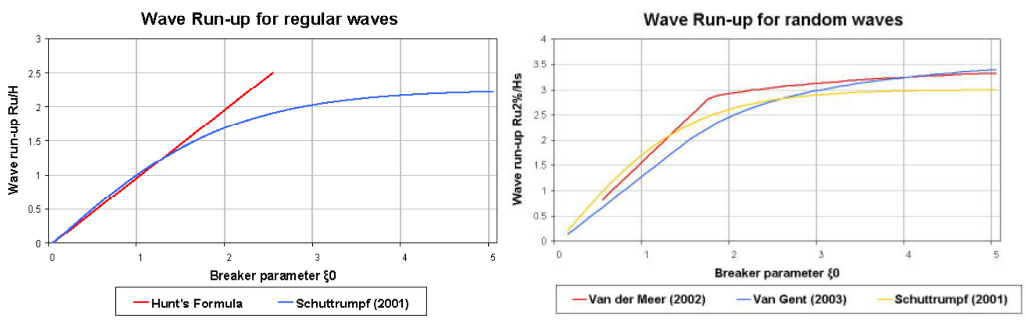


Figure 2.2 Wave run-up levels according to different research

Differences in the results can be explained by different scales (see chapter 3 and 4), different measurement techniques and the fact that these highly turbulent flows are difficult to measure. Schüttrumpf, 2001 finds a slightly lower value for the run-up levels than the other studies for regular waves. Also lower velocities and quite large layer thicknesses were found relative to other research. Since Schüttrumpf, 2001 is the only study found with systematic measurements of run-up levels, velocities and layer thicknesses for regular waves, these results are used for further calculation.

2.2 Characteristics of Vetiver Hedges

In this section the mechanical and hydraulic characteristics of Vetiver grass will be discussed. In order to get a good view on how Vetiver grass may reduce the run-up and handles with the loads. A lot of research has been done to derive the resistance of flow through vegetated channels (Ben Chie Yen, 2002). However, most of the research has been done for submerged vegetation and with open channel flow. The research is focused on finding resistance factors in order to find the resistance for the whole lining. A Vetiver hedge on the other hand is a discrete object in flow.

2.2.1 General Characteristics of Vetiver Grass

Vetiver grass is a stiff stemmed grass. Dunn, 1996 investigated the mechanical properties of three types of stems (vegetative stems, green internodes, dry internodes) of the grass and came up with the values below:

Table 2.2 Mechanical characteristics of Vetiver grass (after Dunn, 1996)

	Veg. stems	Gr. internodes	Dry internodes
Largest Diameter	4.5 mm	8.6 mm	6.2 mm
Smallest Diameter	1.6 mm	6.1 mm	4.9 mm
Mom. of Inertia (Major Axis)	8.2 mm^4	272 mm^4	83 mm^4
Mom. of Inertia (Minor Axis)	1.0 mm^4	120 mm^4	45 mm^4
Modulus of Elasticity E	0.21 GPa	2.6 GPa	4.7 GPa
Bending Angle Yield Point φ	5.1 º	2.4 º	5.6 º
Yield Strength Y	3.7 MPa	7.3 MPa	20.2 MPa

The different stems of the plant grow very dense. Meyer et al., 1995 report 3500 stems per m^2. He reported a diameter of 9.1 mm which is combined stem and leaves. Rough calculations show that 22.7% of the total surface is taken by stems.

2.2.2 Flow Through Dense Planted Hedges

Some tests have been performed with dense planted hedges of Vetiver. The small slips of Vetiver were planted less than 15 cm apart. The result after some time is a very dense hedge which is able to pond water. Flow through a discrete Vetiver grass hedge is drawn in figure 2.3

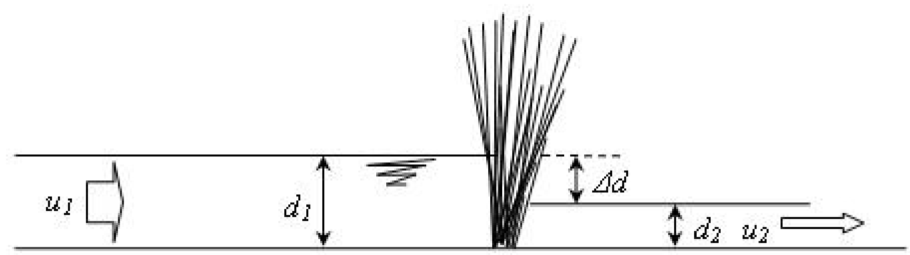


Figure 2.3 Flow through a discrete Vetiver hedges

Dabney, 2003

In order to find a relationship between the discharge and the backwater depth, in Metcalfe (2003) and Dabney (2003) it is tried to determine a Manning's 'n' for the resistance of a barrier of Vetiver grass. Manning's equation is commonly used for determining the resistance for water flowing through vegetation lined channels:

Equation 2.11

$$u = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot \sqrt{i}$$

Where:

V	=	the velocity in this case u_1
n	=	a hydraulic resistance parameter
R	=	the hydraulic radius
i	=	the slope of the bottom

For the hydraulic radius of wide flows, the backwater depths are used. This however, is not useful in this case since the backwater depth is independent of the slope (Dabney, 1996). The 'n' depends on the specific discharge. However, the Manning's 'n' values may serve as indicative values for the backwater depth, only for the slopes tested, which are slopes in the range of 0.03-0.07.

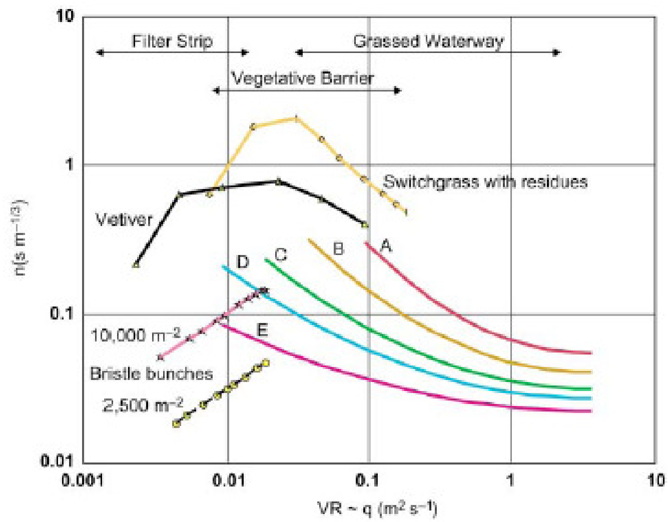


Figure 2.4 'n'-values for different vegetation versus specific discharge (Dabney, 2003)

Metcalfe, 2003

In Metcalfe (2003) 'n' values of about 0.6 $s/m^{1/3}$ were found for hedges of 2 years old with a maximum discharge of 0.06 m^2/s. Also higher values for lower discharges were found which compare good with the values in Dabney, 2003.

Dalton, 1996

In Dalton, 1996 the following formula is derived after Smith, 1982:

Equation 2.12

$$q = x \cdot z_1^{\ a} \cdot \Delta z^b$$

The coefficients ξ, a and b were determined by the use of linear regression analysis. The tests were done on seedlings planted in one row with a space of 125 mm with hedges of different ages. For a hedge of 2 years old the following constants were found:

$$x = 0.66$$
$$a = 1.78$$
$$b = 0.62$$

For all the heights in meters and q is in m^2/s

Dabney, 1996

Dabney (1996) only provides a formula without test results or theoretical considerations.

$$\Delta d = 0.000341 \cdot Re^{1.07} \cdot Veg^{0.17} \cdot Leaf^{0,47} \qquad Re < 11,700$$
$$\Delta d = 0.0762 \cdot Re^{0.49} \cdot Veg^{0.17} \cdot Leaf^{0,47} \qquad Re > 11,700$$

Equation 2.14

In which

Re = the Reynolds Number q/ν		(-)
$Veg = Diameter * M * Width$		(-)
$Diameter$ = The diameter of the stem measured at 5 cm height		(cm)
M = Number of stems per cm^2 at 5 cm height		(cm^{-2})
$Width$ = The width of the hedges		(cm)
$Leaf$ = A dimensionless number related to the number of leaves		(-)

For the larger Reynolds number this formula can be rewritten as:

$$q = \frac{\nu}{0.0762 \cdot Veg^{0.35} \cdot Leaf^{0.96}} \cdot \Delta d^{2.04}$$

In this formula apparently no difference is made between the difference in water level and the water level upstream.

2.2.3 Failure of Vetiver through flow

No specific research has been done about the failure of whole hedges. The failure occurs because of the overloading of a single stem. When stems are loaded, they first bend through an elastic range and then an inelastic range. After that the stems will fail and the stems may break or bend and develop a hinge point (Dunn, 1996 after Rehkugler and Buchele). Because of interactions between the stems in a hedge a single plant failure is hard to calculate from the mechanical properties of a single stem.

Dabney, 1996 report backwater depths up to 0.4 m for Vetiver grass hedges of 0.3 m wide. Meyer, 1995 reports a backwater depth of 0.42 m. Dabney, 1996 also argued that the failure of grass hedges depend on the density of the hedge. A dense hedge will fail with lower discharge because of the higher backwater depth. Sediment and residues will weaken a hedge even more because of filling up the hedge, while not adding to the strength. In Meyer, 1995 tests were performed to determine the sediment trapping capacity for several types of grass hedges. Those tests were carried out in a small indoor flume and the influence of placing the grass in the flume is unknown. It was found that the sediment increased the backwater depth, because the sediment trapped blocked the openings in the hedges. In Temple, 2001 tests with switch grass and water with flowing residues were performed. The result was a weaker hedge because the residues blocked the flow and a higher backwater with lower discharge was the result. With wave run-up however this will not be an issue because of the oscillatory flow.

For single plants of Vetiver grass Chengchun Ke, (2002) reports the ability to remain erect in flows of 0.6-0.8 m deep with velocities up to 3.5 m/s. No other reports on the hydraulic characteristics of single plants could be found.

2.2.4 Summary and Evaluation

The different formulae for the flow through hedges are presented in figure 2.5.

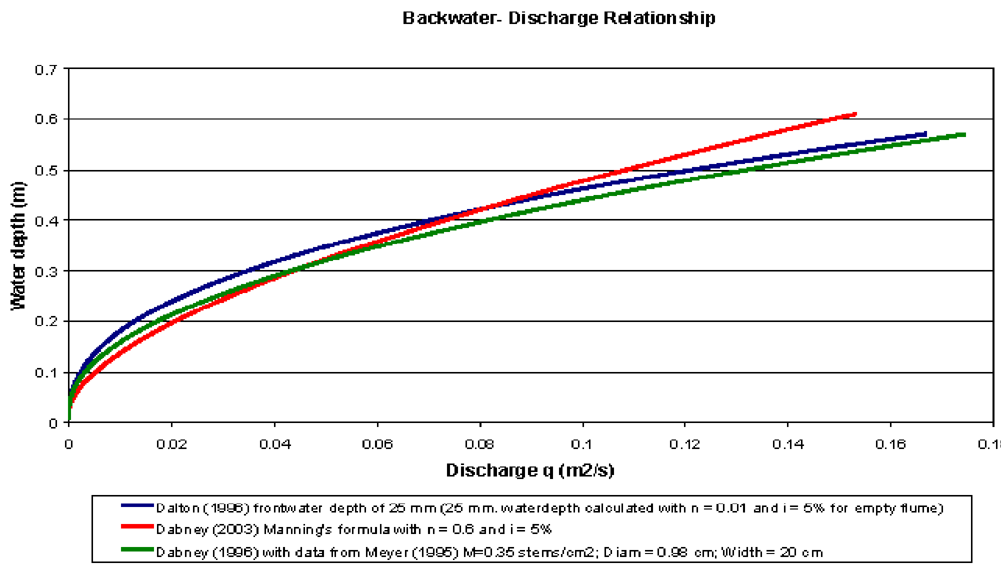


Figure 2.5 backwater specific discharge relationships

The backwater discharge relationships show remarkable similarities. This is mainly because of the fact that the hedges are all tested under the same flow regimes and the formulas are found by curve fitting. The formula of Dabney, 2003 can only be used for the slopes tested since the backwaterdepth is independent of the slope. The use of Manning's formula is not appropriate for a situation with a discrete hedge. Dabney, 1996 does not present any theoretical background or measurements. After personal communication no further information on equation 2.14 could be presented, therefore this formula should be used with great care. In the following chapters the formula presented by Dalton, 1996 will be used. Some some remarks about the values of x, a and b are made below.

