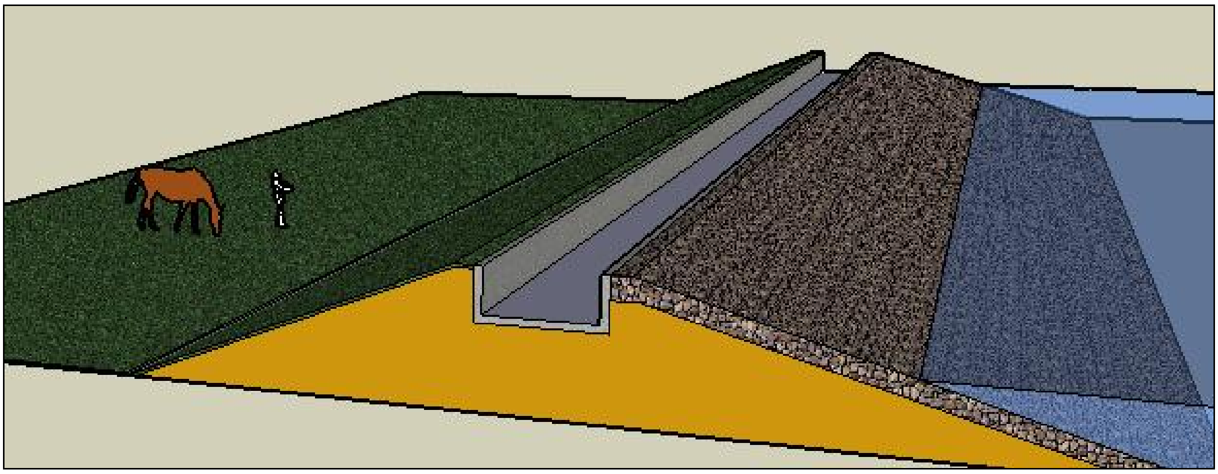
Final report

Wave overtopping aspects of the Crest Drainage Dike
A theoretical, numerical and experimental research



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Colophon

Final report of the master thesis
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Preface

This report is written as a master thesis of Delft University of Technology. The research is performed under the authority of the section of hydraulic engineering and is carried out from October 2006 until May 2007. The research has been carried out in close cooperation with Rijkswaterstaat, the Technical University of Braunschweig and DHV consultancy and engineering.

The main goal of this report is to obtain a proper insight in the physics of the Crest Drainage Dike and to predict the wave overtopping discharge of this type of dike.

The subject of this study gave many opportunities to use the knowledge and to improve the skills I have learned during my education in Delft. In the first place, the analytical way of thinking I have learned, was always a guiding line during all the elements such as the literature study, the execution of the physical experiments, the development of the numerical model, the interpretation of the obtained data and the execution of the case studies. Only because of this training in analytical thinking, I was able to teach myself necessary skills such as the use of Matlab or the setup of the physical experiments.

The struggle against the threats of the sea is something that always has to be improved. This can be done with traditional methods or with the development of new concepts. Even if only one out of every thousand new concepts gives an improvement of the safety against flooding, one should realize that this concept is only found by studying all thousand concepts. Therefore I hope that the theories and predictions in this report contribute in an indirect way to a better and more efficient design of sea dikes.

Paul van Steeg May 2007

Acknowledgements

In writing this report, I have been fortunate to have had help and inspiration from a lot of different people. I would like to thank my graduation committee, Professor J.K. Vrijling, H.J. Verhagen, W.S.J Uijttewaal, A. Kortenhaus, M.D. Groenewoud and M.K. Karelse, for being a great support during this project. The combination of committee members from two universities, Rijkswaterstaat and DHV gave an interesting mix of views on this subject.
Without the support of my parents and friends, and especially my best friend Willemijn, I was not able to complete my studies in this pleasant way. Therefore I would like to thank them for their contribution of this thesis.

Paul van Steeg May 2007

Executive summary

In the framework of the ComCoast project, the concept of the Crest Drainage Dike has been studied regarding the reduction of wave overtopping. This study only focuses on the average wave overtopping discharge. The basic concept of the Crest Drainage Dike is a basin, integrated in the crest of the dike, that collects overtopping water and thus reduces the load on the inner slope of the dike. The collected water in the crest basin is drained landward or seaward through pipes.

The main goal of this report is to identify the physical background of the concept of the Crest Drainage Dike and to predict the wave overtopping discharge as a function of hydraulic and geometric boundary conditions.

Therefore two different types of theoretical studies have been executed. The first study is process-based and serves as a basis for the numerical program that has been developed. Since this model is partly based on several assumptions, several physical model tests have been executed to verify or reject the stated hypotheses. In the physical model tests, several hydraulic and geometric boundary conditions, such as the wave height, the crest freeboard, the use of berms, the wave spectra, the wave steepness and the drain layouts, have been varied.

Since the predictions of the numerical program are well in line with the measured wave overtopping discharges, the numerical program is used to investigate the use of a Crest Drainage Dike in two case studies. The case studies are the Hondsbossche Sea Defence and the Perkpolder Sea Defence. Both dikes are located in the Netherlands

The use of this numerical program gives a better insight in the physical background of the Crest Drainage Dike. The description of these physics is the second part of the theoretical study.
For dikes with severe wave attack, such as the Hondsbossche Sea Defence, only a small fraction of the waves is reaching the crest of the dike. However, the waves that do reach the crest of the dike have a relatively large volume and the buffer capacity of the Crest Drainage Dike limits the effectiveness of the Crest Drainage Dike. Besides this, there is a high statistical uncertainty since the average wave overtopping discharges are determined by only a couple of waves.
For dikes with a lower wave attack, such as the Perkpolder Sea Defence, more waves with a lower volume per wave are overtopping and therefore the concept of the Crest Drainage Dike works well. However, the crest freeboard reduction with the use of the Crest Drainage Dike is in these specific cases is only minor.

Based on the numerical studies and the current Dutch overtopping criteria, the reduction of the crest freeboard with the use of the Crest Drainage Dike is determined and is significantly lower then the assumed reductions in earlier studies.

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List of symbols (report)

a	scale parameter Weibull distribution	(m^3/m)
A	empirical coefficient for wave overtopping	(-)
A_{drain}	surface of a drain	(m^2)
A_0	surface of a reservoir	(m^2)
B	width of berm measured horizontally	(m)
B	empirical coefficient for wave overtopping	(-)
c	dimensionless drain length	(-)
d_h	berm depth in relation to SWL (negative berm is above SWL)	(m)
D_{drain}	drain diameter	(m)
$Dist_{drains}$	distance between two drains	(m)
E_{nn}	variance density spectrum	(m^2/Hz)
f	frequency	(Hz)
f_{peak}	peak frequency	(Hz)
g	acceleration due to gravity	(m/s^2)
h	water level in the crest basin	(m)
h_{max}	maximum height of the crest basin	(m)
H	wave height	(m)
H_{m0}	significant wave height, based on the spectrum	(m)
H_s	significant wave height, average wave height of the 1/3 highest waves	(m)
L_{drain}	length of the drain	(m)
L_{berm}	horizontal length between two points on the slope 1.0 H_{m0} above and 1.0 H_{m0} below middle of berm	(m)
L_0	wavelength in deep water based on $T_{m-1.0}$	(m)
l.s.e	length spreading effect	(-)
m_0	surface of energy density spectrum	(m^2)
m_n	n^{th} moment of energy density spectrum	
N	number of incoming waves	(-)
N_{ov}	number of overtopping waves	(-)
P_v	$P(\underline{V}<V)$ probability that overtopping volume per wave $\underline{V}$ is greater than or same as V	(-)
P_{ov}	probability of overtopping waves ($P_{ov}=n_{ov}/n$)	(-)
q	average wave overtopping discharge per linear meter of crest (traditional dike)	($m^3/s/m$)
q_{drain}	average drained discharge per linear meter of crest	($m^3/s/m$)
q_{mao}	maximum allowed average wave overtopping discharge per linear meter of crest	($m^3/s/m$)
$q_{drain,max}$	maximum drain capacity per linear meter of crest	($m^3/s/m$)
$q_{overtopping}$	average wave overtopping discharge per linear meter of crest (Crest Drainage Dike)	($m^3/s/m$)
$q_{totalovertopping}$	average total overtopping discharge per linear meter of crest	($m^3/s/m$)
Q	average wave overtopping discharge	(m^3/s)
Q^*	dimensionless average wave overtopping discharge	(-)
R_c	crest freeboard in relation to SWL, at position of outer crest line	(m)
R_c^*	dimensionless crest freeboard	(-)
R_{CDD}	crest freeboard (Crest Drainage Dike)	(m)
s_0	wave steepness with L_0 based on $T_{m-1.0}$ ($s_0 = H_{m0}/L_0$)	(-)
t	Duration of a wave record	(s)
t	Duration of draining a crest basin	(s)
t_E	Duration of draining a crest basin until it is empty	(s)
$t_{E,dim}$	Dimensionless duration of draining a crest basin until it is empty	(-)
$t_{E,fundamental}$	non-linear dimensionless duration of draining a crest basin	(-)
$t_{E,L\_drain=0}$	Duration of draining a crest basin untill it is empty without the use of a drain	(s)
T	wave period	(s)
T_m	mean wave period	(s)
T_p	peak period	(s)
$T_{m-1.0}$	spectral wave period (m_{-1}/m_0)	(s)
v	drain velocity	(m/s)
V	volume of overtopping per wave per linear meter of crest (traditional dike)	(m^3/m)

V_{buffer}	buffer capacity of a crest basin	(m^3/m)
V^*_{buffer}	dimensionless buffer capacity	(-)
$V_{crest,end}$	Volume in the crest basin "at the end" of a wave period	(m^3/m)
$V_{crest,start}$	Volume in the crest basin directly after incoming wave	(m^2)
V_{drain}	Volume drained per wave	(m^3/m
$V_{overtopping}$	overtopping volume per wave	(m^3/m)
$V_{totalovertopping}$	total overtopping volume per wave	(m^3/m)
$z_{2\%}$	wave run-up height exceeded by 2% of incoming waves	(m)
Z	reference height for draining a reservoir	(m)
α	angle of average slope	(°)
α	energy scale parameter	(-)
β	angle of wave attack	(°)
γ	peak enhancement factor	(-)
γ_b	influence factor for a berm	(-)
γ_{CDD}	influence factor for a crest drainage dike	(-)
γ_f	influence factor for roughness elements	(-)
γ_v	influence factor for a vertical or very steep wall on a slope	(-)
γ_β	influence factor for angle of wave attack	(-)
θ	fraction of the crest basin that is filled	(-)
μ	entrance loss	(-)
ξ_0	breaker parameter based on $T_{m-1.0}$: ξ_0=tan α/sqrt(s_0)	(-)
σ	peak width parameter	(-)
φ	fraction of the overtopping water that is trapped in the crest basin and drained	(-)

List of symbols (numerical program)

A	Surface cross section crest basin	m^2
a	Scalefactor of the Weibull distribution	m^2
A0	Surface of the crest basin between two drains	m^2
b	Assist parameter to determine the drain capacity	
br	Counter for number of repetitions	-
c	Linear correction factor	-
Ddrain	Diameter of the discharge drains	m^2
Distdrains	Distance between the discharge drains	m
e	Counter for number of situation	-
Efficiencycrest	Percentage of max filling of the crest basin	-
Entrancefriction	Entrancefriction of the drains	-
error	Discrepancy between numerically calculated average wave overtopping discharge and input wave overtopping discharge	%
g	Acceleration due to gravity	m/s^2
g_beta,	Influence factor for angle of attack	-
g_berm	Influence factor for berms	-
g_friction	Influence factor for friction	-
hcrest	Water level in the crest basin	m
Heightcrest	Height of the crest basin	m
Hm0	Significant wave height	m
hmax	Maximum water level in the crest basin	m
i	Wave number in the record	-
ksi	Irribarren parameter	-
L	Wave length	m
Ldrain	Length of the drain	m
maxdrainvolume	Maximum drain volume during one wave	m^2
mu	Entrance factor for drain capacity	-
n	Number of waves in the wave record	-
Nov	Total number of waves overtopping the front of the crest drainage dike	-
NrOfWavesDuringEmptying	Number of waves during emptying a filled crest basin	-
NumberOfWaves	Number of waves in the wave record	-
overtopping	Determination whether the wave overtops the front of the Crest Drainage Dike. 1=overtopping 0=no overtopping	-
parameters	Input parameters	-
percentageOvertopping	Percentage of overtopping water during the wave record	-
Pov	Probability on overtopping per wave (analytically calculated value)	-
Povcheck	Probability on overtopping per wave (numerically calculated value)	-
q	Input value of average overtopping discharge	m^2/s
q_check	Average overtopping discharge (numerically calculated)	m^2/s
q_crestmeasured	q_crest measured in physical experiments	m^2/s
q_drainmean	Average drained discharge (numerically calculated value)	m^2/s
q_overtopmeasured	q_overtopping measured in physical experiments	m^2/s
q_overtopmean	Average overtopping discharge	m^2/s

q_totalovertopmean	Average total overtopping discharge	m^2/s
Rc	Crest freeboard	m
s0	Wave steepness	-
slope	Outer slope of the dike (tanα)	-
subset	Subset of input parameters	-
t	Time	s
T	Wave period	s
t_full	Time to empty a filled crest basin	s
t_storm	Duration of the storm	s
testnr	Number of experiment	-
testpercentage	Percentage overtopping according to the physical tests	-
Theta	Linear factor	-
V2crestmax	Buffer volume with a length spreading effect of two	m^2
Vcrest	Volume of water in the crest	m^2
Vcrestend	Volume of water in the crest basin at the end of a wave period	m^2
Vcrestmax	Maximum volume of water in the crest basin	m^2
Vcreststart	Volume of water in the crest basin directly after the wave impact	m^2
Vdrain	Drained volume during one wave period	m^2
Vovertopping	Overtopped volume during one wave period	m^2
Vcumdrain	Cumulative drained volume	m^2
Vcumovertop	Cumulative overtopping wave volume	m^2
Vcumwave	Cumulative incoming wave volume	m^2
Vw	Incoming wave volume during one wave	m^2
Widthcrest	The width of the crest basin	m
z2	Wave run-up height exceedance by 2% of the incoming waves	m

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1 Introduction

1.1 Background of the Crest Drainage Dike

Based on several separately initiated studies on possible advantages and opportunities for overtopping dikes, a European project called ComCoast (Combined Functions in the Coastal zone) was set up in 2004. The ComCoast project is carried out in the framework of the Interreg IIIb- North Sea program. Interreg III is a Community initiative, which aims to stimulate interregional cooperation in the EU between 2000-2006. It is financed under the European Regional Development Fund (ERDF).

One of the objectives of the ComCoast project is to come up with possibilities of a wider coastal defence zone. Instead of raising and strengthening the dike, the coastal defence zone is widened. This provides opportunities for new spatial developments and different types of functions within the zone. [DHV, 2005]

1.2 The Crest Drainage Dike

Within the framework of ComCoast, the CUR (Civieltechnisch Centrum Uitvoering Research en Regelgeving) issued a request to several parties to develop possible innovative concepts for overtopping dikes. DHV (Dwars, Heederik en Verhey consultancies) responded on this with the concept of the Crest Drainage Dike. The concept of this alternative is to catch the overtopping water in a construction, which is integrated in the crest of the dike. The water caught is discharged through drains either on the inner side of the dike or the outer side of the dike. The conceptual design is given in figure 1-1.

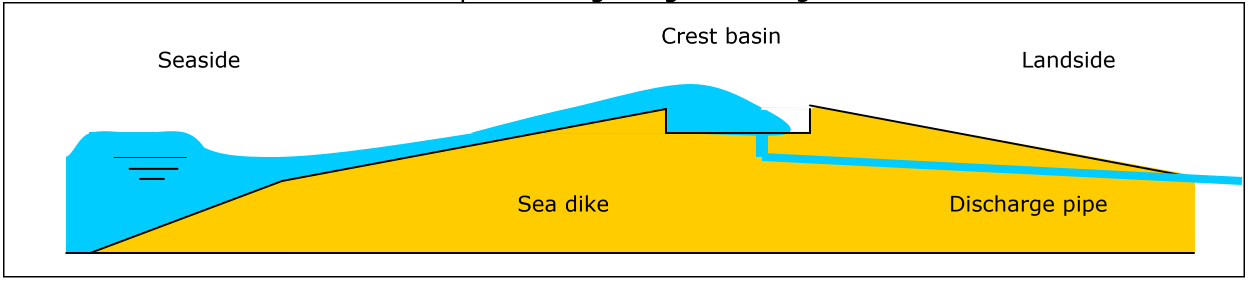


figure 1-1: Conceptual design of the Crest Drainage Dike

The CUR selected this concept to be worked out in more detail. This theoretical study is executed by DHV [DHV, 2005]. In this study it was concluded that "the Crest Drainage Dike is technically and financially feasible and offers good opportunities for recreational and environmental development. The concept requires some further development research however." According to this report, the main technical aspect of the Crest Drainage Dike that requires further research is the amount of overtopping water that is trapped in the crest construction in relation to the remaining overtopping flow.

The next step within ComCoast is to gather further insight into the wave overtopping discharges of the Crest Drainage Dike.

1.3 Problem analysis

In this section, a description of the problem is given. Following from this description, a problem definition is formulated.

1.3.1 Problem description

To determine the feasibility of the Crest Drainage Dike, reference is made to the alternative where traditional dike heightening is applied. The feasibility of the Crest Drainage Dike is largely dependent on the reduction of the crest freeboard when using a Crest Drainage Dike instead of a traditional dike. If this reduction is, for example, 10 cm, the feasibility of the Crest Drainage Dike is less then a situation where a dike heightening of 3 meters is avoided.

The freeboard reduction depends largely on the efficiency of the Crest Drainage Dike. In previous studies, [DHV, 2005], it has been shown that, based on a certain efficiency, the Crest Drainage Dike is feasible. However, it is stressed that the efficiency of the Crest Drainage Dike is only an assumption based on some rough calculations.

1.3.2 Problem definitions

Since there is a lack of physical insight in the efficiency of the Crest Drainage Dike, it is not possible to predict the wave overtopping discharges for this type of dike. Without this information, there is no basis for a proper feasibility study.

1.3.3 Objective

The objective of this report is to gather physical insight in the overtopping aspects of the Crest Drainage Dike and to formulate a model that predicts the average wave overtopping discharge.

1.4 Approach for this study

To get a proper insight in the physical background of overtopping theories regarding the Crest Drainage Dike, use has been made of overtopping theories regarding traditional dikes. This serves as a basis for the overtopping theories that are developed in this report.
Two tracks have been followed. The first track is the development of physical model which is process-based. Here, every single wave that approaches the Crest Drainage Dike is observed, analysed and described. This resulted in a numerical program that can predict the efficiency of the Crest Drainage Dike. This model is based on several hypotheses. To verify or reject the hypotheses, several physical model tests have been executed. The data obtained from these experiments serves as a reference of the numerical model.
The second track is a study where the influence of the geometric and hydraulic parameters is studied. This analysis does not result in a model that can predict efficiencies, but gives a clear understanding how and why the various parameters have a certain weight. This analysis is executed with the use of dimensionless parameters. The second track is partly based on the results obtained from the developed numerical model. This means that the physical background of the second track is already "hidden" in the numerical model. Therefore, it is stated that the second track is only followed to make the physics visible.
Since both tracks are quite theoretical, the developed theories have been projected on four case studies. Two of the cases are fictive dikes with simple geometric and hydraulic boundary conditions. Two study cases are real existing dikes. These are the Hondsbossche Sea Defence and the Perkpolder Sea Defence. Both dikes are situated in the Netherlands.

1.5 How to read this report

This report is written in chronological order. The advantage of this is a clear insight in how the theories are developed. A slight disadvantage is that some theories are sliced into several blocks and sometimes adapted in a later stage. This is always indicated at that specific part.

A study of the overtopping theories regarding traditional dikes is given in chapter 2. An extension of this theory is applied in chapter 3. Here, the overtopping theories are applied to the Crest Drainage Dike. In this chapter, the process-based theories, which result in a numerical model (section 3.4), are explained. Besides this, the influences of the several hydraulic and geometric parameters are studied as well.
Since the numerical model is based on several hypotheses, physical model tests have been executed. A description of these experiments is given in chapter 4. The results of the physical experiments are analysed in chapter 5. This analysis requires some adaptations of the numerical model. Therefore a feedback on the model is given in chapter 6. To obtain experience with the numerical model, two case studies have been

used. This is described in chapter 7. With the use of these case studies, some better understanding regarding the influences of the several parameters is obtained. This is described in section 7.5. Two real existing dikes have been analysed with the use of the numerical model in section 8.3 and 8.4. Based on the theories, the numerical model, the physical model tests and the case studies, a conclusion is given in chapter 8. The above stated process is shown in figure 1-2.

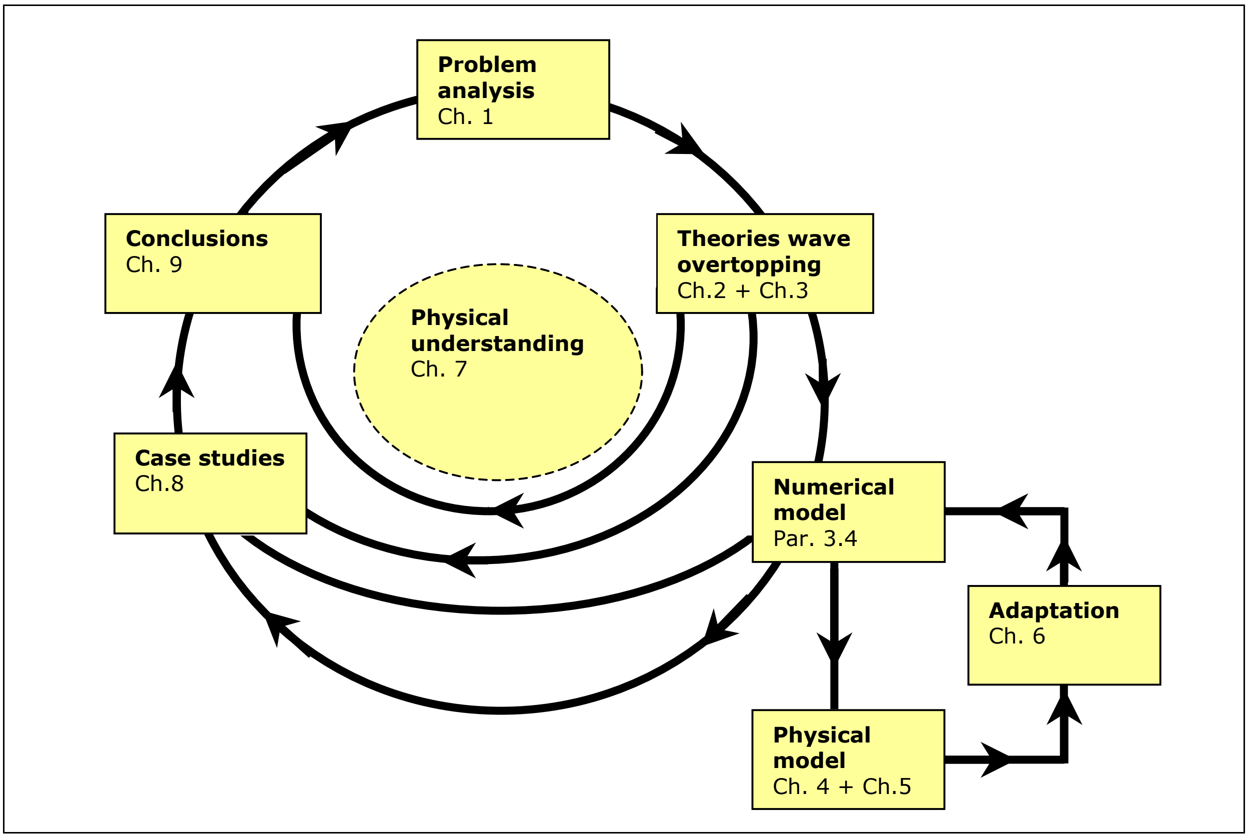
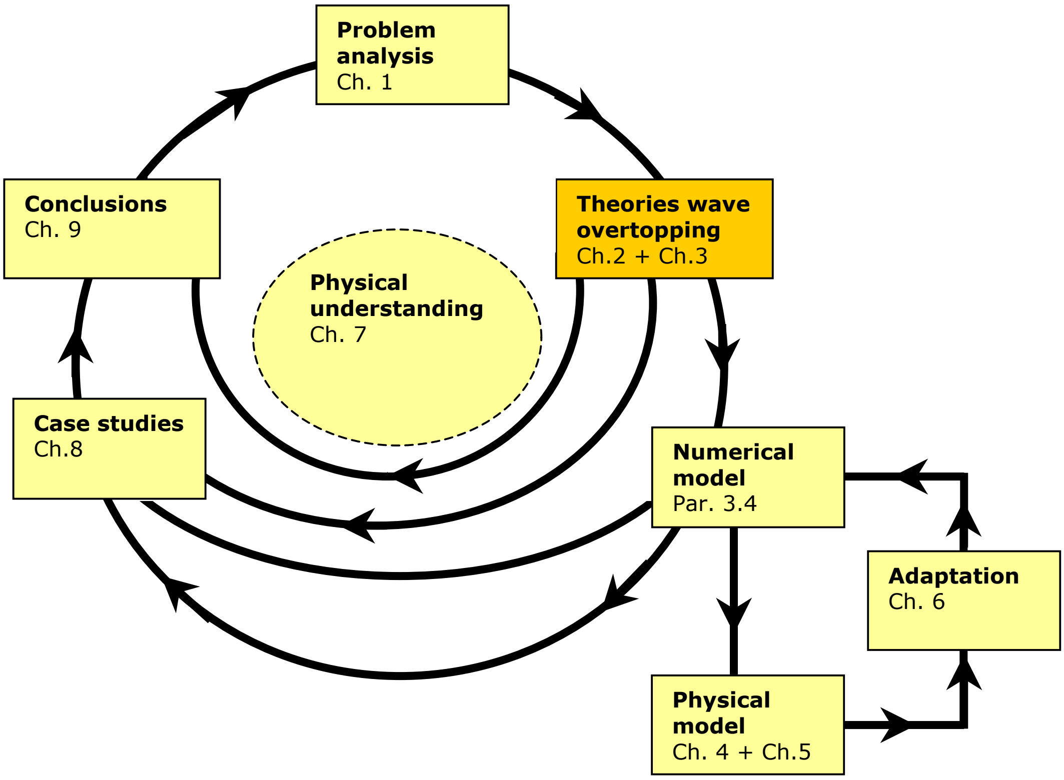


figure 1-2: Overview set up of the report

2 Wave overtopping theory: traditional dikes



2.1 Introduction

Overtopping discharge occurs as a result of waves running up the slope of the seawall. This report does not directly examine the wave run-up but concentrates on the resulting discharge rates. A description of wave run-up can be found in [CIRIA, 2007] and [van der Meer, 2002]. Since this report focuses only on wave overtopping, these theories will be described below. The analysis of the wave overtopping for a traditional dike will form a part of the theories developed for the Crest Drainage Dike. (Chapter 3)

Wave overtopping is usually given as an average discharge per meter of width. Usually this is expressed in m^3/s per m or liters/s per m. Average overtopping rates that are accepted in the Netherlands are [TAW, 1989]:

- 0.1 l/s per m for sandy soil with a poor grass cover
- 1.0 l/s per m for clayey soil with a reasonable good grass cover
- 10l/s per m for a clay covering and a grass cover according to the requirements for the outer slope or for an armored inner slope.

There is still research ongoing to substantiate a better relationship between wave overtopping and the capacity of the inner slope. A method is also given in the Guideline on Safety Assessment. [DWW, 2004] At the moment of writing this report, full-scale wave-overtopping tests regarding the strength of grass are being executed. [ComCoast, 2007] To determine the average overtopping rates, use can be made of [van der Meer, 2002] and [Besley,1999]:, the latter is based on the experimental work of [Owen, 1980]. All these prediction methods have intrinsic limitations to their accuracy since they are based on physical model data. The physical model data from which the design equations have been derived generally exhibit significant scatter. A study by [Douglas, 1985] concluded that calculated overtopping rates, using empirically derived equations, should only be regarded as being within, at best, a factor of 3 of the actual overtopping rate.

General aspects of wave overtopping will be discussed in section 2.2. The influence of a berm and the influence of different wave spectra will be discussed in section 2.3 and section 2.4. Since these theories only consider average overtopping discharges, some basics about solitary overtopping theories will be explained in section 2.5.

2.2 General aspects of wave overtopping

A typical schematisation of a dike is given in figure 2-1. The most important parameters such as the significant wave height (H_s), the wave period ($T_{m-1.0}$) and the freeboard (R_c) are shown.

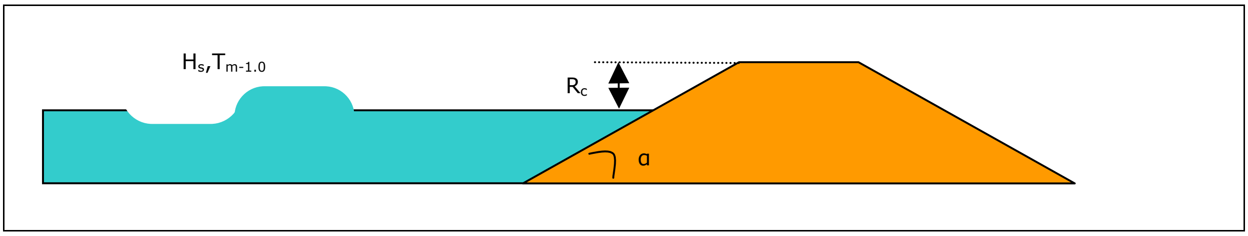


figure 2-1: Schematisation of a traditional dike

The wave that approaches the dike, might break. For waves breaking on a slope, the dimensionless Irribarren number or surf similarity parameter is of crucial importance in all kind of shore problems. The parameter is defined as:

$$\xi_0 = \frac{\tan\alpha}{\sqrt{\frac{H_s}{L_0}}} \qquad \textbf{Equation 2-1}$$

where

ξ_0	=	breaker parameter	(-)
tan α	=	slope steepness	(-)

L_0	=	wavelength in deep water	(m)
H_s	=	significant wave height (average wave height of 1/3 highest waves)	(m)

$$L_0 = \frac{gT_{m-1.0}^2}{2\pi}$$ **Equation 2-2**

where

g	=	acceleration due to gravity	(m/s^2)
$T_{m-1.0}$	=	spectral wave period (m_{-1}/m_0)	(s)

Several breaker types are shown in figure 2-2.

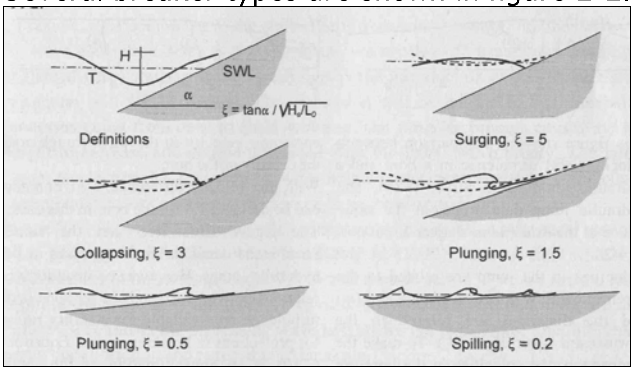


figure 2-2: Breaker types [Schiereck, 2001]

2.2.1 *The use of dimensionless parameters*

In wave overtopping formulae, a dimensionless crest freeboard (R_c*) and a dimensionless overtopping discharge (Q*) are usually used. The exponential relation between these parameters is as follows:

$$Q^* = A \cdot e^{BR_c^*}$$ **Equation 2-3**

Where

A	=	empirically derived coefficient	(-)
B	=	empirically derived coefficient	(-)
R_c^*	=	dimensionless crest freeboard	(-)
Q^*	=	dimensionless average wave overtopping discharge	(-)

The coefficients A and B are still functions of the wave height, slope angle, breaker parameter and several influence factors such as the berm, friction, angle of wave attack and a vertical wall on the slope.

The dimensionless discharge and the dimensionless crest freeboard can be constructed in several ways. An overview of dimensionless parameters that are used by several researchers is given in appendix I, table A-1

2.2.2 *Overtopping rates according to Owen.*

In [Besley, 1999], a method to predict average overtopping discharges is given. This is based on the experimental work, which is presented in [Owen, 1980]. Owen proposed a design method, which is widely used in the civil engineering industry, to calculate the average overtopping discharge on a simply sloping seawall. In this method the discharge and freeboard are made dimensionless as follows:

$$Q^* = \frac{Q}{T_m g H_s}$$ **Equation 2-4**

and

$$R^* = \frac{R_c}{T_m\sqrt{gH_s}}$$ **Equation 2-5**

With

R_c	=	crest freeboard in relation to SWL, at position of outer crest line	(m)
T_m	=	mean wave period	(s)

The dimensionless discharge, Q*, and the freeboard, R*, are related by the following equation:

$$Q^* = Ae^{(-BR^*)}$$ **Equation 2-6**

Where A and B are empirically derived coefficients which depend on the profile of the seawall. Owen derived, or interpolated, values of A and B for simply sloped seawalls ranging in the slope angle from 1:1 to 1:5, these are shown in appendix II, table A-2. An example is given in the box below:

Example

Consider a dike with a smooth slope of 1:4. The angle of attack of the waves is perpendicular to the dike (γ_β=1), no berm is applied (γ_b=1) and there is no friction. (γ_f=1). The relation between Q* and R* can be calculated with the use of Equation 2-6 and table A-2 from appendix II:

$$Q^* = 0.0116e^{(-41.0R^*)}$$

Besides this 'basic' sea-dike, several other aspects such as berms (which will be discussed in section 2.3), slope-roughness, angle of wave attack and returning walls are discussed in [Besley, 1999]. Since the latter three are not in the area of interest of this report, these will not be discussed. However, more information can be found in [Besley, 1999]

2.2.3 Overtopping rates according to van der Meer

The relations given by [van der Meer, 2002] are commonly used in the Netherlands regarding the design of dikes. Van der Meer describes wave overtopping in two formulae, which are linked to each other: one for breaking waves ($\gamma_b\xi_o$ <≈ 2, with γ_b is the influence factor for berms), where wave overtopping increases for increasing breaker parameter ξ_o, and one for the maximum that is achieved for non-breaking waves ($\gamma_b\xi_o$ >≈ 2).

The wave overtopping formula is based on the exponential function with the general form

$$q = A \cdot e^{B \cdot R_c}$$ **Equation 2-7**

With

$$A = \sqrt{gH_{m0}{}^3} \cdot \frac{0.067}{\sqrt{\tan\alpha}} \cdot \xi_0$$ **Equation 2-8**

$$B = \frac{-4.3}{H_{m0}} \cdot \frac{1}{\xi_0\gamma_b\gamma_f\gamma_\beta\gamma_v}$$ **Equation 2-9**

where

H_{m0}	=	significant wave height based on the wave spectrum	(m)
γ_b	=	influence factor a berm	(-)
γ_β	=	influence factor for angle of attack	(-)
γ_f	=	influence factor for roughness elements	(-)
γ_v	=	influence factor for a vertical or very steep	(-)

wall on a slope

(It can be seen that van der Meer uses a different definition of the significant wave height than Owen).

This results into the following overtopping formulae:

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\xi_0 \cdot e^{-4.3\frac{R_c}{H_{m0}}\frac{1}{\xi_0\gamma_b\gamma_f\gamma_\beta\gamma_v}}$$

Equation 2-10

With a maximum of

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2e^{-2.3\frac{R_c}{H_{m0}}\frac{1}{\gamma_f\gamma_\beta}}$$

Equation 2-11

2.3 The influence of a berm

Both Owen and van der Meer did research on the influence of a berm regarding the wave overtopping discharge. The most important parameters in both theories are the berm width (B) and the berm elevation (d_h). An impression of the dimensions of a berm is shown in figure 2-3. Both theories (Owen and van der Meer) will be described in this section.

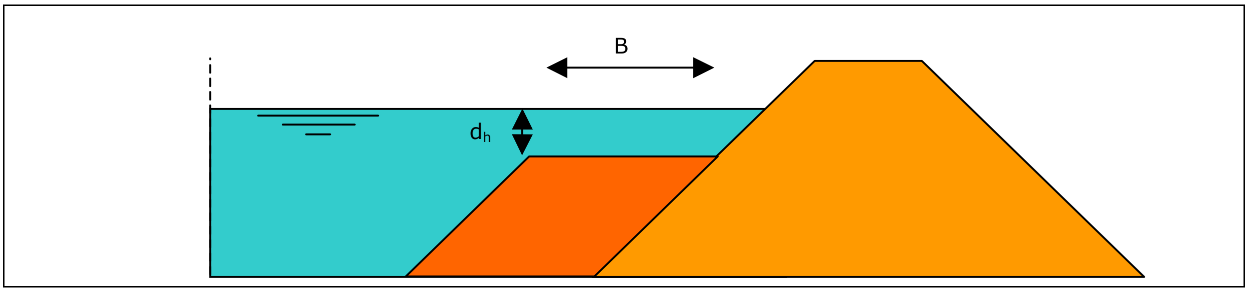


figure 2-3: Schematisation of a berm in front of a sea dike

2.3.1 Influence of a berm according to Owen

Owen found that his empirical relation could easily be applied to bermed structures, however, modified empirical coefficients should be applied. These coefficients depend on the berm width and the berm elevation. Owen derived empirical parameters for several berm layouts. These parameters are shown in [Owen, 1980].

2.3.2 Influence of a berm according to van der Meer

Van der Meer introduced a berm reduction coefficient γ_b in both the dimensionless crest freeboard (R*) and the dimensionless overtopping discharge (Q*). The berm reduction factor depends on the berm width (B) and the berm elevation (d_h). For details of this theory, reference is made to [van der Meer, 2002]. The results of the van der Meer theories regarding berms are shown in figure 2-4.

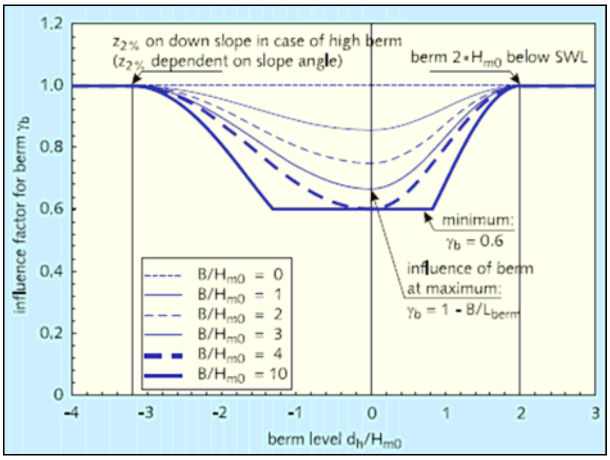


figure 2-4: Influence factor for the use of a berm [van der Meer, 2002]

From this figure, it can be seen that a berm is most effective if it lies on the still water line. ($d_h/H_{m0}=0$). The berm width is optimal when the factor γ_b gets close to 0.6.

2.4 The influence of different wave spectra

Van der Meer uses the spectral period $T_{m-1,0}$ This period can be determined in the following way:

$$T_{m-1,0} = \frac{m_{-1}}{m_0}$$ **Equation 2-12**

where

m_n	=	n^{th} moment of the wave spectrum.	

m_n can be calculated as follows [Battjes, 2001]:

$$m_n = \int_0^\infty f^n E_{\eta\eta}(f)df \quad \text{(n=0,1,etc)}$$ **Equation 2-13**

where

f	=	wave frequency	(Hz)
$E_{\eta\eta}$	=	variance density spectrum	(m^2/Hz)

The wave period $T_{m-1.0}$ gives more weight to the longer periods in the spectrum than the average wave period (T_m) and, independent of the type of the spectrum, it provides the corresponding wave overtopping for the same values and the same heights. In this way, wave run-up and wave overtopping can easily be determined for double peaked and 'flattened' spectra, without the need for other difficult procedures. This theory is supported by physical model tests on wave run-up [van Gent, 2001] and wave overtopping [van Gent 1999].

Van der Meer uses a fixed relationship between the spectral period ($T_{m-1,0}$) and the peak period (T_p). In [van der Meer, 2002] a conversion factor (T_p = 1.1$T_{m-1,0}$) is given. This can be done when a uniform spectrum with a clear peak exist. It is stressed that, for cases where the spectrum has no uniform shape or where no clear peak period is given, $T_{m-1,0}$ should be determined by spectral analysis.

2.5 Solitary wave overtopping on a traditional dike

The average wave overtopping discharge does not say much about the amount of water that will overtop a dike for a single wave. The wave overtopping volumes per wave differs substantially from the average wave overtopping discharge. Using the average wave overtopping discharge (q), the probability distribution function for the wave overtopping volume per wave has been calculated. This is described in [van der Meer, 2002]. The probability distribution function is a Weibull distribution with a shape factor of 0.75 and a scale factor a, which depends on the average wave overtopping discharge and the probability of overtopping waves. [van der Meer, 2002].

The probability distribution function is given by:

$$P_v = P(\underline{V} \le V) = 1 - e^{\left[-\left(\frac{V}{a}\right)^{0.75}\right]}$$ **Equation 2-14**

With

$$a = 0.84 \cdot T_m \cdot \frac{q}{P_{ov}}$$ **Equation 2-15**

where:

P_v	=	probability that wave overtopping per wave V is greater than or same as $\underline{V}$.	(-)
V	=	wave overtopping volume per wave	(m^3/m)
a	=	scale factor of a Weibull distribution	(m^2)
T_m	=	mean wave period (N·T_m is duration of storm of	(s)

		examined period, where N is the number of waves in a storm)	
q	=	average wave overtopping discharge per linear meter of crest	(m^3/m/s)

It is assumed that the wave run-up distribution conforms to the Rayleigh distribution. In this case, the probability of overtopping (P_{ov}) can be calculated as follows:

$$P_{ov} = \exp\left[-\left(\sqrt{-\ln 0.02}\,\frac{R_c}{z_{2\%}}\right)^2\right]$$ **Equation 2-16**

with

$$z_{2\%} = 1.75\gamma_b\gamma_f\gamma_\beta\xi_0 H_{m0}$$ **Equation 2-17**

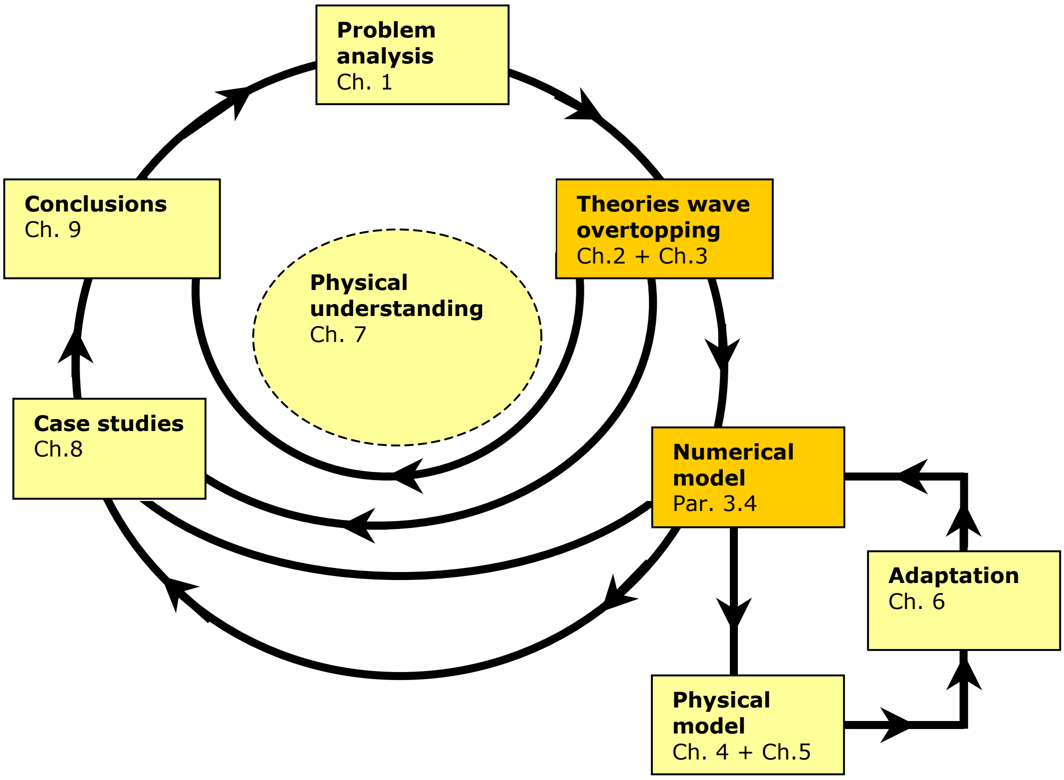
where:

P_{ov}	=	probability of waves passing line 1 ($P_{ov}=N_{ov}/N$)	(-)
R_c	=	crest freeboard in relation to SWL	(-)
$z_{2\%}$	=	2% wave run-up level above the still water line	(m)
H_{m0}	=	significant wave height at toe of the dike	(m)
ξ_0	=	breaker parameter based on $T_{m-1.0}$	(-)
N_{ov}	=	number of overtopping waves	(-)
N	=	number of incoming waves per storm	(-)
γ_b	=	influence factor for a berm	(-)
γ_f	=	influence factor for roughness elements on slope	(-)
γ_β	=	influence factor for angled wave attack	(-)

2.6 Conclusions

Wave overtopping formulae regarding traditional dikes are highly empirical. The commonly used overtopping theories according to Owen and van der Meer have been explained. However, the predicted overtopping rate must be considered as only a factor of 3 of the actual overtopping rates. The influence of a berm and the use of the spectral wave period have been explained as well as the theory regarding solitary wave overtopping.
An overview of wave-overtopping theories regarding traditional dike is given. These theories are derived for simple geometries and do not cover the geometry of the Crest Drainage Dike. Therefore, wave-overtopping theories regarding the Crest Drainage Dike are necessary. This is the subject of the following chapter.

3 Wave overtopping theory: Crest Drainage Dike



3.1 Introduction

In the previous chapter, an overview of overtopping theories regarding traditional dikes has been given. In this chapter, a theoretical approach of the overtopping theories of the Crest Drainage Dike will be given. Since there are hardly publications on wave overtopping that apply to a Crest Drainage Dike, specific wave overtopping theories are developed in this chapter. The goal of this chapter is twofold;

- A development of physical insight in the physical process of the Crest Drainage Dike and its efficiency.
- The development of a model that can predict the efficiency of the Crest Drainage Dike.

Although these goals look like they are the same, there is a significant difference. A physical understanding gives the possibility to understand the influence of the several parameters. The model can only predict the efficiency of the Crest Drainage Dike and might still "hide" the underlying physical processes.

Before this will be done, there is a need to analyze the crest freeboard reduction with the use of a Crest Drainage Dike. This will be done in section 3.2. Section 3.3 discusses some basic design aspects of the Crest Drainage Dike. Several assumptions regarding the physical processes are adopted in section 3.4. Given a described design and several assumptions, a process-based model is developed in 3.5. Section 3.6 deals with the physical insights of the Crest Drainage Dike. In section 3.7, the influence of hydraulic and geometric boundary conditions is discussed. Following from the above stated analysis, hypotheses are collected and presented in section 3.8.

3.2 Analysis of crest freeboard reduction with the use of a crest structure

When applying the Crest Drainage Dike, two wave-overtopping discharges will occur; water will overtop the dike and will be drained through the crest structure and water will overtop the dike and run down the inner slope. This is based on the assumption that no water will be reflected by the crest basin.

One can express the amount of water which is trapped in the crest construction as a fraction of the total overtopping water. (e.g. 70% of the total overtopping water is trapped in the crest construction). This amount of water is given the symbol φ

However, this fraction φ does not express how much crest freeboard reduction can be applied. The aim of this section is to find an expression for the crest freeboard reduction (or avoidance of crest freeboard heightening) which can be applied when using the Crest Drainage Dike. This crest freeboard reduction is given the symbol γ_{cdd} (CDD stands for Crest Drainage Dike). The above described has been subject to a mathematical analysis which is described below.

The crest freeboard of a Crest Drainage Dike can be written as:

$$R_{CDD} = \gamma_{CDD} \cdot R_c$$ **Equation 3-1**

with

$$\gamma_{CDD} = f(\varphi)$$ **Equation 3-2**

And

$$\varphi = f(q_{mao}, g, H_{m0}, \tan\alpha, s_0, \gamma_x)$$ **Equation 3-3**

where

R_{CDD}	=	The crest freeboard when applying a Crest Drainage Dike	(m)
R_c	=	The crest freeboard without the use of a Crest Drainage Dike	(m)

γ_{CDD}	=	The crest freeboard reduction factor when applying a Crest Drainage Dike	(-)
φ	=	The fraction of water which is trapped in the Crest Drainage Dike	(-)
q_{mao}	=	The maximum allowed overtopping water	(m^3/s)
g	=	Acceleration due to gravity	(m/s^2)
H_{m0}	=	The significant wave height	(m)
$\tan\alpha$	=	The angle of the slope	(-)
s_0	=	The wave steepness	(-)
γ_x	=	The various influence factors such as berms etc.	(-)

The overtopping discharge for a traditional sea-dike with no berm, perpendicular wave attack, and no crest wall, has already been discussed in chapter 2. Equation 2-10 (page 9) is rewritten and gives:

(For simplicity reasons all the reduction factors for berms, angles of attack etc. are not shown).

$$q_{overtopping} = \sqrt{gH_{m0}{}^3} \cdot \frac{0.067}{\sqrt{\tan\alpha}} \cdot \frac{\tan\alpha}{\sqrt{s_0}} \cdot e^{\frac{-4.3}{H_{m0}} \frac{\sqrt{s_0}}{\tan\alpha} \cdot R_c}$$ **Equation 3-4**

where

$q_{overtopping}$	=	average wave overtopping discharge per linear meter of crest	($m^3/s/m$)

Suppose a dike is designed in such a way that the maximum allowed overtopping discharge equals the overtopping discharge:

$$q_{mao} = q_{overtopping} = \sqrt{gH_{m0}{}^3} \cdot \frac{0.067}{\sqrt{\tan\alpha}} \cdot \frac{\tan\alpha}{\sqrt{s_0}} \cdot e^{\frac{-4.3}{H_{m0}} \frac{\sqrt{s_0}}{\tan\alpha} \cdot R_c}$$ **Equation 3-5**

or

$$q_{mao} = q_{overtopping} = A \cdot e^{B \cdot R_c}$$ **Equation 3-6**

with

$$A = \sqrt{gH_{m0}{}^3} \cdot \frac{0.067}{\sqrt{\tan\alpha}} \cdot \frac{\tan\alpha}{\sqrt{s_0}}$$ **Equation 3-7**

and

$$B = \frac{-4.3}{H_{m0}} \frac{\sqrt{s_0}}{\tan\alpha}$$ **Equation 3-8**

A new parameter is introduced:

q_{drain}	=	average drained discharge per linear meter of crest	($m^3/s/m$)

The physical meaning of this parameter is shown in figure 3-1.

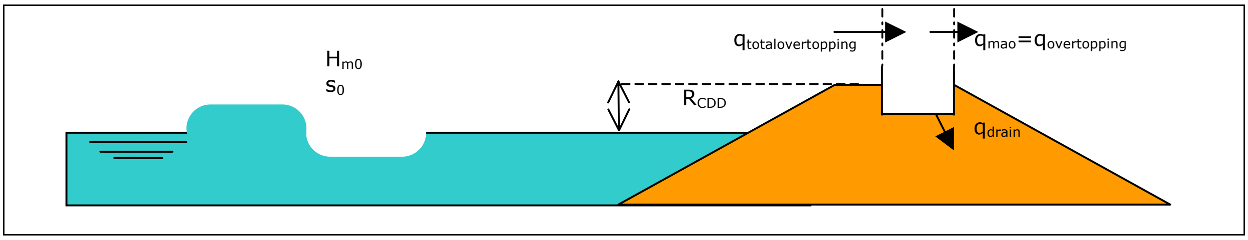


figure 3-1: Schematization of the various discharges

The following relationship yields:

$$q_{totalovertopping} = q_{mao} + q_{drain}$$ **Equation 3-9**

The q_{crest} will be written as a function of the fraction parameter φ:

$$q_{drain} = \varphi \cdot q_{totalovertopping}$$ **Equation 3-10**

Combining Equation 3-9 and Equation 3-10 gives:

$$q_{totalovertopping} = q_{mao} + \varphi \cdot q_{totalovertopping}$$ **Equation 3-11**

Rewriting gives:

$$q_{totalovertopping} = \frac{q_{mao}}{1-\varphi}$$ **Equation 3-12**

Rewriting Equation 2-7(chapter 2 on page 8) gives:

$$R_c = \frac{\ln(\frac{q}{A})}{B}$$ **Equation 3-13**

Combining Equation 3-12 and Equation 3-13 gives:

$$R_{CDD} = \frac{\ln(\frac{q_{mao}}{(1-\varphi)A})}{B}$$ **Equation 3-14**

and

$$R_c = \frac{\ln(\frac{q_{mao}}{A})}{B}$$ **Equation 3-15**

To introduce a Crest Drainage Dike reduction factor, the following parameter is constructed (using Equation 3-14 and Equation 3-15).

$$\boxed{\gamma_{CDD} = \frac{R_{CDD}}{R_c} = \frac{\ln(\frac{q_{mao}}{(1-\varphi)A})}{\ln(\frac{q_{mao}}{A})} = 1 - \frac{\ln(1-\varphi)}{\ln(\frac{q_{mao}}{A})}}$$ **Equation 3-16**

As can be seen in Equation 3-16, the crest reduction factor γ_{CDD} depends on three factors:

- the fraction of water trapped in the crest: φ
- the maximum allowed overtopping water q_{mao}
- the factor A which is a function of H_{m0}, s_0, $\tan\alpha$ and g — H_{m0}, s_0, $\tan\alpha$, g
  Reference is made to Equation 3-7 (page 17) for a definition of A

To obtain some feeling with Equation 3-16, the factors s_0, $\tan\alpha$ and g are kept constant; this leaves only H_{m0}, q_{mao} and φ as variables.

Suppose that:

- s_0 = 0.03
- $\tan\alpha$ = 0.25
- g = 9.81 m/s^2
- no influence of berms, angle of wave attack, friction etc.

The relationship between the freeboard reduction factor (γ_{CDD}), the crest basin efficiency (φ or phi), the maximum allowed overtopping (q_{mao}) and the significant wave height (H_{m0}) is plotted in figure 3-2. This graph is only true for the values as stated above.

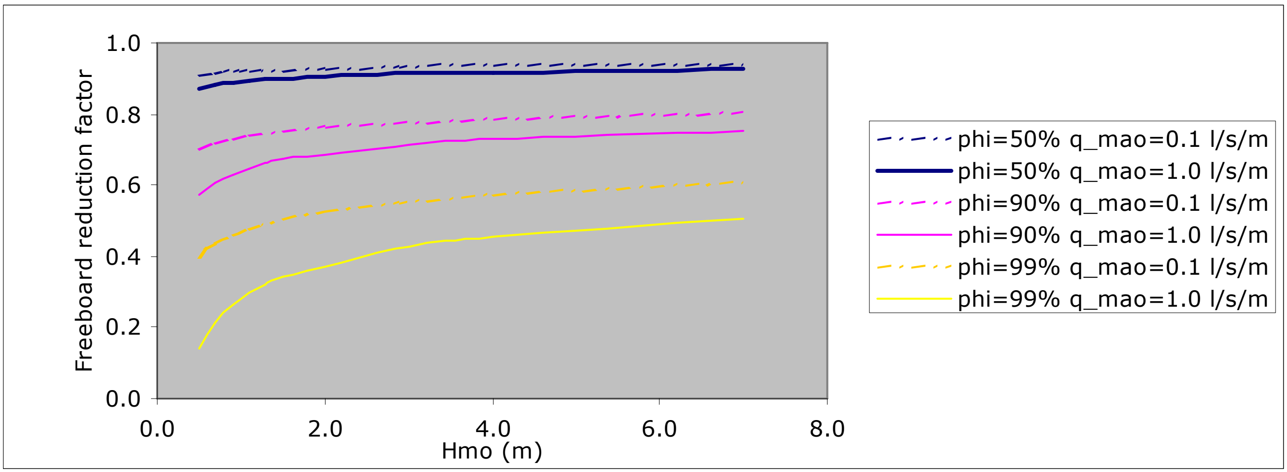


figure 3-2: The crest structure reduction factor γ_{CDD} as function of the wave height (H_{m0}) , the crest basin efficiency (φ) and the maximum allowed average wave overtopping discharge (q_{mao}).

Example
A dike with the following conditions is given:

• Wave steepness	s_0	=	0.03	-
• Maximum average wave overtopping discharge	q_{mao}	=	0.001	$m^3/s/m$
• Slope	$\tan\alpha$	=	0.25	-
• Significant wave height	H_{m0}	=	4	m
• Sum of gamma's	γ_x	=	1	-

Suppose it is possible to collect 90 percent of the overtopping water in the crest construction (φ=0.90). How much can the crest freeboard be lowered?

Using a traditional dike, the crest freeboard is calculated with the use of Equation 2-8, Equation 2-9 and Equation 2-10. This gives a freeboard of R_c=11.4m

To calculate the crest freeboard R_{CDD} with the use of a crest construction, assuming 90 percent reduction of the average wave overtopping discharge, one can use the crest structure reduction factor γ_{CDD}. Applying Equation 3-16 gives a crest structure reduction coefficient of 0.729. Thus, with the use of a crest structure, the freeboard needs to have a height of 0.729 ·11.4m = 8.3 m. This is a crest freeboard reduction of 11.4m - 8.3m = 3.1m.

As illustrated in the example, it is easy to use figure 3-2. However, the fixed wave steepness and the fixed slope angles limit the usefulness of the figure. Therefore, it is desirable to use a parameter that contains all the geometric and hydraulic boundary conditions. This has been done in figure 3-3.

