Velocity and flow depth variations during wave overtopping

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Velocity and flow depth variations during wave overtopping

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Preface

This report is my final thesis for the master study in Hydraulic Engineering at the faculty of Civil Engineering and Geosciences at the Delft University of Technology in the Netherlands.

I would like to thank the graduation committee. Special thanks are given to J.W. van der Meer and H.F.R. Schüttrumpf for their intensively support during the thesis. I also wish to thank the support of Infram, Royal Haskoning, Rijkswaterstaat and the water board “Hunze & Aa’s” during the case study in Delfzijl with the wave overtopping simulator.

Gijs Bosman
Delft, August 2007
Executive summary

Nowadays the protection of our country for high sea levels and heavy storms is daily news. The protection of the coastal zone is never finished, because of new insights and natural shifts. The ComCoast project (COMbined functions in the COASTal defence zones) is originated by ten organisations out of five European countries bordering the North Sea coasts in order to develop innovative solutions for flood protection in coastal areas. Instead of automatically raising the coastal defence zone on places where more protection is needed, ComCoast creates multifunctional flood management schemes with a more gradual transition from sea to land. Part of the new solutions is the wave overtopping resistant dike. This so called “overtopping durable dike”, should withstand wave overtopping during a storm much better than the current ones. More knowledge about the loads on the dike by wave overtopping is therefore needed. At the moment a lot of information is available about wave overtopping discharge. A few decades ago formulae were derived to determine wave overtopping discharges on coastal constructions during storms. With wave and dike parameters one is able to determine the average wave overtopping discharge. But one can expect that an individual overtopping event will have different impacts on the dike construction. The load from a wave overtopping event of 10,000 l/m will probably be much heavier than the load from a wave of 1,000 l/m. Velocities and flow depths on the crest and inner slope will differ between both individual events.

Recently formulae have been derived for maximum flow depths and velocities on the crest and inner slope. These formulae are based on the difference between fictive wave run-up and the crest freeboard. This is a good measure to determine the flow depths and velocities on the dike. These formulae have been calibrated by two independent physical model test programs in different wave flumes. One of the studies has been carried out by Schüttrumpf in Germany (Schüttrumpf 2001). He performed large and small scale model tests and determined the empirical coefficients. The other study has been carried out on small scale by Van Gent in the Netherlands (Van Gent 2002). He calibrated the formulae and determined the empirical coefficients as well. When these two studies are compared there is a large difference that attracts the attention. The empirical coefficient of the flow depth equation was determined to be a factor 2.2 higher by Schüttrumpf than by Van Gent: coefficient \( c_{d2\%} = 0.33 \) (Schüttrumpf 2001) or \( c_{d2\%} = 0.15 \) (Van Gent 2002). They agreed about the empirical coefficient of the velocity equation: coefficient \( c_{u2\%} = 1.37 \) (Schüttrumpf 2001) or \( c_{u2\%} = 1.30 \) (Van Gent 2001). They collectively wrote a paper (Schüttrumpf, Van Gent 2003) and found the test set-up as primary cause for the discrepancy in the flow depth coefficient. The differences between the test set-up and analysis have been studied in the present report. The overtopping time (duration of an overtopping event) and the variation of velocity and flow depth in time have been investigated as well. These quantities are also necessary to be able to give a full description of the loads on the dike during wave overtopping.

Data of measured velocities and flow depths on the crest and inner slope were available from both physical model tests. With this data the reliability of the measurements was analysed first. It was concluded that the velocities measured
during the large scale tests by Schüttrumpf were not correct. The velocity of high turbulent, non stationary, air containing flow appeared to be very difficult to measure. Velocities above 2.5 m/s were not registered. The empirical velocity coefficient found by Schüttrumpf is therefore too low ($C_{u,2\%} = 1.37$). For that matter not only the flow depths, but also the velocities that occurred during Schüttrumpf’s tests were higher than by Van Gent.

Secondly differences in test set-ups were analysed. The present report shows that the outer slope is of great importance in the flow depths and velocities on the crest. This is new knowledge. Schüttrumpf performed his tests on a dike model with an outer slope of 1:6 and Van Gent used a dike model with an outer slope of 1:4. The empirical coefficients appeared to be dependent on the outer slope steepness. Therefore formulae for maximum flow depth and velocity were adapted: the coefficients are written as a function of the outer slope angle. Subsequently a formula for the overtopping time is created, based on the difference between fictive wave run-up and crest freeboard. The overtopping time appeared not to be a function of the outer slope. The variation of flow depth and velocity in time can be approached with a linear function. Furthermore new equations for flow depth and velocity on the crest are presented. The present report proposes to implement an extra transition parameter. If water runs-up on the outer slope and starts to flow on the crest, the vertical velocity component has to change in a horizontal direction. This transition causes flow depths and velocities to decrease with approximately 20%. Finally the present report describes that the residence time of water on the crest for different exceedance levels is Rayleigh distributed.

The new equations for wave overtopping are compared to the results obtained by tests with the wave overtopping simulator. The wave overtopping simulator is a machine which is able to simulate wave overtopping on a dike on full scale. Despite the difficulties in measuring velocities and flow depths during these tests, one can conclude that the wave overtopping simulator works very well. Storms with high overtopping discharges can be simulated accurately for testing strength and stability of a dike.

Overtopping volumes of water are highly turbulent and have a lot of air entrainment, research is needed to study in which way velocities and flow depths can be measured best under such extreme conditions. Furthermore extra research is recommended to find a more complete physical description of the results found in the present study. Especially the shape of the run-up tongue on different outer slopes is crucial for better understanding in wave overtopping loads.
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**Latin symbols**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>empirical coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$a_1$, $a_2$</td>
<td>shape factors</td>
<td>[-]</td>
</tr>
<tr>
<td>b</td>
<td>scaling coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$B_c$</td>
<td>crest width</td>
<td>[m]</td>
</tr>
<tr>
<td>$C_{h,2%}$</td>
<td>empirical coefficient, flow depth (1)</td>
<td>[-]</td>
</tr>
<tr>
<td>$C_{u,2%}$</td>
<td>empirical coefficient, flow velocity (1)</td>
<td>[-]</td>
</tr>
<tr>
<td>$C_{\text{Tovt},2%}$</td>
<td>empirical coefficient, overtopping time (1)</td>
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</tr>
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<tr>
<td>$c_r$</td>
<td>friction coefficient</td>
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<tr>
<td>$c_{\text{trans}, h}$</td>
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<td>[-]</td>
</tr>
<tr>
<td>$c_{\text{trans,ovt}}$</td>
<td>influence due to transition from outer slope to crest</td>
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</tr>
<tr>
<td>$c_{u,s} / c_{u,s}$</td>
<td>coefficients (s=shape)</td>
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<tr>
<td>d</td>
<td>water depth</td>
<td>[m]</td>
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<tr>
<td>f</td>
<td>friction coefficient (=0.02 for smooth coverage and grass)</td>
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<tr>
<td>$f_i$</td>
<td>influence factor due to friction on the inner slope</td>
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<tr>
<td>g</td>
<td>acceleration due to gravity</td>
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<tr>
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<td>[m]</td>
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<td>[m]</td>
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<td>spectral wave height</td>
<td>[m]</td>
</tr>
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<td>[-]</td>
</tr>
<tr>
<td>$k_h$</td>
<td>coefficient</td>
<td>[-]</td>
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<tr>
<td>L</td>
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<td>[m]</td>
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<tr>
<td>$L_0$</td>
<td>deep water wave length</td>
<td>[m]</td>
</tr>
<tr>
<td>$m_n$</td>
<td>n$^{th}$ order moment of the energy density spectrum</td>
<td>[-]</td>
</tr>
<tr>
<td>n</td>
<td>outer slope: cotα or 1:n</td>
<td>[-]</td>
</tr>
<tr>
<td>$P_{\text{ov}}$</td>
<td>probability of an overtopping event</td>
<td>[-]</td>
</tr>
<tr>
<td>$P_v$</td>
<td>probability of an overtopping volume</td>
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</tr>
<tr>
<td>q</td>
<td>average wave overtopping discharge per meter</td>
<td>[m³/s/m]</td>
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<td>$q_{\text{cal}}$</td>
<td>calculated overtopping discharge</td>
<td>[m³/s/m]</td>
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<tr>
<td>r</td>
<td>correlation coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$R_c$</td>
<td>crest freeboard relative to SWL</td>
<td>[m]</td>
</tr>
<tr>
<td>$R_u$</td>
<td>wave run-up</td>
<td>[m]</td>
</tr>
<tr>
<td>$R_{u,x}$</td>
<td>run-up level exceeded by x percent of the incoming waves</td>
<td>[m]</td>
</tr>
<tr>
<td>s</td>
<td>position along the inner slope</td>
<td>[m]</td>
</tr>
<tr>
<td>$s_{m-1,0}$</td>
<td>wave steepness</td>
<td>[-]</td>
</tr>
<tr>
<td>t</td>
<td>time</td>
<td>[s]</td>
</tr>
<tr>
<td>$t_0$</td>
<td>start time data record</td>
<td>[s]</td>
</tr>
<tr>
<td>$t_{\text{test}}$</td>
<td>total test duration</td>
<td>[s]</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Unit</td>
</tr>
<tr>
<td>------------</td>
<td>-------------------------------------------------------</td>
<td>-------------</td>
</tr>
<tr>
<td>$T_{av,oiv}$</td>
<td>wave-averaged overtopping time</td>
<td>[-]</td>
</tr>
<tr>
<td>$T_{m-1,0}$</td>
<td>spectral wave period</td>
<td>[s]</td>
</tr>
<tr>
<td>$T_{oiv}$</td>
<td>overtopping time</td>
<td>[s]</td>
</tr>
<tr>
<td>$T_{p}$</td>
<td>the peak period in the wave spectrum</td>
<td>[s]</td>
</tr>
<tr>
<td>$T_{res}$</td>
<td>total residence time</td>
<td>[%]</td>
</tr>
<tr>
<td>$T_{res,h}$</td>
<td>residence time exceeded by level h</td>
<td>[%]</td>
</tr>
<tr>
<td>$T_{res,u}$</td>
<td>residence time exceeded by level u</td>
<td>[%]</td>
</tr>
<tr>
<td>$u$</td>
<td>velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_0$</td>
<td>velocity at the landward side of the crest</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_{2%}$</td>
<td>maximum velocity exceeded by 2% of the incident waves</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_{front}$</td>
<td>front velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_{max(x)}$</td>
<td>maximum velocity on position $x$</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_{max}$</td>
<td>mean maximum velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_{is}$</td>
<td>velocity at the inner slope</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_s$</td>
<td>velocity component along the inner slope</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_v$</td>
<td>vertical velocity component</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_s$</td>
<td>shear stress</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$V$</td>
<td>number of Ursell</td>
<td>[-]</td>
</tr>
<tr>
<td>$V_{2%}$</td>
<td>volume of an overtopping event</td>
<td>[m$^3$/m]</td>
</tr>
<tr>
<td>$V_{2%}$</td>
<td>overtopping volume exceeded by 2% of the inc. waves</td>
<td>[m$^3$/s/m]</td>
</tr>
<tr>
<td>$x_c$</td>
<td>$x$-coordinate on the crest</td>
<td>[m]</td>
</tr>
<tr>
<td>$x_*$</td>
<td>relative wave run-up in horizontal direction</td>
<td>[m]</td>
</tr>
<tr>
<td>$y_0$</td>
<td>integration coefficient</td>
<td>[m]</td>
</tr>
<tr>
<td>$z_{2%}$</td>
<td>run-up level exceeded by 2% of the incoming waves</td>
<td>[m]</td>
</tr>
<tr>
<td>$z_{5%}$</td>
<td>run-up level exceeded by 5% of the incoming waves</td>
<td>[m]</td>
</tr>
</tbody>
</table>