The formula presented by Dalton can be theoretically explained as follows: The energy loss (ς) through the hedge can be calculated as follows:

Equation 2.15

$$d_1 + \frac{u_1^2}{2 \cdot g} - d_2 - \frac{u_2^2}{2 \cdot g} = \varsigma$$

From this formula the following can be derived:

Equation 2.16

$$q = d_2 \cdot \sqrt{2 \cdot g \cdot (-\varsigma + d_1 - d_2) + u_1^2}$$

For very close hedges u_1 will be very small and the u_1^2 term can be neglected. Since the ς also depends on the velocities, this formula can be rewritten as:

$$q = d_2 \cdot \zeta \cdot \sqrt{2 \cdot g \cdot (d_1 - d_2)}$$

The resulting formula is similar to an equation for a submerged orifice. The ζ in this equation depends on the elasticity and diameter of the stems, the width of the hedge, the density and the water depth. This because the characteristics of the hedge change with the height. Since the stems of Vetiver remain largely erect during flow (Dalton, 1996) the effect of the bending of the stems on ζ can be neglected. For calculations on hedges the following formula is proposed.

Equation 2.17

$$q = d_1^a \cdot \psi \cdot (2 \cdot g \cdot (d_1 - d_2))^{0.5}$$

Where:

ψ = a hedge factor only dependent on the hedge characteristics
a = a factor to be determined by experiments

Dalton, 1996 found a slightly higher exponent than the 0.5 mentioned in the formula. This is mainly because of the curve fitting, because a slightly higher value for a and an exponent of 0.5 will give almost the same results.

It has to be noted that when the flow downstream of the hedge is super-critical, which is actually the case for figure 2.5, theoretical the downstream water depth has no influence on the flow through the hedge.

2.3 Objects in Run-up

The only research that could be found on forces on objects in the run-up zone on dikes and slopes was about the forces acting on a crown-wall of a breakwater. In tests with rough slopes usually only the reduction in run-up is measured for random or regular placed blocks or stones. However, for a slender object protruding the run-up flow on dikes and breakwaters no literature could be found.

To get more information about the effect of a hedge on the run-up first one slender object is considered below. To determine the effect of one object in the run-up zone the energydissipation has to be determined. The exact values of the layer thickness and velocities are hard to determine as can be concluded from the different results from van Gent and Schuttrumpf. Therefore the exact values of the energy loss are also hard to find. However, as a first indication the parameters that determine the energyloss can be determined.

2.3.1 Flow around Objects in Run-up

The resistance of one object is causing the dissipation of energy of the flow. Therefore the forces on a object are important for the reduction of the run-up. The forces per unit length on a stiff object in non-stationary flow can be described by the Morrison-equation:

Equation 2.18

$$F(t) = C_m \cdot A_0 \cdot \rho \cdot \frac{dU}{dt} + C_d \cdot D \cdot \frac{1}{2} \cdot \rho \cdot U \cdot |U| \qquad \text{[kN/m]}$$

Flow in run-up can be described as follows:

Equation 2.19

$$u_{bore}(z) \sim \sqrt{g \cdot (R_u - z)} \qquad \text{[m/s]}$$

$$\frac{dz}{dt} \sim (g(R_u - z))^{0.5} \qquad \text{[m/s}^2\text{]}$$

$$T_r \sim \frac{2}{g} \cdot \sqrt{g(R_u - z)} \qquad \text{[s]}$$

In this T_r is the run-up time, the time between the run-up bore hitting the object and the time for the maximum run-up for the undisturbed situation. Schüttrumpf, 2001 states that the bore velocities near the vicinity of the maximum run-up-level cannot be described by $u_{bore} \sim \sqrt{g \cdot (R_u - z)}$ because of viscous effects. This is assumed to be insignificant for large waves in nature. Since T_r and u depend on (R_u-z) the acceleration also depends on (R_u-z).

Equation 2.20

$$\frac{du}{dt} = f(u, T_r)$$

$$F = f(t, T_r, U, D, \rho, \nu, \frac{du}{dt}, h)$$

For run-up this eventually comes down to:

Equation 2.21

$$F = f(t, (R_u - z), g, D, \rho, \nu)$$

Since we are interested in the energy loss for one run-up period, we have to integrate over T_r and multiply by U this gives:

Equation 2.22

$$\Delta E = f(g, (R_u - z), D, \rho, \nu)$$

It can be seen that the dissipation of energy depend on two geometrical parameters that is, D and (R_u-z). The energy dissipation can be made dimensionless by dimensional reasoning:

Equation 2.23

$$\frac{\Delta E}{\rho \cdot g \cdot (R_u - z)^3 \cdot D} = f\left(\frac{\sqrt{g \cdot (R_u - z)} \cdot D}{\nu}, \frac{(R_u - z)}{D} \right)$$

From this it can be seen that the energy dissipation in one wave run-up cycle depends on the Reynolds number and a dimensionless run-up height parameter.

The run-up parameter can be compared to the Keulegan-Carpenter number for sinusoidal motion:

Equation 2.24

$$K = \frac{U_m \cdot T}{D}$$

For run-up this comes down to:

Equation 2.25

$$\frac{U_{bore} \cdot T_r}{D} = \frac{\sqrt{g \cdot (R_u - z)} \cdot \frac{2}{g} \cdot \sqrt{g \cdot (R_u - z)}}{D} = \frac{(R_u - z)}{D}$$

However, it is noted that the Keulegan-Carpenter parameter is defined over the whole wave cycle while the run-up height parameter is defined over the run-up period only. The Keulegan-Carpenter parameter is used to determine the energy loss per unit length of the object. Since the water depth for run-up depends on (Ru-z) this is not necessary for run-up.

In reality the run-up height is not only dependent on the amount of energy but also on the momentum. A crown wall on a breakwater for instance is not reducing run-up because of energy dissipation but because of reflecting the incoming wave. The exchange of momentum however, is also dependent on the same two dimensionless parameters. For a hedge the different stems interact and tests cannot be done with one stem and then simply multiplied therefore run-up tests will be necessary.

2.4 Conclusion

From literature good information could be found on the wave run-up on smooth slopes and on the hydraulic behavior of Vetiver hedges in stationary flow. More research will be needed on:

- Failure of Vetiver in flow
- Peak forces acting on Vetiver in run-up
- The effect of Vetiver hedges on run-up heights and flow.

To do research on the failure of Vetiver in flow, prototype tests are needed. The complex interactions between stiff stems and leaves are hard to scale. Therefore real Vetiver plants need to be tested. Since full-scale run-up tests are quite expensive, flumes could be used where the loading is represented by a hydraulic jump. Layer thickness, celerity and bore wedge angle need to be the same as with run-up.

To test the reduction of Vetiver hedges on run-up scale tests can be conducted. The hydraulic characteristics of the Vetiver hedges can be scaled, except for failure. The hedges can be represented by scaled artificial objects.

3 SMALL-SCALE RUN-UP TESTS

In order to determine the effect of a Vetiver grass hedge on the run-up, small scale run-up tests have to be conducted. The tests are used to determine the effect of a Vetiver grass hedge on:

- Run-up height
- The amount of water flowing through the hedge

In this chapter the design of the test and the scaling of the Vetiver grass hedge is described. For run-up tests, scaling according Froudes law is used, since a for run-up important parameter ξ_0 has to be the same as in full scale.

Table 3.1 Parameters involved in the tests and their scale factor

Parameter	**Scale**
H (wave height)	n
L_0 (deep water wave length)	n
T (wave period)	$n^{0.5}$
R_u (run-up height)	n
H (layer thickness)	n
v (run-up velocity)	$n^{0.5}$

Two dimensionless parameters are derived in chapter 2. In equation 2.23 and further is stated that the two parameters are very important and should be kept constant.

$$\frac{\sqrt{g\cdot(R_u-z)}\cdot D}{v} \quad \text{and} \quad \frac{(R_u-z)}{D}$$

Since they should be kept constant a conflict occurs between the run-up height and the diameter of the objects. Since the velocities are scaled $n^{0.5}$ according to Reynolds, D should be scaled $1/n^{0.5}$. Which is in conflict with the dimensionless run-up height.

3.1 Modeling a Vetiver Grass Hedge

In order to get useful results in small scale run-up tests the Vetiver hedges need to be scaled. However, the complex interaction between stems and leaves is hard to model. Below several options are reviewed.

Brushes: A brush, like Vetiver grass, consists of small flexible stems which are closely packed together. In a brush the same complex interaction between hairs may occur as in Vetiver grass. Before a brush could be used the number of stems and the bending stiffness should be representative for Vetiver grass. This would again be rather complex since Vetiver grass consists of stems with different bending stiffness.

Cylinders: One single Vetiver clump could be represented by one stiff cylinder.

This would neglect the bending of Vetiver grass. However, as said in chapter 2, the effect of bending can be neglected. The drag factor of a cylinder however, is not constant. The drag factor is related to the Reynolds number. Since we do not know the velocities between closely packed cylinders the drag is hard to determine and is not representative for a Vetiver clump.

Vertical plate with openings: A plate with openings can also represent a hedge of Vetiver grass. The openings in the plate should represent the openings in a Vetiver hedge. This can be established by using the stage-discharge relationship of a Vetiver grass hedge described in chapter two. Again problems may occur because of laminar flow through the openings.

3.1.1 The blocking-factor

A vertical plate on the dike slope is used to model the Vetiver hedge. Vertical slits in the plate have to be cut so the plate is hydraulically representative for a Vetiver hedge.

The blocking factor of the Vetiver hedges is calculated as follows. From the stage discharge graph a specific discharge of 0.08 m^2/s is found at a water level of 0.4 m in front of the hedge. The discharge through a contraction of a channel is calculated using the following formula:

Equation 3.1

$$q = b \cdot \mu \cdot \frac{2}{3} \cdot d \cdot \sqrt{\frac{2}{3} \cdot g \cdot d}$$

μ is the contraction coefficient and depends on the blocking factor, Froude number and the width of the sheet. From Kindsvater an Carter, 1953 an average value of 0.7 for μ is found. For q = 0.08 m^2/s and d = 0.4 m a value for b is found of 0.25. Hence the blocking factor of the Vetiver grass is 75 %. By planting Vetiver in a wider grid or overloading a hedge lower blocking factors could be obtained. Therefore tests with different blocking factors are conducted: 75 % and 60 %.