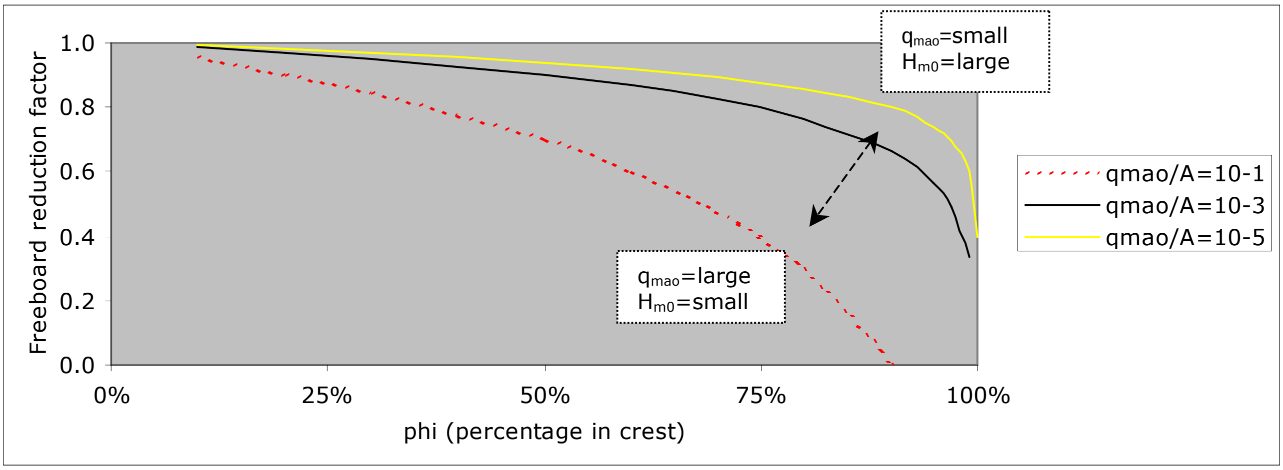


figure 3-3: The reduction coefficient as function of the geometric and hydraulic boundary conditions, the maximum allowed overtopping discharge and the crest basin efficiency

It is difficult to get a 'feeling' with figure 3-3. Therefore, the maximum allowed overtopping and some wave heights are combined to give the corresponding value of q_{mao}/A. (The definition of A is given in Equation 3-7 on page 17). These combinations are shown in table 3-1. These values only valid for tanα = 0.25 and s_0= 0.03.

q_{mao} \ H_{m0}	1m	2m	5m	10m
0,1 l/s/m	2,E-04	6,E-05	1,E-05	5,E-06
1 l/s/m	2,E-03	6,E-04	1,E-04	5,E-05
10 l/s/m	2,E-02	6,E-03	1,E-03	5,E-04
100 l/s/m	2,E-01	6,E-02	1,E-02	5,E-03

table 3-1: the value of q_{mao}/A for different values of q_{mao} and H_{m0}

Example

Suppose the maximum allowed overtopping is 10 l/s/m and the significant wave height is 5m. When using table 3-1, it is obvious to see that q_{mao}/A =1E-3. This is the yellow line in figure 3-3

3.2.1 Conclusions

In the given examples, all parameters such as geometry and hydraulic boundary conditions are usually known. The only unknown parameter is the crest basin efficiency parameter, φ. This parameter is the single most important parameter of interest in this report. If this parameter is known it is possible to investigate the crest freeboard reduction of the Crest Drainage Dike. In other words: If the crest basin efficiency parameter is found, the difference in crest freeboard of a traditional dike and a Crest Drainage Dike can be obtained. To find this crest basin efficiency parameter, it should be realized that this parameter is a function of geometric and hydraulic boundary conditions. The theory in the following sections will describe the process to find the relation between the crest basin efficiency and the parameters it depends on.

3.3 Design of the Crest Drainage Dike

3.3.1 Introduction
To obtain the efficiency of the Crest Drainage Dike, there is a need to specify the layout of the Crest Drainage Dike. Therefore, different layouts are presented in this section. The different layout types are categorized in two groups; Drain layout and buffer layout. The drain layout will be discussed in section 3.3.2, the crest basin layout will be discussed in section 3.3.3

3.3.2 Drain layout
To drain the crest basin, three different alternatives will be discussed. The three alternatives are:

- Porous crest basin
- Pipes
- Overflow

These alternatives will be discussed briefly.

Drain alternative A: A porous crest basin
The basic idea of a porous crest basin is shown in figure 3-4. The advantage of this layout is that the expected drain capacity is relatively high.

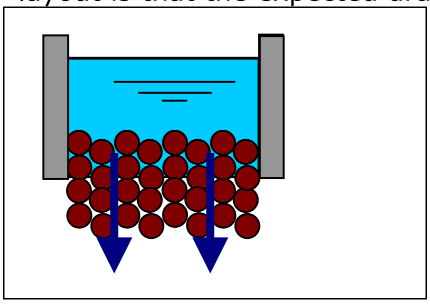
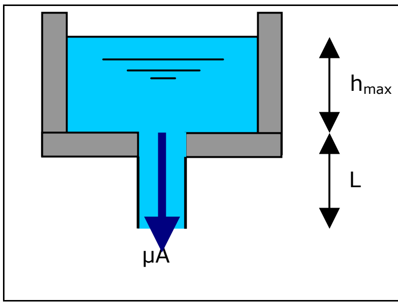


figure 3-4: Drain alternative A: A porous crest basin

Drain alternative B: Draining with the use of pipes
The basic idea of drain alternative B is shown in figure 3-5. Use will be made of pipes which discharge the water in the crest basin. This alternative is based on [DHV, 2005]. The outflow is dependent on the entrance friction (μ), the surface of the discharge pipe (A), the length of the discharge pipe (L_{drain}) and the height of the crest basin (h_{max}).



$$Q = \mu A\sqrt{2g(h + L_{drain})}$$

figure 3-5: Drain alternative B: Draining with the use of pipes.

Drain alternative C: Overflow
The basic idea of drain alternative c is shown in figure 3-6. The water in the crest structure leaves the crest basin through openings on the sides of the crest basin. The physics of this layout are based on the Rehbock weir theory.

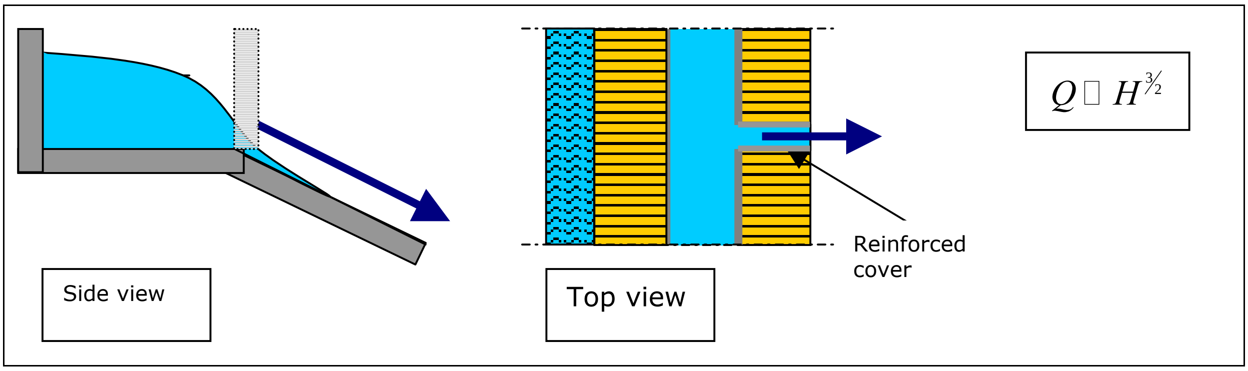


figure 3-6: drain alternative C: Overflow

Conclusions
A closer analysis is required for determining which alternative is favorable. It is stressed that there are many variations possible. (e.g. landward or seaward draining). Designing the optimal drain layout should be performed on the basis of technical, esthetical and financial criteria. Since this is not the main goal of this report, no more attention will be paid to these alternatives.
The feasibility study [DHV, 2005] is based on alternative B: draining with the use of pipes. For continuity reasons, this layout will be used in the further analysis. This however, does not imply that this alternative is also the best option. Since this choice is still open, the physics and models that will be developed in this report will be developed in the most general method possible. The advantage of this is that alternative draining methods could be projected on the developed theories.

3.3.3 Crest basin layout
This section discussed several layout alternatives for the crest basin. Before this will be done, two possible physical scenarios are shown.

It might happen that the incoming water flows over the crest basin. This process is shown in figure 3-7. There is a desire to minimize this overflow.

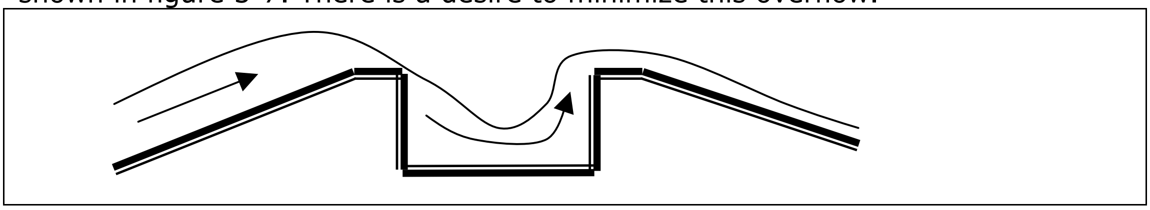


figure 3-7: possible overflow of the crest basin for crest basin A

Another possibility is that water is reflecting from the crest basin. This is illustrated in figure 3-8. This phenomenon is desired since this contributes to the minimization of overtopping water.

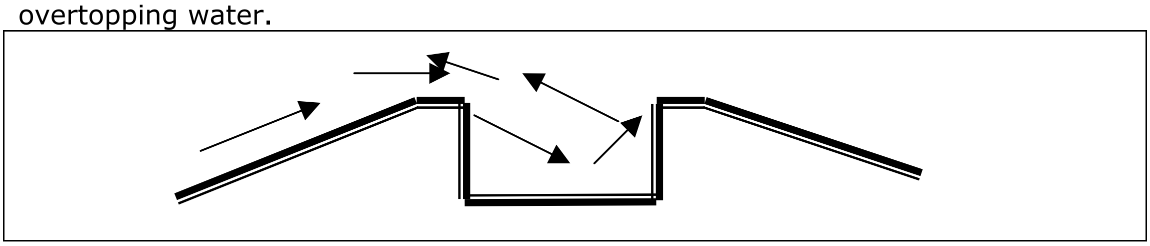


figure 3-8: Reflection of water

The shape of the basin probably determines the amount of overflow and an existence of reflection. Therefore, the design of the crest basin is important regarding the efficiency of the Crest Drainage Dike. Although the design of the crest basin is not the main goal of this report, some alternative crest basin layouts will be presented briefly. In figure 3-9 three crest basin layouts are shown.

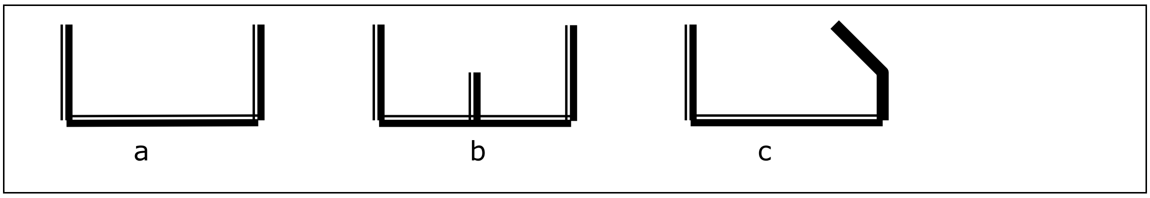


figure 3-9: different crest basin lay-outs

figure 3-9a is the basic layout of the crest basin. It might happen that the wave is simply flowing out of the crest basin as shown in figure 3-7. In [DHV, 2005] it is assumed that the crest basin is 80 percent filled with water before it starts to overflow. To avoid this overflow, a small element could be placed in the middle of the crest basin. See figure 3-10 for an impression.

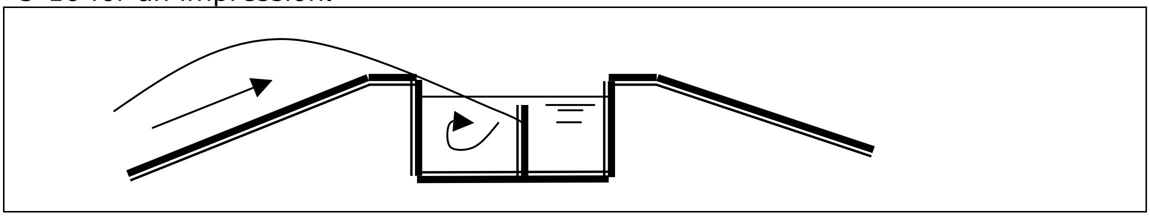


figure 3-10: Alternative crest basin B

Another possible solution is shown in figure 3-9 c. See figure 3-11 for an impression.

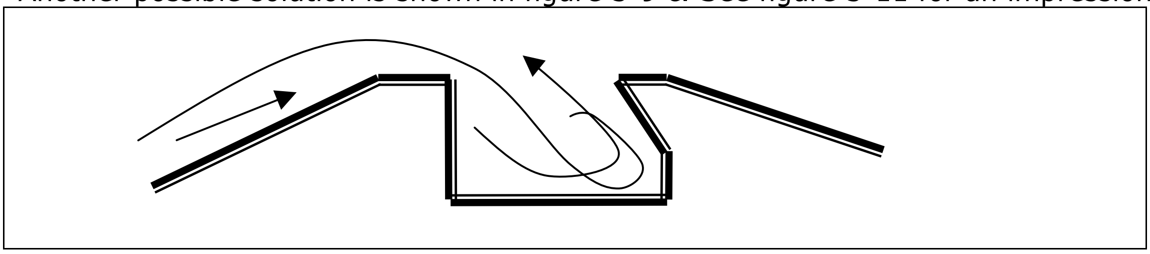


figure 3-11: Alternative crest basin C

For simplicity reasons, a start is made with a crest basin layout as shown in figure 3-9a. The question that rises is in which ratio the crest width and height is optimal. See figure 3-12 for an impression.

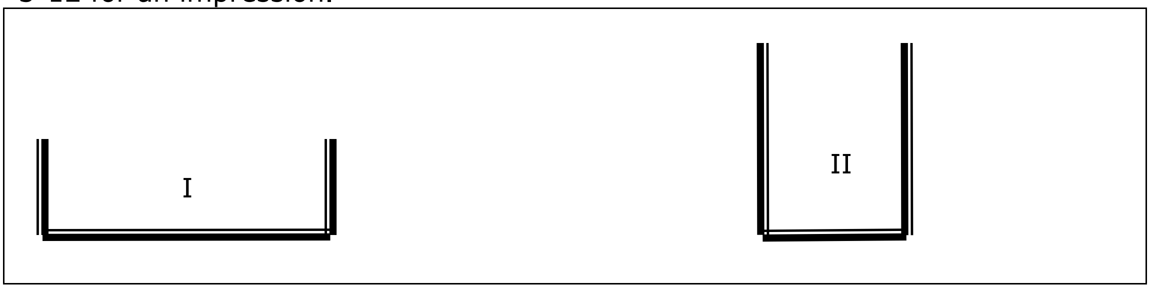


figure 3-12: examples of different ratios between the crest basin width and height

Crest layout II might be more effective since the average water level in the crest basin will be higher and thus a larger drain capacity will be obtained. (Draining theories will be discussed in section 3.5.1)

The theories that will be developed in the report are all based on alternative A (figure 3-9, page 23). However, it is stressed that it is unknown whether this is the best option.

3.3.4 Conclusions

Several options regarding the layout of the draining method and crest basin layout are given. In this report, no analysis regarding an optimal layout is given. In the continuation of this report, the draining method with the use of drains and the crest basin shown in figure 3-9a are the starting points of the analysis. The analysis will be executed in a general way. This gives the possibility to adapt the theories relatively easy for the other alternatives.

3.4 Assumptions regarding a model framework

Before a model framework is constructed several assumptions are formulated in this section.

3.4.1 *Shooting over*

It is assumed that no "shooting over" takes place. (The overtopping on the crest takes the form of a sheet flow). For an impression of overshooting, reference is made to figure 3-13.

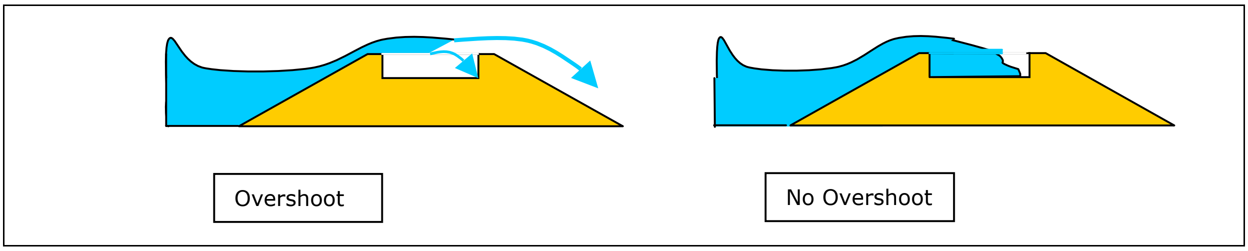


figure 3-13: "Shooting over" of an overtopping wave

3.4.2 *Length spreading effect*

Since wind-generated waves are short crested, the maximum instantaneous overtopping rate does not occur simultaneously along the entire length of the crest construction. This gives the crest basin a chance to spread extreme overtopping rates in longitudinal directions of the dike. In [DHV, 2005], it is assumed that due to this length spreading effect, the buffer capacity is twice as much as the volume of this buffer. This assumption is also applied in this model by increasing the (numerical) width of the crest basin. A schematization of the influence of the length spreading effect is given in figure 3-14.

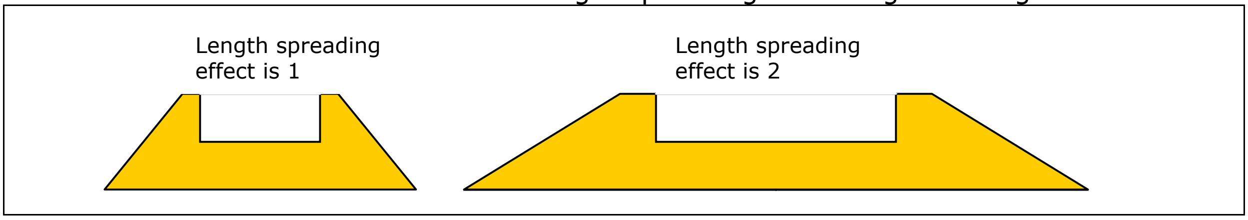


figure 3-14: Schematization of the length spreading effect as applied in the Matlab program.

3.4.3 *Wave grouping*

Wave grouping is a phenomenon, which has no influence on the average wave overtopping discharge of a traditional dike. However, it is very likely that wave grouping might influence the efficiency of a Crest Drainage Dike. This will be discussed on page 34 after some process-based theories are described in section 3.5.2. In this report, it is assumed that there is no wave grouping.

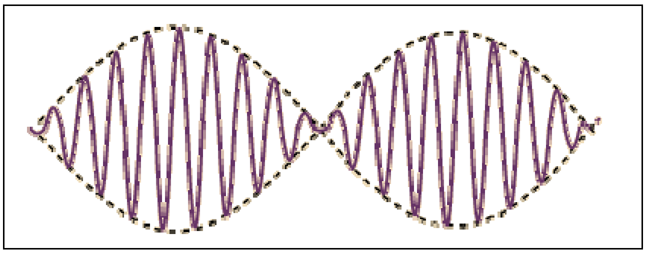


figure 3-15: Illustration of extreme wave grouping

3.4.4 *Physical upper boundary of the Weibull distributed overtopping discharges*

In section 2.5, it is described that the overtopping discharge per wave is Weibull distributed. Therefore, it has no statistical upper boundary. It is unknown if there is a physical maximum and how this should be derived. Therefore, the Weibull distribution will be used in the numerical program. It is assumed that there is no upper boundary.

3.4.5 Reflection

Reflection due to hydraulic process in the crest basin is already discussed in section 3.3. (See figure 3-8 on page 22). It is assumed that there is no reflection as a result of hydraulic processes in the crest basin.

3.5 Model framework

In section 3.2, it is stated that the efficiency of the crest basin is needed to get insight of the influence of the Crest Drainage Dike section. Besides this, a basic layout is given in section 3.3 as a starting point. This gives the possibility to create a numerical model. A first set up of this model is described in this section. Since this is only a basic set up, several specific problems and hypotheses will be formulated. These will be worked out in later phases.

The Crest Drainage Dike is schematized as shown in figure 3-16. As a start, only one wave is considered.

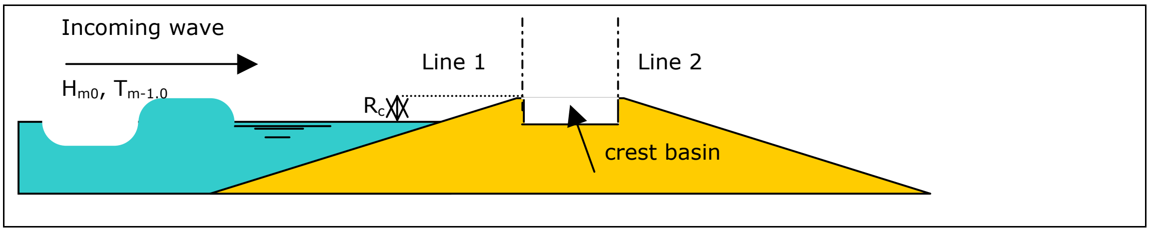


figure 3-16: Schematization of a Crest Drainage Dike

As can be seen in figure 3-16, the crest basin has a certain volume that can work as a buffer for the wave, which is passing line 1. If the volume of the wave, which is passing line 1, is larger than the volume in the crest, a part of the wave will also pass line 2. In this schematization no drain has been applied. A schematization of the above-described theory is shown in figure 3-17.

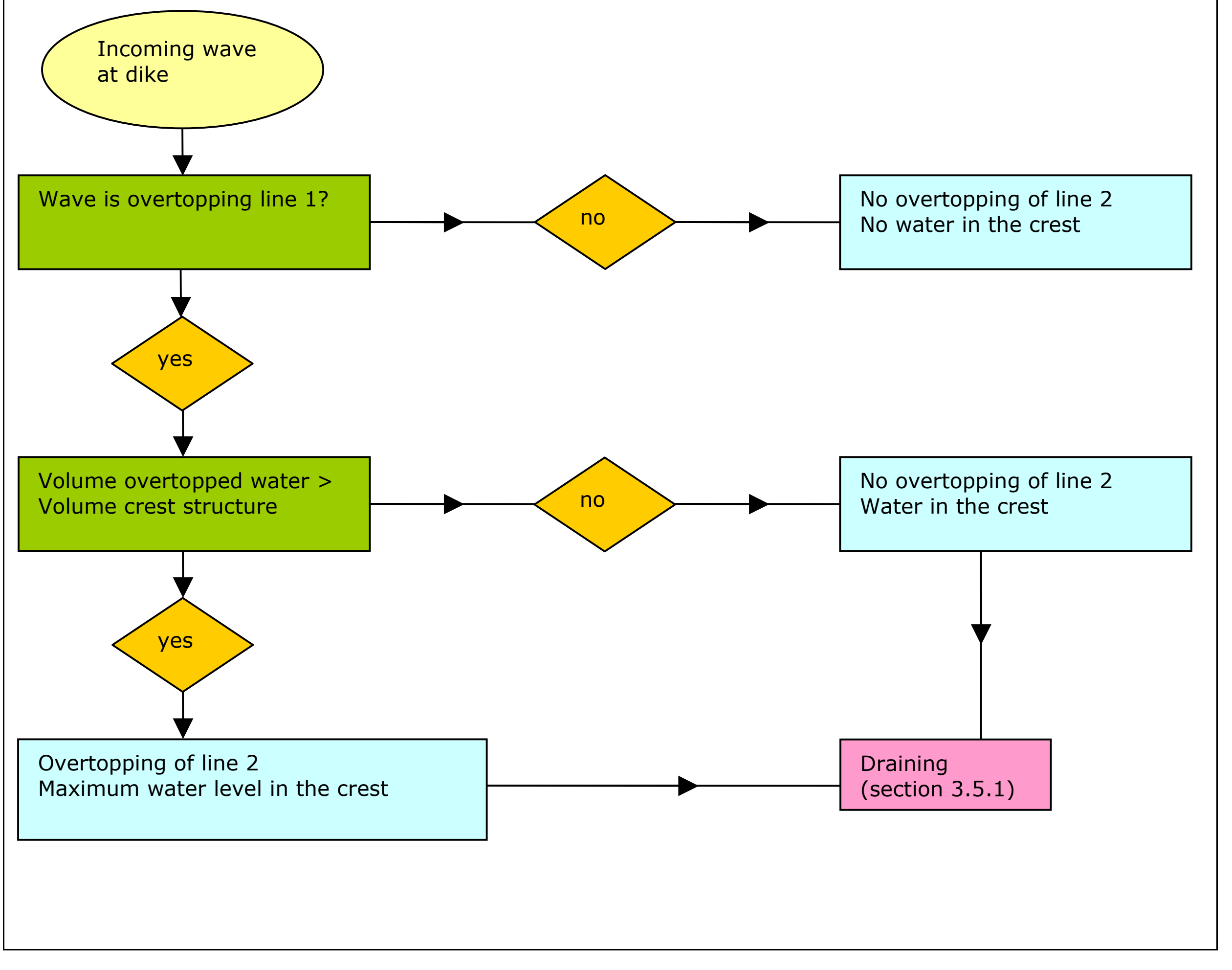


figure 3-17: Flow chart of an incoming wave at the Crest Drainage Dike

This is a simple model that will be the basis of the theory. This basis will be discussed step by step, following the arrow lines in figure 3-17.

Incoming wave at dike
It is assumed that the wave run-up is Rayleigh distributed and the wave overtopping volume per wave is Weibull distributed [van der Meer, 2002]. (See also section 3.4.4 on page 25). Under these assumptions it is possible to determine the probability of passing line 1 (figure 3-16). This theory is explained in section 2.5. It is calculated whether $P_{ov}>0$. If $P_{ov}=0$, no water will pass line 1, the crest will not be filled with water and no water will pass line 2. In other words: no overtopping will take place.

Water that passed line 1
In case water will pass line 1, water will fill the crest basin and a part will possibly pass line 2 (figure 3-16) Following figure 3-17, it should be determined whether water is also passing line 2. In this model, it is assumed that this depends on four parameters:

- The volume of the solitary wave
- The volume of the crest basin
- The length spreading effect
- The effectiveness of the crest basin

The volume of the solitary wave can be determined using the theory explained in section 2.5 (Equation 2-14 and Equation 2-15, page 11)

The volume of the crest basin is determined by two parameters. (Assumed that the crest basin has a rectangular shape): width and height. The volume is expressed in m^3 per m.

The length spreading effect has been explained in section 3.4.2 on page 25. It is assumed that due to this length spreading effect, the buffer capacity is twice as much as the volume of this buffer.

The effectiveness of the crest basin θ is expressed in a maximum possible percentage of fillings. It is assumed that the crest basin is 80% filled before overtopping takes place [DHV, 2005].

The volume of the crest basin, the length spreading effect and the effectiveness of the crest basin can be combined and forms one parameter. This parameter is defined as:

$$V_{buffer} = \theta \cdot l.s.e. \cdot V_{crestbasin} \qquad \textbf{Equation 3-17}$$

Where

V_{buffer}	=	buffer capacity	(m^2)
l.s.e	=	length spreading effect	(-)
$V_{crestbasin}$	=	volume of the crest basin	(m^2)
θ	=	effectiveness of the crest basin	(-)

Water will pass line 2 if the volume of water, which is passing line 1, is more than the effective crest basin volume:
if $V_{water\ passed\ line\ 1} > V_{buffer}$ then water is passing line 2.
If $V_{water\ passed\ line\ 1} \leq V_{buffer}$ then no water is passing line 2.

The amount of water that will pass line 2 is the difference between the volume of water that has passed line 1 and the effective crest basin volume. This is based on the assumption that no water is reflecting.

$V_{water\ passing\ line\ 2} = V_{water\ passed\ line\ 1} - V_{buffer}$
$V_{water\ in\ crest\ basin} = V_{buffer}$

Following figure 3-17, the (partly) filled crest is drained. Since the physics of this draining process requires some thorough analysis, this will be treated separately in the next section.

3.5.1 Draining the crest basin

To obtain a proper model for the Crest Drainage Dike, there is a need to investigate the behavior of reservoirs, which are emptied by the use of a drain. A reservoir, such as used in the Crest Drainage Dike, is shown in figure 3-18. It contains a reservoir with a height h_{max} and a surface A_0. Besides the reservoir there is a drain with drain length L and drain surface μA

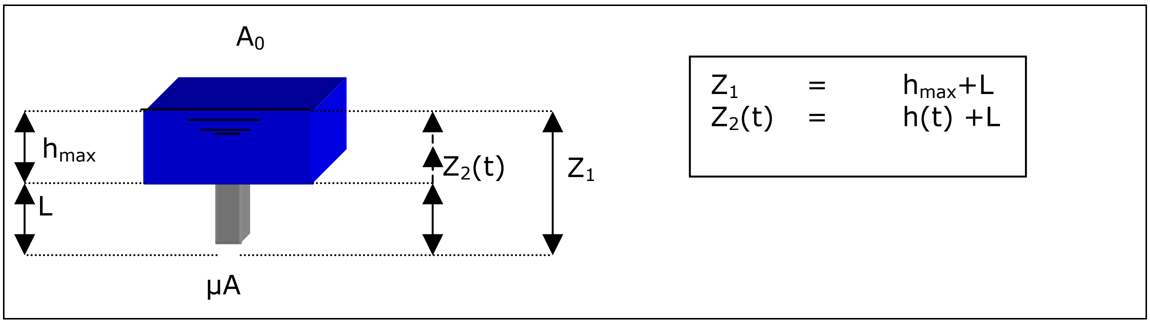


figure 3-18: Schematization of a crest basin with a drain

A proper analysis on the physics of draining a reservoir has been executed and is shown in appendix III. The results from this analysis are discussed in this section.

The water level in the crest basin is expressed as a function of time. This relation is

$$Z_2(t) = \left(\sqrt{Z_1} - \frac{\mu A \sqrt{2g}}{2A_0} \cdot t \right)^2 \qquad \textbf{Equation 3-18}$$

Where

μA	=	net surface of the drain	(m^2)
A_0	=	surface of the reservoir	(m^2)
Z_1	=	water level (measured from bottom of drain)	(m)
Z_2	=	water level (measured from bottom of reservoir)	(m)

This relation is based on the analysis in appendix III. In the relation, it can be seen that the water level in the crest depends on Z_1 and thus also on the length of the drain. (a condition for this is that the drain is totally filled with water). An investigation of the length of this drain has been carried out. The length of the drain influences two physical aspects.

- Dependency of the water level as function of the time
- Total draining time

These two aspects will be discussed briefly. For a full description, one is referred to appendix III. The length of the drain is expressed as a number of maximum water heights in the crest basin and is therefore dimensionless: $L=c.h_{max}$ where c is the dimensionless drain length.

The dependence of the water level as function of the time

The dependence of the water level as function of the time is shown in figure 3-19. The x-axis represents the dimensionless time (t_E =total time to empty a filled crest basin). The y-axis represents the dimensionless water level in the crest basin. (h_{max} is a filled reservoir). It can be seen that the water level reservoirs with a certain drain length behave linearly. In this figure, it can be seen that the water level can be considered linearly as function of the time for a dimensionless drain length that has a value of 1 or larger.

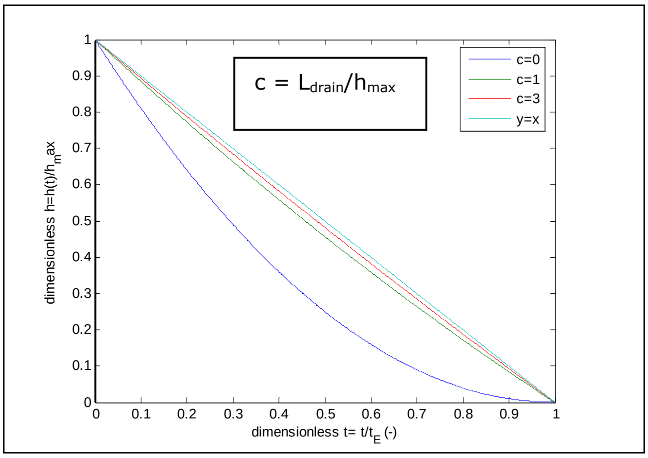


figure 3-19: Crest basin water level with several dimensionless drain lengths.

Based on figure 3-19, the following schematizations are made:

- No drain length $h(t) \propto t^2$

When applying a drain with a certain length, the relation tends to behave linearly:

- Drain length $h(t) \propto t$

Total draining time

The analysis in appendix III shows that the drain has a higher capacity with a larger drain. The results are shown in figure 3-20. The x-axis represent the dimensionless drain length. The y-axis represent the dimensionless time to empty the crest basin. $t_{E,dim}=1$ represents the time to drain a filled crest structure with a drain length of zero meters. For a sound understanding of this figure, reference is made to the example given below.

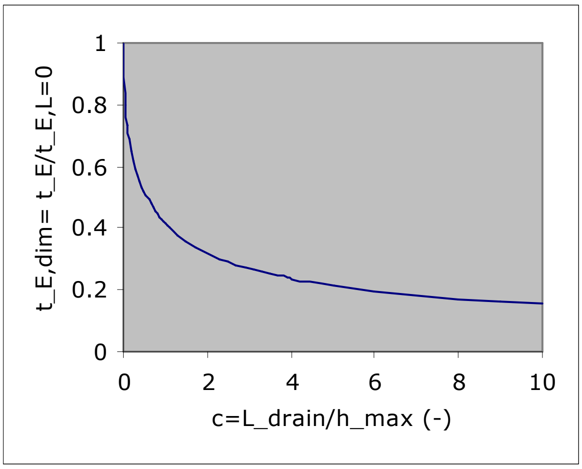


figure 3-20: total draining time as function of the relative drain length.

Example

Suppose that a filled reservoir with a drain length of L_{drain}=0.00m (no drain) needs 100 seconds to empty. The height of the reservoir is 2m

When applying a drain with a drain length of 6m, the following dimensionless parameter can be applied:

$$c = \frac{L_{drain}}{h} = \frac{6m}{2m} = 3$$

In figure 3-20, it can be seen that the total draining time is around 25 percent of its draining time in case the drain length is zero.

It is also possible to use the following formulae (see appendix III):

$$t_{E,\dim} = \sqrt{c+1} - \sqrt{c} = \sqrt{3+1} - \sqrt{3} \approx 0.27 \text{ (=27 \%)}$$

Since the effects of a longer drain length are positive, it is assumed that this phenomenon will be used to optimize the effectiveness of the Crest Drainage Dike. In a real situation, the crest basin height will be around 0.80m and the filling of this structure will be around 80 percent [DHV, 2005]. Therefore the h_{max} is around 0.64meter. It is assumed that it is possible to construct a drain with a height difference (L_{drain}) of around 1 meter. This means that the dimensionless drain length c = L_{drain} /h_{max} = 1m/0.64m = 1.6. Using the theories from Appendix III, it turns out that the draining duration is shortened by 65 percent. The water basin level can be considered as a linear dependent time variable. (see figure 3-19). How this relation can be translated into a linear relation is explained in appendix III.

Epilogue
Since the physics of the drains are identified, it is possible to integrate this into the model. In this model it is assumed that a drain with a certain length will be used and therefore the water level in the crest basin behaves linearly as a function of the time. When using the model, one should realize that the model can not be used for other drain types or drains that are not linearly dependant. The reason for this is that the physics keep "clean" and are not made invisible caused by complicated relations. Besides this, it is very likely that, if a Crest Drainage Dike will be used, the designer will make use of the advantages of the drains with a certain length.
The model will be integrated in such a way, that if there is a tendency to, it is relatively easy to adapt the behavior of the drains.

3.5.2 Theory regarding a wave field

In the previous section, the theory about wave overtopping and draining has been explained for only one single wave. Since there is always a wave field instead of only a single wave, attention needs to be paid to this. The theories explained in the previous sections do also yield for conditions where more then one wave does exist. However, some adaptations are required.
An assumption in the earlier mentioned theories on a single wave is that the crest basin is empty at the moment the overtopping wave starts to fill the crest basin. This assumption is valid for a single wave, but not necessarily for a wave field. In a wave field it is possible that the crest basin is not empty when a wave starts to fill the crest basin since there might be water in the crest basin as a result of the previous overtopping wave(s).
This effect will be included in the model by taking account for the crest basin volume of the previous wave(s) and the draining discharge during these previous waves. An illustration is given in figure 3-21. In this illustration three waves with mean wave period T_m are taken.

The events are schematized as follows:

- An incoming wave has an infinitive small "incoming time" at the beginning of the wave period.
- At this moment the overtopping volume is determined by subtracting the effective crest basin volume from the incoming wave volume. (This is only the case when the incoming wave volume is larger).
- During the mean wave period (T_m), the crest is draining until there is no more water to drain.
- At the end of the wave period the volume in the crest basin is determined by subtracting the drained volume from the crest basin volume directly after the incoming wave.
- When a second wave comes in, the crest basin volume which is still "left" is determined by subtracting the volume of the crest basin, which is still filled with water from the previous wave, from the effective crest basin volume.

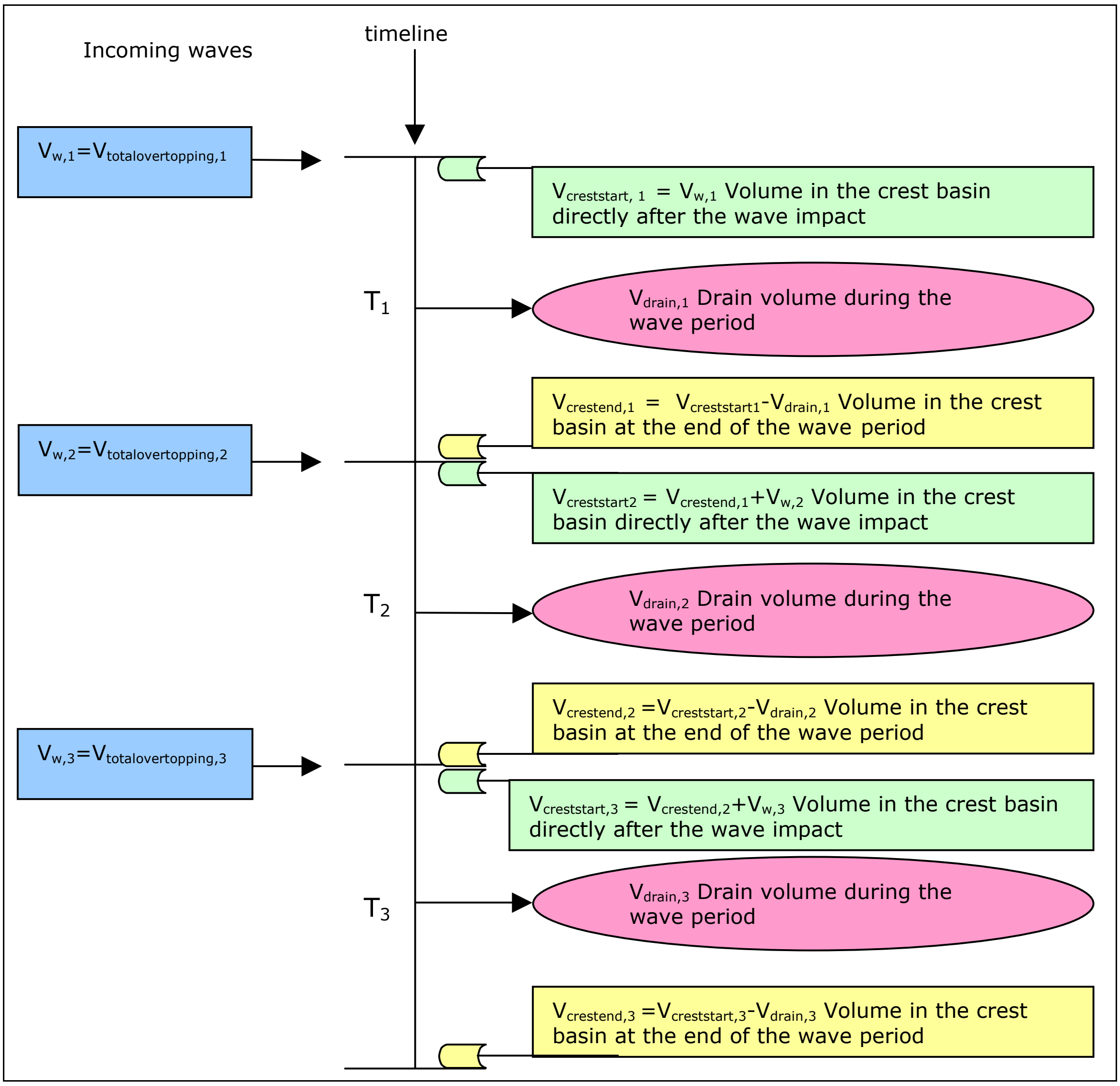


figure 3-21: Schematization of a wave field overtopping the Crest Drainage Dike

Wave grouping
The basic concept of wave grouping is discussed in section 3.4.3 (page 25). To illustrate the possible influence of wave grouping, two examples are given. In figure 3-22 a schematization is shown, where no wave grouping takes place. In figure 3-23 a schematization with wave grouping is shown. The average incoming volume of water is the same in both pictures. The schematization is based the processes shown in figure 3-17 and figure 3-21. The drain capacity in these examples is 1 m^3/m per wave. (usually this is expressed in m^3/m/s but for illustration purposed this unit is shosen). The maximum buffer capacity is 2 m^3/m per wave.

As can be seen in figure 3-22, 6 m^3/m is overtopped during the wave record. In the schematization of figure 3-23, 10 m^3/m is overtopped during the wave record. Therefore, it seems very logical that wave grouping might have a negative influence on the average wave overtopping rates.

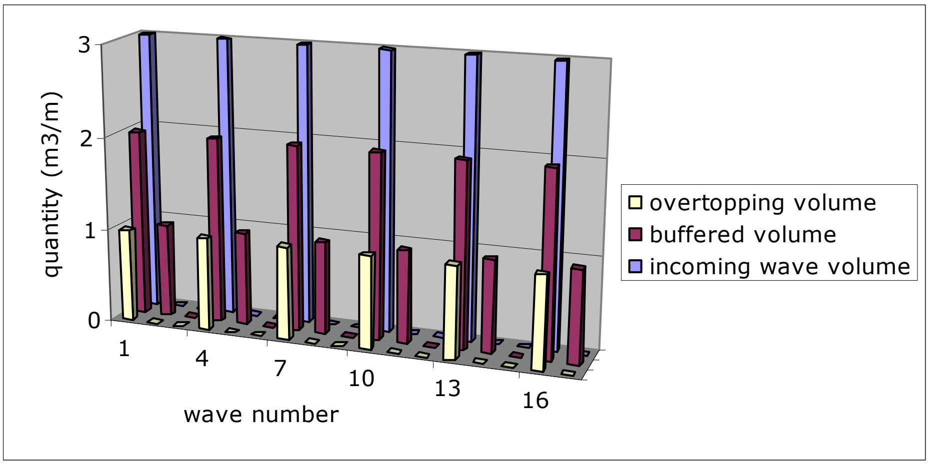


figure 3-22: Schematisation of a wave pattern without wave grouping.

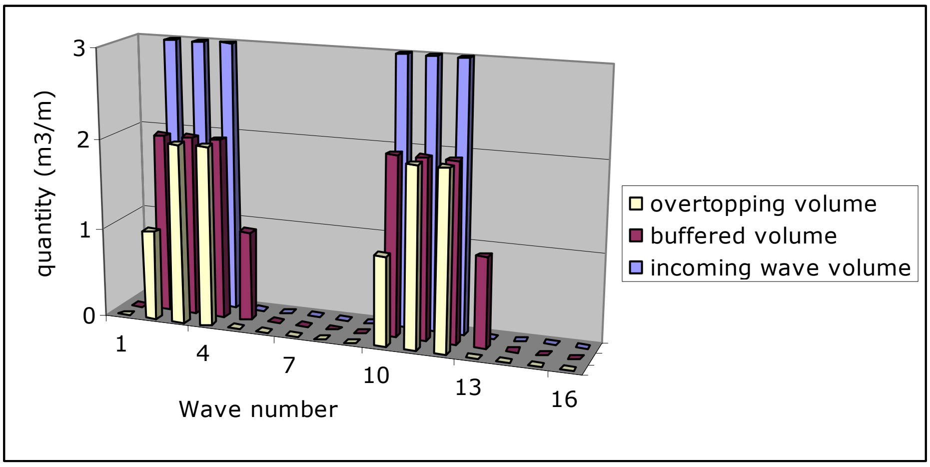


figure 3-23: Schematisation of a wave pattern with wave grouping.

Since there is not much insight in wave grouping, this phenomenon will not be used in the model. However, one should be aware that the phenomenon does exist and it is very likely that it will influence the efficiency of the Crest Drainage Dike.

Conclusions

A model framework for the Crest Drainage Dike is described. This has been done on the basis of several discussed theories. This model is implemented as a numerical computer program. How this is done is discussed in the following section.

3.5.3 Numerical model.

In the previous section a model frame is given. This frame is used for the development of a numerical model. A complete description of this model is given in appendix IV. The program code is given in appendix V. The manual of this program is shown in appendix VI.
The numerical model is designed using the top-down model described in [Mesman, 1991]. This model is chosen for two reasons. The basic idea of this method is that the complex problems will be divided in several small problems, which make it easy to understand the sub-problems and thus the general problem.
Besides understanding the model, it is important that the model can be adapted in the future. The top-down model is easy to adapt since the method gives a good overview of the sub problems. If adaptations are required, redesigning only one (or more) module(s) and not the whole program can do this.

An example of the use of a top-down model is shown in figure 3-24. The methodology as shown in section 3.4 is applied to the Crest Drainage Dike program. An impression how this is implied is given in figure 3-25.

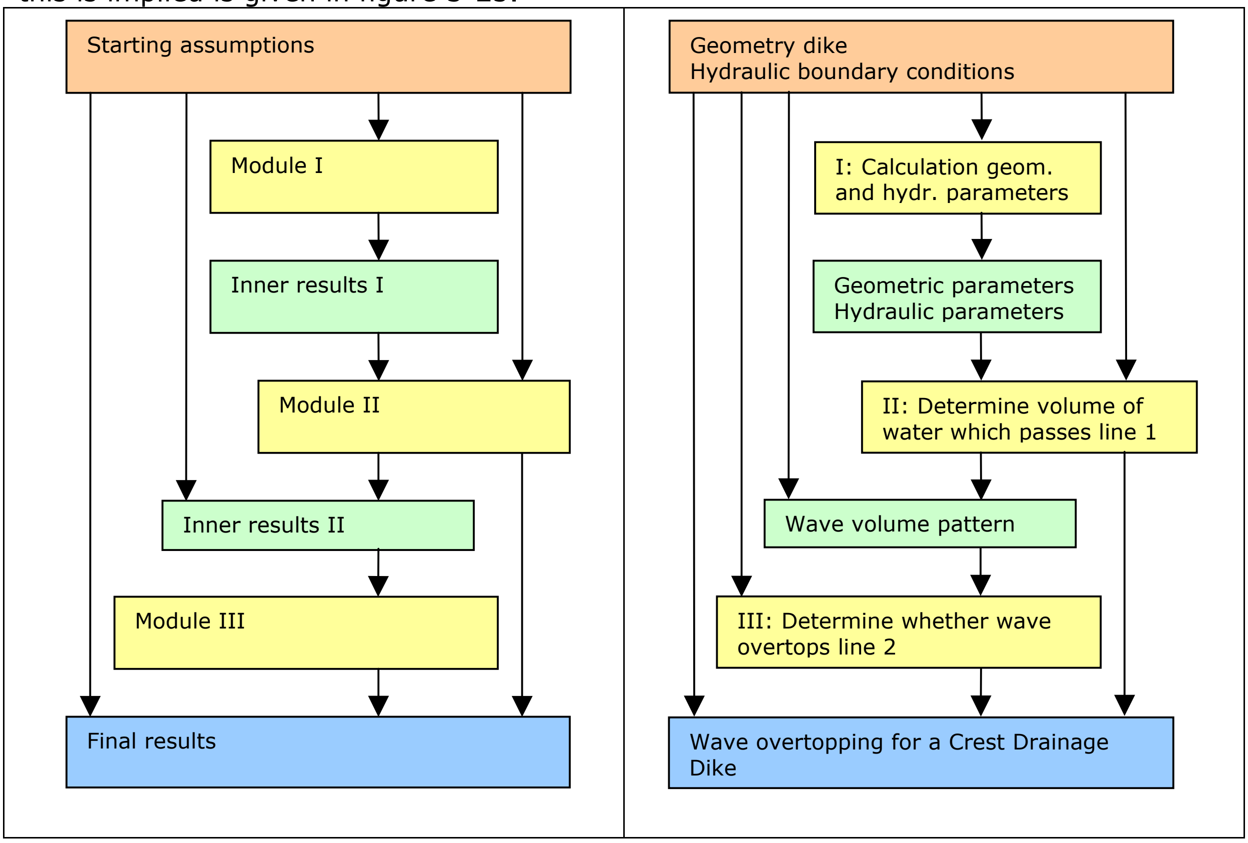


figure 3-24: Top down Model [Mesman, 1991]

figure 3-25: The Crest Drainage Dike model as a top-down model

3.6 The use of dimensionless parameters

3.6.1 Introduction

In the previous section, several process-based theories have been discussed. This resulted in a numerical program that can be used to determine the average wave overtopping rate and the efficiency of the Crest Drainage Dike. However, the described theories give no proper insight in which way the parameters influence the efficiency of the Crest Drainage Dike. Although the described theories are sufficient to create a numerical model, they do not provide a physical insight. Therefore, a different type of analysis will be executed in this section. As a result of this section, a better physical insight in the parameters is obtained.

3.6.2 Dimensionless Crest Drainage Dike parameters.

For wave overtopping analysis regarding traditional dikes, it is common to use dimensionless parameters. The use of these parameters is described in chapter 2. It would be interesting if the parameters that are introduced with the Crest Drainage Dike, could be made dimensionless as well. Mathematically it is not difficult to construct “a” dimensionless parameter. However, the challenge in creating the dimensionless parameters lies within the physical background.

With the introduction of the concept of the Crest Drainage Dike, two new physical aspects are introduced. These physical aspects are:

- Drain capacity
- Buffer capacity

Both physical aspects will be described in the following sections.

Buffer capacity

For the purpose of creating a dimensionless buffer parameter, it seems obvious to use the buffer volume V_{buffer} (see page 28 for an explanation of this parameter). The unit of this volume is m^3 per m. This parameter could be seen as the “defence” parameter. A larger crest basin volume gives a better buffer and thus a better “defence”.
The “attack” parameter should contain the volume of the wave. Since $V_w \propto H \cdot L$, the created dimensionless parameter is temporarily defined as:

$$V^*_{buffer} = \frac{V_{buffer}}{V_{wave}} = \frac{V_{buffer}}{H \cdot L} \qquad \textbf{Equation 3-19}$$

Where

V^*_{buffer}	=	dimensionless buffer capacity	(-)
V_{buffer}	=	buffer capacity of the crest basin (see page 28)	(m^2)
H	=	wave height	(m)
L	=	wave length	(m)

H and L are not specified yet, since there is only a weak physical basis for the above-described theories. It is stressed that this dimensionless parameter has a weak physical basis and can therefore not be considered as ‘the’ dimensionless buffer parameter. A closer analysis is required. This will be done in a later stadium after physical and numerical data are obtained.

Drain capacity

To get a feeling of the physical processes and the dimensions of the drain capacity one could ask the following question:

“How long does it take to empty a filled crest?”

A possible answer is, for example, “10 seconds” or “1 hour”. However, this answer is not dimensionless since the variable ‘time’ has a dimension. Therefore, one could answer this question at the following way:

“The time to empty a filled crest basin is 3 wave periods”

Here, a dimensionless answer is obtained. Taking this in mind, the dimensionless drain capacity is defined as the number of waves that it takes to drain a filled crest basin completely. If this is expressed in time (for example in seconds) one could state that the duration of one wave is the mean wave period (T_m). With n waves, the total time to empty a full crest basin, expressed in seconds is:

$$t_E = nT_m \qquad \textbf{Equation 3-20}$$

Where

t_E	=	time to empty a filled crest basin	(s)
n	=	number of waves	(-)
T_m	=	mean wave period	(s)

The time to empty a filled crest basin can be determined using the methods given in section 3.5.1. In a simplified model the duration time to drain a filled crest is:

$$t_E = \frac{V_{crestbasin}}{Q_{drain}} \qquad \textbf{Equation 3-21}$$

Combining Equation 3-20 and Equation 3-21 gives:

$$n = \frac{V_{crestbasin}}{Q_{drain} T_m} \qquad \textbf{Equation 3-22}$$

This is a dimensionless parameter. However, the physical background is relatively weak. Therefore, a closer analysis will be executed after physical and numerical data are obtained.

To get a good physical insight into this parameter an example is given below.

Example

Suppose a crest basin with the following characteristics: Width=length=height=1m
Thus, the volume is $1m^3$

The wave period is 5 seconds. The drain capacity is 0.1 m^3/s The dimensionless parameter can be calculated with the use of Equation 3-22:

$n=1m^3/(0.1m^3/s * 5s) = 2$

Therefore, it takes the drain 2 mean wave periods to empty a filled crest basin.

3.7 Influence of hydraulic and geometric boundary conditions

3.7.1 Berms

It is difficult to predict whether berms have the same or a different influence on a Crest Drainage Dike compared to a traditional dike. Since berms reduce the amount of overtopping for a traditional dike it seems trivial that this will also happen at a Crest Drainage Dike.
As a first estimate, it is assumed that a berm applied to the Crest Drainage Dike gives the same reduction as a berm applied to a traditional dike.

3.7.2 Influence of different wave spectra

Since there is a lack of developed theories regarding the influence of different wave spectra, it is initially assumed that the shape of the wave spectrum has no significant influence on the efficiency of the Crest Drainage Dike. A boundary condition for this is that the $T_{m-1.0}$ is used as the spectral period.

3.8 Hypotheses

In the previous chapter, several aspects regarding the wave overtopping over a Crest Drainage Dike have been analyzed and discussed. Several assumptions were created during this analysis. To verify these assumptions, hydraulic scale model tests have been executed. Before the tests were executed several hypotheses are created. These hypotheses are categorized as shown below.

Reflection

In section 3.6 it is assumed that no reflection takes place. Therefore the following hypothesis is formulated:

Hypothesis 1: The amount of overtopping water at a traditional dike equals the amount of overtopping water + the amount of drained water at a Crest Drainage Dike. This implies that a negligible amount of water is reflecting from the crest basin.

Use of dimensionless parameters

In section 3.6 it is assumed that the (adapted) dimensionless parameters of van der Meer could be used for the Crest Drainage Dike. Therefore the following hypothesis is created:

Hypothesis 2: Given a certain total overtopping discharge, geometric and hydraulic boundary conditions such as the crest freeboard, the wave height, the wave steepness and the use of berms do not influence the efficiency of the Crest Drainage Dike

Influence of different wave spectra

From the analysis in section 3.7.2, the following hypotheses considering the physics of different wave spectra is stated:

Hypothesis 3 The spectral shape has no significant influence on the efficiency of the Crest Drainage Dike.

Solitary wave overtopping

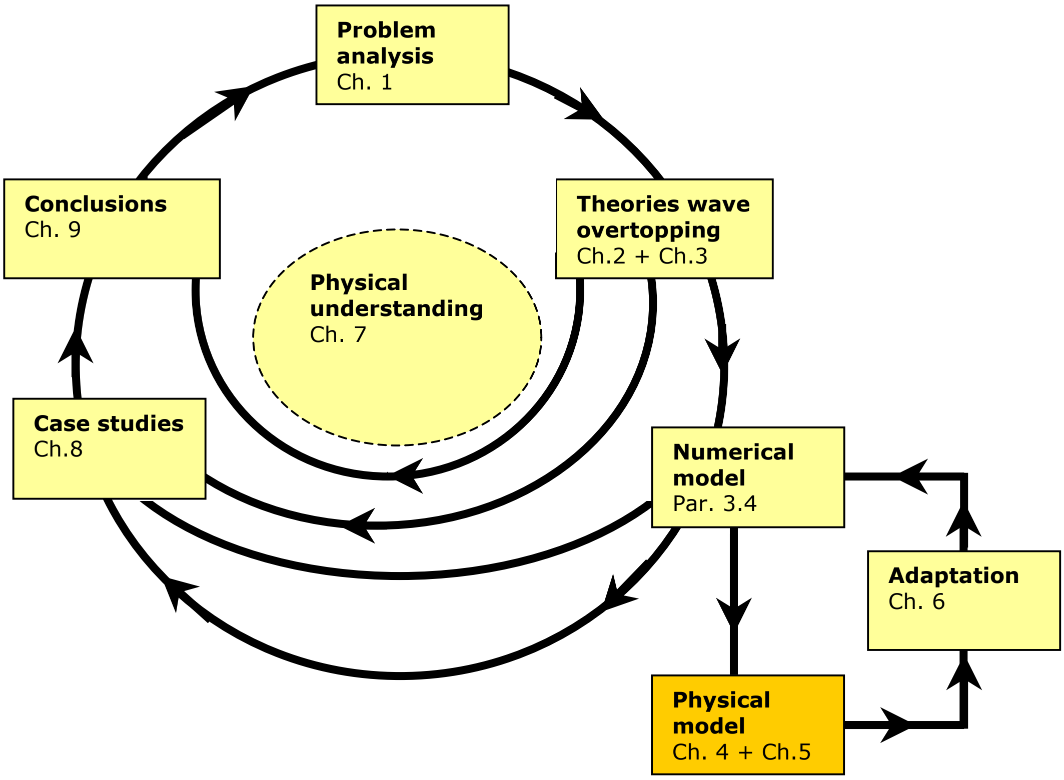
In section 3.2 it is assumed that the crest buffer capacity is 80 percent of its total volume. Therefore, the following hypothesis is formulated:

Hypothesis 4: The reduction factor for the buffer efficiency θ is 0.8

3.9 Conclusions

A conceptual model, which predicts the efficiency of a Crest Drainage Dike is constructed. This model is based on wave overtopping theories, several assumptions, schematisations and hypotheses. With the use of this model, a numerical model is constructed. Since the model is partly based on hypotheses, it is unknown whether the predicted efficiencies are in line with the actual overtopping efficiencies. To check the stated hypotheses, physical model tests have been performed. The execution of these physical model tests is the subject of the next chapter.

4 Physical experiments



4.1 Introduction
As a results of the analysis in chapter 2 and chapter 3, a numerical model, which predicts the efficiency of the Crest Drainage Dike, is created. This model is based on several hypotheses. To verify or reject these hypotheses, several physical model tests have been executed. The goal of this chapter is to describe these physical model tests.
In section 4.2, it is explained how the hypotheses are 'translated' into required physical model tests. The used equipment, materials and measuring instruments are described in section 4.3 and section 4.4 and the set-up of the model tests is described in section 4.5. A description of the execution of the tests is given in section 4.6.

4.2 From hypotheses to tests
The objective of the hydraulic model tests is to verify or reject the hypotheses stated in chapter 3. To investigate which tests are necessary to execute, all the hypotheses will be discussed.

Hypothesis 1: The amount of overtopping water at a traditional dike equals the amount of overtopping water + the amount of drained water at a Crest Drainage Dike. This implies that a negligible amount of water is reflecting from the crest basin.

To verify or reject hypothesis 1, it is necessary to execute test with a traditional dike and compare the results with tests, which have been executed with a Crest Drainage Dike

Hypothesis 2: Given a certain total overtopping discharge, geometric and hydraulic boundary conditions such as the crest freeboard, the wave height, the wave steepness and the use of berms do not influence the efficiency of the Crest Drainage Dike

To verify or reject hypothesis 2, it is necessary to execute tests with different wave steepness, wave heights, crest freeboards and berm layouts.

Hypothesis 3: The spectral shape has no significant influence on the efficiency of the Crest Drainage Dike.