**Greek Symbols**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$</td>
<td>angle of the outer slope</td>
<td>[-]</td>
</tr>
<tr>
<td>$\beta$</td>
<td>inner slope angle</td>
<td>[°]</td>
</tr>
<tr>
<td>$\delta$</td>
<td>boundary layer</td>
<td>[m]</td>
</tr>
<tr>
<td>$\gamma_b$</td>
<td>reduction factor for a berm</td>
<td>[-]</td>
</tr>
<tr>
<td>$\gamma_c$</td>
<td>influence factor, due to friction at crest</td>
<td>[-]</td>
</tr>
<tr>
<td>$\gamma_l$</td>
<td>influence factor for the roughness on the outer slope</td>
<td>[-]</td>
</tr>
<tr>
<td>$\gamma_v$</td>
<td>influence factor for a vertical wall on slope</td>
<td>[-]</td>
</tr>
<tr>
<td>$\gamma_o$</td>
<td>reduction factor for angled wave attack</td>
<td>[-]</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>Von Karman coefficient (which has a value of 0.4)</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$\rho$</td>
<td>density</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$\tau_0$</td>
<td>shear stress</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>$\nu$</td>
<td>viscosity</td>
<td>[Pas]</td>
</tr>
<tr>
<td>$\xi_0$</td>
<td>Breaker or surf parameter</td>
<td>[-]</td>
</tr>
</tbody>
</table>

**Abbreviations**

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>EMS</td>
<td>Electro Magnetic Velocity Meter</td>
</tr>
<tr>
<td>GWK</td>
<td>Grosser Wellenkanal (Large wave flume Hannover)</td>
</tr>
<tr>
<td>MSL</td>
<td>Mean Sea Level</td>
</tr>
<tr>
<td>SWL</td>
<td>Still Water Level</td>
</tr>
<tr>
<td>WG</td>
<td>Wave Gauge</td>
</tr>
</tbody>
</table>
1 Introduction

1.1 History
1.2 ComCoast
1.3 Problem analysis
1.5 Objectives
1.6 Outline of the report
1.1 History

Sea dikes in the Netherlands can be found in the north (Groningen, Friesland, Afsluitdijk) in the northwest (Hondsbossche en Pettermerzeewering) and in the southwest (Zeeland). The characteristics of the Dutch dikes exhibit a toe structure with a rather gently sloping outer slope (1:4). The dikes usually consist of a berm at MSL, a crest with a certain width and an inner slope (usually steeper than the outer slope, around 1:3). The outer slope is in most cases armoured with a placed block revetment, rock or asphalt while the crest and inner slope have usually been covered with grass. When in 1953 a ferocious storm attacked the Dutch coast a large number of dikes collapsed and large areas of land were completely flooded with water. The disaster killed 1835 people in the Netherlands. Many dikes failed because of overtopping waves. Large volumes of water passed the crest of the dike and damage the inner slope of the dikes. The dikes would typically fail when the inner slope slides. After this disaster new guidelines for the coastal defence were made to protect the country in the future for this kind of storms. The storm surge level is determined to a level which has a probability of exceedance of 1/10,000 or 1/4,000 a year. Dissimilarity is due to differences in economical value of the protected area. The crest height of sea dikes is much higher than the storm surge level and based on the amount of overtopping waves. Two percent of the incoming waves are allowed to exceed the crest. This results in a crest freeboard of three to five meters above the storm surge level. In the late twentieth century an extra guideline was added which is the average wave overtopping discharge of overtopping water, typically amounts like one litre every second per meter dike width.

![Figure 1-1 cross section of a typical Dutch sea dike](image)

In the future stronger storms are expected due to climate change. Together with more wave attack and relative sea level rise this can lead to the problem that the dikes won’t be high enough anymore. Wave overtopping can lead to sliding of the inner slope due to erosion and/or infiltration. The original policy of raising dikes may not be sufficient or recommendable in the future. Thus when dikes won’t be raised, more overtopping water can be expected. This doesn’t have to be a problem, if the dike is strong enough to cope with the overtopping water and if there is a good water management for the area behind the dike. It can even be a very good solution to reinforce the dikes instead of raising them. Before it is known if a dike is strong enough or where and how it has to be reinforced, a good understanding of the loads on the dike is needed. How does the overtopping water flow over the crest and inner slope? How high are velocities and flow depths and how do they change in time? A
lot of research has already been carried out. This report is a supplement to these former studies. New insight will be given to the characteristics of overtopping water.

1.2 ComCoast
The present study is part of the ComCoast project. ComCoast stands for COMbined functions in COASTal defence zones. ComCoast is a European project which develops and demonstrates innovative solutions for flood protection in coastal areas. Instead of automatically raise the coastal defence zone where more protection is needed, ComCoast creates multifunctional flood management schemes with a more gradual transition from sea to land. This benefits the wider coastal community and environment whilst offering economically sound options. Better understanding of the characteristics of overtopping water can contribute to new innovative coastal defence solutions. The ComCoast concept focuses on coastal areas comprising of embankments. The five participating countries are Belgium, Germany, the Netherlands, Denmark and the United Kingdom.

1.3 Problem analysis
In the future more wave overtopping will be expected on dikes due to (relative) sea level rise. Erosion and infiltration take place at the crest and inner slope of a dike due to this overtopping. There is more insight needed to be able to understand and predict more accurately the failure mechanisms caused by overtopping waves. A failure mechanism depends on load bigger than strength. This study will focus on the loads.
Several studies have been carried out in the Netherlands by Van Gent (2002) and in Germany by Schüttrumpf (2001a). Both studies give formulae for maximum flow depths and maximum velocities, but there is also a variation in time. The data are available to study this variation, but the study has not yet been carried out. This variation in time of flow depth is important for a determination of the infiltration of water in a dike while a variation in time of the flow velocities give more insight in the amount of erosion. Furthermore the studies carried out in the Netherlands and Germany do not match in the maximum flow depths on the crest. Schüttrumpf determines double heights of maximum flow depths in comparison with Van Gent. On the other hand, both studies do coincide about the maximum flow velocities.
This study will focus on the duration and the variation in time of the overtopping water. Also the volumes and discharges will be investigated. A study will be done to assess the differences between Schüttrumpf and Van Gent. There will be a special focus on the crest of the dike. This comes from the supposition that if one is able to describe the flow conditions of the overtopping water on the crest in the correct way, one is also able to describe the flow depth and velocities on the inner slope. Here only gravity and friction are involved.

1.4 Objectives
The main objectives are to:
• Explain the difference between the studies of Van Gent and Schüttrumpf according to the flow depth and produce a new, univocal formula.
• Study the duration ($T_{ovt}$) and variation (shape) of flow velocities and flow depths in time.
The secondary objective is to:

- Study the results of the measurements with the wave overtopping simulator and compare them with the results of the wave flumes.

1.5 Outline of the report

In chapter 2 the wave overtopping theories about maximum flow velocities and flow depths on the crest and inner slope of a dike will be described. The theories of Schüttrumpf and Van Gent will be explained.

![Image](image-url)

*Figure 1-2 test in the GWK in Hannover in 2001*

In chapter 3 the overtopping theories of Schüttrumpf and Van Gent will be discussed. The measurements and data-processing methods will be validated with test data from both studies.

In chapter 4 a new theory is presented for maximum velocities and flow depths. Also the overtopping time and residence time are discussed. The overtopping theories of the presents study are validated with the tests data of Schüttrumpf and Van Gent. The empirical coefficients will be determined.

In chapter 5 of the study the results of the tests carried out in Delfzijl with the wave overtopping simulator will be discussed.

Finally the conclusions and recommendations are drawn in chapter 6.
2 Wave Overtopping Theory

2.1 Wave run-up
2.2 Overtopping discharge
2.3 Overtopping volumes per wave
2.4 Maximum flow velocity and flow depth on the crest and inner slope
2.5 Conclusions
2.1 Wave run-up

2.1.1 Breaker parameter

The existing wave run-up and overtopping formulae are based on the breaker parameter (or surf similarity parameter). The parameter has been defined by the steepness of the outer slope divided by the root mean square of the wave steepness. The wave steepness is the ratio between the wave height and wave length. For the wave height the significant wave height is used. This wave height is the average of the highest one third of all waves. This is called $H_s$ if the significant wave height is determined from measurements and $H_{m0}$ when this wave height is based on the zero moment of a wave spectrum. In deep water both definitions produce almost the same value, but in shallow water the values can differ 10 to 15%. The other part of the wave steepness is the wave length. The wave length depends on the wave period. For the wave period the spectral mean period $T_{m-1.0}$ is used. This period gives more weight to the longer waves in the spectrum than an average period $T_m$. The longer waves are important for the amount of overtopping. This leads to equation 2-1:

$$\xi_0 = \frac{\tan \alpha}{\sqrt{s_{m-1.0}}} \quad \text{(TAW, 2002), with } s_{m-1.0} = \frac{H_{m0}}{L_0} \text{ and } L_0 = \frac{gT_{m-1.0}^2}{2\pi}$$

Where:

- $\xi_0$ = breaker or surf parameter [-]
- $\alpha$ = angle of the outer slope [-]
- $s_{m-1.0}$ = wave steepness [-]
- $H_{m0}$ = spectral wave height ($H_{m0} = 4\sqrt{m_0}$) [m]
- $m_n$ = nth order moment of the energy density spectrum [-]
- $L_0$ = deep water wave length [m]
- $g$ = acceleration due to gravity [m/s²]
- $T_{m-1.0}$ = spectral wave period ($T_{m-1.0} = \frac{m_n}{m_0}$) [s]

The value of the breaker parameter gives an idea of how the wave breaks on the outer slope. Spilling breaker type, usually found along flat beaches, belong to the lowest values of the breaker parameter, $\xi_0 < 0.5$. Almost no wave energy is reflected back into the sea with this breaker type. A plunging breaker, $0.5 < \xi_0 < 3.3$, is often found at the outer slopes of sea dikes and within the model tests discussed in the present report. Collapsing and surging breaker types correspond with higher values of the breaker parameter. For surging breaker types, $\xi_0 > 5$, more than 50% of the wave energy is reflected back to the sea (in Battjes 2001).

