Preface

This document is the final report of my graduation research in order to fulfill the requirements for obtaining the Master of Science (Ir.) in Civil Engineering. This research has been carried out at Delft University of Technology (DUT), Faculty of Civil Engineering and Geosciences, Section of Hydraulic Engineering with specification in Coastal Engineering.

Executing a graduation research is an integral part of the education program at Civil Engineering and is regarded as the final work of the education in which the student shows his/her skills and knowledge obtained at the faculty. The interest of this research was the relationship between wave run-up, wave overtopping and Vetiver grass hedges. A major part of the study was experimental research in the wave flume of the Stevin III-laboratory at Delft University. This work was under supervised by a graduation committee, including:

Prof.dr.ir.M.J.F.Stive	Chairman DUT, Civil Engineering and Geosciences, Section of Hydraulic Engineering
Ir.H.J.Verhagen	DUT, Civil Engineering and Geosciences, Section of Hydraulic Engineering
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I would like to thank all members of the committee for the fruitful discussions during my research and for useful advices they gave concerning both the execution of the experiments and the interpretation of the results.
I also give my warm thanks to all staffs in Stevin III-laboratory and Botanical Garden who have helped me in various ways during the execution of the experiments.
Further, I would like to thank Hanoi Water Resources University and CICAT-Delft University, who gave me the opportunity to study here.
Next to that, I would like to thank my friends for their support and the great times we had together.
Finally, I give my special thanks to my family who always show such a tremendous patience during the time I studied in TUD. Without their enduring support, my graduation from TUD would certainly not have been possible.

Vu Minh Anh
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Abstract

In order to protect the area inside the coastal flood defend system, the height of a sea dike should be sufficient. Huge sea dikes with revetment protection which are made from traditional ''hard'' material such as block concretes or big rock has given good results. However they are quite costly to implement and materials are not always available. In order to reduce total cost, the traditional revetment could be replaced by cheaper materials. A combination between ''hard'' and ''soft'' materials is a good alternative solution.

In a number of tropical countries Vetiver grass (Vetiveria zizanioides) is well-known as bioengineering species in stabilizing inner slopes, reducing run-off and controlling soil loss. Recently, it has been planted on outer slope as sea dike protection. However the understanding of the processes and properties between waves and Vetiver grasses are still limited. In this research Vetiver grass is investigated as outer slope protection in order to reduce wave overtopping discharges. In consequence sea dike crest can be reduced. Beside it, this research also focuses on addressing the hydraulic effects of flow and Vetiver hedges. Physical model is conducted full-scale of Vetiver grass and wave parameters in front of the hedge. Mature Vetiver grasses are used for testing and wave is simulated by dam-break method.

Experimental results have shown that resistance between flow and Vetiver hedge depends on grass density. In this research, Manning's coefficient is used to describe the resistance; it varies with the changing of flow depth. One interested characteristic of Vetiver grasses showed in this experiment is that its ability to withstand flow of backwater which reaches depths up to nearly 0.4m. The reduction of wave overtopping of more than 60% is measured. The roughness coefficient of Vetiver grass that depends on grass density is found to be varying between 0.33 and 0.41.

An example with the use of Vetiver grass on a sea dike in Vietnam is worked out, by use of the results from the physical model. This example shows that a reduction of 0.5m of the crest height is feasible for upgrading the present sea dike in Nam Dinh, Vietnam. It corresponds with a reduction of 12,6% of the costs in case of two Vetiver hedges are planted on the outer slope. This case shows that Vetiver grass is a good solution for sea dike in order to reduce wave run-up on the outer slop and decreases the cost for upgrading sea dike.

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A	[m^2]	cross section, area
g	[m/s2]	acceleration due to gravity: g = 9.81 m/s^2
h	[m]	depth of flow
h_f	[m]	head loss
H_{m0}	[m]	significant wave height at toe of dike
H_s:	[m]	significant wave height at the toe of the wall
$R_{u2\%}$	[m]	2% wave run-up level above the still water line
ξ_{m0}	[-]	breaker parameter
γ_b	[-]	influence factor for a berm
γ_f	[-]	influence factor for roughness elements on slope
γ_β	[-]	influence factor for angled wave attack
R_h	[m]	hydraulic radius
n	[s/m$^{1/3}$]	the Manning roughness coefficient
u	[m/s]	the velocity of flow
S_f	[m/m]	the slope of energy line
Q_*	[-]	dimensionless discharge
R_*	[-]	dimensionless freeboard
Q	[m^3/s/m]	mean overtopping discharge rate per meter run of seawall
T_m	[s]	wave period at the toe of the wall
R_c	[m]	freeboard of the seawall
R	[-]	is the roughness coefficient
$c_{u2\%}$, $c_{q2\%}$	[-]	empirical coefficient
$h_{2\%}$	[m]	layer thickness exceeded by 2% of the coming waves
$q_{2\%}$	[m/s]	waves run-up discharge exceeded by 2% of incoming wave

1. Introduction

In this chapter first some background information and problems which are reduced when using Vetiver grasses are provided. The objectives of this research are defined later.

1.1. General background

Floods and storms pose risks to many densely populated parts of the world. Effective and efficient protection structures play an important role there. An example is a dike which is covered by grasses or combination between hard revetments and grasses. They cover a significant part of the dike surface like berm, crown and inner slope. Researches and experiments since the mid-eighties have shown that grass coverings had high quality in terms of erosion resistance, reduction of wave attack and slope stability. In the recent decades, since 1980's in specifically Vetiver grass has become popular for its potential use in stabilizing structures, decreasing run-off and flow attack. Nowadays Vetiver grass is applied in several areas China, Thailand, and Vietnam. Following two problems which can be reduced using Vetiver grass are described:

** Wave run-up and overtopping at outer slope*
For earth sea dikes, overtopping is one of the most damaging factors for the inner slope. Eventually a failure of the inner slope may lead a failure of the dike. Vetiver grass planted in the berm of outer slope may have a considerable effect in reducing wave run-up and wave overtopping. Using Vetiver grass is considered as an effective method which has low costs and is able to decrease wave overtopping.

** Erosion and sliding of the inner dike slope*
Erosion and sliding of inner slope can be result from overtopping. Water infiltrates into dike crest, inner slop and reduces the shear resistance of the soil. At the same time, flow forces occur vertically or horizontally at the slope. Supported by the decreasing shear strength the flow forces induce together with the mass forces, creeping deformations parallel to the slope. In order to increase the strength of soil Vetiver grass is planted as amour layer.
These problems with Vetiver grass like sea dike protection give considerable effects in practice. It stimulates the engineering community to take a closer look at the use of Vetiver thanks to its proven abilities to stabilizing soil, reducing flow speed significantly and protecting earth structures more effectively and at a lower cost than many other technologies. However the understanding of processes and properties did not yet supply in very detail. Therefore it is necessary to investigate phenomena around Vetiver grass if you want to understand clearly the physical processes.

Picture 1-1: Fifteen months old Vetiver on the outer batter of the sea dike in Go Cong province, Viet Nam

1.2. Objectives

This research will focus on the first problem mentioned above in section 1.1. Hence the main objectives for this thesis are defined as follows:

''The effectiveness of Vetiver grass to reduce wave overtopping''

The following objectives will be addressed:

- The hydraulics with Vetiver grasses.
- The interaction between flow velocity and flow depth under cases of Vetiver hedge in relation to the reduction of wave overtopping.
- Improving guidelines in designing sea dike dimensions.

1.3. Outline of the report

In the first chapter the problem description and objectives of research are involved. In chapter 2, the knowledge about wave overtopping, wave run-up and Vetiver grass will be illustrated. The following chapter deals with the description of experimental set-up which is use in physical test. Chapter 4 provides an analysis of the measured wave data. In chapter 5 an application of Vetiver hedge at outer slope will be given. Conclusions and recommendations for further study are ending this research in chapter 6.

2. Literature studies

Firstly in this chapter the theories of overtopping which are related to the objectives of this research are described (2.1). Secondly, discussion of Vetiver grass effects in reducing wave run-up and wave overtopping as well as the general information about Vetiver grass (2.3 &2.3) are described. From the knowledge of waves and Vetiver grass, the new ideals can be looked more detailed inside to show that: this phenomenon can be observed through testing in the lab.
In this research, the average overtopping discharge and two wave parameters: layer thickness and velocity are investigated. These parameters were not only chosen because of they are absent in many previous overtopping formulas but also because they are easily measured in laboratory.

2.1. Theoretical Background

Wave overtopping is one of the most important phenomena which could badly influences to constructions. For designing and maintenance works of river and coastal structures, the anticipation of wave overtopping are required. Several guidelines for wave overtopping calculation exist for sea dikes, rubble mound breakwaters or vertical breakwaters. They will be introduced in the following sections:

2.1.1. Wave overtopping formulas

2.1.2.1 Owen formula (1980)

Owen (1980) presents a method to calculate flow discharge of overtopping on an impermeable, smooth and rough bermed slope of seawall as a function of wave height, wave period, and the freeboard.

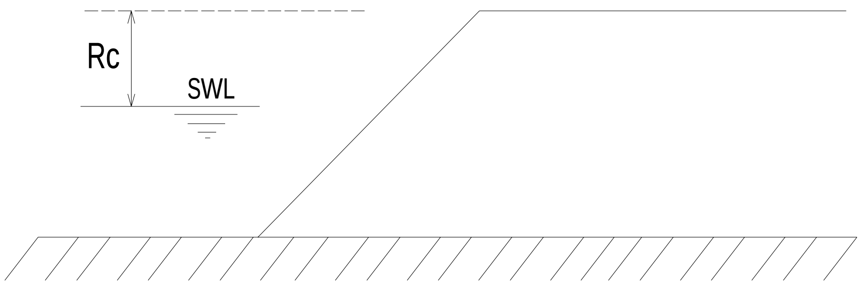


Figure 2-1: Smooth, impermeable, simply sloped seawall

In this method first the dimensionless discharge and freeboard are calculated:

$$Q_* = Q/(T_m.g.H_s) \tag{2-1}$$

$$R_* = R_c/(T_m(g.H_s)^{0.5}) \quad \text{valid for } 0.05 < R_* < 0.3 \tag{2-2}$$

Where:

Q_*:	dimensionless discharge	(-)
R_*:	dimensionless freeboard	(-)
Q:	mean overtopping discharge rate per meter run of seawall	($m^3/s/m$)
T_m:	wave period at the toe of the wall	(s)
g:	acceleration due to gravity	(m/s^2)
H_s:	significant wave height at the toe of the wall	(m)
R_c:	freeboard of the seawall (The height of the crest of the wall above still water level)	(m)

The dimensionless discharge, Q*, and freeboard, R*, are related to following equation where A and B are empirically derived coefficients which depend on the profile of the seawall for the slopes arranging from 1:1 to 1:5:

$$Q^* = A \exp(-B.R^*) \quad (2\text{-}3)$$

Owen extended his work apply for sloped and bermed to cover rough impermeable and rough permeable seawalls:

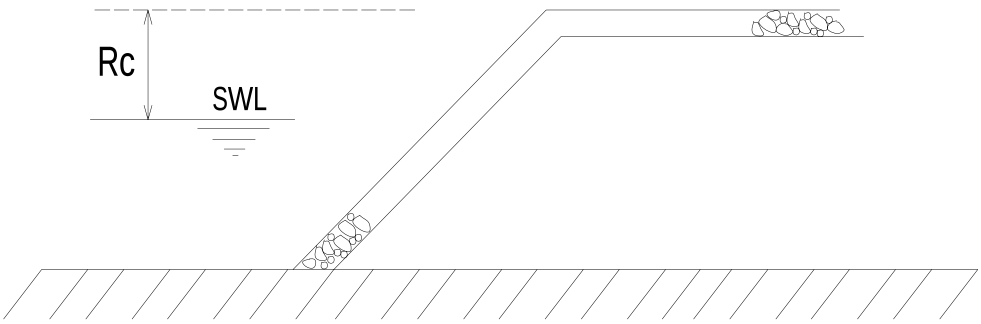


Figure 2-2: Armored seawall

$$Q_* = A\exp(-B.R_* / r) \quad (2\text{-}4)$$

$$R_* = R_c /(T_m (g.H_s)^{0.5}) \quad \text{valid for } 0.05 < R_* < 0.3 \quad (2\text{-}5)$$

$$Q = Q_*.T_m.g.H_s \quad (2\text{-}6)$$

Where: r: is the roughness coefficient (-)

This calculation is based on the relative run-up performance of alternative types of constructions. In order to reduce overtopping, a crest berm has been built to dissipate wave energy. Owen's equation does not take into account crest berm thus the discharges is overestimated.

2.1.2.2 Van der Meer formula (2002)

In the Technical Report Wave Run-up and Wave Overtopping at Dikes (2002), Van der Meer described two formulas to calculate the average waves overtopping with irregular waves. These formulas represent for breaking wave and non-breaking wave according to the following criteria of breaker parameter:

$$\xi_0 = \frac{\tan\alpha}{\sqrt{s_0}} \tag{2-7}$$

For $\xi_0.\gamma_b \leq 2$ wave breaking

$\xi_0.\gamma_b > 2$ wave non-breaking

Where:

ξ_0 :	breaker parameter	(-)
s_0:	wave steepness	(-)
α :	angle of slope	(^{o})
γ_b :	influence factor for the influences of the toe of the breakwater	(-)

In the following graph the dimensionless wave overtopping discharge is plotted on the vertical logarithmic axis for three different relative crest heights:

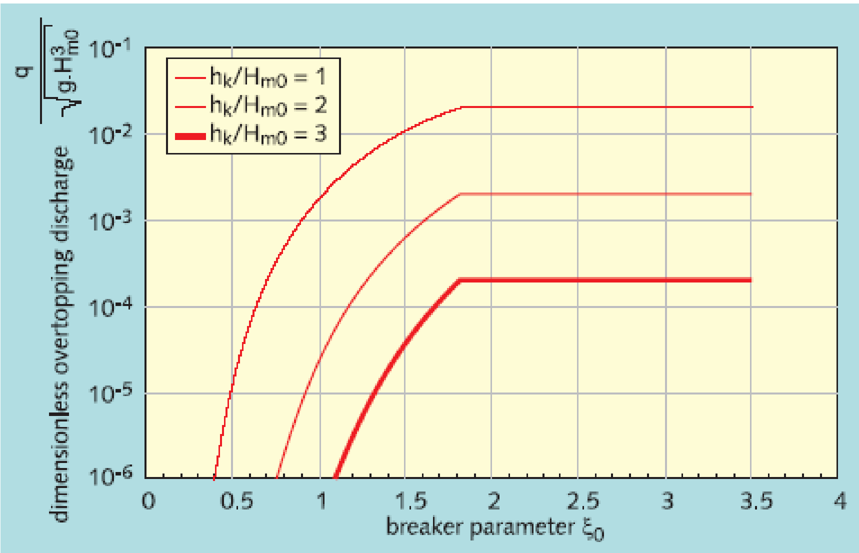


Figure 2-3: Breaker parameter of Van der Meer formula

The overtopping formulas are exponential functions:

$$\frac{Q}{\sqrt{g.H_{m0}^3}} = \frac{0,06}{\sqrt{\tan\alpha}}.\gamma_b.\xi_0.\exp\left(-4,7.\frac{R_c}{H_{m0}}.\frac{1}{\xi_0.\gamma_b.\gamma_f.\gamma_\beta}\right) \tag{2-8}$$

With the maximum of: $$\frac{Q}{\sqrt{g.H_{m0}^3}} = 0,2.\exp\left(-2,3.\frac{R_c}{H_{m0}}.\frac{1}{\gamma_f \gamma_\beta}\right) \tag{2-9}$$

Where:

q:	average wave overtopping discharge	(m^2/s)
g:	acceleration due to gravity	(m/s^2)
H_{m0};	significant wave height at toe of dike	(m)
ξ_0:	breaker parameter	(-)
s_0:	wave steepness	(-)
$T_{m-1.0}$:	spectral wave period at toe of dike	(s)
α:	angle of the slope	(-)
R_c	free crest height above still water line	(m)
γ_b	influence factor for a berm	(-)
γ_f	influence factor for roughness elements on slope	(-)
γ_β	influence factor for angled wave attack	(-)

All formulas above can be applied only for impermeable slopes such as concrete rough slopes. However the formula for wave overtopping with grasses protection on the outer slope has less information, especially for the case of Vetiver grass hedges.

2.1.2.3 Summary

For determining slope stability and designing dikes, average overtopping discharge is a useful parameter. Though many equations are found in literature studies Owen's equation and Van der Meer's equation are chosen for calculating overtopping discharge in this research for these following reasons:

- Firstly, these two formulas are usually used to design dike nowadays. Owen's formula was originally derived from dike design. In this equation the relationship between the dimensionless discharge parameter Q and the dimensionless crest freeboard parameter R is examined. Van der Meer described the average wave overtopping with irregular waves based on breaking waves and non-breaking waves.
- Secondly, in two formulas, especially in Van der Meer's formula, there are features relating to many interesting parameters in this thesis like wave overtopping discharge, coefficients of slope roughness, angles of the slope, etc. During this research, these coefficients can be verified and compared.

2.1.2. Wave Run-up formulas

In normal situations, wave run-up is one of the most important factors causing the overtopping. Reduction of wave run-up means that the total amount of wave overtopping volume over the crest of sea dike decreases. Hence reduction of wave run-up and reduction of wave overtopping discharge are linked together. The general wave run-up equation can be applied on dikes as following:

$$\frac{R_{u2\%}}{H_s} = 1,65.\gamma_h.\gamma_f.\gamma_\beta.\xi_0 \tag{2-10}$$

For large value of ξ_0 ~ 3, at that time wave run-up height can be calculated by the following formula:

$$\frac{R_{u2\%}}{H_s} = \frac{\gamma_f.\gamma_\beta.(4,3 \div 1,6)}{\sqrt{\xi_0}} \tag{2-11}$$

The Coastal Engineering Manual, which is based on run-up measurement acquired during irregular wave rock armor stability experiments, gives design guidance:

$$\frac{R_{u2\%}}{H_{m0}} = \begin{Bmatrix} 0,96.\xi_{m0} \\ 1,17.\left(\xi_{m0}\right)^{0.46} \end{Bmatrix} for \begin{Bmatrix} 1.0 < \xi_{m0} < 1.5 \\ \xi_{m0} > 1.5 \end{Bmatrix} \tag{2-12}$$

In which

H_{m0};	significant wave height at toe of dike	(m)
$R_{u2\%}$:	2% wave run-up level above the still water line	(m)
ξ_{m0}	breaker parameter	(-)
γ_b	influence factor for a berm	(-)
γ_f	influence factor for roughness elements on slope	(-)
γ_β	influence factor for angled wave attack	(-)

From the knowledge of interaction between wave run-up and Vetiver grass, the parameters which are useful for experiments will be derived. This study focus on all phenomena around Vetiver hedges, especially by two parameters: layer thickness and velocity of wave run-up which have direct effect on overtopping volume.

According to Schütrumpf (2001) there is no significant difference in the layer thickness and wave run-up velocity with or without overtopping in model tests. The relationship among run-up, layer thickness and velocity which can be defined as following:

$$u = c_u\sqrt{g(R_u - z)} \tag{2-13}$$

And

$$h = c_h(R_u - z) \tag{2-14}$$

Regular waves: From Schütrumpf (2002) for slope 1:3, 1:4 and 1:6, values found with c_u = 0.94, c_h = 0.284. The value c_h = 0.284 followed from Tautenhain (1981).

Random waves:

$$\frac{u_{2\%}}{\sqrt{g.H_s}} = c_{u2\%}\sqrt{\frac{R_{u2\%} - z}{H_s}} \tag{2-15}$$

$$\frac{h_{2\%}}{H_s} = c_{u2\%} \cdot \frac{R_{u2\%} - z}{H_s} \tag{2-16}$$

Schütrumpf (2002) found $c_{u2\%}$ = 1.37, $c_{h2\%}$ = 0.33. According to Van Gen (2003) those values were $c_{u2\%}$= 1.3, $c_{h2\%}$=0.15.