To verify or reject hypothesis 3, it is necessary to execute tests with different wave spectra, such as narrow wave spectra, wide wave spectra, and double-peaked wave spectra.

Hypothesis 4: The reduction factor for the buffer efficiency θ is 0.8.

To verify hypothesis 4, it is necessary to execute tests with solitary waves.

Since many test results can be used to verify more than one hypothesis, the test series are divided into subsets, which are slightly different than the subsets of hypotheses. The test subsets are shown in table 4-1.

Subset	Description subset
A	Basic parameter Crest Drainage Dike
B	Traditional dike
C	Influence drain capacity
D	Influence crest freeboard
E	Influence wave steepness
F	Influence wave spectrum
G	Influence of a berm
H	Solitary wave overtopping

table 4-1: Overview of test subsets

All the tests are executed in such a way that the average wave overtopping discharge is determined. (Except for the last set of experiments: Solitary wave overtopping).

The several tests will be described intensively before the results will be shown. However, to get a sound understanding of the description of the tests there is a need to know which materials and which equipment is used. This is the subject of the next section.

4.3 Equipment and materials

Wave Flume

The tests are performed in the "Sediment transportgoot" of the Fluid Mechanics Laboratory of Delft University of Technology. The flume is 39m long, 0.8m wide and 0.85m high. The bottom of the flume is flat. At one end of the flume a wave board is placed. At 25.88 m distance from the flume a dike is placed. 17 m of the wave board and in front of the dike wave gauges are placed.

Wave board

The wave flume is equipped with a wave board, which can generate regular and irregular waves. The wave board has active reflection compensation (ARC) and a second order wave generation technique. This means that the second-order effects of the first higher and first lower harmonics of the wave field are taken into account in the wave board motion.

The wave board is controlled by a steering file, which creates a certain wave spectrum. The steering file is created with the program "Delft Auke" where the specific wave conditions can be filled in.

Dike

The dike is placed in the flume at a distance of 25.88 m of the wave board. The slope of the dike is 1:4. The slope is made of a smooth concrete. The top of the dike is placed 0.70m above the bottom of the flume. For an impression of the dike, reference is made to figure 4-1 and figure 4-2.

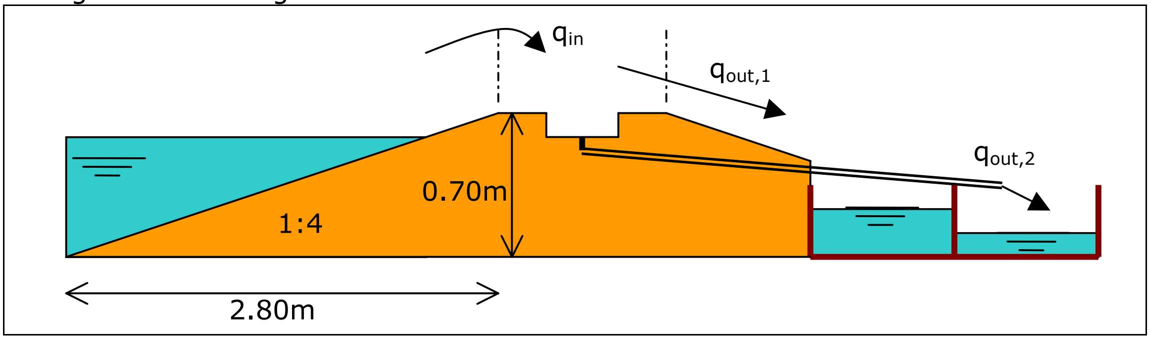


figure 4-1: Schematization of the scale model.

figure 4-2: impression of the scale model

The crest basin

On the crest of the dike, a crest basin is placed. This is illustrated in figure 4-3. This crest basin collects overtopping water. The water is drained through a drain, which is connected to the bottom the crest basin.

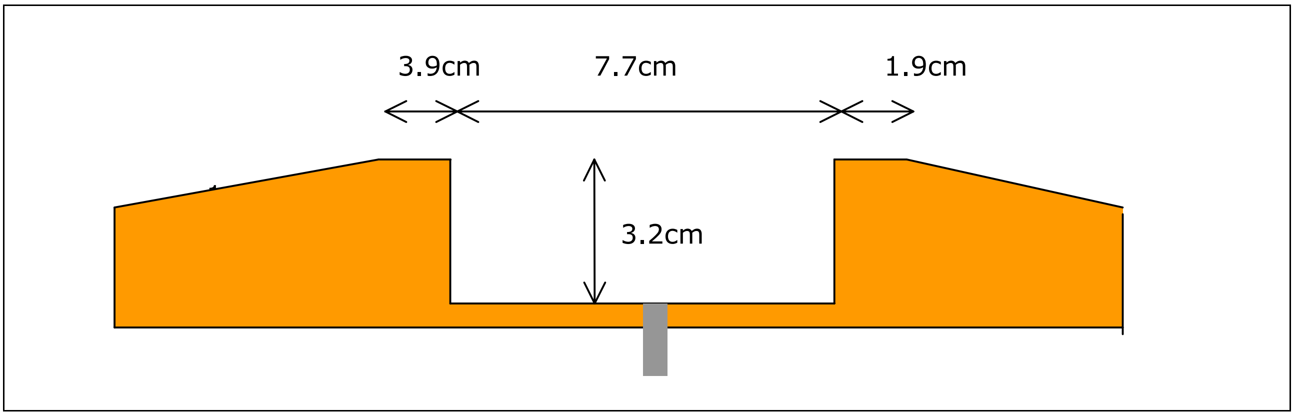


figure 4-3: Dimensions of the crest in the scale model

In some cases, experiments with a traditional dike have been executed. A traditional dike is constructed by closing of the crest basin. This is illustrated in figure 4-4.

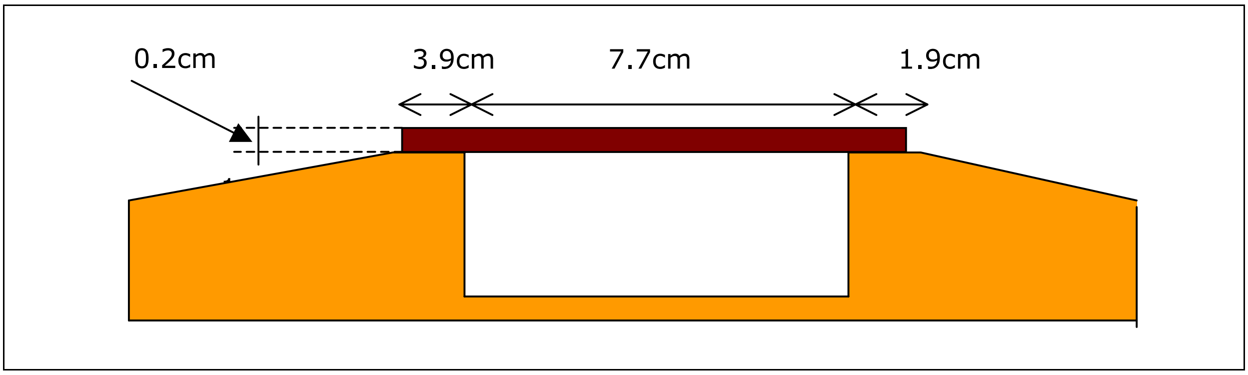


figure 4-4: Dimensions of a closed crest basin

The drains

The drains are designed in such a way that it is easy to change the layout of the drains. In the first test series the drains are connected to a tube which discharges the water into the overtopping tanks. This is illustrated in figure 4-5. After some tests, adaptations are applied in such a way that the water could flow freely and no suction could happen. This is illustrated in figure 4-6 and figure 4-7. Specification of the different drain types are shown in table 4-2.

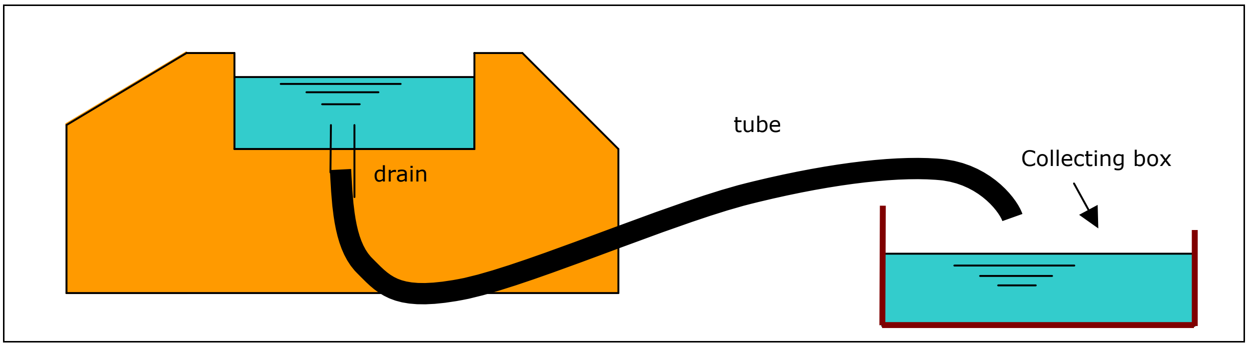


figure 4-5: Schematization of drain type I.

Drain number	L_{drain}	D_{drain}
	(m)	(m)
1	0.050	0.010
2	0.0525	0.020
3	0.055	0.030
tube		0.032

table 4-2: specifications of drain type 1

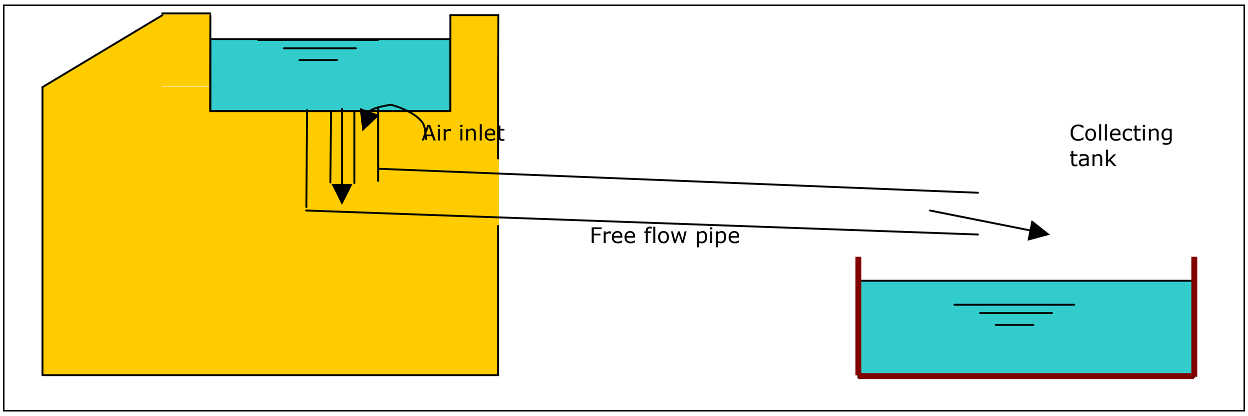


figure 4-6: Schematization of drain type II: Drain with free flow pipe (side view)

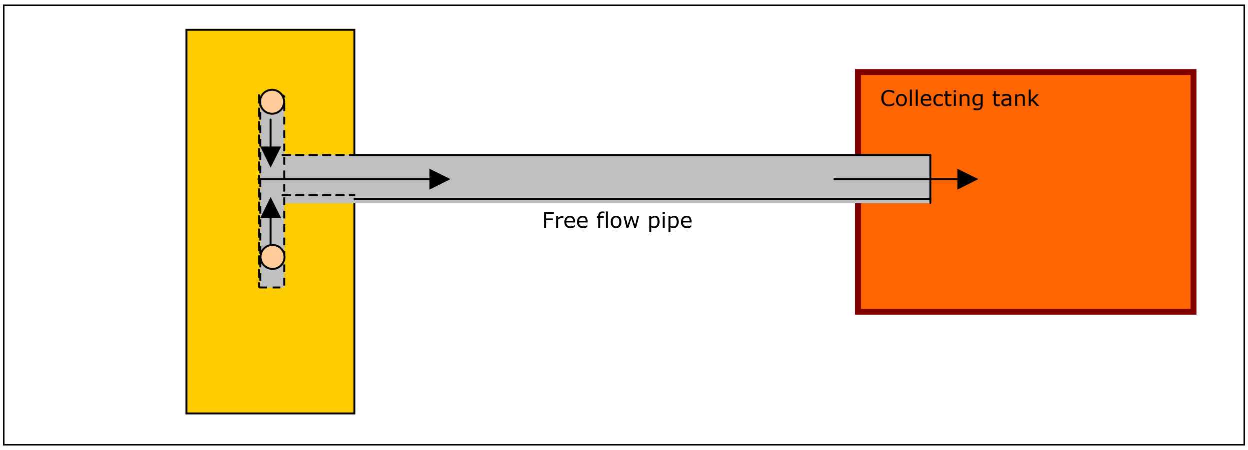


figure 4-7: Schematization of drain type II: Drain with free flow pipe (top view)

Four types of drains are used. All drain types have a circular shape. Variations were in the drain diameter and in the drain length. A schematization of the drains is given in figure 4-8; the specifications are shown in table 4-3.

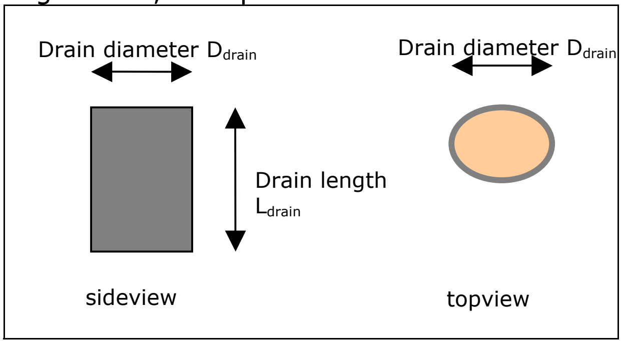


figure 4-8: identification of the drain parameters

Drain number	L_{drain}	D_{drain}
	(m)	(m)
1	0.05	0.010
2	0.10	0.010
3	0.10	0.020
4	0.10	0.030

table 4-3: specification of the used drains for drain type 2

The berms

Several tests with different berms heights are executed. The berm width is in all the test the same. The berms are made of wood with a smooth surface and are attached to the slope. A schematization is given in figure 4-9, the specifications are given in table 4-4.

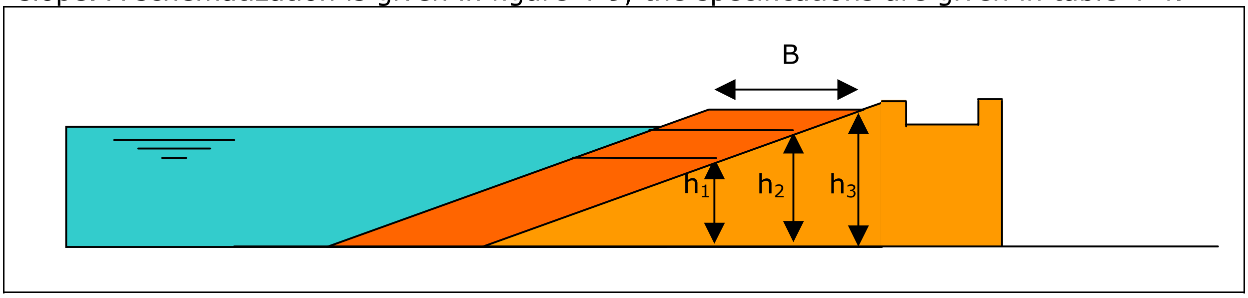


figure 4-9: berm attached to the slope

Berm number	h	B
	(m)	(m)
1	0.526	0.27
2	0.618	0.27
3	0.680	0.27

table 4-4: berm characteristics

Spare tanks for refilling during the tests

With severe wave overtopping, the water level in the flume is not stable since water is loosing. Therefore, there is a need to pump extra water into the flume to guaranty a stable water level. This water is pumped from spare tanks that are placed besides the flume.

4.4 Measuring instruments

The independent variables, which are obtained during the tests, are the following:

- Significant wave height (H_{m0})

  The significant wave height is derived from a spectral analysis with the program Auke

- Wave period ($T_{m-1.0}$)

  The wave period $T_{m-1.0}$ is derived from a spectral analysis. A Matlab code is written to derive this period. Reference is made to appendix VII.

- Water levels in the overtopping tanks
- Overtopping time
- Pressures in the crest basin

The wave height and period are measured with the use of wave gauges, the water levels in the overtopping boxes are measured by a water level meter.

Wave gauges
For the measurements, three analogue wave gauges are used. By using three wave gauges it is possible to separate the measured wave into an incoming and a reflecting wave. These wave gauges measure the water level by measuring the voltage difference between the two poles. This can be translated to water levels easily, since the voltage drop linearly with the water level. Calibrations have been carried out to determine the relationship between the water level and the measured voltage drop. In table 4-5, an overview of the calibrations is given. The range of the gauges is –10V to +10V. This is a range for the water level variations of around 0.50m, which is sufficient for all the tests. The sampling frequency of the gauges is set at 100 Hz.

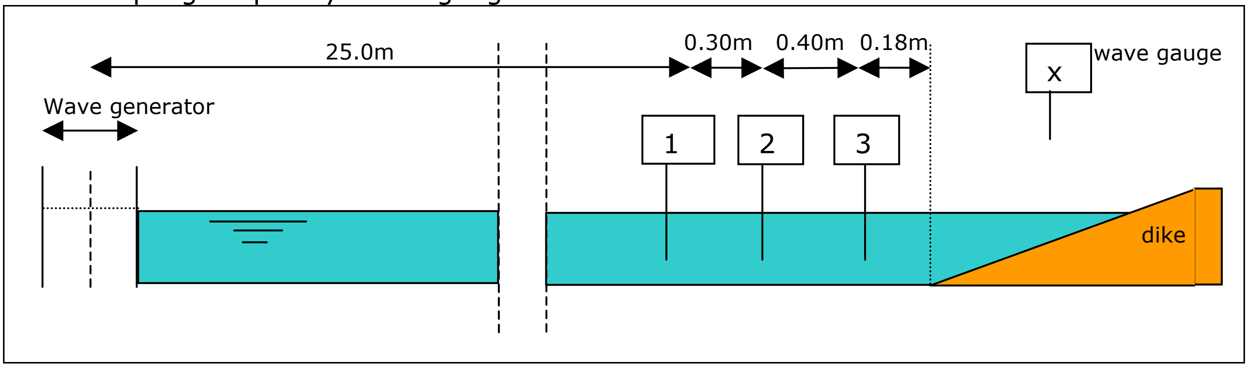


figure 4-10: position of the wave gauges in the model

wave gauge number	sensitivity
	(m/Hz)
1	0.0226
2	0.0246
3	0.0242

table 4-5: characteristics of the wave gauges

Overtopping tanks
Behind the dike, three overtopping tanks are placed. One tank collects all the water, which is overtopping the Crest Drainage Dike (tank1). The second tank is collecting all the water, which is drained through the crest basin (tank 2). The third tank is placed as an extra tank. In case tank 1 or tank 2 is totally filled, water can be pumped into tank 3. In the tanks, water level meters are installed. Characteristics of the overtopping tanks are shown in table 4-6.

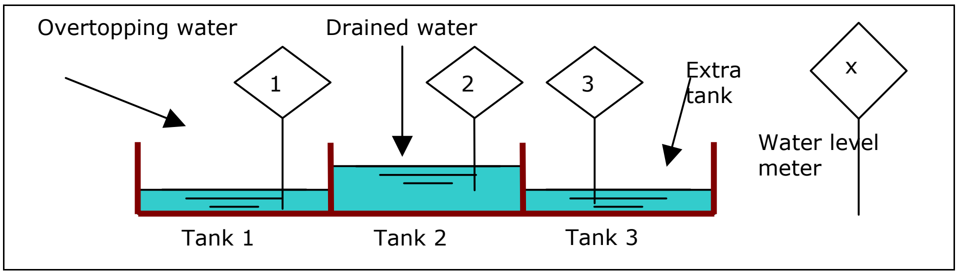


figure 4-11: overview of the overtopping tanks

tank number	surface
	(m^2)
1	0.751
2	0.758
3	0.748

table 4-6: Tank characteristics

Pressure meters
Pressure meters have been placed in the crest basin. These can measure the pressures on the crest basin during wave impact. The obtained data is not analyzed in this report. The measurements have been executed since this only required a small adaptation of the experiments. In case the data are required in a later stadium, there is no need for repeating the experiments. The data obtained from this research can be used for eventual future investigations.

Solitary wave measurement
To obtain data regarding solitary waves, experiments with solitary waves have been executed. This is done by closing off the dike with gates. These gates are opened during one wave. The overtopping water is caught in a small box and is weight. The water level in the crest basin is determined by measuring the water level in the crest basin.

Dependent variables
As described, the wave height and wave period are obtained with the use of wave gauges. The water levels in the overtopping tanks are determined by using the overtopping tanks and water level meters. From these independent variables, dependent variables can be obtained. These are:

Spectral periods such as $T_{m-1,0}$
The program Auke gives the possibility to analyze the measured record. The mean wave period and the peak period are calculated. However, the interest lies in the $T_{m-1.0}$ and this parameter cannot be determined with the program. Therefore, a method to analyze the given wave spectrum is identified and a Matlab code is written. The code of this program is given in appendix VII. Using this code, the $T_{m-1.0}$ can be determined.

Overtopping and drain discharge
The overtopping and drain discharges are determined by calculating the differences in volume in the overtopping tanks.

4.5 Set-up of the test model

In section 4.2, a description of the tests, which are needed to verify the hypotheses, is given. The tests are divided into different subsets. The subsets will be discussed in the sections below.

4.5.1 Subset A: Basic parameters Crest Drainage Dike

To compare all the variation in the tests, a basic set of data is necessary. The variations can be divided into two main aspects: the wave conditions and the geometry of the Crest Drainage Dike. In the experiments B-H, only one parameter (besides the significant wave height) is changed.

The Crest Drainage Dike layout with the basic parameters is shown in figure 4-12.

Wave starting parameters

- Spectrum JONSWAP spectrum
- Wave steepness s_0 = 0.05
- Significant wave height variable

Geometric starting parameters

- Berm none
- Crest freeboard R_c = 0.085m
- Drain type type II (see page 47 for a description of this type)
- Drain diameter D_{drain} = 0.01m
- Drain length L_{drain} = 0.10m

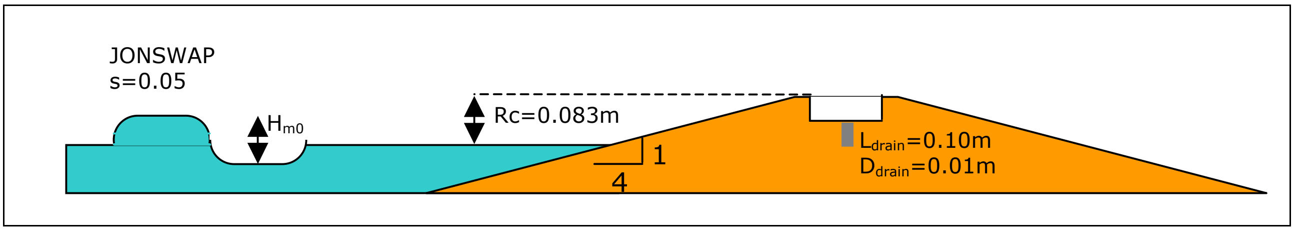


figure 4-12: Basic parameters

All the tests in the subsets B-H are identical to this set-up except for one specific parameter. An overview of the experiments of subset A is given in table 4-7.

Experiment		Number of tests
A01-A07	Basis	7
		7 total

table 4-7: Overview subset A: basis parameters Crest Drainage Dike

The specifications of subset A are shown in appendix VIII, table A-3.

4.5.2 Subset B: Traditional dike

To compare the Crest Drainage Dike with a traditional dike, several tests with a traditional dike have been executed. All the parameters are, except for the crest basin, as stated in figure 4-12. An impression is given in figure 4-4.

Experiment		Number of tests
B01-B05	Traditional dike	5
		5 total

table 4-8: Overview subset B: traditional dike

The specifications of subset B are shown in appendix VIII, table A-4.

4.5.3 Subset C: Influence drain capacity

To identify the influence of the drain capacity, several tests have been executed with different drains. Two types of drain systems have been used. (Drain system I and drain system II) The difference between these two systems is explained in section 4.3. The diameter and the length of the drains is varied. The used drains have been calibrated in specific calibration tests.

Experiment	Draintype	Drain diameter	Drain length	Number of tests
		(m)	(m)	
C01-C02	I	0.01	0.05	2
C03-C07	I	0.02	0.05	5
C08-C15	I	0.03	0.05	8
C16-C21	II	0.01	0.05	6
C22-C29	II	0.02	0.10	8
C30-C37	II	0.03	0.10	8
C38-C42	Calibration			5
				42 total

table 4-9: Overview subset C: Drain influence (draintype II)

The specifications of subset C, draintype I, are shown in appendix VIII, table A-5.
The specifications of subset C, draintype II, are shown in appendix VIIIX, table A-6.

Al the drain capacities and their entrance losses are determine in specific calibration tests. A full description of these calibrations is given in appendix X.

4.5.4 Subset D: Influence crest freeboard

To identify the influence of the crest freeboard, tests have been executed with three different crest freeboard.

Experiment	Crest freeboard R_c	Number of tests
	(m)	
D01-D09	0.045	9
D10-D14	0.125	5
		14 total

table 4-10: Overview of subset D: Influence crest freeboard

The specifications of subset D are shown in appendix VIII, table A-7.

4.5.5 Subset E: Influence wave steepness

To identify the influence of the wave steepness several tests have been executed where the wave steepness is varied. A wave steepness of 3 percent and 5 percent has been tested.

Experiment	Wave steepness s_0	Number of tests
E01-E10	0.03	10
		10 total

table 4-11: Overview subset E: Influence wave steepness

The specifications of subset E are shown in appendix VIII, table A-8.

4.5.6 Subset F: Influence wave spectrum

To identify the influence of the wave spectrum several tests have been executed with the JONSWAP spectrum, the Pierson-Moskowitz spectrum, a double-peaked spectrum and two narrow spectra.

The different shapes of the wave spectra are based on the Pierson-Moskowitz wave spectrum. This spectrum is identified by the following equation:

$$E_{JONSWAP}(f) = \underbrace{\alpha g^2 (2\pi)^4 f^{-5} e^{\left[-\frac{5}{4}\left(\frac{f}{f_{peak}}\right)^{-4}\right]}}_{\text{Pierson-Moskowitz}} \cdot \underbrace{\gamma^{e^{\left[-\frac{1}{2}\left(\frac{\frac{f}{f_{peak}}-1}{\sigma}\right)^2\right]}}}_{\text{Peak enhancement factor}}$$

Equation 4-1

Where

$E_{JONSWAP}(f)$	=	variance density spectrum of a JONSWAP spectrum	(m^2/Hz)
α	=	energy scale parameter	(-)
f	=	frequency	(Hz)
f_{peak}	=	peak frequency	(Hz)
γ	=	peak enhancement factor	(-)
σ	=	peak width parameter	(-)

This shape is adapted by changing the peak enhancement factor. This approach is based on the experimental work described in [Smith, 2004] In this study, it is stated that narrow spectra have a peak enhancement factor of 20 or 100. Wave spectra with a peak enhancement factor of 100 can be considered as regular waves. Common wave spectra are the Pierson-Moskowitz wave spectra (p.e.f. = 1) or the JONSWAP spectrum (p.e.f. =3.3).

In [Smith, 2004], no description of a double peaked spectrum is given. Therefore, as a starting point for double-peaked spectra, use has been made of [RIKZ,2003]. Here, it is stated that double-peaked spectra are created by a summation of two spectra with different peak periods. However, after several iterative processes, no wave spectra with clear identified double-peaked spectra were found. Therefore, a different approach has been followed. Double-peaked wave spectra are obtained by using a peak enhancement factor with a value of 0.1 and 0.5.

The following types of wave spectra have been tested:

Extreme narrow spectrum
In the experiments with an extreme narrow spectrum, the peak enhancement factor is set to 100. This is almost a regular wave.

Narrow spectrum
In the experiments with a narrow spectrum the peak enhancement factor is set to 20.

Pierson-Moskowitz spectrum or deep-sea spectrum
In the experiments with a Pierson-Moskowitz spectrum the spectrum is some wider then the JONSWAP spectrum. The peak enhancement factor is 1.

Double-peaked spectrum
Experiments with a double-peaked spectrum have been executed. Applying a peak enhancement factor of 0.5 and 0.1 creates the double-peaked spectrum that is used.

figure 4-13 gives an impression of the wave input spectra that are used. For illustration purposes, all the wave spectra are normalized. (The surface under the line is the same).

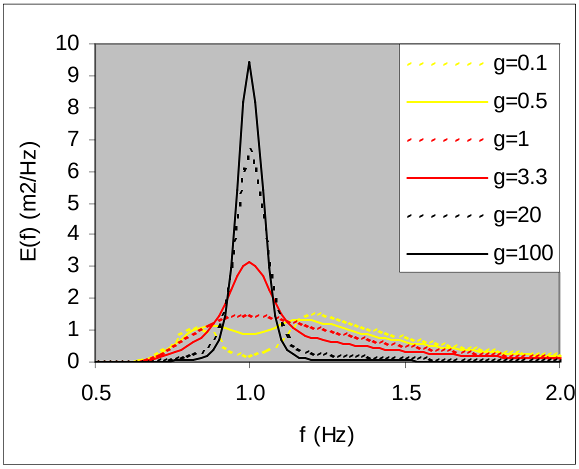


figure 4-13: the used input wave spectra (normalized)

Experiment	Wave type	Peak enhancement factor	Number of tests
F01-F06	Pierson-Moskowitz spectrum	1	6
F07-F12	Narrow spectrum	20	6
F13-F19	Extreme narrow spectrum	100	7
F20-F22	Double-peaked spectrum	0.5	3
F23-F24	Double-peaked spectrum	0.1	2
total			24

table 4-12 Overview subset F: influence wave spectrum

The specifications of subset F are shown in appendix VIII, table A-9.

4.5.7 Subset G: Influence of berms

To identify the influence of a berm, several tests with a berm attached to the dike, have been executed. The berm is placed under the water level, on the water level and above the water level. An overview is given in table 4-13.

Experiment	Berm level d_h	Wave steepness s_0	Number of tests
	(m)	(-)	
G01-G03	H	0.05	3
G04-G06	0	0.05	3
G07-G11	-2/3H	0.05	5
G12-G16	H	0.03	5
G17-G23	0	0.03	7
G24-G28	-2/3H	0.03	5
			28 total

table 4-13 Overview subset G: influence of berms

The specifications of subset G, for s=0.05, are shown in appendix VIII, table A-10
The specifications of subset G, for s=0.03, are shown in appendix VIIIX, table A-11

4.5.8 Subset H: Solitary waves

To analyze solitary waves, several tests with solitary waves have been executed. An overview is given in table 4-14.

Experiment		Number of tests
H01-H16	Solitary waves	16
		16 total

table 4-14 overview subset 14: solitary waves

The specifications of subset H are shown in appendix VIII, table A-12.

4.6 Procedures for execution of the tests

The procedure for the tests is given below.

- Calibrate the wave gauges
- Determine and correct the water level in the flume
- Determine a suitable wave height, wave period and wave spectrum
- Create the input file
- Measure the water level in all overtopping tanks
- Insert the proper drain in the crest structure
- Start the wave board
- Start the wave measurement
- Check the overtopping tanks
- In case the overtopping tanks are filled up to a certain level, add the same amount of water in the flume.
- In case the overtopping tanks are full, pump the water in another overtopping tank
- Stop the test
- Measure the water level in all overtopping tanks
- Analyze the wave record from wave gauges
- Empty the overtopping tanks.
- Conduct a new experiment and start at step 2

4.6.1 Applied corrections
During the experiments, new insights are obtained. Based on these insights, some adaptations have been applied. The single most important adaptation is the layout of the used drains. A start is made with drain type I (see table 4-2 and figure 4-6). However, it turned out that this drain layout did not give well-defined conditioned. This is mainly due to a combination of two physical aspects

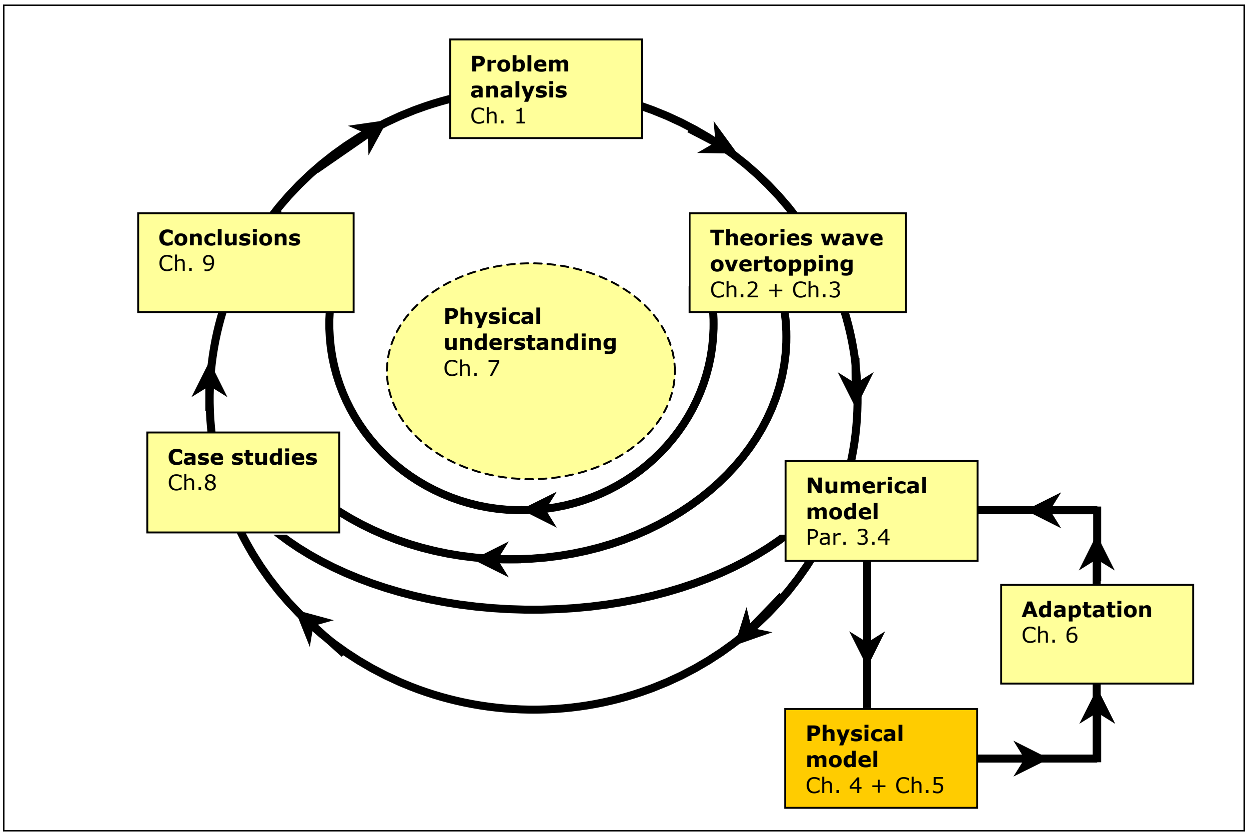
- Interference with Air bubbles.
- Suction due to a large height difference.

This combination gives complicated physical processes that are not covered by the numerical model. Therefore, a comparison between the results of physical model tests with drain type I and the numerical model is not possible. As a result of this observation, the drain layout is adapted to drain type II. (See page 47 for a description). The drain is designed in such a way that it is not possible that air bubbles are entrapped.

4.7 Results of the experiments
The results of the experiments are shown in appendix VIII. Here, the drain and wave overtopping discharges are shown as a function of the hydraulic and geometric boundary conditions.

4.8 Conclusions
The experiments that have been executed are described. The test set-up is adapted during the test series since it was observed that the draining aspects were not in a conditioned set-up. The result of the experiments is a data set in which the overtopping and draining discharges are presented as a function of the geometric and hydraulic boundary conditions. The goal of the experiments is to verify or reject the hypotheses stated in chapter 3. Therefore a thorough analysis of the data is necessary. This analysis is the subject of the following chapter.

5 Analysis of the experimental results



5.1 Introduction

In chapter 4, the executed experiments are described. The result of these experiments is a data set where the wave overtopping and draining discharges are given as a function of the geometric and hydraulic boundary conditions. The experiments are executed to verify or reject the hypotheses stated in section 3.8. Therefore a thorough analysis of the data set is required. This analysis is the main goal of this chapter.

This chapter analyses the results in three different ways. The first method is a comparison of the results with the theory adapted by van der Meer and Owen (section 5.2). The second method is comparing the data with each other (section 5.3). With the third method, a comparison is made between the data and the numerical simulations (section 5.4).

These comparisons display only a part of all the comparisons that are made throughout this report. An overview of all the comparisons that will be executed in this report is shown in table 5-1.

	Theory	**Physical tests**	**Numerical tests**
Theory	Ch. 2 (Owen vs. v.d Meer)	Section 5.2	Ch. 7
Physical tests	-	Section 5.3	Section 5.4
Numerical tests	-	-	-

table 5-1: Comparison between the theories, the physical tests and the numerical tests.

As can be seen from the table, this chapter compares the theory and the physical tests, the physical tests with each other and the physical tests with the numerical tests. Before this will be done a short description of the methods of comparison will be given.

Comparing theory and current test results (total overtopping).

All the test results regarding the overtopping of a traditional dike will be compared with the theory that has been discussed in chapter 2. A commonly used method to compare a theory with measurements is to plot a graph where the measured value is plotted on the x-axis and the calculated value is plotted on the y-axis. When the theory and test results match perfectly, the intersections are, by definition, on the line y=x. This is illustrated in figure 5-1.

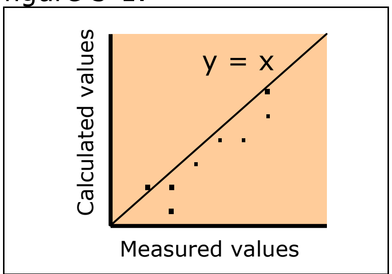


figure 5-1: Illustration of comparing theory and physical model tests.

The results of the physical model tests have been compared with the calculations based on the following theories:

- van der Meer
- Owen

The theories of Owen and van der Meer are described in chapter 2.

Comparing physical model tests with each other

To determine geometric and hydraulic aspects that might affect the physics of the Crest Drainage Dike, several test results will be compared with each other. Test results that will be compared and analysed are:

- Crest freeboard differences (3 series)
- Wave steepness differences (2 series)
- Traditional dike and Crest Drainage Dike. (2 series)
- Wave spectra (6 series)
- Berm differences (4 series)
- Drain lay-outs (4 series)

The most appropriate method would be a comparison as described in the previous section and shown in figure 5-1. This, however, can not be done since the hydraulic and geometric boundary conditions are usually not the same in the compared subsets. The wave heights and wave steepness are in the most cases slightly different and therefore a sound comparison as described above is not possible.

Another possible method of comparing two subsets is to use interpolated values. Therefore, statistical analysis is needed. As a result of this statistical analysis, exponential trend lines would be found and these could be compared. However, since there are not many data points per data set and the errors on the small wave overtopping discharges are quite large this method is not sufficient.

A third possible method to compare the results of the Crest Drainage Dike with overtopping results of a traditional dike, is to make use of the results of other tests such as [van der Meer, 2002] or [Owen, 1980]. However, since wave overtopping discharges should only be regarded as being within, at best, a factor of 3 of the actual overtopping rate [Douglass, 1985] this is not favorable due to large errors.

Since the three above stated methods to compare the results cannot be used, use is made of visual observation and interpretation to compare the results.

Comparing numerical test results with physical test results.
In chapter 3, a numerical model is described. This model is based on the theories described in chapter 2 and chapter 3. The outcomes of this model are compared with the measurements of the physical model tests as shown in figure 5-1.

5.2 Comparing theory and current test results (total wave overtopping discharge)

To check whether the results are in line with empirically derived equations, a comparison with the theory of Owen and van der Meer will be executed. The theories of van der Meer and Owen are treated in chapter 2. The dimensionless parameters of Owen and van der Meer are derived and a comparison based on these dimensionless parameters is given. The test results and the relations according to van der Meer are shown in figure 5-2 and figure 5-3. The 5% percent exceedance limits are shown as well. It is emphasized that in this section only the total wave overtopping discharges are compared.

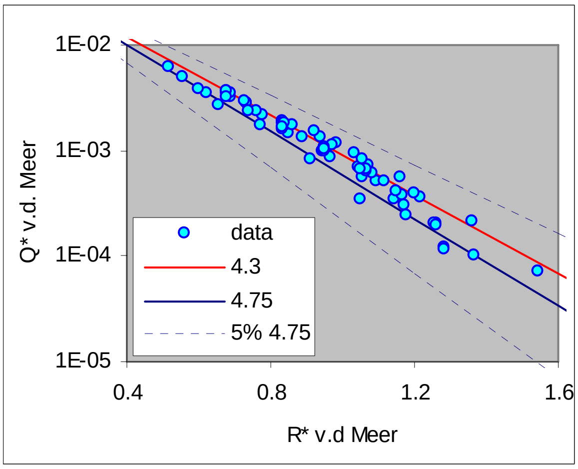


figure 5-2: Results tests and van der Meer equations

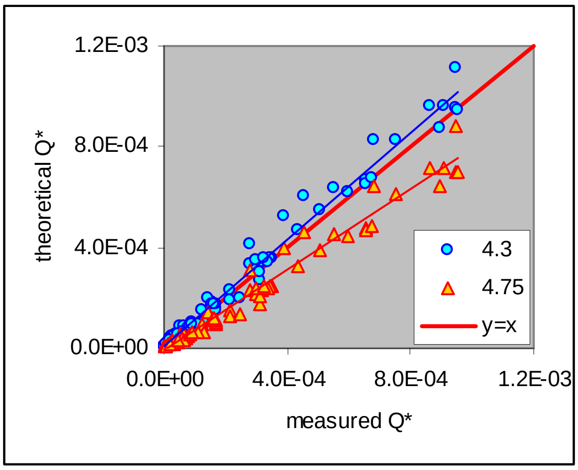


figure 5-3: Comparison measurements and theory (van der Meer).

From figure 5-2 and figure 5-3 it is derived that the measured total wave overtopping discharges are slightly higher than the wave overtopping discharges derived by van der Meer.

The same analysis has been carried out with respect to the wave overtopping discharge according to Owen. This is shown in figure 5-4 and figure 5-5

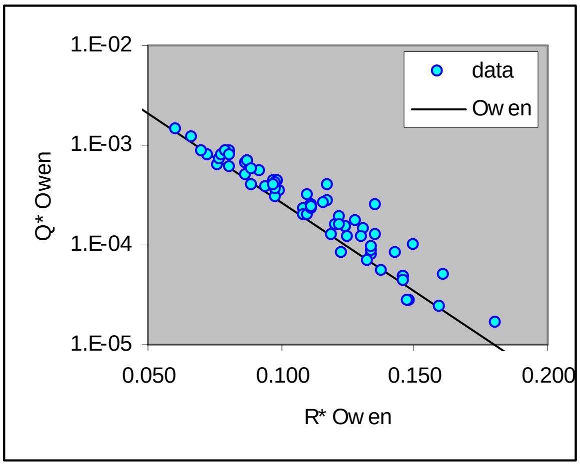


figure 5-4: Results tests a Owen equations

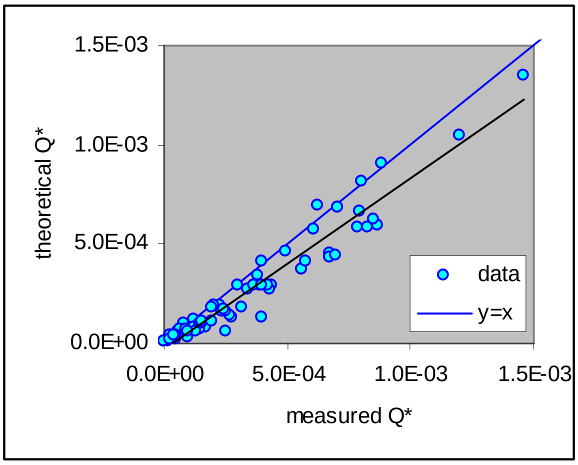


figure 5-5: Comparison measurements and theory (Owen)

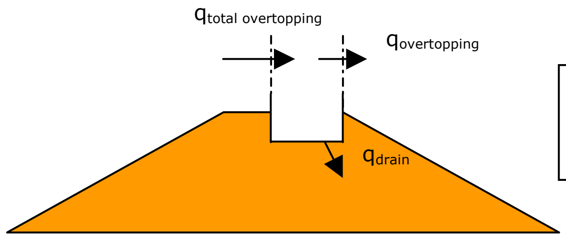
As can be seen in figure 5-3 and figure 5-5, the measured wave overtopping discharges are slightly higher than the theoretical wave overtopping discharges according to the theory of Owen and van der Meer. However, it should be noted that the differences are very small and are within error margins given by van der Meer. Therefore it can be concluded that the determination of the wave overtopping discharges have been executed in a proper way.

5.3 Comparing physical model tests

5.3.1 Introduction

The purpose of this section is to analyze the data and check which parameters have a significant influence on the efficiency of the Crest Drainage Dike.

The crest basin efficiency is expressed as the fraction of overtopping with respect to the total overtopping. See figure 5-6 for an impression.



Crest basin efficiency: $\varphi = \dfrac{q_{drain}}{q_{totalovertopping}}$

figure 5-6: Impression of crest basin efficiency

The crest basin efficiency depends on several hydraulic and geometric parameters. These parameters can be expressed in several dimensionless parameters. But which parameter would be favorable? Which relationship is needed? For engineering purposes it is needed to know the crest basin efficiency as a function of the geometric and hydraulic conditions (which are usually expressed in dimensionless parameter such as the dimensionless crest freeboard). But for scientific purposes, the dimensionless crest freeboard is very unsuitable for the prediction of the crest efficiency, since the relation between the wave overtopping discharges and the geometric and hydraulic boundary conditions is very inaccurate (prediction methods are a factor 3 within the actual overtopping rates). This problem illustrated in figure 5-7.

Geometric and hydraulic boundary conditions
- Wave height
- Crest freeboard
- Berms
- Wave steepness
- Etc

→

Prediction with use of
- Owen
- v.d. Meer

→

Predicted wave overtopping discharge (within a factor 3 of actual overtopping rates).

→

Determination crest basin efficiency
φ(geom/ hydr. BC)

figure 5-7: The crest basin efficiency as a function of the geometric and hydraulic boundary conditions

It is preferred to use the measured total wave overtopping discharges. This implies that the accuracy is much better and that the poor empirical wave overtopping relations do not contaminate the crest basin efficiency results. See figure 5-8 for an impression.

Measured total wave overtopping discharge ($q_{totalovertopping}$)

→

Determination crest basin efficiency
$\varphi(q_{totalovertopping})$

figure 5-8: The crest basin efficiency as a function of the total wave overtopping discharge

However, it should be realized that this method is only suitable for situations where the total wave overtopping discharge can be measured (e.g. laboratories) for specific conditions.

5.3.2 Traditional dike vs. Crest Drainage Dike

In section 3.8 the following hypothesis is given:

> *Hypothesis 1: The amount of overtopping water at a traditional dike equals the amount of overtopping water + the amount of drained water at a Crest Drainage Dike. This means that a negligible amount of water is reflecting from the crest basin.*

To verify this hypothesis the following test series have been used

- Subset A: Basic parameters Crest Drainage Dike (total overtopping)
- Subset B: Traditional dike (total overtopping)

Details of subset A are given in section 4.5.1 (page 51). Details of subset B are given in section 4.5.2 (page 51).

The total wave overtopping discharge ($q_{totalovertopping}$) in subset A is determined by adding the wave overtopping discharge ($q_{overtopping}$) and the drained discharge (q_{drain}). For an impression of the results, one is referred to figure 5-9. To account for slight differences in boundary conditions (such as crest freeboard, wave steepness etc.) the comparison has also been made with the use of dimensionless parameter. The relation with the use of the dimensionless parameters of van der Meer is shown in figure 5-10 and the relation with the use of the dimensionless parameters of Owen is shown in figure 5-11.

It can be seen in figure 5-9, figure 5-10 and figure 5-11 that there is no significant difference in the total wave overtopping discharges. Therefore it can be concluded that the use of a Crest Drainage Dike, which has been used in the experiments, has no significant influence on the amount of reflected water. In other words: hypothesis 1 is verified.

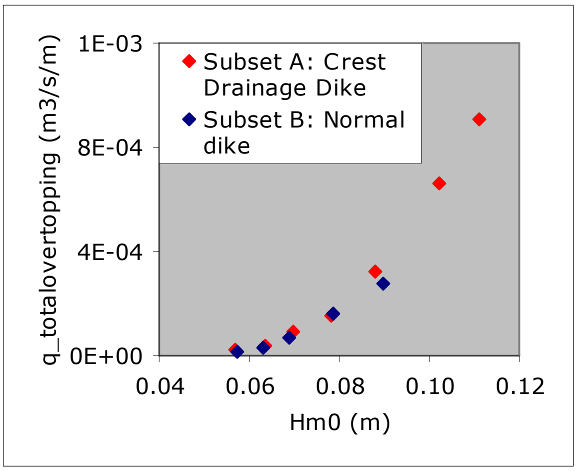


figure 5-9: Comparison of the total overtopping discharges of subset A and subset B

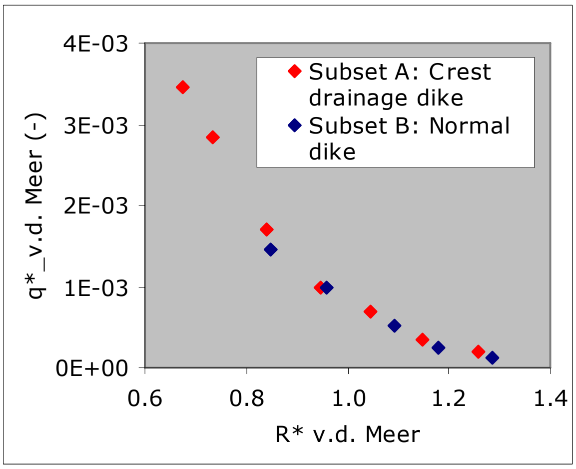


figure 5-10: Comparison subset A and subset B (van der Meer)

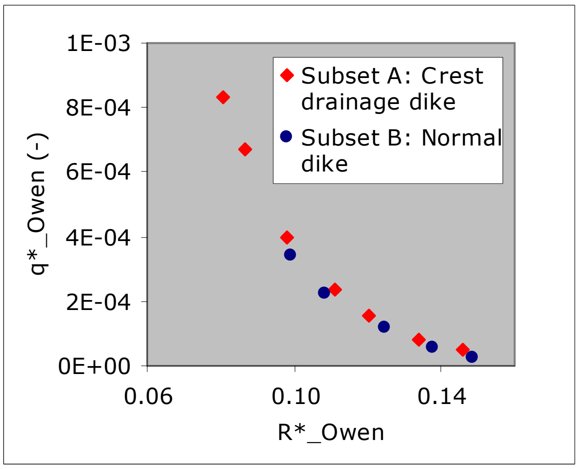


figure 5-11: Comparison subset A and subset B (Owen)

5.3.3 Influence wave steepness

To investigate the influence of the wave steepness, the following subsets have been used:

- Subset A: Basic parameters Crest Drainage Dike.
- Subset E: Wave Steepness

Details of subset A are given in section 4.5.1 (page 51). Details of subset E are given in section 4.5.5 (page 52).

The wave steepness of subset A is 5 percent and the wave steepness of subset E is 3 percent. The relation between the wave height and the percentage of water which is drained is shown in figure 5-12. According to this figure, the crest basin effectiveness is higher for steeper waves. This can be explained since the volume of a wave is less when the wave is steeper. Therefore less water comes in and the crest basin is more effective.

The crest basin effectiveness is plotted vs. the total wave overtopping discharge. This is shown in figure 5-13. Here, it can be seen that there is no significant difference between a wave with a steepness of 3 percent and a wave with a steepness of 5 percent. This can also be seen in figure 5-14, where the drained discharge is plotted vs. the total overtopping discharge. The overtopping discharges are plotted vs. total overtopping discharges in figure 5-15 (normal scale) and figure 5-16 (logarithmic scale).

From these figures, several interesting physical aspects can be obtained. These physical aspects will be discussed in section 5.3.9.

It can be concluded that, given a certain amount of total overtopping, there is no significant difference in the crest basin efficiency between a wave with a steepness of 3 percent and a wave of 5%. However, conclusions about the wave steepness in general cannot be given since the difference in wave steepness is limited. General conclusions could be given if the tests had an extremer wave steepness value (for example 1% and 7%). This is unfortunately not the case.

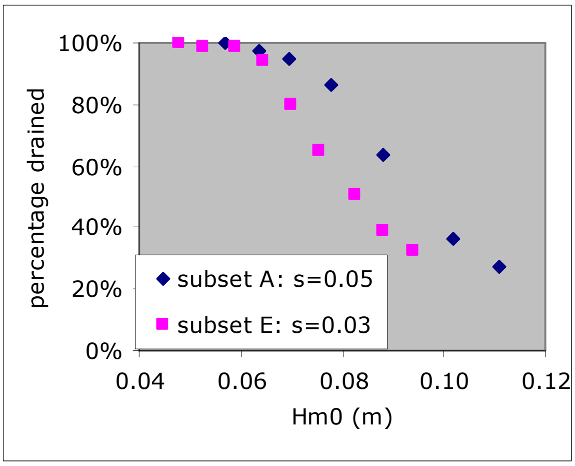


figure 5-12: φ vs. wave height (wave steepness)

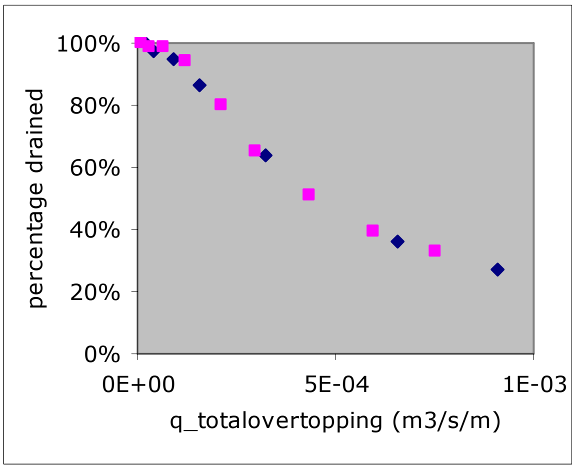


figure 5-13: φ vs. $q_{totalovertopping}$ (wave steepness)

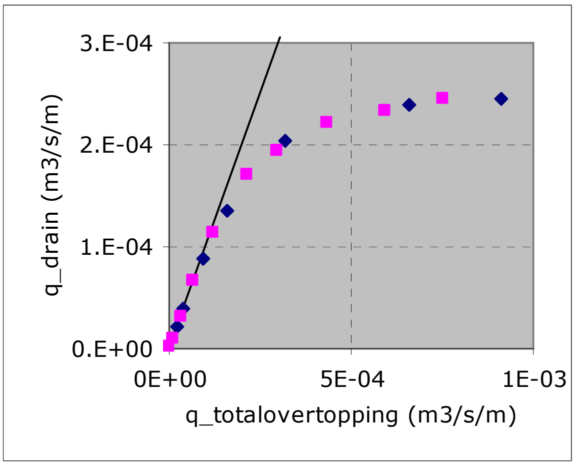


figure 5-14 q_{drain} vs. $q_{totalovertopping}$ (wave steepness)

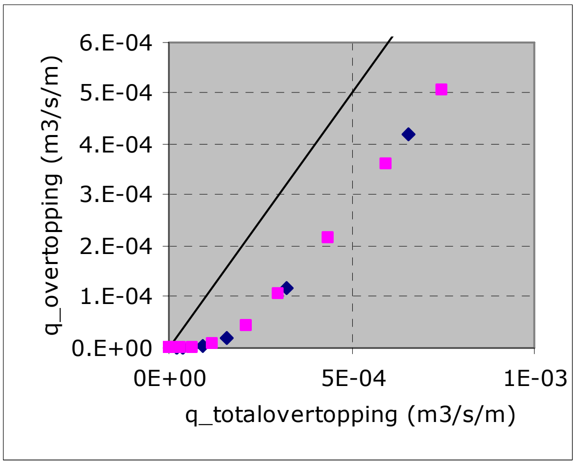


figure 5-15: $q_{overtopping}$ vs. $q_{totalovertopping}$ (wave steepness)

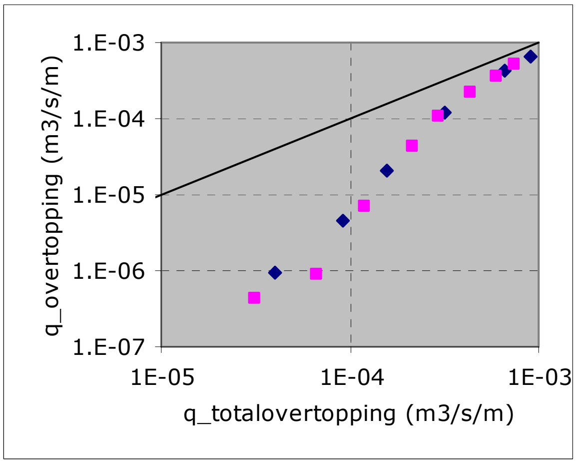


figure 5-16: $q_{overtopping}$ vs. $q_{totalovertopping}$ (wave steepness) (logarithmic scale)

5.3.4 Influence crest freeboard

To investigate the influence of the crest freeboard, use will be made of the following subsets:

- Subset A: basic parameter Crest Drainage Dike.
- Subset D: Influence crest freeboard

Details of subset A are given in section 4.5.1 (page 51). Details of subset D are given in section 4.5.4 (page 52).

The interest lies in the percentage of water that is drained. Therefore, the relation between the significant wave height and the percentage of water that is drained is shown in figure 5-17. It is obvious that the freeboard has a significant influence on the crest basin effectiveness since a lower crest freeboard results more overtopping water.

In figure 5-18, the crest basin effectiveness is plotted as a function of the total wave overtopping discharge ($q_{totalovertopping}$). It can be seen that there is a slight difference between the three test series. The tests with a lower crest freeboard are slightly more efficient. This is also shown in figure 5-19. In this figure, the drained discharge (q_{drain}) is compared with the total wave overtopping discharge ($q_{totalovertopping}$). It can be seen that the drained discharge is slightly higher for the tests with a lower crest freeboard.

It can be concluded that the crest basin efficiency is slightly higher for lower crest freeboards. Therefore, hypothesis 2 is rejected.