2.1.2 Wave run-up

Overtopping occurs when a wave breaks on the outer slope, runs up and exceeds the crest level. In other words: when the wave run-up of a wave is higher than the crest freeboard the wave will overtop. The difference between the wave run-up and the crest freeboard is an indication for the flow depth, flow velocity and discharge. The wave run-up depends on the breaker parameter and therefore on the angle of the outer slope, the wave height and the wave period. Another parameter which is
important for the determination of the run-up, is the surface of the outer slope. A smooth surface (grass, asphalt) will lead to a higher run-up level than a rough surface (rock). Also a berm influences the run-up height. And last but not least, the angle of the wave attack will effect the run-up level. The wave run-up height exceeded by 2% of the incoming waves is given by $z_{2\%}$. This is the wave run-up level, measured vertically from the still water line (elevation $\eta=0$) which is exceeded by 2% of the incoming waves during a storm. This level has been a design guideline for the dikes in the Netherlands. The general formula of the average wave run-up can now be given by equation 2-2:

$$\frac{z_{2\%}}{H_{m0}} = 1.65 \cdot \gamma_b \cdot \gamma_f \cdot \xi_0 \quad \text{(TAW 2002)}$$

With a maximum for values of the breaker parameter $\xi_0$ larger than 1.75:

$$\frac{z_{2\%}}{H_{m0}} = \gamma_f \cdot \gamma_b \cdot \left( 4.0 - \frac{1.5}{\sqrt{\xi_0}} \right) \quad \text{(TAW 2002)}$$

Where:

- $z_{2\%}$ = run-up level exceeded by 2% of the incoming waves [m]
- $\gamma_b$ = reduction factor for a berm [-]
- $\gamma_f$ = reduction factor for the roughness on the outer slope [-]
- $\gamma_b$ = reduction factor for angled wave attack [-]

In Figure 2-1 the wave run-up is given as a function of the breaker parameter only. The red line shows that the wave run-up level ($z_{2\%}/H_{m0}$) increases linearly with the breaker parameter and from a value of $\xi_{op} = 1.65$ the relation becomes almost constant (equation 2-3). This kink in the graph is because of the fact that from the value $\xi_{op} = 1.65$ for the breaker parameter, a lot of reflection of energy takes place back into the sea. The value 1.65 in equation 1-2 is the slope of the equation which is determined in a empirical way.
2.2 Overtopping discharge

Wave overtopping is the mean discharge per linear meter of width \((q \text{ [l/m/s]})\) that is reaching the inner slope of the dike (or structure). Wave overtopping formulae are mostly exponential functions with the general formula given by Owen (in d’Angremond 2001):

\[
Q^* = \frac{q}{\sqrt{gH_s}} = a \exp(-b \frac{R_c}{\gamma H_s})
\]

The coefficients \(a, b\) have the following values according to the TAW of 2002:

\[
\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan \alpha}} \gamma_{b0} \xi \exp \left( -4.3 \frac{R_c}{H_{m0}} \frac{1}{\gamma_0 \gamma_{b0} \gamma_r \gamma_{vr}} \right) \quad \text{(TAW 2002)}
\]

With a maximum of:

\[
\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp \left( -2.6 \frac{R_c}{H_{m0}} \frac{1}{\gamma_0 \gamma_{r}} \right) \quad \text{(TAW 2002)}
\]

Where:

- \(q\) = average wave overtopping discharge per meter \([\text{m}^3/\text{s/m}]\)
- \(R_c\) = crest freeboard relative to SWL \([\text{m}]\)
- \(\gamma_v\) = influence factor for a vertical wall on slope \([-\text{]}\)

Guidelines (TAW 2002) suggest the following allowable discharges on the inner slope:

- 0.1 l/s/m for sandy soil with a poor grass mat
- 1.0 l/s/m for clayey soil with a reasonable good grass mat
- 10.0 l/s/m for a clay covering and a grass mat according to the requirements for the outer slope or for a armoured inner slope

These guidelines are based on mean discharges, but a volume of 3600 liter in one hour, thus a discharge of 1 l/s caused by several small waves might have a different effect on the dike than the same volume caused by one single wave in one hour.
2.3 Overtopping volumes per wave

The different overtopping volumes during a storm can be described by a Weibull distribution, equation 2-7.

\[ P_v = P(V \geq V) = \exp\left( -\left( \frac{V}{a} \right)^{0.75} \right), \quad \text{with} \quad a = \frac{0.84T_m q}{P_{ov}} \quad (\text{Van der Meer, 2006a}) \quad 2-7 \]

Where:
- \( P_v \) = probability of an overtopping volume
- \( V \) = volume of an overtopping event
- \( P_{ov} \) = probability of an overtopping event

**Example:**

In order to give an example of the Weibull distribution of the different overtopping volumes during a storm a few parameters have been assumed, typical for the Dutch dike coasts:

- Wave height, \( H_s = 2 \text{ m} \)
- Mean wave period, \( T_m = 4.7 \text{ s} \)
- Outer slope, \( \cot \alpha = 4 \)
- Number of waves, \( n_{waves} = 4600 \)

With a crest freeboard of \( R_c = 3.32 \text{ m} \) the overtopping rate is \( P_{ov} = 2.0 \% \), thus 92 waves will overtop. The overtopping discharge \( q = 0.73 \text{ l/s/m} \) has been determined with equation 2-5 and the Weibull distribution with equation 2-7. The largest overtopping event is \( V_{max} = 883 \text{ l/m} \). The overtopping distribution of the example has been plotted in Figure 2-2:

![Figure 2-2 Distribution of overtopping volumes](image_url)
2.4 Maximum flow velocity and flow depth on the crest and inner slope

Recent studies have been carried out to determine overtopping velocities and flow depths. Schütttrumpf (2001b) and Van Gent (2002) give formulae for the maximum velocity and maximum flow depth on the crest and the inner slope. The following section will describe in detail the knowledge gained from tests by Schütttrumpf and Van Gent and the discrepancy between the results of their researches. The theories are based on the difference between the fictive 2% wave run-up $z_{2\%}$ and the crest freeboard $R_c$. Indicated with the letter "A" in Figure 2-3. In fact is "A" the remainder of the wave run-up and an initial measure to quantify the flow depths and velocities on the crest, thus the higher the remainder of the run-up, the higher the velocities and flow depths.

![Figure 2-3 Remainder of the wave run-up: "A"](image1)

![Figure 2-4 Definition of $x_c$ on the crest](image2)

2.4.1 Theory of Schütttrumpf

Formulae were derived by Schütttrumpf for maximum flow depths and maximum flow velocities on the crest and inner slope. All formulae have been based on the 2% wave run-up. Schütttrumpf does not use equations 2-2 and 2-3 for the 2% wave run-up. He has determined the 2% wave run-up for irregular waves with a (continues) hyperbolic function, see equation 2-8.

$$\frac{z_{2\%}}{\gamma_1\gamma_b H_s} = 3 \tanh(0.655 \xi)$$  \hspace{1cm} (Schütttrumpf 2001a) \hspace{1cm} 2-8

The formulae for maximum flow depth and maximum velocity are listed below. A difference is made between the seaside of the crest, landside of the crest and the inner slope.

2.4.1.1 Seaward side of the crest:

The maximum flow depth exceeded by 2% of the incident waves at the seaward side of the crest (position 1) is presented in equation 2-9 (definition of $x_c$ is shown in Figure 2-4):

$$\frac{h_{2\%}(x_c = 0)}{H_s} = c_{h_{2\%}} \left( \frac{Z_{2\%} - R_c}{H_s} \right)$$  \hspace{1cm} (Schütttrumpf 2001b) \hspace{1cm} 2-9
Wave Overtopping Theory

Where:
- $h_{2\%}$ = maximum flow depth exceeded by 2% of the incident waves [m]
- $x_c$ = x-coordinate on the crest [m]
- $c'_{h,2\%}$ = empirical coefficient (0.33) [-]

And for the maximum flow velocities on the seaside of the crest equation 2-10 was found (based on a simple energy equation (Schüttrumpf 2001a):

$$\frac{u_{2\%}(x_c = 0)}{\sqrt{gH_s}} = c'_{u,2\%}\sqrt{\frac{z_{2\%} - R_c}{H_s}}$$  \hspace{1cm} (Schüttrumpf 2001b)  \hspace{1cm} 2-10

Where:
- $u_{2\%}$ = maximum velocity exceeded by 2% of the incident waves [m/s]
- $c'_{u,2\%}$ = empirical coefficient (1.37) [-]

Details of the derivation of equations 2-9 and 2-10 are described by Schüttrumpf (Schüttrumpf 2001a) and confirmed by Schüttrumpf (Schüttrumpf 2001b) and Van Gent (Van Gent 2002).

2.4.1.2 Landward side of the crest:
The development of the layer thickness on the crest depends on the width of the crest $B_c$ and the x-coordinate on the crest $x_c$. According to Schüttrumpf the flow depth on the crest at every position can therefore be described in equation 2-11:

$$\frac{h_{2\%}(x_c)}{H_s} = c''_{h,2\%}\left(\frac{z_{2\%} - R_c}{H_s}\right)\exp\left(-c''_{h,2\%}\frac{x_c}{B_c}\right)$$  \hspace{1cm} (Schüttrumpf 2001b)  \hspace{1cm} 2-11

Where:
- $h_{2\%}$ = maximum flow depth exceeded by 2% of the incident waves [m]
- $B_c$ = crest width [m]
- $c''_{h,2\%}$ = empirical coefficient (0.75 - 1.11) [-]

Thus the alteration of the maximum flow depth on a dike crest is considered by Schüttrumpf to be an exponential function. The maximum velocity remains almost constant on the dike and is only influenced by the roughness of the dike. The maximum flow velocity is given in equation 2-12:

$$\frac{u_{2\%}(x_c)}{\sqrt{gH_s}} = c''_{u,2\%}\sqrt{\frac{z_{2\%} - R_c}{H_s}} \exp\left(-f\frac{B_c}{2h_{2\%}(x_c)}\right)$$  \hspace{1cm} (Schüttrumpf 2001b)  \hspace{1cm} 2-12

Where:
- $u_{2\%}$ = maximum velocity exceeded by 2% of the incident waves [m/s]
- $f$ = friction coefficient (= 0.02 for smooth coverage and grass) [-]
- $c''_{u,2\%}$ = empirical coefficient (1.37) [-]
The maximum velocity in this formula depends on the maximum flow depth on the crest.