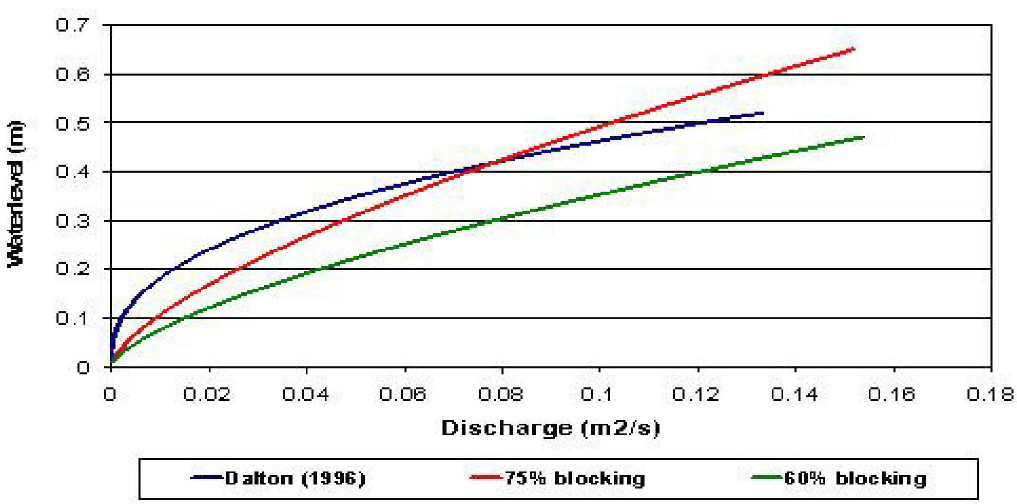


Figure 3.1 Water level discharge graph for Vetiver hedges and different blocking factors

3.1.2 Distribution of the slits

Making the slits too small would overestimate the reduction of run-up because of laminar flow through the slits. This is inevitable since the flow is oscillatory and the velocities are near zero near the moment of maximum run-up. The effect of this can be reduced by creating wider slits.

Making the slits too wide would also overestimate the reduction of the run-up. In front of a Vetiver hedge the water surface is assumed horizontal along the hedge. The openings in the hedge are too small to get large differences in water level along the hedge. With very wide gaps in the plates flow in the middle of the gaps is not affected by the blocking on the sides. In figure 3.1 the different flow profiles through the different opening width presented

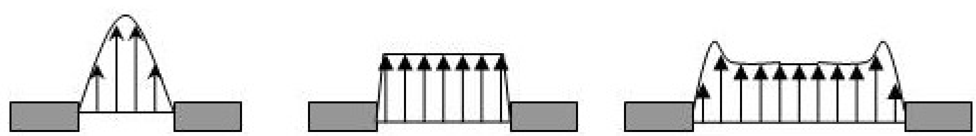


Figure 3.1 Flow profiles through openings in the hedge.

Only for the middle turbulent flow profile hardly any wall effects occur. Because the flow profile and velocities are hard to determine several distributions of slits were tested in a wave flume of 80 cm wide. The results for the different plates with the same blocking factor can only be similar, when the middle flow profile is mainly present.

The same flow profile is obtained in stationary flow. The blocking factor of 75 percent resembles vetiver in stationary flow. Given the rectangular flow profile the flow through the openings can be considered quasi-steady. And so a plate with slits may resemble a vetiver hedge very well in run-up.

Table 3.2 Table with different hedges tested. Hedges with two different blocking factors were used.

Name	Blocking	Number of slits	Width of slits	Spacing
A	0%	1	80 cm	-
B	75%	8	2.5 cm	10 cm
C	75%	4	5 cm	20 cm
D	60%	16	2 cm	5 cm
E	60%	8	4 cm	10 cm
F	60%	4	8 cm	20 cm

3.2 Test Design

For the dike a slope of 1:3 is designed since this is a common slope angle for dikes. Run-up is measured by placing another slope 1:3 behind the hedge and measuring the run-up height. In figure 3.3 the test set-up is shown:

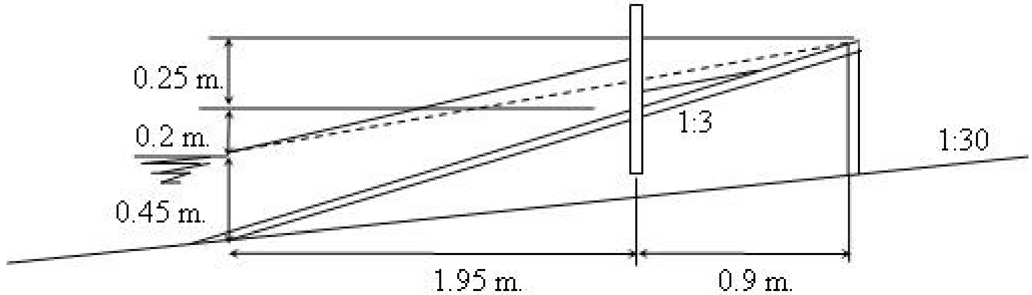


Figure 3.3 Test set-up used to measure the reduction of the run-up.

The amount of water passing through the hedge is determined by putting a box behind the hedge and measuring the amount of water of a controlled number of waves.

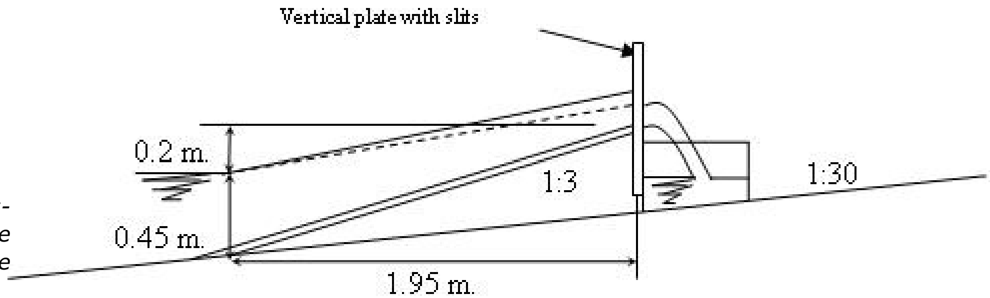


Figure 3.4 Test set-up used to measure the reduction of the overtopping.

For the slope and the vertical plates laminated wooden sheets of 18 mm are used. Each of the different plates is tested for 6 different regular wave climates:

Table 3.3 Waves used for testing. V_{max} is calculated by using 0.94 for c_u this is proposed by Schüttrumpf (2001) see chapter 2

Nr.	H	T	L_0	ξ	R_u (Hunt)	V_{max}
1	125 mm.	2.28 s	8.1 m	2.7	0.34 m	1.10 m/s
2	128 mm.	2.5 s	9.7 m	2.9	0.37 m	1.25 m/s
3	135 mm.	2.5 s	9.7 m	2.8	0.38 m	2.03 m/s
5	150 mm.	2.5 s	9.7 m	2.7	0.40 m.	1.32 m/s
4	140 mm.	2.70 s	11.3 m	3	0.42 m	1.23 m/s
6	160 mm.	2.88 s	12.95 m	3	0.48 m	1.50 m/s

For some plates additional tests were conducted with other waves these are shown in table 3.4

Table 3.4 Waves used for testing.

Nr.	H	T	L_0	ξ	R_u (Hunt)	V_{max}
7	0.12	2.08	6.75	2.5	0.3	0.99
8	0.14	2.25	7.90	2.5	0.35	1.21
9	0.16	2.4	8.98	2.5	0.4	1.4

In table 3.5 the plates with different blocking factors are described. The maximum Reynolds number given is for waves of 0.1 m which generate a flow through the slits. Using higher waves will give even higher Reynolds numbers.

Table 3.5 The hedges and the Reynolds number and dimensionless run-up number. $(R_u\text{-}z)/D$ is calculated for the smallest wave. $Re_{max,1}$ is calculated by using the max. velocity from the first wave and the width of the slits

Name	Blocking	No. of slits	Width of slits	$(R_u\text{-}z)/D$	$Re_{max,1}$
A	0%	1	80 cm	-	$8.8*10^5$
B	75%	8	2.5 cm	1.7	$2.7*10^4$
C	75%	4	5 cm	0.85	$5.5*10^4$
D	60%	16	2 cm	3.4	$2.2*10^4$
E	60%	8	4 cm	1.7	$4.4*10^4$
F	60%	4	8 cm	0.85	$8.8*10^4$

The Reynolds numbers seem high enough to ensure turbulent flow. However, these are calculated by the velocities of the undisturbed run-up. The hedge itself will slow down the run-up velocities and pond water.

3.3 Test Procedure

The run-up is measured by use of a point gauge. A series of waves are monitored and the average run-up of the regular waves is being used. A picture of the upper part of the slope is visible in figure 5.3.

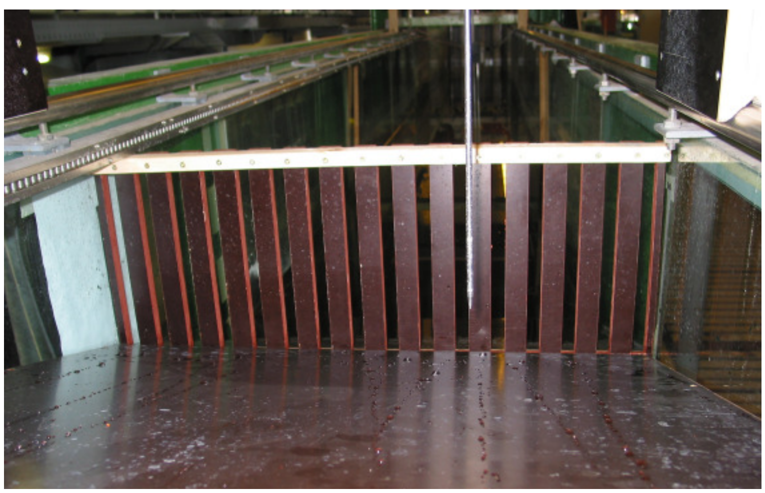


Figure 3.5 A Picture of the test set-up. Looking down from the crest. The point gauge for measuring the run-up level and the hedge behind the plate is clearly visible. This part of the slope could be removed to measure the overtopping

The amount of water flowing through the hedges is measured by collecting water that is passing through the hedge in a box. This is established by removing the slope above the hedge. First the wave generator was started so no start effects would be measured. When a controlled number of waves passed the hedges, first the box was closed than the wave generator was stopped. The water collected in the box was pumped in another box that could be weighed in order to determine the amount of water.

Figure 3.6 The box that was used to collect the water. The pump was used to pump water to a weighing device. In the background the hedge is visible.

4. RESULTS OF THE TESTS

The results of the tests conducted are presented here. First the general processes observed are discussed. This is a description of how a Vetiver hedge is reducing the run-up. Then an error analysis is described before the measured results are presented.

4.1 Processes observed

When the run-up tongue is hitting the Vetiver hedge, part of the water is blocked so the run-up is reduced. This is off course the most important part of the reduction of the run-up. (see figure 4.1)

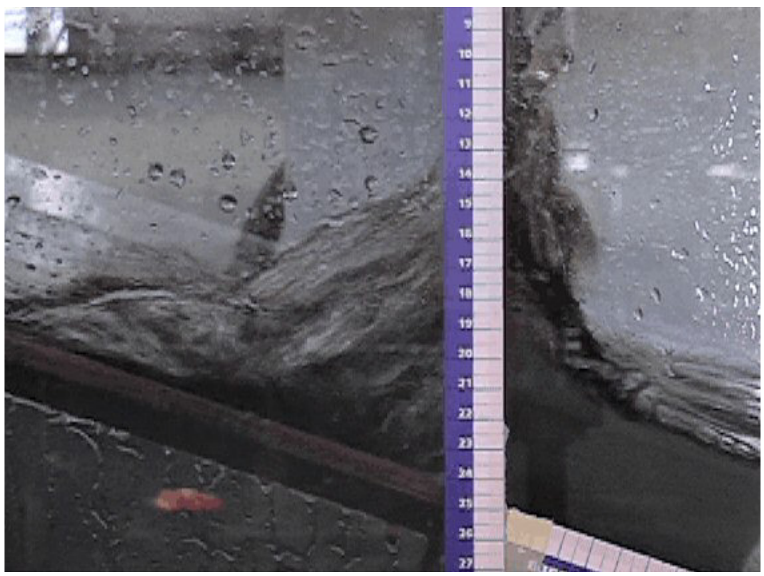


Figure 4.1 Side view of the run-up bore hitting the hedge. (behind the measuring tape). The water is partly splashing up against the hedge and partly transmitted.