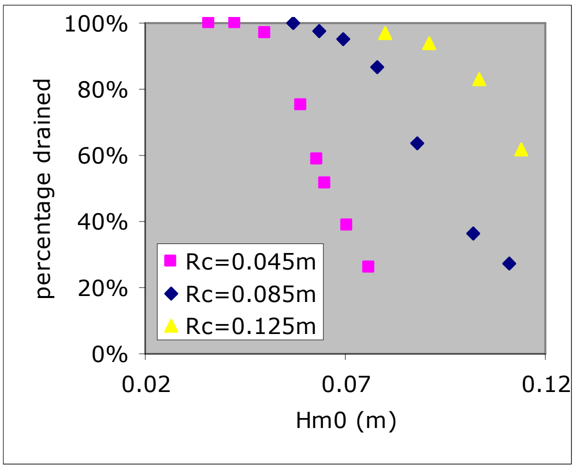


figure 5-17: φ vs. H_{m0} (Crest freeboard)

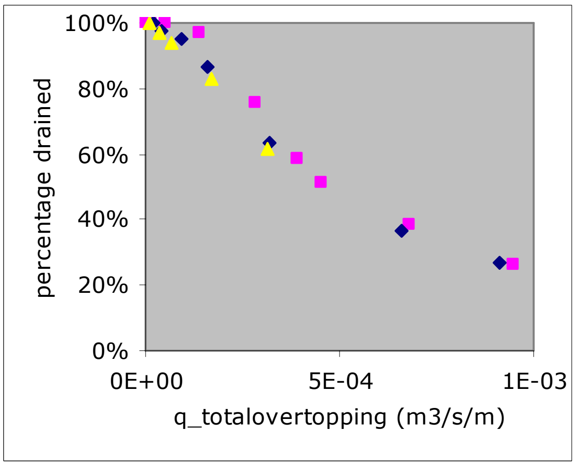


figure 5-18: φ vs. $q_{totalovertopping}$ (Crest freeboard)

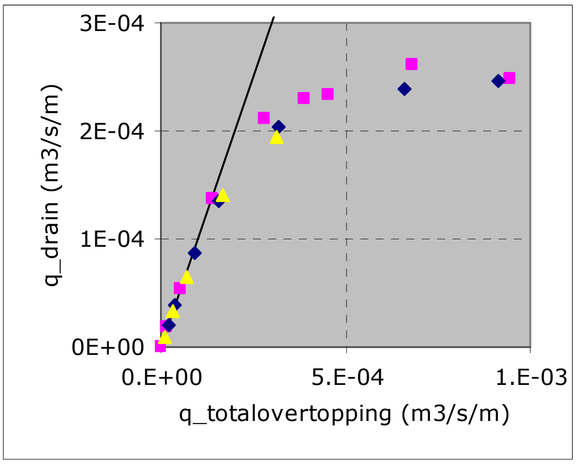


figure 5-19 q_{crest} vs. $q_{totalovertopping}$ (Crest freeboard)

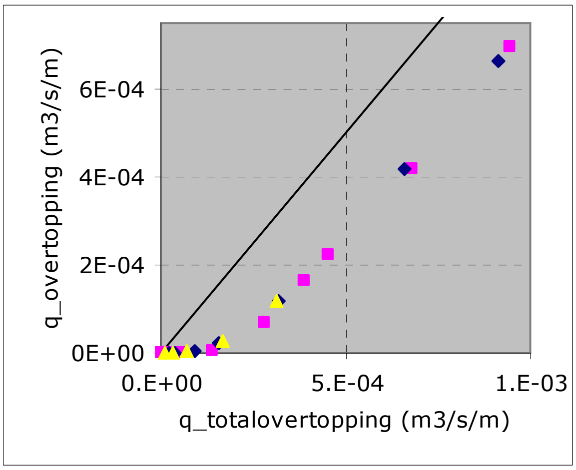


figure 5-20: $q_{overtopping}$ vs. $q_{totalovertopping}$ (Crest freeboard)

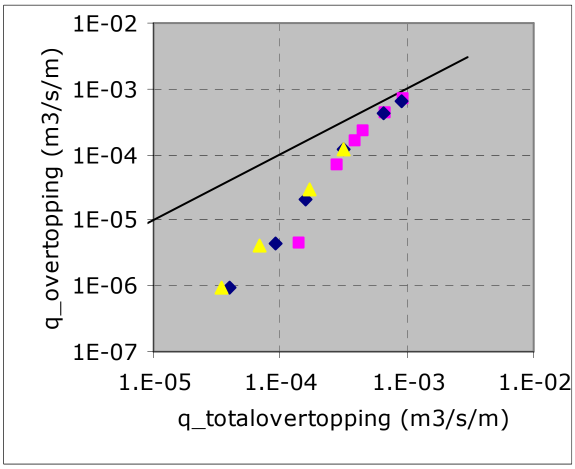


figure 5-21: $q_{overtopping}$ vs. $q_{totalovertopping}$ (Crest freeboard) (logarithmic scale)

5.3.5 Wave spectra

In section 3.8, the following hypothesis is given:

> *Hypothesis 3: The spectral shape has no significant influence on the efficiency of the crest drainage dike.*

To check this hypothesis, use will be made of the following subsets:

- Subset A: basic parameter Crest Drainage Dike.
- Subset F: Influence wave spectrum

Details of subset A are given in section 4.5.1 (page 51). Details of subset F are given in section 4.5.6 (page 52).

The relation between the significant wave height and the percentage of drained water, is shown in figure 5-22. To account for differences in the amount of total overtopping, the relation between the total wave overtopping discharge ($q_{totalovertopping}$) and the crest basin efficiency is shown in figure 5-23. In this figure, it can be seen that the experiments with a high peak enhancement factor (almost regular waves) are less efficient than the experiments with a low peak enhancement factor. There is no significant difference between the Pierson-Moskowitz spectrum (g=1), the JONSWAP spectrum (g=3.3) and the double-peaked spectra (g=0.5 and g=0.1).

This means that hypothesis 3 is in general not true. However, for engineering purposes where such highly peaked wave spectra do not exist, hypothesis 3 has been verified.

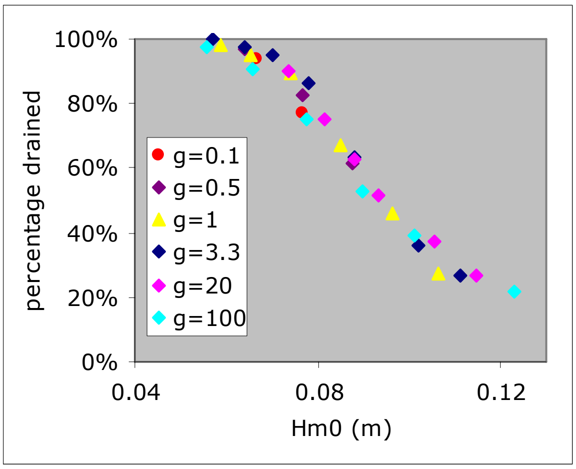


figure 5-22: : φ vs. H_{m0} (wave spectra)

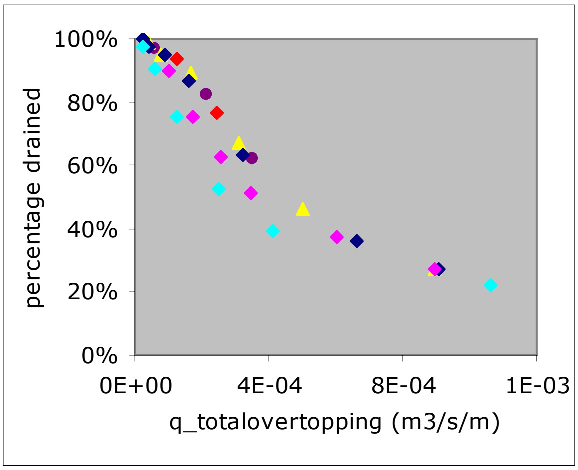


figure 5-23: φ vs. $q_{totalovertopping}$ (wave spectra)

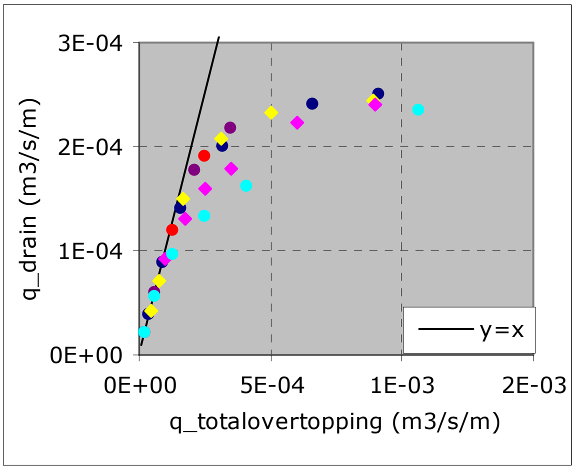


figure 5-24: q_{drain} vs. $q_{totalovertopping}$ (wave spectra)

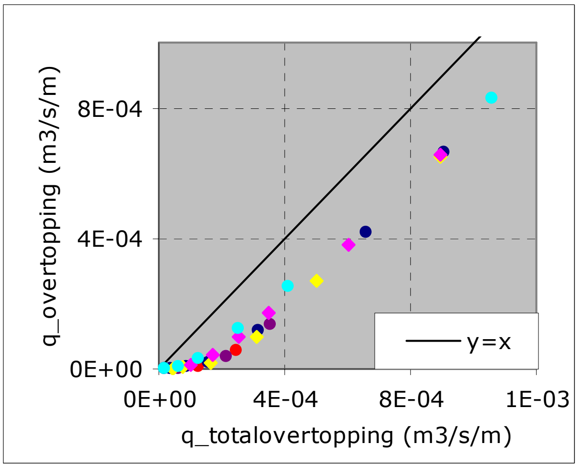


figure 5-25: $q_{overtopping}$ vs. $q_{totalovertopping}$ (wave spectra)

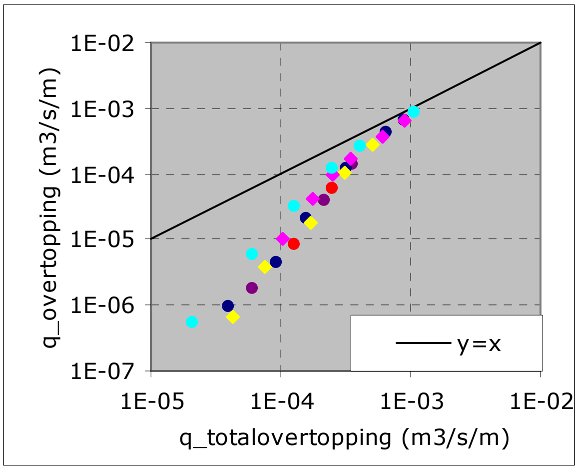


figure 5-26: $q_{overtopping}$ vs. $q_{totalovertopping}$ (wave spectra) (logarithmic scale)

5.3.6 Berms

To investigate the influence of berms, use have been made of the following subsets:

- Subset A: Basic parameter Crest Drainage Dike.
- Subset G: Influence berms

Details of subset A are given in section 4.5.1 (page 51). Details of subset G are given in section 4.5.7 (page 54).

The tests with berms were executed using two types of waves with different wave steepness (3% and 5%). It is shown in section 5.3.3 that the wave steepness has only a minor influence on the crest basin efficiency. Therefore, the results will not be treated separately but will be used as one set of parameters.
The relation between the significant wave height and the percentage of drained water is shown in figure 5-27. The differences between the 3% and the 5 percent waves are clearly seen. This influence does not exist for the relation between the total wave overtopping discharge and the crest basin efficiency. This is shown in figure 5-28.

A berm that is placed under the water level gives a slightly higher crest basin efficiency than a situation where no berm is placed. Surprisingly, a berm that is placed on the still water line gives slightly lower crest basin efficiencies. Since there is no theoretical back up for this influence and the differences are very small it can only be concluded that there might be a small difference.

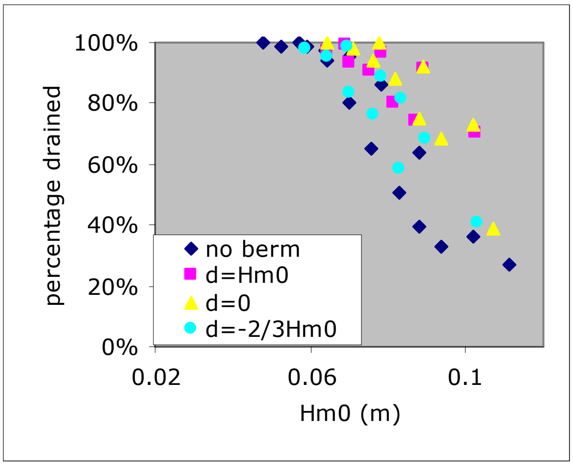


figure 5-27: φ vs. H_{m0} (berms)

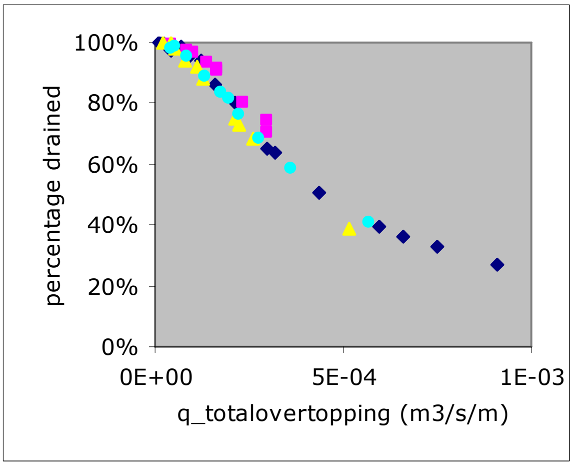


figure 5-28: φ vs. $q_{totalovertopping}$ (berms)

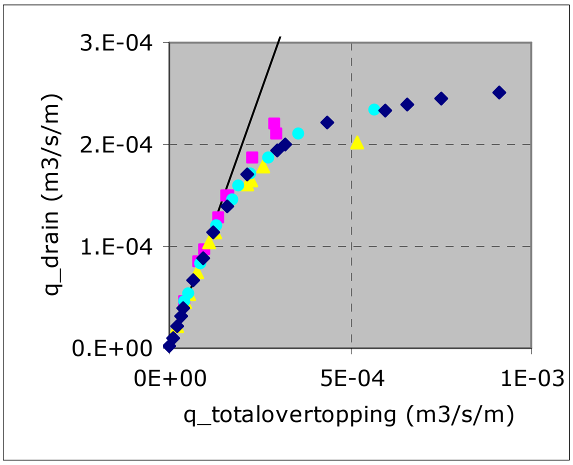


figure 5-29: q_{drain} vs. $q_{totalovertopping}$ (berms)

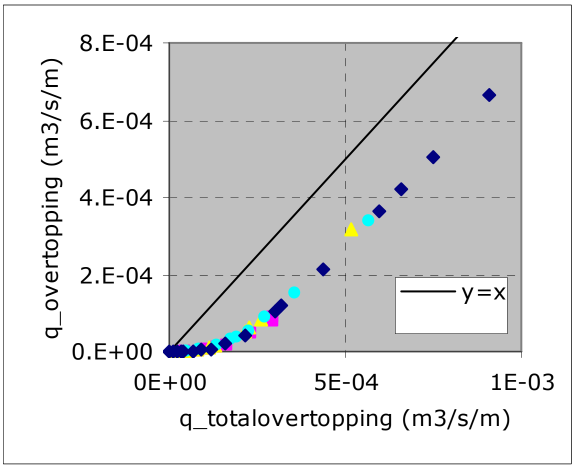


figure 5-30: $q_{overtopping}$ vs. $q_{totalovertopping}$ (berms)

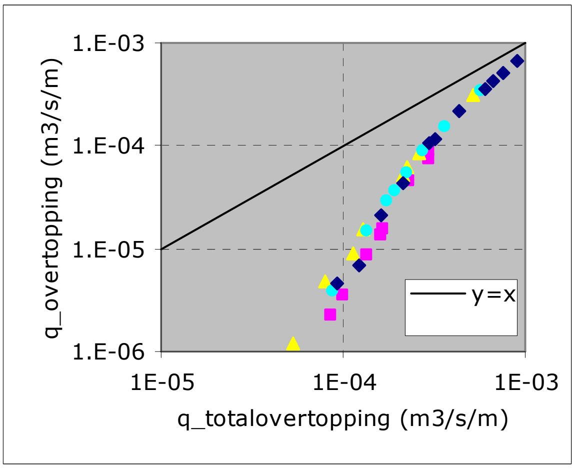


figure 5-31: $q_{overtopping}$ vs. $q_{totalovertopping}$ (berms) (logarithmic scale)

5.3.7 Comparison of all the results with the same drain parameters

Since the influence of the wave steepness, the crest freeboard, the berms and the wave spectra is very small and there appears to be no reflection in the crest basin, the results will temporarily be used as one single set. This gives the opportunity to show all the data in one graph and analyze the results. The graphs are shown in figure 5-32 and figure 5-33. All the data sets that have been discussed in section 5.3.2 until section 0 are displayed in this figure. Only the tests from subset F with narrow wave spectra (g=20) and extreme narrow wave spectra (g=100) are not taken into account since these conditions do not happen in nature and the results are significantly different.

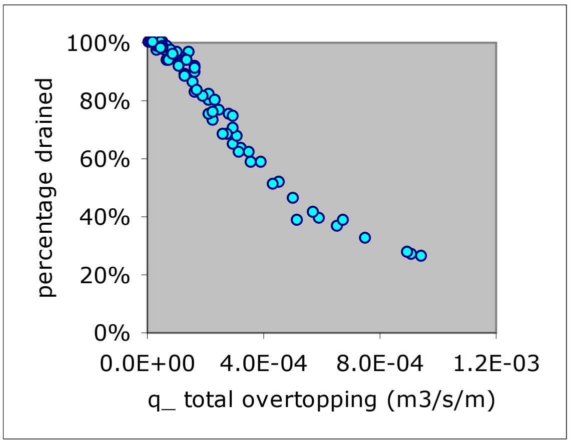


figure 5-32: φ vs. $q_{totalovertopping}$ (D_{drain} = 0.01m

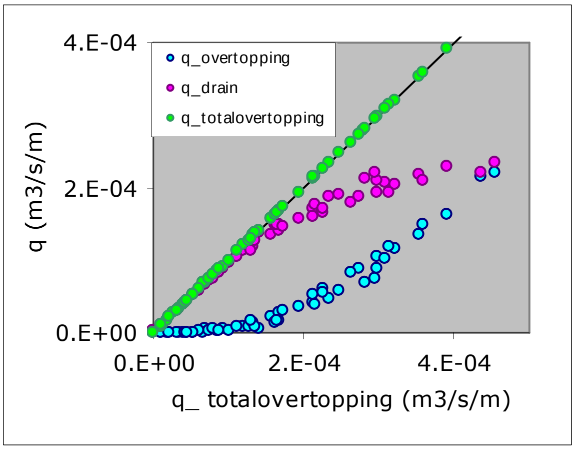


figure 5-33: q_{crest}, $q_{overtopping}$ and $q_{totalovertopping}$ vs. $q_{totalovertopping}$ (D_{drain} = 0.01m)

In figure 5-33, three types of discharges are shown. The green dots represent the total overtopping discharge ($q_{totalovertopping}$). These are by definition on the line y=x. The pink dots represent the discharges that are trapped in the crest basin and drained. The blue dots represent the average wave overtopping discharge ($q_{overtopping}$).
It can be seen that for a small amount of overtopping water, the amount of water that is drained is the same. Therefore the draining discharge starts at the line y=x. As a result of the drained discharge, the overtopping discharges start with the value zero. With an increasing total wave overtopping discharge, the draining discharges are growing as well and it can clearly be seen that the draining discharges go to an upper limit. This limit represents the maximum discharge capacity. With the increasing total wave overtopping discharge, the wave overtopping discharge is increasing as well.

As is shown in figure 5-32 there are only minor deviations between the different subsets. However, since there have been no variations in the drain parameters not much can be said about the physics of the Crest Drainage Dike. Therefore a sound analysis of the tests with different drain diameters is necessary before the physics of the Crest Drainage Dike will be studied in detail. This is the subject of the next section.

5.3.8 Influence drain diameter

To account for the influence of the drain capacity, a comparison between the different drain types is executed. The following subsets are used for this comparison.

- Subset A: Basic parameters Crest Drainage Dike
- Subset D: Influence crest freeboard
- Subset E: Influence wave steepness
- Subset F: Influence wave spectrum (except g=20 and g=100)
- Subset G: Influence berm

Subset A, D, E, F, G are all considered to have the same characteristics (see section 5.3.7 on page 73) and will therefore be considered as one subset with the following characteristics:

Drain diameter	D_{drain} = 0.01m
Drain length	D_{drain} = 0.10m

This subset is compared with the following subsets:

- Subset C: Influence drain diameter

In figure 5-34, the percentage of water that is drained is plotted as a function of the significant wave height Since the significant wave height does not say much about the amount of total overtopping discharge (due to variations in the crest freeboard, wave steepness, use of berms etc.), the crest basin efficiency as function of the total overtopping discharge is plotted in figure 5-35. From this figure, it can be seen that the drains with a smaller diameter (D_{drain} =0.01m) are draining less than the drains with a larger diameter (D_{drain}=0.02m and D_{drain}=0.03m).

In figure 5-36 it is clearly visible that more water is drained when using larger drains. In figure 5-37 it is showed that less water is overtopping. It can also be seen that the drain with a smaller length (D_{drain}=0.01m, L_{drain}=0.05m) is slightly less efficient then a longer drain (D_{drain}=0.01m, L_{drain}=0.10m).
In figure 5-36, it can be seen that the smaller drains (blue and yellow dots) have a significant smaller discharge capacity since they reach an upper limit. The larger drains (the green and pink dots) drain significant more water and “follow” the line y=x a longer time. The maximum drain capacity of the larger drains has not been reached since there is no upper limit visible.

In figure 5-37, the remaining wave overtopping discharge is plotted as a function of the total wave overtopping discharge. In this graph, it can be seen that the remaining overtopping water is significantly lower when a larger drain is applied. For the smaller drain (blue dots), it can be seen that the residual wave overtopping discharge is “following” the line y=x with a certain delay. This can be explained by the fact that the drain capacity is constant (because it has reached its upper limit). The vertical distance between the line y=x and the linear part of the blue dots equals the maximum drain capacity. This cannot be seen for the larger drains since the upper drain capacity has not been reached.

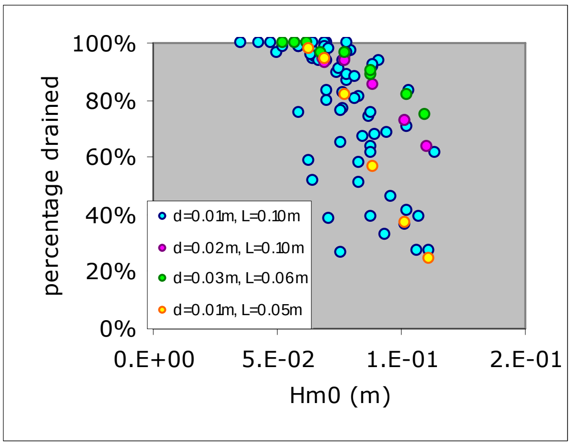


figure 5-34: φ vs. H_{m0} (drains)

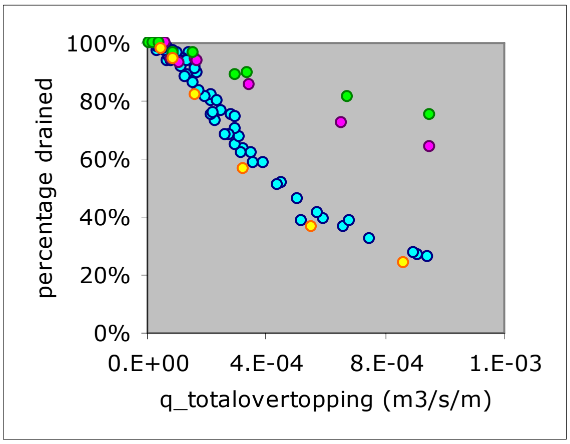


figure 5-35: φ vs. $q_{totalovertopping}$ (drains)

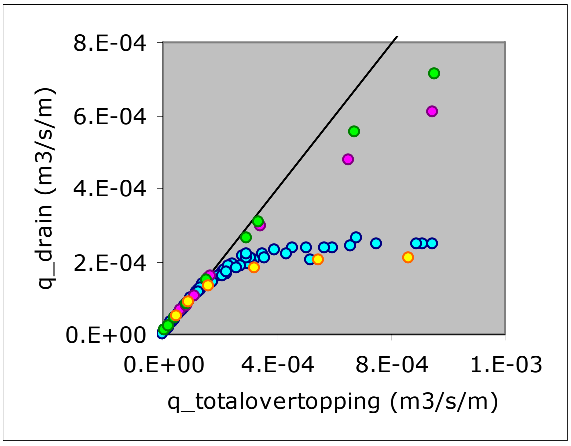


figure 5-36: q_{drain} vs. $q_{totalovertopping}$ (drains)

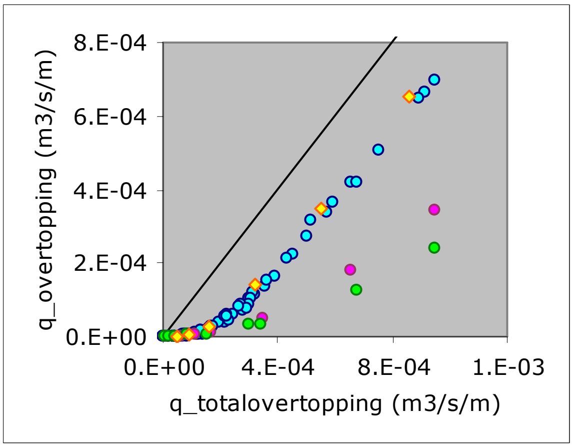


figure 5-37: $q_{overtopping}$ vs. $q_{totalovertopping}$ (drains)

5.3.9 Physical interpretation of the test results

The figures shown in section 5.3.8 show a clear relation between the drain parameters and the crest basin efficiency. However, this does not say much about dimensionless parameters. These are needed to "translate" the physical model test to real existing situations. The use of dimensionless parameters for traditional dikes has been explained in section 2.2.1 (page 7). In section 3.6 (page 37), some dimensionless parameters are suggested to use in overtopping theories regarding the Crest Drainage Dike. In this section, the wave overtopping discharges are made dimensionless with the maximum drain capacity.

Dimensionless total wave overtopping discharge
In figure 5-38 and figure 5-39, the total overtopping discharge ($q_{totalovertopping}$) is made dimensionless with the maximum drain capacity ($q_{drain,\ max}$). This maximum drain capacity is determined with specific calibration tests. (see appendix X). The graphs are based the data shown in figure 5-35 (page 75). A reference line is drawn in the figure. This reference line indicates the drained percentage in case the drain would always drain its maximum capacity and no wave would directly overtop the crest basin. For example: suppose the total wave overtopping discharge is twice the maximum of the drain capacity. This means that the dimensionless total overtopping parameter is 2. Suppose the drain is draining with its maximum capacity. The crest basin efficiency would be ½ = 50 %.
The mathematical expressions for the asymptotes of figure 5-38 and figure 5-39 are:

- $y = 1$ for $0 \leq x \leq 1$
- $y = 1/x$ for $x > 1$

where
y = percentage of water that is drained
x = dimensionless total overtopping discharge ($q_{totalovertopping}/ q_{drain,max}$)

Dimensionless drain discharge
In figure 5-40 and figure 5-41, the draining discharges are made dimensionless with the maximum drain capacities. The graphs are based on the data shown in figure 5-36 (page 75). A value of 1 indicates that the drain is draining with its maximum capacity. A reference line is drawn in the figures. These reference lines indicate the theoretical maximum drained discharge. When the total overtopping capacity is below the maximum drain capacity, the crest efficiency is, by definition, 100 percent. This is indicated with the line z=x. Since it is by definition impossible to drain more water than the total wave overtopping discharge, it is impossible to get a point on the left side of the black reference line. The mathematical expressions for the asymptotes of figure 5-40 and figure 5-41 are:

- $z = 1$ for $x > 1$
- $z = x$ for $0 \leq x \leq 1$

Where
z = dimensionless drain parameter ($q_{drain}/q_{drain,max}$)
x = dimensionless total overtopping discharge ($q_{totalovertopping}/ q_{drain,max}$)

Dimensionless wave overtopping discharge
In figure 5-42 and figure 5-43, the overtopping discharges are made dimensionless with the maximum drain capacity. The graph is based on the data shown in figure 5-37 (page 75). The line p=x is plotted to indicate the overtopping discharges in case no Crest Drainage Dike is used. (For a traditional dike, the wave overtopping discharge is by definition equal to the

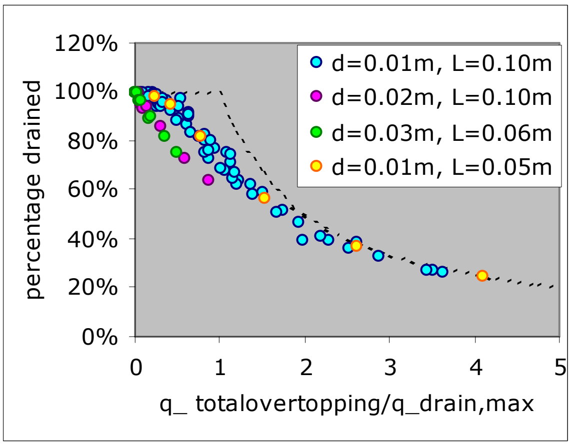


figure 5-38: φ vs. $q_{totalovertopping}$ (dimensionless)

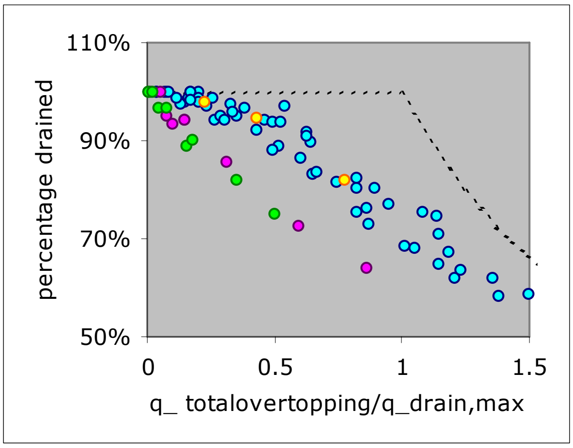


figure 5-39: φ vs. $q_{totalovertopping}$ (dimensionless, zoomed scale)

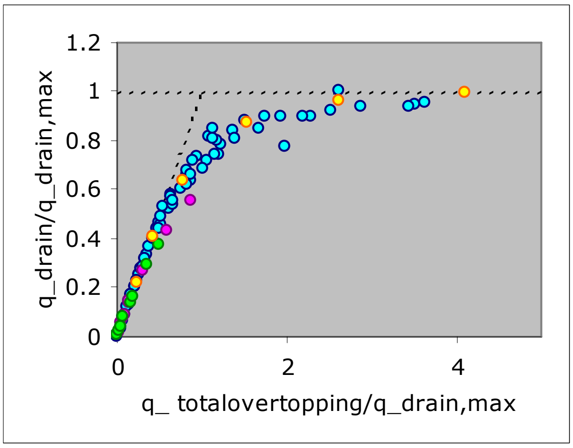


figure 5-40: Dimensionless q_{drain}

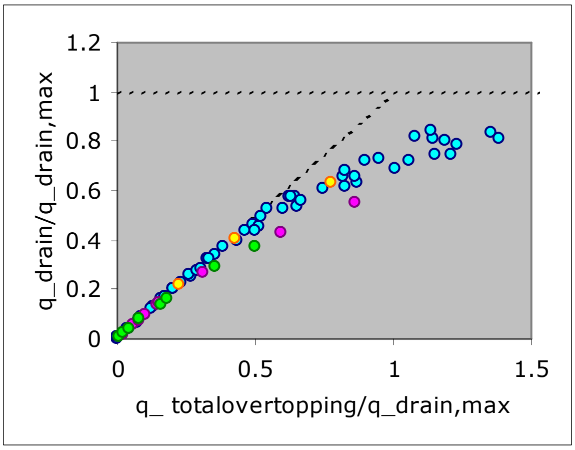


figure 5-41: Dimensionless q_{drain} (zoomed scale)

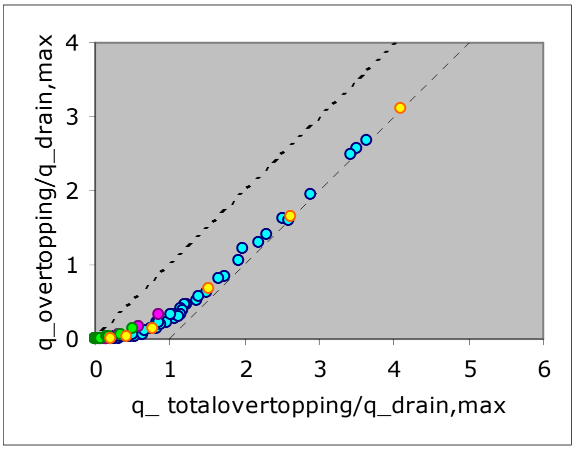


figure 5-42: dimensionless $q_{overtopping}$

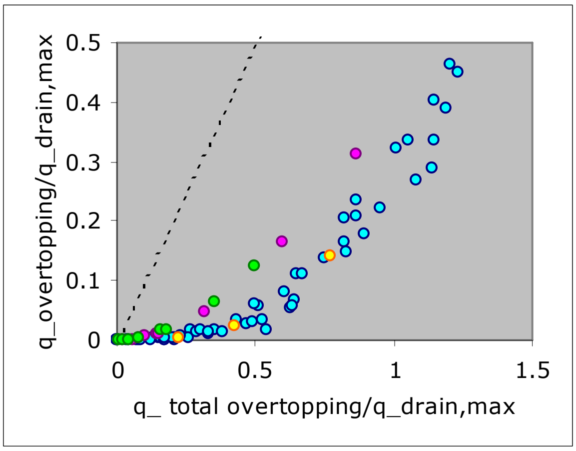


figure 5-43: dimensionless $q_{overtopping}$ (zoomed scale)

total overtopping discharge). It can be seen that the overtopping discharge is "following" the line p=x with a certain delay. This is caused by the draining discharge. When the drain capacity reaches its maximum capacity, the overtopping discharge is the total discharge minus the drained discharger. Expressed in formulae:

$$q_{overtopping} = q_{totalovertopping} - q_{drain} \quad \textbf{Equation 5-1}$$

Dividing Equation 5-1 with $q_{drain,max}$ gives:

$$\frac{q_{overtopping}}{q_{drain,\max}} = \frac{q_{totalovertopping}}{q_{drain,\max}} - \frac{q_{drain}}{q_{drain,\max}}$$ **Equation 5-2**

For a larger total wave overtopping discharge, the drain is draining with its maximum capacity thus:

$q_{drain} = q_{drain,\max}$ For a large $q_{totalovertopping}$, Equation 5-2 is rewritten as:

$$\frac{q_{overtopping}}{q_{drain,\max}} = \frac{q_{totalovertopping}}{q_{drain,\max}} - 1$$ **Equation 5-3**

or

p = x-1

where

p = dimensionless overtopping discharge ($q_{overtopping}/q_{drain,max}$)
x = dimensionless total overtopping discharge ($q_{totalovertopping}/ q_{drain,max}$)

Conclusion
Using dimensionless overtopping discharges, it is possible to construct theoretical maximum efficiencies as shown in figure 5-38 until figure 5-43. The theoretical lines can be compared with the actual efficiencies.
In figure 5-38 until figure 5-43, it can be seen that the tests where a larger drain diameter has been used, have a relatively lower efficiency then the smaller drains. The distance between the theoretical maximum (the black reference lines) and the measured points is assumed to be a function of the (dimensionless) crest basin volume. A temporary dimensionless crest basin volume is suggested in section 3.6 (page 37). An extension of this theory is given in section 7.5 (page 100). This is done with the use of numerical data and insights obtained in case studies.

5.3.10 Solitary waves
In section 3.8 the following hypothesis is given:

Hypothesis 4: The reduction factor for the buffer efficiency θ is 0.8

To verify or reject hypothesis 4, the following subset is used:

- Subset H: solitary waves

Details of subset A are given in section 4.5.8 (page 54). A visualization of the results is shown in figure 5-44.

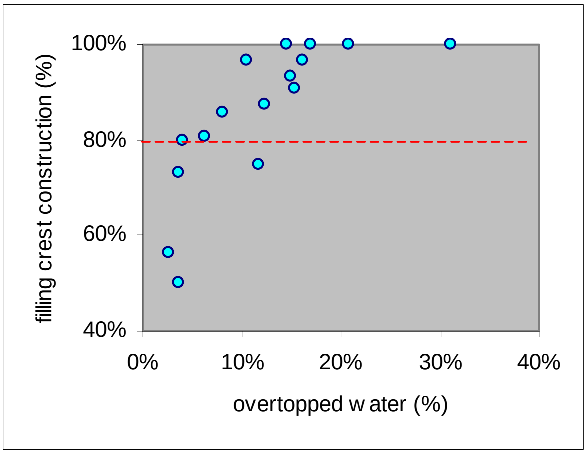


figure 5-44: Results of subset H

The x-axis represents the percentage of water that has overtopped the Crest Drainage Dike. The y-axis represents the percentage of the crest basin that is filled with water. In the figure, it can be seen that if the crest basin is filled with less than 80 % of its own volume, the percentage of overtopping water is less then 5%. For high overtopping volumes, the crest can be regarded as totally filled. Although the stated hypothesis cannot be verified or rejected by the implications of these results, the assumed 80% efficiency seems to be a good estimate of the efficiency. Therefore, this estimate will be maintained in the numerical program.

5.4 Comparing the numerical and physical test results.

To check the numerical model described in section 3.5.3, a comparison between the results of the model and the results of the physical model tests is made. This is done for all the subsets. Three groups of subsets are plotted separately. The groups are divided based on the difference drain capacities.
The groups are divided like this since there seems no significant difference in the results of the subgroups where a drain diameter of 1 cm is chosen. Therefore, these tests are all considered as one subset. The other two subsets are the tests where a drain diameter of 2cm and 3 cm is chosen. The comparison between the numerical model and the physical model is shown in figure 5-45, figure 5-46 and figure 5-47.

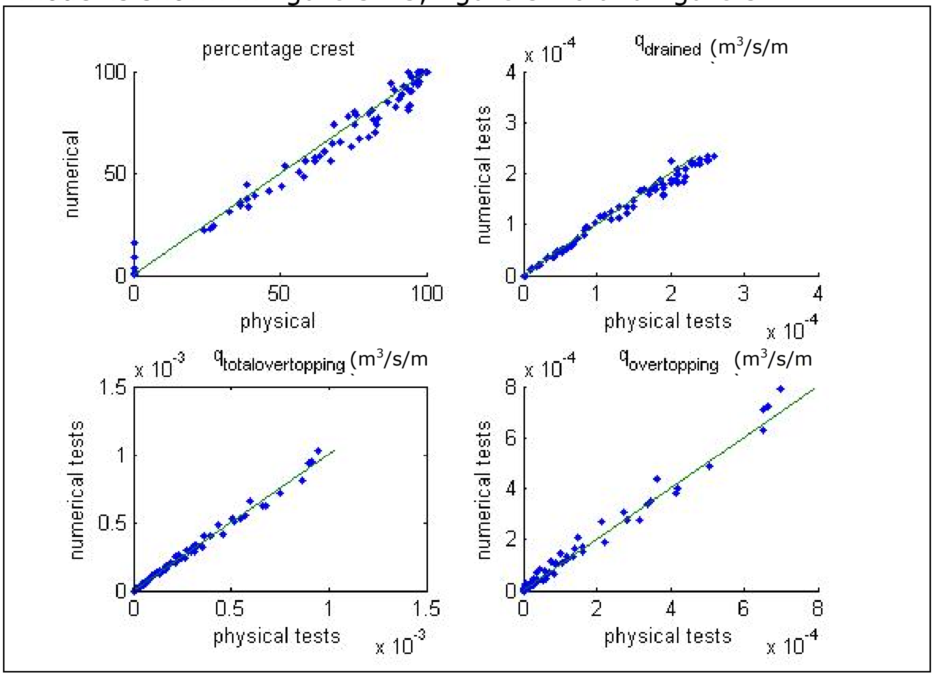


figure 5-45: comparison between model and test results for D_{drain}=0.01

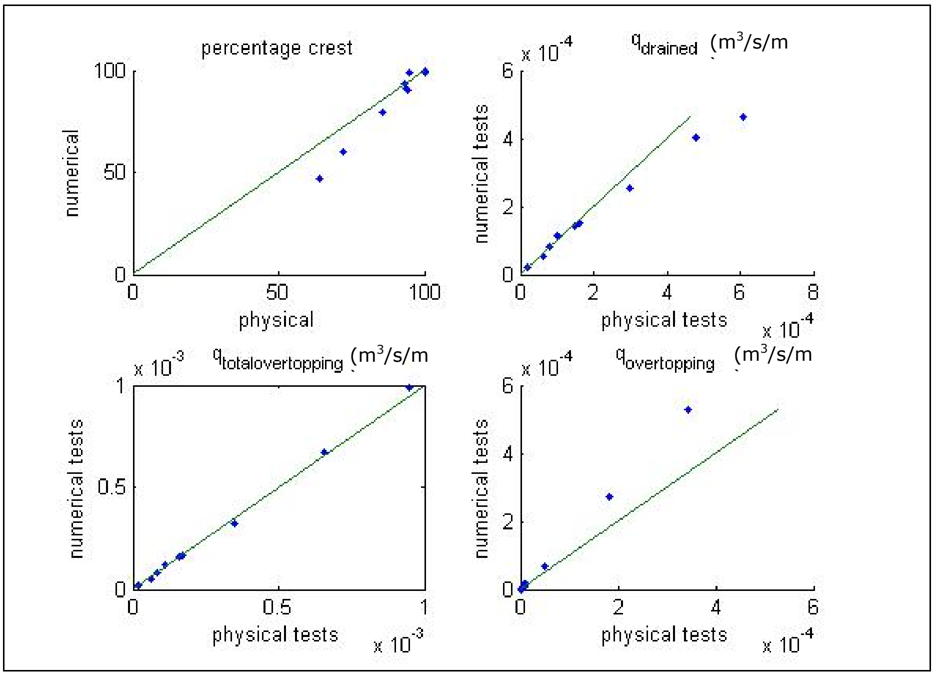


figure 5-46: comparison between model and test results for D_{drain}=0.02

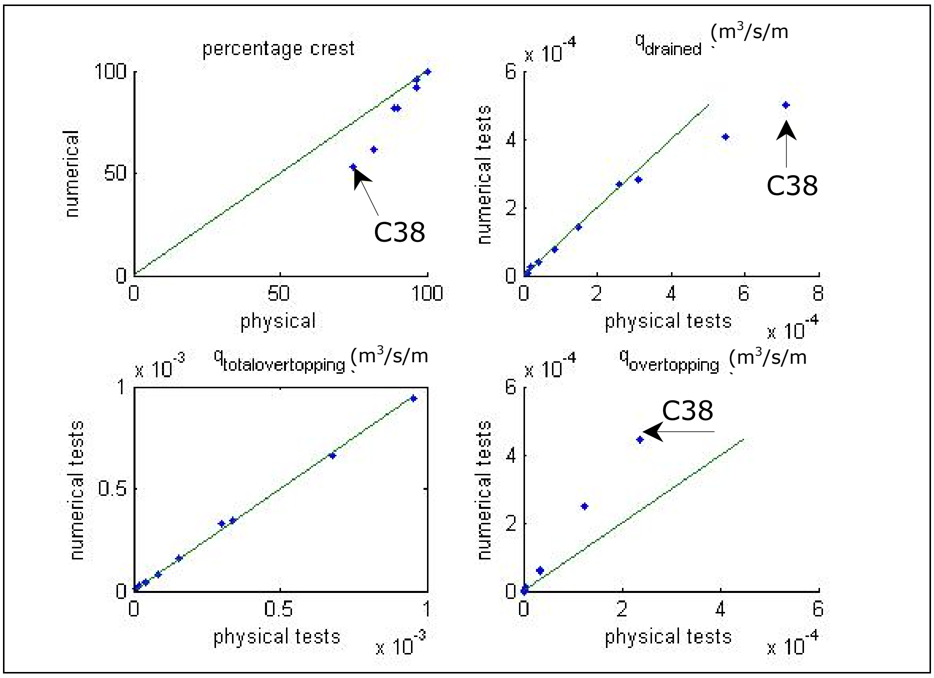


figure 5-47: comparison between model and test results for D_{drain}=0.03

The graphs present four different physical aspects of the Crest Drainage Dike. The graph on top left (percentage crest) represents the percentage of water that is trapped in the crest basin (the crest basin efficiency). The graph on top right ($q_{drained}$) represents the average discharge through the drain. The graph on the bottom left shows the total overtopping discharge ($q_{totalovertopping}$). The graph on the bottom right shows the overtopping discharge ($q_{overtopping}$).

As can be seen in figure 5-45, the numerical model predicts the actual overtopping rates very accurately. However, in the graph on top right it can be seen that for higher amounts of drained water, the numerical model under predicts the actual values of drained discharges. This is also visible in the graph on the bottom left. Since less water is drained, more water is overtopping.

This phenomenon is better visible in figure 5-46 and figure 5-47. Here it is very obvious that the drained discharge are strongly under predicted and that considerably more overtopping water is predicted than the actual rate of overtopping. This influences the crest basin efficiency considerably as can be seen in the figures on the top left. The calculated crest basin efficiency is considerably lower than the actual crest basin efficiency. Therefore a closer analysis of the model regarding this error is necessary. This will be done in the next chapter.

5.5 Conclusions

The test results from the physical model are compared with current wave overtopping relations (Owen and van der Meer), are compared with each other and are compared with the numerical model.

The total wave overtopping rates fit very well in the relation, which is given by van der Meer and Owen. Therefore it can be concluded that the determined total overtopping volumes are very well in line with the current overtopping models.

The subsets that have been compared with each other show only slight differences regarding the influence of wave steepness, crest freeboard, berms or spectral shape. It is emphasized that this statement does not hold for the total wave overtopping discharge but only for the crest basin efficiency, given a certain amount of total overtopping.

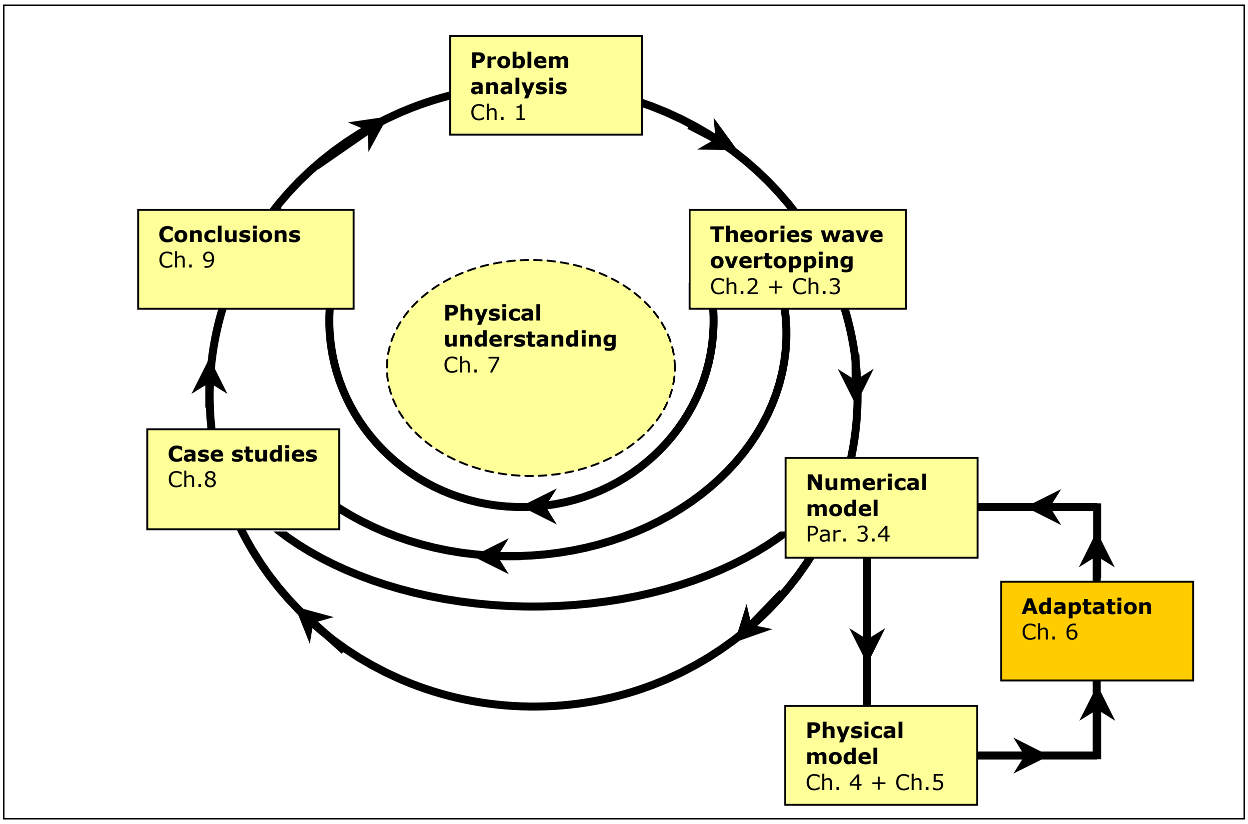
A traditional dike (without a crest structure) is compared with a Crest Drainage Dike. Since there is no significant difference in the total overtopping discharges it can be concluded that there is no reflection in the crest basin. This, however, is only known for crest basins with this specific layout.

The drain capacity shows a significant difference in the crest basin efficiency. The larger the drain diameter, the more water it can drain. However, when the actual drain discharge is made dimensionless with the maximum drain capacity, it turns out that the efficiency of larger drains is lower. This can be explained by the fact that the crest basin volume (the buffer capacity) is relatively smaller when using a larger drain.

The numerical model predicts the actual overtopping rates very accurately when using a drain diameter of 1 cm. However, for drains with a diameter of 2cm or 3cm, the numerical model under predicts the crest basin inefficiency. Therefore a closer analysis of the numerical model is necessary and will be the subject of the next chapter.

It has been shown that, for relatively large drain capacities, an error occurs in the numerical model. Since there is no insight in the cause of this error, there is a need to identify this error and eventually adapt the model. Therefore it is needed to have a feedback on the model. This feedback is the subject of the following chapter.

6 Feedback on the model based on the physical experiments.



6.1 Introduction

In section 5.4 (page 80) it is shown that the numerical model under predicts the actual crest basin efficiency when using larger drains. This chapter gives a closer analysis on the numerical model and will investigate the error in the numerical model. The test results will be used for this.

A closer analysis of the problem is given in section 6.2. In section 6.3, an analysis of the influence of the error regarding the engineering field is given. Conclusions are given in section 6.4.

6.2 Analysis of the numerical program with respect to errors

In section 5.4, it is described that the predicted crest basin efficiency is lower than the actual crest basin efficiency. Since the total wave overtopping discharge ($q_{totalovertopping}$) is always very accurately predicted (see figure 5-47 on page 81). Therefore, the only possible reason for the error is that the predicted drain capacity is lower than the actual drain capacity. Regarding draining, two phenomena regarding an error might occur in the numerical program:

- Drain error A: The crest basin is empty. Water is approaching and, according to the numerical program, directly overtopping (partially) while this is not happening in reality.

- Drain error B: The drain capacity in the numerical model is different from the actual drain capacity.

The error in the numerical program could only be the result of an error in these two phenomena. This error is only significant for larger overtopping rates. Therefore a closer look to these phenomena will be executed. Use will be made of the numerical program. With this program a test where the prediction was significant different than the actual rate is used. The physical test with the largest predicted error observed is test C39. This is marked in figure 5-47 on page 81. The parameters of this test are used as variables in the run which is shown in figure 6-1.

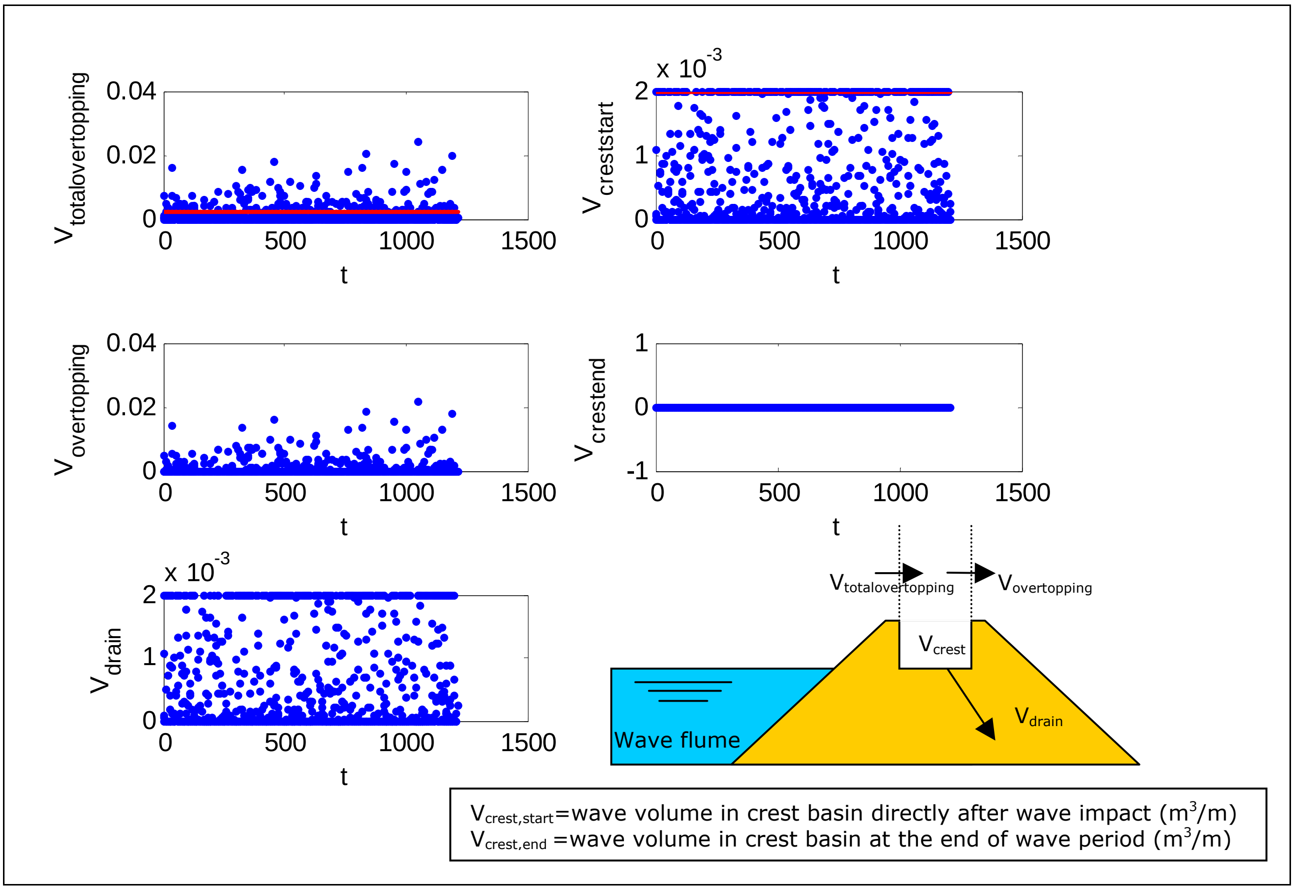


figure 6-1: numerical process of test C38

In figure 6-1, several aspects of the physical process are shown. Every blue dot represents a volume that belongs to one individual wave. For a proper explanation of the physical aspects of figure 6-1, reference has been made to figure 3-21 (page 34). In the figure on top left, the incoming volume per wave is displayed as function of the time. The red line indicates the buffer capacity. It can be seen that several waves are larger than the crest basin and will therefore directly overtop. The volume of water in the crest basin, directly after the wave came in the crest basin, is showed in the figure on top right. By definition, this volume is never larger than the crest basin volume. The waves that are overtopping the crest basin are showed in the figure (middle, left). This figure is identical to the part above the red line in the figure on top left. This should only be identical if an incoming wave arrives at an empty crest. It can be seen in the figure (middle, right) that the crest basin is always empty at "the end" of a wave period. The figure (bottom, left) shows the drain capacity. It can be seen that the drain can empty a filled crest basin in one wave period since the drain capacity is quite often equal to the crest basin volume of a filled crest (2.10^{-3} m^3/m per wave)

As can be seen in figure 6-1 (middle, right) the draining discharge is large enough to empty the crest basin in only one single wave. Therefore the crest "at the end" of a wave is always empty. Therefore it can be concluded that drain error B (Drain error B is described on page 86) in the numerical model does not exist.

The non-existence of drain error B implies that the error is caused by drain error A. To identify the characteristics of this error, a thorough analysis is required.

In the model, it is assumed that the incoming wave volume is coming in during an infinitesimally small duration. After determining whether the wave is overtopping or not, the drain starts to drain. (This schematisation is shown in figure 3-21 on page 34). However, in a real situation the "wave incoming time" is not infinitesimally small but a certain fraction of the wave period. For illustration purposes the wave incoming time is assumed to be a quarter of the wave period.

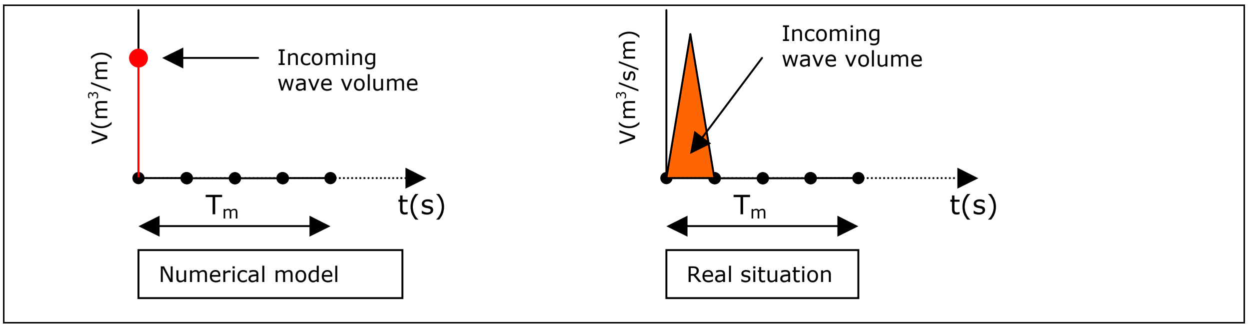


figure 6-2: incoming time in the numerical model and in reality.

Suppose the drain capacity is designed in a way where a filled crest basin is emptied during one wave period. (Which actually is the case in test C38 as can be seen in figure 6-1 (middle, right). In this case the wave is also draining during the "wave incoming time. In case of test C38, this means that 25% of a crest basin volume is already drained at the moment that the wave stops to overtop. This implies that the buffer capacity is larger due to the drained quantities during the wave impact. An illustration is given in figure 6-3.