2.4.1.3 Inner slope

A theoretical function was developed by Schüttrumpf based on the momentum equation and the continuity equation. This function can only be solved iteratively. The model assumes constant flow and hydrostatic pressure at the surface of the dike. Boundary conditions are the flow depth and velocity at the landward side of the crest. The formula derived by Schüttrumpf for the velocity on the inner slope is given in equation 2-13.

\[
    u_n(s) = \frac{u_0 + \frac{k_1 h_0}{f} \tanh \left( \frac{k_1 k_2}{2} \right)}{1 + \frac{f \cdot u_0}{h_0 k_1} \tanh \left( \frac{k_1 k_2}{2} \right)} \quad \text{(Schüttrumpf 2001a)}
\]

with: \( k_1 = \sqrt{\frac{2f \cdot g \sin \beta}{h_0}} \) and \( k_2 = -\frac{u_0}{g \sin \beta} + \frac{u_0^2}{\sqrt{(g \sin \beta)^2 + 2s}} \)

Where:

- \( u_n \): velocity at the inner slope \([\text{m/s}]\)
- \( s \): position along the inner slope \([\text{m}]\)
- \( u_0 \): velocity at the landward side of the crest \([\text{m/s}]\)
- \( h_0 \): flow depth at the landward side of the crest \([\text{m}]\)
- \( \beta \): inner slope angle \([\text{o}]\)

The flow depth can be determined with the local velocity, see equation 2-14.

\[
    h_n(s) = \frac{u_n h_0}{u(s)} \quad \text{2-14}
\]

Where:

- \( h_n \): flow depth at the inner slope \([\text{m}]\)

2.4.2 Theory of Van Gent

The hydrodynamic processes on the dike can be determined with several formulae. All these formulae depend on the 2% wave run-up values. The research carried out by Van Gent has been performed with different equations for the wave run-up value than mentioned in section 2.1.2. Van Gent uses for the determination of the 2% wave run-up equations 2-15 and 2-16 below.

\[
    \frac{z_{2\%}}{\gamma_1 \gamma_p H_z} = c_0 \cdot \xi_0 \quad \text{for} \quad \xi_0 \leq \rho \quad \text{(Van Gent 2002)}
\]

\[
    \frac{z_{2\%}}{\gamma_1 \gamma_p H_z} = c_0 \cdot \xi_0 \quad \text{(Van Gent 2002)}
\]
\[
\frac{Z_{2\%}}{\gamma_l \gamma_h H_s} = \frac{c_1 - c_2}{c_0} \quad \text{for} \quad \xi_0 > p \quad \text{(Van Gent 2002)} \tag{2-16}
\]

With: \( p = 0.5 \frac{c_1}{c_0} \)

The parameters for the 2\% run-up equations are listed below in Table 2-1. The quantities of the parameters have been determined by Van Gent in his study to wave run-up on dikes with shallow foreshores.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>( c_0 )</th>
<th>( c_1 )</th>
<th>( c_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Z_{2%} )</td>
<td>1.35</td>
<td>4.7</td>
<td>4.1</td>
</tr>
</tbody>
</table>

*Table 2-1 Parameters for the 2\% run-up values by Van Gent (Van Gent 2002)*

The formulae found by Van Gent for maximum flow depths and maximum flow velocities on the crest and inner slope will be discussed in the upcoming sections.

**2.4.2.1 Seaward side of the crest:**

The maximum flow depth and maximum velocity equations exceeded by 2\% of the incident waves at the seaward side of the crest (position 1) are given with:

\[
\frac{h_{2\%}(x_c = 0)}{H_s} = c_{h,2\%} \left( \frac{Z_{2\%} - R_c}{\gamma_l H_s} \right) \quad \text{(Van Gent 2002)} \tag{2-17}
\]

\[
\frac{u_{2\%}(x_c = 0)}{\sqrt{g} H_s} = c_{u,2\%} \sqrt{Z_{2\%} - R_c} \quad \text{(Van Gent 2002)} \tag{2-18}
\]

Where:

- \( h_{2\%} \) = maximum flow depth exceeded by 2\% of the incident waves \([\text{m}]\)
- \( u_{2\%} \) = maximum velocity exceeded by 2\% of the incident waves \([\text{m/s}]\)
- \( x_c \) = x-coordinate on the crest \([\text{m}]\)
- \( c_{h,2\%} \) = empirical coefficient (0.15) \([-]\)
- \( c_{u,2\%} \) = empirical coefficient (1.30) \([-]\)
- \( \gamma_l \) = influence factor for the roughness on the outer slope \([-]\)

**2.4.2.2 Landward side of the crest:**

Van Gent developed formulae for the landward side of the crest. Maximum flow depths are 33\% lower at the landward side than at the seaward side. Equation 2-20 is valid for a crest width larger than the wave height \( (B_c > H_{mo}) \).

\[
\frac{h_{2\%}(x_c = B_c)}{H_s} = c_{h,2\%} \left( \frac{Z_{2\%} - R_c}{\gamma_l H_s} \right) \quad \text{(Van Gent 2002)} \tag{2-19}
\]

\[
\frac{u_{2\%}(x_c = B_c)}{\sqrt{g} H_s} = c_{u,2\%} \sqrt{\gamma_l} \sqrt{Z_{2\%} - R_c} \left( 1 + c_{u,2\%} \frac{B_c}{H_s} \right) \quad \text{(Van Gent 2002)} \tag{2-20}
\]
Where:

- \( h_{2\%} \) = maximum flow depth exceeded by 2% of the incident waves [m]
- \( u_{2\%} \) = maximum velocity exceeded by 2% of the incident waves [m/s]
- \( C_{\text{h},2\%} \) = empirical coefficient (0.10) [-]
- \( C_{\text{u},2\%} \) = empirical coefficient (1.70) [-]
- \( C_{\text{u}',2\%} \) = empirical coefficient (0.10) [-]
- \( \gamma_l \) = influence factor due to friction on the outer slope [-]
- \( \gamma_c \) = influence factor due to friction at crest [-]

2.4.2.3 Wave overtopping parameters along inner slopes of dikes

At the landward slope super-critical flow is assumed. A stationary one-dimensional shallow-water equation along the inner slope can be written as equation 2-21:

\[
u \frac{du}{ds} + g(\cos \beta \frac{dh}{ds} - \sin \beta) + \frac{1}{2} f_i \frac{u^2}{h} = 0, \text{ where } s=0 \text{ at the crest.} \quad 2-21
\]

This equation can be solved to determine the flow depth and velocity on the inner slope:

\[
h_u(s) = \left( h_0 u_0 \right) \left( \frac{b_1}{b_2} + \mu \exp(-3b_1 b_2^2 s) \right) \quad (\text{Van Gent 2002}) \quad 2-22
\]

\[
u_u(s) = \frac{b_1}{b_2} + \mu \exp(-3b_1 b_2^2 s) \quad (\text{Van Gent 2002}) \quad 2-23
\]

With:

\[
b_1 = \sqrt{g \sin \beta} \quad b_2 = \sqrt{\frac{0.5 f_i}{h_0 u_0}} \quad \mu = u_0 - \frac{b_1}{b_2}
\]

Where:

\( f_i \) = influence factor due to friction on the inner slope [-]

For smooth surfaces is the friction coefficient 0.005.
2.4.3 Overall view of the theories of Schüttrumpf and Van Gent

Schüttrumpf and Van Gent have collectively written a paper about their wave overtopping studies (Schüttrumpf, Van Gent 2003) and combined their theories. In this section an overall view of their combined theories is given.

2.4.3.1 Wave run-up

The hyperbolic equation 2-8 by Schüttrumpf and equations 2-15 and 2-16 by Van Gent as well as equation 2-2 and 2-3 of the TAW for the 2% wave run-up have been plotted in Figure 2-5.

![Wave run-up](image)

**Figure 2-5 2% Wave run up by Schüttrumpf, Van Gent and the TAW**

Van Gent and the TAW use two different equations for a transition between plunging and surging breaking waves while Schüttrumpf uses one (hyperbolic) equation. The linear equation for 2% wave run-up for plunging waves by Van Gent and TAW is repeated below:

\[
\frac{z_{2\%}}{\gamma_1 \gamma_\rho H_s} = c_0 \cdot \xi_0 \quad \text{for} \quad \xi_0 \leq p \quad \text{(Van Gent 2002)}
\]

(2-24)

The parameters, corresponding to the 2% wave run-up were \(c_0 = 1.35\) and \(p = 1.5\) by Van Gent (the red line in Figure 2-5 is plotted with these parameters). The value \(c_0 = 1.65\) in the TAW. Schüttrumpf has determined higher values for the 2% wave run-up for breaker parameter \(\xi \leq 1.5\), indicated with the blue line in Figure 2-5 and corresponds quite well with the TAW. For higher values of the breaker parameter a large deviation occurs between all studies. The present study focuses on breaker parameters in the plunging range, thus in the lower range where: \(\xi \leq 1.5\).
2.4.3.2 Maximum flow depth and flow velocity

The general formulae for maximum flow depth and maximum flow velocity at the seaward side of the crest have been described in equation 2-25 and equation 2-26:

\[
\frac{h_{2\%}(x_c = 0)}{H_s} = c'_{h,2\%}\left(\frac{z_{2\%} - R_c}{\gamma_s H_s}\right) \quad \text{(Schüttrumpf, Van Gent 2003)} \tag{2-25}
\]

\[
\frac{u_{2\%}(x_c = 0)}{\sqrt{gH_s}} = c'_{u,2\%}\left(\frac{z_{2\%} - R_c}{\gamma_s H_s}\right)^{0.5} \quad \text{(Schüttrumpf, Van Gent 2003)} \tag{2-26}
\]

The coefficients \(c'_{h,2\%}\) and \(c'_{u,2\%}\) have been determined by Schüttrumpf and Van Gent in different physical model tests. The values are listed in Table 2-2.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(c'_{h,2%})</td>
<td>0.33</td>
<td>0.15</td>
</tr>
<tr>
<td>(c'_{u,2%})</td>
<td>1.37</td>
<td>1.30</td>
</tr>
</tbody>
</table>

Table 2-2 Coefficients of Schüttrumpf and Van Gent at the seaward side of the crest

According to Schüttrumpf and Van Gent the discrepancy between the results can be explained by different model set-ups (different dike geometries and instruments) and test programs (Schüttrumpf, Van Gent 2003).

On the landward side of the crest the general formulae can be described by equations 2-27 and 2-28:

\[
\frac{h_{2\%}(x_c)}{h_{2\%}(x_c = 0)} = \exp\left(-c''_{h,2\%}\frac{X_c}{B_c}\right) \quad \text{(Schüttrumpf, Van Gent 2003)} \tag{2-27}
\]

\[
\frac{u_{2\%}(x_c)}{u_{2\%}(x_c = 0)} = \exp\left(-c''_{u,2\%}\frac{X_c \cdot f}{h_{2\%}(x_c)}\right) \quad \text{(Schüttrumpf, Van Gent 2003)} \tag{2-28}
\]

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(c''_{h,2%})</td>
<td>0.89</td>
<td>0.40</td>
</tr>
<tr>
<td>(c''_{u,2%})</td>
<td>0.5</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 2-3 Coefficients of Schüttrumpf and Van Gent at the landward side of the crest

On the inner slope Schüttrumpf uses a Navier-Stokes equation and Van Gent a One-dimensional shallow water equation. The One-dimensional shallow water equation of Van Gent was preferred above the Navier-Stokes equation (Schüttrumpf, Van Gent 2003). The last mentioned equation has to be solved iteratively while the One-dimensional equation is even easier to use for practical solutions. The one-
dimensional shallow water equation for the maximum velocity on the inner slope has been given in equation 2-29.

\[ u_{m}(s) = \frac{b_{1}}{b_{2}} + \mu \exp(-3b_{1}b_{2}^{2}s) \]  
(Schüttrumpf, Van Gent 2003)  
2-29

With:

\[ b_{1} = \sqrt{g \sin \beta} \quad b_{2} = \sqrt{\frac{0.5f_{d}}{h_{0}u_{0}}} \quad \mu = u_{0} - \frac{b_{1}}{b_{2}} \]

Equation 2-29 has an asymptote for \( s \to \infty \), equation 2-30:

\[ \lim_{s \to \infty} u_{m}(s) = \sqrt{\frac{2g \cdot h_{0} \cdot u_{0} \cdot \sin \beta}{f}} \]  
(Schüttrumpf, Van Gent 2003)  
2-30

With the continuity equation the maximum flow depth can be determined, see equation 2-31.

\[ h_{m}(s) = \frac{h_{0}u_{0}}{u_{m}(s)} \]  
(Schüttrumpf, Van Gent 2003)  
2-31

2.5 Conclusion

Schüttrumpf determined the coefficient \( c'_{h,2\%} \) to be a factor 2.2 higher than Van Gent. In Figure 2-5 one can see that the 2% wave run-up for \( \xi \leq 1.5 \) is higher by Schüttrumpf than by Van Gent. This 2% wave run-up value is very dominant in the formulae for calculation of flow depths and velocities on the dike. The discrepancy of the coefficient \( c'_{h,2\%} \) between both studies would even be more than a factor 2.2, if in both studies the same wave run-up equation had been used. In other words: there occurred a large difference in flow depths on the crest during the experiments. According to Schüttrumpf and Van Gent; this discrepancy between the studies has been caused by the differences in the model set-up and test program. The next chapter will focus on these differences between the experiments. Also the data-processing in both studies will be analysed as well.
3 The experiments carried out by Schüttrumpf and Van Gent

3.1 Introduction
3.2 Tests Schüttrumpf
3.3 Tests Van Gent
3.4 Test programmes
3.5 Filter techniques
3.6 Regular waves analysis
3.7 Data analysis Schüttrumpf
3.8 Data analysis Van Gent
3.9 Conclusions
3.1 Introduction

In this section the experiments of Schüttrumpf and Van Gent will be analysed. A part of the test data of both experiments has been made available for the present study. This data has been used to validate the measurements. This will be described in the first two sections. The different test programmes will be discussed hereafter. In section 3.5 filter techniques are described and validated with regular wave data of Schüttrumpf. In the last two sections of this chapter this filter technique is used to analyse the data that was available in the present study.