The water that is blocked by the hedge creates a reflected wave from the hedge. (see figure 4.2) This wave is running down the slope to the water. Breaking of a wave is related to the steepness of the water surface. This is caused by the combination of the incoming and reflected wave. Since the hedge is altering the reflected wave, the breaking is also changed. The change of the breaking is determined by the breaker parameter (ξ) and the dimensionless run-up height (R_u/z).

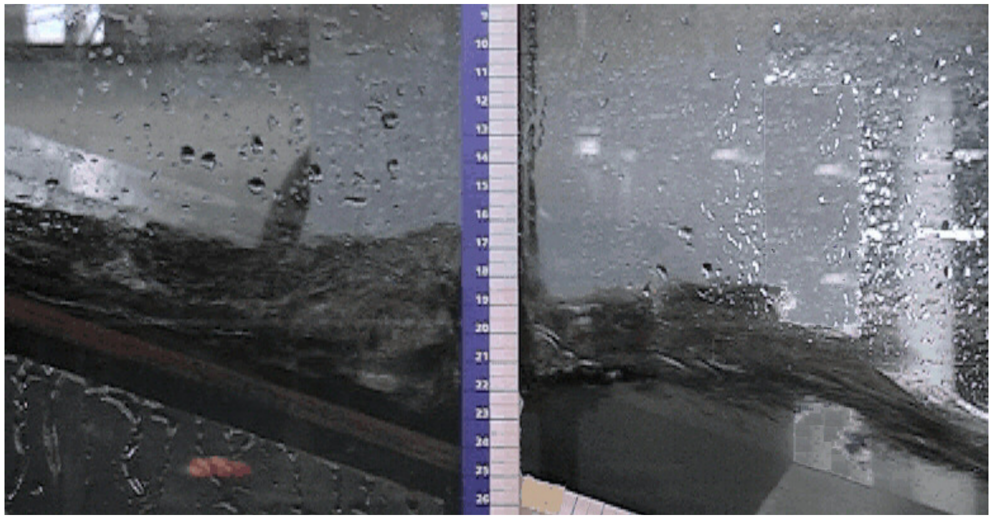


Figure 4.2 Side view of the reflected wave of the run-up bore. The maximum run-up level has not yet been reached. Yet a wave on top of the run-up tongue is running down.

The water that is passing through the hedge after reaching its maximum run-up level is also running down. This water is again slowed down by the hedge on its way back down the slope. The run-up tongue of the new wave has to run-up over a layer of water flowing down the slope. This will reduce the run-up of the new wave. The reduction depends on the dimensionless run-up height (R_u/z)

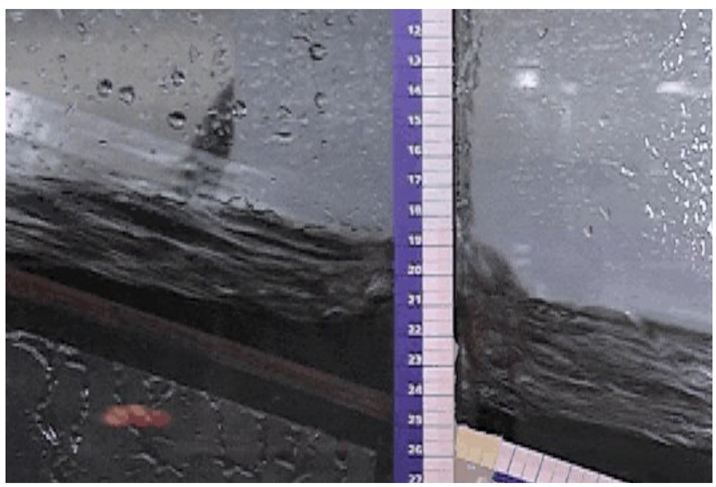


Figure 4.3 Water running down the slope is also slowed down by the hedge.

4.2 Error Analysis

Three different sources for errors can be distinguished: scale effects, measurement errors and model errors. These are discussed before the results of the tests are presented.

4.2.1 Scale Effects

In most cases it is impossible to simulate properly all of the processes involved. In order to get representative results the scale of the tests need to be adapted to decrease the scale effects. The scale effects concerning the viscosity of water were already described in the previous chapter. The plates were designed such that the viscosity does not play an important role.

The surface tension of the air-water surface can play a role in the wave celerity for small waves. For depths over 2 centimeters and periods of over 0.35 seconds this does not play an significant role (Schüttrumpf, 2001).

During the impact of the run-up tongue compressibility may play a role. The air water mixture in the leading bore is far more compressible than water. Tests with different scales to determine the forces on a crown wall give no big differences (Martin, 2000). This is only true for a run-up tongue hitting the crown wall. If waves break on the wall compressibility may play an important role (Martin, 2000).

4.2.2 Model errors

The model is made of wooden plates. These plates of 18 mm thick may widen during the tests because of the water. The slits in the plates are cut with an accuracy of about 1 mm. These errors are insignificant for the tests especially when comparing different results.

The wave generator was ordered to create waves of one wave height and one period. Although the wave generator is equipped with an automatic reflection compensator, different run-up heights are found in the tests. Differences of the order of 6 mm were found in the run-up. This corresponds with a wave height difference of 2 mm.

The overtopping water was collected in a box. From this box the water was pumped into another box hanging under a weighing device. While pumping water from one box to another box errors can be made because of residues in the pump and the boxes. This error is estimated at 2 liter.

4.2.3 Measuring errors

The following measurement devices are used:

Table 4.1 Errors occurring because of the measuring device.

Device	Measured Parameter	Error
Point gauge	Run-up	± 1 mm
Weighing device	Amount of water	± 1 kg.
Electronic wave gauge	Water level	± 0.5 %

The electronic wave gauge records a water level every 0.02 seconds. The measurements are used to measure the decrease in water level, after the overtopping test is conducted. The records are averaged and the water level is determined before and after (see Appendix 1). By doing so the error increases.

To decrease the errors, averages are used to measure the effect for one single wave. The run-up is measured for several waves and an average is noted. For the overtopping a controlled number of waves is allowed to overtop and the complete volume is measured. The combined measuring and model errors then become as follow:

Table 4.2 Combined measuring and test set-up errors.

Parameter	Error
Average run-up	3 mm.
Overtopping volume	$3*10^{-3}$ m^3/number of waves
Water level	0.5 mm

4.3 Quantitative Results

The reduction of the run-up is related to the energy dissipation. In chapter 2 the energy dissipation for flow around one object was mentioned. The dimensionless numbers from chapter 2 and the flow profiles from chapter 3 are used below to explain the results of the tests.

In chapter two dimensionless parameters are determined that affect the dissipation of energy and the exchange of momentum during run-up with a slender object:

Equation 4.1

$$\frac{\Delta E}{\rho \cdot g \cdot (R_{usmooth} - z)^3 \cdot D} = f\left(\frac{\sqrt{g \cdot (R_{usmooth} - z)} \cdot D}{\nu}, \frac{(R_{usmooth} - z)}{D} \right)$$

This is only true for one object. For a group of objects, like the plate with the slits, the energy loss per unit length can be calculated by multiplying the energy loss by the density, by n/l, in which n is the number of stems and l is the hedge length. By doing so the interaction between the different objects is neglected. The flow pattern around the objects interfere with each other and this should be taken into account too. Full expression for the energy loss per unit length then can be described as:

Equation 4.2

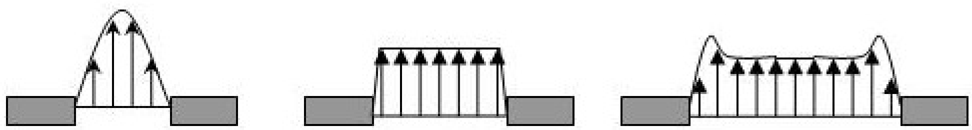
$$\frac{\Delta E}{\rho \cdot g \cdot (R_{usmooth} - z)^3 \cdot D} / l = f\left(\frac{\sqrt{g \cdot (R_{usmooth} - z)} \cdot D}{\nu}, \frac{(R_{usmooth} - z)}{D}, \% \right) \cdot \frac{n}{l}$$

The energy loss per unit length is linked to the volume behind the run-up hedge at the moment of maximum run-up. The energy of a run-up tongue can for instance be calculated by calculating the potential energy of the run-up tongue at the moment the maximum run-up height is reached. The reduction of the run-up volume is determined by the same parameters as above:

Equation 4.3

$$\frac{\Delta V}{V} / l = f\left(\frac{\sqrt{g \cdot (R_{usmooth} - z)} \cdot D}{\nu}, \frac{(R_{usmooth} - z)}{D}, \% \right) \cdot \frac{n}{l}$$

For the D not only the size of the objects can be used but also the size of the openings can be used as input. The dimensionless parameters then determine the flow pattern inside the openings. The expected patterns from chapter 3 are repeated here:



The left flow pattern can be obtained when the Reynolds term is small, so the flow in the opening is laminar. The right flow profile can be obtained when the dimensionless run-up height is very small. The opening is too large compared to the run-up height. The flow in the middle of the gaps is not affected by the blocking of the sides. For a hedge of vetiver the water level is assumed horizontal in front of the hedge. For too wide slits the water level is not horizontal along the hedge.

The plates are designed to obtain the middle profile with both a high Reynolds number and a high relative run-up height parameter. The relative run-up parameter is now an expression for the shape of the flow pattern which should be rectangle. This does not mean that the other flow patterns do not perform during a run-up cycle. However, compared to the total run-up time these patterns can be neglected if the results from the tests with the different slits are equal. The volume overtopping through the hedge can now be described as:

Equation 4.4

$$\frac{V_{hedge}}{l} \sim D \cdot \frac{n}{l} \cdot T_r \cdot u_{smooth} \cdot h_{smooth} \cdot f\left(\frac{\sqrt{g \cdot (R_{usmooth} - z)} \cdot D}{\nu}, \frac{(R_{usmooth} - z)}{D}, \% \right)$$

$$\frac{V_{hedge}}{l} \sim \frac{D \cdot n \cdot \frac{2}{g} g \cdot (R_{usmooth})^2}{l} \cdot f\left(\frac{\sqrt{g \cdot (R_{usmooth} - z)} \cdot D}{\nu}, \frac{(R_{usmooth} - z)}{D}, \% \right)$$

And the change of the volume can be described as follows:

$$\frac{V_{smooth}}{l} \sim (R_{usmooth})^2$$

Equation 4.5

$$\frac{V_{hedge}}{V_{smooth}} = \frac{D \cdot n}{l} \cdot f\left(\frac{\sqrt{g \cdot (R_{usmooth} - z)} \cdot D}{\nu}, \frac{(R_{usmooth} - z)}{D}, \% \right)$$

Because of the fact that for the same blocking factors the same results are expected the function between the parenthesis can only depend on (Ru-z). The change of D and n is not allowed to change the result. The reduction then results in the following:

Equation 4.6

$$\frac{V_{hedge}}{V_{smooth}} \sim \frac{n \cdot D}{l} \cdot f\left(R_{usmooth} - z\right)$$

The problem with the dependency of (Ru-z) is that a dimension of length has been introduced. The dimensions both on the left and right hand side of the equations have to be the same. This can only be reached if:

$$f\left(R_{usmooth} - z\right) = constant$$

Equation 4.7

$$\frac{V_{hedge}}{V_{smooth}} = c \cdot \frac{D \cdot n}{L}$$

So when a constant reduction is measured, the flow can be assumed to be quasi-steady. And the forces are drag dominant. (see paragraph 3.1.2)

4.3.2 Overtopping

The amount of water passing through the hedge during one run-up cycle was measured by collecting the water in a box. By doing so the amount of water passing through the hedge is overestimated, because no water running down the slope will reduce the run-up of the next wave. For overtopping on smooth slopes the overtopping can be calculated by determining the imaginary run-up tongue (Schüttrumpf, 2001).