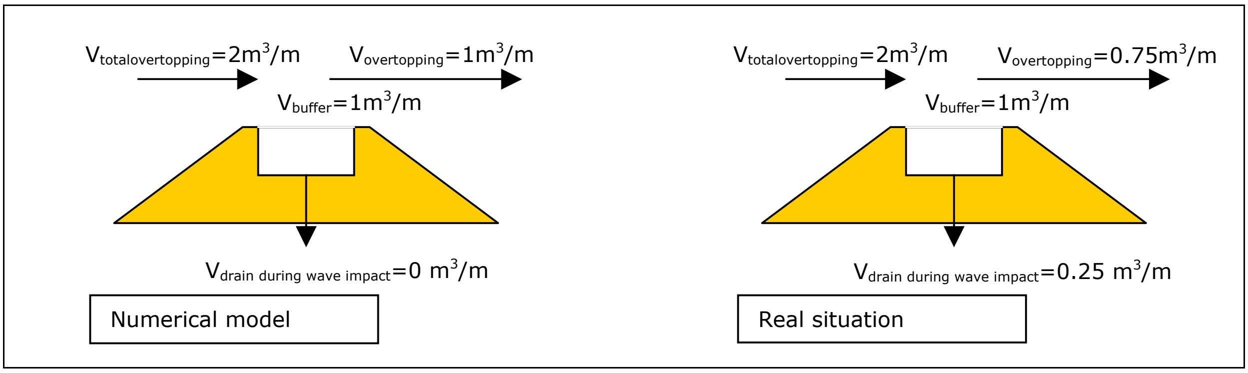


figure 6-3: Comparison numerical model and real situation for larger drain capacities.

In the example given in figure 6-3, a wave with a volume of 2m^3/m is overtopping. The buffer capacity is 1m^3/m. According to the numerical model, 1m^3/m is buffered in the crest basin and 1m^3/m is overtopping the Crest Drainage Dike. However, in reality, the drain is draining during the impact time of the wave (1/4 of the wave period is assumed). Suppose that a relatively large drain is used. The capacity of this drain is 1m^3/m per wave period. This implies that, during the wave impact, 0.25·1m^3/m is 0.25m^3/m is drained and that 0.75m^3/m is overtopping.

The influence of the error is the amount of water that is drained during the "wave incoming time" assuming that the drain is draining with its maximum capacity. This amount is, in case of test C38, the drain capacity times the wave incoming time. Since the drain capacity has a magnitude of around 1 crest basin per wave, the error is ¼ crest basin volume per wave. The error should be in the order of 25 percent per wave. It is emphasized that this is only valid for waves that have an incoming volume of at least 1.25 times the crest basin volume.

It can be concluded that the error is significant when the drain capacity is large compared with the crest basin volume. To give an indication, the dimensionless parameter 'n', which represents the necessary draining time of a filled crest basin, expressed in a number of waves, is applied. This parameter is described in section 3.6. (page 37). The values of this parameter 'n' corresponding to the different drains used in the physical model are shown in table 3-1.

Drain diameter D_{drain}	**Dimensionless discharge n**	**Error**
(m)	(-)	
0.01m	8	Small
0.02m	2	Large
0.03m	0.9	Large

table 6-1: Dimensionless discharge for the used drains

With a relatively larger drain capacity, the error of the numerical model becomes larger. This uncertainty is illustrated in figure 6-4. In figure 6-4 it can clearly be seen that the error in the numerical model becomes larger with a larger dimensionless drain capacity n.

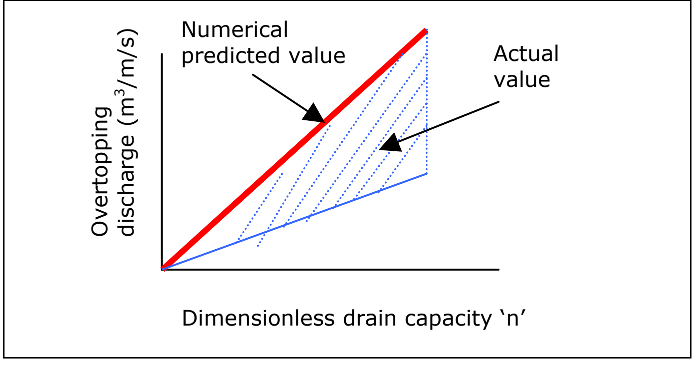


figure 6-4: Error of the numerical model as function of the relative drain capacity

Adapting the numerical program can solve this problem. Before an eventual adaptation will be executed, the influence of the error for engineering purposes will be examined in the following section.

6.3 Influence of the error in the engineering field.

To examine the error described in the section 6.2, a comparison with practical situations will be used. This is only done in a rough way to estimate the order of magnitude of the dimensionless drain discharge n (see section 3.6 for an explanation of the dimensionless parameter 'n'). The crest lay out as given in [DHV, 2005] will be used as a reference. In this study, the following dimensions are suggested:

The crest basin volume: 1.3m^3/m

Pipe diameter: 0.40m
Distance pipes 30-35m
Discharge pipe 0.43-0.61 m^3/s (depending on the water height in the crest construction)

The following values will be used for the indication:

Pipe diameter 35m
Discharge pipe 0.5 m^3/s

The value of the wave period depends on the local situation. As a first guess a wave period (at the toe of the dike) of 12 seconds is assumed.

With the use of Equation 3-22 (page 38) the following dimensionless drain parameter 'n' is obtained:

$$n = \frac{V_{crest} \cdot \mathrm{Dist}_{\mathrm{drains}}}{\mathrm{Q}_{\mathrm{drain}} T} = \frac{1.3 \, {}^{m^3}\!/_{m} \cdot 35m}{0.50 \, {}^{m^3}\!/_{s} \cdot 12s} = 7.5$$

Applying a wave period of 6 seconds, the dimensionless parameter n=15.

These examples show that for situations as described in [DHV, 2005] the model can be used, when accepting a moderate error. (see section 6.2). Even if a smaller dimensionless discharge is obtained, the model can be used when the crest basin efficiency is high. (>80%) see figure 5-47. However, when using the numerical model, it is always necessary to check this dimensionless parameter.

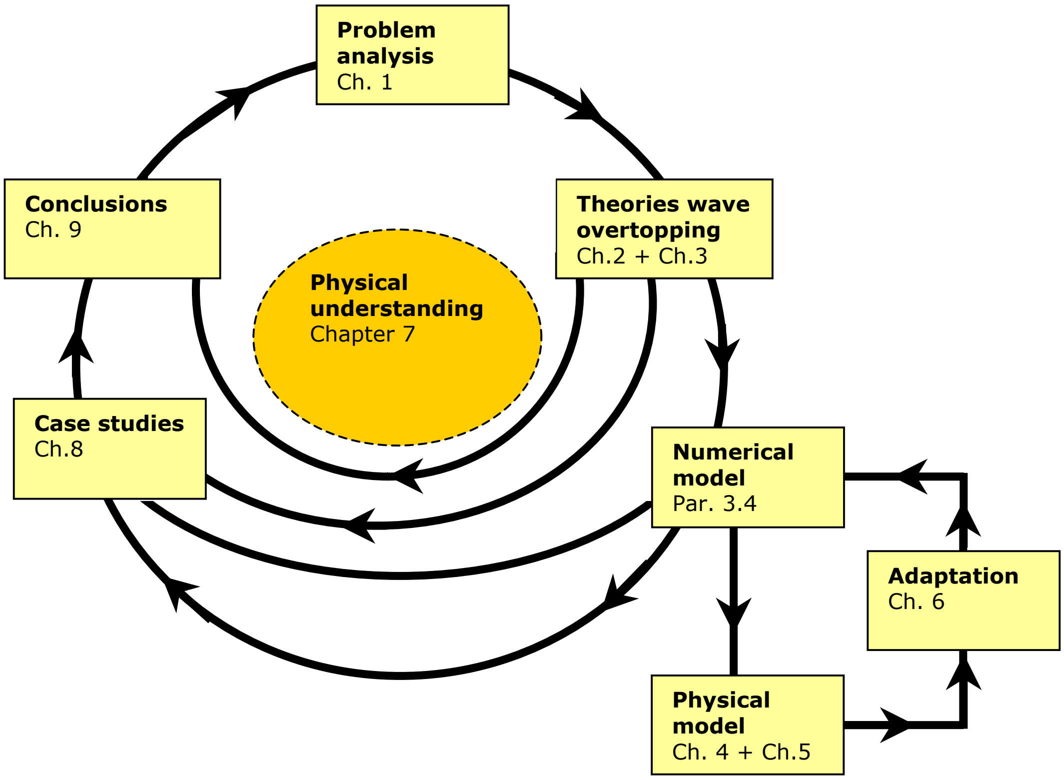
6.4 Conclusion

The error in the numerical model is identified. The reason that there is an error in the numerical model is due to the assumption that the "wave overtopping time" is infinitesimally small. This means that according to the model no draining could take place during the wave overtopping time and that the total overtopping will be larger. For smaller dimensionless drain discharges this error is negligible. Since the practical application area of the model is usually with relatively small dimensionless drain discharges, the model will not be adapted. However, when using the model, one should always check whether the model could be used or not.

6.5 Epilogue

Based on the physical model tests, the numerical model predicts overtopping rates which are within an acceptable error. However, there is still no insight in the influence of several parameters since there is a lack of numerical experiments. Therefore, some experience with the numerical program has been gathered. This is described in the following chapter.

7 Reflection on the theories based on numerical experience



7.1 Introduction

To obtain a better insight in the physics of the Crest Drainage Dike, this chapter presents two examples with fictive dikes. In these examples the focus is not on quantification of overtopping discharges but only on the physical processes. The description of the two examples is given in section 7.2. An analysis of the statistical uncertainty is given in section 7.3. Section 7.4 describes the wave overtopping processes for both dikes. A feedback on earlier described theories is given in section 7.5.

7.2 Two examples: the fictive Schrobbelse Sea Defence and Knaspelpolder Sea Defence

To illustrate the wave overtopping processes of the Crest Drainage Dike, use is made of two fictive dikes with a simple geometry. The hydraulic and geometric boundary conditions for both dikes are equal except for the significant wave height and the crest freeboard. The significant wave height for the Knaspelpolder Sea Defence is 1m, the significant wave height for the Schrobbelse Sea Defence is 1m. The hydraulic and geometric parameters are shown in table 7-1 and figure 7-1.

Slope	α	1:4	
Water level	SWL	0	(m+ reference level)
Wave steepness	s_0	5	(%)
Reduction factors	γ_β, γ_b γ_f γ_v	1	-
$T_{/m-1.0}$ / T_m		1	-

table 7-1: The hydraulic and geometric parameters of the fictive Schrobbelse Sea Defence and Knaspelpolder Sea Defence.

All the influence factors such as berms, angle of attack, friction etcetera are negligible and are therefore set to the value of 1. There's no influence of a shallow foreshore. It is assumed that the $T_{m-1.0}$ equals the T_m (This is usually not the case but this case study is only executed for illustration purposes).

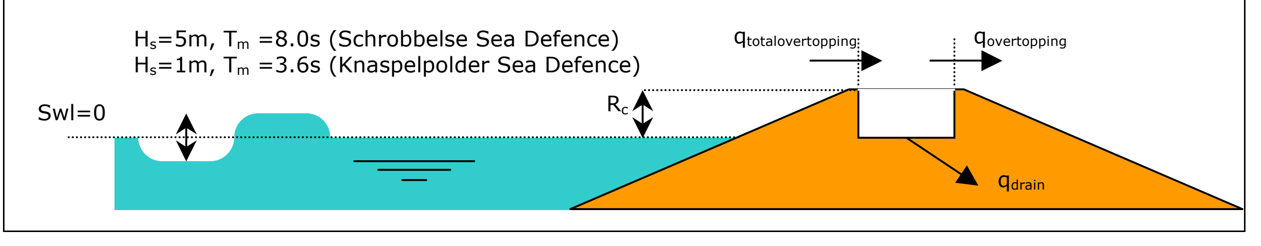


figure 7-1: The dimensions of the fictive Schrobbelse Sea defence

The layout of the applied crest basin and drains are equal in both situations. The dimensions of the discharge pipes and the crest basin are based on the feasibility study [DHV, 2005] and have the following characteristics:

$q_{drainmax}$ = 16 l/s/m
buffer capacity = 1.6m^3/m

In section 6.3 (page 89), it is explained that the numerical model gives a significant error for situations where the dimensionless drain parameter "n" has a certain value. This parameter is described in section 3.6. With the use of Equation 3-22 on page 38, n is determined and has a value of 12.5 for the Schrobbelse Sea Defence and a value of 28 for the Knaspelpolder Sea defence. This implies that the error in the numerical prediction is very small and that the numerical model can be used for these examples.

7.3 Statistical uncertainty in determining the wave overtopping discharge

7.3.1 Introduction

Since the numerical program is creating wave volumes randomly (This is explained in section 3.5), there is an uncertainty in the predicted reduction of wave overtopping and the predicted average total overtopping discharge ($q_{totalovertopping}$). An illustration of the random process is given in figure 7-2. In this simulation 5000 waves are generated.

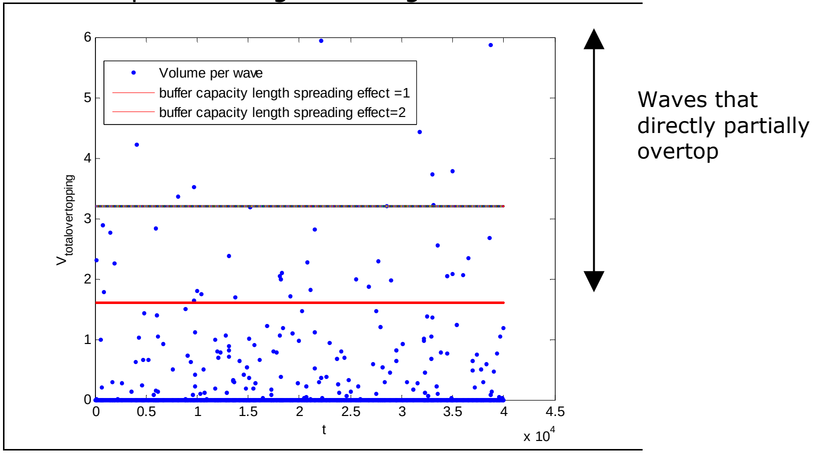


figure 7-2: The wave overtopping pattern of the fictive Schrobbelse Sea Defence.

In figure 7-2, it can be seen that during 5000 waves, only a couple of waves overtop the crest basin directly. (The red line indicates the volume of the crest basin, the black line indicates the volume of the crest basin assuming a length spreading effect of 2). Therefore there is a large statistical uncertainty. In theory, this problem could be solved by lengthening the wave record to infinite lengths, but this makes no sense since the duration of a storm during design conditions is only a couple of hours.

7.3.2 Analysis of the statistical uncertainty

The analysis of the statistical uncertainty of the wave overtopping discharges is executed in two steps:

- Statistical uncertainty of the total overtopping discharge ($q_{totalovertopping}$)
- Statistical uncertainty of the reduction in overtopping discharge

The above-described example is tested for several crest freeboards (with its corresponding wave overtopping discharge) for the two fictive dikes. Every test is repeated 1000 times (with a 3 hours storm per test). This amount of data gives the opportunity to execute a statistical analysis on the data and obtain the accuracy of the predictions. To give an indication of the statistical analysis, the results of the Schrobbelse Sea Defence are shown in figure 7-3 to figure 7-6.

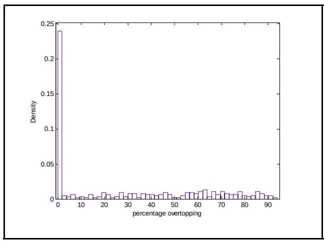


figure 7-3: Density plot for the percentage of overtopping for the Schrobbelse Sea Defence. (q=0.1 l/s/m).

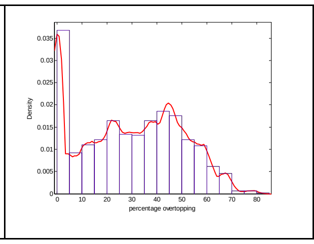


figure 7-4: Density plot for the percentage of overtopping for the Schrobbelse Sea Defence. (q=1 l/s/m)

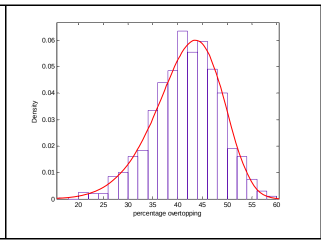


figure 7-5: Density plot for the percentage of overtopping for the Schrobbelse Sea Defence. (q=10 l/s/m)

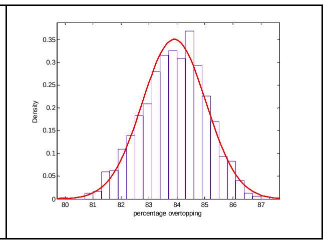


figure 7-6: Density plot for the percentage of overtopping for the Schrobbelse Sea Defence. (q=100 l/s/m)

Statistical uncertainty of the total overtopping discharge ($q_{totalovertopping}$)

The numerical program uses the average total overtopping discharge ($q_{totalovertopping}$) as an input variable. However, this does not imply that the numerical calculated average total overtopping discharge is the same. (due to the random character of the wave volume generation). Therefore, both fictive dikes have been analysed regarding this average total overtopping discharge. Every test is based on 1000 runs. With the obtained numerical data, figure 7-7 (Schrobbelse Sea Defence and figure 7-8 (Knaspelpolder Sea Defence) are constructed.

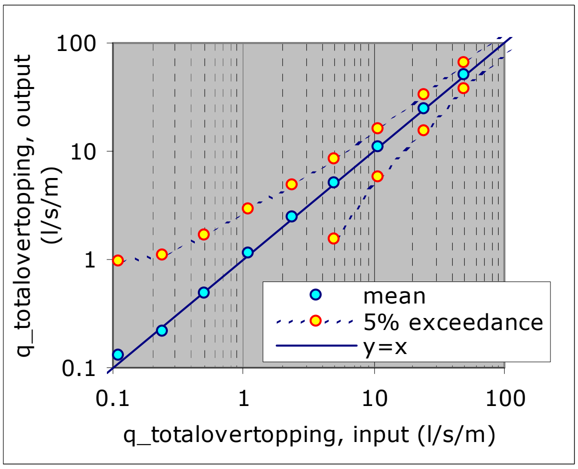


figure 7-7: Statistical uncertainty $q_{totalovertopping}$, (Schrobbelse Sea defence).

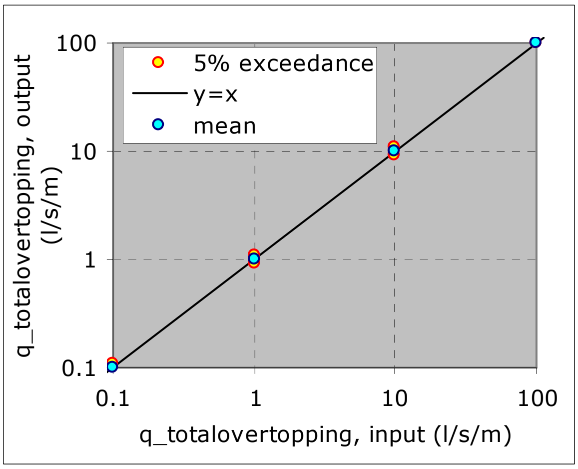


figure 7-8: Statistical uncertainty $q_{totalovertopping}$, (Knaspelpolder Sea defence).

In figure 7-7, it can be seen that the actual total overtopping discharge has a relative high uncertainty at the Schrobbelse Sea Defence. The relative uncertainty becomes larger for smaller overtopping discharges. The mean predicted average total overtopping quantity equals the input parameter $q_{totalovertopping,input}$.

In figure 7-8, it can be seen that the statistical uncertainty is relatively small for the Knaspelpolder Sea Defence. The total number of waves that are overtopping explains the difference in uncertainty. During a 3 hour storm, 1350 waves are generated at the Schrobbelse Sea Defence. At the Knaspelpolder Sea Defence, 3000 waves are generated. However, the interest is not the number of generated waves but in the number of waves that are overtopping. This can be calculated using Equation 2-16 (page 12). The number of overtopping waves at the Knaspelpolder Sea Defence during a 3 hour storm is 1060. The number of waves overtopping the Schrobbelse Sea Defence during a 3 hour storm is 90. The uncertainty becomes lower for a larger number of waves during a time record.

Statistical uncertainty of the reduction in overtopping discharge
The analysis described in the previous section is repeated for the percentage of overtopping.
Results of the statistical analysis are shown in figure 7-9 and figure 7-10

As can be seen in figure 7-9 and figure 7-10, the 5% exceedance line is not drawn for smaller overtopping rates. (0.1 l/s/m) This is due to the fact that no distribution function was found to describe the outcomes. (This is due to the fact that in many simulations the total overtopping rates are equal to zero). It would take a thorough statistical analysis to determine the 5% exceedance limit for these low overtopping rates. This has not been done in this report since the focus is not on statistical quantification but on the understanding of the physics.

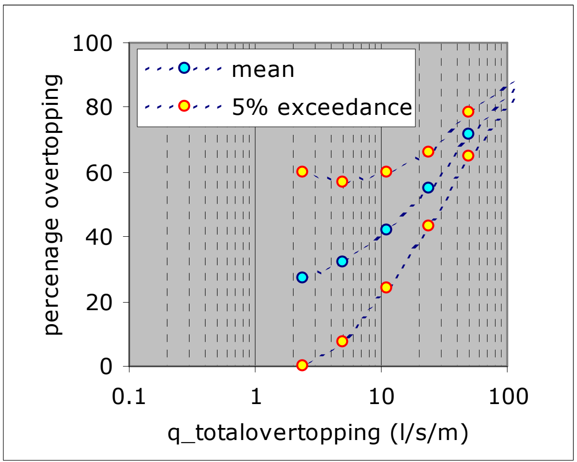


figure 7-9: Statistical uncertainty percentage overtopping (Schrobbelse Sea defence).

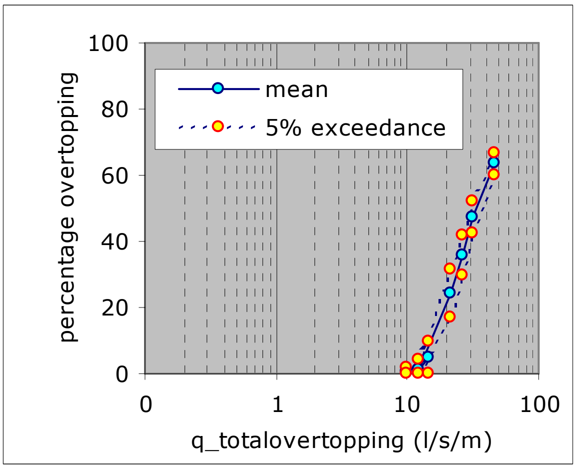


figure 7-10: Statistical uncertainty percentage overtopping (Knaspelpolder Sea defence).

It can be seen that the uncertainty is relatively high for the Schrobbelse Sea Defence. The uncertainty is relatively low for the Knaspelpolder Sea Defence. The explanation for the difference is due to the difference in the number of overtopping waves. (1060 waves are overtopping waves at the Knaspelpolder Sea Defence and 90 waves are overtopping the Schrobbelse Sea defence).

7.4 Physical difference between the two fictive dikes

The two fictive case studies shown in section 7.3 a significant difference in efficiency under the same hydraulic load. (Expressed in a average total overtopping discharge). A thorough analysis is given with the use of figure 7-11 and figure 7-12.

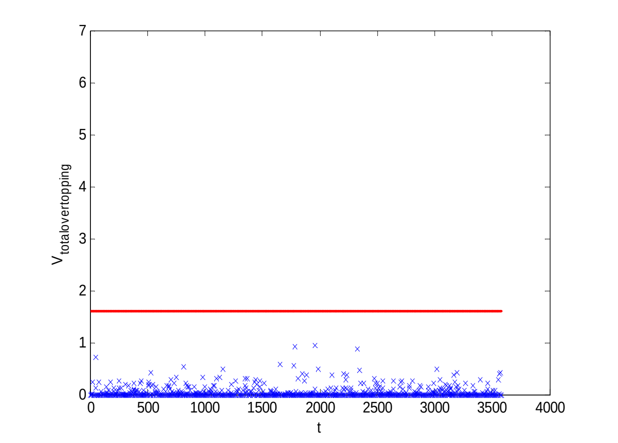


figure 7-11:Wave volume pattern Knaspelpolder Sea Defence. $q_{totalovertopping}$ =10 l/s/m

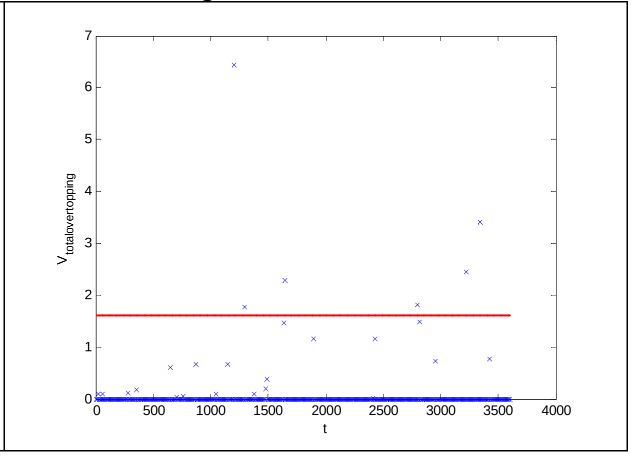


figure 7-12: Wave volume pattern Schrobbelse Sea Defence. $q_{totalovertopping}$ =10 l/s/m

In figure 7-11, the wave-overtopping pattern of the Knaspelpolder is shown as a function of the time. The y-axis represents the volume per wave. (m^3/m). The red line in the figure represents the volume of the crest basin (assuming no length spreading effect). In figure 7-12 the wave pattern of the Schrobbelse Sea Defence is shown. In these two figures the difference between the physics of the two given examples is shown. It can be seen that for the Schrobbelse Sea Defence (figure 7-12) less waves are overtopping but the waves that are overtopping have a relatively large volume. At the Knaspelpolder, much more waves are overtopping but the wave overtopping discharge per wave is lower. In the run that is showed, all the overtopping waves have a smaller volume than the crest basin volume and therefore the crest basin efficiency is much higher.

Taking the above described in mind, the physical explanation for the difference between the two given examples is found. However, there is a need to find a parameter that describes the physical difference as shown in figure 7-11 and figure 7-12. To find this parameter, a closer look will be taken at the theory about overtopping volumes per wave. This theory is already been discussed in section 2.5 (page 11). Since this theory is important regarding the physical behavior of the Crest Drainage Dike, a feedback on the theory is given below. The wave overtopping volumes are Weibull distributed. This probability distribution function is given by

$$P_v = P(\underline{V} \le V) = 1 - e^{\left[-\left(\frac{V}{a}\right)^{0.75}\right]} \qquad \textbf{Equation 7-1}$$

where

P_v	=	probability that wave overtopping per wave V is greater than or same as $\underline{V}$	(-)
V	=	wave overtopping volume per wave	(m^3/m)
a	=	scale factor of a Weibull distribution	(m^3/m)

$$a = 0.84 \cdot T_m \cdot \frac{q_{totalovertopping}}{P_{ov}} \qquad \textbf{Equation 7-2}$$

This scale factor ‘a’ is the only parameter that can be changed and influence the distribution of the waves. The scale factor depends on the mean wave period, the average wave overtopping discharge and the probability on overtopping per wave. The probability on overtopping per wave is defined in equation Equation 2-16 on page 12 as

$$P_{ov} = e^{\left[-\left(\sqrt{-\ln 0.02}\frac{R_c}{z_{2\%}}\right)^2\right]}$$ **Equation 7-3**

The scale factor and the probability on overtopping are calculated for the 2 fictive case studies. The results are shown in table 7-2.

	Crest heigth	2% wave run-up height	Probability on overtopping	Average Wave period	Average overtopping discharge	Scale parameter
	R_c (m)	$z_{2\%}$(m)	P_{ov} (-)	T_m (s)	q (l/s/m)	a (m^3/m)
Schrobbelse Sea defence	8.14	9.78	0.067	8.0	10	1.0
Knaspelpolder Sea Defence	1.01	1.96	0.353	3.6	10	0.085

table 7-2: Difference in parameters for the Schrobbelse Sea Defence and the Knaspelpolder Sea Defence.

In table 7-2 it can be seen that there is a significant difference in the scale factor parameter (more than a factor ten). It can also be seen that 35% of the waves at the Knaspelpolder overtop a traditional dike. For the Schrobbelse Sea defence this is only 7 percent. Since the total amount of overtopping is in both situations the same (10 l/s/m) this has a significant influence on the overtopping volumes per wave. A clear indication of this influence is given in figure 7-13.

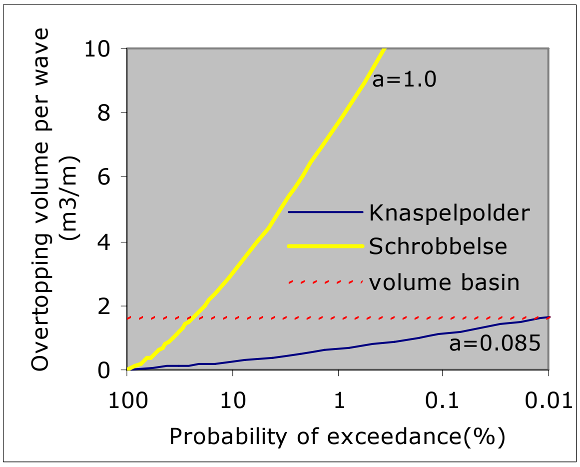


figure 7-13: Probability distribution function of the overtopping wave volumes per wave for the Knaspelpolder and the Schrobbelse Sea Defence.

In figure 7-13, it is shown that the probability that the wave volume is larger than the crest basin volume is much larger for the Schrobbelse Sea Defence (25% per overtopping wave) than for the Knaspelpolder Sea Defence (0.01% per overtopping wave). This physical difference is not identified in the theories described in chapter 2. Therefore, a feedback on the theories described in chapter is given in the following section.

7.5 Feedback on the theory based on the numerical experience

7.5.1 Introduction

In section 7.4 a physical aspect has been identified with the use of numerical experiments. This section discusses this physical aspect and will combine this with a theory that is based on the dimensionless parameter analysis given in section 3.6 (page 37).

7.5.2 Dimensionless buffer capacity

The dimensionless buffer parameter is defined in section 3.6 (page 37) as:

$$V^*_{buffer} = \frac{V_{buffer}}{V_{wave}} = \frac{V_{buffer}}{H \cdot L} \qquad \textbf{Equation 7-4}$$

In section 3.6, it is stated that this parameter has a weak physical basis and that a closer analysis is required. This will be done based on the two examples given in section 7.3.

In Equation 7-4, no clear definition of H and L is given. With the experience obtained from physical and numerical experiments, it is clear that this dimensionless parameter is incorrect since the amount of water, which comes in the crest basin is also dependent on other parameters such as the freeboard, the use of berm etc. Using the solitary overtopping theories, it turns out that the Weibull scale parameter 'a' determines the scale of the overtopping discharge per wave). For an explanation of this scale parameter one is referred to section 2.5 (page 11) and section 7.4 (page 98). Therefore, the dimensionless buffer parameter is redefined as:

$$V^*_{buffer} \square \frac{V_{wave}}{V_{buffer}} \square \frac{a}{V_{buffer}} = \frac{T_m \dfrac{q_{totalovertopping}}{P_{ov}}}{V_{buffer}} = \frac{q_{totalovertopping} \cdot T_m}{V_{buffer} \cdot P_{ov}} \qquad \textbf{Equation 7-5}$$

The parameter P_{ov} represents many parameters. To show the influence of these parameters, Equation 7-5 is reformulated with the use of Equation 2-1 (page 6), Equation 2-16 (page 12), Equation 2-17 (page 12), Equation 3-17 (page 28) and Equation 7-5. This gives the following equation:

$$V^*_{buffer} = \frac{q_{totalovertopping} T_m}{l.s.e \cdot V_{crestbasin}} e^{\sqrt{-\ln 0.02}\frac{R_c}{1.75\gamma_\beta \gamma_b \gamma_f \gamma_v H_{m0}} \frac{\sqrt{s_0}}{\tan\alpha}} \qquad \textbf{Equation 7-6}$$

where

$q_{totalovertopping}$	=	average total overtopping discharge per linear meter of crest	(m^3/s/m)
T_m	=	mean wave period	(s)
l.s.e	=	length spreading effect	(-)
$V_{crestbasin}$	=	volume of the crest basin	(m^3/m)
R_c	=	crest freeboard	(m)
γ_β, γ_b, γ_f, γ_v	=	influence factors for angle of attack, berms, roughness elements and vertical walls	(-)
H_{m0}	=	significant wave height	(m)
s_0	=	waves steepness	(-)
$\tan\alpha$	=	slope angle	(-)

Equation 7-6 is constructed to show that all the geometric and hydraulic boundary conditions are in the dimensionless buffer parameter. However, to obtain a better insight in the physical process, it is recommended to use Equation 7-5.

Since all the geometric and hydraulic boundary conditions are included, this dimensionless parameter is the single most important parameter in this report considering the understanding of the physical process.

However, it is stressed that the suggested dimensionless parameter does not include the drain capacities and therefore does not represent all the physical aspects of wave overtopping at a Crest Drainage Dike.

7.5.3 Combining the dimensionless buffer capacity with the dimensionless drain capacity

To combine the dimensionless buffer parameter with the dimensionless drain parameter, use will be made of figure 5-40 (page 77), which is reprinted as figure 7-14. For a sound understanding of the dotted lines (the asymptotes), and the dimensionless axes, one is referred to section 5.3.9 (page 76).

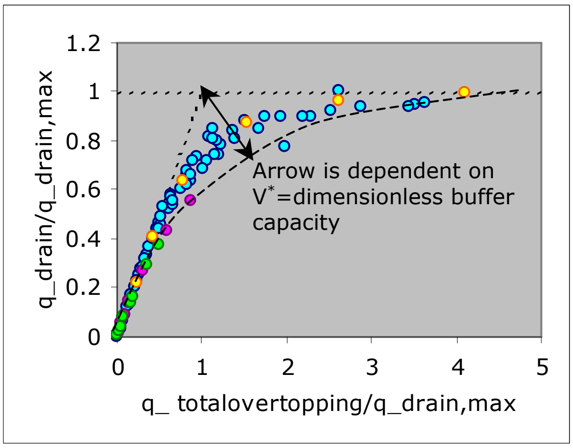


figure 7-14: Influence of the dimensionless crest basin volume

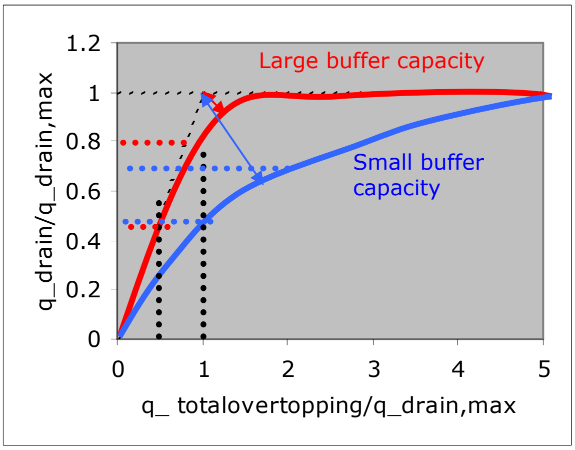


figure 7-15: Illustration of difference in dimensionless overtopping

where

q_{drain}	=	average drained discharge per linear meter of crest	(m^3/s/m)
$q_{drain,max}$	=	maximum draining discharge per linear meter of crest	(m^3/s/m)
$q_{totalovertopping}$	=	average total overtopping discharge per linear meter of crest	(m^3/s/m)
V^*_{buffer}	=	dimensionless buffer capacity	(-)

In figure 7-14, the influence of the dimensionless crest basin volume (V^*) is visualized with the arrow. Although this visualization has no consequences for the numerical model, it helps to understand the physics behind the efficiency of the Crest Drainage Dike.
A larger dimensionless crest basin parameter indicates a relatively smaller crest basin. In the figure it can be seen that an infinitesimally large basin (and thus a small dimensionless parameter) gives a maximum draining efficiency. (since the dotted lines are the limit efficiencies). This is illustrated with figure 7-15 where the red line represent a situation with a relatively large crest basin and the blue line represents a situations with a relatively small crest basin.

7.5.4 Example

To exemplify the theory described in the previous section, an example is given. Two fictive dikes will be used. Both dikes have the same geometric and hydraulic boundary conditions. The only difference is the wave pattern. The fictive wave pattern is shown in figure 7-16. The wave period in both situations is 10 seconds and the average wave overtopping discharge is 10 l/s/m. (or 100 liters per wave per meter). The "red" dike has a very regular wave pattern and every wave has almost the same wave overtopping discharge. The “blue” dike has an irregular wave pattern.

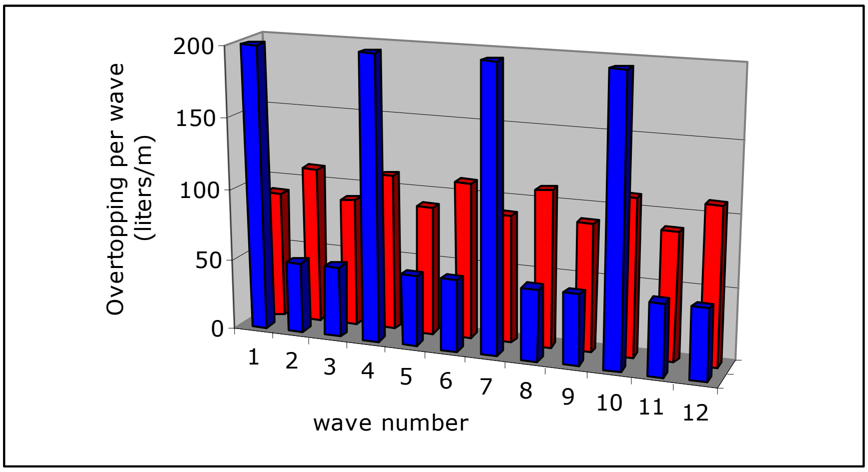


figure 7-16: wave pattern for two fictive situations

figure 7-16 is linked to figure 7-15 and can be seen as a schematization of figure 7-11 and figure 7-12 on page 98 (only schematically, not numerically). The red wave pattern in figure 7-16 is linked to the red line in figure 7-15. The blue wave pattern is linked to the blue line. In figure 7-15, the red line indicates a very efficient draining capacity since this line is close to the asymptotes. The blue line in figure 7-15 has a relatively large distance from the asymptotes and has therefore a relatively smaller efficiency. (One could say that the red line is a situation with a relatively large crest basin and the blue line is a situation with a relatively small crest basin). The parameters of this example are given in table 7-3. It is stressed that this example is only used for illustration purposes and that the used values cannot be considered as proper values.

			Red line	**Blue line**
Maximum drain capacity	$q_{drain,max}$	(l/s/m)	10	10
Total overtopping discharge	$q_{totalovertopping}$	(l/s/m)	10	10
Value x-axis in figure 7-15	$x = \frac{q_{totalovertopping}}{q_{drain,\max}}$	(-)	1	1
Value y-axis in figure 7-15	$y = \frac{q_{drain}}{q_{drain,\max}}$	(-)	0.8 (read from graph)	0.5 (read from graph)
Drain discharge	q_{drain}	(l/s/m)	8	5
Overtopping discharge	$q_{overtopping}$	(l/s/m)	2	5

table 7-3: example dimensionless parameters with $q_{drain,max}$=$q_{totalovertopping}$

In table 7-3, it can be seen that the total wave overtopping discharge ($q_{totalovertopping}$) equals the maximum drain capacity ($q_{drainmax}$). This implies that, with an infinitesimally large crest basin, all the water would be drained and no water would overtop the Crest Drainage Dike. (Indicated by the asymptotes).
The relative total overtopping discharge (the x-axis in figure 7-15) is 10/10=1. From figure 7-15, it can be seen that the corresponding y-value is 0.8 for the red line and 0.5 for the blue line. Since this y-value represents the dimensionless draining discharge, the actual drained discharge (q_{drain}) can be derived for both situations. (8 l/s/m for the red line, 5 l/s/m for the blue line)

Suppose that for both situations the maximum drain capacity is doubled. The corresponding values of both situations are shown in table 7-4.

			Red line	Blue line
Maximum drain capacity	$q_{drain,max}$	(l/s/m)	20	20
Total overtopping discharge	$q_{totalovertopping}$	(l/s/m)	10	10
Value x-axis in figure 7-15	$x = \frac{q_{totalovertopping}}{q_{drain,\max}}$	(-)	0.5	0.5
Value y-axis in figure 7-15	$y = \frac{q_{drain}}{q_{drain,\max}}$	(-)	0.45 (read from graph)	0.25 (read from graph)
Drain discharge	q_{drain}	(l/s/m)	9	5
Overtopping discharge	$q_{overtopping}$	(l/s/m)	1	5

table 7-4: example dimensionless parameters with $q_{drain,max}=2 \cdot q_{totalovertopping}$

From table 7-4, it can be concluded that increasing the drain capacity ($q_{drain,max}$) does make sense for the situation which is represented by the red line. One could say that in this situation the bottleneck is the drain capacity. Increasing this capacity gives a better result in terms of reduction of the overtopping discharge. For the situation that is represented by the blue line, the increase in drain capacity does not give better results. This can be explained by the fact that the bottleneck is the buffer capacity.

Although there is no numerical support, one could state that the red line represents the Knaspelpolder Sea Defence (see figure 7-11 on page 98) and the blue line represent the example of the Schrobbelse Sea Defence (see figure 7-12 on page 98).

The V^* for both examples described in section 7.3, is determined with the use of Equation 7-5 (page Equation 7-5)

- V^*(Knaspelpolder Sea Defence) = 0.06
- V^*(Schrobbelse Sea Defence) = 0.75

These results are well in line with the theory. The dimensionless crest basin parameter (V^*) is small (thus a relatively large crest basin) for the Knaspelpolder and is more efficient than the Schrobbelse Sea defence, which has a relatively large dimensionless crest basin parameter (and thus a smaller crest basin). However, it is stressed that the physical analysis is far from complete since:

- The influence of wave grouping is not considered in this theory. Reference is made to section 3.5.2 on page 25 for an explanation of wave grouping.
- Shooting-over is not considered in this theory. Reference has been made to figure 3-13 on page 25 for an explanation of shooting-over.
- Reflection is not considered. Reference is made to figure 3-8 on page 22 for an impression of reflection.
- The influence of the drain capacity is not represented in the dimensionless crest basin parameter. Several attempts to include the drain capacity into this parameter have been executed but this did not result into a parameter that reflects the physics in a proper way.

7.5.5 Analytical expression of the combined dimensionless parameters

Since the most interest is in the crest basin efficiency, the above stated theories are projected on the theories in section 3.2 (page 16). A relation between the effectiveness of the Crest Drainage Dike and the reduction in crest freeboard is given in figure 3-3 (page 20). An analytical relation is given in Equation 3-16 (page 18). In this paragraph, it is stated that, if the crest basin efficiency is found, the problem is solved. Therefore, a closer analysis regarding the crest basin efficiency has been carried out.

With the use of figure 7-14, the following equation is derived

$$\frac{dy}{dx} = \frac{d\left(\frac{q_{drain}}{q_{drain,\max}}\right)}{d\left(\frac{q_{totalovertopping}}{q_{drain,\max}}\right)} = \frac{dq_{drain}}{dq_{totalovertopping}} = \varphi \qquad \textbf{Equation 7-7}$$

Using Equation 7-7, it can be seen that the derivative of the relation shown in figure 7-14 equals the crest basin efficiency parameter. However, since there is a lack of an analytical expression of the relation shown in figure 7.21, it is not possible to project this on Equation 3-16 (page 18) or figure 3-3 (page 20).

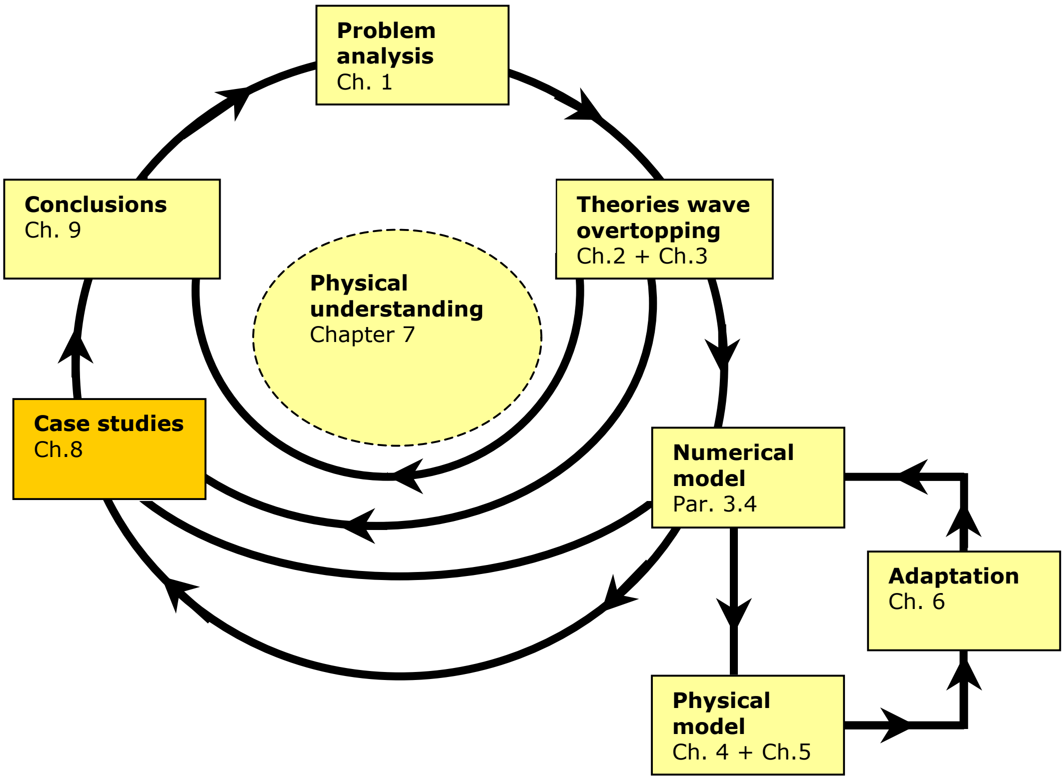
Therefore, there is a need to express the relation shown in figure 7-14 analytically. Combining this with this expression used in equation 3-3 (page 16), a full analytical description of the crest freeboard reduction could be given. A thorough analysis to identify this expression has been executed. However, the relation is unfortunately not found. Below some mathematical starting points for this expression shown in figure 7-14 are given.

- $0 \leq x$
- $0 \leq y < 1$
- Asymptote $y\left(\lim_{x \downarrow 0}\right) = x$
- Asymptote $y\left(\lim_{x \to \infty}\right) = 1$
- Derivative $0 \leq \frac{dy}{dx} \leq 1$ for all values of x>0
- Derivative $\frac{dy}{dx}\left(\lim_{x \downarrow 0}\right) = 1$
- Derivative $\frac{dy}{dx}\left(\lim_{x \to \infty}\right) = 0$
- The formula contains a scale parameter (here V^* can be used)

Many attempts to find an expression have been executed. However, no expression is found. The most expressions that were found are conflicting with the following mathematical boundary condition:

- $0 \leq \frac{dy}{dx} \leq 1$ for all values of x>0

8 Case studies



8.1 Introduction

In the previous chapters, a numerical model has been developed to calculate the efficiency of the Crest Drainage Dike. Since this model is quite theoretical and the results are not unambiguous for practical applications, two examples are given in this chapter. These examples are two fictive dikes and two real existing dikes. The goal of this chapter is to get insight into the consequences of the use of a Crest Drainage Dike. The consequences are expressed in a reduction (or avoided heightening) of the crest freeboard.

In the case studies a comparison is made between the use of a traditional dike and the use of a Crest Drainage Dike. For this purpose overtopping calculation are needed. These overtopping calculations are based on the van der Meer relations described in chapter 2. These calculations have been carried out with the use of the program PC-Overtop [van der Meer, 2002]. These calculations can only be used for the traditional dike and for the Crest Drainage Dike in case the total wave overtopping discharge ($q_{totalovertopping}$) is needed. The overtopping discharge of a Crest Drainage Dike is calculated with the use of the numerical program, which has been discussed in chapter 3. This numerical program needs input parameters such as the total wave overtopping discharge, the $z_{2\%}$ etc. The results from the program PC-Overtop will be used for this. A schematization is given in figure 8-1 and figure 8-2.

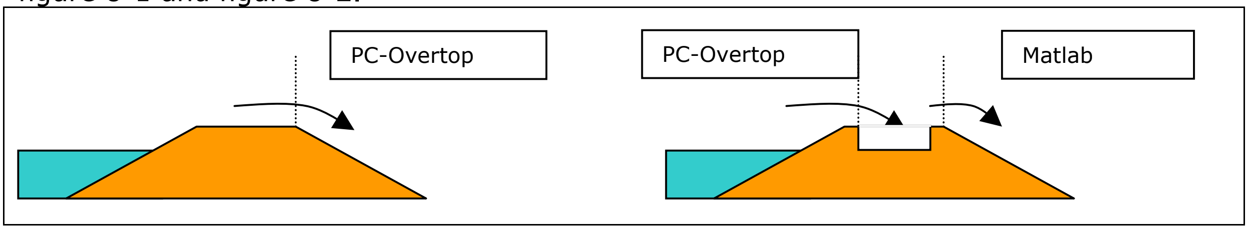


figure 8-1: Overview of methods of calculations

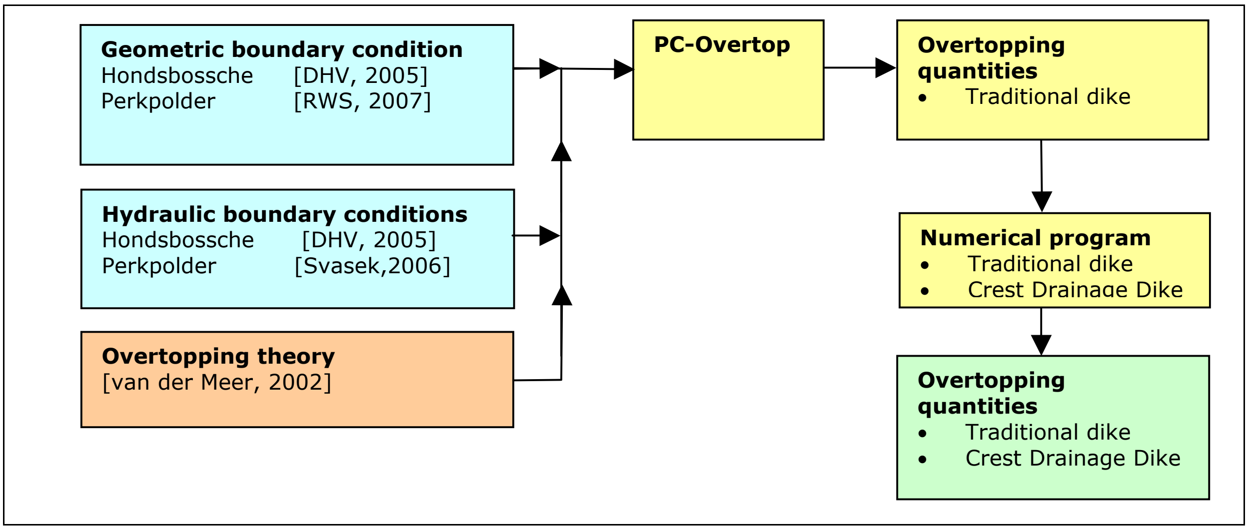


figure 8-2: Overview of the analysis process of the Hondsbossche and Perkpolder Sea Defences

The Hondsbossche Sea Defence is a sea defence in the Netherlands with a relatively severe wave attack. It is expected that the hydraulic boundary conditions become more severe in the near future and the total overtopping discharge is expected to increase.

The Perkpolder Sea Defence is a dike that does not exist yet but is planned to be built in the near future. This dike will be built in a harbour. The wave attack is relatively low. The locations of the Hondsbossche Sea Defence and the Perkpolder Sea Defence are shown in figure 8-3.

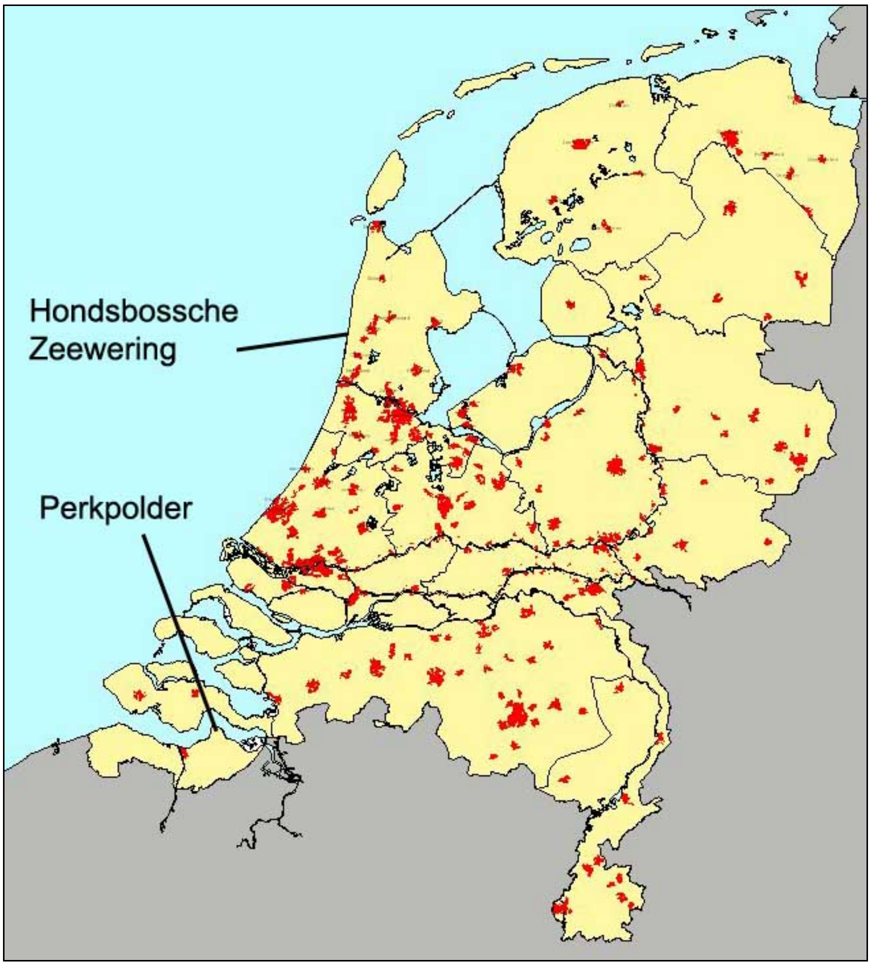


figure 8-3: Locations of the case studies

8.2 Description of the proposed Crest Drainage Dike

The design of a Crest Drainage Dike with the use of a numerical program is an iterative process. Therefore some starting assumptions regarding the layout of the crest basin and the drainage pipes are required. These starting assumptions are based on [DHV, 2005] and are shown in table 8-1. during the design process, these parameters are adapted

Crest basin width		2.0	(m)
Crest freeboard	R_c	1.0	(m)
Crest basin efficiency coefficient	θ	0.8	-
Drain diameter	D_{drain}	0.40	(m)
Distance drains	$Dist_{drains}$	35	(m)
Entrance friction coefficient	ξ	0.8	-
Length drains	L_{drain}	1	(m)

table 8-1: Dimensions of the crest basin and the drain.

Drain capacity

The drain length is 1m. This implies that the water level in the crest basin can be considered linearly dependent on the time. For an explanation of this, one is referred to appendix III (see figure A-4 on page VIII). Using the dimensions given in table 8-1, the maximum drain capacity is determined:

$$Q = A\sqrt{2g\xi(h_{\max} + L_{drain})} = 0.66 \text{m}^3/\text{s/drain}.$$

Since these drains are placed every 35m. The discharge per meter width is 19 l/s/m. The volume of the crest basin and the length spreading effect determines the buffer capacity.

Buffer capacity

The volume of the crest basin is fixed in all the scenarios. The buffer capacity is 0.8*1.0m*2.0m = 1.6 m^3/m

The length spreading effect (A description of the length spreading effect is given in section 3.4) is a variable that will be used in some of the case studies. Since the only effect of this length spreading effect is the increase of the buffer capacity, the length spreading effect can be simulated in the Matlab program by increasing the crest width in the program (see figure 3-14 on page 25 for an illustration).

8.3 Case study I: The Hondsbossche Sea Defence

8.3.1 Introduction

The Hondsbossche Sea Defence is a real existing dike in the Netherlands and has a relative severe wave attack. The sea defence has a length of 5 kilometers. A picture of the Hondsbossche Sea Defence is shown in figure 8-4. It is expected that the hydraulic boundary conditions become more severe in the near future and the total overtopping is expected to get larger. The following aspects of the Hondsbossche Sea Defence are discussed:

- Present situation — section 8.3.2
- Future situation with a Crest Drainage Dike as suggested by [DHV, 2005]. — section 8.3.3
- Future situation with a Crest Drainage Dike in combination with dike heightening. — section 8.3.4
- Future situation with a Crest Drainage Dike with adapted parameters. — section 8.3.5
- Conclusions — section 8.3.6

figure 8-4: Hondsbossche Sea Defence

8.3.2 *Present situation*

Boundary conditions and assumptions present situation
To determine the hydraulic and geometric boundary conditions, the case study that has been executed in [DHV, 2005] will be followed. The determined boundary conditions for the present situation regarding the present dike are shown in table 8-2 and figure 8-5.

H_s	$T_{m-1,0}$	T_m	β	Swl	t
[m]	[s]	[s]	[^{0}]	[m]	[s]
2.90	12.60	10.95	11	4.72	10800

table 8-2: Input parameters for the Hondsbossche Sea Defence at the present situation [DHV, 2005]

X	Y
13.87	-0.43
19.84	1
40.55	3.82
52.26	4.93
66.28	5.72
86.5	12.02

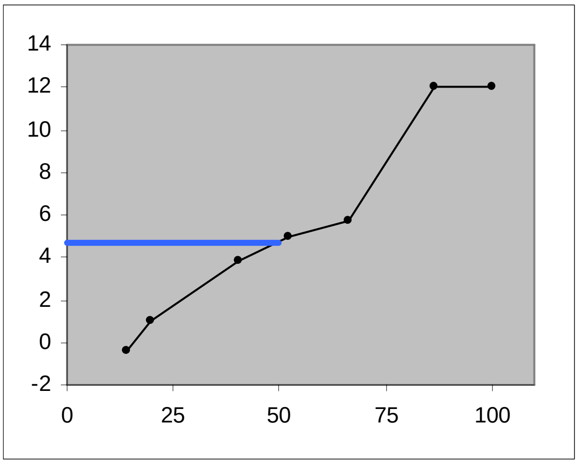


figure 8-5:A schematisation of the dimensions of the Hondsbossche Sea Defence at the present situation [DWW, 2002]

Use has been made of the program PC-Overtop [van der Meer, 2002] to calculate the wave overtopping discharge for the present situation. The results of this calculation are shown in table 8-3.

q	$Z_{2\%}$+swl
[l/s/m]	[m]
1.6	11.75

table 8-3: Wave overtopping discharge at the Hondsbossche Sea Defence for the present conditions

These results are the same as determined by [DHV, 2005]. In table 8-3, it can be seen that the average wave overtopping discharge at design conditions is 1.63 l/s/m according to the calculations of PC-Overtop. However, this program does not give a probabilistic analysis. It is possible to give a probabilistic analysis with the use of the numerical program. Therefore, this analysis is executed in the following section.