3.2 Tests Schüttrumpf

In small scale model wave overtopping tests have been carried out by Schüttrumpf (Schüttrumpf 2001a). The results were validated and extended by large scale model tests carried out in the large wave flume in Hannover (Schüttrumpf 2001b). The tests were part of the project "Loading of the inner slope of sea dikes by wave overtopping". Data from that test have been made available and were used for the present overtopping study. In this section the model set-up will be described and the different instruments used for the measurements will be discussed. The validity of the measurements will be reflected by the use of the data from the tests with regular waves.

![Figure 3-1 Model set-up GWK Hannover](image)

3.2.1 Model set-up

The tests have been carried out in the Large Wave Flume or GWK (Grosser Wellenkanal) in Hannover. The flume has a total length of 324 m, a width of 5 m and a depth of 7 m. Regular waves can be generated in the flume with a wave height up to 2.0 m. A model of a dike has been built in the flume (Figure 3-1). The tests were carried in order to gain more insight in the hydrodynamic processes on a dike during wave overtopping. To achieve this objective, more than a hundred measuring devices were installed. These instruments measured the surface elevation, pressure (hydrostatic or dynamic), the velocity, the flow depth and the discharge of the water. Figure 3-2 gives an overall view of the different positions on the dike where a part of the measure instruments were installed. On every of the ten positions indicated in the figure velocity propellers, flow depth gauges and pressure cells were installed. Each of the instruments has its own characteristics and restrictions. These characteristics are essential for analysing the data. During the tests in Hannover three video cameras were also installed. This video footage is a good support in
order to estimate the usefulness of the data at the different positions. The different instruments will be described in the upcoming sections by examples of wave data measured with regular waves.

![Horizontal distance to begin wave flume](image)

**Figure 3-2 Positions of velocity, pressure and flow depth instruments on crest and inner slope**

### 3.2.2 Flow velocity measurements

Velocity has been measured during the test by several micro propellers (Figure 3-3). The propellers generate electricity like a dynamo. The resistance is assumed negligible. Two kinds of micro propellers were used, different in size and in range. The bigger ones have a diameter of 2.2 cm and can measure velocities up to 10 m/s. The smaller ones, with a diameter of 1.5 cm, have a range up to 5 m/s. The instruments were installed about one centimetre above the surface. Flow depths of less than one centimetre couldn’t be measured, but also velocity data measured at small flow depths, whereby the propeller isn’t fully submerged, must be used cautiously. On the crest the (five) propellers were installed in the centre (in the middle between the walls). At the inner slope the propellers were installed on the right hand side, close to the wall. This is expected to have consequences for the measured data on the inner slope, because the flow on the inner slope is concentrated in the middle, like a kind of tongue (Figure 3-4). This tongue shape may be caused by the wall roughness and/or because of the fact that the dike model is manmade. The result is that the flow depths in the centre will be thicker than on the sites. If the propellers aren’t fully submerged, they won’t measure correctly. They also measure the velocity too late and too short, because the main body has already passed by.

![Micro propeller dimensions](image)

**Figure 3-3 Micro propeller dimensions (Source, Schüttrumpf, 2001b)**

![Tongue of overtopping water](image)

**Figure 3-4 Tongue of overtopping water on the inner slope. One of the velocity propellers is indicated by the arrow.**
A closer look at the measured data will give some insight in what happens to the velocity of the overtopping water. In Figure 3-5 some parameters are given of a test carried out on the 31st of May 2001. The wave height was 1 meter and the wave period was 7.5 seconds. In Figure 3-5 the velocity of the water is shown as function of time at the different positions. In the figure one can see that the data from position one, at the seaside of the crest, is not a good reference point. The data is disturbed by a lot of air entrainment during the overtopping event. The velocity propeller cannot recognize a flow direction, thus a peak at the end of an event can also be generated by water that flows back into the flume. In addition, the maximum velocity remains almost constant on the crest. The water accelerates on the inner slope due to gravity.
3.2.3 Validation flow velocity data
A way to validate the velocity data is to compare the measured maximum velocities with the front velocity of an overtopping event.

\[
\bar{u}_{\text{max}} = \frac{u_{\text{max}}(A) + u_{\text{max}}(B)}{2}
\]

\[
u_{\text{front}} = \frac{dx}{dt} = \frac{x_t(B) - x_t(A)}{t_0(B) - t_0(A)}
\]

Where:
- \(u_{\text{max}}\) = mean maximum velocity [m/s]
- \(u_{\text{max}}(x)\) = maximum velocity on position \(x\) [m/s]
- \(u_{\text{front}}\) = front velocity [m/s]
- \(x_t(x)\) = distance on the crest of position \(x\) [m]
- \(t_0(x)\) = start time data record on position \(x\) [s]

The mean maximum velocity (equation 3-1) and the front velocity (equation 3-2) on the crest have been determined between positions 3 and 5 (Figure 3-2). The comparison has been made at the end of the crest in order to have little influence from the high turbulent seaside. On the inner slope the comparison was made between positions 7 and 8.

![Figure 3-6 Velocity comparison on the crest (left) and the inner slope (right)](image)

In the left graph in Figure 3-6 it can be seen that on the crest a high deviation is present when the velocity is beyond 2.5 m/s. In fact the mean maximum of the velocity does not become higher than 3 m/s while the front velocity even exceeds above 5 m/s. This is a very large deviation.

The graph on the right in Figure 3-6 is a comparison of the velocity on the inner slope. Front velocity and mean maximum velocity are almost equal in this case. It seems that the deviation in the front velocity and the mean maximum velocity on the crest must have something to do with the location of the velocity propeller, because from the measurements on the inner slope it can be concluded that the calibration of
the propellers have been carried out well. It can have something to do with the vertical distribution of the velocity and thus the position of the velocity propeller above the bottom. The deviation is above 2.5 m/s and it is important to know what flow depth corresponds to this velocity. Figure 3-7 gives the front velocities with the corresponding maximum flow depth.

![Figure 3-7 Front velocities with corresponding maximum flow depths on the crest](image)

The propellers were situated 1 cm above the bottom and have a diameter of 2.2 cm, indicated with the black horizontal dotted lines in Figure 3-7. The red vertical dotted line is the 2.5 m/s velocity line. The figure does not show a direct correlation between the maximum front velocity and maximum flow depth, however the velocity propeller has been submerged by the flow during the moment the maximum passed by.

![Figure 3-8 Velocity data record on the inner slope](image)

On the inner slope the flow has a more stationary character, in comparison to the flow on the crest. The maximum velocity of one overtopping event occurs for a longer period on the inner slope (Figure 3-8), while the maximum velocity on the
The experiments carried out by Schüttrumpf and Van Gent
crest only exists for one tenth of a second. That is probably the reason why the
velocity can be measured correctly on the inner slope.

The vertical velocity distribution of a turbulent \textit{stationary} flow is described in 1932 by
Nikuradse (in Battjes 2002) with the following formula:

\[
\frac{u(y)}{u_*} = \frac{1}{\kappa} \ln \left[ \frac{y}{y_0} \right]
\]

3-3

Where:
- \( u(y) \) = velocity on position \( y \) [m/s]
- \( u_* \) = shear stress velocity [m/s]
- \( \kappa \) = Von Karman coefficient (which has a value of 0.4) [-]
- \( y_0 \) = integration coefficient [m]

The shear stress velocity for turbulent flows is given as:

\[
u_* = \sqrt{\frac{\tau_0}{\rho}} \quad \text{with:} \quad \tau_0 = \rho c_r u^2
\]

3-4

Where:
- \( \tau_0 \) = shear stress [N/m²]
- \( \rho \) = density [kg/m³]
- \( c_r \) = friction coefficient [-]

The friction coefficient \( c_r \) corresponds with the Reynolds-number of the flow. The
integration coefficient \( y_0 \) in equation 3-3 is based on the nominal thickness of the
boundary layer:

\[
y_0 = \frac{\delta}{105} \quad \text{with:} \quad \delta = 11.6 \frac{v}{u_*}
\]

3-5

Where:
- \( \delta \) = boundary layer [m]
- \( v \) = viscosity [Pas]

In Figure 3-9 are the average velocity profiles drawn for two turbulent stationary
flows with:

\[
\begin{align*}
&u_0 = 2.5 \text{ m/s, } h = 0.06, \ c_r = 0.0016 \\
&u_0 = 4.0 \text{ m/s, } h = 0.12, \ c_r = 0.0015
\end{align*}
\]

The dotted lines indicate the position of the velocity propeller. The boundary layer is
small and does hardly influence the measurements.
On the crest a non-stationary flow occurs, therefore the vertical velocity distribution is different. The boundary layer cannot develop and is assumed to look like the blue lines in Figure 3-10.

The blue line on the right in Figure 3-10 has a maximum velocity of 4.5 m/s. The measured velocity will be beneath 3 m/s, due to location of the velocity propellers above the dike model.

The accuracy of the front velocity will be much higher than the measured velocity on one single spot in the vertical distribution profile. The front velocity must therefore be used on the crest. Schüttrumpf has used the measured velocities during his data-processing.
3.2.4 Flow depth measurements

Flow depth has been measured by depth gauges and pressure cells. The gauges consist of several pins and a wire (Figure 3-11). A potential difference has been applied between the two sides. When one or more of the pins will be submerged by the flow, the resistance will be lower and the flow depth can be calculated. The distance between the pins was two centimetre, starting with the first pin on one centimetre. Besides these depth gauges static pressure cells were installed. With these pressure cells the flow depth could be derived as well. A third measuring method was done with video cameras. The cameras gave the possibility to have an extra check on the measured flow depths. Because the video tapes can be played frame by frame, an insight of the whole overtopping process was given.