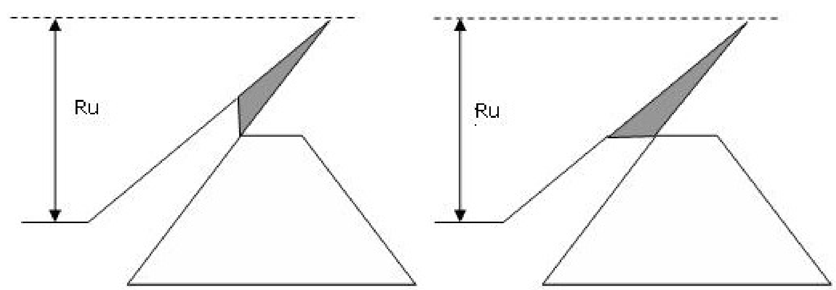


Figure 4.5 Calculation of the overtopping volumes

The run-up volume on the left side of the figure 4.5 needs to be measured. Because of the overestimation the overtopping volumes found in tests with a smooth slope, found a better fit with calculations which use the volume of the run-up tongue above the crest level (right side of figure 4.5). (Battjes, 1976). The measured data were compared with calculations of the overtopping volume, calculated by using c_h= 0.284 and the measured run-up levels.

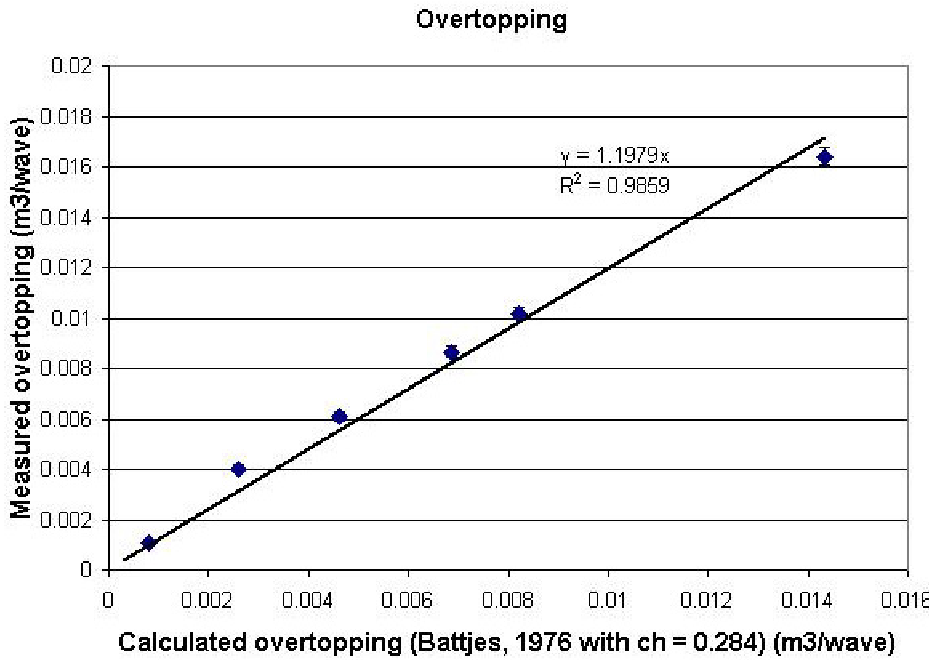


Figure 4.6 Measured overtopping volumes versus the calculated overtopping volumes.

The measured data are all larger than the calculated data, because of the absence of a horizontal dike crest in the tests. The difference between the calculated data and the measured data is about 20%. The calculation of the overtopping itself gives an overestimation of 42%. So the total amount of overestimation of the run-up tongue is 69 %. However, it is assumed that this is the same for the overtopping with the hedges. The reduction in the volume of overtopping can be seen in the graph below for the plates with 75% blocking.

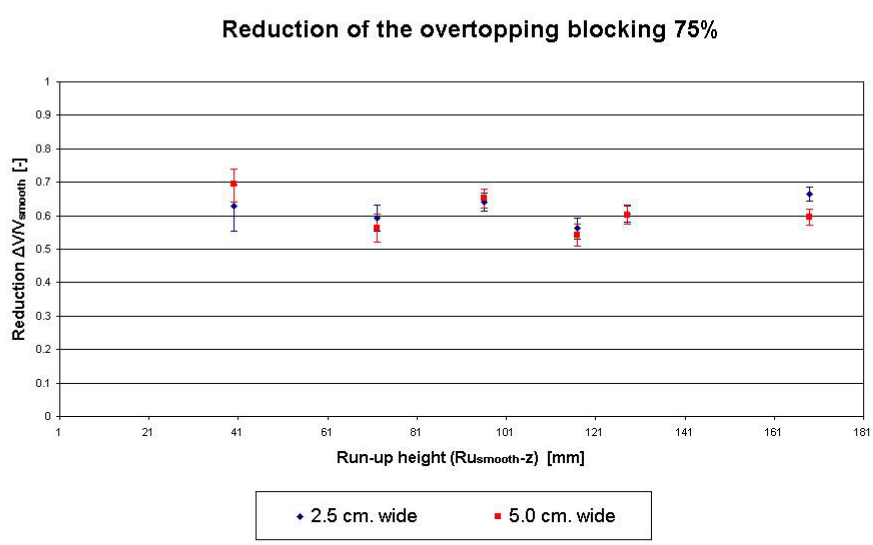


Figure 4.7 Reduction of the overtopping volume with a blocking factor of 75%

The reduction of the overtopping volume is considerable. The volume is reduced by over 55%. No significant differences could be found between the different hedges with the different widths of the gaps except for the largest run-up. The reduction is almost constant as expected.

The tests were conducted with different breaker parameters. Because of that the small differences can be explained. The influence of the breaker parameter may be the effect of the reflection of the waves. On the other hand, it is more likely that different breaker types create different amounts of turbulence in the tip of the run-up tongue. In the formulas for the reduction of the run-up the average velocities are used, which gives an underestimation of the forces. Since the differences are small no further tests are conducted to measure the influence of the breaker parameter.

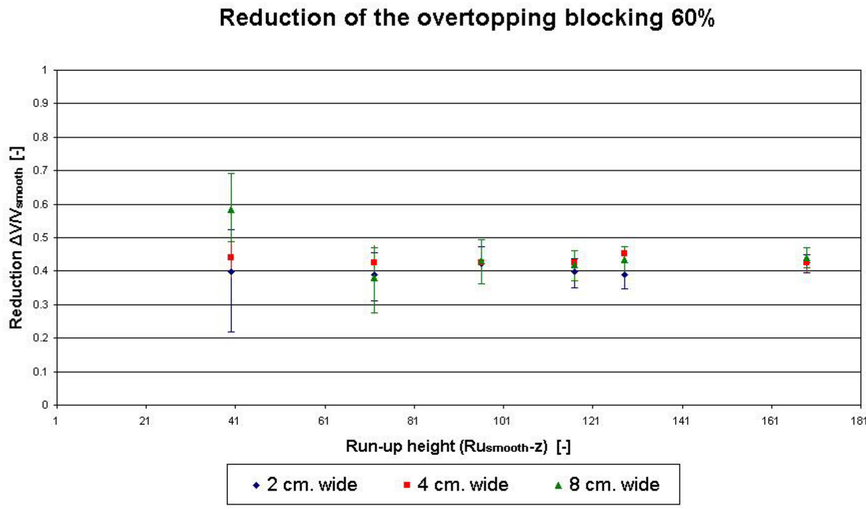


Figure 4.8 Reduction of the overtopping volumes with a blocking factor of 60%

The reduction of the overtopping is much smaller than with the plate with 75 % blocking as can be expected. The reduction is about 40% as can be seen from figure 4.8.

4.3.1 Run-up height

The run-up height and the run-up volume are strongly linked to each other. Therefore run-up height should give the same trend as the run-up volume. The results should be independent of the run-up height and only depend on the blocking factor.

For the hedges with a blocking factor of 75% the results are presented below:

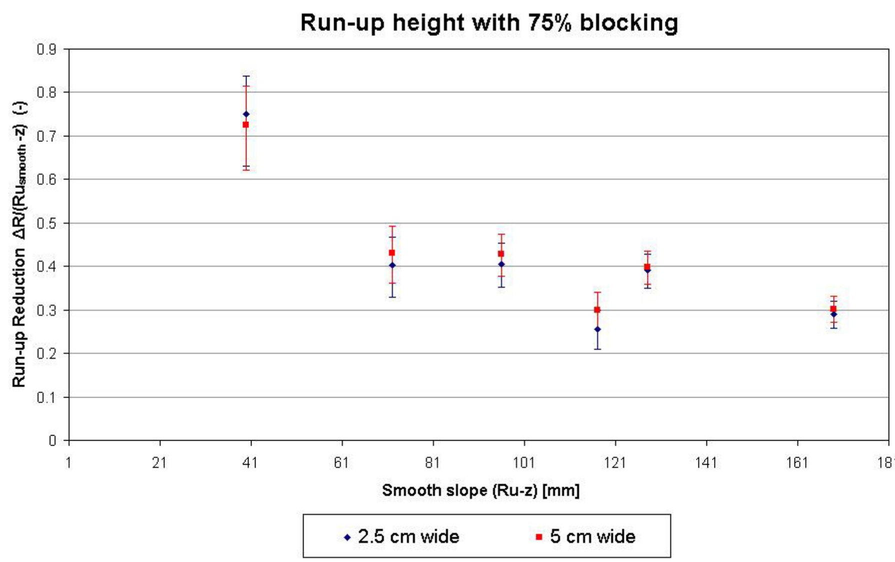


Figure 4.9 Reduction of the run-up height with 75% blocking.

From this graph it can be seen that no differences significant are found in the run-up height between the different slit width. On the other hand, no clear trend can be observed. The reduction is over 25 % in all cases. In figure 4.10 the results from the hedge with the narrow slits are presented together with some additional measurements. The reduction is now a function the breaker parameter:

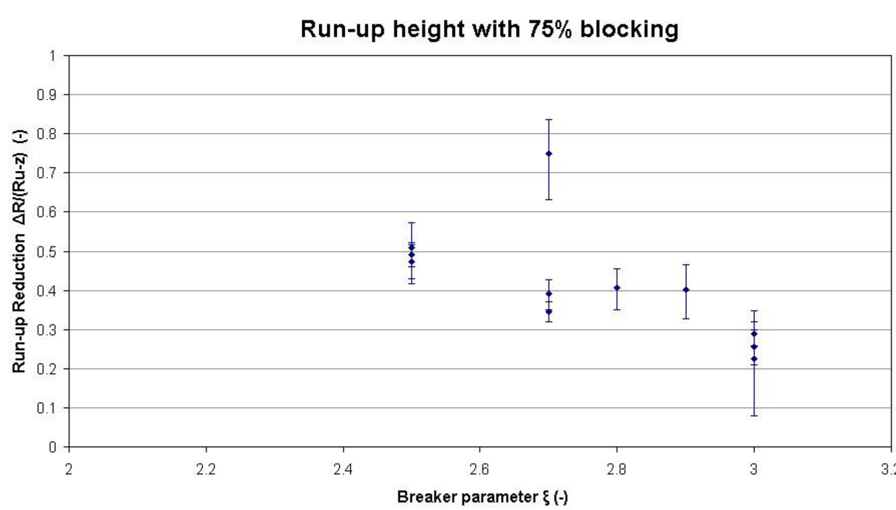


Figure 4.10 Reduction of the run-up versus breaker parameter.