With:

R_u:	2% wave run-up level above the still water line	(m)
H_s:	Significant wave height	(m)
$c_{u2\%}$, $c_{q2\%}$:	empirical coefficient	(-)
$h_{2\%}$:	layer thickness exceeded by 2% of the coming waves	(m)
$q_{2\%}$:	waves run-up discharge exceeded by 2% of the incoming waves	(m^2/s)
ξ_0 :	is breaker parameter	(-)
γ_b	influence factor for a berm	(-)
γ_f	influence factor for roughness elements on slope	(-)
γ_β	influence factor for angled wave attack	(-)

In an effort to avoid wave overtopping, the crest level of sea dike can be increased based on the design water level and the design waves. If wave overtopping cannot be avoided, dikes have to be designed in such a way that overtopping water has no consequences on the stability of the dike crest. Therefore, wave overtopping must be described by the associated overtopping flow velocities and layer thicknesses which are responsible for erosion and infiltration, but not only by average overtopping rates. Hence, in this study the waves overtopping parameters are translated into wave run-up flow which is based on wave run-up flow velocities and layer thicknesses is used instead of average overtopping rates.

2.2. Vetiver grass

Vetiver grass (Vetiveria zizanioides) is well-known for the important features that it occupies minimal space and virtually non competitive with adjacent crops. Vetiver grass is also known as grass for roads stabilization, railroad embankments, river banks, canals, water management, etc. It can grow in many areas with various temperatures, including warmer areas like Japan, China, New Zealand, Australia, France, Argentina, Chile, the United States and Canada.

Picture 2-1: Vetiver distribution in the world

The Vietnamese name of Vetiver grass is *cỏ Hương bài, cỏ Hương lau*, the Latin name is Vetiver zizanioides or Khus Khus (Urdu/Hindi), Secate violetta (Spanish), Xieng Geng Sao (Chinese). There are 12 different varieties of the grass, three famous grasses which are used are: V.zizanioides in Asia, V.nigratana in Southern Africa, and V.nemoralis in South East Asia.

Characteristics of Vetiver

Vetiver grass is a special coarse perennial grass found in the tropics of the Europeans, Asians and Africans that belonged to the tribe Andropogoneae. Vetiver Zizanioides has proven to be ideal for soil and moisture conservation.

Picture 2-2: Leaf and root of vetiver

The physical attributes of the grass plant which determine the effectiveness of grass for protection are:

1. Length and stiffness of sward
2. Surface area of grass leafs
3. Strength and depth of root structure
4. Density of rhizomes, stolons and surface root structure

The following information gives an overview picture of Vetiver grass and its effectiveness for sea dike protection:

Culm: Vertiver grass is a prolific tiller growing naturally in clumps with thin, long, and erect leaves. The Vetiver clumps may grow densely tufted in a big cluster or scattering over the nearby space.

Leaf: Vetiver leaves are narrow, long and coarse. It also has stiff erect stems that can grow up to 1.5m high after 2-3 months, forming a dense hedgerow to effectively slow down surface runoff. It can reduce 60-73% runoff and trap 90-98% sediments (Kon and Lim, 1991; Xia et al., 1996).

Two characteristics of Vetiver grass above are useful for protecting the surface area under conditions of waves, currents, wind, etc.

Roots: Vetiver roots are the most useful part. Different from almost other grasses, the roots of Vetiver grass penetrate vertically, they are able to withstand tunneling and cracking of the soil. It has a vigorous and massive root system that can penetrate 5cm thick layer of asphalt concrete (Henchaovanich, 1998). The average root strength is 75 MPa and roots will improve shear strength of soil by 30÷40%

For the purpose of stabilizing slopes this character is highly appreciated, it can combine the soil and reduce significant erosion.

One of remarkable point when applying Vetiver grass on the outer slope of sea dikes is that the environment is saline. Vetiver grasses are inundated and submerged for several hours by the last high tide. Many measurements in the laboratory and fieldwork showed that Vetiver grass can survive under high saline environment, only the leaf burned, but the plants kept growing (Paul Truong, 1992). The results also indicated that more mature Vetiver plants can tolerate and survive in highly saline condition and inundation much better than younger plants. This is the reason why the Vetiver grass should be planted when the storm season is over.

2.3. Application of Vetiver grass

In the previous parts all equations for wave overtopping were applied to concrete, pitch stone or rough slopes and the parameters which can reduce wave overtopping were mention. Some studied for wave run-up and wave overtopping include the effect of short grass. However there is lack of information about the effect of long and stiff grass hedges. In this research Vetiver grass is planted in the berm of the outer slope in order to investigate their influence on wave overtopping. There are several small topics which are provided to support to that objective: the hydraulic of wave run-up flow in the slope through the Vetiver grass hedges, the characteristics of waves in front and behind grass hedges, and the effect of grass on flow characteristics will be taken into account.

2.3.1. The relationship with wave run-up and Vetiver hedges

In literature little information is available on the hydraulics of flow through Vetiver grass hedges, especially in relation with different slopes. Studies have been focused on the ability of Vetiver grass hedges in order to trap sediment in flows (Mayer *et al.,* 1995), the hydraulics characteristics of discrete hedge (Dalton, 1997) or the physical properties of the plant (Dunn and Dabney, 1996). The effect of the slope on the relationship between wave run-up and Vetiver grass hedges has not been properly considered.

In this study the characteristic of flow through Vetiver grass hedges on slope are investigated in order to reduce the flow velocity, water depth and increase energy loss. As a consequence the total overtopping will decrease. Studies Dean (1978, 1979), Knutson (1982, 1988), Moeller et al. (1996, 1999, 2002), and Hansen (2002) have shown that vegetable may damp wave energy. The wave height and wave steepness of incoming waves are reduced by the higher resistance of grasses. It means that wave energy is lost after flowing through the vegetations. Vetiver hedges provide high resistance to flow. As a result it has a large impact on flow of the water.

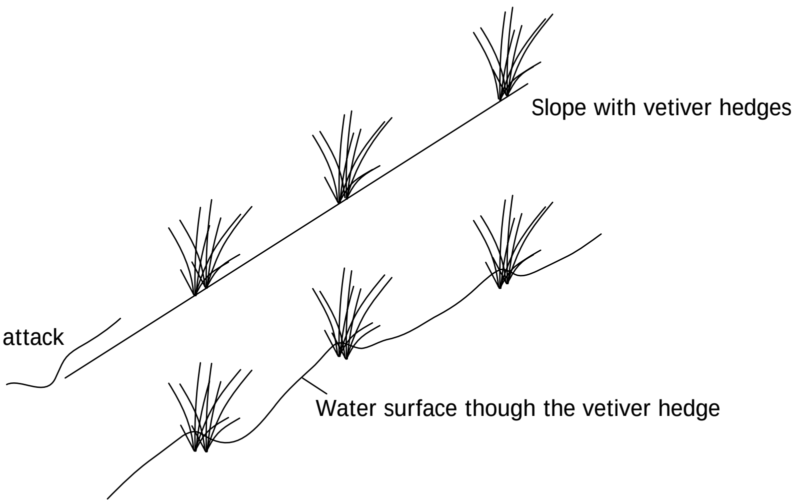


Figure 2-4: Slope with Vetiver grass hedge

In order to deduce wave run-up and wave overtopping, Vetiver hedges are grew at berm or directly at outer slope. It dissipates wave energy and protects the slope from erosion; the berms are constructed on the upstream slope at the mean sea level. Water surface though the Vetiver hedge describes in figure (2-4).The energy loss though the hedge can be calculated by Dalton formula as following:

$$y_1 + \frac{u_1^2}{2.g} - y_2 - \frac{u_2^2}{2.g} = \Delta\varsigma \qquad (2\text{-}17)$$

With

y_1:	water level in front of the hedge	(m)
y_2:	water level behind the hedge	(m)
u_1:	flow velocity in front of the hedge	(m/s)
u_2:	flow velocity behind the hedge	(m/s)
$\Delta\varsigma$:	head loss difference	(m)

2.3.2. Flow through the Vetiver hedge

For non-breaking waves, wave motion is essentially irrotational, except within a thin boundary layer. Therefore, the wave evolution can be described as potential flow theory in a short time. In areas where the flow occurs through the vegetation, the characteristics of the flow are mainly determined by types and density of vegetation as well as the depth and velocity of the flow. Estimating flow resistance of vegetation and the impact of vegetation to hydraulic performance may have a significant effect on the conveyance of flow (Järvelä, 2002).

2.3.2.1. Vetiver hedge resistance

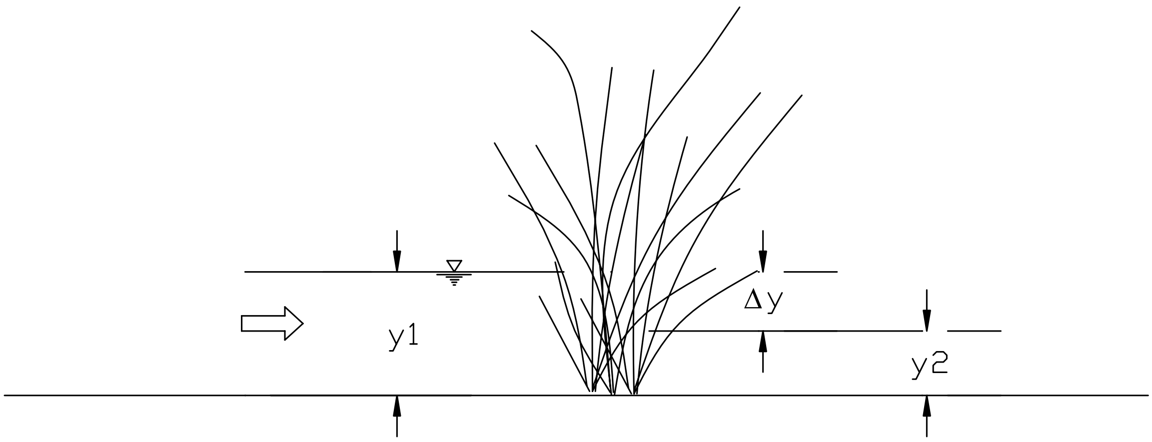


Figure 2-5: Water through Vetiver hedge

Flow resistance describes influences of friction to the flow. Magnitude of the resistance can be given as a resistance coefficient, such as the Manning's n coefficient. This coefficient are described the relationship between flow velocity and flow depth in the river. Resistance is commonly represented by parameters such as Manning's roughness coefficient (n), Chezy's resistance factor (C). Partryk and Bosmajian (1975) developed a quantitative procedure for predicting the Manning's value for non-submerged vegetation. The analytical result showed that the n value increased as depth if the vegetation density remained relatively constant with the resistance. Chiew and Tan (1992) published their field observation on the resistance of non-submerged grass to water flow. Their research showed the flow resistance was independent of water depth. Dabney et al. (2003) found a relationship between discharge and the back water depth by using Manning resistance. In his research, the resistance factor of various grasses, include Vetiver, have determined with the related Manning factor. The different results above are perhaps caused due to the different density and kinds of non-submerged vegetation in their researches. It also means that the effect of non-submerged vegetation of flow resistance is not clear yet and needs to further studies.

Manning formula has become the most widely used of all uniform-flow formulas for flow in natural channel (Chow, 1959). Owing to its simplicity of form and to satisfactory result to practical applications, it can be defined as:

$$u_1 = \frac{1}{n}.\sqrt{S_f}.R_h^{\frac{2}{3}} \tag{2-18}$$

In which:

R_h:	hydraulic radius	(m)
n:	the Manning roughness coefficient	($s/m^{1/3}$)
u_1:	the velocity of flow up stream	(m/s)
S_f:	the slope of energy line	(-)

Einstein (Smith et al. 1990) did the test in wave flume, the channel had the same configure at different profiles. He found that the roughness of the grass wall is negligible compared to the roughness of bottom. For the calculation, the water depth y was used instead of hydraulic radius R. Rewriting formula (2-18) yields:

$$u_1 = \frac{1}{n}.\sqrt{S_f}.h^{\frac{2}{3}} \tag{2}$$

2.3.2.2. Vetiver Hedge failure

The useful characteristics of Vetiver grass which are different from other grasses are resistance against bending or breaking, ability to detain (pond) significant amount of water, and its durability during prolonged flows. The mechanical behavior of foliage leaves in response to static and dynamic mechanism. When loading under their own weight or subjected to externally applied force, like wind or flow, petioles simultaneously bend and twist. They operate as cantilevered beams. The first phase of bending is an elastic process, if the load is higher, the leaves will break down.

Dabney et al., 1996 did tests with different grasses which had to withstand various rates of flow. Also attention was paid the resistance against bending or breaking of the stem. Vetiver grass has the greatest ability to withstand flow of backwater which reaches depths up to nearly 0.4m. The backwater depth was found to be nearly independent from flume slope. An increase of the depth depends on the hedge which was related to this grass characteristic like stem diameter, stem density, and hedge width, the existing grass leaf characteristics and the Reynolds flow.

Dalton et al. (1996) carried out tests at the University of Southern Queensland, Australia. He showed that Vetiver planted in row made flow run slowly through the hedge. The flow with their thick hedge and stiff stems can grow up to 0.6m high. Hydraulic characteristics of Vetiver hedges under deep flows were showed as:

- The flow of water through a hedge can be described by a simple equation relating discharge to the upstream and downstream depths of the hedge, with upwards is of 90% of the variation in discharge which was described by the equation (Dalton et al. 1966).

- To determine the hydraulic characteristics of Vetiver hedge, three rows of Vetiver were established in an outdoor flume. Mature hedges of Vetiver can stand up in flow which has the height of 0.8m and substantially reduce flow height by 0.3m

When applying Vetiver grass on a slope, the hedge resistance is still an unknown coefficient because it is a relatively new surface protection material. In this research, this coefficient is one of interesting factors which makes clarifies its impact on the flow water depth and the flow velocity.

Hedge strength, the ability to remain erect, is related to **its stem density** and **its individual stem strength**. Meyer at al. (1995) and Dabney, et al. (1995) found that both ***stem moment of inertia*** and ***modulus of elasticity*** were important resisting characteristics in keeping hedge from failure. In general, modulus of elasticity increases along with stem age until maturity. In comparison with wood, a stem's elastic limit was reached when it defected approximately 10% at a hedge of 150mm. Dabney (1996) found that modulus of elasticity of stems of several grasses increased with stem age. Vetiver grass barriers develop strength by the higher density stem whereas switch grass barriers are strong because of high modulus of elasticity, similar to that of oak and of their intact mature stems.

2.3.3. Summary

In this chapter the basic of wave run-up, wave overtopping and aspects of Vetiver hedge related to overtopping flow have been given. The formulas are widely used for deigning structure but mainly for hard structures. Therefore, some aspects of Vetiver grass related to overtopping flow are still in questions:

- What is the influence between Vetiver hedges and run-up flow on outer slope, compared with different materials?
- How is the process of run-up flow with the presence of Vetiver hedges?
- What are the dominate properties and characteristics of Vetiver grass on overtopping?

In order to answer these above questions, a physical model have been setup and tested in this study. The next chapter experiment setup will represent in more details.

3. Experiments

3.1. General information

For an understanding the processes and properties of the wave flow on the outer slope of the sea dike with the Vetiver hedges, physical modeling is indispensable. Use of a numerical model in this stage would be too much influenced by the assumptions on the governing hydraulic processes between Vetiver grasses and wave flow. Physical testing will lead to more insight in the physical processes. Therefore, physical tests have been chosen. Former researches were concentrated on the interaction between wave celerity and wave run-up height with the amour layer of concrete, rock or submerged vegetations (see chapter 2). In that approaches, the hydraulic processes of wave to stiff grass like Vetiver grass still lack of information.

Auke Algera (2006) had a research at Delft university, namely '' Run-up reduction through Vetiver grass''. In that research Vetiver grass was used to reduce wave run-up, however Vetiver grass hedges was modeled as vertical plates with vertical slit. All characteristics of Vetiver hedge and phenomena around it were assumed. It is necessary to understand the processes and reaction between real Vetiver grass and flow run-up, and between Vetivel grass and run-up overtopping if applying Vetiver grass on outer slope as sea dike protection. Hence in this study real Vetiver grass is used for physical experiments. Normally in the laboratory scale physical model of hydraulic structures and hydraulic boundary conditions are used. In scale model hydraulic structures, boundary conditions like wind, wave, tidal, and Vetiver grass must be taken with the same scale. However up to now scale factors for Vetiver characteristics like stem moment of inertia, modulus of elasticity, etc, have not been provided. The defining characteristics of Vetiver grass in a scale model have no foundation to be trusted. Avoiding these limitations real Vetiver grass is used for experiments, boundary conditions of flow depths, flow velocities in front of Vetiver hedge are taken as the real situation. The boundary data are used with reference from Algera's (2006) experiments.

A different approach in this research which is described in the following sections is the impermanent wave flow. The wave itself is considered a limited knowledge and it is simulated by dam-break phenomenon. When waves were generated by using dam-break method, wave parameters in front of Vetiver hedge change easily. Wave comes to the coast the bore propagation is somehow similar to a dam break wave (Vischer and Hager 1998, Chanson et al. 2000a). If only the wave run-up process and maximum wave run-up celerity are taken into account the non-breaking wave can be simplified as a propagating bore, which is analogous to a shock wave. As explain in section (2.3.2) in a short time, dam-break wave can be considered as flow in small distance. The wave maker which can produce the process as dam-break has been designed.

These above reasons are supplied for setting up the physical model. The interesting part is only around the Vetiver hedge. Vetiver grasses and wave parameters just in front of grass are consider as full scale. It requires the method that can simulate wave component before the grass easily and similar with real situation. Thus in this model, the wave conditions only in front of Vetiver hedge are taken into account as full scale. The parameters which are interested in this research and will be measured in this experiment are:

* Just in front of Vetiver hedge
    - Flow depth
    - Flow velocity
    - The time duration

* Behind Vetiver hedge
    - Flow depth
    - Flow velocity
    - Average energy head loss through Vetiver hedge
    - Average value of wave topping through Vetiver hedge

* Characteristics of Vetiver hedge
    - Density of hedge
    - Vetiver resistance.

3.2. Experimental set-up

In the first part of this chapter, general information about experiment is given. The next parts include experimental set up, required equipments and pre-test. A part of experimental processes is introduced in the last part.

3.2.1. General descriptions

The tests were conducted in a 15m long and 0.4m wide rectangular glass-walled flume with the depth of 0.4m in the Fluid Mechanic Laboratory at the Faculty of Civil Engineering and Geosciences, DUT. The dimensions of the flume determine the boundary conditions for all components which needed to set up in this experiment. The experimental set-up consists of the following components as in figure (3-1).

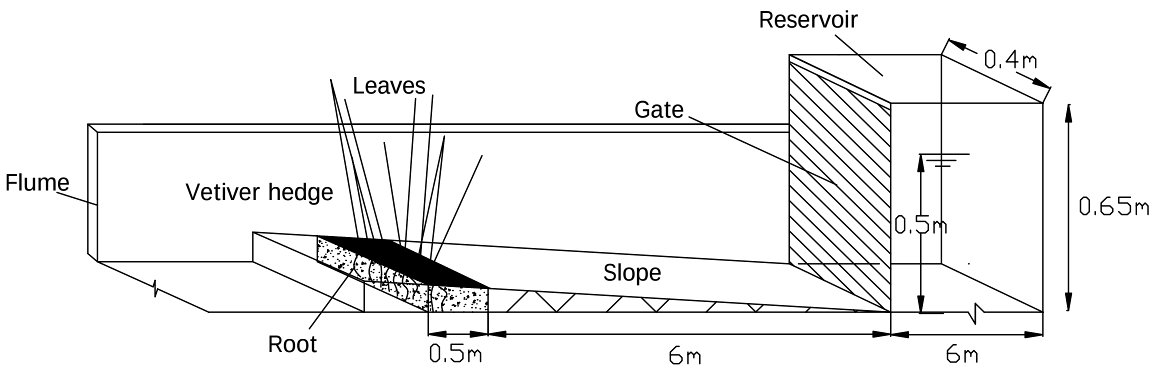
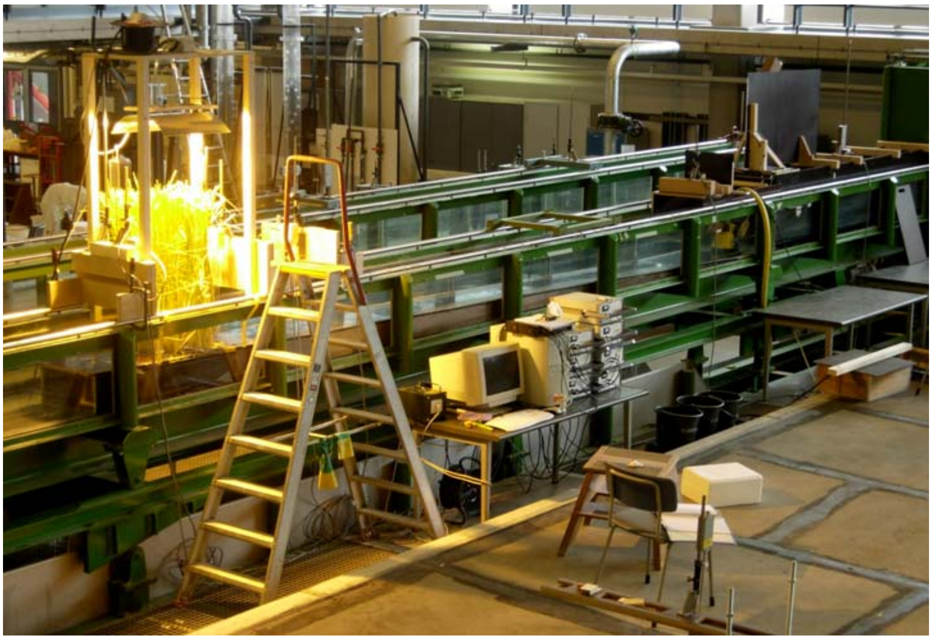


Figure 3-1: Sketch of experimental set-up

All the components of the experiment are based on the glass-walled structures; some additional parts have been built:

- The slope is constructed as a simple berm slope using ply wood, with the slope angle of 1:30 and the total length of 6m.
- A hedge of Vetiver grass is planted at the upper end of the slope with the length of 0.5m.
- The reservoir is constructed using wood on one side of the flume. Water will be supplied into the reservoir by the water system which is available in the laboratory. The reservoir is used for wave making by opening the vertical hinge gate.
- An opening mechanism is setup to control the opening of gate which is made from thickness wood and steal.
- An artificial light source is put above the position of Vetiver grass in the experimental flume in order to imitate natural conditions for grass alive during the tests.