The flow depth gauges were positioned on the same spots as the velocity propellers (see Figure 3-2); in the centre on the crest and at the wall on the inner slope. This positioning gave problems during measuring at the inner slope as with the velocity data, due to the fact that the main tongue of the wave passes by in the centre (Figure 3-4). The pressure cells on the other hand, were situated in the middle of the model between the two walls on the crest as well as on the inner slope. Probably the pressure cells will be much more useful for the data-processing than the depth gauges on the inner slope, because the cells have measured the main wave tongue.

3.2.5 Validation flow depth data

As stated above: the flow depth has been measured in three ways: by depth gauges, by pressure cells and by video cameras. A validation of the flow depth data can be performed in two ways. One: comparing the data of the depth gauges with the data measured by the pressure cells. Two: comparing the data with the video footage. In Figure 3-12 two plots of the data from a wave gauge are shown in comparison with
the video footage. The flow depths from the video were read from the wall of the wave flume every two frames. That means every 0.08 seconds, since 1 second contains 25 video frames. The data were classified in integers. Especially on the seaside of the crest the turbulence of the flow makes it hard to read the flow depth correctly on the wall. However, it can be seen that the lines match pretty well in Figure 3-12 (with correlation coefficients of 0.9 and 0.8). Although this is a very global comparison, it gives a good indication that the measurements with the depth gauge are quite correct and that will definitely not generate a difference of a factor 2.2 with data of Van Gent.

Figure 3-13 shows the difference between the measurements of the depth gauge and the pressure cell of position "3" (middle of the crest) of test 31050009.

![Figure 3-13 Depth measurements at the middle of the crest.](image)

One can see that the depth gauge gives a gradual signal, while the pressure cell has a much more direct signal and thus a larger resolution. This direct signal can especially be seen in the last three seconds of the graph. The steps in the signal of the depth gauge are caused by the pins situated two centimetres above each other. The correlation \( r = 0.9 \) is good between the signals. On the other hand, the difference between the maxima is very large, the depth gauge and the pressure cell have a maximum of 18 cm and 14 cm respectively. It looks like the pressure cell misses the maximum peak or the depth gauge does not measure correctly.

Figure 3-14 shows two data records of test 09050006 at the crest. Besides the fact that the maxima of the pressure cell are smaller than the ones from the depth gauge, some overtopping events do not even exist on the pressure cell’s data record. The ones at \( t=380 \) and \( t=432 \) are only measured by the depth gauge. The total correlation between the pressure cell and the wave gauge of the whole data record for this test is \( r = 0.58 \). The large peak in the data record of the depth gauge at \( t=385 \) is probably caused by spray and will be filtered away during data-processing.
Due to the lower maxima and the missing overtopping events in the data record of the pressure cell, the pressure cell data record will not be used. Flow depths measured with the depth gauges showed a good similarity with the video footage, therefore the data records of the depth gauges will be used in the further analysis in the present report.
3.3 Tests Van Gent
Van Gent carried out wave overtopping tests (Van Gent 2002). A part of the data has been made available for the present study. In this section the measurements will be discussed and validated.

3.3.1 Model set-up
The tests were small scale tests and carried out in autumn 2001. The length of the used wave flume is 55 m, and the flume has a width of 1m and a height of 1.2 m. Figure 3-15 gives a schematization of the dike configurations that were used by Van Gent in his tests.

![Figure 3-15 Model set-up tests Van Gent (source: Van Gent, 2002)](image)

Different water levels, different crest widths and different inner slope angles were used. In the present report only data from configuration (A) have been used. The crest width in this configuration was 0.2 m and the outer slope was 1:2.5. The blue dots in Figure 3-15 roughly indicate the locations of the measurement devices, there were two positions on the crest and three on the inner slope. The overtopping discharge was measured in the overtopping box as well.

![Figure 3-16 Positions of instruments on the dike model](image)
The experiments carried out by Schüttrumpf and Van Gent

Figure 3-16 gives the exact positions of the instruments on the dike model used in the tests of Van Gent for configuration (A). The chosen position numbers are more or less corresponding with the position numbers of Schüttrumpf test (Figure 3-2).

3.3.2 Flow velocity measurements
The flow velocity was measured with small velocity propellers. The propellers had a diameter of 1.0 cm and were installed 0.1 cm above the bottom of the dike model. The propellers were capable of measuring velocities in the range of 0.5 m/s to 4 m/s.

3.3.3 Validation flow velocity data
The flow velocity data have been checked in the same way as the data of Schüttrumpf, shown in Figure 3-17.

![Figure 3-17 velocity check data Van Gent](image)

One can conclude from Figure 3-17 that the velocity propellers measured the velocity correct during the experiments of Van Gent. One of the main reasons that during these test no large deviation occurred, like in the tests of Schüttrumpf, is due to the occurrence of lower velocities and therefore lower flow depths. The propellers do not experience disadvantages of the vertical velocity distribution. Van Gent’s velocity measurements carried out with velocity propellers are adequate.
3.3.4 Flow depth data
The flow depths have been measured with depth gauges which consist of two wires, see Figure 3-18. A potential difference was applied between the two wires. Water between the two wires decreases the electrical resistance and a higher voltage will be measured between the wires. In this way the flow depth can be calculated. These instruments can be used to measure the water elevation of waves in the flume itself in a simple manner. Problems with this kind of instruments can occur when a flow is measured. It can happen that water with a certain velocity is running up onto the wires and decreasing the electrical resistance. In that case the gauges measure higher flow depths than in reality occurred.

3.3.5 Validation flow depth data
Van Gent did not use extra instruments like pressure cells to measure the flow depth. Besides that, there was no video material available for the present study. The only way to validate the flow depth data, was to compare it with the measured discharge in the overtopping reservoir at the landside of the dike model.

![Figure 3-18 depth gauge](Source: Van Gent 2002)

![Figure 3-19 measured (red) and calculated (blue) volume of part test 1.07](image)

Figure 3-19 shows the measured and calculated volumes of a part of test 1.07 of Van Gent. The measured volume is the volume measured during the test, in this case
The experiments carried out by Schüttrumpf and Van Gent

with the water level in the overtopping reservoir. The calculated volume is the volume that can be calculated with the integration over time of the measured velocity and flow depth data (full definitions in appendix A). One can see that the calculated volume is lower than the actual volume. This can be explained by the fact that wave gauges and velocity propellers do not measure very small flow depths. Thus some of the overtopping volume cannot be calculated, while all the overtopping water will end up in the reservoir behind the dike model and is measured. This makes the red line steeper. However, all the individual overtopping events can be found back in both lines. Although an extra validation of the flow depth data couldn’t be made, it was concluded that, considering the difference between the measured and calculated discharge, flow depth measurements have been carried out correct during the tests.

3.4 Test programmes

Van Gent carried out 118 tests with irregular waves, 56 of them were double-peaked wave energy spectra which were obtained by super positioning two TMA-spectra (see appendix B). All other tests were normal TMA-spectra. In all tests were at least 1000 waves generated. Schüttrumpf also carried out 24 tests with regular waves. 33 tests were carried out with TMA spectra and 146 tests with natural wave spectra. The natural wave spectra are mostly multi peaked and based on the theoretical TMA spectra. Multi peaked spectra are typical for the German and Dutch North Sea coast. Table 3-1 indicates the test programmes of Schüttrumpf and Van Gent. For all wave parameters and overtopping parameters the minima and maxima are presented.

<table>
<thead>
<tr>
<th>d [m]</th>
<th>R_c [m]</th>
<th>H_s [m]</th>
<th>T_m-1.0 [s]</th>
<th>T_p [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>min</td>
<td>max</td>
<td>min</td>
<td>max</td>
<td>min</td>
</tr>
<tr>
<td>Schüttrumpf</td>
<td>3.50</td>
<td>5.00</td>
<td>1.00</td>
<td>2.50</td>
</tr>
<tr>
<td>Van Gent</td>
<td>0.30</td>
<td>0.50</td>
<td>0.10</td>
<td>0.30</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>s_om [-]</th>
<th>ε_50-1 [-]</th>
<th>z_2% [m]</th>
<th>P EVT [%]</th>
<th>q_{meas} [l/s/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>min</td>
<td>max</td>
<td>min</td>
<td>max</td>
<td>min</td>
</tr>
<tr>
<td>Schüttrumpf</td>
<td>0.01</td>
<td>0.06</td>
<td>0.73</td>
<td>3.57</td>
</tr>
<tr>
<td>Van Gent</td>
<td>0.02</td>
<td>0.03</td>
<td>1.16</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Table 3-1 Ranges of parameters varied by Schüttrumpf and Van Gent in their tests
3.5 Filter techniques

Figure 3-20 shows three data records of one overtopping test. The upper record was made by a velocity propeller, the middle one by a pressure cell and the third data record by a depth gauge. All data records were measured in the middle of the crest.

Three circles in the upper data record indicate three maximum velocities which deviate, compared to the rest of the data. The green line indicates a more convincing maximum velocity. These peaks do have a big influence in the calculations of the maxima and the data have to be filtered in order to get rid of the "incorrect" data. In the record of the pressure cell (red in Figure 3-20) a strange peak is also visible. Probably this is not a correct flow depth. The third record, the data record from the depth gauge, also shows two deviations. This is caused by spray of the overtopping water, briefly reaching the higher pins on the depth gauge. Figure 3-21 is a frame of a video which has been made during the test. Here one can see this spray is very well visible.
3.5.1 Mean-phase filtering

Regular waves can best be filtered by the mean-phase principle. This means that an average is taken of the data with respect to the phase of the wave. In the present study data records of tests with irregular waves will be discussed as well. Mean-phase filtering cannot be used for irregular waves. Therefore moving-average filtering will be applied on the regular wave data. This moving average filtering can also be used for irregular waves. Both filter techniques can be compared to each other concerning the regular waves data.

![Flow velocity and depth graphs](image)

*Figure 3-22 Filtering by the mean-phase principle of test 31050010 with regular waves (2 upper figures are raw data, 2 lower figures are filtered)*

For the specific test shown in Figure 3-22 (which is the same test as in Figure 3-20) it can be seen that the large peaks were filtered. The maximum velocity has become 2.92 m/s and the maximum flow depth has become 21.7 cm (from the depth gauge). The large peak of more than 35 cm completely disappeared. The calculated discharge of the filtered data becomes a little bit higher than the raw data (and is quite large with almost 160 l/s/m). $T_{overt}$ is about 6.4 s which is 67% of the mean period.

3.5.2 Moving-average filtering

Maximum flow depth and maximum flow velocities can also be calculated by taking the average of all events (Figure 3-23). In this case there are eight overtopping events and the mean maximum velocity, after applying the moving-average filter,
becomes 2.93 m/s. The maximum flow depth becomes 22.8 cm. This is a very small difference with the mean-phase filtering technique. The discharge after filtering becomes slightly lower than in the raw data.