The influence of the breaker parameter is visible in this graph. A higher breaker parameter gives less reduction. A breaker parameter of 3 gives half the reduction as the breaker parameter of 2.5. Only one point doesn't fit, this can be caused by the low Reynolds number because of the fact that this point is measured at very low run-up.

The reduction seems constant for different run-up heights if the breaker parameter is kept constant. This means that the relative reduction of the run-up height is independent of (Ru-z).

The breaker parameter determines the breaking of a wave. A surging wave with a high breaker parameter has less turbulence at the tip of the run-up tongue than plunging waves.

The reduction is determined by using an average velocity. In the case of higher turbulent flow because of the differences in velocities are the forces on the objects are larger. This increases the reduction of the run-up.

In figure 4.11 the results for the hedges with a blocking factor of 60% are presented. Also no significant differences between the hedges, although the widest slits tend to give less reduction in run-up.

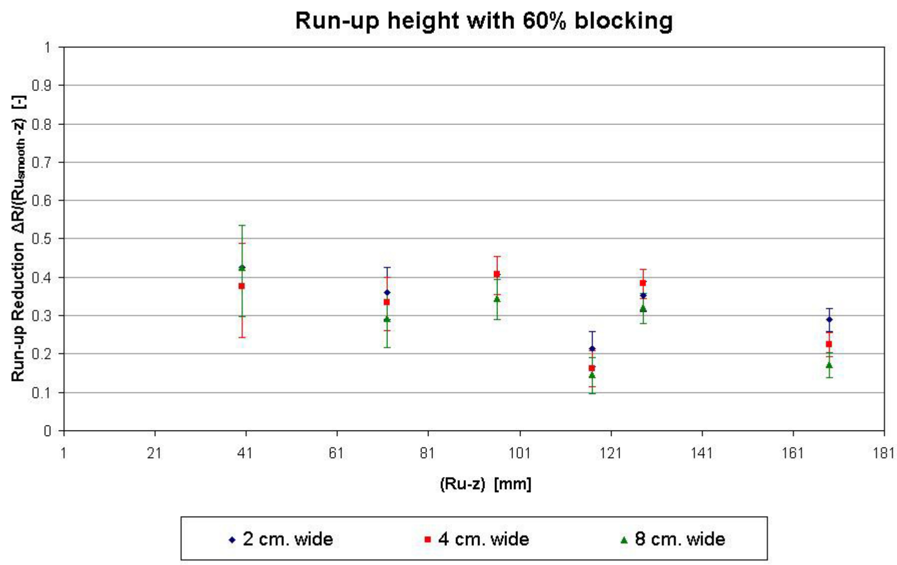


Figure 4.11 Run-up reduction with 60% blocking.

In figure 4.12 the reduction again is set out versus the breaker parameter.

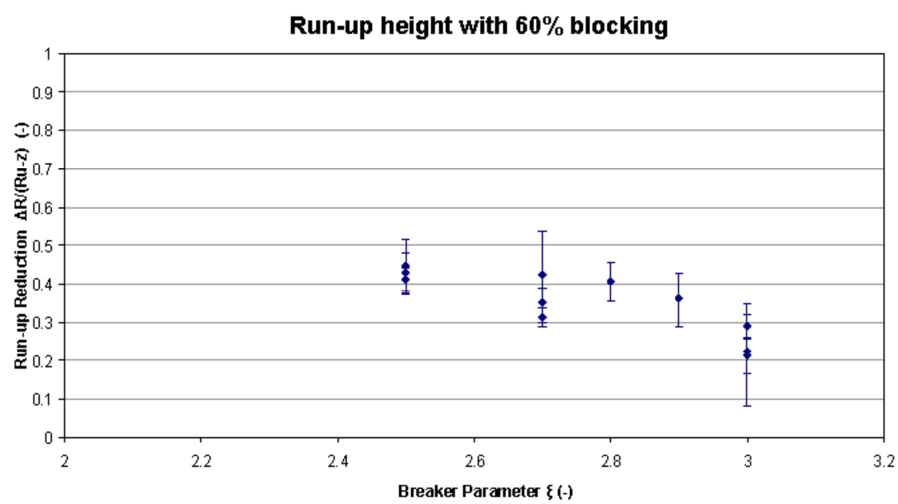


Figure 4.12 Run-up reduction versus the breaker parameter

In this graph the same trend as with the hedges with a blocking factor of 75% is visible. The relative reduction ΔRu/(Ru-z) seems independent of the run-up height, but depends on the breaker parameter.

The differences between the two blocking factors are small especially for the larger breaker parameter. The reduction of the run-up is for both blocking factors about 20-25 % at a breaker parameter of 3.

4.3.3 Conclusion and Evaluation

The theory that the reduction is independent of the run-up height could not be rejected. Both the relative reductions of the run-up height and the overtopping volume were found constant over the run-up height. The run-up height reduction by both hedges is at least 20% for the breaker parameters tested. The reduction of the volumes is at least 55% for the 75% blocking hedge and around 40% for the hedge with a blocking of 60%.

For the Reynolds numbers, the relative run-up height number and blocking factors tested, the reduction of the run-up height through a plate with vertical slits depends mainly on the breaker parameter. The differences in the results for the different breaker parameters are larger than the differences between the blocking factors tested. The run-up height is determined by the fast flowing tip of the run-up tongue. For a Vetiver hedge with higher blocking factors at the lower parts of the hedge the reduction of the run-up height could be more. The reduction of the run-up height is difficult to use for further calculation since the differences between the different breaker parameters are large.

The reduction of the overtopping volume is much more than the reduction of the run-up height. The influence of the breaker parameter on the overtopping volumes is very small.

Since the main reason to plant Vetiver on a dike is to decrease the overtopping volumes, the reduction of volume is a better parameter to use for further calculation than the reduction of the height.

Forces on the plates in the tests are drag dominant. This means that the flows through the plates are quasi-steady. The forces on the plate are mainly caused by the velocities and not by the acceleration or deceleration of the flow.

It is expected that for higher run-up heights the flow remains drag dominant. (Ru-z)/D is only getting larger for higher run-up so the flow profile in the opening remains a rectangle. For a real Vetiver hedge the D is getting smaller this also causes (Ru-z)/D to increase so for a Vetiver hedge the flow is also drag dominant. This is similar to the behavior with short waves through vegetation.

A higher Keulegan-Carpenter parameter creates more drag dominant flow.

The hedges were designed to simulate the drag of Vetiver grass in steady flow. Since the flow through the openings in the run-up tests were quasi-steady it can be concluded that the plates with the slits are a good model for a Vetiver hedge.

5. VETIVER GRASS ON A DIKE

To determine the effect of Vetiver grass hedges on a dike an example is worked out below. In this example a conventional dike with conventional armour layer and a dike covered with Vetiver grass are designed and compared. The differences in use of material and costs will are shown. In this example the wave climate is determined by some rules of thumb, Therefore the examples can only be used for comparison and indication.

5.1 Location

The location is in Vietnam in the coastal district southeast of Ho Chi Minh City. This Can Gio district is a large biosphere reserve recognized by the UNESCO. It covers 75,740 hectares and is dominated by mangroves both brackish and salt water species. About 58,000 people are living within the boundaries of the reserve of which 54,000 live in the transition area (1997). The district is the poorest district of the Ho Chi Minh province. The biosphere reserve is expected to be a site where eco-tourism and different sustainable economic activities can be implemented to develop the area. (UNESCO, 2000). In the south of the district there are some villages along the coast that need protection from the waves coming from the sea.

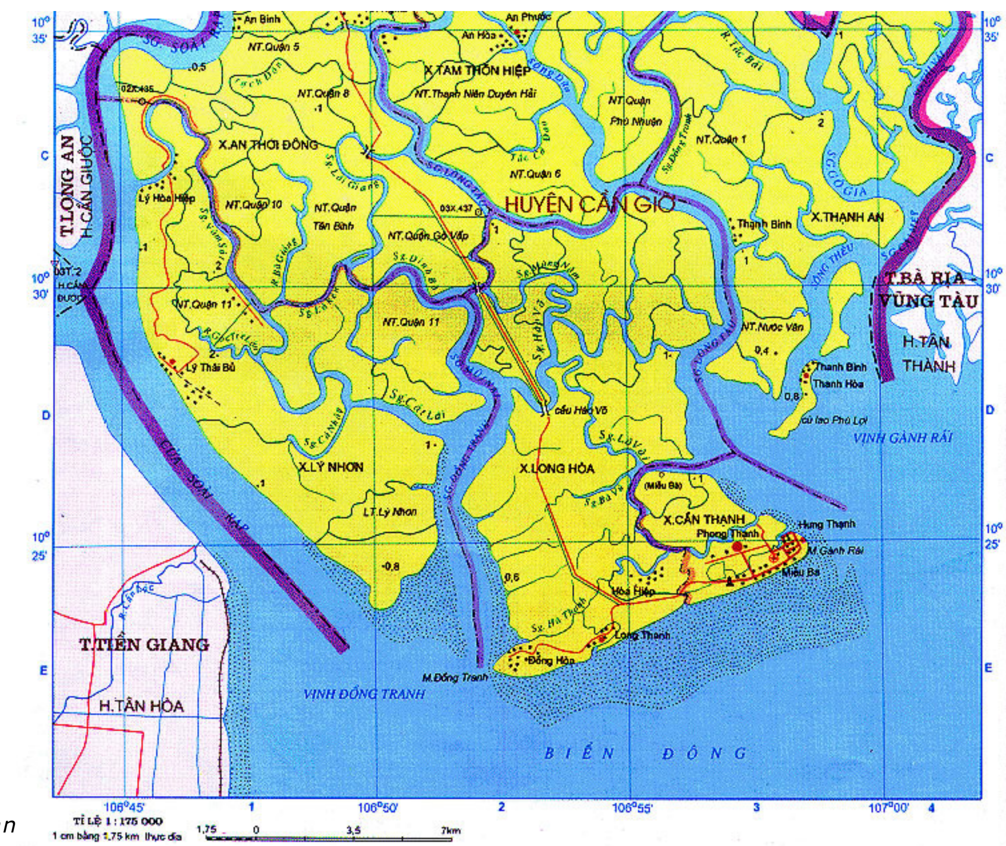


Figure 5.1 Map of the Can Gio district

There is a large mud flat lying in front of the coast. This mud flat is protecting the coast most of the time from wave attack. However during storms the water level may rise because of wind set-up. The typhoon Linda (1997), one of the severest typhoons of the century caused wind set-up of 1 meter at sea and 4 meters inside the delta. No levels of set-up along the coast are known. A set-up of 2.5 meters along the coast seems a good assumption. From this maximum water level the wave climate at the toe of the dike can be determined by the following rule of thumb (d'Angremond, 2001):

Equation 5.1

$$H_s \leq 0.55 \cdot h$$

Breaking of the waves causes the wave spectrum to change; this is neglected in this example. The spectral wave period is assumed to be 7 seconds, so the value of the breaker parameter becomes 2.5.