Picture 3-1: Experimental facilities in laboratory of fluid mechanic

3.2.2. Wave maker

There are several options for generating waves:

- By using a chute
- By using a water tank
- By using a small reservoir

In the first option, wave is simulated with the chute which runs from the pipe bend. By this way the water velocity could be generated too high in comparison with the dimension of the flume and Vetiver hedge which has chosen. Thus, this option is not selected.

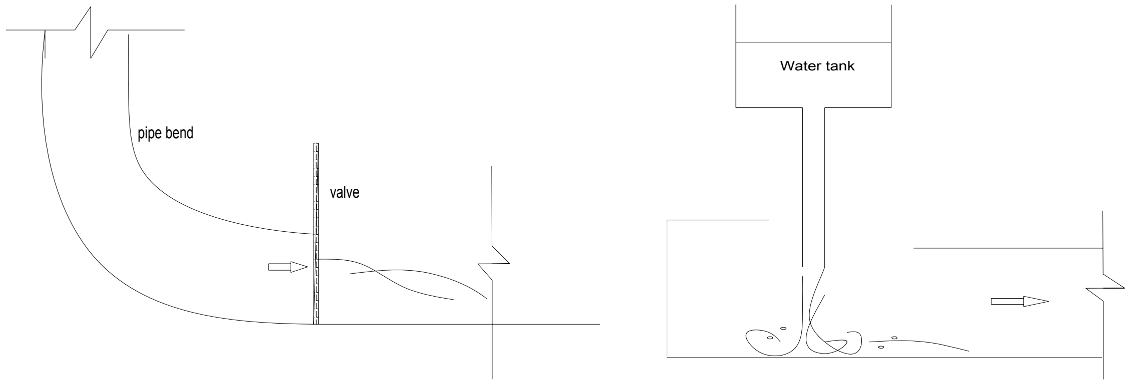


Figure 3-2: The first and second concept of wave maker

In the second option, a surge waves can be generated by downstream of the free-falling nappe. A dominant wave characteristic of an advancing bore is its highly initial momentum. Hence the wave front usually travels fast with high momentum. Because in this study the normal wave conditions are considered so this option is not choosen.

In the last option, the water pressure in a small reservoir is used to generate wave velocity by opening the vertical hinge gate. The wave characteristics can be controlled easily by changing reservoir configuration, the water level inside the reservoir or varying the gate opening. That is the main reason for selecting this concept. The wave maker includes two main components the reservoir and the gate of the reservoir. The dimensions and the design of these components will be presented in the following sections.

3.2.3. The Reservoir

The reservoir dimensions have much influence on the wave characteristics (wave height, wave celerity). The parameters which play an important role in the experiments are the volume of reservoir, the water level inside reservoir and the wave condition in front of the Vetiver hedge. These decide the characteristics of the dam-break wave behind the gate and the wave parameter in front of Vetiver hedges. In this research the solitary plunge wave is investigated. In order to control the magnitudes of the wave from gate, the reservoir is designed base on:

- The wave parameters just in front of Vetiver grass which are taken in reference with the values of Algera's experiments (Wave celerity of around 2m/s, wave height of around 0.13m), see Auke Algera (2006), chapter 3.
- The theory of dam break on dry channel (appendix A) can be applied to calculate the wave celerity and wave depth which can be happened, corresponding to the water level inside the wave maker.

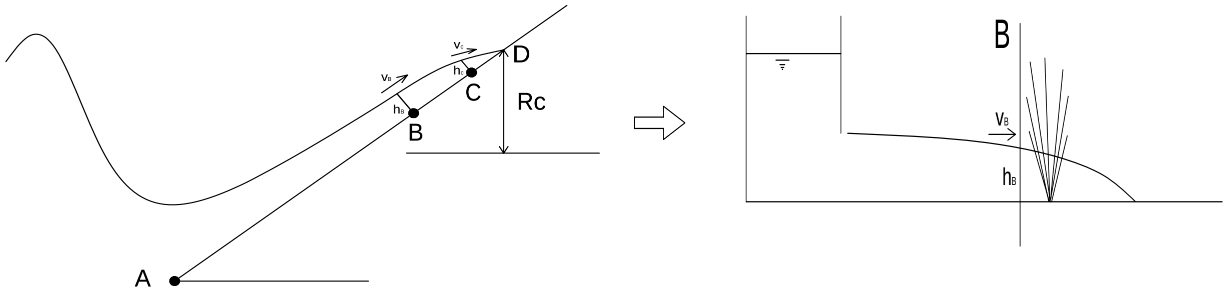


Figure 3-3: Simulating wave run-up by the dam break method

However, the wave parameters are sensitive with variation of water level in the reservoir and the time for gate opening so the wave should be calibrated by doing pre-tests. The water level, the configuration of reservoir or the gap of space for gate opening can be adjusted adequately to make different sets of wave parameters.

The following steps are applied to calculate reservoir dimension:

- Assume the variation of wave height and wave celerity at the position in front of Vetiver hedge.
- Estimate the water level inside the reservoir which can produce the same value wave height and wave celerity in front of Vetiver hedge.
- Sketch profile of the dam-break wave with initial water level equals to the supposed the water level inside the reservoir.
- Check the wave parameters before Vetiver hedge which are estimated with the dam-break theory and the supposed ones. If they are equal, the supposed initial the water lever inside the reservoir are taken to design and to do the experiments.
- From the profile of the dam-break wave which has maximum wave parameters, estimate the required total water volume.
- Design the volume of the reservoir bases on required water volume.

In figure (3-4) the sketch gives the dam-break wave progress at one point with the maximum water level inside reservoir of 50cm and wave parameters in front of Veitver hedge of u=2m/s and a wave height of 13cm. Because of the dam-break traveling progress, the wave height and the wave celerity change with time. By sketching the wave profile and dividing it into many small parts, the total water volume can be calculated by the trapezoidal method.

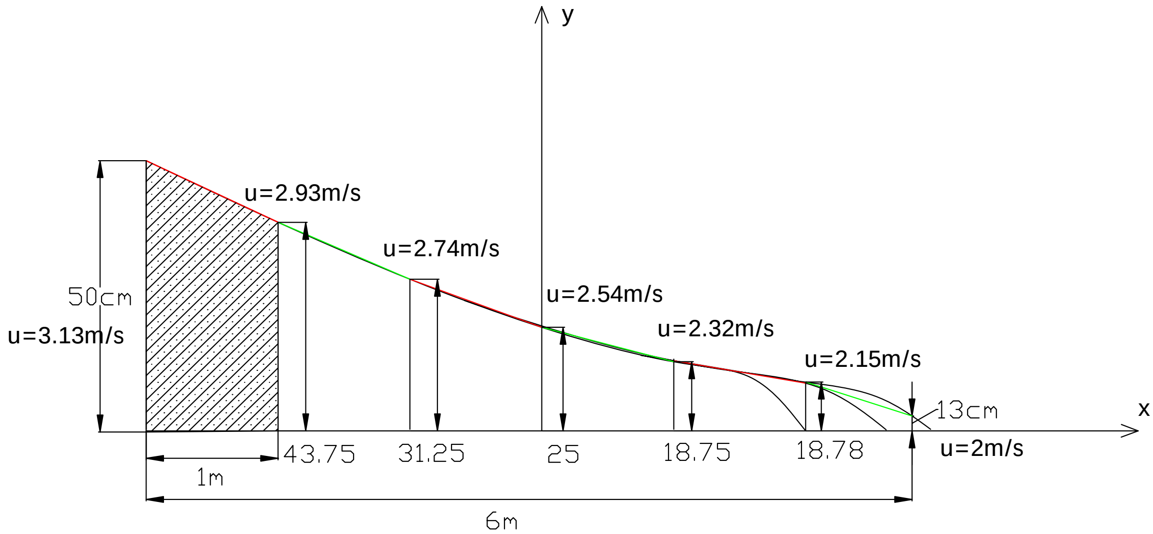
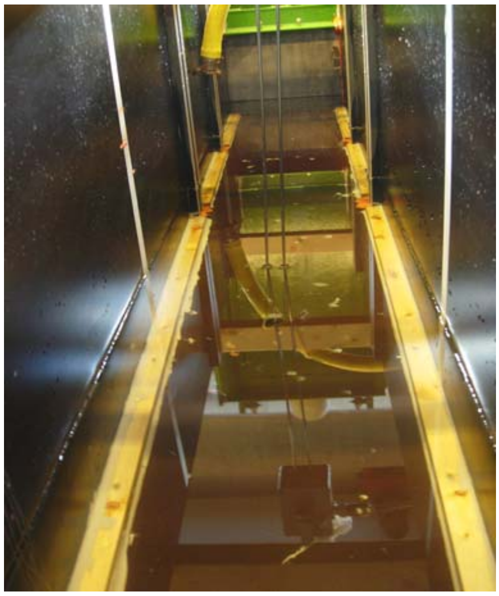


Figure 3-4: Dam-break wave profile

The calculation is done for the length of 6m (from the gate to the end of slope) and flume width of 0.4m. The result the total volume of the designed reservoir is between 400-500l. In the final design the reservoir has the dimensions of Length- Height- Width: L x H x B = 3 x 0.65 x 0.4 m^3.

This calculation is considered for the largest dimension so various sets of wave height and wave celerity can be generated easily. The height of the reservoir is designed large enough to produce required wave parameter. Finally the gate of the reservoir is constructed with the opening space higher than 50cm. The gate is made from thickness wood and steal.

Picture 3-2: Outside and inside of the reservoir

3.2.4. The Vetiver hedge

Vetiver grass used in this research was collected from Botanical garden, Delft University of Technology. It was planted in a greenhouse under the temperature of approximate 20 degrees Celcius, and the humidity of 85%. This condition is similar with the natural condition in Thailand and Vietnam. The Vetiver grass which used during the tests had 6 months aged and the average height was 1.5m. Each mature cluster of Vetiver grass includes 12-15 culms with the average diameter of 3cm. The leaves of the Vetiver were cut at about 50cm on the top and the root length was only 20cm.

Picture 3-3: Vetiver grass in greenhouse

The Vetiver grass is planted in a process as follows:

- Firstly, the Vetiver grass was planted in a container which has the same size as the required size in the test Length- Depth- Width: 0.5-0.2-0.4m, at least 3 weeks before the test with the highest density is 530 stem per m^2.
- Secondly, the Vetiver grass was collected after 3 weeks growing in the container in order to be sure that its root already combined. Then they were put into the position in the flume.

In these experiments three different densities of the Vetiver grass have been used. A series of tests are conducted with different stem densities. Firstly the density is 530 stems per square meter, after that density reduces a half to the value of 265 stem stems per square meter. Lastly the density has only 160 stems per square meter. These densities value are equivalent to the total munber of stems for testing are 106, 53, 32 respectively (table 3-1). The Reduction of the stem density is done by cutting off randomly each stem, but trying to maintain the regular distribution of the stems.

Total of culms in the experiments	Grass density (stem per m^2)
106	530
53	265
32	160

Table 3-1: Different grass densities in the experiments

3.2.5. Measuring equipment

Different pressure transducers are used to record water depth and to measure flow velocity along the flume, especially in the area around the Vetiver grass. To record the data, a PC is connected to the transducers.

The water layer thickness is measured along the flume by using a wave Gauge Height Meter (GHM). The locations of the gauges (GW) are indicated in figure (3.5) as follows:

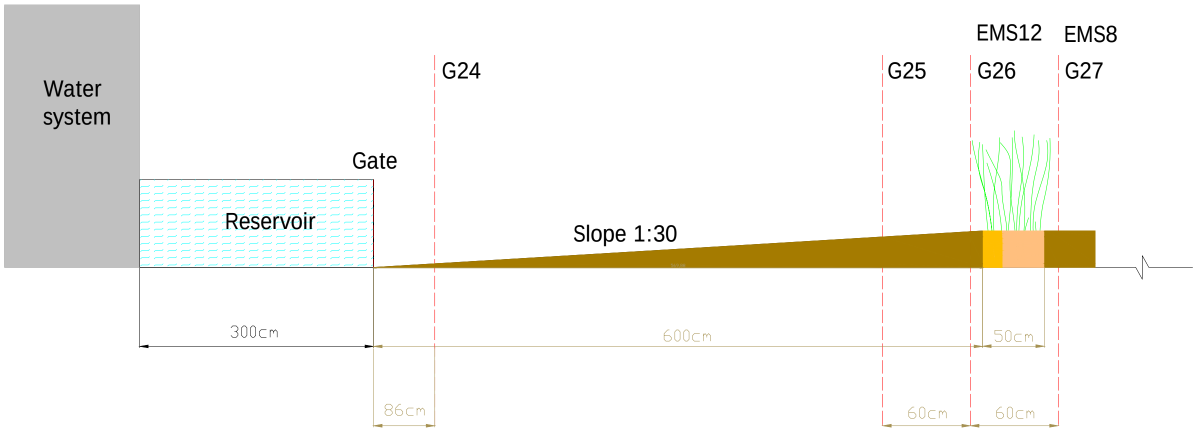


Figure 3-5: Position of measurements

1.
2. G23 is in the middle of the reservoir (this data only to check the water level inside the reservoir
3. G24 is behind the gate at the distance of 84cm.
4. G25 is placed at a location G26 and G27
5. G26 is in front of the Vetiver hedge
6. G27 is behind the Vetiver hedge

The system of GHM consists of a probe and a control unit, which converts the low-level signals of the probe to a high-level voltage representing the liquid level. The wave gauges are used to measure the distance between the free surface and the bed level with the accurate data ±0.1mm. The wave gauges have arranges of wave height 5, 10, 20 and 50cm, and have the range of output control by +/- 10V. The observed data has the is linearly related to the voltage with the accuracy of +/- 0,5% of the range selected.

Flow velocity is measured by using an Electromagnetic Liquid Velocity Meter (EMS). The EMS can be generally applied for flow –monitored purpose in open channels and full or partial filled pipes. The instrument consists of 3 basic parts: the probe with pre-amplifier, the Control-unit in universal Carrying Case and connection-cables. The connection cables between this box and the Control-unit may then reach up to 1000 meters.

The schematized break-dam wave is defined by the maximum water layer thickness, the maximum water velocity and the time duration. The water layer thickness can be measured by GHM gives accurate data. But measuring velocity by EMS is quite difficult because the individual time is too short and this parameter is quickly changing in a particular time.

Video images are recorded by the video camera during the tests for monitoring the process of the break-dam wave and the interaction between the Vetiver hedge and the wave component to obtain the dimensions and velocities of the wave through the Vetiver hedge. The video camera also records the changes of the wave in front of and behind the hedge and the characteristics of the Vetiver hedge such as density, elasticity which can affect on the wave flow in the slope. The video camera is used for recording the progress of the wave and to check the time.

The Dasylab9 program is used for recording measured data. The main software which is used for processing and analyzing data is Matlab.

3.3. Experimental pre-tests

Before starting the experiments, several pre-tests are carried out. The shape of the waves are generated and checked with the basic of the dam-break theory. Furthermore the results from the pre-tests are used to check if the dimensions of model set-up was correct or not and to calibrate the accurate of instruments. Because the accuracy of all the next steps depend on the accuracy of collected data from the pre-test measurements so this step is importance. During the experimental pre-tests some interesting phenomena with remarkable notices are realized:

1. There are 2 noticeable points for checking the wave shape: Firstly, designed wave is based on the dam break theory so the characteristics of the wave shape must follow this theory. Secondly, the wave characteristics need to be similar with the designed wave components: wave celerity and wave layer thickness at the position in front of the Vetiver grass. As can be seen from figure (3-6) after about two seconds, the wave layer thickness reaches the maximum value and then reduces slowly. The phenomenon which appears just few seconds later is back curve. However this research does not focus on that phenomenon, only the data of the first coming waves is used. Water layer thickness and velocity at the positions G26 and G27 are collected; from that wave components can be controlled.

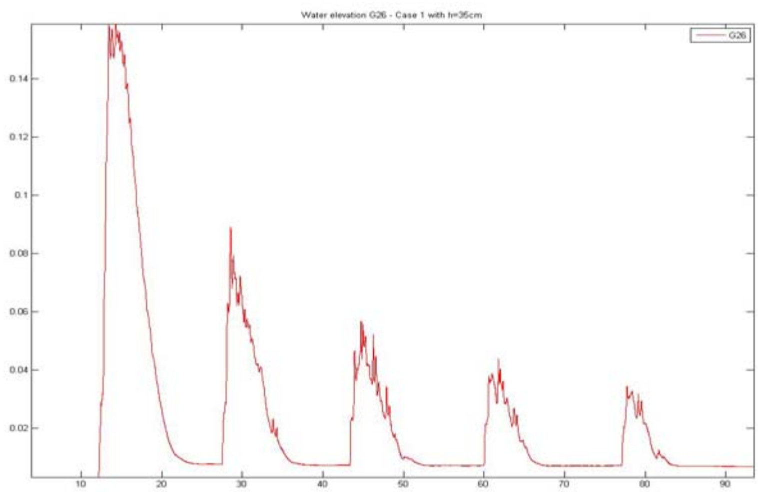


Figure 3-6: Wave elevation at gauge G26

2. The volume of the reservoir is checked by changing the different water level inside the reservoir in order to decide which one is the most suitable with expected wave components. The wave velocity and wave layer thickness showed that their value do not depend much on the total volume of water inside reservoir, only the water level inside the reservoir and the time for the opening the gate proved as important factors.

3. The accuracy of the measurements can be checked under the same experiment set-up (water level and configuration of reservoir), at that time the waves should have the same characteristics. But in the pre-tests do not show the same value under different tests, only the trend is the same. Therefore it is necessary to repeat the tests which have the same condition for several times and using average data may give more accurate result.

Test Series

In this study teh experiments with different set-ups are carried out. The combination of various water levels, grass densities produce 16 different scenarios. With each concrete case one grass density is combined with 4 different water levels and testes repeatedly 10 times.

Case	Density of grass ($Culm/m^2$)	Water level inside reservoir h_r (cm)
Case 1	Without grass	50, 45, 40, 35
Case2	530	50, 45, 40, 35
Case 3	265	50, 45, 40, 35
Case 4	160	50, 45, 40, 35

Table 3-2: The experiment scenarios

Testing procedure

For equipment: for the equipments the following procedure is carried out

- Calibrate all equipments before using.
- As the wave gauges require that the measuring head must always be under the water. Hence they must be placed at the positions of wave gauges by slots.
- With the EMS, the position of head measurement is placed at least 10mm from the bottom.
- Check the computer.
- Check the program for recording data: Dasylab9.

For reservoir

- Close the gate.
- Checks all positions which are water leaked and fix them.
- Fill water inside the reservoir until matching required level.

Slope and Vetiver Grass

Except the first case of experiment without the grass, in the rest ones the grass should be avoided touching equipments around it.

- Count total culms used in the experiments.

- Check the slope under the grass, especially the area next to the glass wall (after water flow through this area, there is an amount of earth follows the flow, so it is necessary to add more earth in order to keep the same slope for the next test).
- Remove all the water in the slope.