<table>
<thead>
<tr>
<th>Test names</th>
<th>$H$ [m]</th>
<th>$T$ [s]</th>
<th>$\xi$ [-]</th>
<th>$q$ [l/s/m]</th>
<th>$q_{g}^{\text{cal}}$ [l/s/m]</th>
<th>$u_{\text{max}}$ [m/s]</th>
<th>$h_{\text{max}}$ [cm]</th>
<th>$q_{\text{cal}}$ [l/s/m]</th>
<th>$u_{\text{max}}$ [m/s]</th>
<th>$h_{\text{max}}$ [cm]</th>
<th>$q_{\text{cal}}$ [l/s/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>31050009</td>
<td>1.00</td>
<td>7.50</td>
<td>1.56</td>
<td>75.8</td>
<td>66.7</td>
<td>2.73</td>
<td>12.7</td>
<td>63.5</td>
<td>2.54</td>
<td>12.8</td>
<td>64.2</td>
</tr>
<tr>
<td>31050010</td>
<td>1.00</td>
<td>9.50</td>
<td>1.98</td>
<td>154</td>
<td>159</td>
<td>2.92</td>
<td>21.7</td>
<td>161</td>
<td>2.93</td>
<td>22.8</td>
<td>155</td>
</tr>
<tr>
<td>31050011</td>
<td>0.70</td>
<td>9.50</td>
<td>2.36</td>
<td>60.0</td>
<td>53.5</td>
<td>2.12</td>
<td>12.7</td>
<td>62.2</td>
<td>2.11</td>
<td>12.3</td>
<td>52.3</td>
</tr>
<tr>
<td>13060009</td>
<td>0.75</td>
<td>10.00</td>
<td>2.40</td>
<td>60.6</td>
<td>54.3</td>
<td>2.40</td>
<td>11.7</td>
<td>46.5</td>
<td>2.32</td>
<td>11.7</td>
<td>52.4</td>
</tr>
<tr>
<td>14060002</td>
<td>1.10</td>
<td>5.00</td>
<td>0.99</td>
<td>12.4</td>
<td>16.0</td>
<td>1.40</td>
<td>4.8</td>
<td>26.5</td>
<td>1.52</td>
<td>5.3</td>
<td>13.7</td>
</tr>
</tbody>
</table>

Table 3-2 Five tests with regular waves Hannover

The measured $q$, in column five, is the discharge that has been measured during the tests. The raw $q_{g}^{\text{cal}}$, in column six, is the discharge which can be calculated from the
raw data record (full definitions in appendix A). One can see that with the mean-
phase filtering technique larger deviances of the mean discharge take place than with
the moving-average filtering technique. However, the two filter techniques show
almost the same maximum flow depths and maximum velocities. These are very high
discharges, not only because we are dealing with regular waves, but also because
the tests have been done with very long wave periods.

3.6 Regular waves analysis

In this section the present filter techniques will be used to analyse the tests with
regular waves carried out by Schüttrumpf in Hannover. The results will be compared
to the results found by Schüttrumpf (Schüttrumpf 2001a). He found formulae for
maximum flow depth and maximum flow velocity for regular waves. He used the
remainder of the wave run-up, which exists of the difference between the wave run-
up and the crest freeboard. The wave run-up for regular waves has been given in
equation 3.7 (which is the same as equation 2.8 but with different coefficient values).

\[ R_u = H \cdot 2.25 \tanh(0.5 \cdot \xi_u) \]  
(Schüttrumpf 2001a)  3-6

Where:

- \( R_u \) = wave run-up [m]
- \( H \) = wave height [m]

The remainder of the wave run-up, the difference between the fictive wave run-up
and the crest freeboard, has been given in equation 3-8 One has to notice that this
relation has been defined in horizontal direction.

\[ x_* = x_z - x_A = n \cdot H \cdot 2.25 \tanh(0.5 \cdot \xi_u) - (n \cdot R_c) \]  
(Schüttrumpf, 2001a)  3-7

Where:

- \( x_* \) = relative wave run-up in horizontal direction [m]
- \( n \) = outer slope 1:n [-]

![Flow depths of regular waves (GWK Hannover)](image)

*Figure 3-24 Maximum flow depth seaward side of the crest, regular waves.*
Schüttrumpf found a linear correlation between $x_*$ and the maximum flow depth. The slope of the linear equation has been 0.055 (Schüttrumpf 2001a). Applying the moving-average filter technique, described in the previous section, a linear correlation with a slope of 0.051 has been found, see Figure 3-24. Data records of only five regular wave tests were used here, which probably caused the small difference between the two slopes.

Schüttrumpf uses equation 3.8 to determine the velocity.

$$\frac{u_s}{\frac{\pi H}{T}} = n \cdot \xi_d \cdot \sqrt{\frac{(R_u - R_c)}{H}}$$  \hspace{1cm} (Schüttrumpf 2001a) \hspace{1cm} 3-8

There exists also a linear correlation between equation 3-9 and the flow velocity on the crest. Schüttrumpf finds a linear equation through the origin with a slope of 0.75 (Schüttrumpf 2001a). With the present filter technique a slope of 0.78 is found (Figure 3-25).

![Flow velocities of regular waves (GWK Hannover)](image)

*Figure 3-25 Maximum velocity seaward side of the crest, regular waves.*

Concluded from the analysis with regular waves; the present moving average-filter technique can be used as analysis method for irregular waves.
3.7 Data analysis Schüttrumpf’s tests

Five tests were used for the analysis with irregular waves in the present report. The parameters have been listed in Table 3-3. The 2nd column from the right shows the discharge $q_{\text{meas}}$ that has been measured during the tests with the overtopping tank. The last column indicates the discharge calculated from the velocity and flow depth data after a moving-average filter has been applied on the data records (full definitions in appendix A).

<table>
<thead>
<tr>
<th>Test name</th>
<th>$d$ [m]</th>
<th>$R_c$ [m]</th>
<th>$H_s$ [m]</th>
<th>$T_{m-1,0}$ [s]</th>
<th>$T_p$ [s]</th>
<th>$P_{\text{overt}}$ [%]</th>
<th>$L_0$ [m]</th>
<th>$s_{\text{run}}$ [-]</th>
<th>$z_{\text{20}}$ [-]</th>
<th>$z_{\text{20}}$ [l/s/m]</th>
<th>$q_{\text{meas}}$ [l/s/m]</th>
<th>$q_{\text{cal}}$ [l/s/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>310500006</td>
<td>5.00</td>
<td>1.00</td>
<td>0.73</td>
<td>6.71</td>
<td>6.17</td>
<td>16.0</td>
<td>70.4</td>
<td>0.01</td>
<td>1.64</td>
<td>1.72</td>
<td>11.2</td>
<td>9.8</td>
</tr>
<tr>
<td>090500006</td>
<td>5.01</td>
<td>0.99</td>
<td>0.87</td>
<td>3.90</td>
<td>4.37</td>
<td>22.0</td>
<td>23.7</td>
<td>0.04</td>
<td>0.87</td>
<td>1.34</td>
<td>-</td>
<td>4.8</td>
</tr>
<tr>
<td>250500003</td>
<td>5.01</td>
<td>0.99</td>
<td>0.88</td>
<td>4.20</td>
<td>2.98</td>
<td>25.2</td>
<td>27.6</td>
<td>0.03</td>
<td>0.93</td>
<td>1.43</td>
<td>7.6</td>
<td>7.7</td>
</tr>
<tr>
<td>160600003</td>
<td>4.70</td>
<td>1.30</td>
<td>0.92</td>
<td>4.16</td>
<td>3.59</td>
<td>7.3</td>
<td>27.0</td>
<td>0.03</td>
<td>0.90</td>
<td>1.46</td>
<td>1.5</td>
<td>1.4</td>
</tr>
<tr>
<td>240500009</td>
<td>5.01</td>
<td>0.99</td>
<td>0.90</td>
<td>4.14</td>
<td>4.02</td>
<td>9.1</td>
<td>26.7</td>
<td>0.03</td>
<td>0.91</td>
<td>1.43</td>
<td>3.7</td>
<td>4.7</td>
</tr>
</tbody>
</table>

Table 3-3 Parameters of the five tests in Hannover with irregular waves.

$P_{\text{overt}}$ is percentage of overtopping. This means that for example in the first test 16% of all incoming waves overtopped the dike model.

For the five tests mentioned in Table 3-3 an analysis has been carried out, using the moving-average filter technique. The 2% run-up values are all determined with the hyperbolic function of Schüttrumpf (equation 2-8). The 2% flow depths were the values that were exceeded by 2% of the incoming waves, not to be confused with the amount of overtopping events. The coefficients can be calculated for the different formulae and compared with coefficients found by Schüttrumpf. Equations 2-25 to 2-28 were used to determine the coefficients. Figure 3-26 shows the coefficients $c'_h2\%$ and $c''h2\%$ for the maximum flow depth on the crest found in the present study with the moving average filter. The red lines indicate the fits of Schüttrumpf.

![Figure 3-26 Fits of 2% maximum flow depths against formulae on the crest, irregular waves](image-url)
In Figure 3-27 the coefficients for the maximum velocity on the crest are demonstrated. The red lines indicate the fits of Schüttrumpf.

![Graph showing Seaside and Landside](image)

*Figure 3-27 Fits of 2% maximum flow velocities against formulae on the crest, irregular waves*

A comparison between the coefficients found in the present study and by Schüttrumpf has been given in Table 3-4. Small deviations in the flow depth coefficients can be caused by the fact that the coefficients in the present study have been based on five tests and not on 146 tests by Schüttrumpf. The velocity coefficient $c'_{u\%2}$ differs a lot. In this study the 2% values are defined by the front velocities instead of the measured velocity, explained in the previous chapter. On the other hand is the coefficient $c''_{u\%2}$ on the landside of the crest almost equal. That means that the front velocities and measured velocities show the same alteration on the crest.

<table>
<thead>
<tr>
<th></th>
<th><em>Schüttrumpf (200b1)</em></th>
<th><em>Present study</em></th>
</tr>
</thead>
<tbody>
<tr>
<td>$c'_{h,2%}$</td>
<td>0.33</td>
<td>0.31</td>
</tr>
<tr>
<td>$c''_{h,2%}$</td>
<td>0.89</td>
<td>0.62</td>
</tr>
<tr>
<td>$c'_{u,2%}$</td>
<td>1.37</td>
<td>1.64</td>
</tr>
<tr>
<td>$c''_{u,2%}$</td>
<td>0.5</td>
<td>0.5</td>
</tr>
</tbody>
</table>

*Table 3-4 Comparison between coefficients, irregular waves*
3.8 Data analysis Van Gent’s tests

The data records of the velocity, flow depth and discharge on the landside of the crest and on the middle of the inner slope of three tests were available. An overall view of the involved test parameters are presented in Table 3-5.

<table>
<thead>
<tr>
<th>Test</th>
<th>$d$ [m]</th>
<th>$R_c$ [m]</th>
<th>$H_s$ [m]</th>
<th>$T_{m-1.0}$ [s]</th>
<th>$T_p$ [s]</th>
<th>$P_{out}$ [%]</th>
<th>$L_0$ [m]</th>
<th>$S_{om}$ [-]</th>
<th>$\xi_0$-1 [-]</th>
<th>$z_{2%}$ [-]</th>
<th>q-meas. [l/s/m]</th>
<th>q-cal. [l/s/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.05</td>
<td>0.40</td>
<td>0.20</td>
<td>0.150</td>
<td>2.14</td>
<td>2.49</td>
<td>0.51</td>
<td>7.15</td>
<td>0.021</td>
<td>1.73</td>
<td>0.38</td>
<td>2.19</td>
<td>2.38</td>
</tr>
<tr>
<td>1.06</td>
<td>0.40</td>
<td>0.20</td>
<td>0.140</td>
<td>1.79</td>
<td>1.99</td>
<td>0.27</td>
<td>5.00</td>
<td>0.028</td>
<td>1.49</td>
<td>0.30</td>
<td>0.52</td>
<td>0.44</td>
</tr>
<tr>
<td>1.07</td>
<td>0.40</td>
<td>0.20</td>
<td>0.135</td>
<td>1.51</td>
<td>1.59</td>
<td>0.14</td>
<td>3.56</td>
<td>0.038</td>
<td>1.28</td>
<td>0.25</td>
<td>0.14</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Table 3-5 Parameters of the three tests of Van Gent with irregular waves.