Uncertainties in the present situation
The numerical Matlab program is used in combination with PC-Overtop to check the uncertainty in the average wave overtopping discharge for a 3-hour storm (Run I) and a 1-hour (Run II) storm duration. 1000 simulations have been used to obtain data for a statistical analysis. The input parameters that are used in the numerical program are showed in table 8-4 and table 8-5.

H_{m0}	R_c	s_0	q_totaal	tan(α)	$z_{2\%}$	Influence factors
(m)	(m)	(m)	(m3/s/m)	(-)	(m)	(-)
2,90	7,30	0.012	1.6*10-3	0.168	7.03	1

table 8-4: input parameters for the numerical simulation for the Hondsbossche Sea defence, present situation.

Run	Number of waves
I	985 (3 hours)
II	328 (1 hour)

table 8-5: Overview numerical tests for the scenario with the present conditions of the Hondsbossche Sea Defence. (no use of Crest Drainage Dike).

The statistical results are shown in table 8-6, figure 8-6 and figure 8-7. The statistical analysis is executed with the use of a Weibull distribution function and an Epanechnikov distribution. No analysis regarding the best fitting distribution has been executed. The distributions have been used to determine the 5% exceedance limits. It is stressed that the statistical analysis is only used to determine a rough indication of the uncertainty and not to get a very accurate figure.

Run	Storm duration	$q_{totalovertopping}$	$q_{totalovertopping}$ 5% upper boundary (Epanechnikov)	$q_{totalovertopping}$ 5% upper boundary (Weibull)
		(l/s/m)	(l/s/m)	(l/s/m)
I	3 hours	1.6	3.0	2.9
II	1 hour	1.3	4.3	-

table 8-6: Results basic scenario Hondsbossche Sea Defence (no use of Crest Drainage Dike)

As can be seen in table 8-6, figure 8-6 and figure 8-7, there is a significant uncertainty in the total wave overtopping discharge. For a 3 hours storm duration the wave overtopping discharge is on average 1.6 l/s/m with a 5% exceedance value of 3 l/s/m. For the 1 hours storm the 5% exceedance value is even more. Therefore, it is shown that the length of the storm duration has a significant influence on the variance of the average wave overtopping discharge. This aspect is also discussed in section 7.3.2 (page 95).

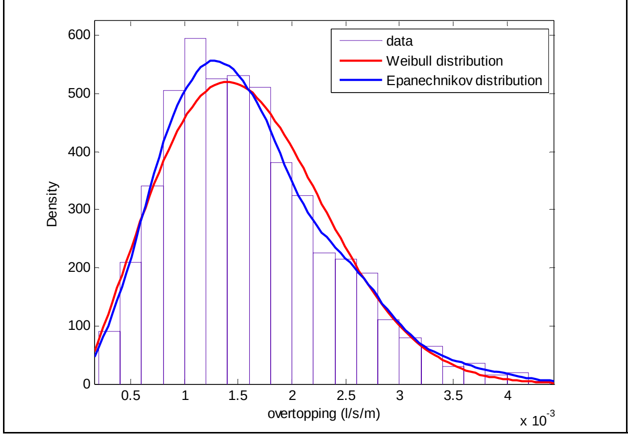


figure 8-6: Distribution of the wave overtopping discharge of the Hondsbossche Sea Defence (present situation with a storm duration of 3 hours)

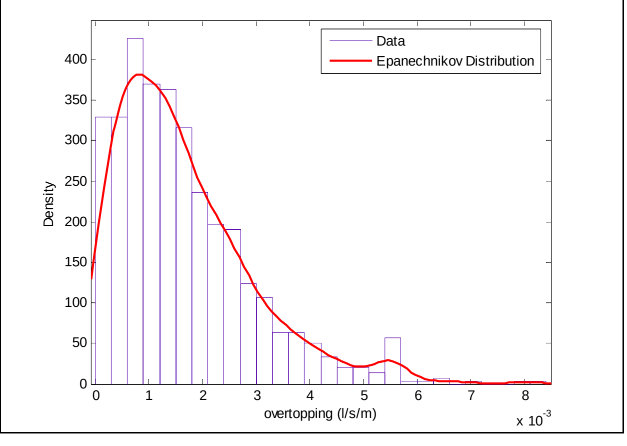


figure 8-7: Distribution of the wave overtopping discharge of the Hondsbossche Sea Defence (present situation with a storm duration of 1 hours).

8.3.3 *Future situation*

In the analysis executed by [DHV, 2005] several assumptions regarding the future conditions of the Hondsbossche Sea Defence are adopted. Boundary conditions that are use are adopted from [DWW, 2002a] and [DWW, 2002b]. The used parameters are shown in table 8-7. Influences of the angle of attack, berms, roughness etc. are not shown in this table. Their influence is determined with the use of PC-Overtop and is represented in the 2% wave run-up parameter. For a definitions of the 2% wave run-up paramter, reference is made to Equation 2-17(page 12).

H_{m0}	s_0	$q_{\_totaal}$	$z_{2\%}$	R_c	tan(a)
(m)	(-)	(m^3/s/m)	(m)	(m)	(-)
3.30	0.010	0.015	9.03	6.90	0.168

table 8-7: Hydraulic and geometric boundary conditions for the Hondsbossche Sea Defence in the future situation.

As a start, the proposed design of [DHV,2005] regarding the lay-out of the crest basin and the drains is tested with the numerical program. The layout parameters are shown in table 8-8.

Diameter drain	Distance between drains	Crest basin width	Crest basin height	Efficiency crest θ
D_{drain} (m)	$Dist_{drain}$ (m)	(m)	h_{max} (m)	(-)
0.4	35	2m	1	0.8

table 8-8: Layout crest basin and drains as suggested by [DHV,2005] for the Hondsbossche Sea Defence

Four different simulations will be used to investigate the efficiency of the Crest Drainage Dike. The variations in the four runs are the variations in the length spreading effect and in a 1 hour and a 3-hour storm. An overview is given in table 8-9.

Run	Length spreading effect	Storm duration
	l.s.e. (-)	t_{storm} (hours)
III	1	3
IV	1.5	3
V	2	3
VI	1	1

table 8-9: variable parameters future scenario for the Hondsbossche Sea Defence

The assumptions that are used are shown in table 8-10. It can be seen that the entrance friction factor has a value of 0.8. This value has the same magnitude that is derived from the calibration tests in the physical experiments (This is explained in chapter 4). The length of the drain is set to 1m. Making this drain longer might have a positive influence on the draining discharge. However, a condition for this is that the drain is totally filled. Since it is unknown whether this is true or not, this drain length is chosen. The efficiency of the crest basin is set to a value of 0.8. This assumptions is backed up by the physical experiments described in chapter 4. However, it is stressed that this only yields for the conditions as applied in these specific tests.

Entrance friction factor	Length drain	Efficiency crest basin	Reflection in the crest basin
(-)	L_{drain} (m)	(-)	
0.8	1	0.8	none

table 8-10: Assumptions for the Hondsbossche Sea Defence

The numerical program is used to simulate the four schematizations. 1000 runs per schematization have been used to investigate the statistical uncertainty of the results. The statistical analysis is not shown in this report but is equal to the earlier described statistical analyses. The results of the statistical analysis are shown in table 8-11 and figure 8-8.

Run	Total wave overtopping	5% exceedance value	Wave overtopping	5 % exceedance value
	$q_{totalovertopping}$	$q_{totalovertopping}$	$q_{overtopping}$	$q_{overtopping}$
	(l/s/m)	(l/s/m)	(l/s/m)	(l/s/m)
III	15	19	9.0	12.3
IV	15	19	7.5	11.0
V	15	19	6.6	9.8
VI	15	22	8.7	15.1

table 8-11: Results future scenario Hondsbossche Sea Defence

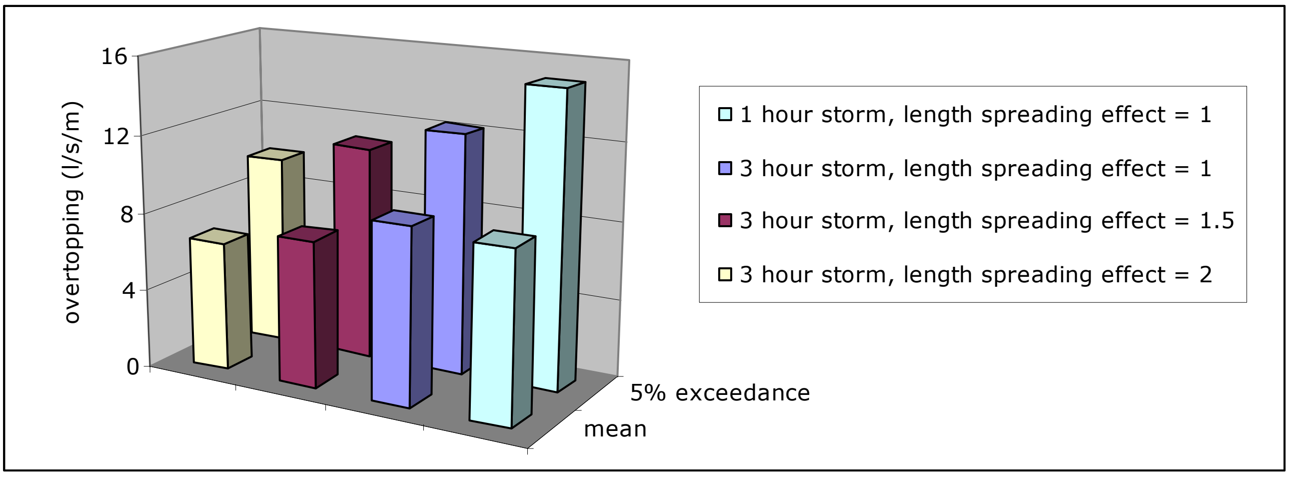


figure 8-8: The overtopping results for the Hondsbossche Sea Defence (future scenario) with the use of a Crest Drainage Dike.

In figure 8-8, the influence of the length spreading effect is very clear to see; the wave overtopping discharge is smaller with a larger length spreading effect. However, although the effect is visible, it should be noted that the influence is small. The influence of duration of the storm regarding the statistical uncertainty can be seen but is relatively small. It can be seen that the wave overtopping discharge in all the simulations do no satisfy the condition that the overtopping is 1 l/s/m.

The results shown in table 8-11 and figure 8-8 show three interesting phenomena. The most important result is that none of the simulation result into a satisfactory level of wave overtopping discharge. In the most positive case the average wave overtopping discharge is still 6.4 l/s/m. (the corresponding 5% exceedance value is 8.0 l/s/m)
The influence of the length spreading effect is clearly visible. With a larger length spreading effect, less overtopping takes place. This is in line with the theories about the buffer capacities. The influence of the storm duration is moderate and is only visible in the 5% exceedance value. A 1-hour storm gives a slightly higher 5% exceedance value than a 3 hours storm.

To get a good insight in the physical process of the Crest Drainage Dike applied to the Hondsbossche Sea Defence, the numerical process has been visualized and is shown in figure 8-9.

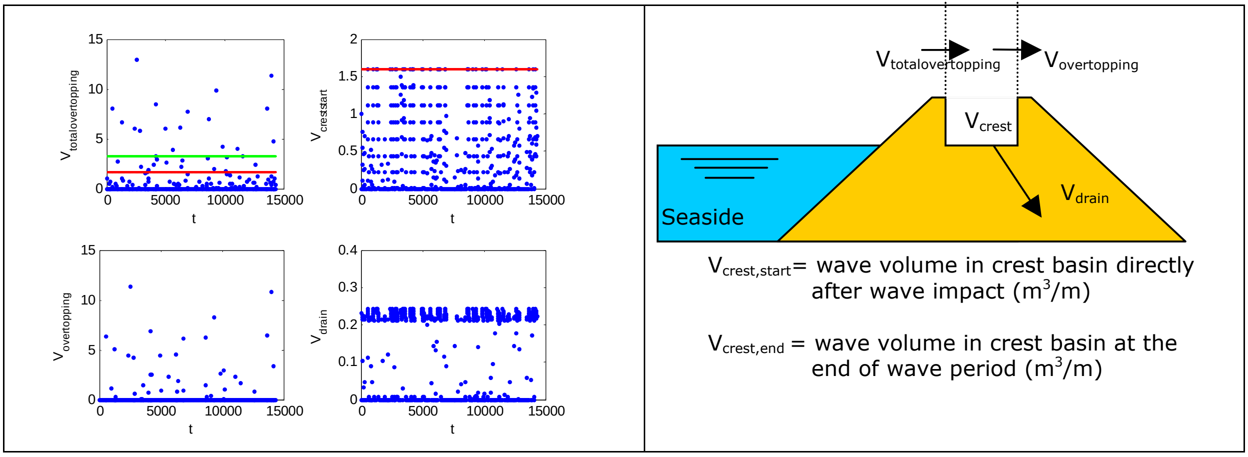


figure 8-9: Overtopping process of the future scenario Hondsbossche sea defence with the use of a Crest Drainage dike (length spreading effect is 1)

The red line indicates the maximum buffer capacity with a length-spread reduction factor of 1. The green line indicates the buffer capacity with a length-spread reduction factor of 2. The incoming wave volume is showed in the figure on top left. The actual overtopping volumes are shown in the figure (down, left). The water volume in the crest is shown in the figure (top, right). The drain capacity is showed in the figure (down, right). It can be seen in the figure (top, left) that a significant number of overtopping waves have a larger volume than the crest basin and therefore overtop the crest basin.

It can be seen in figure 8-9, that the most overtopping waves have a larger volume than the volume of the crest basin and therefore also overtop the crest basin. It is even possible to give a rough estimate of the total wave overtopping discharge with the use of this graph. Counting gives 17 waves overtopping the green line. The average is estimated on (8000-3200) =4800 liters per wave. This gives 81600 liters in 15000 seconds or an average wave overtopping discharge of 5.4 l/s/m. According to the Matlab model this is on average 6.4 l/s/m.
Only a small fraction of the waves have a volume that is in-between the volume of the crest and the volume of the crest with a length spreading factor of 2. (Between the red and the green line). This explains the small difference between the simulations with the difference length spreading factor.

Since it can be concluded that, in this case, the Crest Drainage Dike cannot be used as a substitution for traditional crest heightening, a combination of traditional crest heightening and the Crest Drainage Dike will be analyzed in the next section.

8.3.4 Traditional dike heightening in combination with the use of the Crest Drainage Dike

The necessary crest freeboard in case no Crest Drainage Dike would be used, is determined with PC-Overtop. The input parameters of table 8-7 and table 8-8 are used for this simulation. The results are shown in table 8-12.

Wave overtopping discharge	Crest freeboard	Needed heightening
(l/s/m)	(m+NAP)	(m)
0.1	17.85	5.83
1	15.12	3.10
10	12.39	0.37
100	9.66	-2.36

table 8-12: Wave overtopping discharge and corresponding crest freeboards for the Hondsbossche Sea defence in the future situation.

In table 8-12, it can be seen that, if a maximum wave overtopping discharge of 1 l/s/m is required, a dike heightening of 3.10 is necessary. Allowing a wave overtopping discharge of 10 l/s/m, only 37cm dike heightening is required. In the following analysis, it is assumed that 1 l/s/m is the criterion. Here, two combinations of dike heightening and the use of a Crest Drainage Dike are discussed.

- Run VII   1m dike heightening + Crest Drainage Dike
- Run VIII   2m dike heightening + Crest Drainage Dike

For both scenario's, the storm duration is three hours, the length spreading effect is two and the layout of the drain and the basin is the same as in the previous example. The geometric and hydraulic parameters, which are influenced by heightening the dike, are calculated with PC-Overtop and are shown in table 8-13.

Run	Heightening crest freeboard	Crest Freeboard	Wave run-up height	Average total wave overtopping discharge
	ΔR_c	Rc	$z_{2\%}$	$q_{totalovertopping}$
	(m)	(m)	(m)	(l/s/m)
VII	1	7.9	9.308	8.1
VIII	2	8.9	9.45	4.0

table 8-13: Input parameters for the Hondsbossche Sea Defence in the future situation with a combination of dike heightening and a Crest Drainage Dike

For both scenarios, the wave overtopping discharge is calculated with the use of the numerical program. The results are shown in table 8-14, figure 8-10, figure 8-11 and figure 8-12.

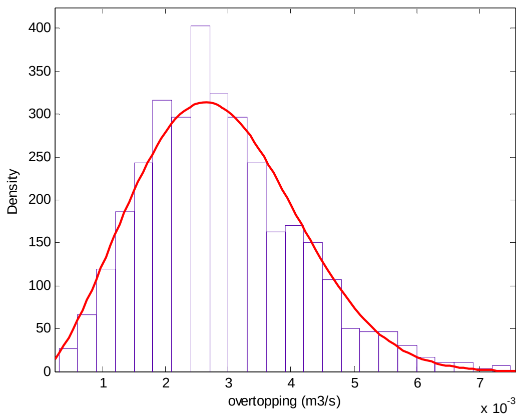


figure 8-10: Distribution for the Hondsbossche Sea Defence. (1m dike heightening + CDD)

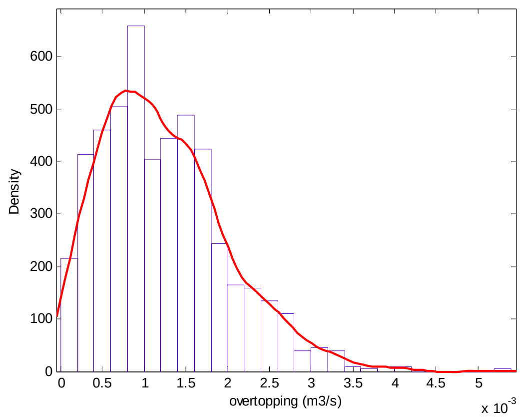


figure 8-11: Distribution for the Hondsbossche Sea Defence. (1m dike heightening + CDD)

Run	**Average wave overtopping discharge**	**5% exceedance**
	$q_{overtopping}$	$q_{overtopping}$
	(l/s/m)	(l/s/m)
VII	2.8	5.1
VIII	1.1	2.7

table 8-14: results combination dike heightening and Crest Drainage Dike

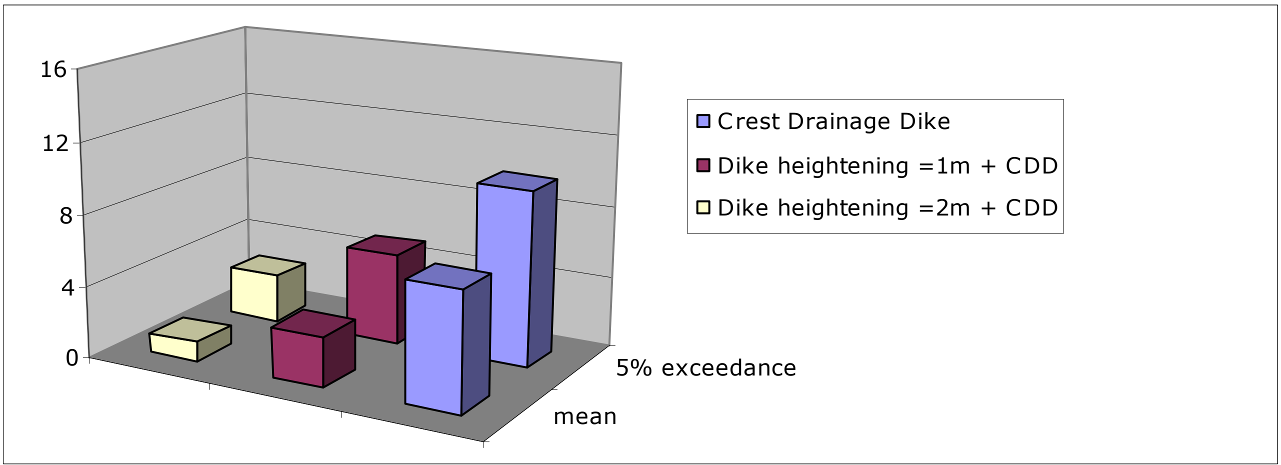


figure 8-12: Results for the Hondsbossche Sea Defence in the future situation.

In figure 8-12 and table 8-14, it can be seen that for all the situations, the wave overtopping discharge is still higher than 1 l/s/m. Therefore, this is not a proper solution and different layouts will be discussed in the following section.

8.3.5 Alternative crest drainage dike parameters

Since the earlier assumed parameters do not work in a satisfactory way, these will be adapted. Numerical experiments with larger drain diameters, smaller drain distances, and larger crest basins have been executed and are presented in this section. This analysis is carried out as a first order estimate. No statistical analysis has been carried out. To create a small standard deviation of the wave overtopping discharge, a storm duration of 24 hours is used. It is stressed that for an optimal design, a statistical analysis as shown the previous section, is required.

With the use of larger drain capacities, the numerical model is under predicting the actual drain capacities. Reference is made to the analysis of the numerical model shown in section 6.2 on page 86. Therefore, the results should only be regarded as an estimate of the actual results.

This is also true for physical processes in a larger crest basin (e.g reflection). Since the numerical model is not backed up by physical experiments regarding larger crest basins, the accuracy of the model is unknown.

In figure 8-13, the wave overtopping discharge is plotted as a function of the drain diameter (0.4m-0.8m) and the basin width (2m-5m). The basin height has a fixed value (1m) as well as the distance of the drains (35m) and the length spreading effect (2). In the figure, it can be seen that none of the suggested combinations result into a wave overtopping discharge that is less than 1 l/s/m.

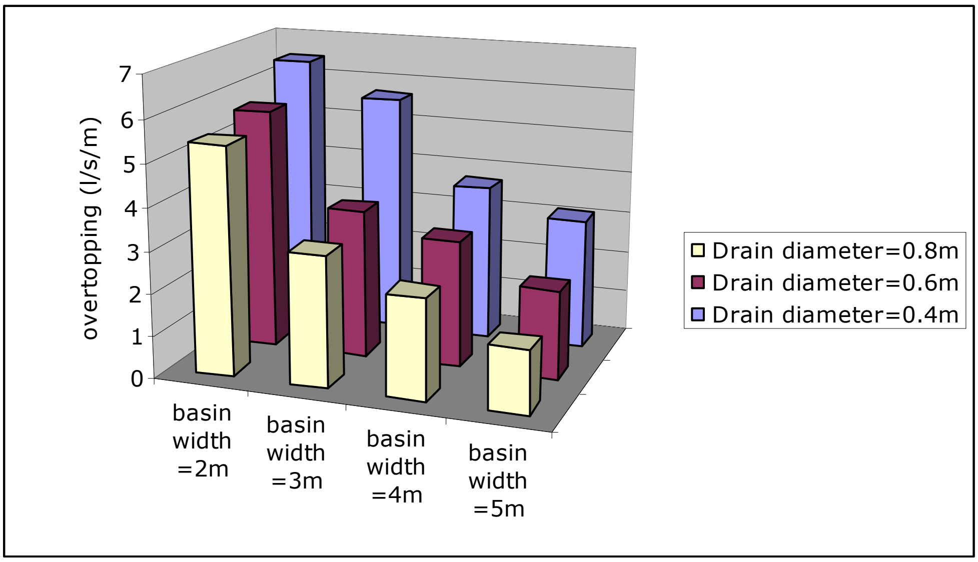


figure 8-13: Results Hondsbossche Sea Defence (Drain distance =35m, Basin height =1m)

In figure 8-14, the wave overtopping discharge is plotted as a function of the drain distance (5m-35m) and the width of the basin (2m-5m). The basin height is a fixed parameter (1m) as well as the drain diameter (0.8m) and the length spreading effect (2). It can be seen that some combinations result into a situation where an overtopping discharge is obtained that is less than 1 l/s/m.

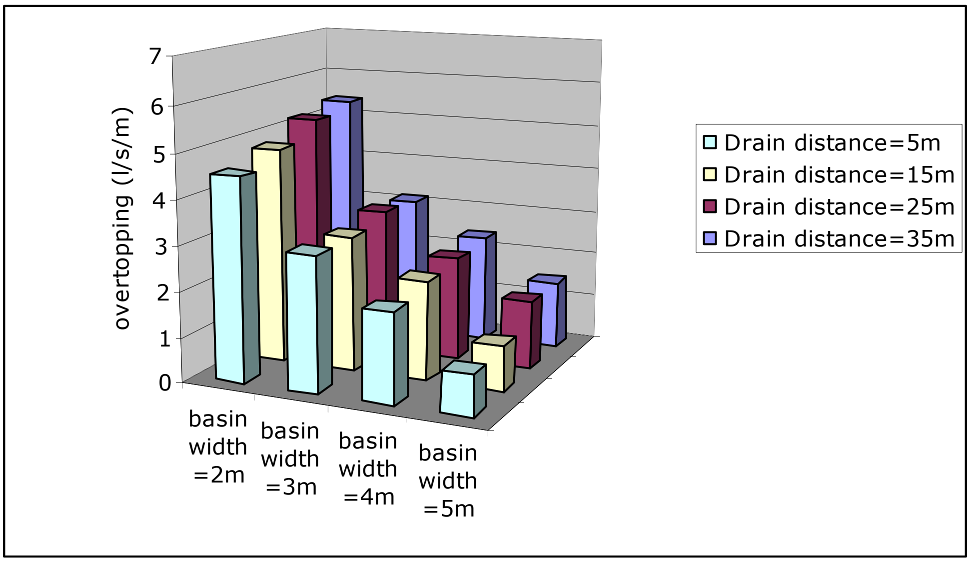


figure 8-14:Results Hondsbossche Sea Defence (Drain diameter=0.8, Basin height = 1m)

In figure 8-15, the wave overtopping discharge is plotted as a function of the width of the basin (3m-5m) and the height of the basin (1m-2m). Fixed parameters are the drain distance (15m) and the drain diameter (0.8m). It can be seen that several combinations lead to a situation where an average wave overtopping discharge that is less than 1 l/s/m is obtained.

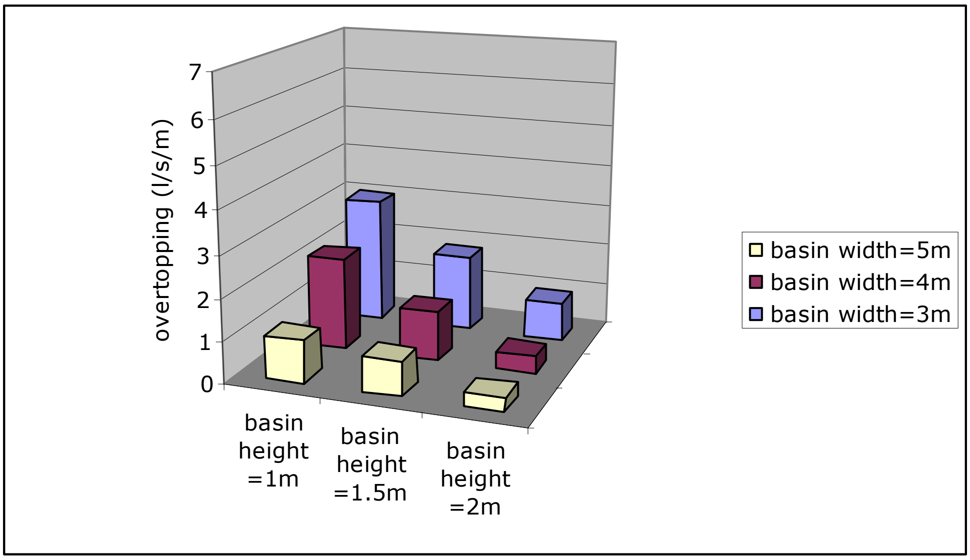


figure 8-15: Results Hondsbossche Sea Defence (Drain diameter =0.8, Drain distance =15m)

In figure 8-16, the wave overtopping discharge is plotted as a function of the width of the basin (3m-5m) and the height of the basin (1m-2m). Fixed parameters are the drain distance (15m) and the drain diameter (1.0 m). It can be seen that several combinations lead to a situation where an average wave overtopping discharge that is less than 1 l/s/m is obtained. The differences with figure 8-15 are negligible. This implies that a further increase of the drain capacity does not result into a significant higher efficiency.

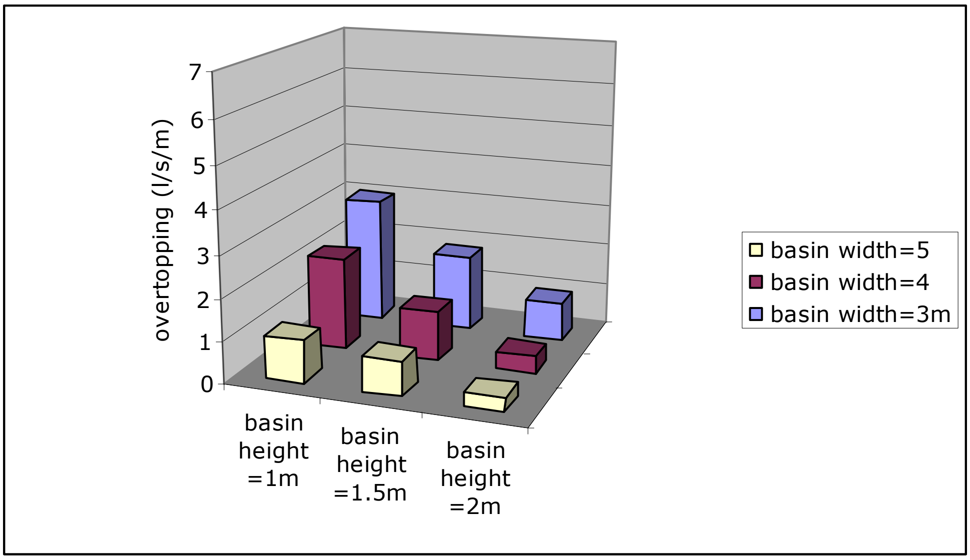


figure 8-16: Results Hondsbossche Sea Defence (Drain diameter=1m, Drain distance=15m)

The runs that lead to an average wave overtopping discharge that is less than 1 l/s/m are shown in table 8-15. It is stressed that these values are based on the numerical model.

Drain diameter	Drain distance	basin width	basin height
(m)	(m)	(m)	(m)
0.8	5	5	1
0.8	15	4	2

table 8-15: Required dimensions of the drains and the crest basin for the Hondsbossche Sea Defence in the future situation.

For both situations displayed in table 8-15 the validity of the numerical program should be check with the use of the dimensionless parameter n. This parameter is explained in section 3.6 (page 37). The relation between the validity of the numerical program and the dimensionless parameter n is explained in section 6.2 (page 86).

The alternative with a basin width of 5m and a basin height of 1m has a ‘ n value’ of 1. This implies that the numerical model is under predicting the actual drain quantities. Therefore, it is assumed that the dimensions of the drains might be some smaller.

The alternative with a basin width of 4m and a basin depth of 2m has a ‘n value’ of 6. This implies that the numerical model is predicting the drain quantities with only a moderate error.

8.3.6 Conclusions

In this case study, three possible solutions for the Hondsbossche Sea Defence are discussed:

- Traditional Dike heightening
- Reinforcing the inner slope of the dike
- Applying a Crest Drainage Dike

A dike heightening of 3.10 is required when applying traditional dike heightening. Suppose that the inner slope is reinforced in such a way that 10 l/s/m is allowed, only 34 cm of dike heightening is required.
Applying a Crest Drainage Dike at the Hondsbossche Sea Defence is possible. However, the dimensions of the crest basin are significant larger than is suggested in previous feasibility studies [DHV, 2005].

The studies in this chapter are based on several assumptions. The most important assumption that has not been verified yet is the value of the length spreading effect. In the studies in this report, it is assumed that the length spreading effect has a factor of 2. If this value is lower, larger crest basins are required.

8.4 Case study II: The Perkpolder Sea defence

8.4.1 Introduction

The following text has been adopted from the ComCoast website:

"The pilot project Perkpolder concerns a project in which a former car ferry harbor and an adjacent polder are in the picture for compensation of estuarine wetland nature by the Ministry of Transport, Public Works and Water Management and the Province of Zeeland. This new nature area compensates a lost of estuarine nature due to the already carried out deepening of the Westerschelde for container shipping to Antwerp. The project is combined with the development of some economic and recreational activities of the community of Hulst. Depending on the way the new nature area will be realized, adjustments of the present embankments may be needed in which the ComCoast concept may be applied"

A part of the plan is to create a dike around the harbor. (see figure 8-17) In this section a comparison between the use of a Crest Drainage Dike and a traditional dike will be executed. Since the goal of the section is to give an order of magnitude regarding the crest freeboard differences between a traditional dike and a Crest Drainage Dike, the comparison will only be done using rough calculations. Therefore it is stressed that the calculations cannot be used for other purposes.

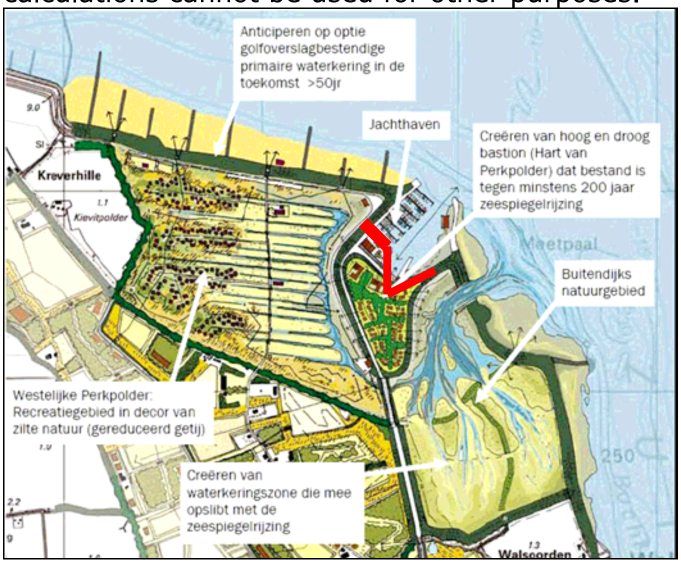


figure 8-17: Overview of the Perkpolder.

8.4.2 Analysis

Boundary conditions
The boundary conditions consist of hydraulic and geometric boundary conditions. The hydraulic boundary conditions are based on a study executed by Royal Haskoning and Svasek [Svasek, 2006]. The hydraulic boundary conditions are based on the predicted conditions of the year 2060.
The geometric boundary conditions are not determined yet but are formulated as a result of discussions with the client.

Hydraulic boundary conditions
- Design water level 6.65m + NAP [Svasek, 2006]
- Significant wave height 1.1m [Svasek, 2006]
- Peak period 3.5s [Svasek, 2006]

Geometric boundary condition
- Slope dike $\tan(\alpha)=0.25$
- Bermlevel 6.65m+NAP
- Berm width 5m
- Slope berm 1:25

Assumptions
- $T_p = 1.21T_g$ [DWW, 2001]
- $T_{m-1,0} = 1.1\ T_g$

Analysis crest freeboard reduction
The necessary crest freeboard of a traditional dike is determined with the use of PC-Overtop. The results of this study are shown in table 8-16.

Average wave overtopping discharge	Crest freeboard	Difference with 1 l/s/m
$q_{overtopping}$	R_c	ΔR_c
(l/s/m)	(m+NAP)	(m)
1	7.49	0
10	7.15	0.34
100	6.81	0.68

table 8-16: Results PC-Overtop for the Perkpolder Sea Defence.

From table 8-16, it is derived that a traditional dike with an overtopping criterion of 1 l/s/m implies a minimal crest freeboard of 7.49m + NAP.

Crest Drainage Dike
From table 8-16 it can be derived that a reduction of 34cm can be derived if an overtopping of 10l/s/m is accepted. A reduction of 68cm is possible if 100 l/s/m is accepted. Both situations are analyzed in the following sections.

Reduction of 68cm(100 l/s/m)
Several numerical runs have been executed. Variables are the drain layout and different crest basin layouts. Since there are many combinations possible, only a few parameters are changed. The variable parameters are:
- Drain diameter (0.4m-1.0m)
- Drain distance (10m-30m)
- Crest basin width (2m-4m)

The fixed parameters are
- Drain length 1m
- Length spreading effect 2
- Height crest basin 1.0m

The results of the 42 runs are shown in table 8-17 and table 8-18 and are plotted in figure 8-18 and figure 8-19.

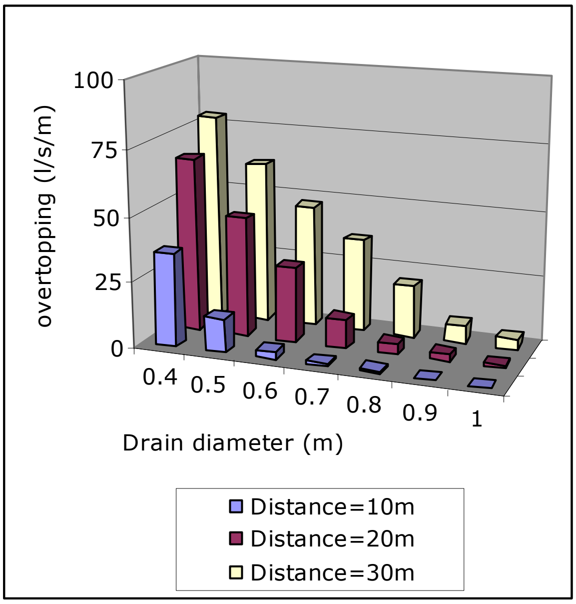


figure 8-18: Results wave overtopping discharge Perkpolder Sea Defence with a crest buffer width of 2m.

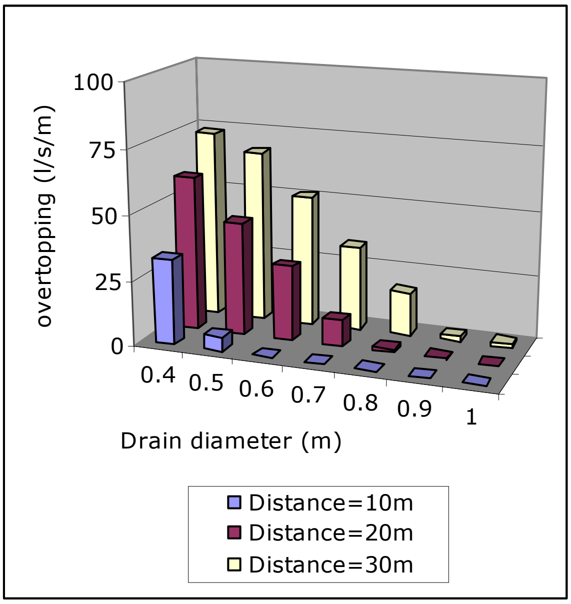


figure 8-19: Results wave overtopping discharge Perkpolder Sea Defence with a crest buffer width of 4m.

Drain diameter (m)	0.4	0.5	0.6	0.7	0.8	0.9	1
Distance=10m	36	13	2.9	1.1	0.9	0	0
Distance=20m	67	46	29	11	4	3	1
Distance=30m	79	62	47	36	20	7	4

table 8-17: Results wave overtopping discharge Perkpolder sea Defence with a crest buffer width of 2m.

Drain diameter (m)	0.4	0.5	0.6	0.7	0.8	0.9	1
Distance=10m	33	5.4	0.2	0	0	0	0
Distance=20m	60	44	29	10	1.3	0.2	0
Distance=30m	73	67	51	33	17	2.1	1.4

table 8-18: Results wave overtopping discharge Perkpolder Sea Defence with a crest buffer width of 4m.

figure 8-18 shows the wave overtopping discharge as a function of the drain diameter and the distance of the drains. It can be seen that both aspects have a large influence on the wave overtopping discharge. figure 8-19 is the same figure as figure 8-18. The only difference is the width of the crest basin. It can be seen that the influence of a larger buffer capacity is moderate. The results show that several combinations could be applied to obtain an overtopping discharge which is less than 1 l/s/m. These combinations are shown in table 8-19.

Crest basin width	Diameter drain	Distance drains
(m)	(m)	(m)
2	0.8	10
2	1.0	20
4	0.6	10
4	0.9	20

table 8-19: Possible layouts Perkpolder Sea Defence to obtain a crest freeboard reduction of 68 cm.

The dimensionless parameter ‘n’ has a value of 4-5 for the basins with a width of 2m and a value of 13-14 for a basin with a width of 4m. The dimensionless parameter ‘n’ is explained in section 3.6 (page 37). It is shown in section 6.2 (page 86) that for n-values of 8 or higher, the accuracy of the model is good and that the model is slightly under predicting the draining discharges for lower values of n.

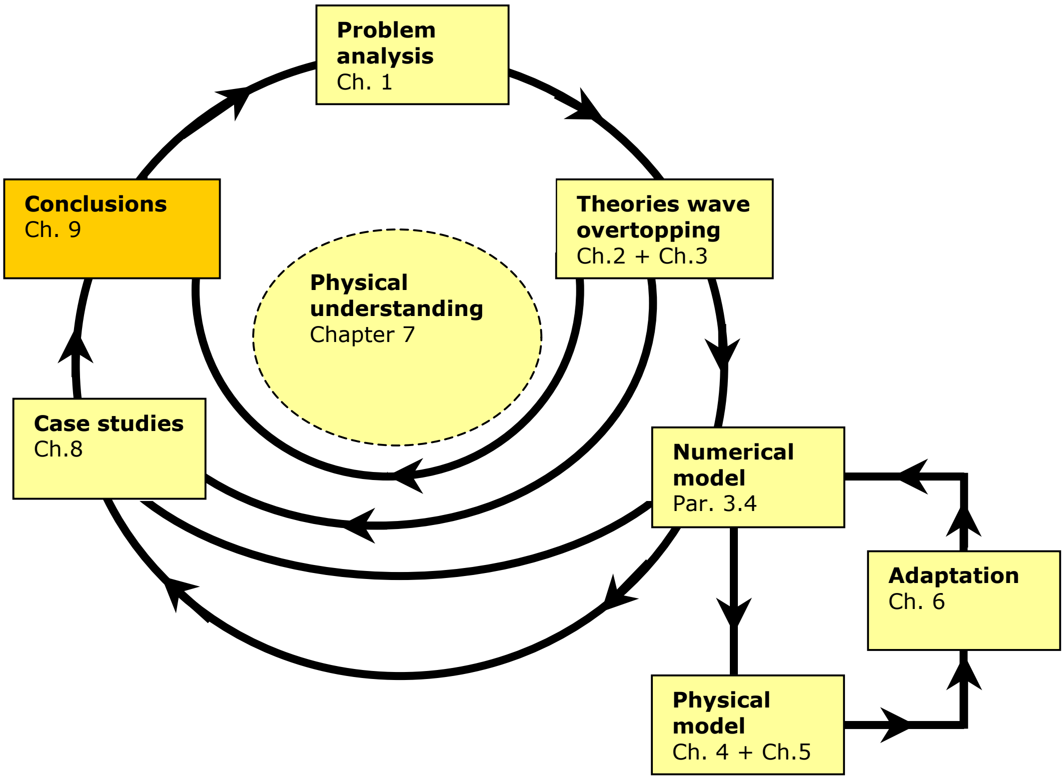
Reduction of 34cm (10 l/s/m)
The analysis as shown in the section above has also been executed for a crest freeboard reduction of 34 cm. This implies that the average wave overtopping discharge is 10 l/s/m (This can be seen in table 8-16). Since the overtopping is only moderate, this analysis has only been carried out for a crest width of 2m. The results of this analysis are shown in table 8-20.

Crest basin width	Diameter drain	Distance drains
(m)	(m)	(m)
2	0.2	10
2	0.3	20
2	0.4	40

table 8-20: Possible layouts Perkpolder Sea Defence to obtain a crest freeboard reduction of 34 cm.

8.4.3 Conclusions
Several numerical experiments have been executed to determine the layout of the drains and the crest basin. This resulted in several possible combinations of the drain diameter, the width of the crest and the distance between the drains. Besides these parameter, several other parameters such as the depth of the crest basin, the length of the drains and the length spreading effect, can be adapted.
It is stressed that the values are derived with the numerical program. This program is based on several assumptions that are not all verified.

9 Conclusions and recommendations



9.1 Conclusions

A numerical computer model, which can predict the effectiveness of the Crest Drainage Dike, is constructed. This model is based on a theoretical analysis and is verified with data obtained from physical model experiments.

The Crest Drainage Dike with a layout as suggested in the feasibility study [DHV, 2005], has only a minor influence on the required crest freeboard of the dike. This is based on two numerical case studies (the Hondsbossche Sea Defence and the Perkpolder Sea Defence) and the current Dutch wave overtopping criteria.

The wave attack on the Hondsbossche Sea Defence is characterized by a small fraction of overtopping waves with a relatively high volume per wave. Therefore, the buffer capacity limits the effectiveness of the Crest Drainage Dike. According to the numerical study, a basin with a width of 4m and a height of 2m is necessary to avoid a future heightening of the crest freeboard. Drains with a diameter of 80cm need to be placed every 15m. Several other layouts are possible but the layout presented here serves only as an indication of the required dimensions.

The wave attack on the Perkpolder Sea Defence is characterized by a relatively large fraction of overtopping waves with a relatively small wave volume per wave. However, a slight decrease in the crest freeboard implies a relatively large increase in the total wave overtopping discharge. Therefore, the drain capacity limits the effectiveness of the Crest Drainage Dike. To decrease the freeboard with 68cm, several layouts are possible. A possible layout is a crest basin with a width of 2m and a height of 1m. Drains with a diameter of 1.0m needs to be placed every 20m.

The numerical model is based on several assumptions that have not been verified in this study. The five most important assumptions are:

- <u>No wave grouping takes place</u>. The numerical model takes no account for wave grouping. Wave grouping might have a serious negative influence on the buffer capacity and thus the effectiveness of the Crest Drainage Dike.

- <u>The length spreading effect has due to the short-crested character of waves a value of 2</u>. This length spreading effect has a direct relationship with the buffer capacity. A smaller value has a negative influence on the effectiveness of the Crest Drainage Dike.

- <u>The determined volumes per wave are Weibull distributed</u> and have therefore no physical upper boundary. If such an upper boundary does exist, this might have a positive influence on the effectiveness of the Crest Drainage Dike.

- <u>No “overshoot” of water takes place</u>. The overtopping water is considered as a sheet flow and will not ’fly’ over the crest. The existence of an overshoot has a negative influence on the effectiveness of the Crest Drainage Dike.

- <u>No reflection takes place in the crest basin</u>. This has been tested in the physical experiments for a specific layout. Here, no reflection took place. However, this has not been tested for other layouts such as a wider or deeper crest basin. Reflection has a positive influence on the effectiveness of the Crest Drainage Dike.

Since most of the assumptions in the numerical model are in the advantage of the effectiveness of the Crest Drainage Dike it is very likely that the actual effectiveness of the Crest Drainage Dike is lower than the calculated effectiveness and that larger dimensions of the layout are required.

9.2 Recommendations

Feasibility
The required dimensions of the buffer capacity and the draining pipes, which are obtained in this report, are significant larger than the dimensions obtained in the feasibility study [DHV, 2005]. It is assumed that these larger dimensions influence the costs of the Crest Drainage Dike considerable. Therefore it is recommended to reconsider the feasibility of the Crest Drainage Dike.

Wave overtopping criteria
According to the Dutch guidelines, the overtopping criteria should be expressed in an average wave overtopping discharge. However, it has been shown that an average wave overtopping discharge only characterizes a small fraction of the physical aspects of overtopping. This gives serious doubts on the current Dutch overtopping criteria. Since these criteria form the basis of the conclusions regarding the Crest Drainage Dike, it is recommended to improve the current Dutch overtopping criteria. Based on these new criteria, the conclusions of this report should be reconsidered.

Verification of the assumptions
The influence of the assumptions stated in the conclusions might have a serious influence on the effectiveness of the Crest Drainage Dike. In case a better model is required, it is recommended to study the assumptions which are adopted in this report regarding the influence of:

- wave grouping
- length spreading effect
- the existence of an upper boundary of the Weibull distributed wave volumes
- overshooting
- reflection of waves

•

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I. Dimensionless wave overtopping parameters

In chapter 2, the dimensionless overtopping parameters derived by Owen and van der Meer have been discussed. Several researchers have used other dimensionless parameters. An overview of these parameters is given in table A-1.

Autor	Q*	R*
Saville and Caldwell (1953)	$\frac{q}{H}\frac{T}{L}$	$\frac{R_c}{H}$
Sibul (1955)	$\frac{qT}{H^2}$	$\frac{R_c}{H}$
Paape (1960)	$\frac{qT}{HL}$	$\frac{R_c}{H\tan^{\frac{3}{2}}\alpha}$
Weggel (1976)	$\frac{q}{\sqrt{hH^3}}$	$\frac{R_c}{T\sqrt{gH}}$
Owen (1980)	$\frac{q}{T_m gH_s}$	$\frac{R_c}{T_m\sqrt{gH_s}}$
Ahrens and Heimbaugh (1988)	$\frac{q}{\sqrt{gH_s^3}}$	$\frac{A_{98}-R_c}{H_s}$
Waal and van der Meer (1992)	$\frac{q}{\sqrt{gH_s^3}}$	$\frac{R_c}{\tan\alpha T_m\sqrt{gH_s}}$
Pilarczyk (1994)	$\frac{q}{H_s gT_m\sqrt{\tan\alpha}}$	$\frac{R_c}{\tan\alpha T_m\sqrt{gH_s}}$
Pedersen (1996)	$\frac{qT_m}{L^2}$	$\frac{R_c}{H_s}$
Hedges and Reis (1997)	$\frac{q}{\sqrt{gA_{98}^3}}$	$\frac{R_c}{\tan\alpha T_m\sqrt{gH_s}}$
Franco and Franco (1999)	$\frac{q}{\sqrt{gH_s^3}}$	$\frac{R_c}{H_s}$

table A-1:overview of dimensionless crest freeboard and overtopping discharges [Schüttrumpf, 2001]

q	=	average overtopping discharge per linear meter of crest	(m^2/s)
R_c	=	Crest freeboard	(m)
A_{98}	=	wave run-up height	(m)
L	=	wave length at the toe of the structure	(m)
L_0	=	wave length at deep water	(m)
H	=	wave height for regular waves	(m)
H_s	=	significant wave height	(m)
T	=	wave period for regular waves	(s)
T_s	=	significant wave period	(s)
g	=	acceleration due to gravity	(m/s^2)

II. Dimensionless parameters

According to [Owen, 1980] the following equation yields for the dimensionless discharge (Q^*) and the dimensionless freeboard (R_c^*)

$$Q^* = \frac{Q}{T_m g H_s}$$ **Equation 0-1**

And

$$R_c^* = \frac{R_c}{T_m \sqrt{g H_s}}$$ **Equation 0-2**

$$Q^* = A e^{-BR^*}$$ **Equation 0-3**

Where

Q	=	average wave overtopping discharge	(m^3/s)
Q^*	=	dimensionless average wave overtopping discharge	(-)
R_c	=	crest freeboard	(-)
R^*	=	dimensionless crest freeboard	(-)
T_m	=	mean wave period	(s)
g	=	acceleration due to gravity	(m/s^2)
H_s	=	significant wave height	(m)
A	=	empirical coefficient	(-)
B	=	empirical coefficient	(-)

A and B are empirically derived coefficients. The coefficients are shown in table A-2.

Seawall slope	A	B
1:1	$7.94.10^{-3}$	20.1
1:2	$9.39.10^{-3}$	21.6
1:3	$1.09.10^{-2}$	28.7
1:4	$1.16.10^{-2}$	41.0
1:5	$1.31.10^{-2}$	55.6

table A-2: Coefficients Owen

III. Draining aspects

To obtain a proper model for the Crest Drainage Dike, there is a need to investigate the behavior of reservoirs, which are emptied by the use of a drain.

A reservoir, such as used in the Crest Drainage Dike, is shown in figure A-1. It contains a reservoir with a height h_{max} and a surface A_0. Besides the reservoir there is a drain with drain length L and drain surface μA

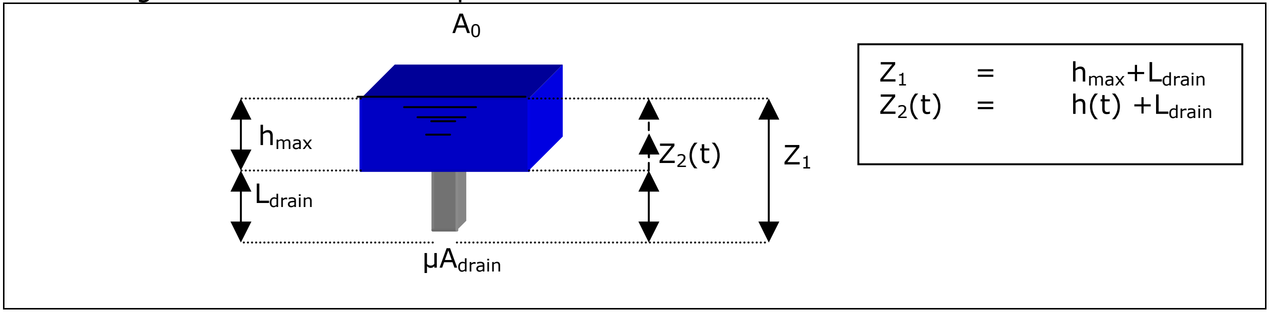


figure A-1: Schematization of a crest basin with a drain

Z_1	=	water level at t_1	(m)
Z_2	=	water level at t_2	(m)
μ	=	entrance loss	(-)
A_{drain}	=	drain surface	(m^2)
g	=	acceleration due to gravity	(m/s^2)
A_0	=	surface of the reservoir	(m^2)

To get insight in the water level as function of the time some analysis is required.

A start will be made with the balance of volume

$$A_0 \frac{dz_0}{dt} = -Q \qquad \textbf{Equation 0-4}$$

A quasi-stationary approach gives

$$Q = \mu A_{drain} \sqrt{2g(z_0 - z_1)} \qquad \textbf{Equation 0-5}$$

$$z_o - z_1 = Z \qquad \textbf{Equation 0-6}$$

Combining the above formulas will give:

$$A_0 \frac{dZ}{dt} + \mu A_{drain} \sqrt{2gZ} = 0 \qquad \textbf{Equation 0-7}$$

This is a simple 1st order differential for Z. For a certain given Z_1 for $t=t_1$, this can be integrated:

$$t_2 - t_1 = \frac{A_0}{\mu A_{drain}\sqrt{2g}} \int_{Z_2}^{Z_1} \frac{1}{\sqrt{Z}} dz \qquad \textbf{Equation 0-8}$$

(A_0 is independent and is therefore placed before the integral)

Integration gives:

$$t_2 - t_1 = \frac{2A_0}{\mu A_{drain}\sqrt{2g}} \left[Z^{\frac{1}{2}} \right]_{Z_2}^{Z_1} = \frac{2A_0}{\mu A_{drain}\sqrt{2g}} (\sqrt{Z_1} - \sqrt{Z_2}) \qquad \textbf{Equation 0-9}$$

Choosing $t_1=0$ and rewriting gives

$$Z_2(t) = \left(\sqrt{Z_1} - \frac{\mu A_{drain}\sqrt{2g}}{2A_0} \cdot t \right)^2$$ **Equation 0-10**

This is the direct relationship between the water level in the reservoir and the time

In case L=0:

$$h(t) = \left(\sqrt{h_{\max}} - \frac{\mu A_{drain}\sqrt{2g}}{2A_0} \cdot t \right)^2$$ **Equation 0-11**

This relation is shown in figure A-2. The x and y axes are made dimensionless with respectively t/t_E and $h(t)/h_{max}$

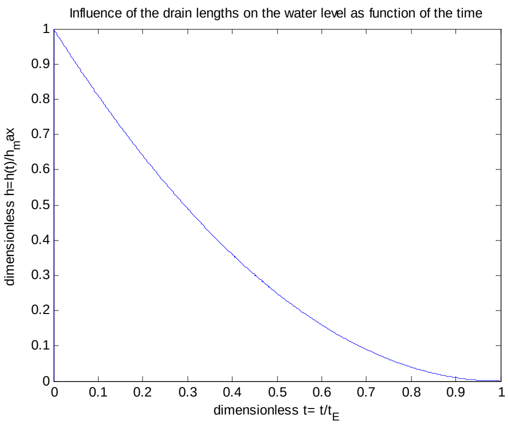


figure A-2: Relation between the draining time and the water level in the crest basin

From this graph it is easy to see that the last bit of water is very inefficiently drained. The last 20 percent of water takes 50% of the time. It would be favorable if this part of water could be drained more efficiently. Before this will be treated, an analysis of the total draining time will be given.

Total draining time of a reservoir
From the previous analysis, it is possible to determine the necessary time to drain the total volume of the reservoir.

In case the total reservoir is empty:

- $Z_1 = h_{max} + L_{drain}$
- $Z_2 = L_{drain}$ (assuming that the volume in the drain is negligible)

$$L = \left(\sqrt{h_{\max} + L_{drain}} - \frac{\mu A_{drain}\sqrt{2g}}{2A_0} \cdot t \right)^2$$ **Equation 0-12**

Rewriting gives

$$t_2 = t_E = \left(\sqrt{h_{\max} + L_{drain}} - \sqrt{L_{drain}} \right) \cdot \frac{2A_0}{\mu A_{drain}\sqrt{2g}}$$ **Equation 0-13**

Where

t_E	=	Duration of draining a crest basin until it is empty	(s)

III.I Influence of drain length L on the time to empty the reservoir
Since there is an expression for the total draining time, analysis on the influence of the drain length L can be applied.

The length of the drain has an influence on the total time, which is necessary to empty the reservoir. First, some parameters will be made dimensionless.

The drain length will be expressed in terms of the reservoir height:

$$L_{drain} = c \cdot h_{\max}$$ **Equation 0-14**

The time to empty the crest structure will be made dimensionless with the total time in case the drain length is zero. ($t_{E,L=0}$).

$$t_{E,\dim} = \frac{t_E}{t_{E,L_{drain}=0}}$$ **Equation 0-15**

Where

$t_{E,\dim}$	=	dimensionless duration of draining a crest basin untill it is empty	(-)

Applying Equation 0-15 gives

$$t_{E,L_{drain}=0} = \sqrt{h_{\max}} \frac{2A_0}{\mu A_{drain}\sqrt{2g}}$$ **Equation 0-16**

$$t_E = \left(\sqrt{h_{\max} + L_{drain}} - \sqrt{L_{drain}}\right)\frac{2A_0}{\mu A_{drain}\sqrt{2g}} = \left(\sqrt{h_{\max} + ch_{\max}} - \sqrt{ch_{\max}}\right)\frac{2A_0}{\mu A_{drain}\sqrt{2g}}$$ **Equation 0-17**

$$t_{E,\dim} = \frac{t_E}{t_{E,L_{drain}=0}} = \frac{\left(\sqrt{h_{\max} + ch_{\max}} - \sqrt{ch_{\max}}\right)\frac{2A_0}{\mu A_{drain}\sqrt{2g}}}{\sqrt{h_{\max}}\frac{2A_0}{\mu A_{drain}\sqrt{2g}}}$$ **Equation 0-18**

Rewriting gives:

$$\boxed{t_{E,\dim} = \sqrt{c+1} - \sqrt{c}}$$ **Equation 0-19**

This relationship is shown in figure A-3

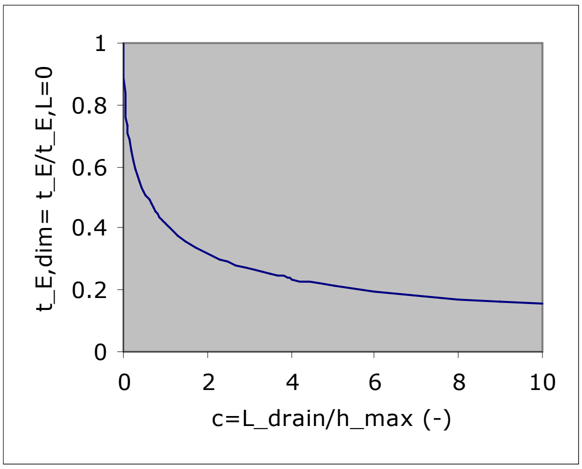


figure A-3: relation between the drain length and the total draining time

figure A-3 may be difficult to interpret. Therefore an example will be given in the box below:

Example

Suppose that a filled reservoir with a drain length of L_{drain}=0.00m (no drain) needs 100 seconds to empty. The height of the reservoir is 2m

When applying a drain with a drain length of 6m, the following dimensionless parameter can be applied:

$$c = \frac{L_{drain}}{h_{\max}} = \frac{6m}{2m} = 3$$

In figure x, it can be seen that the total draining time is around 25 percent of its draining time in case the drain length is zero.