The procedure of the measurements during the tests as follows:

1. Close the gate and remove all the water in the slope, fill water into the reservoir to match required level.
2. Turn on the program for recording wave height and wave velocities.
3. Open the gate as quickly as possible (to avoid the delay time).
4. Note all strange phenomena happened during every step of the test.
5. Repeat all the steps ten times.

With the different grass densities the pictures are taken and several recorded tests are made from the beginning until the second back wave. This data is used for checking or illustrating later.

Picture 3-4: Overview of experimental set-up

4. Measurement results and discussions

In this chapter, firstly the data processing and process observation are described. Secondly all the results from physical experiments are formulated and analyzed in the following contents:

- Resistance of the Vetiver hedge
- Energy head loss through the Vetiver grass
- Overtopping discharge
- The reduction of wave run-up with different Vetiver grass densities

All results have been plotted using rough data of gauges G26, G27 and the data of EMS8, EMS12. They are translated into the wave characteristics, overtopping discharge, energy loss, etc.

4.1. Data observation and data processing

In this part wave characteristics from measured data is processed and introduced. Collected data from the experiments includes the first and the second propagation of wave run-up. However the data in the first propagation is only used. The general processes observed about wave hit into Vetiver hedge is described in the following series pictures (4-1) during the first propagation. Series pictures give only one process of the wave around Vetiver hedge. The beginning time in the series pictures is described after opening the gate about 2 seconds; at that time the wave start to meet the Vetiver hedge.

- Firstly, series pictures in the left side show that as time goes by a part of wave is blocked in front of Vetiver hedge. As the result the wave run-up is reduced, this phenomenon is the most interested in this research.
- Secondly, after about 3 continuing seconds amount of water is kept in front of Vetiver hedge and made a small pond. The increasing of water level in font of and the difference of water level in front of and behind Vetiver grass is higher and get the maximum value after about 10 seconds. During this time, water flow still passes through the Vetiver hedge. However the total water through the hedge is too small in compared with the total water coming in front of the hedge.
- The following series pictures in the right side show that the difference between water level in front of and behind Vetiver hedge start to reduce. A part of the total amount of water which is trap in front of Vetiver hedge continues through the hedge. The rest one will run down the slope, it causes the reflected wave from the hedge. The interaction between the back curve, coming wave and Vetiver hedge do not show in the series pictures (4-1). In face this phenomenon is interesting, the combination of reflected wave and the next coming wave can change the breaking wave. The model setup is designed for total water which is though Vetiver hedge can be run out and does not give any affection on the coming wave.

Other information in the real situation the water that is passing through the hedge after reaching its maximum run-up level is running down. This water is against the run-up height of new coming wave. The reduction depends on the dimensionless run-up height and Vetiver hedge characteristics.

The process observation in picture (4-1) gives the overview and the primary knowledge about the interaction of wave and Vetiver hedges that is plant on the outer slope in order to reduce overtopping discharge.

Picture 4-1: The wave hit the Vetiver hedge (start from the left side to right side)

In order to explain how to use the data, one of examples will be explained. The measured data which is used in this research is descried like in figure (3-6). After that the data is translated into energy head loss and discharge overtopping. This research uses the maximum value of those data in front of and behind Vetiver grass to calculate. For example, the figure (4-1) describes the process of one wave energy head loss, in which the energy in front of the grass is green line and behind is red line.

- From A÷B (green line) and A÷C (red line): the program starts recording measured data. At that time the wave does not come to the measured positions, data is recorded in dry environment. So the measured data is not accurate, that reason why it could not be used.
- The process after the point D is the second wave (back curve), in the real situation the back curve much affects on different parameters like wave height, wave velocity, energy loss, reflection coefficient, etc… It makes phenomena around grass becoming more complicated. This research focus only on the first wave process.
- The data which is collected between points B and D (green line), points C and D (red line) describes all the first wave propagation, it is clear that wave changes in time. In a short time at the beginning it increases rapidly and reaches the maximum value, and then it gradually falls down.

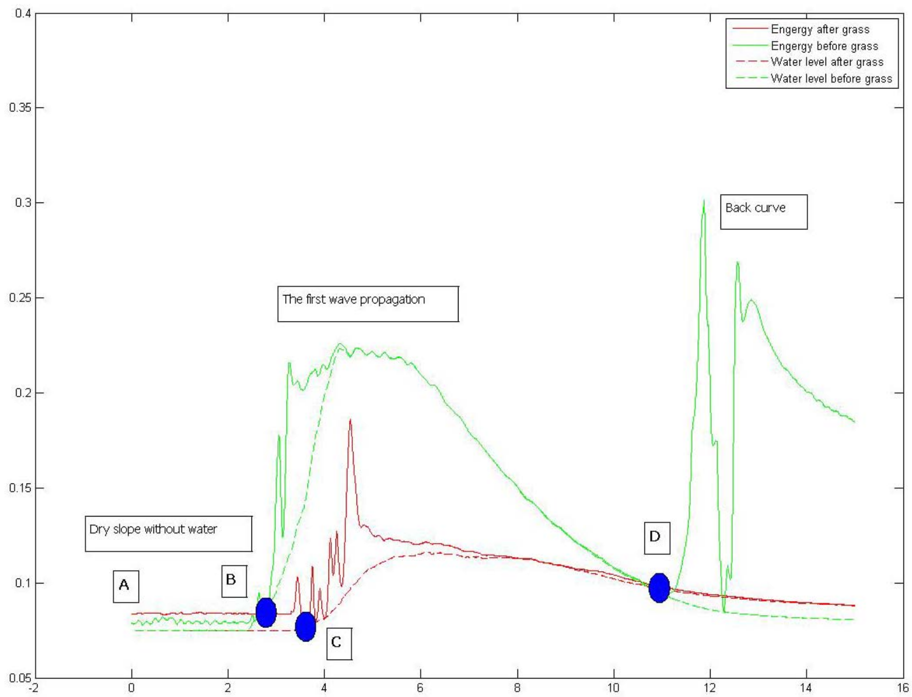


Figure 4-1: Process of individual wave energy

In this research, total energy loss and total discharge are plotted by Matlab. The way of calculating these results are also shown in the figure (3-7). For example, total energy head loss in front of grass calculated by integrating the area in which upper boundary is green line from the first point B to the end point D, and lower boundary is axle-axis. This method is also applied for calculating total discharge overtopping.

4.2. Flow through Vetiver hedge

4.2.1. Resistance of Vetiver hedge

The hydraulic characteristics of the Vetiver hedges at various discharges and water depths are determined from the experiment in the wave flume. A series of laboratory experiments is carried out to investigate the influence of Vetiver grass density and flow depth on the resistance which imposed by the Vetiver grasses. Einstein (Smith et al. 1990) found that the roughness of the grass wall is negligible compared to the roughness of bottom. For the calculation, the water depth y was used instead of hydraulic radius R. Rewriting formula (2-18) yields:

$$u_1 = \frac{1}{n}.\sqrt{S_f}.R_h^{\frac{2}{3}} \qquad \rightarrow \qquad n = \frac{1}{u_1}.\sqrt{S_f}.h^{\frac{2}{3}}$$

The following calculation is applied:

- Water depth is taken the average value of maximum water depth in front of and beside Vetiver hedge. Water depth changes with the difference of grass densities.
- The energy slope S_f is calculated by taking average value from different measurements of the energy line H(m). At the same grass density, energy slope depends on measured water depths and flow velocities in front of and beside Vetiver hedge. In this experiment the Vetiver hedge length and width has the same value. The length of hedge is called x, so energy slope can calculate as the formula: $S_f = \frac{-dH}{dx}$.
- u_1 is the velocity of flow in front of Vetiver hedge.

The calculated roughness coefficient n is plotted that against the flow depth in figure (4-2).

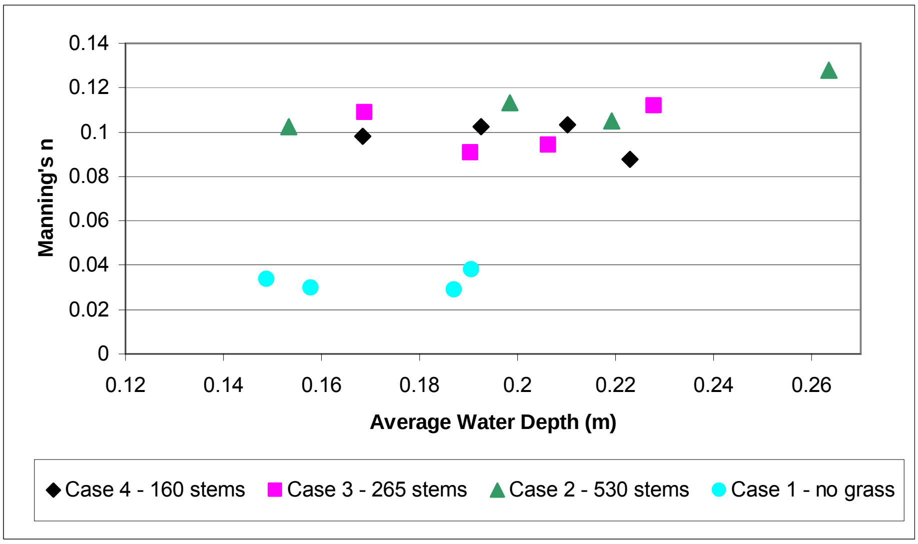


Figure 4-2: The variation of roughness coefficient against the flow depth

In the figure (4-2) shows the relationship between Manning's coefficient and the Vetiver densities. It varies with different grass densities far more than in case of un-vegetated bed slope. In case without grass, Manning coefficient n does not change much with the increasing of water depth. The value of Manning coefficient is approximately constant. The varying of its values is only between 0.029 and 0.038.

The changing of Manning coefficient depends on the grass densities, the flow velocity and the water depth. This value of the highest density of Vetiver grass can reach the value nearly 2.5 times in compared with the case of without grass. It is logical; the friction coefficient is higher in the case of bottom with Vetiver grasses. It can be seen in experimental results that resistance generally have no plain trend when the water depth increases. However it is clear that the Vetiver roughness increase with the higher of water depth. This shows that not only the presence of Vetiver grasses increases Manning's substantially but also Manning's varies with flow depth. Vetiver grass procedures a higher resistance for the bed than in case without Vetiver grass, it reaches the highest value at case of 530 stems per square meter. Therefore it has large impact on water depth. Resistance of flow through the Vetiver hedge depends on grass density and water depth.

In open channel, Manning coefficient increases with the bigger density of grass and higher water depth. However in this experiment the tendency not as clear as expected. The following explanation would be used:

1. In this simplified experiments which based on drag concept is proposed to evaluate the roughness coefficient for unsubmerged Vetiver grasses. In many previous studies like Paul Truong (2004), Juha Jarvela (2004), Nehal and Yan Zhong Ming (2005) Manning coefficient shown obvious result in relationship with depth flow and grass density, especially for open channel with unsubmerged vegetation along channel under permanent flow. The difference in this experiment is that it done with non-permanent flow.
2. The calculating in this research uses formula (2-19).

- Average water depth is correct value with the accuracy of measurement of water depth in front of and beside Vetiver hedge.
- Energy slope depends on both measured water depths and flow velocities in front of and beside Vetiver hedge. In this experiment the testing time for one experiment is too short, around 10 seconds. Also because of characteristic of instrument for measuring velocity EMS, it could not measure exact value of velocity in too short time. That is the reason why energy slope could not fully give accurate values.
- In open channel velocity which is used for calculation can measure at exact position. But in this experiment for calculating the velocity in fron of Vetiver hedge is taken. That value is assumed to equal with the velocity in the middle of the hedge. The difference between velocity in front of and in the middle of Vetiver hedge could make the mistake in calculation.

According to Ree (1949), when the depth continues to increase it is expected that resistance would reach a point of inflection and lower rate of decrease of resistance as flows submerge the vegetation and tend toward a streamlined condition. While the same trend remained where overall resistance decreased with the depth, the turning point where resistance began to decrease with increasing depth was not observed. However in this experiment all tests done for non-submerged waters and the point of inflection is outside the range of collected data.

4.2.2. Relationship between water depth and grass density

In this research non-permanent flow through the Vetiver hedge which is simulated from dam-break is applied, the relationship between these parameters may give different information. According to experiment set-up in the following steps water level inside reservoir, water depth around Vetiver hedge and density of Vetiver grass are described in a relationship.

In this experiment the change of the water level inside the reservoir means that the corresponding wave height generated by dam break from the same water level and flow depth in front of Vetiver hedge, also changes. The difference between water levels and velocity in front of and behind Vetiver hedge directly effect to energy slope S_f and average water depth. The more difference between these water levels the higher steep of the energy slope. And the Vetiver density may change this difference. Especially in this research is the affection of water level in side reservoir which is initial condition for simulate wave dam-break to water level in front of Vetiver hedge.

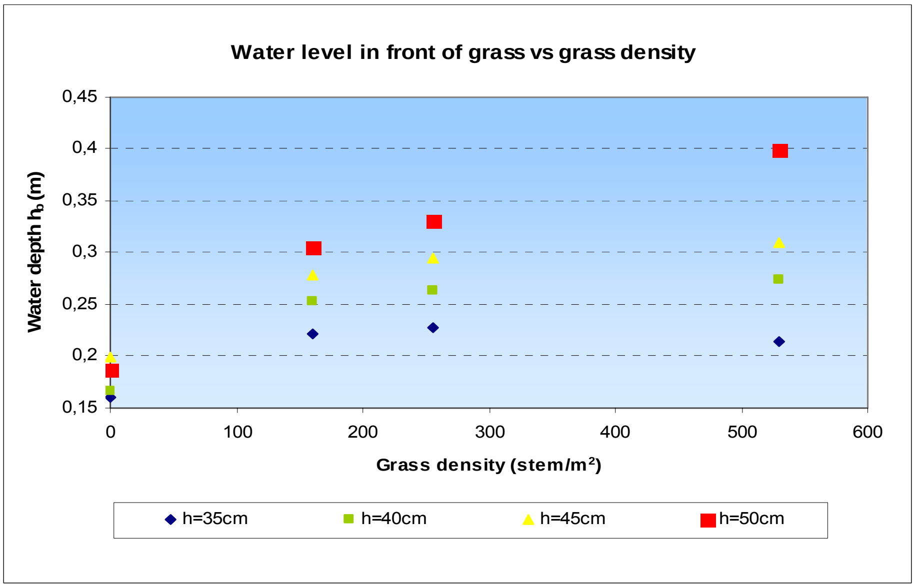


Figure 4-3: Water level in front of Vetiver hedge

In the figure (4-3) and (4-4), the water depth in front of the Vetiver hedge (h_b) is plotted depending on the water depth inside reservoir (h_r) and grass density (d). It can be seen that the higher density of Vetiver grass, the more increasing of the water depth in front of the grass but that value reduces behind the grass. The water depths with different grass densities have the same trend, but the changing of these values behind the grass are less than the one inn front of. Figure (4-3) illustrates the water depth in front of grass h_b increases quickly with the increasing density of Vetiver grass, but that value slowly rises up from the density around of

200 stems per square meter. Except in the case of water level inside reservoir is 50cm, water depth in font of Vetiver hedge h_b continues rising with density of the grass higher than 200 stems per square meter.

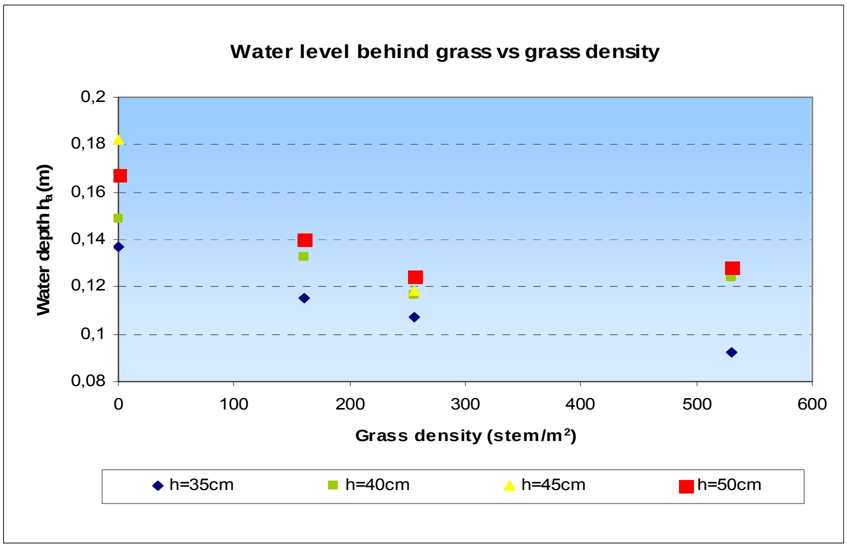


Figure 4-4: Water level behind Vetiver hedge

The trend of the water depth behind the grass h_a decreases with higher density. The significant reduction of water level after through Vetiver grass can be seen under small wave height (corresponding to a small water level inside reservoir). But the reduction of water depth h_a only occurs under density of Vetiver is smaller than around 250 stems/m^2. When the density increases larger than about 250 stems per square meter, water level reduce a little. That trend with water level insides reservoir 35cm reduces with higher density. Figure (4-4) shows value of water depth after grass (h_a) at water level inside reservoir 45cm and 50cm are approximately equal. Even with the density of 250 stems per square meter, that value at 40cm also has nearly the same results.

	Case 1 - without grass			Case 4 - smallest density of grass		
Water level inside reservoir (cm)	Water level In front of grass (m)	Water level behind grass (m)	Reduction (%)	Water level In front of grass (m)	Water level behind grass (m)	Reduction (%)
35	0,161	0,1367	15,093	0,222	0,115	48,198
40	0,167	0,1488	10,898	0,2527	0,1324	47,606
45	0,1994	0,1818	8,826	0,279	0,1412	49,391
50	0,187	0,167	10,695	0,3059	0,14	54,233
	Case 3			Case 2 - largest density of grass		
35	0,2281	0,1074	52,915	0,2144	0,0923	56,950
40	0,264	0,1167	55,795	0,2732	0,1238	54,685
45	0,2943	0,1181	59,871	0,3096	0,1289	58,366
50	0,3313	0,1245	62,421	0,3987	0,1285	67,770

Table 4-1: Layer thickness through Vetiver grass

From the table (4-1) under that case without grass, value of the water depth in front and behind Vetiver grass (G26 and G27) are approximately equal, its value at gauge G27 (h_a) only 0,02m lower than the water depth (h_b) at gauge G26. In cases with grass, that value is at least ten times bigger. The average increasing of water level are: 0,12; 0,16 and 0,18m respectively for the cases 4, 3, and 2. The explanation for the case one is that testing time is too short in narrow distance between two gauges, hence bottom dissipation between them is inconsiderable.

The changing of the water depth behind the grass is not so much under different grass densities but that value in front of it changes significantly. The changing of the water depth especial for value in front of Vetiver grass depends on the grass density. It shows clearly for a high water level inside the reservoir:

- In the case of various densities and water levels inside reservoir are smaller than 50cm, the difference of water depths in front of the grasses change around 0.005m when water levels rise.
- This value increases suddenly when water level rises to 50cm high and corresponding to highest density (case 4). It is 0.05m higher compared with water level at 45cm high under the same density.

* Discussion

From the experimental results present in figure (4-2) and (4-3), it is found that the water depth behind the Vetiver hedge decreases with the increasing of the grass density for the grass density less than 250 stems per square meter. With the density higher than this value, the water depth decreases a little. Therefore the value of grass density at 250 stems per square meter may have s significant meaning for design as an optional value of grass density if we want to have the lowest water depth behind the Vetiver hedge.

The water level behind grass has a tiny changing under various water levels inside reservoir and grass densities in compared the value in front of grass. The water level in front of the Vetiver hedge can have the high value under dense density of Vetiver hedge and high water level inside the reservoir. That value reaches a significant value of 0.4m under density of 530 stems/m^2 and of 50cm water high inside reservoir. This value can be explained by the following reasons:

- In a short time, amount of large water comes to Vetiver hedge.
- Vetiver grass has stiff erect stems; under dense density it makes itself a strong barrier.

Because of these reason the water is blocked in front of the grass and there for the water level at this location increased to high value. Through the experiments, it again confirms that Vetiver grass can withstand a thick layer of water against flow. In highest density, water column of 0.4m depth corresponding to the water level of 50cm inside reservoir can be established. This characteristic of Vetiver is significant in comparison with other grass for preventing because it can prevent considerable water through the hedge.