5.2 Design of the Dike

The height of a dike is determined by the amount of overtopping allowed. In the Netherlands the following design rule from van der Meer, 2001 is used to determine the crest height:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0,067}{\sqrt{\tan\alpha}} \cdot \gamma_b \cdot \xi_0 \cdot \exp\left(-4,3 \cdot \frac{h_k}{H_{m0}} \cdot \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right)$$

With a maximum of

Equation 5.2

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0,2 \cdot \exp\left(-2,3 \cdot \frac{h_k}{H_{m0}} \cdot \frac{1}{\gamma_f \cdot \gamma_\beta}\right)$$

In this equation q is the average discharge allowed the other factors are the same as in equation 2.3. The discharge allowed is determined by the state of the inner slope. In the guidelines the following average discharges are mentioned:

0.1 l/m per s for sandy soil and a bad cover
1.0 l/m per s for clay with a reasonable well grass cover
10 l/m per s for a good grass cover or an armour layer.

The conventional dikes in the area consist of clay, covered with a geo-textile and an armour layer of placed concrete blocks or placed granite stones on a granular filter. This armour layer has a roughness of 0.95. The inner slope is unprotected and just covered with some low grasses and weed. The outer slope has a slope angle of 1:3. The inner slope is assumed 1:2.5. If an average overtopping discharge of 0.1 l/m per second is used as a design rule, a dike height of 4.57 meters will be necessary. The cross section of the dike is drawn in figure 5.2.

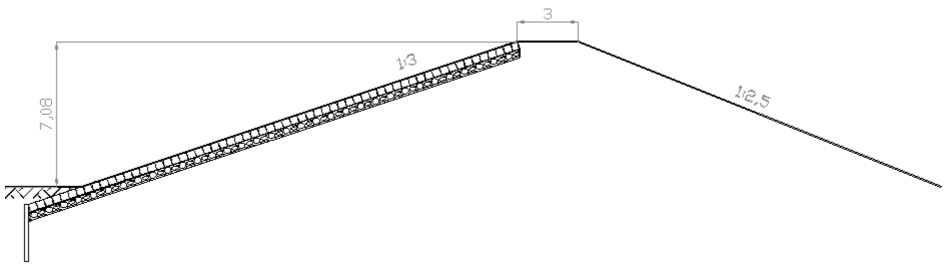


Figure 5.2 Cross-section of the dike without Vetiver grass.

Since a hedge of Vetiver grass will reduce the volume of overtopping by 55 % the crest levels can be decreased. The new crest levels are given in the table 5.1:

Table 5.1 Crest heights for different numbers of hedges on the outer slope

Nr. of hedges	q allowed without hedge	h_k above SWL
No hedge	0.1	4.58
1 hedge	0.22	4.12
2 hedges	0.49	3.67
3 hedges	1.1	3.21

The reduction by multiple hedges is calculated by multiplying the reduction of one hedge. In reality some water will remain between the hedges and the reduction is even more.

The number of hedges to be planted on a slope is limited by the length of the slope and the strength of the hedges. The hedges need to be planted with a spacing of one meter. This allows people to move through the hedges for maintenance and inspection. Near the still water line the hedges will be overloaded and water will flow over the hedges. Water flowing over the hedges is a different mechanism from water flowing through the hedges.

In chapter 2 it can be found that there is limited knowledge of the strength of mature hedges in flow. The failure mechanism is unknown. For one stem Dunn, 1996 states that after overloading of a stem, the stem will break or develop a hinge point. For a hedge the following extreme failure mechanisms are possible or a combination of those:

- Breaking or hinging of several weaker stems at low heights. A more open hedge remains that can cope with the extra discharge through the hedge.

- Breaking or hinging of all stems at a certain height. The lower part can cope with the load and when overloaded the extra flow will simply flow over the hedge.

- Brending through an elastic range so after overloading the hedge turns back in its original state.

In further calculations the height of 0.4 m is considered as the height, where the stems will break or develop a hinge point or bend. This height can be exceeded by some of the run-up layers, but most of the water overtopping the dike needs to flow through the hedges. Other wise the proposed reduction is not feasible. In figure 5.3 a probability distribution function for wave overtopping is shown:

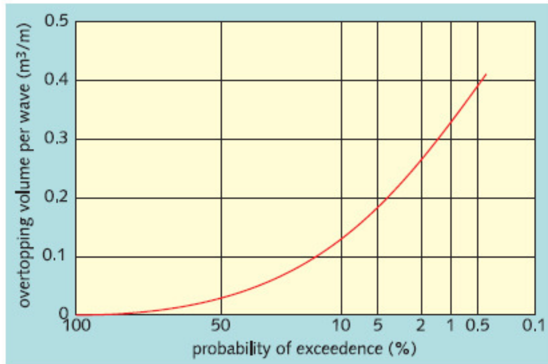


Figure 5.3 probability distribution function for wave overtopping volumes per wave; q = 1 l/s per m width, Tm = 5 s and Pov = 0.10 van der Meer (2002)

Waves with a high probability overtop the dike often but the damage caused is small. Larger waves cause more damage, however their occurrence is very low. Vetiver grass should be able to withstand the most damaging waves.

In this case the 1% wave run-up height is assumed as a design run-up height. For determining the $R_{u1\%}$ van Gent (2002) is used:

Equation 5.3

$$\frac{R_{u1\%}}{(\gamma \cdot H_s)} = c_0 \cdot \xi_0 \qquad for \quad \xi_0 \leq p$$

$$\frac{R_{u1\%}}{(\gamma \cdot H_s)} = c_1 - \frac{c_2}{\xi_0} \qquad for \quad \xi_0 \geq p$$

$$c_2 = 0.25 \cdot \frac{c_1^{\,2}}{c_0} \xi_0 \qquad \mathrm{p} = 0.5 \cdot \frac{\mathrm{c}_1}{\mathrm{c}_0}$$

$$for\ R_{u1\%} \quad \mathrm{c}_0 = 1.45 \quad \mathrm{c}_1 = 5.1$$

In this case the average $R_{u1\%}$ is 4.31 meters. The layer thickness of the run-up tongue can be calculated for a smooth slope by the equation proposed by Schüttrumpf, 2001:

Equation 5.4

$$h = 0.284 \cdot (R_u - z)$$

This is perpendicular to the slope for perpendicular to the hedge c_h should be 0.3. If two hedges are planted the lower hedge is planted 33 cm lower than the crest. The higher hedge is planted on the top of the slope. The lower hedge is planted at 3.37 m from the still water line. In this case the layer thickness is 0.28 m for a $R_{u1\%}$. This is well below the 0.4 m height of the hedge. However, the 0.28 m is calculated for smooth slopes and reflection of part of the wave is not taken into account. So a small part of the water will splash over the hedge , this is considered negligible.

If the height of the hinge or bending point is taken at 0.6 m a third hedge may be considered. A third hedge would need to be planted 0.66 meter below the crest level. At this level the water layer thickness is about 0.53 m for $R_{u1\%}$. Again this is only for a smooth slope and probably a lot of water is flowing over the hedge and the reduction of 55% is not feasible. So, if only water flowing through the hedge is considered, 2 hedges is probably the maximum number of hedges needed

If water is flowing over the hedges, the hedges can be interpreted as artificial roughness elements. Planting more hedges below the 2 already planted, will also have effect on the run-up. This is not tested however, in van der Meer, 2002 roughness elements are described. The hedges can be described as ridges. For small ridges a roughness factor of 0.75 is found. For a cover with small grass, that is sod forming, the roughness is 1,0. Since the roughness is not investigated a safe roughness factor of 0.95 is used for overloaded Vetiver grass the same as for the placed blocks. The new dike can be seen in the picture below:

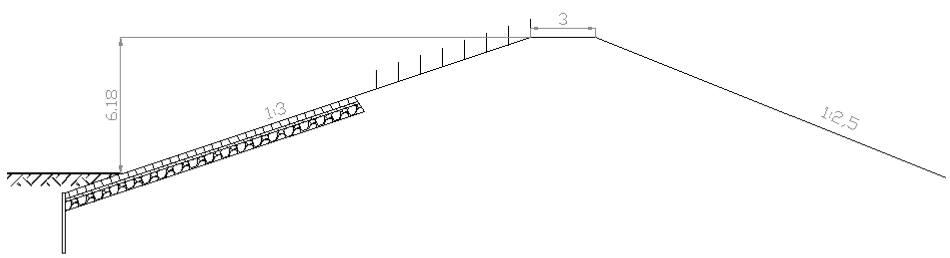


Figure 5.4 Cross section of a dike with Vetiver grass

When an average overtopping discharge of 1 liter/m per second is allowed, considerably more water is flowing over the dike. Allowing more water to overtop the dike can only be done if a good inner slope is present. A good inner slope can be very expensive in construction or management. The heights of a conventional dike and a dike with hedges is described below:

Table 5.2 Crest heights for different numbers of hedges on the outer slope

Nr. of hedges	q allowed without hedge	h_k above SWL
No hedge	1	3.26
1 hedge	2.2	2.81
2 hedges	4.9	2.36

The $R_{u1\%}$ is 4.31 meters and water will flow over the hedges, since the layer thickness will be 0.45 meters at the top of the slope. This is a small amount from the total number of waves spilling over the crest of the dike. Over 10% of the waves will overtop this dike since $R_{u10\%}$ is 3.31 m. However, the 1% wave does considerable more damage. So for a well managed inner slope the reduction of the run-up by flowing through a Vetiver hedge is small. If the Vetiver hedges can be considered as artificial roughness elements the construction costs can still be decreased. The dike with Vetiver as is drawn below:

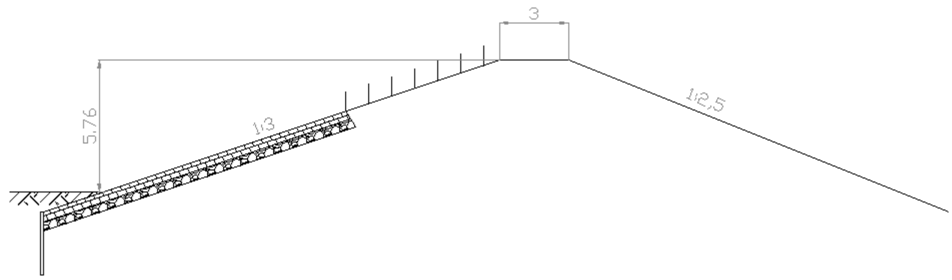


Figure 5.5 Cross section of a dike with Vetiver grass (q= 1.0 l/m/s)

From the calculations above it can be concluded that the effect of Vetiver hedges can be considerable if the average discharge over the dike is small. If the inner slope is protected with a good armour layer planting of Vetiver hedges on the outer slope is not very useful.

5.5 Construction Costs

For the construction costs the following specific costs were applied.

Table 5.3 Prices of different materials

Material	Unit	Price
Clay	m^3	$ 5
Vetiver hedge	m	$ 4
Armour layer	m^3	$ 12

The thickness of the armour layer is assumed to be 0.4 m. The construction costs of the different dikes are described below:

Table 5.4 Costs of the different dikes

Dike	Price/m
Conventional dike q = 0.1 l/m/sec.	$ 871
Vetiver dike q= 0.1 l/m/sec.	$ 691
Conventional dike q = 1.0 l/m/sec.	$ 603
Vetiver dike q = 1.0 l/m/sec	$ 588

In this calculation the extra measures for allowing 1.0 l/m/s over the dike are not taken into account. This might be very expensive and the lower dike may eventually cost more than the dike with an allowed overtopping of 0.1 l/m/s. The results of the conventional dike and the Vetiver dike with the higher overtopping can be compared because, both might need the measures for protection of the inner slope.

5.4 Management Considerations

The height where the stems will break or develop a hinge point is very important in these calculations. The height may be change due to management techniques. By pruning the Vetiver hedge at 0.4 m height the old stems may enforce the lower part and stimulate breaking or hinging at a height of 0.4 m.