It is also possible to use the formulas:

$$t_{E,\dim} = \sqrt{c+1} - \sqrt{c} = \sqrt{3+1} - \sqrt{3} \approx 0.27$$

III.II Influence of drain length L_{drain} on the water level as function of time

In the previous sections, it is shown that a longer drain length gives a considerably shorter draining duration. However, this does not say much about the water level as function of the time. It is very likely that, due to the use of a drain with a certain length, the water level as function of the time will also change.

An analysis has been carried out to investigate this influence. The results of this analysis are shown in figure A-4.

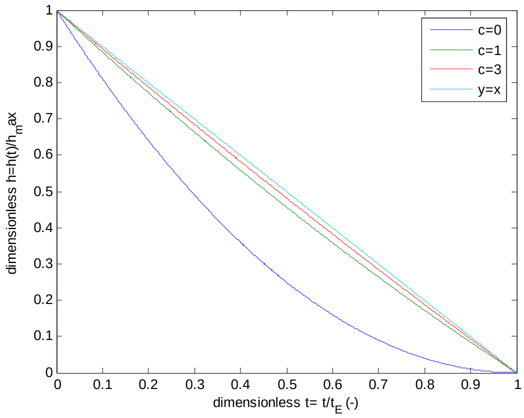


figure A-4: Influence of the drain length on the water level as function of time

This relation is based on Equation 0-10.It can be seen that the shape of the relation will considerably change. When a longer drain is used, the relation tends to be linear.

III.III Conclusions regarding the drain analysis

In this chapter two phenomena of a reservoir are analyzed.

The duration of emptying a filled reservoir

When applying a drain with a certain length, the duration of emptying the reservoir will be considerably shortened. For example, when a drain with a length of 25 percent of the reservoir height is used, the draining time will be shortened with 40 percent

The relation between the water level in the reservoir and the time
The water level in a reservoir where no drain length is applied will depend on the time squared:

- No drain length $h(t) \propto t^2$

When applying a drain with a certain length, the relation tends to behave linearly:

- Drain length $h(t) \propto t$

Since the effects of a longer drain length are positive it is assumed that this phenomena will be used to optimize the effectiveness of the Crest Drainage Dike. In a real situation the crest structure height will be around 0.80m and the filling of this structure will be around 80 percent. Therefore the h_{max} is around 0.64meter.

It is assumed that it is possible to construct a drain with a height difference (L) of around 1 meter. This means that the dimensionless drain length $a = \frac{L_{drain}}{h_{\max}} = \frac{1m}{0.64m} = 1.6$

Applying Equation 0-9, it turns out that the drain duration is shortened with 65 percent

In figure A-4 it is obvious to see that for engineering purposes a linear relation can be used.

III.IV Lineairisation of h(t)

As described in the previous section, the draining time of a reservoir has the following shape:

$$t_{E,fundamental} = \left(\sqrt{h_{\max} + L_{drain}} - \sqrt{L_{drain}}\right) \cdot \frac{2A_0}{\mu A_{drain}\sqrt{2g}} \quad \textbf{Equation 0-20}$$

Where

h_{max}	=	maximum water level in the reservoir	(m)
L_{drain}	=	length of the drain	(m)
A_0	=	surface of the reservoir (independent on the water level)	(m^2)
μ	=	entrance losses of the drain	(-)
g	=	acceleration due to gravity	(m/s^2)

A linear relation is preferred and this is constructed as follows:

$$t_{E,linear} = \frac{V_{crestbasin}}{Q_{drain}} = \frac{A_0 h_{\max}}{v \mu A_{drain}} \quad \textbf{Equation 0-21}$$

with

$$v = \sqrt{2g(\theta h_{\max} + L_{drain})} \quad \textbf{Equation 0-22}$$

Combining this gives:

$$t_{E,linear} = \frac{V_{crestbasin}}{Q_{drain}} = \frac{A_0 h_{\max}}{\sqrt{2g(\theta h_{\max} + L_{drain})}\mu A_{drain}} \quad \textbf{Equation 0-23}$$

The main question that yields is: What is θ?

To answer this question the linear draining time will be compared with the theoretical draining time:

$$t_{E,linear} = t_{E,fundamental} \quad \textbf{Equation 0-24}$$

$$\left(\sqrt{h_{\max}+L_{drain}}-\sqrt{L_{drain}}\right)\cdot\frac{2A_0}{\mu A_{drain}\sqrt{2g}}=\frac{A_0 h_{\max}}{\sqrt{2g(\theta h_{\max}+L_{drain})}\mu A_{drain}}$$ **Equation 0-25**

L is made dimensionless by expressing it in h_{max}: $L=ch_{max}$

$$2\left(\sqrt{h_{\max}+ch_{\max}}-\sqrt{ch_{\max}}\right)=\frac{h_{\max}}{\sqrt{(\theta h_{\max}+ch_{\max})}}$$ **Equation 0-26**

rewriting gives:

$$\theta(L=ch_{\max})=\frac{1}{4\left(\sqrt{c+1}-\sqrt{c}\right)^2}-c$$ **Equation 0-27**

c=0	(no drain is used)	:	θ = 1/4
c→∞	(a drain with infinitive length	:	θ↑1/2

This relation is shown in figure A-5

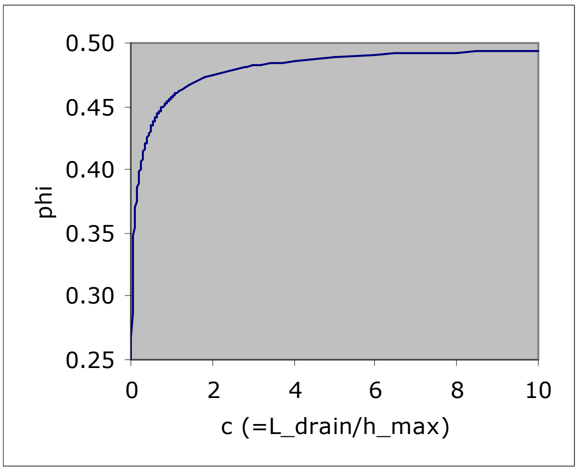


figure A-5: Relation between the drain lenght and the correction factor θ

IV. Numerical model

IV.I Introduction

In section 3.4, a set-up for a numerical model is given. This appendix describes this model in full detail. All the equations used in this section are based on the analysis in chapter 3. As is stated in section 3.4, the design of the model is based on a top-down design method. [Mesman, 1991]. Applied to the used model, figure A-6 is obtained.

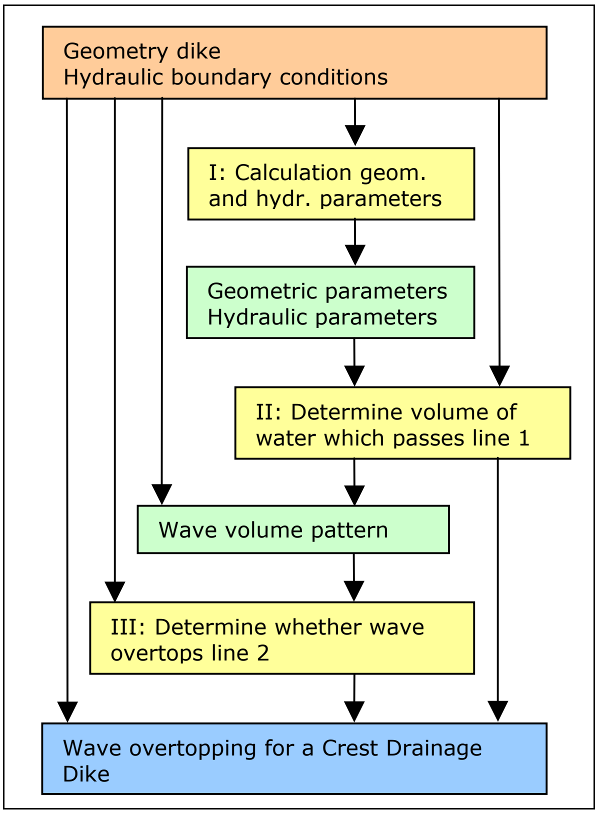


figure A-6: top-down designing

Use will be made of a Program Structure Diagram (PSD). The construction shown figure A-6 is 'translated' into a PSD. The PSD is shown in figure A-7.

Main program

loadvariables			
for br=1:1000 (nr of repetitions for statistical analysis)			
	for e=1:1 (nr of situations for comparison)		
		GeometricBC	(module I)
		HydraulicBC	(module I)
		WeibullDistributedWaves	(module II)
		OvertoppingProcess	(module III)
		Results	
PlotOptions			

figure A-7: Program Structure Diagram main program

This main program will be treated step by step. It can be seen that this structure makes it easier to understand and adapt the program. The loading of variables is described in section IV.II. Module I, the geometric and hydraulic boundary conditions is described in section 3.4. Since the only used procedures are the use of formulas, these will be summed up.

Module II, the determination of the volume of the incoming waves, is described in section IV.IV. Since there are several procedures used, use will be made of a PSD to explain the processes.
Module III, the overtopping process, is described in section IV.V. Several procedures are used and therefore use has been made of a PSD.

IV.II Load variables

Variables are loaded with the use of two input files. In these files, the hydraulic and geometric parameters can be adjusted. The program reads the following parameters:

General parameters		
Acceleration due to gravity	g	[m/s^2]
Geometry dike		
Crest freeboard	Rc	[m]
Slope	tan(α)	[-]
Influence factor for berms	g_berm	[-]
Influence factor for friction	g_friction	[-]
Influence factor for angle of attack	g_beta	[-]
Geometry crest basin		
Width of the crest basin	Widthcrest	[m]
Height of the crest basin	Heightcrest	[m]
Efficiency of the crest	Efficiencycrest	[-]
Geometry drains		
Length of the drain	Ldrain	[m]
Diameter of the drains	Ddrain	[m]
Distance between the drains	Distdrains	[m]
Entrance friction	Entrancefriction	[-]
Hydraulic parameters		
The number of waves	NumberOfWaves [-]	
The significant wave height	Hm0	[m]
Average wave overtopping (line 1)	q	[m^3/s/m]
Wave steepness	s_0	[-]

IV.III Calculation geometric and hydraulic parameters

In the first module, "Calculation geometric and hydraulic parameters", parameters will be calculated which can be derived from the output parameters. To get a good overview of these calculations three subsets are created; geometry crest basin, geometry drain and hydraulic boundary conditions.

Geometry crest basin

The geometry of the crest basin is shown in figure A-8.

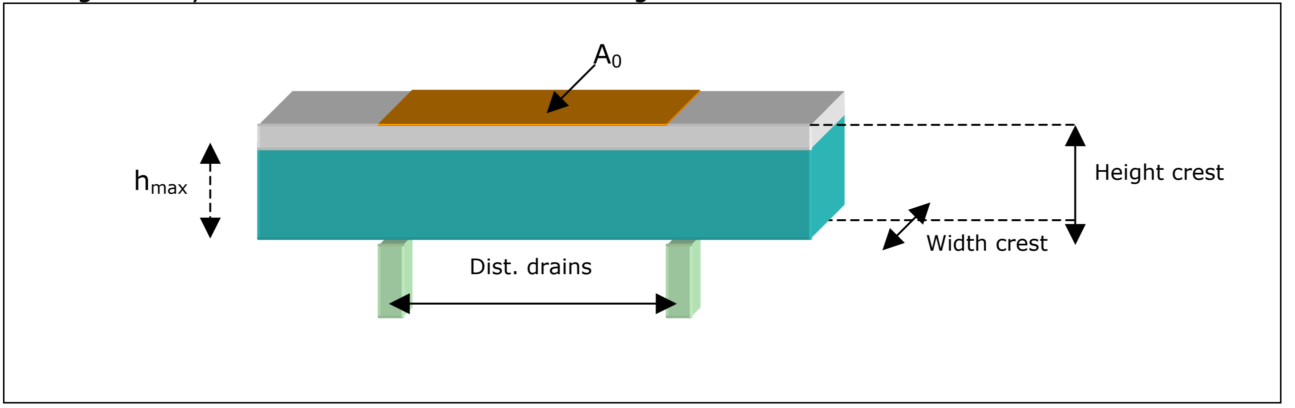


figure A-8: Geometry crest basin

The surface of the crest basin is calculated by multiplying the distance of the drains with the width of the crest.

$$A_0 = Distdrains \cdot Widthcrest \qquad (m^2) \qquad \textbf{Equation 0-28}$$

The maximum water level in the crest is calculated by multiplying the crest efficiency with the height of the crest basin.

$$h_{\max} = Efficiencycrest * Heightcrest \qquad (m) \qquad \textbf{Equation 0-29}$$

The maximum crest basin volume is calculated by multiplying the maximum water level with the crest width.

$$V_{crest\max} = h_{\max} \cdot Widthcrest \qquad (m^2) \qquad \textbf{Equation 0-30}$$

Geometry drain

The entrance friction factor μ

$$\mu = \sqrt{\xi} \qquad (-) \qquad \textbf{Equation 0-31}$$

The surface of the drain

$$A = \tfrac{1}{4}\pi D_{drain}^2 \qquad (m^2) \qquad \textbf{Equation 0-32}$$

The time to empty a filled crest

$$t_{fullcrest} = \left(\sqrt{(h_{\max} + L_{drain}} - \sqrt{L_{drain}}\right) \cdot \frac{2A_o}{\mu A_{drain}\sqrt{2g}} \qquad (s) \qquad \textbf{Equation 0-33}$$

Hydraulic boundary condition

The wavelength

$$L = \frac{H_{m0}}{s_0} \qquad (m) \qquad \textbf{Equation 0-34}$$

The wave period

$$T = \sqrt{\frac{2\pi L}{g}} \qquad (s) \qquad \textbf{Equation 0-35}$$

Duration of the storm

$$t_{storm} = NumberOfWaves \cdot T \qquad \text{(s)} \qquad \textbf{Equation 0-36}$$

Irribarren parameter

$$\xi = \frac{slope}{\sqrt{s_0}} \qquad \textbf{Equation 0-37}$$

Wave run-up height exceeded by 2% of the waves

$$z_2 = 1.75 H_{m0} \gamma_{berm} \gamma_{friction} \gamma_{\beta} \xi \qquad \text{(m)} \qquad \textbf{Equation 0-38}$$

Probability on overtopping per wave

$$P_{ov} = e^{-\left(\sqrt{-\log(0.02)}\frac{R_c}{z_2}\right)^2} \qquad \text{(-)} \qquad \textbf{Equation 0-39}$$

Scalefactor of the Weibull distribution

$$a = 0.84 T_m \frac{q}{P_{ov}} \qquad \text{(-)} \qquad \textbf{Equation 0-40}$$

An overview of module I is given in figure A-9.

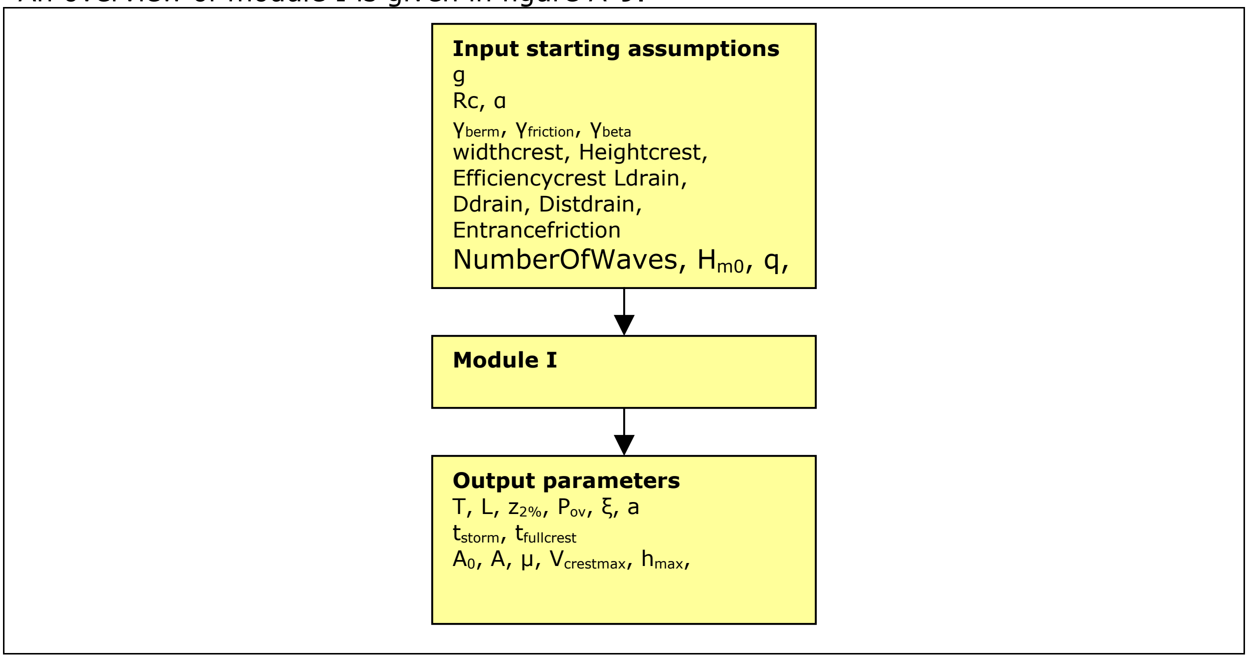


figure A-9: overview input and output of module I

IV.IV Determine the volume of water which passes line 1

Whether a wave overtops line 1, is decided by using the theory described in section 3.3. To translate this theory into a program, a PSD is used. This PSD is shown in figure A-10.

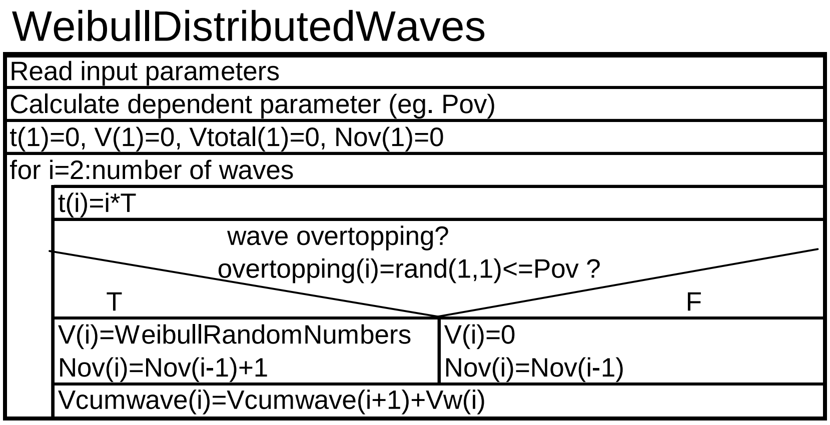


figure A-10: PSD for the generation of waves

To apply this theory use has been made of a Weibull random number generator. This generates random numbers using the scale factor 'a' and the shape factor 0.75 (see section 3.3).

The output of this module is the overtopping of line 1 as function of the wave number. Besides this overtopping per wave V_w, the total wave volume which has passed line 1 until that time is recorder ($V_{cumwave}$) as well as the number of wave which are overtopping (N_{ov})

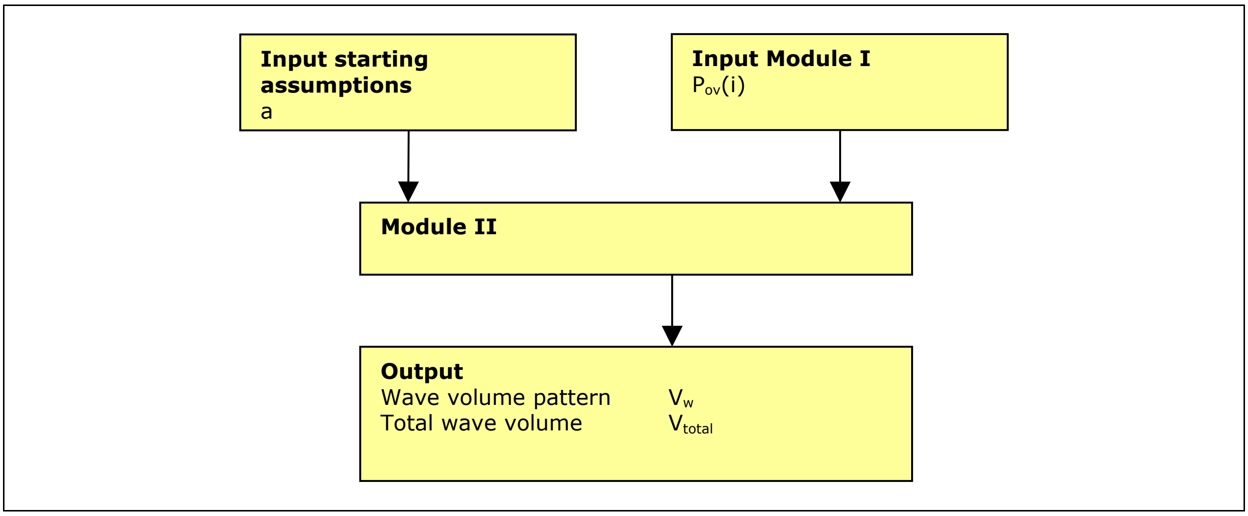


figure A-11: Overview input and output of module II

IV.V Determine whether wave passes line 2.

To determine whether a wave passes line 2, the theory described in section 3.3 will be used. This implies that three "if statements" need to be used. How these are applied can be seen in the program structure diagram in figure A-12.

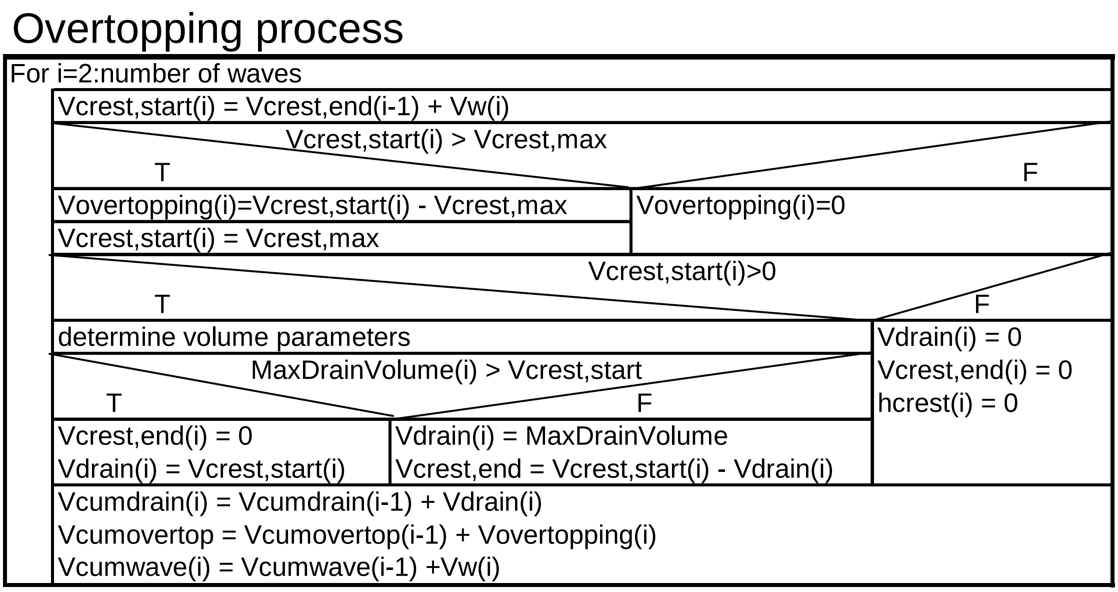


figure A-12: PSD for the overtopping process

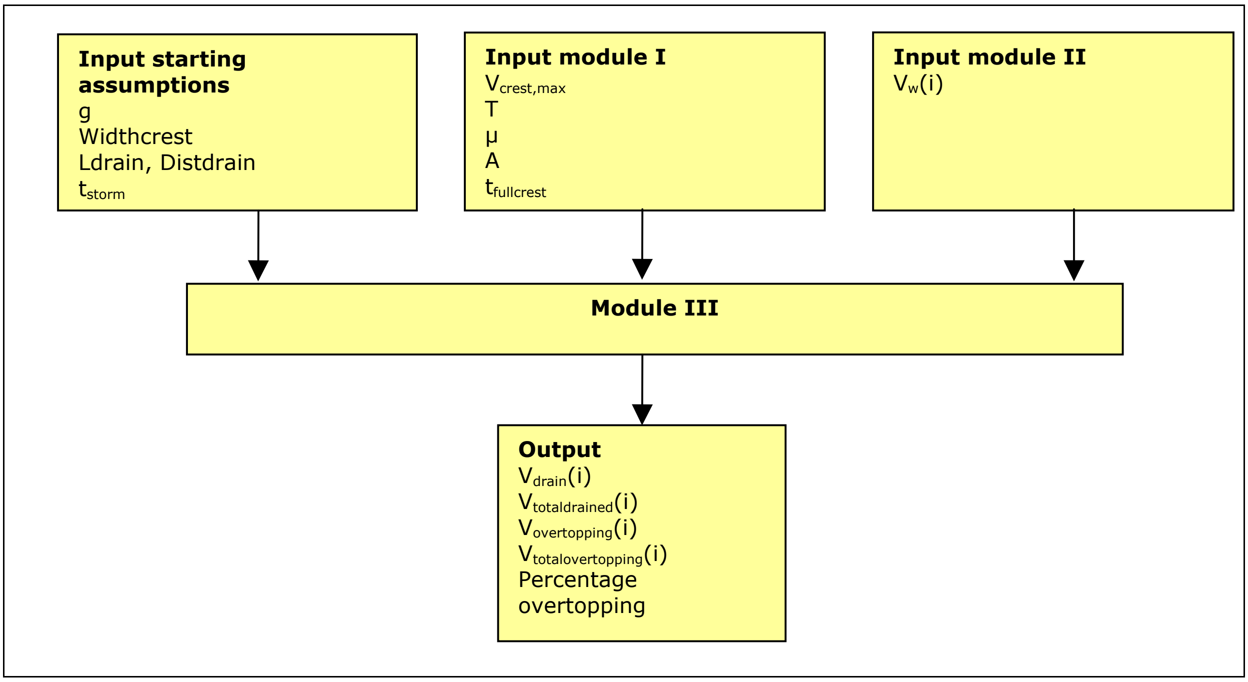


figure A-13: Overview input and output of module III

V. Matlab code numerical program

This appendix gives an overview of the Matlab code that is constructed. A manual of how this code can be used is given in appendix IX.

```
%Main program
%_______________________________________________________________________
%_______________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%_______________________________________________________________________
%_______________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%_______________________________________________________________________
%_______________________________________________________________________
%This program calculates the overtopping quantities at a Crest Drainage
%Dike. For simple structures, no support of other programs is needed. For
%structures with a more complicated geometry, the use of the computer
%program PC-Overtop is neccessary to determine the z2%.

%This program is part of the graduation work at Delft University of
%Technology. The program is based on the theories developed in the written
%thesis.

clear
clf
loadvariables                   %imports the input parameters
for br=1:1                      %nr. of repetitions (statistical)
    for e=1:1                   %nr. of situations (comparison)
        GeometricBC             %calculates geom. param. crest basin
        HydraulicBC             %calculates hydraulic BC param.
        WeibullDistributedWaves %creates wave volumes with Weibull distr.
        OvertoppingProcess      %calculates overtopping
        Results                 %Calculates resulting parameters
    end;
end;
PlotOptions                     %several plotoptions
```

```
%load variables
%______________________________________________________________________________
%______________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%______________________________________________________________________________
%______________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%______________________________________________________________________________
%______________________________________________________________________________
subset                  =   load('allsubsets.txt');
parameters              =   load('parametersA.txt');

testnr                  =   subset(:,1);
                            %test number                          (-)

Hm0                     =   subset(:,2);
                            %wave height                          (m)

Rc                      =   subset(:,3);
                            %crest freeboard                         (m)

s0                      =   subset(:,4);
                            %wave steepness                       (-)

q                       =   subset(:,5);
                            %average wave overtopping        (m3/m/s)

Ddrain                  =   subset(:,6);
                            %Diameter of the drains               (m)

Ldrain                  =   subset(:,7);
                            %Lenght of the drains                 (m)

Distdrains              =   subset(:,8);
                            %(distance between the drains)        (m)

Entrancefriction        =   subset(:,9);
                            %entrancefriction of the drains       (-)

%testpercentage         =   subset(:,10);
                            %percentage overtopping...
                            %according to physical tests

%q_crestmeasured        =   subset(:,11);
                            %q_crest measured in...            (m2/s)
                            %physical experiments

%q_overtoppingmeasured  =   subset(:,12);
                            %q_overtopping measured in...      (m2/s)
                            %physical experiments

g                       =   parameters(1);
                            %acceleration due to gravity       (m/s2)

%geometry dike (input)
g_berm                  =   parameters(2);
                            %influence factor for berms           (-)

g_friction              =   parameters(3);
                            %influence factor for friction        (-)

g_beta                  =   parameters(4);
                            %influence factor for angle of attack (-)

slope                   =   parameters(5);
                            %outer slope of the dike (tan(a))     (-)

%geometry crest (input)
Widthcrest              =   parameters(6);
                            %width of the crest basin             (m)

Heightcrest             =   parameters(7);
                            %height of the crest basin            (m)

Efficiencycrest         =   parameters(8);
                            %efficiency of the crest basin        (-)

%hydraulic boundary conditions (input)
NumberOfWaves           =   parameters(9);
                            %Number of waves in the record        (-)
```

```
%geometricBC
%_________________________________________________________________________
%_________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%_________________________________________________________________________
%_________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%_________________________________________________________________________
%_________________________________________________________________________
A0(e)           =   Distdrains(e)*Widthcrest;
                    %Surface of reservoir                           (m2)

hmax            =   Efficiencycrest*Heightcrest;
                    %Maximum waterlevel in the crest basin          (m)

Vcrestmax       =   Widthcrest*Heightcrest*Efficiencycrest;
                    %Maximum volume in the crest                    (m2)


%geometry drain (output)
mu(e)           =   sqrt(Entrancefriction(e));
                    %entrance factor                                   (-)

A(e)            =   0.25*pi*Ddrain(e)^2;
                    %surface drain                                    (m2)

b(e)            =   mu(e)*A(e)*sqrt(2*g)/(2*A0(e));
                    %assist parameters                           (m^0.5/s)

t_full(e)=(sqrt(hmax+Ldrain(e))-sqrt(Ldrain(e)))*2*A0/(mu*A(e)*sqrt(2*g));
                    %neccessary time to empty a filled crest basin    (s)
```

```
%hydraulicBC
%_________________________________________________________________________
%_________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%_________________________________________________________________________
%_________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%_________________________________________________________________________
%_________________________________________________________________________
L(e)                    =   Hm0(e)/s0(e);
                            %wave length                            (m)

T(e)                    =   sqrt(2*pi*L(e)/g);
                            %wave period                            (s)

t_storm(e)              =   NumberOfWaves*T(e);
                            %duration of storm                      (s)

n(e)                    =   round(t_storm(e)/T(e));
                            %number of waves                        (-)

ksi(e)                  =   slope/sqrt(s0(e));
                            %irribarren parameter                   (-)

z2(e)                   =   1.75*Hm0(e)*g_berm*g_friction*g_beta*ksi(e);
                            %wave run up height exceeded by 2% of   (m)
                            %the waves

Pov(e)                  =   exp(-(sqrt(-log(0.02))*Rc(e)/z2(e))^2);
                            %probability on overtopping per wave    (-)

a(e)                    =   0.84*T(e)*q(e)/Pov(e);
                            %scalefactor of Weibull distribution    (m2)
```

```
%WeibullDistributedWaves
%___________________________________________________________________________
%___________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%___________________________________________________________________________
%___________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%___________________________________________________________________________
%___________________________________________________________________________
t(e,1)                      =   0;
Vw(e,1)                     =   0;
Vcumwave(e,1)               =   0;
Nov(e,1)                    =   0;

for i=2:n(e)
    t(e,i)=i*T(e);
    overtopping(e,i)        =   rand(1,1)<=Pov(e);

    %determine whether overtopping takes place or not. no overtopping=0
    %overtopping=1
    if overtopping(e,i)     ==  0;
        Vw(e,i)             =   0;
        Nov(e,i)            =   Nov(e,i-1);
    else
        Vw(e,i)             =   WeibullRNG(a(e),0.75,1);
                                %Creation of Weibull distributed volumes
        Nov(e,i)            =   Nov(e,i-1)+1;
                                %Counting of number of overtopping waves
    end;
    Vcumwave(e,i)           =   Vcumwave(e,i-1)+Vw(e,i);
                                %Counting of the incoming wave volume
end;
q_check(br,e)               =   Vcumwave(e,i)/t_storm(e);
                                %this can be compared with the input..
                                %mean overtopping discharge
Povcheck(e)                 =   Nov(e,n(e))/n(e);
                                %This can be compared with the...
                                %theoretical Pov
```

```
%WeibullRNG
%____________________________________________________________________________
%____________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%____________________________________________________________________________
%____________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%____________________________________________________________________________
%____________________________________________________________________________
%This file is based on a file created by David Vannucci (1 june 2003)

function [WeibullRandomNumbers]=WeibullRNG(scale,shape,noOfRandomNumbers)

WeibullRandomNumbers = scale.*( -log(1-rand(noOfRandomNumbers,1))).^(1/shape);
```

```
%Overtopping process
%___________________________________________________________________________
%___________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%___________________________________________________________________________
%___________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%___________________________________________________________________________
%___________________________________________________________________________

Vcrest(e,1)                 =   Vw(e,1);
Vcumwave(e,1)               =   Vw(e,1);
Vcumovertop(e,1)            =   0;
Vcumdrain(e,1)              =   0;
t(e,1)                      =   0;

Vovertopping(e,1)=0;
Vcrestend(e,1)=0;
for i=2:n(e)
    Vdrain(e,i)             =   0;
    t(e,i)                  =   i*T(e);
    Vcreststart(e,i)        =   Vcrestend(e,i-1)+Vw(e,i);

    %determine whether overtopping takes place or not
    if Vcreststart(e,i)>Vcrestmax
        Vovertopping(e,i)   =   Vcreststart(e,i)- Vcrestmax;
        Vcreststart(e,i)    =   Vcrestmax;
    else
        Vovertopping(e,i)=0;
    end;

    %determine whether there is water in the crest basin
    if Vcreststart(e,i)>0;
        hcrest(e,i)         =   Vcreststart(e,i)/Widthcrest;
        c(e,i)              =   Ldrain(e)/hcrest(e,i);
        Theta(e,i)          =   (1/(4*(sqrt(c(e,i)+1)-sqrt(c(e,i)))^2))-c(e,i);
        maxdrainvolume(e,i) =
sqrt(2*g*(Theta(e,i)*hcrest(e,i)+Ldrain(e)))*mu(e)*A(e)/Distdrains(e)*T(e);

        %Determine if the crest basin is drained totally
        if maxdrainvolume(e,i)>Vcreststart(e,i)
           Vcrestend(e,i)       =   0;
           Vdrain(e,i)          =   Vcreststart(e,i);
        else
           Vdrain(e,i)          =   maxdrainvolume(e,i);
           Vcrestend(e,i)       =   Vcreststart(e,i)-Vdrain(e,i);
        end
    else
       Vdrain(e,i)          =   0;
       Vcrestend(e,i)       =   0;
       hcrest(e,i)          =   0;
    end

    %Determine the cumulative quantities
    Vcumdrain(e,i)              =   Vcumdrain(e,i-1)+Vdrain(e,i);
    Vcumovertop(e,i)            =   Vcumovertop(e,i-1)+Vovertopping(e,i);
    Vcumwave(e,i)               =   Vw(e,i)+Vcumwave(e,i-1);
end;
```

```
%Results
%_________________________________________________________________________
%_________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%_________________________________________________________________________
%_________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%_________________________________________________________________________
%_________________________________________________________________________

error                               =    ((Vcumwave(n)-Vcumovertop(n)-Vcumdrain(n)-
Vcrestend(n))/Vcumwave(n))*100;

NrOfWavesDuringEmptying(e)          =    t_full(e)/T(e);

percentageOvertopping(br,e)         =
 Vcumovertop(e,n(e))/(Vcumovertop(e,n(e))+Vcumdrain(e,n(e))+Vcrestend(e,n(e)))*100;

q_overtopmean(br,e)                 =    Vcumovertop(e,n(e))/(n*T(e))

q_drainmean(br,e)                   =    Vcumdrain(e,n(e))/(n*T(e))

q_totalovertopmean(br,e)            =    Vcumwave(e,n(e))/(n*T(e))
```

```
%plotoptions
%_______________________________________________________________________________
%_______________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%_______________________________________________________________________________
%_______________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%_______________________________________________________________________________
%_______________________________________________________________________________

V2crestmax  =   Vcrestmax*2;
                %this is the buffer capacity with a length spreading effect
                %of two

%plot(t,Vw,'.',t,Vcrestmax,'r',t,V2crestmax)
%xlabel('t(s)'); ylabel('V_t_o_t_a_l_o_v_e_r_t_o_p_p_i_n_g(m^2)'); title('');
%legend('Volume per wave','buffer capacity length spreading effect =1','buffer capacity
length spreading effect=2');
%%this plot option represents only the incoming wave

%cummulative(t,Vcumwave,Vcumovertop,Vcumdrain)
%%this plot option represents the cummulative wave overtopping

%plotprocess(t, Vw, Vcrestmax,Vcreststart,Vovertopping,Vcrestend,Vdrain,hcrest,hmax,
V2crestmax);
%%this plot options represents the process of overtopping

%plotcomparison(q_overtoppingmeasured,q,q_crestmeasured,testpercentage,percentageCrest,
q_drainedmean,q_check,q_overtoppingmean);
%volume(Vw, Vcrestmax);
%%this plot option compares the physical experiments with the numerical
%%experiments
```

```
%plotprocess
%____________________________________________________________________________
%____________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%____________________________________________________________________________
%____________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section hydraulic Engineering
%____________________________________________________________________________
%____________________________________________________________________________

function plotprocess(t, Vw,
Vcrestmax,Vcreststart,Vovertopping,Vcrestend,Vdrain,hcrest,hmax, V2crestmax);

title('test');
subplot(2,2,1)
plot(t,Vw,'.',t,Vcrestmax,'r', t,V2crestmax,'g')
xlabel('t (s)'); ylabel('V_t_o_t_a_l_o_v_e_r_t_o_p_p_i_n_g (m^2)'); title('');

subplot(2,2,2)
plot(t,Vcreststart,'.',t,Vcrestmax,'r')
xlabel('t (s)'); ylabel('V_c_r_e_s_t_s_t_a_r_t (m^2)'); title('');

subplot(2,2,3)
plot(t,Vovertopping,'.')
xlabel('t (s)'); ylabel('V_o_v_e_r_t_o_p_p_i_n_g (m^2)'); title('');

subplot(2,2,4)
plot(t,Vdrain,'.')
xlabel('t (s)'); ylabel('V_d_r_a_i_n (m^2)'); title('');
```

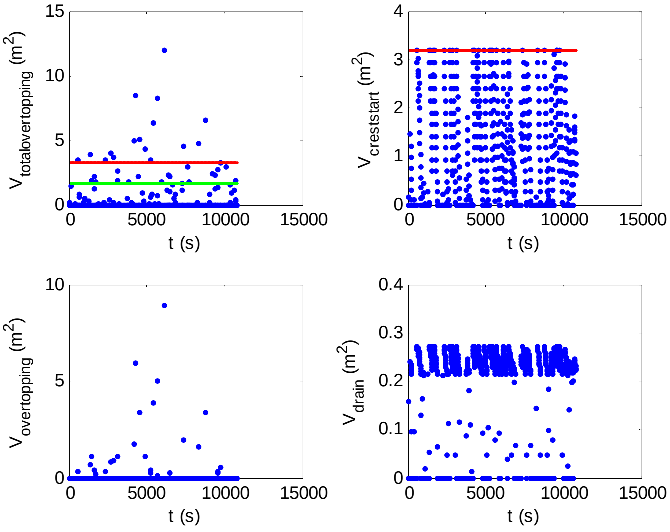
```
%Cumulative
%________________________________________________________________________
%________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%________________________________________________________________________
%________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%________________________________________________________________________
%________________________________________________________________________
function cummulative(t,Vcumwave,Vcumovertop,Vcumdrain);

subplot(2,2,1);
hold
plot(t,Vcumwave)
hold
xlabel('t(s)'); ylabel('Vtotalwave (m^2)'); title('');

subplot(2,2,2)
plot(t,Vcumovertop)
hold
xlabel('t(s)'); ylabel('Vtotalovertopping(m^2)'); title('');

subplot(2,2,3)
plot(t,Vcumdrain)
hold
xlabel('t(s)'); ylabel('Vtotaldrained(m^2)'); title('');
```

```
%plotcomparison
%________________________________________________________________________
%________________________________________________________________________
%Overtopping Discharges of a Crest Drainage Dike
%version 1.0
%P. van Steeg, April 2007
%________________________________________________________________________
%________________________________________________________________________
%Delft University of Technology
%Faculty of Civil Engineering and Geosciences
%Section Hydraulic Engineering
%________________________________________________________________________
%________________________________________________________________________
function
plotcomparison(q_overtoppingmeasured,q,q_crestmeasured,testpercentage,percentageCrest,q
_drainedmean,q_check,q_overtoppingmean);

x_1max=max(percentageCrest);
x_1=[0:x_1max/100,x_1max];
y_1=x_1
subplot(2,2,1);
hold
plot(testpercentage,percentageCrest,'.',x_1,y_1);
hold
xlabel('physical'); ylabel('numerical');title('percentage crest');
hold

x_2max=max(q_drainedmean);
x_2=[0:x_2max/100,x_2max];
y_2=x_2
subplot(2,2,2);
hold
plot(q_crestmeasured,q_drainedmean,'.',x_2,y_2);
hold
xlabel('physical tests');ylabel('numerical tests');title('q_d_r_a_i_n_e_d');
hold

x_3max=max(q_check);
x_3=[0:x_3max/100,x_3max];
y_3=x_3;
subplot(2,2,3);
plot(q,q_check,'.',x_3,y_3);
hold
xlabel('physical tests');ylabel('numerical
tests');title('q_t_o_t_a_l_o_v_e_r_t_o_p_p_i_n_g');
hold

x_4max=max(q_overtoppingmean);
x_4=[0:x_4max/100,x_4max];
y_4=x_4;
subplot(2,2,4);
hold
plot(q_overtoppingmeasured,q_overtoppingmean,'.',x_4,y_4);
hold
xlabel('physical tests');ylabel('numerical tests');title('q_o_v_e_r_t_o_p_p_i_n_g');
hold
%grid;
```

VI. Manual for the numerical program

Manual numerical program Crest Drainage Dike

P. van Steeg
April 2007



Introduction
This is the manual of the numerical program that determines the wave overtopping discharge of the Crest Drainage Dike. Goal of this manual is to explain how the program can be used. For an understanding of how the program is designed, one is referred to [van Steeg, 2007].

The program is written in Matlab. Therefore, Matlab is needed to use this program. For using statistical analysis, it is preferred to use Matlab version 7. If a version is used that does not contain a statistical toolbox, the program can still be used. However, for the statistical analysis one should use another program. (Or a different routine in Matlab. This however is not discussed in this manual)

The program and the underlying theories are not totally completed. Therefore, it is stressed that the outcomes should only be regarded as an indication. Some of the processes are based on assumptions that have not been verified yet. For a description, one is referred to [van Steeg, 2007].

Description of the program
The most important concept of the program is to calculate the wave overtopping discharge of a Crest Drainage Dike. Since the created wave volumes are generated from a distribution function, the possibility exists to repeat the program several times and execute statistical analysis with the obtained numerical dataset. Besides this it is also possible to use the program regarding a traditional dike with a simple geometry.

The program uses several input files where the geometric and hydraulic boundary conditions can be filled in.

To get a good feeling with the physical process, it is possible to plot the physical processes. The outputs of the program are several graphs were the incoming, drained and overtopping discharges are plotted as a function of the time.

Files and programs that are needed
Two types of files are needed to run the program. These are the program files (*.m) and the input files (*.txt) All files have to be placed in the working folder of Matlab. The overage overtopping of a traditional dike is required. This can be calculated with the use of PC Overslag or simple design rules such as described in [Besley, 1999] or [van der Meer, 2002]

The program files are

- Crestdrainagedike.m
- Loadvariables.m
- GeometricBC.m
- HydraulicBC.m
- WeibullDistributedWaves.m
- WeibullRNG.m
- OvertoppingProcess.m
- Results.m
- Plotoptions.m
- Plotprocess.m
- PlotComparison.m
- Cumulative.m

The input files are

- general.txt
- layoutdike.txt

- layoutcrestbasin.txt
- hydraulics.txt
- physicaltests.txt

Input data
The program uses several input files. How to use the input files is discussed below.

General
In the input file "general" several options are given regarding the preferences of the calculation.
An example of the input file "general" is shown in figure A-14

%	general.txt	
%		
0	%Statistical analysis?	(1=yes, 0=no)
1000	%nr of repetitions	(1 for no statistical analysis)
0	%comparions with physical model?	(1=yes, 0=no)"
1	%nr of physical tests to compare with	(1 if no comparisons)
0	%calculate z2%? (only for simple structures!!)	(1=yes, 0=no)

figure A-14: example of input file "general"

Statistical analysis	If a statistical analysis is desired enter the value "1" In case no statistical analysis is desired enter the value "0".
nr of repetitions	Enter the number of desired repetition of the program. If no statistical analysis is required, enter the value "1". For statistical analysis it is advised to use 100 or 1000 runs.
Comparison with physical model	Enter the value "0" if no comparison is desired.
Nr of physical tests to compare with	Enter the value "1" if no comparisons are desired.
Calculate z2%	The program can only calculated the z2% for simple structures. Therefore it is optional to calculate the z2% with this program or enter this manually (in another input file). In case you want a calculated z2%, enter the value "1". If you prefer to enter the z2% manually (e.g. after a calculation with PC Overslag), enter the value "0".

Hydraulics
In the input file "Hydraulics" it is possible to enter the hydraulic boundary conditions. See figure A-15 for an impression of this input file.

%	hydraulics.txt			
%				
3.3	%	Hm0	(significant wave height)	(meters)
0.01	%	s0	(wave steepness)	(-)
0.015	%	q	(average wave overtopping)	(m2/s)
3	%	t_hours	(Duration if storm)	(hours)
9.81	%	g	(acceleration due to gravity)	(m/s2)
9.03	%	z2	(z2%)	(meters)

figure A-15: example of input file "Hydraulics"

It is assumed that the input parameters are obvious. The q and z2% should be calculated with the program PC Overslag. It is preferred that the z2% is calculated with the program PC Overslag, since the numerical program can only calculate the z2% for simple geometries. (see also the input file "general")

Layoutdike
In the input file "layoutdike" , the value regarding the geometric parameters of the dike can be entered. An example of this input file is given in figure A-16.

%	layoutdike.txt			
%				
0.25	%	slope	"(outer slope of the dike, tan(a))"	(-)
6.9	%	Rc	(crest height)	(meters)
1	%	g_berm	(influence factor for berms)	(-)
1	%	g_friction	(influence factor for friction)	(-)
1	%	g_beta	(influence factor for angle of wave attack)	(-)

figure A-16: example of input file "layoutdike"

It is stressed that the program cannot combine several reduction parameters. The reduction parameters are only use to calculate the z2%. If the z2% is calculated manually, enter for the influence factors the value "1".

Layoutcrestbasin
In the input file "layoutcrestbasin" , the values regarding the layout of the crest basin can be entered. An example of this input file is shown in figure A-17.

%	layoutcrestbasin.txt			
%				
0.4	%	Ddrain	(Diameter of the drain)	(meter)
1	%	Ldrain	(Length of the drains)	(meter)
35	%	Distdrains	(Distance between two drains)	(meter)
0.8	%	Entrancefriction	(entrancefriction)	(-)
2	%	Widthcrest	(Width of the crest basin)	(meter)
1	%	heigthcrest	(Height of the crest basin)	(meter)
0.8	%	Efficiencycrest	(Efficiency of the crest basin)	(meter)
2	%	LengthSpreadingEffect	(LengthSpreadingEffect)	(-)

figure A-17: example of input file "layoutcrestbasin"

Physicaltests
The input file "physicaltests" can be used in case there is a need to compare the physical experiments with the numerical experiments. In this input file, the measured values can be entered. An impression is given in figure A-18

%	physicaltests.txt			
%				
50	%	testpercentage	(percentage overtopping according to physical experiments)	(%)
0.01	%	q_crestmeasured	(measured discharge trapped in the crest basin)	(m2/s)
0.01	%	q_overtoppingmeasured	(measured overtopping discharge)	(m2/s)

figure A-18: example of input file "Physicaltest"

Run the program
To run the program, make sure that all the input files have been filled in correctly and type the command "CrestDrainageDike" in the Matlab command.

Output
Three output values are given in the command line. These values are respectively the overtopping, drained and total discharges. It is stressed that, due to the fact that a random number generator is used in the program, two runs might provide different answers. Several other parameters can be found in the workspace of Matlab. Depending on the input file "general", process-based plots are displayed. (A plot is shown in case there are no repetitions and no comparisons).

VII. Matlab code: Calculation spectral periods

The program Auke gives the possibility to analyze the measured record. The mean wave period and the peak period are calculated. However, the interest lies in the $T_{m-1.0}$ and this parameter cannot be determined with the program. Therefore, a method to analyze the given wave spectrum is identified and a Matlab code is written. Using this code, the $T_{m-1.0}$ can be determined.

```
% P. van Steeg
% november 2006
% This programma calculates the mo, the m-1 and the Tm-1,0
% Before this program is used, there is a need to load
% the wave spectra in the following layout:
%
% column 1: frequencies
% column 2: variance density E(f)

f=data(:,1);
E=data(:,2);
for i=1:length(E)-1
   deltaf(i)=f(i+1)-f(i);
   deltam0(i)=(f(i+1)-f(i))*E(i);
   deltam1(i)=((f(i+1)-f(i))*E(i))/f(i);
end;
plot(f,E,'*')
xlabel('f (Hz)')
ylabel('E(f) (m2/Hz)')

m0=sum(deltam0)
m1=sum(deltam1)
Tm=m1/m0
```

VIII. Overview test series

The results of the several executed tests are given in this appendix. To give a good overview, the input, measured and calculated values are shown in the same tables as the hydraulic and geometric boundary conditions. All the input files, measured wave records and output files are stored at the supplementary DVD.