4.3. Energy loss

4.3.1. Quantitative results

The total force on a slender cylinder is assumed to be the sum of a drag force and inertia force as Morison eqution:

$$F(t) = C_m \frac{1}{4}\pi D^2 \rho \frac{dU}{dt} + C_d A \frac{1}{2}\rho U\left|U\right| \tag{4-1}$$

The force acting on the plants should include the relative motion between the fluid and the plants. In the work of Menzen et.al the plant motion has been neglected. For this reason plant-induced forces acting on the fluid can be expressed without swaying motion and inertial motion, the losses are consider to be due to drag force:

$$F(t) = C_d A \frac{1}{2}\rho U^2 \tag{4-2}$$

With C_d is drag coefficient, ρ is density of water, u is mean velocity and A is characteristic interested area.

According to Petryk and Bosmajian (1975) in the case where the roughness element of vegetation cover the bed, energy losses has been expressed in teams of the average boundary shear and Darcy friction f so $\Delta E = f(C_d, A, u, f, n)$

Waves which are simulated by break-dam can be described as function of water level inside reservoir h_r, (equation from appendix A): $u_{bore}(z, t) \sim \sqrt{h_r}$

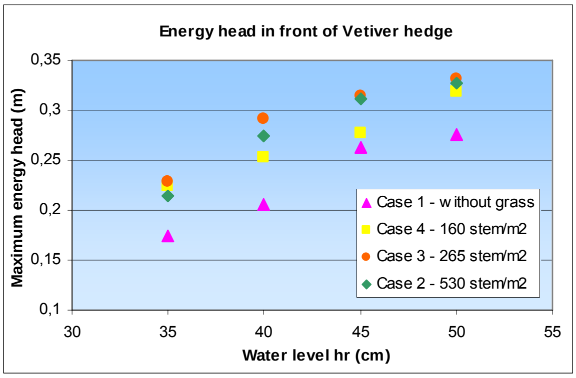
Beside it in the case of an oscillating flow in wave, the value of the Keulegan-Carpenter number can be used to describe the relation between the drag coefficient and the flow regime $K = \frac{u_c T_p}{b_v}$. This formula not only shows the relationship between velocity u and plant stem diameter b_v but also gives the dependence of vegetation height and plant density d_g.

Combination for flow in wave through the Vetiver hedge energy losses is considered as function $\Delta E = f(C_d, A, u, f, n, h_r, b_v, d_g)$

In this experiment these dependence of energy losses with grass density of Vetiver, water level inside reservoir which simulate wave dam-break and flow regime can be verified. These conclusions are a part content of previous section (4.2). A proper process - based approach should consider to support to above section. One remarkable existence that should reduce the accuracy of energy loss is measured velocities.

4.3.2. Measurement results

The energy head for individual wave is calculated bases on Dalton formula (2-17), the mathematical expression of the hydraulic processes give in appendix B. In the figure (4-5) both the relationship between maximum energy head in front of / behind Vetiver hedge and water level inside reservoir are described.



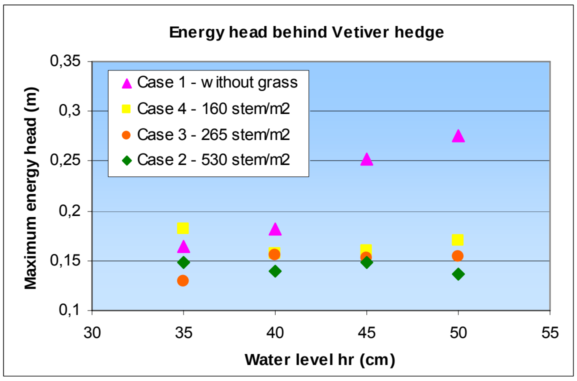


Figure 4-5: Overview energy head in front of and beside Vetiver grass

As shown in the figure (4-5), the energy head increases together with the increase of water level inside the reservoir. The energy head in front of the hedge in case without grass always has the smallest value under variation of water level inside reservoir. This value for behind Vetiver grass is highest, however energy head in front of and behind Vetiver grass have approximately value. Therefore the difference between energy head after passing two waves gauges is considered equal.

The energy head at cases that have Vetiver grass becomes closer with the rise of water level inside reservoir, that judge can give the ideal that energy loss may independent with grass density under high water level inside reservoir. Energy head loss behind grass in case with Vetiver hedge changes little, especially from water level at 45cm to 50cm high, as well as the result is taken before grass which has the same water level.

The difference of energy heads in front of and after Vetiver hedge under various grass densities and water levels inside reservoir are described in figure (4-6).

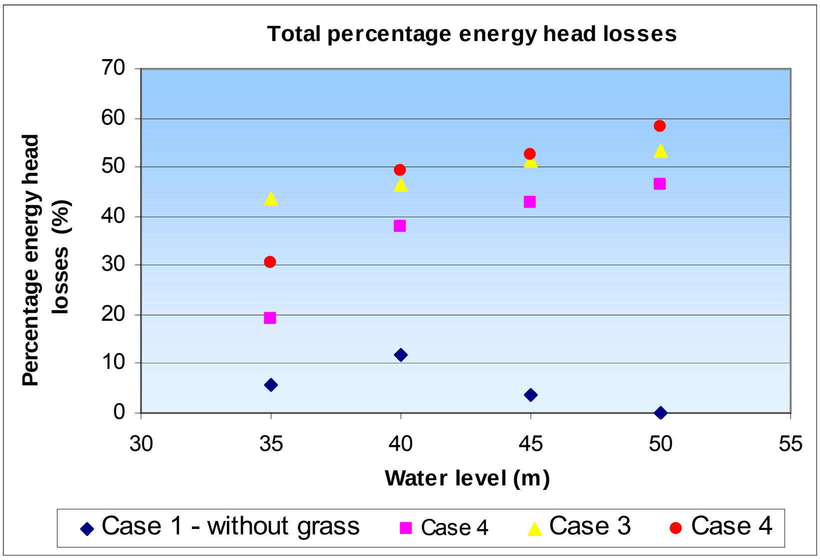


Figure 4-6: Total energy loss through Vetiver grass

It is clear seen in figure (4-6) that under the cases with Vetiver grass the reduction of energy head loss increase with the higher water level inside reservoir. However the reduction of energy head loss though the Vetiver grass is considerable in cases which have Vetiver grass. It can reduce over 40% flow energy in the case of smallest grass density and even nearly 60% energy flow in the case of highest density and highest water lever inside reservoir. On the other hand, there is no clear trend for the energy loss related to the different water levels inside reservoir. It can be seen that with the same grass density, the percentage of energy loss is independent from water level inside reservoir.

* *Discussion*

From energy point of view, energy head in front of Vetiver hedge increases when the grass density increases. Because the difference of velocity upstream and downstream of the hedge is small, the energy loss can be approximated by the water level difference. Hence the energy loss can be considered as energy head loss. According to the formula of Dalton energy loss is a function of flow velocity and water level which is the same result with quantitative result in the first part. In this experiment the results show that the water level is more important than the flow velocity. Especial in case of dense grasses, the flow velocity is limited. This reason explain the result of energy head losses between two high density at case 3 and case 4 have approximately result under high water level inside reservoir. At that time the difference of water depth in front of Vetiver hedge is small and wave celerity through hedge is limited by dense grasses.

As mention in the section (2.3.2) the Vetiver barrier will be broken under high pressure. It means that with increasing of energy head loss, Vetiver grass should be bending. In this experiment the test carried out of small variation of water levels inside reservoir and extreme case of the water levels are not test. For further studies, trials with high water levels may be needed in order to find out the relationship between the energy head loss, the water level inside reservoir and critical bending of the Vetiver grass.

In the case without grass both the energy loss and the reduction of energy loss are received the smallest value. The main parameter which has much affect on the reduction of energy loss is the roughness coefficient. In this experiment one again the result affirms that the slope with roughness of Vetiver grass has more effectiveness than slope without grass.

4.4. Overtopping discharge

The maximum overtopping discharge through the Vetiver grass hedge for specific wave is calculated by the measured data of wave gauges G26, G27 and flow velocity of EMS8 and SMS12. The calculation is given at appendix B which is linked with reduction of energy loss through the Vetiver hedge.

The experimental results in this research are overestimated in comparison with real situation because the collected data includes the water following down the slope.

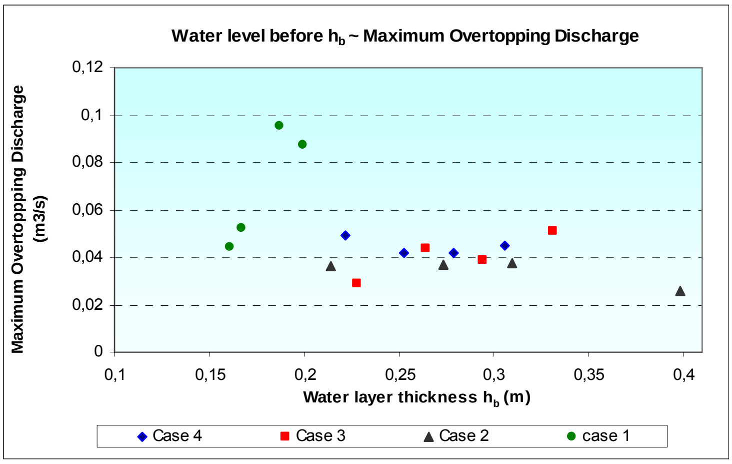


Figure 4-7: Water level in front of vs discharge for various Vetiver densities

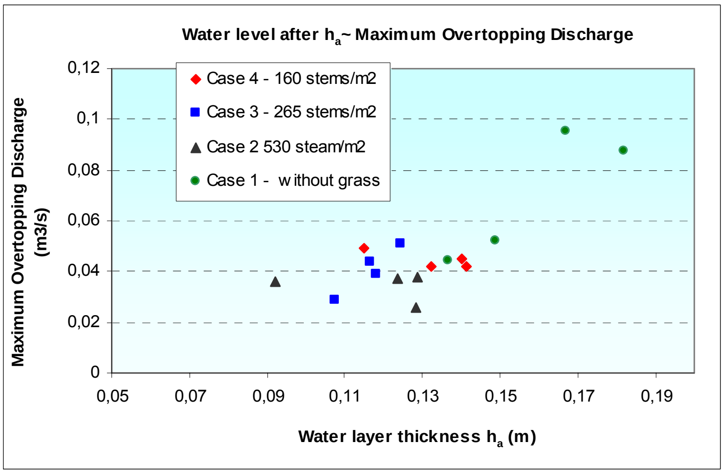


Figure 4-8: Water level behind vs discharge for various Vetiver densities

The measured water depth around Vetiver hedge and overtopping discharge are plotted in figure (4-7) and figure (4-8). These figures show the relationship between the water depth inside reservoir and overtopping discharge which depends on the Vetiver density. In case without grass, the overtopping discharge increases with increasing of water depth and reach the value much more higher than in the cases with grass under the same water level inside reservoir. In the cases with grass the rising of maximum overtopping discharge is not significant, it changing is very small with different grass density. However the value of layer thickness in font of Vetiver hedge under increase faster than the one behind Vetiver hedge when rise the water level inside reservoir. The relationship between the water layer thickness, the maximum overtopping discharge and the grass density do not follow linear equation.

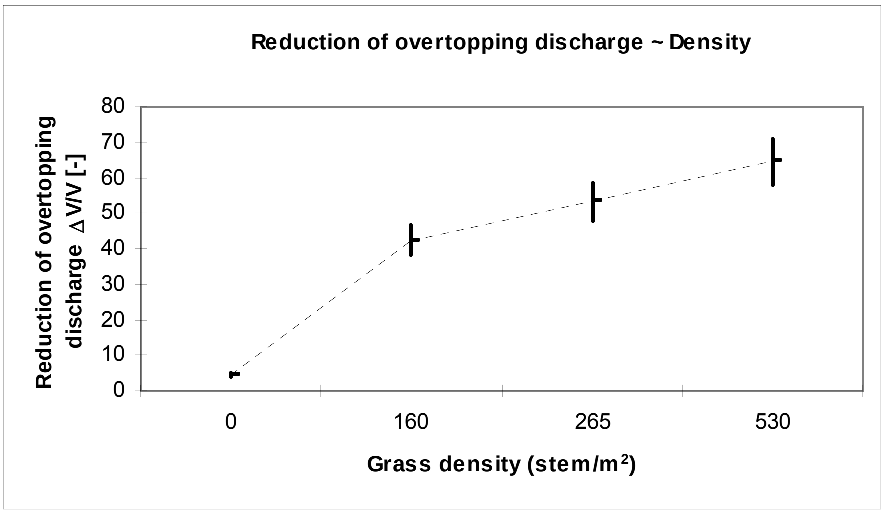


Figure 4-9: Reduction of overtopping discharge under different densities of Vetiver hedge

From the measure data corresponding with each density of Vetiver grass, the average overtopping discharge reduction is calculated. In the figure (4-9) this result is plotted. It shows that the reduction of overtopping discharge increase with the higher Vetiver grass density. This overtopping discharge reduction from the density of zero to 563 stems per square meter rise significantly from a value of just fewer than 5% up to a value of nearly 65%. That trend still increase with the increasing of density which is larger than 530 stem per square meter.

Waves are made by using the dam break propagation in this study. The total water which made individual wave can be controlled by limited water. It means that the flow through the Vetiver hedge can only be considered as continuous flow in a short time. The following figure illustrates the reduction of discharge overtopping and the water level inside the reservoir.

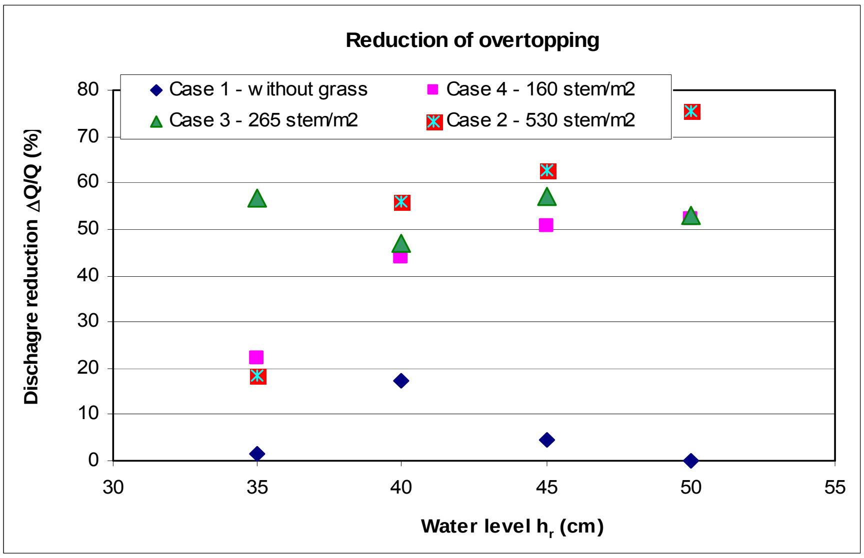


Figure 4-10: Reduction of overtopping discharge

It can be seen from figure (4-10) that for a smooth slope without grass, the percentage of discharge reduces less than 20%. It means that almost amount of water (about 80%) passes through the interested area (between G26 and G27). This value increases at the higher density of grass. At the grass density in case 4 and case 3, this value reaches to the maximum value of 51%, 57% respectively. The value is significant in case 2 for the highest density and various water levels inside the reservoir. The total overtopping discharge reduces at least 18% and at most 75%. This result gives a proof that the density of 530 culms per square meter will exert reducing effect on overtopping effectively.

Corresponding water level inside the reservoir of 40cm and 45cm, the values of reduction overtopping discharge with three different Vetiver grass densities do not change so much in compared water level 35cm and 50cm. This result may provide the idea that in normal situation of wave conditions, the density hedge of grass not necessary too high but still has the same influence on reduction of discharge overtopping.

* *Discussion:* In the literature there is little available information which bases hydraulic description on the flow through a dense grass hedge. Klassen and van der Zwaard (1974) simply derived effective value of the Chezy for flood plain transected by hawthorn hedgerows. The flow of water through more extensive vegetation (Turner, 1978; Chanmeesri, 1984; Smith, 1982) can be quantified by empirical discharge depth equations. Recently study at University of Queensland provided an empirical equation of relationship between discharge and flow depth. In this study more information on the relationship of the density of Vetiver hedge and maximum overtopping discharge and its age, are described. Further more the effectiveness of hedge maturity and grass density on the reduction of overtopping has been also provided. The different point in this study is that the wave run-up is made using dam-break propagation.

It should be noticed that the measured data and its described above still has limited aspects. It is recommended for further studies to extend in the following aspects:

- The relationship between discharge and flow depth
- The difference of hedge resistance between continued flow and dam-break flow

4.5. The reduction of wave run-up

In order to have a realistic value of the reduction of wave run-up on the slope, one example in Nam Định province in Vietnam has been chosen for the calculation, following four steps:

- Calculate wave run-up on the slope of sea dike bases on information of wave, wind, tidal, sea dike profile, etc…
- Estimate the average ration between water level in front and behind Vetiver hedge from measured data.
- Recalculation the wave run-up on outer slope in case planted Vetiver grass hedge on the slope.
- Define total reduction of wave run-up.
- Compare the wave parameters in two different cases with and without grass in order to find influence factor for roughness elements of Vetiver grass on the slope.

Step 1:

** Boundary conditions:*

The following table shows the parameters of wave, tidal, wind, etc in the Gulf of Tonkin at Nam Dinh area:

Name of dimension	Unit	Value
Maximum value of storm surge	(m)	0.8÷3.4
Tidal amplitude	(m)	1.84÷2.19
Beach slope	(-)	1:250÷1:800
Grain size	(mm)	d_{50} =0.157
Wave period	(s)	7÷10
Wave height	(m)	0.8÷2.0

Table 4-2: Basic data of Nam Dinh area

The final design the outer slope is 1:3, the surge levels during spring tide is up to 5.0m above mean sea level (MSL), the significant wave height is 2.5m high and the wave period is 10s. In this figure (4-8) the over view of sea dike and data which is used to calculate is illustrated:

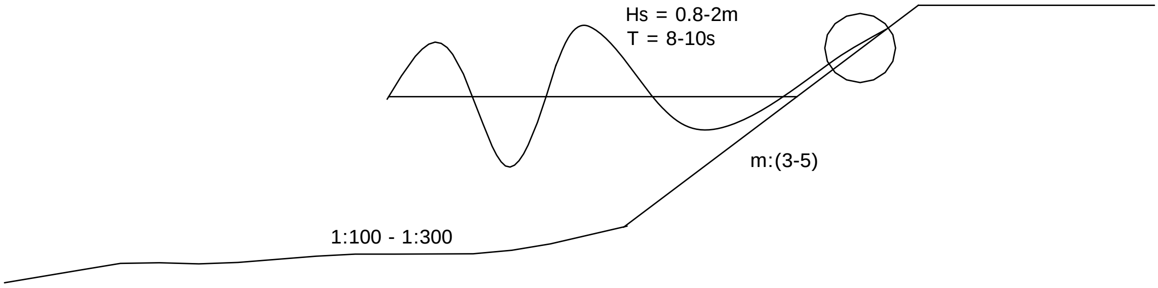


Figure 4-11: Wave boundary conditions

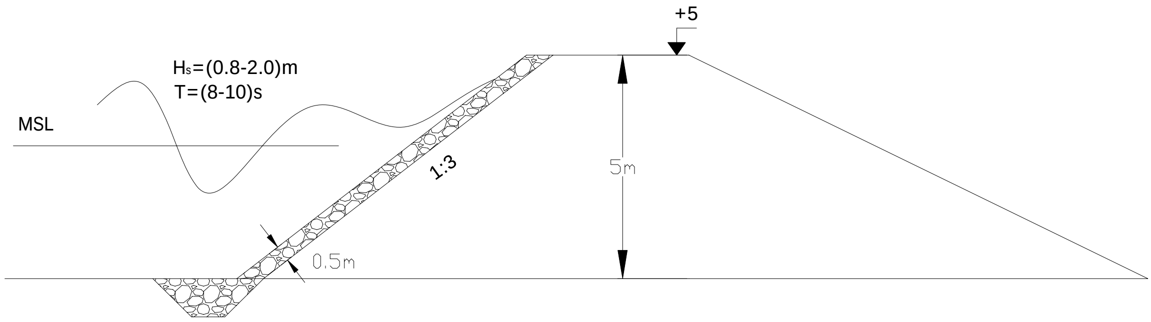


Figure 4-12: Profile of sea dike

* *Calculation:* According to Boundary condition (wind, wave, tidal, sea dike profile), result of wave run-up, water level, and wave velocity on the slope without grass are calculated:

- The wave overtopping is calculated based on formula (2-10) for irregular wave. It can see that wave run-up as function of Iribarren parameter, wave height and influence factor for roughness $R_{u2\%} = f(H_s, \gamma_\beta, \xi_0)$. In this calculation influence factor for berm and wave attack angle equal with 1.
- Influence factor of berm, oblique wave attack are inconsiderable (equal 1), compared with the influence of roughness which gives the main parameter to change wave run-up height

H (m)	0,8	1	1,2	1,4	1,6	1,8	2
T (s)	10	10	10	10	10	10	10
L_0 (m)	156,21	156,21	156,21	156,21	156,21	156,21	156,21
ξ	4,66	4,17	3,80	3,52	3,29	3,11	2,95
$R_{u2\%}$ (m)	1.04	1.374	1.72	2.08	2.47	2.86	3.26
u (m/s)	2.994	3.45	3.86	4.26	4.63	4.98	5.33
h (m)	0.294	0.39	0.49	0.59	0.70	0.81	0.93

Table 4-3: Result of wave run-up in real situation

Step 2: As introduced in the Measuring equipment part (3-2) the measurement points are locate in front of and behind the Vetiver hedge. The measured values of wave celerity and wave height are rearranged in the table (4-4) for water level and (4-5) for wave celerity:

Water level inside reservoir (cm)	Case 1 - without grass			Case 4 - smallest density of grass		
	Layer thickness in front of grass (m)	Layer thickness behind grass (m)	Rate Behind/ In front of	Layer thickness in front of grass (m)	Layer thickness behind grass (m)	Rate Behind/ In front of
35	0,161	0,1367	0,849	0,222	0,115	0,518
40	0,167	0,1488	0,891	0,2527	0,1324	0,524
45	0,1994	0,1818	0,912	0,279	0,1412	0,506
50	0,187	0,167	0,893	0,3059	0,14	0,458
Average			0,886			0,501
	Case 3			Case 2 - largest density of grass		
35	0,2281	0,1074	0,471	0,2144	0,0923	0,431
40	0,264	0,1167	0,442	0,2732	0,1238	0,453
45	0,2943	0,1181	0,401	0,3096	0,1289	0,416
50	0,3313	0,1245	0,376	0,3987	0,1285	0,322
Average			0,422			0,406

Table 4-4: The ration of water level

For wave celerity:

Water level inside reservoir (cm)	Case 1 - without grass			Case 4 - smallest density of grass		
	Velocity In front of grass (m/s)	Velocity Behind grass (m)	Rate Behind/ In front of	Velocity In front of grass (m/s)	Velocity Behind grass (m)	Rate Behind/ In front of
35	1,255	1,078	0,859	1,401	1,231	0,879
40	1,466	1,187	0,810	1,465	0,8351	0,570
45	1,302	1,358	1,043	1,544	0,766	0,496
50	1,683	1,84	1,093	1,891	0,738	0,390
Average			0,951			0,584
	Case 3			Case 2 - largest density of grass		
35	1,5	0,4198	0,280	1,05	0,2512	0,239
40	1,95	0,5723	0,293	1,61	0,4098	0,255
45	1,982	0,7855	0,396	1,85	0,416	0,225
50	1,783	0,8421	0,472	1,72	0,5037	0,293
Average			0,360			0,253

Table 4-5: The ration of wave celerity

Under cases which have the same density of Vetiver grass assume to have the same effluence of roughness. Hence in the average value is used to calculate in the following table:

Case	1	4	3	2
Rate of wave celerity (-)	0,951	0,584	0,360	0,253
Rate of wave height (-)	0,886	0,501	0,422	0,406

Table 4-6: Relationship between wave celerity and wave height around Vetiver hedge

Step 3:

Recalculation the wave run-up parameters on outer slope in case Vetiver hedge is planted on the slope. The following table gives the result for the highest wave parameters which has corresponding height of 2m and period of 10s at toe of the dike. This value bases on measured ration of wave thickness. During the experiment time and analysis data measurement of wave height gave more accurate values, therefore the ration of wave height is chosen for calculating.

	Real value	Case 1	Case 4	Case 3	Case 2
		Without grass	Smallest density	Middle density	Highest density
Grass density (stem/m^2)	-	0	160	256	530
Ration of wave height (-)	-	0.886	0.501	0.422	0.406
$R_{u2\%}$ (m)	3.26	2.49	1.41	1.19	1.14
Reduction of wave run-up (%)	-	23.73	56.85	63.64	65.1

Table 4-7: Reduction of wave run-up

The relationship between grass density of Vetiver hedge and wave run-up is illustrated in the following figure:

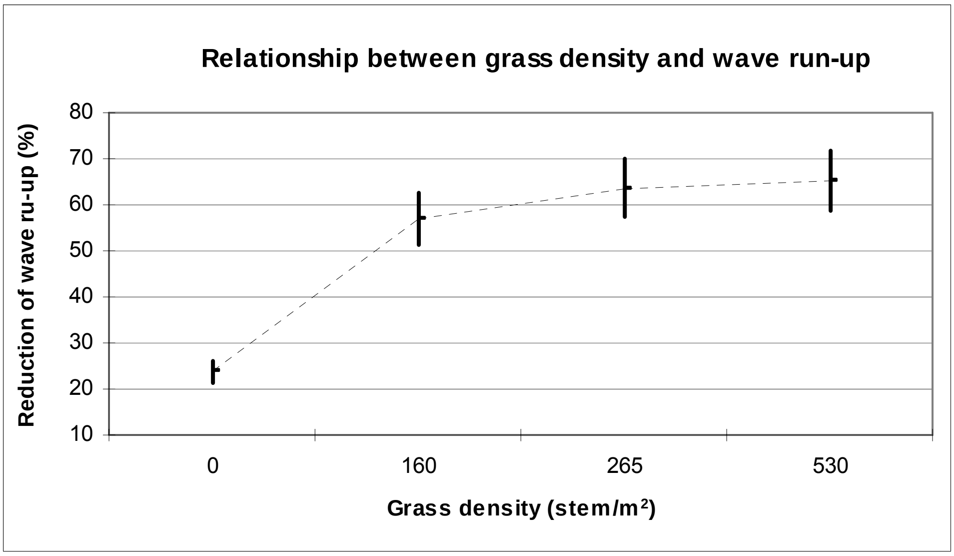


Figure 4-13: Reduction of wave run-up with different density of grass

As can be seen in figure above the reduction of wave run-up depends on the Vetiver density. At the beginning without grass, the revetment is protected by riprap or only earth, the reduction of wave run-up is small, just above 20%. Appling one row of Vetiver grass on outer slope which perpendicular to the direction of wave run-up and is planted at the location at the same height with mean sea level. The wave run-up height reduces significantly. This value increases together with the higher density of grass. It increase from just above 20% to nearly 65%. It can be seen clearly that with the grass density from about of 250 stems per square meter, the reduction wave run-up rise slightly when grass density is higher. Hence, it is a suitable density can be suggested for designing sea dike with application of Vetiver grass.

Step 4:

Applying Vetiver hedge on the outer slope, the reduction of wave run-up is significant. This reduction depends on influence factor for roughness γ_β .

The wave run up can be described as follows: $R_{u2\%} = f(H_s, \gamma_\beta, \xi_0)$, in this calculation wave boundary condition is the same in case of applying Vetiver hedge. Therefore influence factor for roughness of Vetiver hedge can be found $\gamma_{\beta-Vetiver} = \dfrac{R_{u2\%-Vetiver}}{R_{u2\%-Riprap}} . \gamma_{\beta-Riprap}$

The following table (4-8) gives the result of roughness of Vetiver grass hedge with various of densities:

	Unit	Value		
$R_{u2\%\text{-Riprap}}$	m	3,26		
		Case 4	Case 3	Case 2
Grass density	stem/m^2	160	256	530
$R_{u2\%\text{-Vetiver}}$	m	2.41	1.19	1.14
γ Vertiver	[-]	0,410	0,345	0,332

Table 4-8: Roughness of Vetiver hedge with different densities

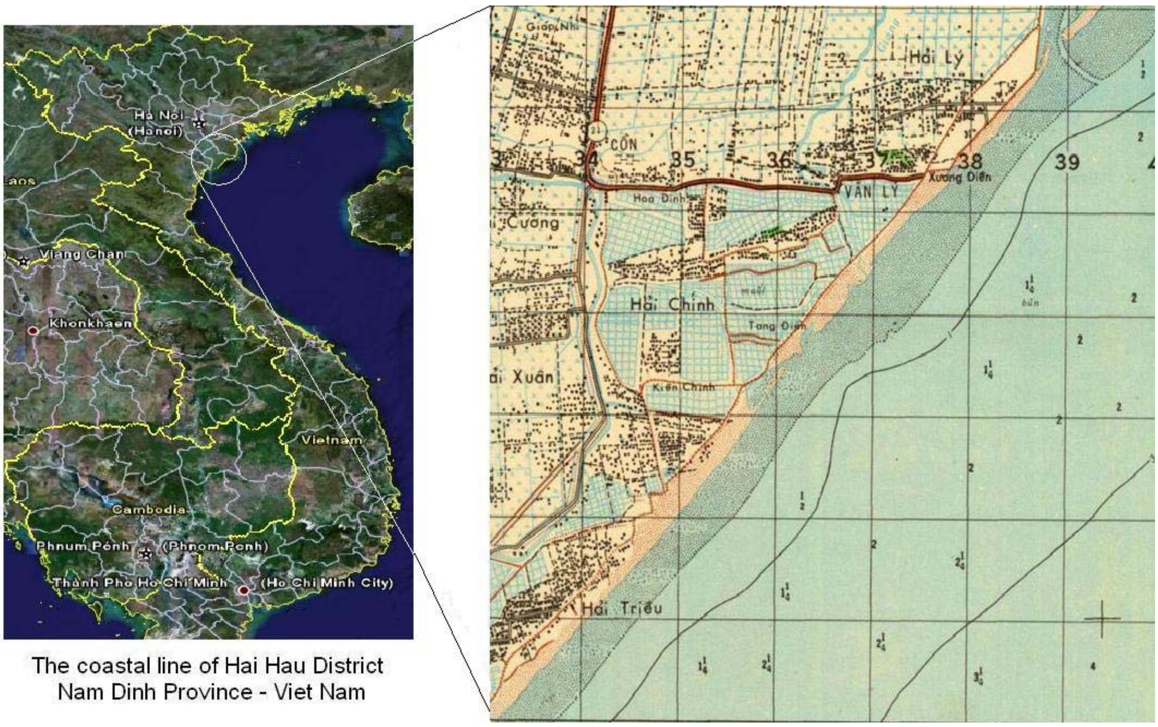
To summarize after applying the experiment result in practice the roughness coefficient of Vetiver grass is around from 0.33 to 0.41 depends on grass density. The reduction wave run-up when introducing Vetiver hedges on outer slope shows the good results. One Vetiver hedge with density of 200 stems per square meter can reduce 60% run-up height.

5. Application

5.1. General information

One of an important objective in this study is the application of experimental data analysis results of using Vetiver grass to reduce wave overtopping in practice. As subsequence of section (4.4) and (4.5), the following presents how Vetiver grass can be applied for sea dike. At a certain extend this is considered as design guidelines for wave overtopping reduction at sea dike by introducing the Vetiver grass.

The application is made for case study of Nam Dinh sea dike system in Northern part of Vietnam. Nam Dinh is situated in the South East of the Red River delta. It borders Ha Nam in the North, Thai Binh in the East, Ninh Binh in the West and the the Gulf of Tonkin sea in the South-East. The province covers total 1,676 square km, with population of 1,934,000. There are three major parts in Nam Dinh, one of them is coastal area including 72km length of coastal line and 4 large rivers mouth-estuary systems. In this region sea dike are the most dominate for Nam Dinh coastal flood defence system. Picture (5-1) shows the map of Vietnam and zoom-im a part of Nam Dinh coastal area at Hai Hau district in Nam Dinh:



Picture 5-1: Location of Nam Dinh province

5.2. Sea Dikes with Vetiver grass

5.2.1. Boundary conditions and dike height criteria

In the previous section (4.5) coastal boundary conditions includes coastal topography, hydraulic boundary condition (waves, wind, sea level) and measured rate of velocity and rate of water depth are already calculated. The case study presents in this chapter shed light on the solutions which solve the problem of wave overtopping in practice. In order to help the readers have more information inside situation in Nam Dinh province. Following part gives the description of sea dike system in details.

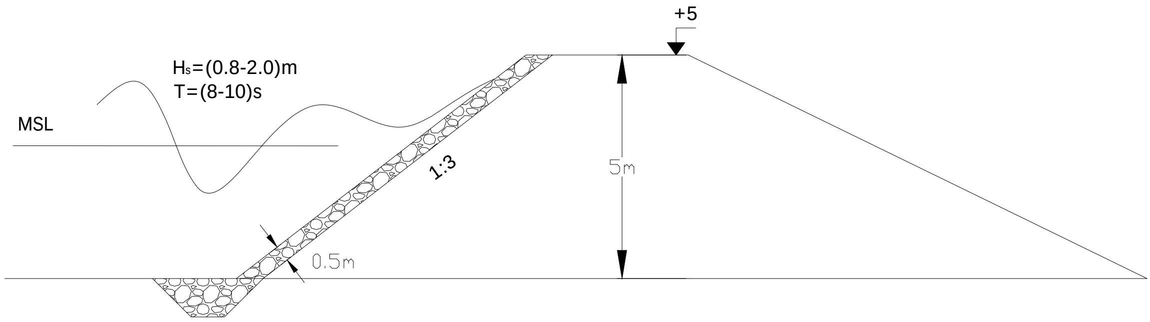


Figure 5-1: Profile of sea dike

In Nam Dinh, the most severe disaster is foreshore erosion and damage of sea dike system which often causes by the impact actions from the sea water. The typical cross shore profile of the sea dike in Nam Dinh is illustrated in figure (5-2), crest level varies from +4.5 to +5.5 depend on the location. The outer slope has steepness of 1 over 3. This is normally protected by rock revetment or concrete block revetments. The inner slope is normally unprotected with slope angle 1 over 2.5. Crest width is about 4-6m which often combines as maintain road. The thickness of revetment is 20-50cm (Phan, 1996; HWRU, 2000) and it is made by rock. The erosion of outer and inner slopes, sometime even dike crest is caused by too much wave overtopping. As the result runoff will happen from the crest. Observed data and simple calculated data give the same result, the present height of sea dike is not sufficient. Wave overtopping always occurs during the tidal period. Some recent year, while traditional protected method such as concrete embankment, bamboo or even mangrove are not effective, the combination of bamboo and Vetiver grass is introduce as other alternatives.

Several pictures were taken after severe Damrey typhoon in September 2005 which landed at Nam Dinh coastal zone and hit the sea dike system there. Influences on dike system are provided more details in appendix D. From that one can not only see the serious problems caused by the typhoon but also the effectiveness of Vetiver grass on sea dike.

In this chapter the application of experimental result is applied to determine the dike height. There are several exiting methods for sea dike design, however in this study the dike height is based on the allowed overtopping discharge criteria for inner slope and Van Der Meer formula.

(1) ''A safe approach if no significant overtopping is allowed '', it means that the crest height should not be lower than the 2% wave run-up level. The criterion for dike height design in this study bases on resistance that against erosion and local sliding of crest and inner slope due to wave overtopping. In Dutch Guideline of designing dam assumes that the following average overtopping rates are allowed for inner slope:

- 0,1 l/s per m for sandy soil with a poor turf
- 1,0 l/s/ per m for clayey soil with relatively good grass
- 10.0 l/s per m for clay protective layer and grass according to standards for an outer slope or with a revetment construction

(2) According to Van Der Meer formula (2001) wave overtopping for non-breaking wave (ξ_0 >2) is given:

$$\frac{Q}{\sqrt{g.H_{m0}^3}} = \frac{0,06}{\sqrt{\tan\alpha}}.\gamma_b.\xi_0.\exp\left(-4,7.\frac{R_c}{H_{m0}}.\frac{1}{\xi_0.\gamma_b.\gamma_f\gamma_\beta\gamma_v}\right) \tag{5-1}$$

5.2.2. Steps for calculation

Without Vetiver hedge

- Choose allowed discharge for inner slope q = 0.1 (l/s per m) = 0.0001 (m^2/s).
- Calculate wave run-up, wave celerity, free crest height R_c and required sea dike height.

With Vetiver hedges

- Calculate with Vetiver hedges are planted on the outer slope.
- Re-calculate wave run-up, wave celerity, overtopping discharge in front of Vetiver hedges.
- Estimate correspond discharge in front of hedge and free crest height R_c
- Calculate the reduction of free crest height R_c

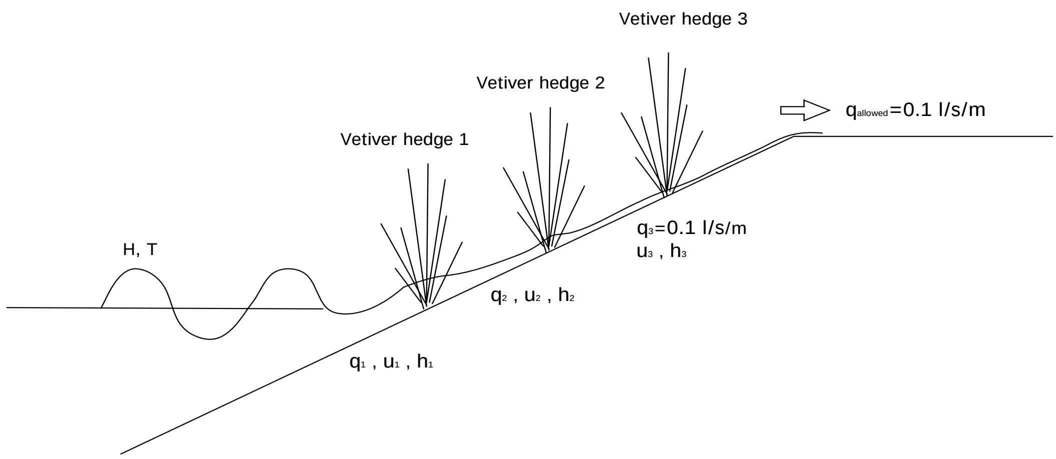


Figure 5-2: Calculation scheme

5.2.3. Results

In case of only one hedge, Vetiver hedge is planed at the same level with SWL or a little bit higher. The hedge is grown across outer slope, perpendicular with wave direction with density of 200 stems per square meter. It means that with Vetiver hedge of 0.5m width, 10 big clusters are assumed, each cluster have around 20 stems. Under this grass density it can reduce at least 60% wave run-up, see more detail from figure (4-13). However these results are just based on limited experiments with nature Vetiver grass. It should be verified and validated before the widely application is made in practice.

In case of two hedges, the second one should be grown with the distance of 1m. Initial moment, flow depth, flow velocity of wave run-up after through the first Vetiver hedge reduce significantly. Hence reduction of wave run-up after second hedge assume bigger than 55%. The number of Vetiver hedges on the slope is limited; it lies on length of slope and strength of Vetiver hedge.

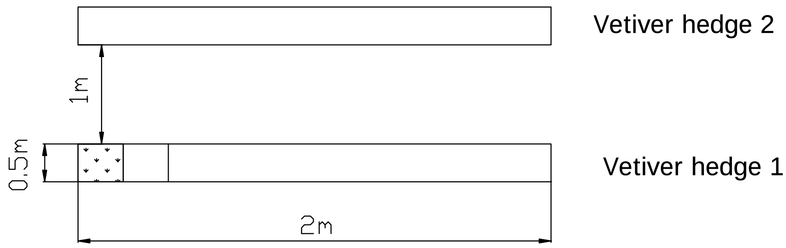


Figure 5-3: Top view of hedges

From sections (4-4) and (4-5) with grass density is chosen 200 stem/m^2, the reduction of wave overtopping is about 45%, see figure (4-9). The rate of velocity and water level (the measure data behind grass divide for the value in front of) which are calculated base on measured data which show in figure (5-5). Corresponding density of 200 steams per square provides the 45% reduction of flow velocity and water layer thickness. According to the results from section (4-5) the factor for roughness of Vetiver grass is taken 0.35.