Since maintenance and inspection will be needed, paths have to be cut through the hedges. These will become week spots in the dike armour layer. These paths and the inner slope behind it need extra protection.

In between the hedges the slope should be protected with small good sod forming grass. Regular pruning of Vetiver grass, so sunlight can break through, might help growing the grass in between the hedges.

Because of all these management measures, the management of Vetiver grass hedges can be more expensive than the management of a conventional armour layer. However in countries with low wages like Vietnam and Bangladesh the reduction of the construction costs will be considerably more than the extra management costs

5.5 Conclusions

The reduction of the costs can be 20% as the example above shows. However, the result highly depends on the average discharge allowed. For low average discharges Vetiver can be very effective. The calculations above show that some assumptions have to be made before the effect could be calculated. In further research the following could be investigated:

- Roughness when the run-up tongue flows over the Vetiver hedge.
- Failure mechanism and the maximum load.

6 CONCLUSIONS AND RECOMMENDATIONS

In the first chapter a problem definition was described. From that the objective of this research was derived. The objective of this research is repeated below:

"Determine the effect of Vetiver grass hedges on wave run-up"

A relationship has to be found between the wave height, hedge characteristics and run-up level. The following questions related to this objective will need to be answered:

- What is the hydraulic resistance of Vetiver grass hedges
- What is the effect of different planting configurations on the reduction of the wave run-up?

In this chapter the results are compared to the objective stated. The conclusions are followed by recommendations for further research.

6.1 Conclusions

Tests show that a plate with slits can represent a hedge of Vetiver grass in run-up. A plate with slits and a blocking factor of 75% has about the same drag factor as a Vetiver grass hedge. .

The theory that the reduction of run-up only depends on the blocking factor could not be rejected. Small scale run-up tests show that a plate with 75% gives a reduction of at least 55% of the overtopping volumes. The run-up height is reduced by more than 20%.

A more open hedge with a blocking factor of 60% blocking reduces the overtopping volume by 40%. The run-up height is also reduced by more than 20%.

The reduction of the run-up height depends amongst others on the breaker parameter. Since the overtopping volume is less dependent on the breaker parameter, this should be used for further calculation.

For dikes with a low average overtopping discharge allowed (0.1 l/m/ second), two hedges placed on the top of the slope have the most effect. It can reduce the height of the dike by about 0.9 m. With an average discharge of over 1 l/m/s the effect of a Vetiver hedge is considerably less.

6.2 Recommendations

In some calculations in this report assumptions have been made. These assumptions have to be verified. This report can be the part of a larger research on Vetiver grass. A possible lay-out of this research can be seen below:

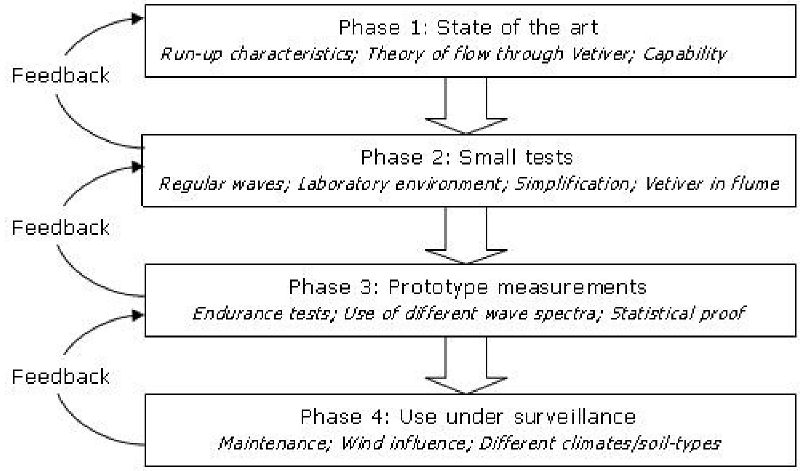


Table 5.4 Possible lay-out of the research on Vetiver grass

The research in this report is part of phase one and two. Phase two is not finished and can be extended by the following research:

- Assumptions about the failure mechanisms in chapter 5 are very crucial for the reduction of the costs. The failure of Vetiver hedges can be tested in flumes where the forces on Vetiver during wave run-up are simulated by a hydraulic jump. Layer thickness celerity and bore wedge angle need to be the same as with run-up.
- The roughness factor of Vetiver grass hedges when overloaded was assumed to be 0.95. It is very well possible that the reduction may be even larger. This needs to be tested.

For Phase three the following can be said:

- Scaling of the hedges is good possible. Large scale tests in laboratory flumes tend to be very expensive. However, they need to be done to get statistical prove of the reduction of a Vetiver hedge in run-up. This is necessary because design rules have to be created and should contain statistical prove.

Phase four can also be started:

- It is yet not possible to use Vetiver grass in design rules because of the reason stated above. Planting Vetiver on a dike for different testing purposes can be done, however without relying on it. If no other options are possible to strengthen a dike Vetiver grass can yet be a last option.

Next to more research on Vetiver grass in the run-up zone other investigation should also be started. Research on the use of Vetiver grass for reinforcement of the inner slope of a dike. This would allow more overtopping and would also lower the dike. If good results with Vetiver on the inner slope are acquired the use of Vetiver on the outer slope could be abandoned.

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APPENDIX 1 PROCESSING OF THE MEASUREMENTS

The results presented in chapter 4 are derived from the measurements as described in this appendix. The processing is done for every test. In this appendix example B6 is used which means the hedge named B and wave climate 6 (see chapter 3). The measurements are described below:

Table I.I Measured values during run-up tests.

Measured	Value
Water level measurement (h)	50 mm
Measured run-up height with hedge (Ru)	378 mm
Measured run-up height smooth slope	427 mm
Height hedge at the crest side (z)	258 mm

Table I.II Calculated quantities

Quantity	Value
(Ru-z)	120 mm
(Ru_{smooth} - z)	169 mm
Reduction (Ru_{smooth} - Ru)/(Ru_{smooth} - z)	0.29

After the values are known the error-margins are calculated in the table below. The values for the measurement errors are determined in chapter 4.

Table I.III Determining the error margins for the run-up tests

Error margin		Value
$(Ru\text{-}z)_{max}$	120+3 mm	123 mm
$(Ru\text{-}z)_{min}$	120-3 mm	117 mm
$(Ru_{smooth}\text{-}z)_{max}$	169+3 mm	172 mm
$(Ru_{smooth}\text{-}z)_{min}$	169-3 mm	166 mm
Reduction max	(172-117)/172	0.32
Reduction min	(166-123)/166	0.26

For the calculation of the overtopping the following procedure is used:

Table I.IV Measurements and calculated quantities of the overtopping test

Measured	Value
Overtopping Volume	136 liters
Number of waves	30
Average volume	4.53 liters
Volume per meter width (V*1/0.8)	5662 mm^2

The water level is recorded with the wave gauge. The recorded values are shown in the graph below. On the right the water level is averaged over the wave period.

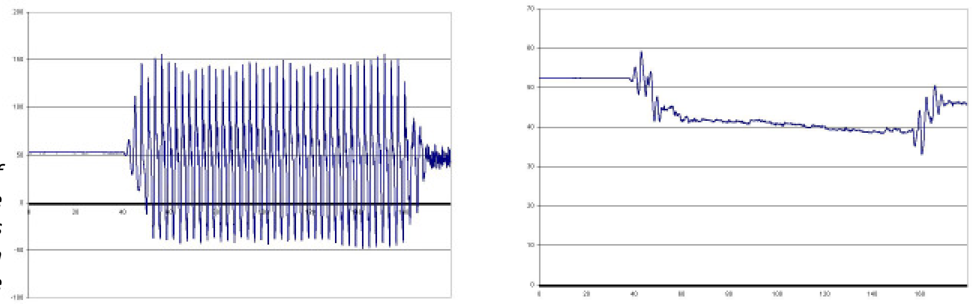

Figure I.I The output of the wave gauge. The negative waterlevel is because of the conversion to the same levels of the point gauge.

Table I.V Start and end value of the waterlevel

Measured	Value
h_{start}	52.3 mm.
h_{end}	46.6 mm.

The overtopping volume is a function of $(R_u\text{-}z)^2$. Therefore not the average water level was taken as reference level, but the water level at 1/3 of the difference. This is the water level at which the average run-up volume measured is taking place. From this reference water level the volume has to be calculated to 50 mm.

Table I.VI Determining the reference waterlevel

Quantity	Value
$H_{reference}$	50.4 mm.
Waterlevel difference	-0.44 mm.

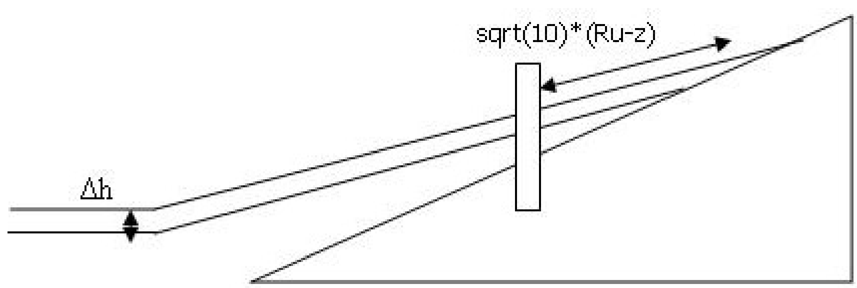

Table I.VII Table with the conversion of the average volume to the volume at h = 50 mm.

Quantity	Value
(Ru-z)	120 mm
Sqrt(10) * (Ru-z) * Δh	-162 mm^2
Overtopping volume at h=50 mm	5504 mm^2

The same is done for the overtopping for with the smooth slope. The overtopping for the smooth slope is 16403 mm^2. Now the error margins can be established the errors are described in chapter 4 and repeated below:

Table I.IIX Table with the measurement errors

Parameter	Error
Average run-up	3 mm
Overtopping volume	$3*10^{-3}$ m^3/number of waves
Water level	0.5 m

Now the maximum and minimum values can be calculated:

Quantity	Value
Overtopping Volume$_{min}$	133 liters
Number of waves	30
Average volume$_{min}$	4.43 liters
$h_{start\ max}$	52.8 mm.
$h_{end\ max}$	47.1 mm.
$H_{reference\ max}$	50.9 mm.
Waterlevel difference	-0.94 mm.
Volume per meter width (V*1/0.8)	5541 mm^2
(Ru-z)	123 mm
Sqrt(10) * (Ru-z) * Δh	-350.5 mm^2
Overtopping volume at h=50 mm	5195 mm^2

Table I.IX Determining the minimum overtopping volume

Quantity	Value
Overtopping Volume$_{min}$	139 liters
Number of waves	30
Average volume$_{min}$	4.63 liters
$h_{start\ max}$	51.8 mm.
$h_{end\ max}$	46.1 mm.
$H_{reference\ max}$	49.5 mm.
Waterlevel difference	+0.54 mm.
Volume per meter width (V*1/0.8)	5791 mm^2
(Ru-z)	117 mm
Sqrt(10) * (Ru-z) * Δh	+19 mm^2
Overtopping volume at h=50 mm	5811 mm^2

Table I.X Determining the maximum overtopping volume

The same procedure is used to calculate the values for a smooth slope

Parameter	Value
V	16403 mm^2
V_{max}	16749 mm^2
V_{min}	16055 mm^2

Table I.XI Overtopping values for the smooth slope

The maximum and minimum reduction are calculated below:

Parameter	Value
Reduction (16403-5504)/16403	0.66
Reduction$_{max}$ (16749-5195)/16749	0.69
Reduction$_{min}$ (16055-5811)/16055	0.64

Table I.XII Reduction of the overtopping and the error margins.