Hin	Significant wave height as used in the input file
Hout	Significant wave height (measured value)
Tm-1.0	Spectral wave period (calculated from the spectrum)
Tp	Peak wave period
Tp/T	Factor between peak period (Tp) and spectral wave period (Tm-1.0)
q_crest	measured discharge through the crest/ drain
q_over	measured discharged which is overtopping
q_total	total discharge
Rc	crest freeboard
Hi/Ho	Factor between input wave height and measured wave height
s	Wave steepness
ksi	Irribarren parameter
R*	Dimensionless crest freeboard according to v.d. Meer
Q*total	Dimensionless total overtopping according to v.d. Meer
Overt	percentage of overtopping water

Subset A: Basic parameters Crest Drainage Dike

Diameter drain	0.01m
Length drain	0.10m
number of drains	2

	test	Hin	Hout	Tm-1,0	Tp	Tp/T	q_crest	q_over	q_totaal	Rc	Hi/Ho	s	ksi	R*	Q*total	overt
		(m)	(m)	(s)	(s)	(-)	m3/s/m	m3/s/m	m3/s/m	(m)	(-)	(-)		(-)	(-)	(%)
A01	86	0.114	0.057	0.906	0.944	1.04	2.1E-05	0.00E+00	2.1E-05	0.085	2.00	0.044	1.19	1.260	2.1E-04	100%
A02	87	0.124	0.064	0.943	0.973	1.03	3.9E-05	9.39E-07	4.0E-05	0.085	1.94	0.046	1.17	1.146	3.4E-04	98%
A03	88	0.135	0.070	0.987	1.035	1.05	8.8E-05	4.52E-06	9.2E-05	0.085	1.94	0.046	1.17	1.046	6.8E-04	95%
A04	89	0.152	0.078	1.031	1.060	1.03	1.4E-04	2.13E-05	1.6E-04	0.085	1.95	0.047	1.15	0.946	1.0E-03	86%
A05	90	0.171	0.088	1.093	1.130	1.03	2.0E-04	1.17E-04	3.2E-04	0.085	1.94	0.047	1.15	0.840	1.7E-03	64%
A06	91	0.194	0.102	1.164	1.187	1.02	2.4E-04	4.19E-04	6.6E-04	0.085	1.90	0.048	1.14	0.733	2.8E-03	36%
A07	92	0.209	0.111	1.210	1.220	1.01	2.5E-04	6.65E-04	9.1E-04	0.085	1.88	0.049	1.13	0.676	3.5E-03	27%

table A-3: Overview Subset A: basic parameters Crest Drainage Dike

Subset B: Traditional dike

Draintype none

	test	Hin	Hout	Tm-1,0	Tp	Tp/T	q_crest	q_over	q_totaal	Rc	Hi/Ho	s	ksi	R*	Q*total	overt
		(m)	(m)	(s)	(s)	(-)	m3/s/m	m3/s/m	m3/s/m	(m)	(-)	(-)		(-)	(-)	(%)
B01	183	0.114	0.057	0.905	0.945	1.04	none	none	1.2E-05	0.087	1.99	0.045	1.18	1.284	1.2E-04	none
B02	186	0.124	0.063	0.941	0.974	1.03	none	none	2.8E-05	0.087	1.96	0.046	1.17	1.179	2.4E-04	none
B03	56	0.133	0.069	0.972	1.028	1.06	none	none	6.8E-05	0.087	1.94	0.047	1.16	1.093	5.2E-04	none
B04	57	0.150	0.079	1.038	1.104	1.06	none	none	1.6E-04	0.087	1.91	0.047	1.16	0.957	9.9E-04	none
B05	184	0.171	0.090	1.097	1.131	1.03	none	none	2.8E-04	0.087	1.91	0.048	1.15	0.848	1.4E-03	none

table A-4: Overview subset B: traditional dike

Subset C: Drain influence

Draintype I
Diameter drain variable
Length drain 0.10m
number of drains 1

	test	Hin	Hout	Tm-1,0	Tp	Tp/T	q_crest	q_over	q_totaal	Rc	Hi/Ho	s	ksi	R*	Q*total	overt
		(m)	(m)	(s)	(s)	(-)	m3/s/m	m3/s/m	m3/s/m	(m)	(-)	(-)		(-)	(-)	(%)
Diameter Drain = 0.01m																
C01	60	0.170	0.089	1.087	1.134	1.043	7.4E-05	1.76E-04	2.5E-04	0.085	1.91	0.048	1.14	0.840	9.3E-04	30%
C02	63	0.133	0.068	0.971	1.014	1.044	4.5E-05	2.12E-05	6.6E-05	0.085	1.96	0.046	1.16	1.076	1.6E-04	68%
Diameter Drain = 0.02m																
C03	61	0.170	0.089	1.088	1.147	1.054	1.6E-04	1.19E-04	2.8E-04	0.085	1.90	0.048	1.14	0.837	6.3E-04	57%
C04	62	0.170	0.089	1.089	1.130	1.038	1.8E-04	9.81E-05	2.8E-04	0.085	1.91	0.048	1.14	0.839	5.2E-04	65%
C05	64	0.133	0.068	0.971	1.013	1.043	7.2E-05	3.92E-06	7.6E-05	0.085	1.96	0.046	1.16	1.075	3.0E-05	95%
C06	66	0.169	0.087	1.079	1.127	1.045	1.7E-04	9.06E-05	2.6E-04	0.085	1.93	0.048	1.14	0.854	4.9E-04	66%
C07	67	0.200	0.102	1.127	1.165	1.034	2.0E-04	2.37E-04	4.4E-04	0.085	1.97	0.051	1.10	0.758	1.1E-03	46%
Diameter Drain = 0.03m																
C08	51	0.170	0.089	1.090	1.134	1.040	2.4E-04	6.37E-05	3.0E-04	0.085	1.91	0.048	1.14	0.837	3.4E-04	79%
C09	52	0.199	0.106	1.169	1.210	1.035	3.8E-04	1.85E-04	5.6E-04	0.085	1.88	0.050	1.12	0.716	7.6E-04	67%
C10	53	0.084	0.040	0.783	0.784	1.001				0.085	2.13	0.041	1.23	1.750		
C11	54	0.117	0.060	0.914	0.953	1.042	3.1E-05	1.25E-06	3.2E-05	0.085	1.96	0.046	1.17	1.218	1.2E-05	96%
C12	55	0.133	0.068	0.970	1.018	1.049	7.9E-05	4.31E-06	8.3E-05	0.085	1.95	0.046	1.16	1.075	3.3E-05	95%
C13	65	0.133	0.069	0.973	1.024	1.053	7.8E-05	3.82E-06	8.1E-05	0.085	1.93	0.047	1.16	1.067	2.9E-05	95%
C14	68	0.19	0.084	1.006	1.022	1.016	1.6E-04	1.47E-05	1.7E-04	0.085	2.29	0.053	1.08	0.933	8.9E-05	91%
C15	69	0.242	0.095	1.048	1.051	1.003	2.5E-04	5.00E-05	3.0E-04	0.085	2.55	0.055	1.06	0.844	2.6E-04	83%

table A-5: Overview subset C: Drain influence (draintype I)

Draintype II
Diameter drain variable
Length drain 0.10m
number of drains 2

	test	Hin	Hout	Tm-1,0	Tp	Tp/T	q_crest	q_over	q_totaal	Rc	Hi/Ho	s	ksi	R*	Q*total	overt
		(m)	(m)	(s)	(s)	(-)	m3/s/m	m3/s/m	m3/s/m	(m)	(-)	(-)		(-)	(-)	(%)
Diameter Drain = 0.01m, length drain =0.05m																
C16	188	0.124	0.063	0.942	0.974	1.034	4.66E-05	1.02E-06	4.8E-05	0.085	1.96	0.045	1.17	1.151	8.8E-06	98%
C17	187	0.135	0.069	0.985	1.023	1.039	8.50E-05	4.93E-06	9.0E-05	0.085	1.95	0.046	1.17	1.050	3.7E-05	95%
C18	118	0.152	0.078	1.031	1.059	1.027	1.3E-04	2.95E-05	1.6E-04	0.085	1.96	0.047	1.16	0.948	1.9E-04	82%
C19	119	0.171	0.089	1.096	1.130	1.031	1.8E-04	1.40E-04	3.2E-04	0.085	1.92	0.047	1.15	0.833	7.3E-04	57%
C20	120	0.194	0.101	1.156	1.161	1.004	2.0E-04	3.48E-04	5.5E-04	0.085	1.92	0.049	1.13	0.740	1.5E-03	37%
C21	121	0.209	0.111	1.210	1.220	1.008	2.09E-04	6.52E-04	8.6E-04	0.085	1.88	0.049	1.13	0.676	2.5E-03	24%
Diameter Drain = 0.02m																
C22	78	0.114	0.057	0.906	0.942	1.040	2.1E-05	0.00E+00	2.1E-05	0.085	2.00	0.045	1.18	1.258		100%
C23	79	0.124	0.062	0.942	0.973	1.033	6.3E-05	0.00E+00	6.3E-05	0.085	1.98	0.045	1.18	1.159		100%
C24	80	0.135	0.068	0.975	1.013	1.039	7.9E-05	4.41E-06	8.4E-05	0.085	1.97	0.046	1.17	1.068	3.4E-05	95%
C25	80a	0.135	0.069	0.984	1.026	1.043	1.0E-04	7.47E-06	1.1E-04	0.085	1.96	0.046	1.17	1.054	5.6E-05	93%
C26	82	0.152	0.077	1.031	1.059	1.027	1.5E-04	9.35E-06	1.6E-04	0.085	1.97	0.047	1.16	0.950	6.0E-05	94%
C27	81	0.152	0.077	1.031	1.059	1.028	1.6E-04	1.02E-05	1.7E-04	0.085	1.96	0.047	1.16	0.950	6.5E-05	94%
C28	83	0.169	0.089	1.097	1.113	1.015	3.0E-04	5.00E-05	3.5E-04	0.085	1.90	0.047	1.15	0.833	2.6E-04	86%
C29	84	0.194	0.102	1.159	1.172	1.011	4.8E-04	1.81E-04	6.6E-04	0.085	1.90	0.049	1.13	0.736	7.8E-04	72%
C30	85	0.209	0.110	1.211	1.230	1.016	6.1E-04	3.43E-04	9.5E-04	0.085	1.89	0.048	1.14	0.677	1.3E-03	64%
Diameter Drain = 0.03m, length drain =0.10m																
C31	70	0.105	0.052	0.873	0.904	1.035	9.1E-06	0.00E+00	9.1E-06	0.085	2.02	0.044	1.19	1.367	0.0E+00	100%
C32	71	0.114	0.057	0.904	0.945	1.045	1.9E-05	0.00E+00	1.9E-05	0.085	2.00	0.045	1.18	1.262	0.0E+00	100%
C33	72	0.124	0.062	0.938	0.985	1.050	4.3E-05	0.00E+00	4.3E-05	0.085	1.99	0.045	1.18	1.166	0.0E+00	100%
C34	73a	0.135	0.068	0.977	1.015	1.039	8.2E-05	2.82E-06	8.5E-05	0.085	1.98	0.046	1.17	1.068	2.2E-05	97%
C35	74	0.152	0.078	1.029	1.060	1.030	1.5E-04	5.13E-06	1.5E-04	0.085	1.96	0.047	1.15	0.951	3.3E-05	97%
C36	75	0.171	0.088	1.100	1.130	1.027	2.6E-04	3.33E-05	3.0E-04	0.085	1.94	0.047	1.16	0.835	1.8E-04	89%
C37	75a	0.171	0.088	1.093	1.130	1.034	3.1E-04	3.41E-05	3.4E-04	0.085	1.94	0.047	1.15	0.839	1.8E-04	90%
C38	76	0.194	0.103	1.165	1.173	1.007	5.5E-04	1.24E-04	6.8E-04	0.085	1.89	0.048	1.14	0.730	5.3E-04	82%
C39	77	0.209	0.110	1.212	1.250	1.031	7.1E-04	2.38E-04	9.5E-04	0.085	1.90	0.048	1.14	0.678	9.1E-04	75%

table A-6: Overview subset C: Drain influence (draintype II)

Subset D: Influence crest freeboard

Draintype	II
Diameter drain	0.01m
Length drain	0.10m
number of drains	2

	test	Hin	Hout	Tm-1,0	Tp	Tp/T	q_crest	q_over	q_totaal	Rc	Hi/Ho	s	ksi	R*	Q*total	overt
		(m)	(m)	(s)	(s)	(-)	m3/s/m	m3/s/m	m3/s/m	(m)	(-)	(-)		(-)	(-)	(%)
D01	99	0.057	0.023	0.652	0.660	1.012	0.0E+00	0.00E+00	0.0E+00	0.045	2.52	0.034	1.35	1.473	0.0E+00	100%
D02	100	0.088	0.036	0.725	0.750	1.034	1.8E-05	0.00E+00	1.8E-05	0.045	2.44	0.044	1.20	1.052	3.5E-04	100%
D03	101	0.100	0.043	0.770	0.780	1.013	5.4E-05	0.00E+00	5.4E-05	0.045	2.35	0.046	1.17	0.909	8.4E-04	100%
D04	102	0.110	0.050	0.836	0.876	1.048	1.4E-04	4.55E-06	1.4E-04	0.045	2.20	0.046	1.17	0.772	1.7E-03	97%
D05	103	0.120	0.059	0.903	0.936	1.037	2.1E-04	6.94E-05	2.8E-04	0.045	2.03	0.046	1.16	0.657	2.7E-03	75%
D06	107	0.125	0.063	0.923	0.956	1.037	2.3E-04	1.62E-04	3.9E-04	0.045	1.98	0.047	1.15	0.623	3.4E-03	59%
D07	105	0.130	0.065	0.941	0.977	1.039	2.3E-04	2.21E-04	4.6E-04	0.045	2.00	0.047	1.15	0.602	3.8E-03	51%
D08	104	0.140	0.071	0.977	0.987	1.010	2.6E-04	4.17E-04	6.8E-04	0.045	1.98	0.047	1.15	0.556	5.0E-03	39%
D09	106	0.150	0.076	1.016	1.047	1.031	2.5E-04	6.97E-04	9.5E-04	0.045	1.97	0.047	1.15	0.515	6.2E-03	26%
D10	98	0.140	0.070	0.981	1.010	1.030	9.5E-06	0.00E+00	9.5E-06	0.125	2.00	0.047	1.16	1.543	7.1E-05	100%
D11	93	0.160	0.080	1.038	1.060	1.021	3.3E-05	9.70E-07	3.4E-05	0.125	2.00	0.048	1.15	1.364	2.1E-04	97%
D12	94	0.180	0.091	1.093	1.120	1.025	6.5E-05	4.15E-06	6.9E-05	0.125	1.98	0.049	1.13	1.215	3.5E-04	94%
D13	95	0.200	0.104	1.160	1.175	1.013	1.4E-04	2.87E-05	1.7E-04	0.125	1.93	0.049	1.13	1.073	7.2E-04	83%
D14	96	0.220	0.114	1.209	1.220	1.009	1.9E-04	1.20E-04	3.1E-04	0.125	1.93	0.050	1.12	0.981	1.2E-03	62%

table A-7: overview subset D: influence crest freeboard

Subset E: Influence wave steepness

Draintype	II
Diameter drain	0.01m
Length drain	0.10m
number of drains	2

	test	Hin	Hout	Tm-1,0	Tp	Tp/T	q_crest	q_over	q_totaal	Rc	Hi/Ho	s	ksi	R*	Q*total	overt
		(m)	(m)	(s)	(s)	(-)	m3/s/m	m3/s/m	m3/s/m	(m)	(-)	(-)		(-)	(-)	(%)
E01	108	0.077	0.041	0.917	0.948	1.034	2.2E-06	0.00E+00	0.0E+00	0.085	1.878	0.031	1.42	1.467	0.0E+00	
E02	109	0.087	0.048	0.971	1.019	1.049	1.1E-05	0.00E+00	1.1E-05	0.085	1.808	0.033	1.39	1.282	1.2E-04	100%
E03	110	0.096	0.053	1.014	1.079	1.065	3.1E-05	4.17E-07	3.1E-05	0.085	1.827	0.033	1.38	1.171	3.0E-04	99%
E04	111	0.106	0.059	1.065	1.140	1.070	6.7E-05	9.05E-07	6.8E-05	0.085	1.795	0.033	1.37	1.053	5.5E-04	99%
E05	112	0.116	0.065	1.110	1.177	1.060	1.1E-04	6.97E-06	1.2E-04	0.085	1.789	0.034	1.37	0.966	8.6E-04	94%
E06	113	0.125	0.070	1.157	1.271	1.099	1.7E-04	4.26E-05	2.1E-04	0.085	1.787	0.034	1.37	0.890	1.3E-03	80%
E07	114	0.135	0.076	1.175	1.273	1.083	1.9E-04	1.05E-04	3.0E-04	0.085	1.782	0.035	1.33	0.843	1.7E-03	65%
E08	115	0.144	0.083	1.218	1.320	1.084	2.2E-04	2.14E-04	4.4E-04	0.085	1.746	0.036	1.32	0.778	2.2E-03	51%
E09	116	0.154	0.088	1.253	1.366	1.090	2.3E-04	3.62E-04	6.0E-04	0.085	1.746	0.036	1.32	0.732	2.8E-03	39%
E10	117	0.164	0.094	1.294	1.394	1.077	2.5E-04	5.06E-04	7.5E-04	0.085	1.748	0.036	1.32	0.687	3.2E-03	33%

table A-8: Overview subset E: influence wave steepness

Subset F: Influence wave spectrum

Draintype	II
Diameter drain	0.01m
Length drain	0.10m
number of drains	2

	test	Hin	Hout	Tm-1,0	Tp	Tp/T	q_crest	q_over	q_totaal	Rc	Hi/Ho	s	ksi	R*	Q*total	overt
		(m)	(m)	(s)	(s)	(-)	m3/s/m	m3/s/m	m3/s/m	(m)	(-)	(-)		(-)	(-)	(%)
Pierson-Moskowitz (gamma =1.0)																
F01	122	0.124	0.059	0.937	0.903	0.96	4.2E-05	6.65E-07	4.2E-05	0.085	2.10	0.043	1.21	1.198	3.9E-04	98%
F02	123	0.135	0.065	0.983	0.984	1.00	7.2E-05	3.84E-06	7.6E-05	0.085	2.06	0.043	1.20	1.084	6.0E-04	95%
F03	124	0.152	0.074	1.030	1.044	1.01	1.5E-04	1.75E-05	1.7E-04	0.085	2.05	0.045	1.18	0.970	1.1E-03	90%
F04	125	0.171	0.085	1.088	1.156	1.06	2.08E-04	1.01E-04	3.1E-04	0.085	2.02	0.046	1.17	0.860	1.7E-03	67%
F05	126	0.194	0.096	1.153	1.156	1.00	2.3E-04	2.71E-04	5.0E-04	0.085	2.01	0.046	1.16	0.761	2.3E-03	46%
F06	127	0.209	0.107	1.210	1.300	1.074	2.4E-04	6.50E-04	8.9E-04	0.085	1.96	0.047	1.16	0.689	3.5E-03	27%
Narrow spectrum Jonswap (gamma=20)																
F07	128	0.140	0.0737	1.006	1.021	1.01	9.16E-05	1.03E-05	1.0E-04	0.085	1.90	0.047	1.16	0.997	7.0E-04	90%
F08	129	0.154	0.0817	1.0405	1.0605	1.02	1.31E-04	4.32E-05	1.7E-04	0.085	1.88	0.048	1.14	0.916	1.0E-03	75%
F09	130	0.164	0.088	1.078	1.087	1.01	1.59E-04	9.54E-05	2.5E-04	0.085	1.86	0.048	1.14	0.852	1.4E-03	63%
F10	131	0.173	0.0935	1.1097	1.1	0.99	1.79E-04	1.69E-04	3.5E-04	0.085	1.85	0.049	1.13	0.803	1.7E-03	51%
F11	132	0.194	0.1054	1.1728	1.2	1.02	2.23E-04	3.78E-04	6.0E-04	0.085	1.84	0.049	1.13	0.715	2.5E-03	37%
F12	133	0.204	0.1146	1.227	1.265	1.03	2.41E-04	6.54E-04	9.0E-04	0.085	1.78	0.049	1.13	0.656	3.3E-03	27%
Extreme narrow Jonswap (gamma =100)																
F13	135	0.11	0.05594	0.899	0.92	1.02	2.06E-05	5.54E-07	2.1E-05	0.085	1.97	0.044	1.19	1.281	2.1E-04	97%
F14	136	0.128	0.0659	0.964	0.987	1.02	5.55E-05	5.91E-06	6.1E-05	0.085	1.94	0.045	1.17	1.101	4.9E-04	90%
F15	137	0.146	0.0776	1.0195	1.048	1.03	9.58E-05	3.18E-05	1.3E-04	0.085	1.88	0.048	1.14	0.959	8.2E-04	75%
F16	138	0.164	0.0898	1.0775	1.08	1.00	1.33E-04	1.20E-04	2.5E-04	0.085	1.83	0.050	1.12	0.843	1.3E-03	53%
F17	139	0.183	0.101	1.1355	1.152	1.01	1.61E-04	2.52E-04	4.1E-04	0.085	1.81	0.050	1.12	0.755	1.8E-03	39%
F18	140	0.21	0.123	1.226	1.23	1.00	2.34E-04	8.29E-04	1.1E-03	0.085	1.71	0.052	1.09	0.633	3.6E-03	22%
Double topped Jonswap (gamma=0.5)																
F19	148	0.14	0.06407	0.962	0.996	1.035	5.91E-05	1.78E-06	6.1E-05	0.085	2.19	0.044	1.19	1.118	5.0E-04	97%
F20	146	0.164	0.0769	1.0484	1.08	1.030	1.77E-04	3.80E-05	2.2E-04	0.085	2.13	0.045	1.18	0.937	1.4E-03	82%
F21	147	0.183	0.0877	1.106	1.14	1.031	2.18E-04	1.35E-04	3.5E-04	0.085	2.09	0.046	1.17	0.832	1.9E-03	62%
Double topped Jonswap (gamma=0.1)																
F22	152	0.138	0.0667	1.019	0.938	0.921	1.20E-04	8.10E-06	1.3E-04	0.085	2.07	0.041	1.23	1.035	9.6E-04	94%
F23	153	0.16	0.0766	1.066	1.011	0.948	1.90E-04	5.75E-05	2.5E-04	0.085	2.09	0.043	1.20	0.923	1.5E-03	77%

table A-9: Overview subset F: influence wave spectrum

Subset G: Influence berms

Draintype	II
Diameter drain	0.01m
Length drain	0.10m
number of drains	2

	test	Hin	Hout	Tm-1,0	Tp	Tp/T	q_crest	q_over	q_totaal	Rc	Hi/Ho	s	ksi	R*	Q*total	overt
		(m)	(m)	(s)	(s)	(-)	m3/s/m	m3/s/m	m3/s/m	(m)	(-)	(-)		(-)	(-)	(%)
B=3H, d=H																
G01	181	0.135	0.069	0.986	1.023	1.038	4.5E-05	1.56E-07	4.5E-05	0.085	1.96	0.045	1.17	1.051	3.4E-04	100%
G02	154	0.152	0.078	1.033	1.084	1.049	9.7E-05	3.56E-06	1.0E-04	0.085	1.95	0.047	1.16	0.944	6.3E-04	96%
G03	155	0.171	0.089	1.095	1.131	1.033	1.49E-04	1.37E-05	1.6E-04	0.085	1.92	0.048	1.15	0.834	8.5E-04	92%
G04	156	0.1938	0.1026	1.159	1.187	1.024	2.10E-04	8.73E-05	3.0E-04	0.085	1.89	0.049	1.13	0.734	1.3E-03	71%
B=3H, d=0																
G05	160	0.152	0.07793	1.031	1.059	1.026	4.50E-05		4.5E-05	0.085	1.95	0.047	1.15	0.946	2.9E-04	100%
G06	161	0.171	0.0889	1.091	1.130	1.035	1.04E-04	8.97E-06	1.1E-04	0.085	1.92	0.048	1.14	0.837	6.0E-04	92%
G07	162	0.1938	0.1018	1.153	1.215	1.054	1.65E-04	6.09E-05	2.3E-04	0.085	1.90	0.049	1.13	0.740	9.8E-04	73%
B=3H, d=-2/3H																
G08	178	0.135	0.0695	0.983	1.023	1.041	5.28E-05	7.69E-07	5.4E-05	0.085	1.94	0.046	1.17	1.051	4.0E-04	99%
G09	173	0.152	0.0783	1.033	1.059	1.025	1.19E-04	1.48E-05	1.3E-04	0.085	1.94	0.047	1.15	0.942	8.5E-04	89%
G10	176	0.162	0.0832	1.061	1.108	1.044	1.58E-04	3.62E-05	1.9E-04	0.085	1.95	0.047	1.15	0.890	1.1E-03	81%
G11	174	0.171	0.0897	1.096	1.130	1.031	1.87E-04	8.74E-05	2.7E-04	0.085	1.91	0.048	1.14	0.830	1.4E-03	68%
G12	175	0.1938	0.1028	1.167	1.200	1.028	2.34E-04	3.36E-04	5.7E-04	0.085	1.89	0.048	1.14	0.728	2.4E-03	41%

table A-10: Overview subset G: influence berm (s≈0.05)

Draintype II
Diameter drain 0.01m
Length drain 0.10m
number of drains 2

	test	Hin	Hout	Tm-1,0	Tp	Tp/T	q_crest	q_over	q_totaal	Rc	Hi/Ho	s	ksi	R*	Q*total	overt
		(m)	(m)	(s)	(s)	(-)	m3/s/m	m3/s/m	m3/s/m	(m)	(-)	(-)		(-)	(-)	(%)
B=3H, d=H																
G13	157	0.116	0.064	1.105	1.192	1.079	8.3E-05	2.22E-06	8.6E-05	0.085	1.80	0.034	1.36	0.972	6.2E-04	97%
G14	158	0.125	0.070	1.150	1.255	1.091	1.3E-04	8.56E-06	1.4E-04	0.085	1.79	0.034	1.36	0.895	8.7E-04	94%
G15	159	0.135	0.075	1.175	1.257	1.070	1.5E-04	1.52E-05	1.6E-04	0.085	1.80	0.035	1.34	0.846	9.5E-04	91%
G16	179a	0.144	0.081	1.214	1.294	1.066	1.9E-04	4.62E-05	2.3E-04	0.085	1.77	0.035	1.33	0.786	1.2E-03	80%
G17	180	0.154	0.087	1.250	1.364	1.091	2.2E-04	7.54E-05	3.0E-04	0.085	1.77	0.036	1.32	0.738	1.4E-03	74%
B=3H, d=0																
G18	163	0.12	0.064	1.109	1.185	1.068	2.2E-05	0.00E+00	2.2E-05	0.085	1.80	0.03	1.37	0.968	1.6E-04	100%
G19	164	0.13	0.071	1.145	1.225	1.070	5.2E-05	1.21E-06	5.3E-05	0.085	1.76	0.035	1.34	0.893	3.4E-04	98%
G20	165	0.14	0.076	1.165	1.257	1.079	7.4E-05	4.77E-06	7.9E-05	0.085	1.77	0.036	1.32	0.846	4.5E-04	94%
G21	166	0.14	0.082	1.208	1.294	1.071	1.1E-04	1.56E-05	1.3E-04	0.085	1.77	0.036	1.32	0.789	6.7E-04	88%
G22	167	0.15	0.088	1.245	1.333	1.070	1.6E-04	5.29E-05	2.1E-04	0.085	1.75	0.036	1.31	0.737	1.0E-03	75%
G23	168	0.16	0.094	1.279	1.390	1.087	1.8E-04	8.34E-05	2.6E-04	0.085	1.75	0.037	1.30	0.695	1.1E-03	68%
G24	182	0.18	0.107	1.407	1.558	1.107	2.0E-04	3.16E-04	5.2E-04	0.085	1.68	0.035	1.34	0.591	1.7E-03	39%
B=3H, d=-2/3H																
G25	177	0.106	0.059	1.058	1.139	1.077	4.4E-05	7.82E-07	4.5E-05	0.085	1.81	0.033	1.37	1.064	0.00	98%
G26	169	0.116	0.064	1.106	1.179	1.066	8.3E-05	3.86E-06	8.7E-05	0.085	1.81	0.034	1.36	0.973	0.00	96%
G27	170	0.125	0.070	1.151	1.255	1.090	1.5E-04	2.91E-05	1.7E-04	0.085	1.78	0.034	1.36	0.893	0.00	83%
G28	171	0.135	0.076	1.174	1.257	1.071	1.7E-04	5.39E-05	2.2E-04	0.085	1.78	0.035	1.33	0.842	0.00	76%
G30	172	0.140	0.083	1.216	1.294	1.064	2.1E-04	1.50E-04	3.6E-04	0.085	1.69	0.036	1.32	0.779	0.00	58%

table A-11: Overview subset G: Influence berms (s≈0.03)

Subset H: Solitary waves

test	H	waterlevel	mass box+water	mass box	mass water	filling cb	Vcb	overtopped	total	overtopped
	(m)	(cm)	(g)		(kg)	(%)	(m3)	(m3)		(%)
H01	0.09	0.314	2948	2882	0.066	80%	1.55E-03	6.60E-05	1.62E-03	4.1%
H02	0.09	0.311	2936	2882	0.054	73%	1.42E-03	5.40E-05	1.48E-03	3.7%
H03	0.09	0.306	2912	2882	0.03	56%	1.09E-03	3.00E-05	1.12E-03	2.7%
H04	0.09	0.304	2918	2882	0.036	50%	9.73E-04	3.60E-05	1.01E-03	3.6%
H05	0.10	0.314	2980	2876	0.104	81%	1.57E-03	1.04E-04	1.67E-03	6.2%
H06	0.10	0.316	3115	2876	0.239	88%	1.70E-03	2.39E-04	1.94E-03	12.3%
H07	0.10	0.320	3207	2876	0.331	100%	1.95E-03	3.31E-04	2.28E-03	14.5%
H08	0.10	0.320	3758	2876	0.882	100%	1.95E-03	8.82E-04	2.83E-03	31.2%
H09	0.10	0.316	3050	2901	0.149	86%	1.67E-03	1.49E-04	1.82E-03	8.2%
H10	0.10	0.317	3222	2901	0.321	91%	1.77E-03	3.21E-04	2.09E-03	15.4%
H11	0.10	0.312	3093	2901	0.192	75%	1.46E-03	1.92E-04	1.65E-03	11.6%
H12	0.10	0.318	3240	2920	0.32	93%	1.81E-03	3.20E-04	2.13E-03	15.0%
H13	0.10	0.319	3286	2921	0.365	97%	1.88E-03	3.65E-04	2.24E-03	16.3%
H14	0.10	0.319	3171	2949	0.222	97%	1.88E-03	2.22E-04	2.10E-03	10.6%
H15	0.10	0.320	3446	2934	0.512	100%	1.95E-03	5.12E-04	2.46E-03	20.8%
H16	0.10	0.320	3335	2938	0.397	100%	1.95E-03	3.97E-04	2.34E-03	16.9%

table A-12 overview subset 14: solitary waves

IX. Manual for using the data on the DVD

Introduction
The physical experiments that have been executed used specific input files. As a result of the experiments several output files are generated. These files have been stored. This paper is a brief manual for this stored data.

Contents
The experiments have been categorized. One is referred to the main report [van Steeg, 2007] for a detailed description of the categorized subsets. The input and output files have been stored in the same subsets. The following input and output files have been saved per experiment:

Input files:

These files contain several input parameters. The program Auke uses these input files.

Hireg00.XXX
These files are the steering files. A steering file is created with the file hireg00.pcf

Output files

Hdeep.asc
This file contains the raw measurements at "deep water". For an explanation, one is referred to the main report

Hteen.asc
This file contains the raw measurements at the toe of the dike. For an explanation, one is referred to the main report.

Output
This file contains the calculated wave heights as function of the time.

Spectrum
This file contains the wave spectrum of the measured wave record. This file has been used to calculate the $T_{m-1.0}$

Stats
This file contains the calculated output values of the measured wave record.

X. Calibration of the drains

To determine the maximum discharge capacity it is necessary to calibrate the different drains that have been used. The drainage system is schematized as shown in figure A-19

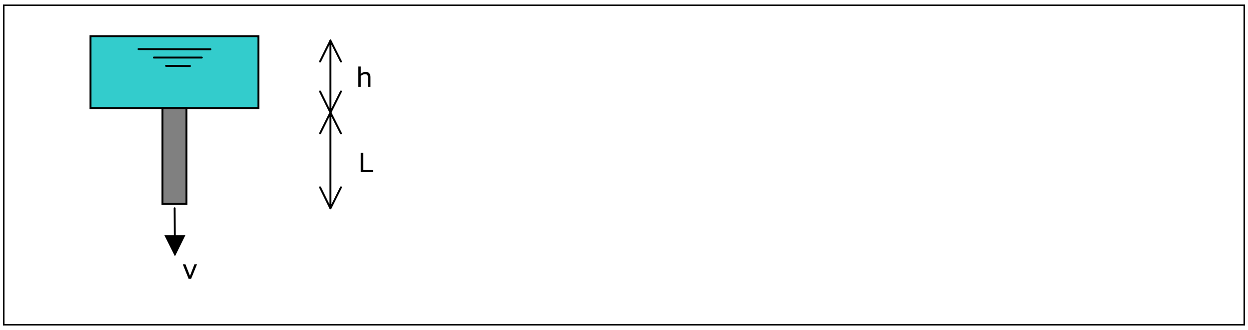


figure A-19: schematization of the drainage system

The discharge of this system can be calculated with the following formulae

$$Q_{drain} = vA_{drain}$$ **Equation 9-1**

$$v = \sqrt{2g\xi(\Delta h + L_{drain})}$$ **Equation 1-2**

$$A_{drain} = \tfrac{1}{4}\pi R^2$$ **Equation 1-3**

Where

Q_{max}	=	maximum drain discharge	(m^3/s)
v	=	velocity in the drain	(m/s)
g	=	acceleration due to gravity	(m/s^2)
ξ	=	friction parameter	(-)
Δh	=	water height above the drain	(m)
L_{drain}	=	length of the discharge pipe	(m)
A_{drain}	=	drain surface	(m^2)
$V_{crestbasin}$	=	volume of the crest basin	(m^3)
t	=	time	(s)

Rewriting gives:

$$\xi = \frac{\left(\frac{V_{crestbasin}}{t}\frac{1}{\pi R^2}\right)^2}{2g(\Delta h + L_{drain})}$$ **Equation 1-4**

Objective is to find the ξ for the different drains. Since, R, g, ΔH and L are known for all the tests, only V and t have to be determined. When these values are obtained, ξ can be calculated with the formulae shown above.

X.I Test set up for calibration of the drain.

Use have been made of an box in which an overflow is created. A constant inflow guaranties a fixed waterlevel. See figure A-20 for a schematisation and Figure A-21 for an impression of the calibration tests.

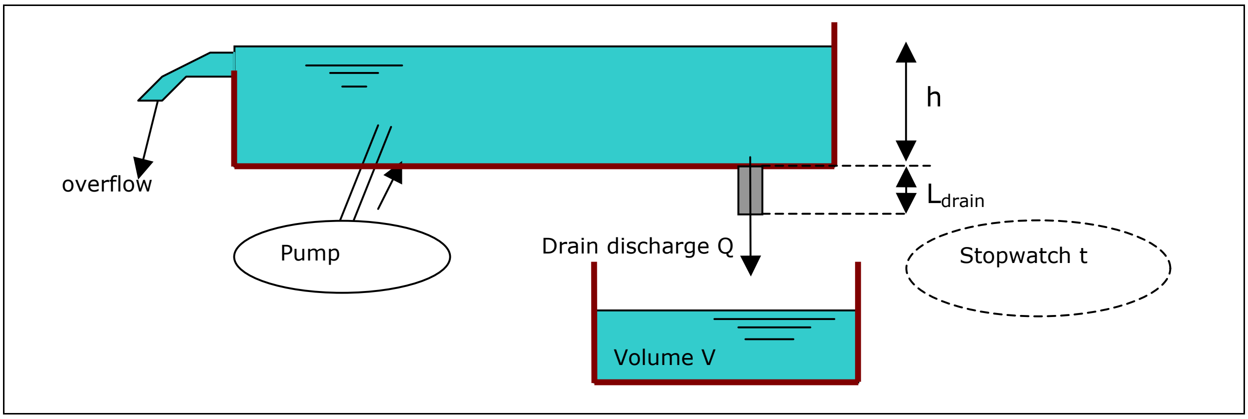


figure A-20: schematisation of the calibration test

Figure A-21: Impression of the setup of the calibration tests

The water level h was slightly different for every calibration test. These values are shown in table A-13.

Calibration test number	**diameter drain**	**length drain**	**water level**
	D_{drain}	L_{drain}	h
	(m)	(m)	(m)
205	0.01	0.05	0.313
206	0.02	0.0525	0.303
207	0.03	0.055	0.303
208	0.01	0.1	0.313
209	0.02	0.1	0.303

table A-13: specifications of the calibration tests

Calibration test number	**V**	**t**	**ξ**
	(m^3)	(s)	(-)
205	0.0529	295	0.73
206	0.0910	133	0.68
207	0.1391	90	0.68
208	0.0311	0172	0.65
209	0.1188	160	0.71

table A-14: results of the calibration tests

It needs to be emphasized that these values only yield for free flow. The obtained values cannot be used for situations where a tube is attached to the drains

XI. Media attention

During the graduation project, there were several media who paid attention to the concept of the Crest Drainage Dike. The published articles are shown in the sections below.

- Delta (Newspaper of Delft University of Technology)
- Civiele Techniek (Technical newspaper)
- Teleac radio (Radio fragment: interview with H.J. verhagen and P. van Steeg)
- Contact (Newspaper for alumni of Delft University of Technology)
- Technisch weekblad (Technical newspaper)
- Delft Blauw (Television program)

XI.I Delta

Een geul met uitzicht

Dijken verhogen is niet altijd de beste verdediging tegen zeestormen. Geulen in de waterkeringen kunnen volgens onderzoekers van Civiele Techniek en Geowetenschappen ook dijkdoorbraken voorkomen.

Tomas Van Dijk

De stormvloedwaarschuwingsdienst van Rijkswaterstaat gaf hoogwateralarm af, tijdens de storm van vorige week. Maar in het vloeistoflaboratorium van Civiele Techniek en Geowetenschappen was niets te merken. Toch beukten ook daar golven tegen de dijk.
Sinds enkele weken bootst student waterbouwkunde Paul van Steeg (24) er stormen na in een veertig meter lang aquarium. Op een oude pc met heuse floppydisk heeft hij een programmaatje draaien dat een golfmachine aanstuurt. Hij kan op schaal precies dezelfde golven maken als de golven die voorkomen aan de Noordzeekust, in de Ooster- en Westerschelde en rond de Waddenzee.
Wat hij test is een nieuw concept. De dijk aan het eind van de watergoot heeft een geul op de top. Golven van een decimeter hoog - geschaald naar ware grootte zouden ze zo'n anderhalf â twee meter zijn - breken tegen de waterkering en komen spetterend in de geul terecht. Via afvoerputjes en kanaaltjes wordt het water afgevoerd.

Het onderzoek van Van Steeg en zijn begeleider ir. Henk Jan Verhagen van Civiele Techniek en Geowetenschappen maakt deel uit van het Europese onderzoeksprogramma Comcoast. Doel van het project is te kijken of het mogelijk is de kans op dijkdoorbraken langs de kust te verkleinen, zonder de waterkeringen te verhogen. "Waterkeringen bezwijken meestal doordat het overslaande water het talud aantast aan de binnenzijde van de dijk", vertelt Van Steeg. "Dit is wat in 1953 in Zeeland is gebeurd. Maar als het water dat over de dijk stroomt gecontroleerd wordt afgevoerd, zoals in dit model, blijft de dijk heel."
Door de klimaatverandering zal de zeespiegel de komende honderd jaar naar verwachting zo'n 38 centimeter stijgen. Dat Nederland de dijken moeten verhogen om pas te houden met deze stijging, lijkt voor de hand te liggen. Een flinke klus. Zeker gezien het feit dat 24 procent van de primaire waterkeringen in Nederland nu al niet voldoet aan de huidige veiligheidseisen. Dit stelde de beroepsvereniging voor ingenieurs Kivi Niria enkele maanden geleden in een brandbrief aan het ministerie van verkeer en waterstaat.

Slopen
Maar is het wel gunstig de dijken overal evenredig met de verwachte zeespiegelstijging te verhogen? De twee Delftenaren denken van niet. Op sommige plaatsten zou het volgens hen goedkoper zijn dijken wat minder op te krikken, maar tegelijkertijd wel te voorzien van geulen.
"Een dijk die je een meter verhoogt, moet je ook zes meter verbreden", legt Verhagen uit. "Deze maatregel is relatief goedkoop, maar als er aan de voet van de dijk gebouwen staan die je moet weghalen, kan het toch duur uitpakken. Bovendien is het niet altijd sociaal acceptabel om woningen te slopen, of ze nou duur zijn of niet."
Verhagen: "Uiteindelijk willen we computermodellen maken. Maar daarvoor moeten we eerst experimenteren in een waterbak om gegevens te verzamelen. We modelleren de typen golven en variëren de diameter van de afvoergoten. Het water moet wel snel genoeg wegstromen, anders heeft de geul geen zin. Met drukmeters in de dijk meten we hoe sterk de golven tegen de geul slaan en hoe stevig deze dus gemaakt moet worden."
Van Steeg studeert in mei af op dit experimentele werk. Als de resultaten veelbelovend zijn, zal Rijkswaterstaat een stukje van de Ellewoutsdijk aan de Westerschelde voorzien van een geul om de Delftse dijk te testen.
De dijk wordt maar een meter diep, zodat mensen die er onverhoopt in mochten vallen, niet hun been breken. Daarnaast hebben fietsers en voetgangers uitzicht over het water, vanuit de fiets- en voetpaden die in de geul komen te liggen.

XI.II Civiele Techniek

(this is a concept text)

De “Crest Drainage Dike”: een overslagbestendige dijk

Inleiding

In de komende decennia zullen klimaatveranderingen een zwaardere belasting van de zeeweringen langs de Noordzee veroorzaken. De zeespiegel stijgt en het land daalt, waardoor het zoute water steeds verder zal doordringen in de kustgebieden. Traditioneel wapenen we ons hiertegen door onze dijken steeds verder te verhogen en te verbreden. Het wordt steeds belangrijker om alternatieven voor de traditionele manier van dijkversterken te ontwikkelen Het project ComCoast (zie kadertekst) ontwikkelt hiervoor nieuwe opties, waarbij gestreefd wordt naar meer geleidelijke overgangen van zee naar land, zgn. brede waterkeringszones. Deze overgangsgebieden creëren nieuwe kansen voor zowel het milieu als de mens en bieden duurzame oplossingen om te anticiperen op toekomstige ontwikkelingen.

Overslagbestendige dijken

Binnen ComCoast wordt o.a. onderzoek gedaan naar het overslagbestendig maken van dijken. Onze huidige dijken zijn zo hoog dat er eigenlijk nooit water over heen zal komen. Het binnentalud is namelijk slecht tegen overslaand water bestand. Alleen tijdens zeer extreme stormen kan een, geringe acceptabele, hoeveelheid water over de dijk komen. Bij het principe van een brede waterkeringszone bestaande uit twee parallelle dijken, mag er best eens wat water over de zeewaartse dijk komen, zolang dit maar in de bufferzone wordt opgevangen. De dijk moet dan wel tegen overslaand water bestand zijn.

Ontwerp van de Crest Drainage Dike

In april 2005 heeft het Civieltechnisch Centrum Uitvoering Research en Regelgeving (CUR) namens het project ComCoast het verzoek gedaan aan diverse marktpartijen om innovatieve concepten te ontwikkelen voor een overslagbestendige dijk. Uitgangspunt daarbij was dat de dijk niet verder verhoogd behoefde te worden om de verwachte toekomstige belastingtoenames (zeespiegelrijzing en zwaardere stormcondities) te weerstaan. DHV bedacht het concept van de Crest Drainage Dike (CDD). Dit is een soort van goot (woelbak) welke is ingegraven in de kruin van de dijk. Tijdens een storm wordt het water in deze goot opgevangen en via drainage buizen gecontroleerd afgevoerd, hetzij landwaarts, hetzij zeewaarts van de dijk. Hierdoor wordt de hoeveelheid overslaand water beperkt en behoeft het binnentalud niet extra versterkt te worden. Tijdens normale omstandigheden kan de goot worden gebruikt voor recreatieve doeleinden zoals een fiets- en wandelpad.

Vervolgens is de CDD, tezamen met twee andere ideeën, geselecteerd om verder uitgewerkt te worden middels een theoretische studie. Deze studie is november 2005 door DHV opgeleverd. Uit de studie volgt dat het concept van de CDD technisch en financieel haalbaar zou moeten zijn en dat het goede mogelijkheden op recreatief vlak biedt. Verder geeft de studie nog enkele aanbevelingen voor nadere studie en onderzoek.

Het belangrijkste onderdeel van het nader onderzoek is na te gaan hoe effectief de CDD in werkelijkheid is. Hoeveel water wordt er daadwerkelijk door de drain afgevoerd en hoeveel water gaat er alsnog over de dijk. Daarvoor zijn fysische modelproeven noodzakelijk. Deze proeven verschaffen ook meer inzicht in de optredende stroomsnelheden en laagdikte van het water langs het talud en op de kruin.

Fysische modelproeven

In 2006 zijn in de golfgoten van de TU Delft en Technischen Universität Carolo-Wilhelmina zu Braunschweig (Duitsland) prototypes van de CDD gemaakt.

Proeven op de TU Delft
In het laboratorium van de TU Delft is een CDD op schaal 1:25 nagebouwd in een tweedimensionale proef. In een veertig meter lange goot met een breedte van 0,8 meter met glazen wanden worden stormen nagebootst met behulp van een golfmachine. De diameter van de afvoergoten wordt daarbij gevarieerd.

In Delft heeft Paul van Steeg (afstudeerder) proeven gedaan met verschillende typen golven. Ook het effect van de aanwezigheid van een berm voor de dijk is onderzocht. Het kan zijn dat de golven er dan anders overheen slaan. Er is gemeten hoeveel water er via de kruinconstructie afgevoerd wordt en hoeveel water er over alsnog over de dijk heen slaat. Ook zijn waterdrukken gemeten waaruit men kan afleiden hoe sterk de constructie van de goot moet worden.

De experimenten in Delft zijn afgerond en de resultaten dienen nog geanalyseerd te worden. Er kan echter al wel vastgesteld worden dat het type golf (het golfspectrum) dat opgewekt wordt, niet veel invloed heeft op de effectiviteit van de CDD. Een vergelijkbare conclusie kan worden getrokken met betrekking tot de invloed van een al dan niet aanwezige berm. Dit is van belang om te weten aangezien vele dijken in Nederland een berm hebben.

De invloed van de verhouding tussen de afmetingen van de drain en de afmetingen van de kruinconstructie is een complex fenomeen. Het is duidelijk dat beide aspecten een grote invloed hebben op de werking van de CDD. Er dient gezocht te worden naar een optimale verhouding tussen beide.

Proeven op de TU Braunschweig
In het laboratorium van de TU Braunschweig is een dijk op schaal 1:17 nagebouwd in een tweedimensionale proef. In een negentig meter lange goot met een breedte van 2 meter worden stormen nagebootst met behulp van een golfmachine. De diameter van de afvoergoten wordt gevarieerd.

In Duitsland bekijkt men vooral de lay-out van de goot en doet men onderzoek naar invloed van de helling van het buitentalud. Men test er de invloed van de breedte en diepte van de goot op de effectiviteit ervan door deze te variëren.

Daarnaast wordt de instroom- en uitstroomzijde van de goot, in samenspraak met de betrokkenen, verder geoptimaliseerd. Zo geldt dat, indien blijkt dat er nog steeds veel water over dijk stroomt, alternatieven voor de landwaartse zijde van de goot dienen te worden bedacht en te worden getest. Eveneens wordt bekeken of het water, dat over het buitentalud richting de kruin stroomt, gemakkelijk in de goot stroomt. Als dat niet het geval is dienen alternatieven voor de goot (zeewaartse zijde) te worden bedacht en getest.

Onderzoek naar de technische uitvoerbaarheid
Naast de modelproeven in de laboratoria wordt bij DHV in Amersfoort door twee afstudeerders van de Hogeschool Rotterdam o.a. onderzoek gedaan naar de overgangsconstructies in het ontwerp. Een starre constructie in een dijklichaam vereist een grondige beschouwing. Hoe gedraagt de CDD zich bij zettingen? Hoe worden de hoge stroomsnelheden weerstaan op de overgangsconstructies?

Verder wordt nog nader ingegaan op het de drainage buizen, de uitstroomconstructie van de buizen en het verdere ontwerp van het achterland om het water verder in op te vangen. De studie is pas kortgeleden van start gegaan.

Vervolgstappen
De resultaten van de diverse hiervoor beschreven nadere studies worden rond de zomer

van 2007 verwacht en zullen op de websites van DHV (www.dhv.nl) en ComCoast (www.comcoast.org) gepubliceerd worden.

Kadertekst:

ComCoast - 'COMbined functions in COASTal defence zones' - is een Europees project dat innovatieve oplossingen ontwikkelt en presenteert om kustgebieden te beschermen tegen overstromingen. Rijkswaterstaat heeft de leiding over dit project. Naast Nederland doen nog vier andere Noordzeelanden mee: Groot-Brittannië, Duitsland, België en Denemarken. In totaal doen er tien partners mee aan het project. De partners delen hun kennis en ervaring, en zoeken naar de beste oplossingen voor de gehele Europese kustverdediging, bestaande uit dijken en duinen. ComCoast loopt van 1 april 2004 tot 31 december 2007. Het Interreg IIIB-programma voor de Noordzee van de Europese Unie financiert samen met de projectpartners de projectkosten van 5,8 miljoen euro.

ComCoast richt zich op het ontwikkelen van multifunctionele waterkeringszones langs de kust die een geleidelijker overgang bieden van zee naar land, die de bevolking en het milieu in de kuststreken ten goede komen en die economisch haalbaar zijn.

Het concept richt zich in eerste plaats op zeedijken:
• om betaalbare en duurzame alternatieven te bieden voor het keer op keer verhogen van de bestaande waterkering
• om een win-winsituatie te creëren voor zowel het waterbeheer in een bredere kuststrook als voor multifunctioneel landgebruik, en
• oplossingen te vinden voor de ruimtelijke ontwikkelingsbehoefte van de kuststreek.

Het doel van ComCoast is:
• onderzoek naar het ruimtelijk mogelijkheden voor brede waterkeringsszones voor bestaande en toekomstige locaties in de 'North Sea Interreg IIIb'-regio
• het creëren en toepassen van nieuwe methoden voor het waarderen van multifunctionele waterkeringszones vanuit economisch en sociaal oogpunt
• het ontwikkelen van innovatieve technische oplossingen om dijken bestendig te maken tegen overslaande golven voor het vereiste veiligheidsniveau
• het verbeteren en toepassen van strategieën om belanghebbenden bij het geheel te betrekken met de nadruk op participatie door het publiek
• het in praktijk brengen van de beste multifunctionele waterkeringszones op ComCoast-proeflocaties
• het delen van kennis in de 'Interreg IIIb North Sea'-regio.

XI.III Teleac radio

A radio interview about this graduation project has been given during the physical experiments. In this interview, the student, P. van Steeg, and his mentor, Ir. H.J. Verhagen, explain to Teleac radio the backgrounds of the Crest Drainage Dike. This interview, which takes 7:30 minutes, can be found online at:

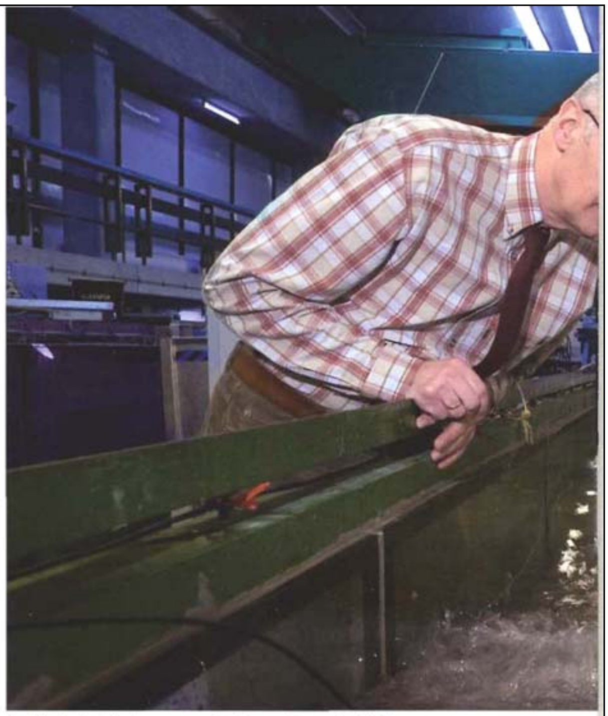
http://www.teleac.nl/radio/index.jsp?nr=134903&news_nr=1279278. (press "beluister deze uitzending" and scroll to 40:10 minutes)

XI.IV Contact

TEKST NATALIE HANSSEN FOTO'S JAAP OLDENKAMP

Het nieuwe denken over dijken

De strijd tegen stijgend zeewater kan eleganter gevoerd worden dan met het ophogen van dijken. Dat is tenminste de inzet van het Europese ComCoast (combined functions in coastal defence) project. TU-studenten helpen mee aan twee kansrijke innovaties.



Ir. Henk Jan Verhagen en Paul van Steeg (rechts)

Met de klimaatverandering zullen de stormen in Europa heviger worden. De combinatie van hardere wind, hogere waterstanden en hogere golven maakt dat bestaande dijken zwaarder belast worden. "Dan kun je een dijk verhogen; dat is een relatief goedkope optie. Maar niet altijd wenselijk. Een verhoging vraagt ook om een forse verbreding, en dat zijn in bebouwd gebied dure vierkante meters. Soms is de verhoging zelf niet wenselijk, bijvoorbeeld omdat je dan niet meer over de dijk kunt kijken."

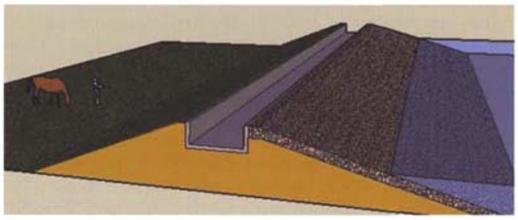
Aan het woord is ir. Henk Jan Verhagen. Verhagen is universitair hoofddocent waterbouwkunde en voorzitter van de Nederlandse gebruikersgroep van ComCoast, een Europees project wat innovatieve oplossingen ontwikkelt om kustgebieden tegen overstroming te beschermen. "Het uitgangspunt van ComCoast is bestaande zeedijken ook zonder verhoging te kunnen gebruiken bij zwaardere belasting. Daarbij wordt naar multifunctioneel gebruik van de kustzone gekeken. We hebben een wedstrijd uitgeschreven om tot innovaties te komen. Uit tien ideeën zijn de twee meest kansrijke initiatieven gekozen. Smart grass reinforcement, speciale matten van geotextiel die het natuurlijke gras op het binnentalud verstevigen en bij golfoverslag weerbaarder maken. En crest drainage dikes, dijken met een geul om wateroverslag op te vangen en gecontroleerd af te voeren. Deze ideeën worden verder uitgewerkt."

Slimme matten De matten om het binnentalud van een dijk te verstevigen, haalden onlangs nog de landelijke media. In Delfzijl werd een proef gestart met een overslagsimulator, om te testen of de matten inderdaad versteviging geven. De simulator produceert 'plonzen', die hetzelfde effect geven als een hele golfoverslag. Handig, want de werking van gras laat zich niet verschalen. Bovendien is het lang wachten op een storm die de golven over de dijken doet slaan, en volgens de statistieken eens per 10.000 jaar voorkomt.

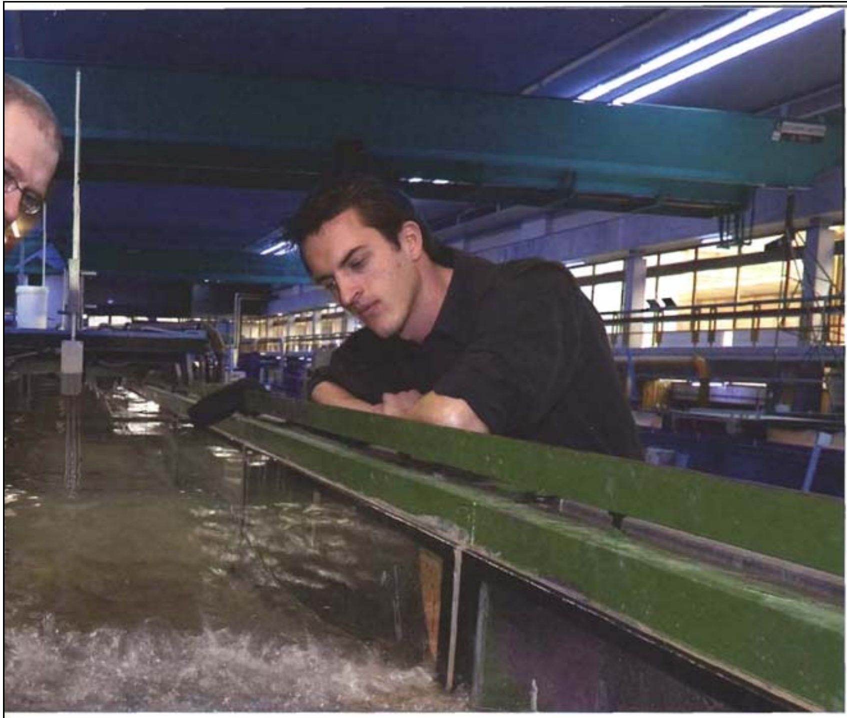
"De oorzaak van dijkdoorbraken ligt doorgaans in het wegslaan van het binnentalud bij wateroverslag," legt Verhagen uit. "Dat is ook wat in 1953 gebeurde. Natuurlijk gras geeft veel versterking, maar kent ook problemen. Er ontstaan gaten op kale plekken, bijvoorbeeld op plaatsen waar een paaltje staat, of een molshoop zit. Die gaten worden groter en zorgen uiteindelijk dat de dijk doorbreekt. We verwachten dat als er een verstevigende mat onder zit, er minder schade zal ontstaan." Verhagen was bij de ingebruikname. "Redelijk spectaculair en heel leuk om te zien. In de komende weken wordt een serie golfoverslagen gedaan om te kijken hoe het water over de dijk loopt, en of het gras heel blijft onder verschillende condities."

Het onderzoek wordt gedaan door ingenieursbureaus Infram, de bedenkers van de overslagmachine, en Haskoning, die de matten ontwikkelde. De overslagmachine maakt plonzen van 4 meter breed van maximaal 14000 liter per plons. Onderzocht wordt de duur van een overslag, maar ook de laagdikte van het water als een functie van de tijd. Student Gijs Bosman is vanuit de TU Delft bezig de berekeningen uit te werken.

Dure Geulen Het tweede idee om een betere dijk te krijgen zonder verhoging, de crest drainage dike van ingenieursbureau DHV, is wel op schaal te simuleren. In samenwerking met de Technischen Universität Carolo-Wilhelmina zu Braunschweig onderzoekt TU Delft



Crest drainage dike



de mogelijkheid om water dat over de dijk slaat, op de kruin op te vangen in een geul in die kruin. Paul van Steeg, student waterbouwkunde, studeert af binnen het project. "Bij de crest drainage dikes wordt een deel van het water dat over de dijk slaat, opgevangen en gecontroleerd afgevoerd. Landwaarts of terug de zee in, dat zal per plek verschillen. We verwachten de overslag met 75 tot 90 procent te reduceren, waardoor de dijk veel beter stand houdt. Duitsland bekijkt vooral de lay-out van de betonnen geul. De afmetingen, maar ook de vorm, een ronde vorm zou bijvoorbeeld een betere golfterugslag kunnen geven. Ook doen ze onderzoek naar de helling van het buitentalud. Hier aan de TU Delft modelleren we verschillende typen golven. Daaruit kunnen we straks concluderen of het evengoed werkt bij de Waddenzee als in Zeeland. Daarnaast onderzoeken we of het systeem evengoed werkt als er een berm voor de dijk ligt, want dat komt nogal eens voor in Nederland. Het kan zijn dat de golven er dan anders overheen slaan. Ook de waterdruk meten we, zodat we af kunnen leiden hoe sterk het beton moet worden."

Modelleren In het laboratorium heeft Paul een dijk op schaal 1:25 nagebouwd in een tweedimensionale proef. Naar goed Hollands gebruik verzakt ook het zand van de dijk op schaal, maar gelukkig hangt de helling nog op schroeven. In de veertig meter lange goot met glazen wanden bootst Paul stormen na met behulp van een golfmachine. Hij modelleert golven zoals ze voorkomen in de Noord- en Waddenzee, en in de Ooster- en Westerschelde. "Hoewel het aanvankelijk niet de bedoeling was, varieer ik ook met de diameter van de afvoergoten. Ik kijk hoeveel er door de drain wordt afgevoerd en hoeveel water er bij een bepaalde golfslag nog over de dijk heen komt. Daaruit kun je afleiden hoe laag de dijk kan blijven. Ik varieer ook met de berm. De testen zijn ongeveer afgerond, nu begint de analyse. Eind mei is die denk ik klaar. Als u dit rapport tenminste een beetje goed vindt," zegt hij met een schuin oog naar zijn afstudeerbegeleider Henk Jan Verhagen. "Ja, dit rapport vind ik een beetje goed," antwoordt die vrolijk. "Nog een paar metingen," gaat Paul verder, en dan opschrijven en conclusies trekken. En de gaten vinden – een onderzoek is natuurlijk nooit af, ik wil ook aanbevelingen doen wat verder uitgezocht moet worden."

Gepielepeuter "Het is leuk om 'voor het echie' iets te doen," vertelt Paul later. "Als de resultaten veelbelovend zijn, gaat Rijkswaterstaat het in de praktijk testen. Zelf als eruit komt dat het geen goed systeem is, heb ik een toevoeging geleverd." Lastig aan laboratoriumwerk vindt hij dat er altijd veel foutjes in het modelleren sluipen. Als het waterpeil bijvoorbeeld een millimeter zakt doordat er water over de rand slaat, heeft dat invloed op de metingen en moet er dus water teruggegoten worden. "Dat is echt gepielepeuter, maar je moet het wel oplossen. Dat kost tijd. Frappant was dat alle verschillende typen golven die ik heb getest, hetzelfde resultaat gaven. Dat had ik niet verwacht. Het versimpelt het probleem natuurlijk wel – welke golf er ook opvalt, je kunt hetzelfde sommetje toepassen." Hij gaat na zijn afstuderen werken bij Deltares, de nieuwe naam van het onderzoeksinstituut dat ontstaan is door een fusie van het Waterloopkundig Laboratorium, GeoDelft en een deel van de specialistische diensten van Rijkswaterstaat. Die sollicitatie komt voort uit de interesse die hij kreeg tijdens dit onderzoek.

Anders denken Hoe vernieuwend ook, de afvoergeul is kostbaar. Er zijn stevige betonconstructies nodig, soms over kilometers lengte. De vraag is of het ook een economische oplossing is. Verhagen denkt niet dat het heel veel toegepast zal worden, al ziet hij wel plekken waar het nuttig kan zijn. "Eigenlijk staan ze op de tweede plaats; de matten zijn levensvatbaarder. Het levert veel op en kost weinig."

Hij verwacht dat per district andere oplossingen gekozen zullen worden in Nederland. "Het meest voor de hand liggende is nog steeds een dijk verzwaren. Andere oplossingen zijn de dijk overslagbestendig maken, het buitentalud flauwer maken of een kade plaatsen. Theoretisch kun je ook land teruggeven aan de zee. Het zal altijd een combinatie blijven in Nederland – elke dijk heeft een ander aandachtspunt. Met deze twee innovaties verandert het landschap veel minder ingrijpend dan met een dijkverzwaring. Het enige wat ingrijpend verandert, is de veiligheid. Buiten dat is natuurlijk een zeer grote verandering dat we kijken naar wateroverslag in plaats van dijkverzwaring. Een heel nieuwe manier van denken." ■

XI.V Technisch weekblad

Waterschappen willen geld voor kustbescherming

Er moet iets gebeuren aan de Nederlandse kustverdediging. Een extra rij Waddeneilanden of zeedijken waar golven overheen kunnen slaan, bijvoorbeeld.

[...]

Verder werkt Rijkswaterstaat binnen het Europese project ComCoast aan Sea Defenceen, waar onder gecontroleerde omstandigheden golven overheen kunnen slaan. Het water kan dan achter de dijk worden weggepompt.

Grasversterking

De overslagbestendige dijken moeten wel worden verstevigd, omdat anders uitholling ontstaat. Verschillende ingenieursbureaus opperden hiervoor al plannen. Royal Haskoning en Infram kwamen met een systeem voor 'grasversterking'. Eerst leggen zij een kunststof geogrid op de kruin en de binnenzijde van de dijk, waar het gras doorheen moet groeien. Zo ontstaat een verhoogde weerstand tegen afschuiving en oppervlakte-erosie door golf-overslag. 'Volgend jaar maart testen we dit versterkingssysteem in een golfoverslagsi-mulator op een dijk van het Waterschap Hunze en Aa's in Groningen', zegt ir. Gert Jan Akkerman van Haskoning.

Ook DHV ontwierp een eigen versie van de overslagbesten-dige dijk. De zogenaamde Crest Drainage Dike heeft in plaats van een versterkt binnentalud een holle bak bovenop de dijk, waarin het overslaande water wordt opgevangen. Met drainagepijpen is het water uit de bak af te voeren. Onder nor-male omstandigheden dient de holle bak als fietspad. De universiteiten van Delft en Hannover pikten het idee van DHV op en prototypes zijn in de maak. 'Rond december zouden daar de eerste proeven mee moeten plaatsvinden', aldus Martijn Karelse van DHV. [...]

figure A-22: Article in Technisch Weekblad [Technisch weekblad, 2006]

XI.VI Delft Blauw

On the 26th of May 16:00h, the television program “Delft Blauw” will be broadcasted on RTL5 (Dutch television). In this show, the mayor of Rotterdam, Opstelten, will interview P. van Steeg and ask him about aspects of the Crest Drainage Dike. This can also be found on www.delftblauw.nl