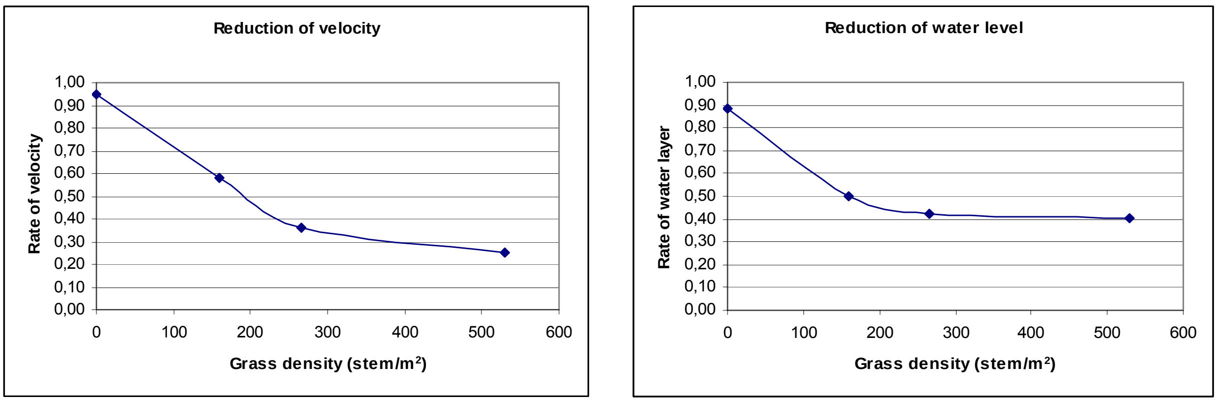


Figure 5-4: Reduction of velocity and water level

Calculation of allowed overtopping discharge and dike crest free board for different case gives the results as in table (5-1). The sea boundary condition for these cases is wave height of 2m at toe of sea dike and wave period of 10s. In case of no Vetiver hedge on outer slop, the sea dike crest is much higher in order to ensure the overtopping criteria.

N_0. Hedge	γ_f (-)	Crest height R_c above SWL (m)	Crest height of the sea dike (m)
No hedge	0.75	5.60	7.60
1 hedge	0.42	5.37	7.37
2 hedges	0.4	5.11	7.11
3 hedges	0.37	4.86	6.86

Table 5-1: Crest height of sea dike

Table (5-1) illustrates the required dike crest height under the allowed discharge 0.0001m^2/s. If one hedge is planted, crest height reduce 0,23m. The reduction of crest height is 0.49m and 0,74m respectively with these cases of two and three hedges. Picture (5-6) gives the cross section of sea dike with crest height without Vetiver hedge and it position after planting two hedges on the outer slope.

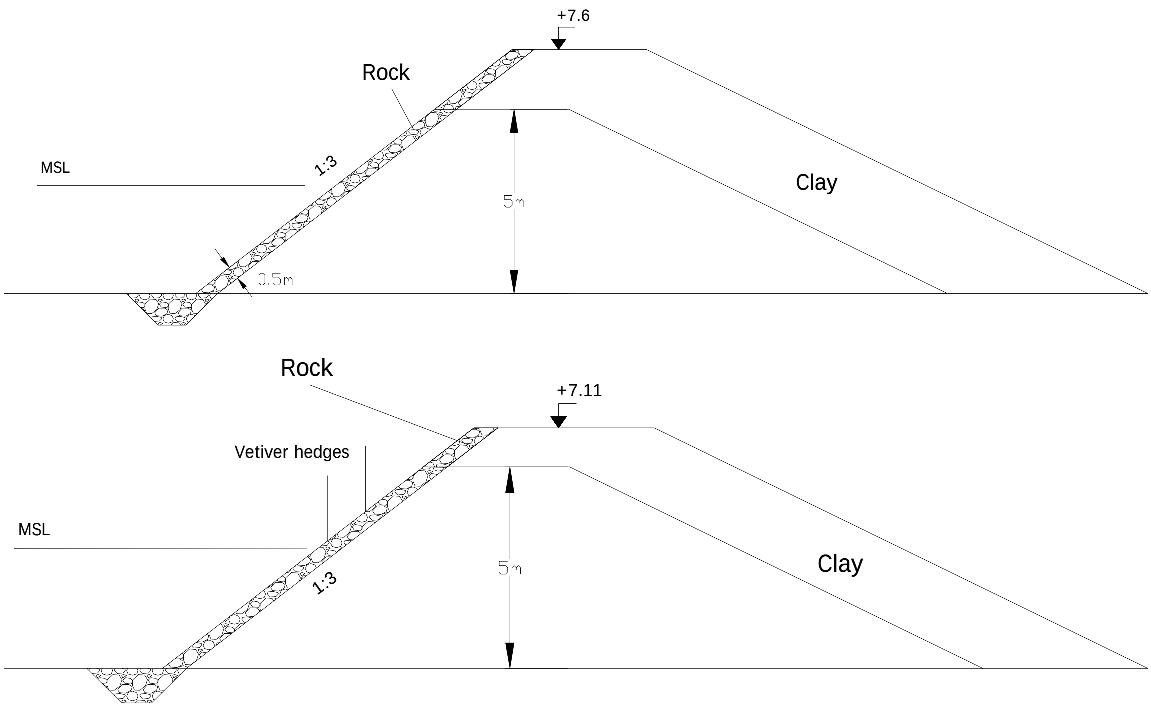


Figure 5-5: Required cross section of sea dike

Results in this chapter are under limited condition. The backwater wave and an affection of next wave are not account. In face the result will be change in practice.

5.2.4. Construction cost analysis

Calculating construction cost for upgrading present sea dike system in order to meet the allowed discharge 0,0001 m^2/s , the following special costs are supplied:

Material	Unit	Price
Clay	m^3	$ 5
Vetiver hedge	m^2	$ 4
Amour layer	m^3	$ 12

Table 5-2: Material cost

From the figure (5-6) upgrading of the present sea dike is described. The revetment which has the layer thickness of 50cm is made by rock and cover the all the outer slope; the rest of dike body is made by clay. In following steps more detail of sea dike are illustrated

- The present sea dike system at level of +5
- In case without Vetiver grass hedge, the crest level is upgraded from level of +5 to +7,6m
- Applying two Vetiver hedges on outer slope, the crest level is upgraded from level of +5 to +7,11m
- Assume that the amour thickness layer is 0.5m

	Material	Total	Unit	Price/m	Total price/m
Without grass	Clay	26,38	m^3	131,9	
	Amour	1,30	m^3	15,60	$ 147,50
Two Vetiver hedges	Clay	22,46	m^3	112,3	
	Amour	1,055	m^3	12,66	

Table 5-3: Cost for upgrading present sea dike

The table (5-4) show s the total cost which need to upgrading one meter length of the sea dike system, total price in case of without grass is more than 1,14 times larger than in case of applying two Vetiver hedge on the outer slope. This means that by introducing Vetiver hedges on outer slope, the cost for upgrading the present dikes is reduced about 12,6% total amount of money. This is considerable number in practical case since the sea dike system is often very long.

5.2.5. Summarizes

The case study give above show the available reduction of cost for maintaining present sea dike in Nam Dinh, Vietnam. It can reduce 12,6% total costs for one meter length. However this result will be changed depends on the total Vetiver hedges which are planted on the slope, average discharge allowed and the way to distribute grass on the slope.

6. Conclusions and Recommendations

6.1. Conclusions

The work presented in this study is an attempt to investigate the hydraulic characteristics of Vetiver grass hedges, an effectiveness of Vetiver hedge to reduce wave overtopping and the development of guideline for designing sea dikes with Vetiver hedge. It is possible to draw certain conclusions.

Firstly, the experimental results with different slope with and without Vetiver grass show a significant effect of grass on flow resistance. It is descried by the Manning coefficient. Manning coefficient provide some indication of the resistance properties of a dense planting of Vetiver with water level around Vetiver hedge and water level inside reservoir. The resistance is general higher with the increasing of flow depth and Vetiver grass density.

Secondly, it appears that there is a hydraulic reaction between Vetiver hedges and the flow through it. The lower portion of Vetiver grass forms a very stiff barrier that effectively dams the flow. Investigating the effect of dense Vetiver grass, under Vetiver grass density of 530 stems per square meter gives the largest flow depth. It made 0.4m water depth before Vetiver hedge.

Further more, the reduction of energy head loss though the Vetiver grass is considerable. The total energy head loss under the highest wave height can reduce over 45% in case of the smallest grass density (160 stems/m^2) and even nearly 60% in the biggest density (530 stems/m^2). The same positive results show in reduction of maximum overtopping discharge. Vetiver hedge reduces 45% total discharge correspond with grass density of 200 steams per square meter. This value is even higher for an increased grass density. The present study also found the roughness coefficient of Vetiver grass is various from 0.33 to 0.41 depends on grass density.

Finally, Vetiver hedge shows a successful evident to reduce wave run-up. The total wave run-up reduction increases up to 60% at density of 200 stems per meter and even higher for higher grass density of Vetiver hedges. As a result, the level crest of a sea dike would reduce 0.53m if two Vetiver hedges are planed on the outer slope with an allowable overtopping discharge 0.0001m^2/s.

6.2. Recommendations

In this research, the experiment was conducted under limited conditions of non-breaking wave overtopping, and wave run-up on slope with perpendicular direction. The influence of berm, toe, angle of wave attack and foreland are not considered. Further research which includes these factors is recommended.

One remarkable point which has been realized during the experiments is that the wave parameters (wave height and wave celerity) can be changed depends on the time for opening the gate. In this experiment the gate was opened manually. It explains the reason of collecting strange values sometimes. Hence for further study wave should be simulated by using machine.

The Vetiver grass which was used in this research was obtainted from botanical garden and was 6 months old. Because of the limited space in the flume, the leaves of Vetiver were cut about 50cm on the top and the roots had only 20cm depth. Vetiver grass lived under artificial conditions during this experiment; they still grew quite well, especially their root, only the leaves color changes less green than usual. However the exact conditions and the growing rate of Vetiver were not measured. In order to understand its development, trials are necessary to carry out as well as the condition when using Vetiver on outer slope under saline conditions,

Also attention should be paid to ecological aspects of applying Vetiver grass. For earth dam with Vetiver grass can accommodate insects and animals to grow. Their activities inside sea dike body give addition holes as the same problem appears after grasses die. It should be pay attention if apply Vetiver grass into practice.

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Appendices

A. Analogy with dam break on dry channel

Beginning with Saint Venant's early work (1848;1871a,b), several investigators have studied the phenomena of propagation of shallow wave in open-channel flow. In the past century, a substantial knowledge has been developed to describe one-dimensional flood propagation (Seddon 1900; Thomas 1983; Stoker 1957; Ponce and Simons 1977). In consideration of an ideal dam break surging over a dry river bed, the method of characteristics may be applied to solve completely the wave profile (Henderson 1996, Montes 1998). For a horizontal rectangular channel, at a given time, the free-surface profile between the wave front is a parabola:

$$\frac{x}{t} = 2\sqrt{gd_0} - 3\sqrt{gd} \qquad \text{(A0-1)}$$

For $-\sqrt{gd_0} \leq \frac{x}{t} \leq 2\sqrt{gd_0}$

Where:

x: the longitudinal direction (x=0 is the dam location)

t: the time (t=0 is the instantaneous dam break)

d: flow depth

d_0: the initial reservoir water depth

After dam break, the flow depth, discharge at the origin x=0 are Constance:

$$d_{x=0} = \frac{4}{9}d_0 \qquad \text{(A0-2)}$$

$$Q_{x=0} = \frac{8}{27}d_0\sqrt{gd_0}B \qquad \text{(A0-3)}$$

And the celerity of the wave front equals:

$$C_s = 2\sqrt{gd_0} \qquad \text{(A0-4)}$$

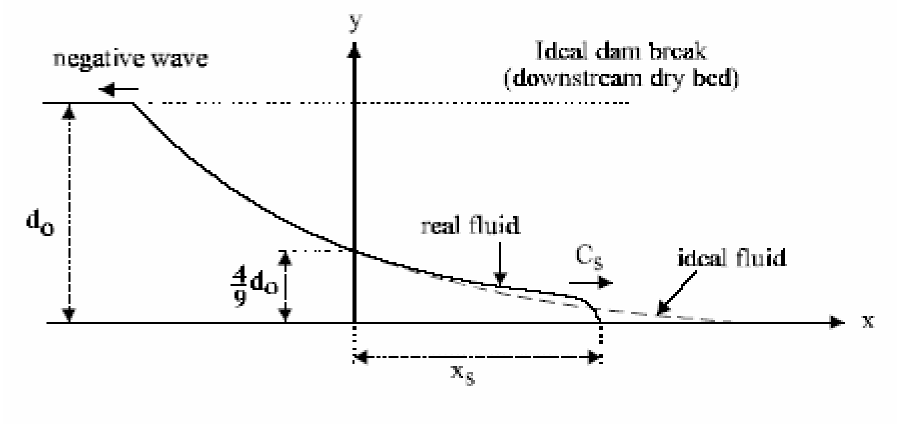


Figure A0-1: Dam break profile

Although equation (0-1) to (0-4) assume no boundary friction, face bottom friction effects significantly on the propagation of the leading tip and it is taken into account the flow resistance. Whitham (1955) developed an analogy between the wave front and a turbulent boundary layer, he estimated the wave front celerity for horizontal dry bed:

$$\frac{C_s}{\sqrt{gd_0}} = \left(0.5002 + 1.45362\left(\frac{f}{8}\sqrt{\frac{gt^2}{d_0}}\right)^{0.4255}\right)^{-1} \tag{A0-5}$$

This equation was applicable only for $\frac{C_s}{\sqrt{gd_0}} > \frac{2}{3}$

Furthermore, he showed that the wave front shape would follow:

$$\frac{d}{d_0} = \sqrt{\frac{f}{4}.\frac{x_s - x}{d_0}}\left(2 - 3,452\left(\frac{f}{8}t\sqrt{\frac{g}{d_0}}\right)^{1/3}\right) \tag{A0-6}$$

In a further development, at the location x >0, equation (5-1) predicts an increasing water depth with increasing time:

$$d = \frac{4}{9}d_0\left(1 - \frac{3}{2}\frac{x}{\sqrt{gd_0 t}}\right)^2 \tag{A0-7}$$

B. Wave propagation

A mathematical expression of the hydraulic processes on the slope of the experimental setup can be found applying the equation of continuity and the equation of motion for open watercourses (first derived by De Saint-Venant). If the resistance term as well as the local accelerations are neglected in these equations (stationary flow), the equation of motion reduces to the equation of Bernoulli (along a streamline): $\frac{d\varsigma}{ds} = 0$

With energy head: $\varsigma = y + \frac{u^2}{2g}$ (m)

Torricelli used the equation of Bernoulli to predict wave propagation like the flow velocity. This parameter depends on the opening space. When the energy head of a water particle in the opening is taken equal to the energy head of a water particle in the surface of the water inside the reservoir, the flow velocity can be approximated:

$$u = \sqrt{2g\Delta\varsigma} \tag{B0-1}$$

The discharge becomes:

$$Q = \mu A u \tag{B0-2}$$

In which

μ:	contraction coefficient	
A:	whole area of outflow opening	(m^2)
u:	flow velocity	(m/s)
g	acceleration due to gravity (m/s^2)	
$\Delta\varsigma$	head difference	(m)
Q	wave volume	(m^3/s)

Thus we have the relationship between discharge and energy head

$$\varsigma = y + \frac{Q^2}{2g\mu^2 A^2} \tag{B0-3}$$

C. Matlab codes for Calculation

Data is analyzed by Matlab, in the following part one example will be showed the main code of program which has been used during my research.

```
%read data from files
{With the same condition, the test repeats 9 or 10 times}
X1 = dlmread('C:\Documents and Settings\c1238167\Desktop\Thesis
new\20060804\test1-35.ASC',';',7,0);
X2 = dlmread('C:\Documents and Settings\c1238167\Desktop\Thesis
new\20060804\test2-35.ASC',';',7,0);
X3 = dlmread('C:\Documents and Settings\c1238167\Desktop\Thesis
new\20060804\test3-35.ASC',';',7,0);
X4 = dlmread('C:\Documents and Settings\c1238167\Desktop\Thesis
new\20060804\test4-35.ASC',';',7,0);
X5 = dlmread('C:\Documents and Settings\c1238167\Desktop\Thesis
new\20060804\test5-35.ASC',';',7,0);
X6 = dlmread('C:\Documents and Settings\c1238167\Desktop\Thesis
new\20060804\test6-35.ASC',';',7,0);
X7 = dlmread('C:\Documents and Settings\c1238167\Desktop\Thesis
new\20060804\test7-35.ASC',';',7,0);
X8 = dlmread('C:\Documents and Settings\c1238167\Desktop\Thesis
new\20060804\test8-35.ASC',';',7,0);
X9 = dlmread('C:\Documents and Settings\c1238167\Desktop\Thesis
new\20060804\test9-35.ASC',';',7,0);

% the matrix correspond with each time testing
f1 = X1(:,1);f11 = find(f1==33);
X1(:,1)= X1(:,1)- 33;
X1(:,2)= X1(:,2)- mean(X1(1:f11,2));X1(:,3)= X1(:,3)- mean(X1(1:f11,3));X1(:,4)=
X1(:,4)- mean(X1(1:f11,4));X1(:,5)= X1(:,5)- mean(X1(1:f11,5));
X12 =X1(66000:96000,:)
t=X12(:,1);
f2 = X2(:,1);f21 = find(f2==6.5);
X2(:,2)= X2(:,2)- mean(X2(1:f21,2));X2(:,3)= X2(:,3)- mean(X2(1:f21,3));X2(:,4)=
X2(:,4)- mean(X2(1:f21,4));X2(:,5)= X2(:,5)- mean(X2(1:f21,5));
X22 =X2(13000:43000,:)
f3 = X3(:,1);f31 = find(f3==60);
X3(:,2)= X3(:,2)- mean(X3(1:f31,2));X3(:,3)= X3(:,3)- mean(X3(1:f31,3));X3(:,4)=
X3(:,4)- mean(X3(1:f31,4));X3(:,5)= X3(:,5)- mean(X3(1:f31,5));
X32 =X3(120000:150000,:)
f4 = X4(:,1);f41 = find(f4==23);
X4(:,2)= X4(:,2)- mean(X4(1:f41,2));X4(:,3)= X4(:,3)- mean(X4(1:f41,3));X4(:,4)=
X4(:,4)- mean(X4(1:f41,4));X4(:,5)= X4(:,5)- mean(X4(1:f41,5));
X42 =X4(45600:75600,:)
f5 = X5(:,1);f51 = find(f5==7);
X5(:,2)= X5(:,2)- mean(X5(1:f51,2));X5(:,3)= X5(:,3)- mean(X5(1:f51,3));X5(:,4)=
X5(:,4)- mean(X5(1:f51,4));X5(:,5)= X5(:,5)- mean(X5(1:f51,5));
X52 =X5(14400:44400,:)
f6 = X6(:,1);f61 = find(f6==7.5);
X6(:,2)= X6(:,2)- mean(X6(1:f61,2));X6(:,3)= X6(:,3)- mean(X6(1:f61,3));X6(:,4)=
X6(:,4)- mean(X6(1:f61,4));X6(:,5)= X6(:,5)- mean(X6(1:f61,5));
```

```
X62 =X6(15200:45200,:)
f7 = X7(:,1);f71 = find(f7==16);
X7(:,2)= X7(:,2)- mean(X7(1:f71,2));X7(:,3)= X7(:,3)- mean(X7(1:f71,3));X7(:,4)=
X7(:,4)- mean(X7(1:f71,4));X7(:,5)= X7(:,5)- mean(X7(1:f71,5));
X72 =X7(31600:61600,:)
f8 = X8(:,1);f81 = find(f8==14);
X8(:,2)= X8(:,2)- mean(X8(1:f81,2));X8(:,3)= X8(:,3)- mean(X8(1:f81,3));X8(:,4)=
X8(:,4)- mean(X8(1:f81,4));X8(:,5)= X8(:,5)- mean(X8(1:f81,5));
X82 =X8(27800:57800,:)
f9 = X9(:,1);f91 = find(f9==16);
X9(:,2)= X9(:,2)- mean(X9(1:f91,2));X9(:,3)= X9(:,3)- mean(X9(1:f91,3));X9(:,4)=
X9(:,4)- mean(X9(1:f91,4));X9(:,5)= X9(:,5)- mean(X9(1:f91,5));
X92 =X9(32200:62200,:)

% the total matrix except the exotic results
G24 = [X12(:,1:2)          X22(:,2) X32(:,2) X42(:,2) X52(:,2) X62(:,2) X72(:,2)
X82(:,2) X92(:,2)];
G25 = [X12(:,1)   X12(:,3) X22(:,3) X32(:,3) X42(:,3) X52(:,3) X62(:,3) X72(:,3)
X82(:,3) X92(:,3)];
G26 = [X12(:,1)   X12(:,4) X22(:,4) X32(:,4) X42(:,4) X52(:,4) X62(:,4) X72(:,4)
X82(:,4) X92(:,4)];
G27 = [X12(:,1)   X12(:,5) X22(:,5) X32(:,5) X42(:,5) X52(:,5) X62(:,5) X72(:,5)
X82(:,5) X92(:,5)];
V8x = [X12(:,1)   X12(:,6) X22(:,6) X32(:,6) X42(:,6) X52(:,6) X62(:,6) X72(:,6)
X82(:,6) X92(:,6)];
V12x= [X12(:,1)   X12(:,8) X22(:,8) X32(:,8) X42(:,8) X52(:,8) X62(:,8) X72(:,8)
X82(:,8) X92(:,8)];

% the result at these wave gauges and EMS
plot(G24(:,1),G24(:,2),'b-',G24(:,1),G24(:,3),'g:',G24(:,1),G24(:,4),'r--
',G24(:,1),G24(:,5),'c-',G24(:,1),G24(:,6),'m-',G24(:,1),G24(:,7),'y--
',G24(:,1),G24(:,8),'k-',G24(:,1),G24(:,9),'b:',G24(:,1),G24(:,10),'c:');
legend('test 1','test 2','test 3','test 4','test 5','test 6','test 7','test
8','test 9');
plot(G25(:,1),G25(:,2),'b-',G25(:,1),G25(:,3),'g:',G25(:,1),G25(:,4),'r--
',G25(:,1),G25(:,5),'c-',G25(:,1),G25(:,6),'m-',G25(:,1),G25(:,7),'y--
',G25(:,1),G25(:,8),'k-',G25(:,1),G25(:,9),'b:',G25(:,1),G25(:,10),'c:');
legend('test 1','test 2','test 3','test 4','test 5','test 6','test 7','test
8','test 9');
plot(G26(:,1),G26(:,2),'b-',G26(:,1),G26(:,3),'g:',G26(:,1),G26(:,4),'r--
',G26(:,1),G26(:,5),'c-',G26(:,1),G26(:,6),'m-',G26(:,1),G26(:,7),'y--
',G26(:,1),G26(:,8),'k-',G26(:,1),G26(:,9),'b:',G26(:,1),G26(:,10),'c:');
legend('test 1','test 2','test 3','test 4','test 5','test 6','test 7','test
8','test 9');
plot(G27(:,1),G27(:,2),'b-',G27(:,1),G27(:,3),'g:',G27(:,1),G27(:,4),'r--
',G27(:,1),G27(:,5),'c-',G27(:,1),G27(:,6),'m-',G27(:,1),G27(:,7),'y--
',G27(:,1),G27(:,8),'k-',G27(:,1),G27(:,9),'b:',G27(:,1),G27(:,10),'c:');
legend('test 1','test 2','test 3','test 4','test 5','test 6','test 7','test
8','test 9');

%Change measures data from unit of Hz into standard unit
V8 = V8x (:,2:9);   V8 = V8*0.25;
V12 = V12x (:,2:9); V12 = V12*0.25;
H26 = G26 (:,2:9);  H26 = H26*0.025;
H27 = G27 (:,2:9);  H27 = H27*0.01;

% Calculate the average value at each measure point
V8 = mean (V8,2);
V12 = mean (V12,2);
H26 = mean (H26,2)+0.075;
```

```
H27 = mean (H27,2)+0.075;
t=G26(:,1);

% plot energy head loss and total energy head loss
E8 = H27+ V8.^2/(2*9.81);
E12 = H26+ V12.^2/(2*9.81);
r=(E12-E8)./E12*100;
plot(t,E8,'r-',t,E12,'g-');
legend('Engergy after grass','Engergy before grass');
title('Energy loss though the vetiver grass - Case 1 with h=35cm');
xlabel('t [s]');
ylabel('E [m]');
axis([0 16 0 0.5]);

% find the maximum value of energy loss
M8=max (E8(f1),E8(f2));
M12=max (E12(f1),E12(f2));
r=M12/M8

% plot velocity and water level
plot(t,V8,'r-',t,V12,'g-');%velocity
legend('Velocity after grass','Velocity before grass');
title('Velocity though the vetiver grass - Case 1 with h=35cm');
xlabel('t [s]');
ylabel('V [m/s]');

plot(t,H27,'r-',t,H26,'g-');
legend('Water level after grass','Water level before grass');
title('Water level though the vetiver grass - Case 1 with h=35cm');
xlabel('t [s]');
ylabel('h [m]');
```

D. Sea dikes with Vetiver protection

In this appendix, several pictures have taken after a big typhoon came to Hai Hau, Nam Dinh, Viet nam last year (12/2005) and its influence on dike system. From these pictures, it shows the problems still exit there and also the effectiveness of Vetiver grass with earth dam.

Details of outer slope of Nam Dinh sea dikes with Vetiver grass protected

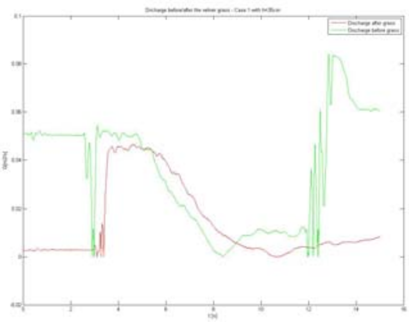
Erosion of dikes slope which has been protected by Vetiver grasses


Sea dikes with Vetiver grass on outer slope

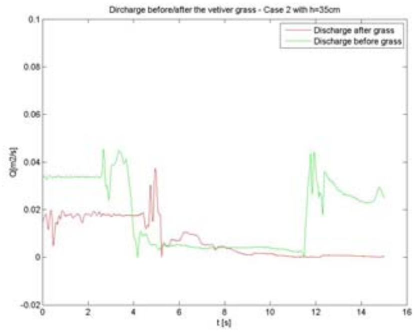

Sea dike without protected of Vetiver grass

From the observed situation, after the storm, there was less damage to Vetiver protected sea dikes than the one without Vetiver grass.

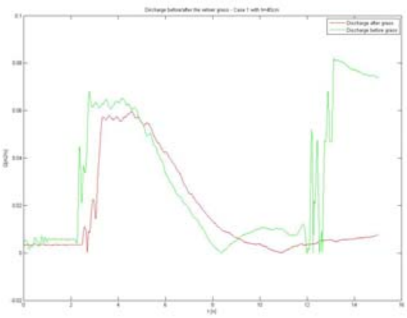
E. Overtopping Discharge



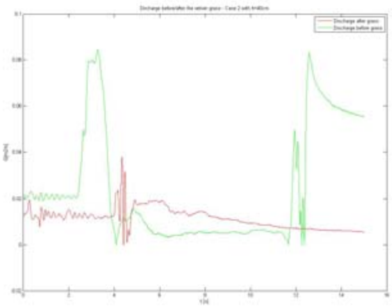
Discharge in case 1 – h=35cm



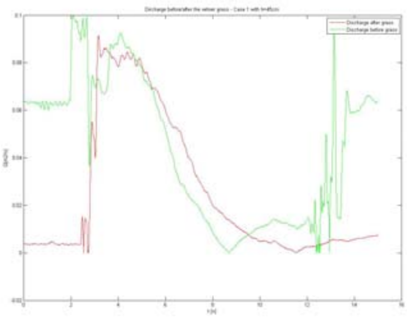
Discharge in case 2 – h=35cm



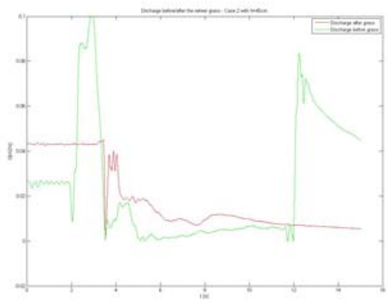
Discharge in case 1 – h=40cm



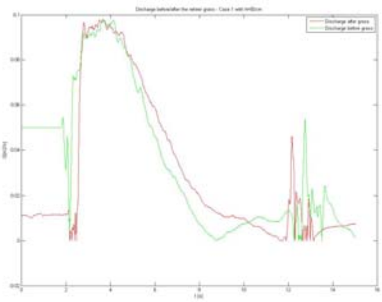
Discharge in case 2 – h=40cm



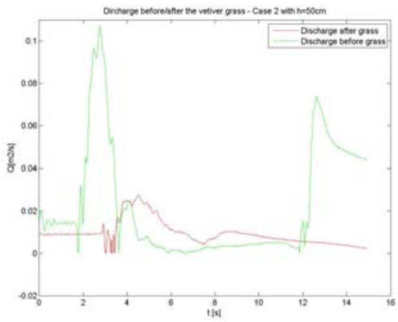
Discharge in case 1 – h=45cm



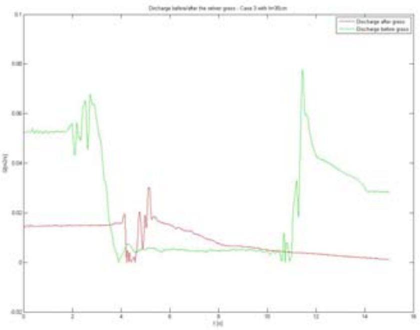
Discharge in case 2 – h=45cm



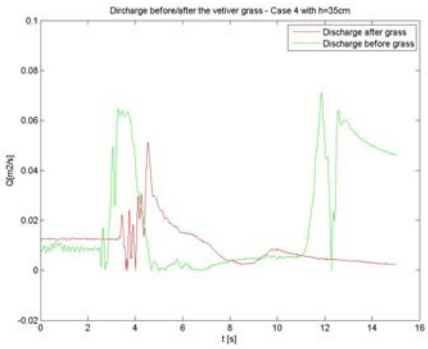
Discharge in case 1 – h=50cm



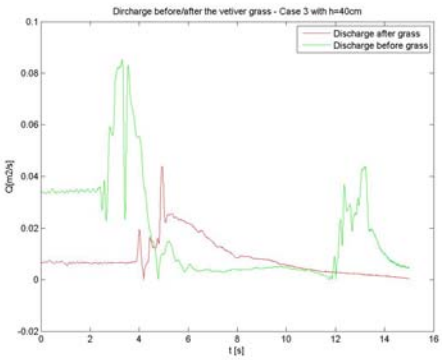
Discharge in case 2 – h=50cm



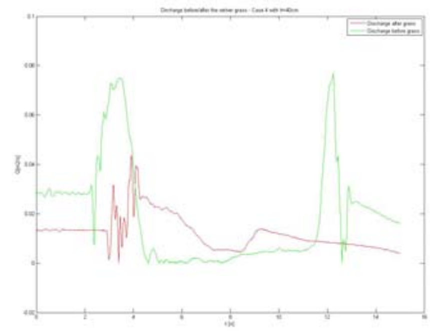
Discharge in case 3 – h=35cm



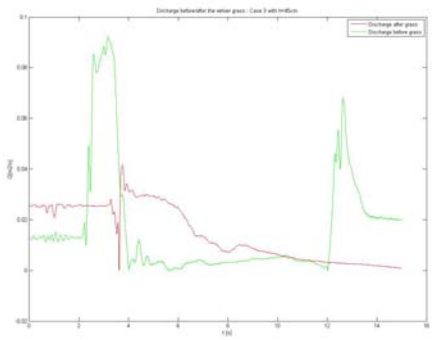
Discharge in case 4 – h=35cm



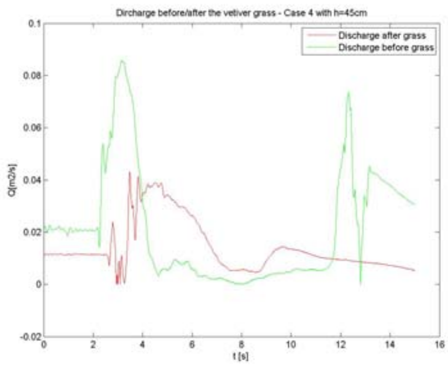
Discharge in case 3 – h=40cm



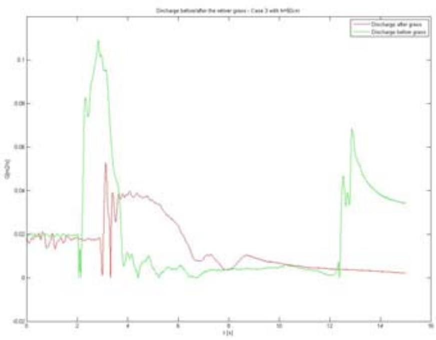
Discharge in case 4 – h=40cm



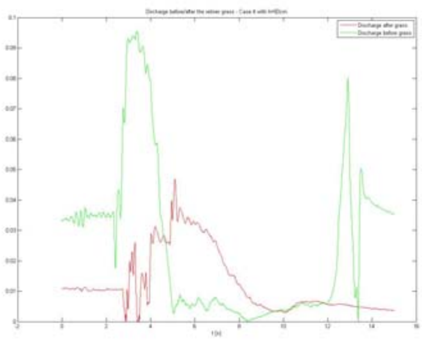
Discharge in case 3 – h=45cm



Discharge in case 4 – h=45cm

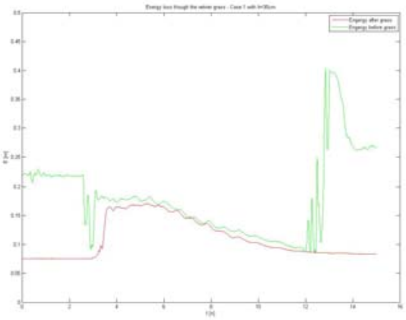


Discharge in case 3 – h=50cm

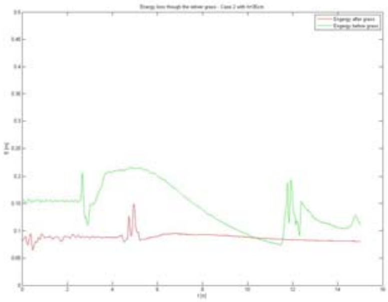


Discharge in case 4 – h=50cm

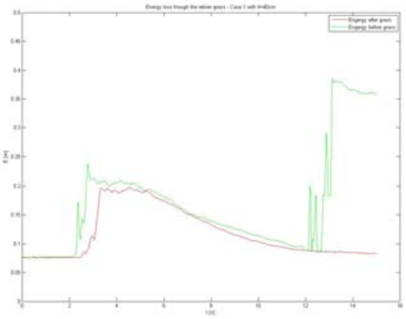
F. Energy Head



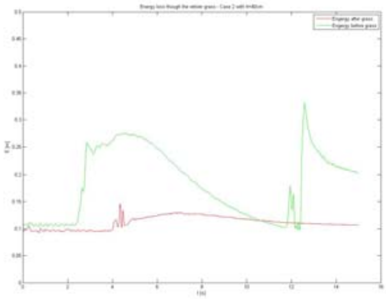
Energy head in case 1 – h=35cm



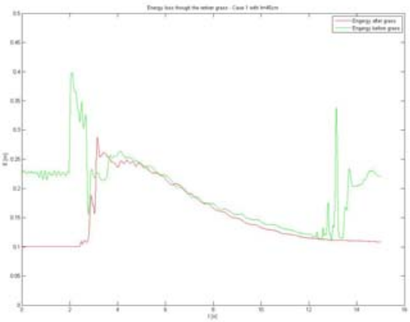
Energy head in case 2 – h=35cm



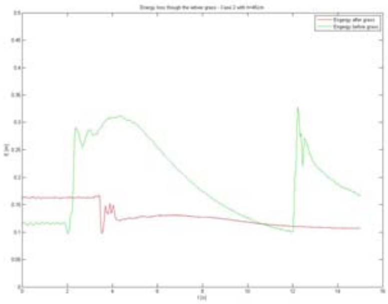
Energy head in case 1 – h=40cm



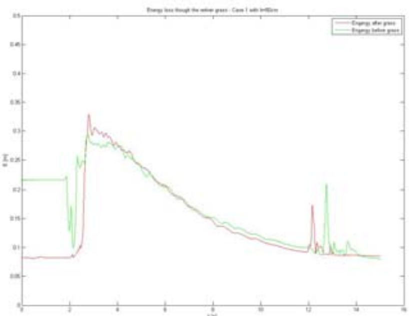
Energy head in case 2 – h=40cm



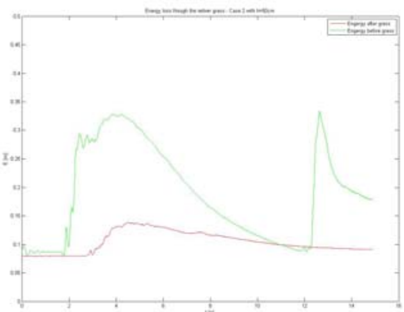
Energy head in case 1 – h=45cm



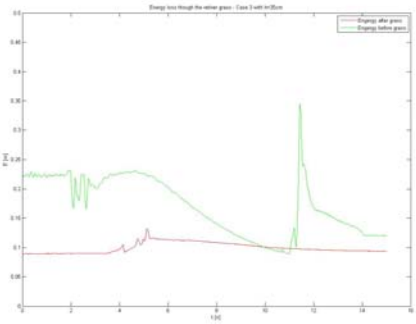
Energy head in case 2 – h=45cm



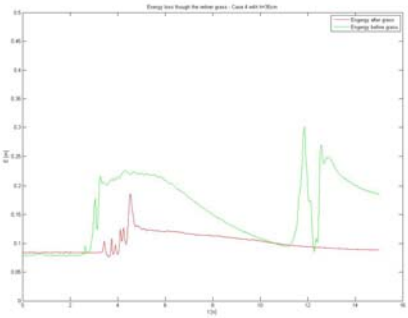
Energy head in case 1 – h=50cm



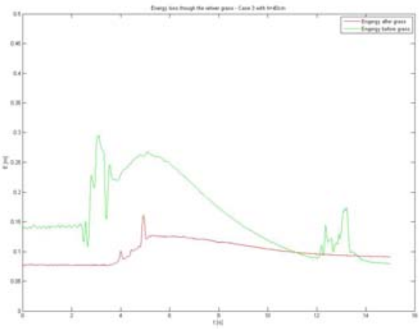
Energy head in case 2 – h=50cm



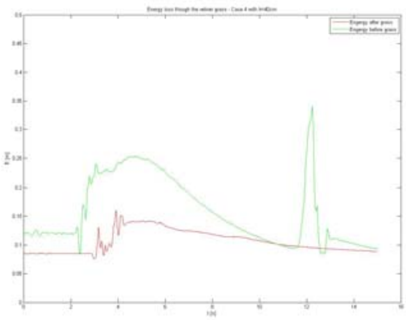
Energy head in case 3 – h=35cm



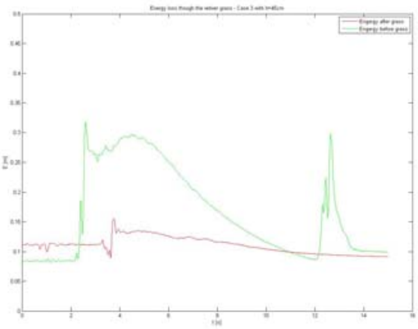
Energy head in case 4 – h=35cm



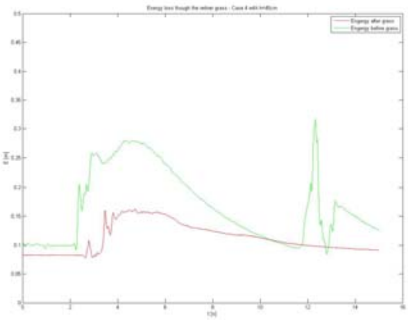
Energy head in case 3 – h=40cm



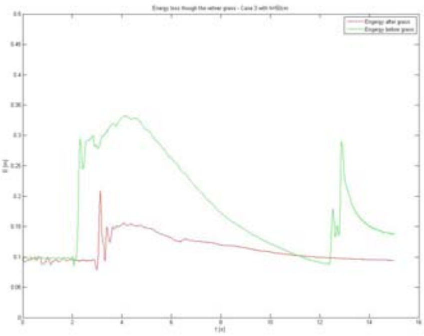
Energy head in case 4 – h=40cm



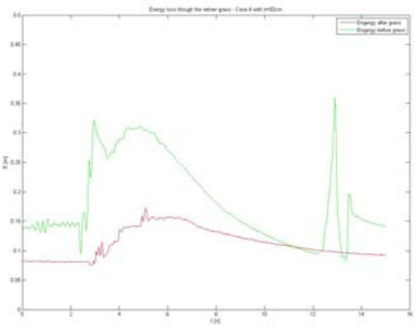
Energy head in case 3 – h=45cm



Energy head in case 4 – h=45cm



Energy head in case 3 – h=50cm



Energy head in case 4 – h=50cm