The 2% values of the maximum flow depth and maximum flow velocity were resolved with the moving-average filter technique of the present study. The 2% values calculated by Van Gent have been made available. In Table 3-6 a comparison has been made between the 2% values calculated within this study and the values calculated by Van Gent.

<table>
<thead>
<tr>
<th>Test</th>
<th>Van Gent $h_{2%}/H_s$ [-]</th>
<th>Present study $h_{2%}/H_s$ [-]</th>
<th>Diff. [%]</th>
<th>Van Gent $u_{2%}/gH_s^{0.5}$ [-]</th>
<th>Present study $u_{2%}/gH_s^{0.5}$ [-]</th>
<th>Diff. [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.05</td>
<td>0.139</td>
<td>0.137</td>
<td>-1.89</td>
<td>1.294</td>
<td>1.218</td>
<td>-5.92</td>
</tr>
<tr>
<td>1.06</td>
<td>0.068</td>
<td>0.064</td>
<td>-5.26</td>
<td>1.041</td>
<td>0.949</td>
<td>-8.89</td>
</tr>
<tr>
<td>1.07</td>
<td>0.033</td>
<td>0.039</td>
<td>+15.6</td>
<td>0.695</td>
<td>0.677</td>
<td>-2.59</td>
</tr>
</tbody>
</table>

Table 3-6 2% values on the landward side of the crest

The flow depth differences are very small (5% is less than 1 mm). In the present study the velocity with the moving-average filter is around 0.1 m/s lower than Van Gent calculated.

Figure 3-28 Fits of 2% maximum flow depths against formulae on the crest, irregular waves
The experiments carried out by Schüttrumpf and Van Gent

The fits between the 2% flow depth values and the formula on the seaside and the landside of the crest are plotted in Figure 3-28. Equations 2-25 and 2-27 were used to determine the coefficients $c'_{h,2\%}$ and $c''_{h,2\%}$, and the 2% wave run-up was calculated with equation 2-15 of Van Gent. One can see that the coefficients determined by Van Gent are just slightly different than the ones determined in the present study.

![Seaside and Landside Graphs](image)

*Figure 3-29 Fits of 2% maximum flow velocities against formulae on the crest, irregular waves*

Figure 3-29 shows the coefficients $c'_{u,2\%}$ and $c''_{u,2\%}$ that were found by fitting the velocity data with equations 2-26 and 2-28. Table 3-7 gives an overall view of the coefficients found in the present study and Van Gent’s study.

<table>
<thead>
<tr>
<th></th>
<th>Van Gent (2002)</th>
<th>Present study</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c'_{h,2%}$</td>
<td>0.15</td>
<td>0.14</td>
</tr>
<tr>
<td>$c''_{h,2%}$</td>
<td>0.40</td>
<td>0.31</td>
</tr>
<tr>
<td>$c'_{u,2%}$</td>
<td>1.30</td>
<td>1.36</td>
</tr>
<tr>
<td>$c''_{u,2%}$</td>
<td>0.5</td>
<td>0.5</td>
</tr>
</tbody>
</table>

*Table 3-7 Comparison between coefficients, irregular waves*

The differences can be explained by the fact that in the present study only data of the test set-up of configuration A were used. Van Gent fitted the formula on more data sets.
3.9 Conclusions

Experiments Schüttrumpf
The flow depth data from the depth gauges can best be used on the crest, while the pressure cells are more reliable on the inner slope. The velocity measurements with the propellers show large deviation in comparison to the determined front velocities when \( u > 2.5 \text{ m/s} \) (Figure 3-6). Therefore the front velocity must be used within the data-processing. The front velocities on different locations on the dike model can be determined very accurate, because Schüttrumpf carried out measurements on ten locations.

Experiments Van Gent
The velocity measurements with the velocity propellers are adequate (Figure 3-17). The flow depth measurements from the depth gauges show good results as well.

Analysis
The analysis in both studies has been carried out correctly, only the velocity 2% values of the tests of Schüttrumpf are higher when the front velocities are used. This means that both velocities and flow depths were higher during the tests of Schüttrumpf in comparison to Van Gent.

Theory
Taking the high correlation in both studies of Schüttrumpf and Van Gent into consideration, the conclusion can be drawn that the principles of equations 2-25 to 2-28 are correct. The fictive wave run-up minus the crest freeboard has been a good measure for the flow depths and velocities on the crest. There still is a discrepancy between both studies; not only the flow depth but also the velocities were higher during the tests of Schüttrumpf. The difference between the flow depths is a factor 2.2 and the difference between the velocities a factor 1.5. The discrepancy may be due to differences in the model set-ups.
The experiments carried out by Schüttrumpf and Van Gent
4 Calibration of new wave overtopping formulae

4.1 Introduction
4.2 New theory for maximum velocity and flow depth
4.3 Overtopping time
4.4 Velocity and flow depth variation in time
4.5 Residence time
4.6 Comparison other dataset
4.7 Conclusions
4.1 Introduction
The differences between the studies of Schüttrumpf and Van Gent will be explained in the first section of this chapter by the difference in the outer slope. Test data from both studies will be used to validate the new theory. The overtopping time will also be described in the second section of this chapter. In the third section the variation in time of flow depth and velocity will be discussed. Section 4.5 describes some information of the residence time of water on the crest. In the last section a comparison has been made with other datasets.

4.2 New theory for maximum velocity and flow depth

4.2.1 The influence of the outer slope
Schüttrumpf and Van Gent conducted experiments on dikes with different outer slopes. Schüttrumpf uses an outer slope of 1:6 and Van Gent 1:4. The formulae for maximum flow depth and maximum velocity on the crest have been based on the remainder of the wave run-up: the 2% run-up level minus the crest freeboard. This has been defined in vertical direction, see Figure 4-1.

![Figure 4-1 The remainder of the wave run-up: "A"](image)

When a smooth slope is assumed only the gravitational force limits the wave run-up height. The red arrow in Figure 4-1 indicates the velocity of the water along the slope. The following example will describe two different dike configurations with the same value for "A". Table 4-1 gives overtopping parameters for two different dike configurations. The parameters have been determined for one wave characteristic and two different outer slopes.

<table>
<thead>
<tr>
<th>Dike configuration</th>
<th>Slope</th>
<th>$\alpha$ [°]</th>
<th>$H$ [m]</th>
<th>$T$ [s]</th>
<th>$s_{vm}$ [-]</th>
<th>$\xi$ [-]</th>
<th>$z_{2%}$ [m]</th>
<th>slope $s$ [m]</th>
<th>$R_c$ [m]</th>
<th>$A$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>1:4</td>
<td>14.0</td>
<td>1</td>
<td>5</td>
<td>0.026</td>
<td>1.56</td>
<td>2.58</td>
<td>10.6</td>
<td>1.58</td>
<td>1.0</td>
</tr>
<tr>
<td>(2)</td>
<td>1:6</td>
<td>9.46</td>
<td>1</td>
<td>5</td>
<td>0.026</td>
<td>1.04</td>
<td>1.72</td>
<td>10.5</td>
<td>0.72</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table 4-1 Example with one wave and two different outer slopes

The crest freeboards have been chosen in such a way that the remainder of the wave run-up "A" will be equal for both dike configurations. The lengths of the slopes of the 2% run-up values "s" will almost be equal as well (around 10.5 m, see the 9th column). This situation has been drawn in Figure 4-2. The vertical wave run-up $z_{2\%}$
for dike configuration (1) will be much higher than the $z_{2p5}$ for configuration (2). That means that the vertical velocity component of the water in point "S" will also be higher than in point "P" in order to achieve the top, point "R". The vertical velocity component in points "Q" en "T" will be the same, because the vertical distance to point "R" is also the same. The velocity along the slope in points "Q" en "T" will not be the same.

![Diagram](image)

Figure 4-2 Situation sketch for a wave generating the same remainder of the wave run-up at two dikes with a different outer slope

Figure 4-3 shows the difference between the velocities along the outer slope for both dike configurations. The vertical velocity component for both configurations should be the same, because the remaining run-up level is the same.

![Diagram](image)

Figure 4-3 Velocity along the outer slope for two dike configurations with the same vertical velocity components

With the use of Figure 4-3 equation 4-1 can be formulated for point "Q" en "T":

$$ u_v = u_s \sin \alpha_1 = u_s \sin \alpha_2 \rightarrow \frac{u_{s1}}{u_{s2}} = \frac{\sin \alpha_2}{\sin \alpha_1} $$

4-1

Where:

- $u_v$ = vertical velocity component [m/s]
- $u_s$ = velocity component along the inner slope [m/s]
In the example is $\sin \alpha_2 / \sin \alpha_1 = 0.68$, this means that $u_{s1} = 0.68 \cdot u_{s2}$ or $u_{s2} = 1.47 \cdot u_{s1}$. Thus the velocity is almost one and a half times higher at point "Q" in comparison to point "T" in the direction along the slope in Figure 4-2. Figure 4-4 shows the relation between the outer slope angle and the velocity along the slope. The two red dots represent the two outer slope angles of the example.

![Figure 4-4 Outer slope angle against dimensionless velocity along the slope](image)

This phenomenon influences the velocities on the crest during wave overtopping. The empirical coefficient $c_{u,2\%}'$ has to be a function of the outer slope angle, equation 4-2.

$$c_{u,2\%}' = \frac{a}{\sin \alpha}$$ \hspace{1cm} 4-2

Where:

- $a$ = empirical coefficient \hspace{1cm} [-]
- $\alpha$ = outer slope angle \hspace{1cm} [°]

The relation between the maximum velocity and maximum flow depth is described below in equation 4-3 and will be used to determine coefficient "a" of equation 4-2. The relation has been determined with the use of equations 2-9 and 2-10, details of the derivation of those equations are described by Schüttrumpf (Schüttrumpf 2001a).

$$\frac{u_{2\%}}{\sqrt{gH_s}} = c_{u,2\%}' \left( \frac{Z_{2\%} - R_c}{H_s} \right)^{0.5} = c_{u,2\%}' \left( \frac{A}{H_s} \right)^{0.5}$$

$$\frac{h_{2\%}}{H_s} = c_{h,2\%}' \left[ \frac{Z_{2\%} - R_c}{H_s} \right] = c_{h,2\%}' \left( \frac{A}{H_s} \right)$$

Where:

- $b$ = scaling factor \hspace{1cm} [-]

It is assumed that the relation between flow depth and velocity must be scaled with a factor $b$. 
The relation between the two coefficients considering equation 4-3 becomes:

\[ c'_{h, 2\%} = \left( \frac{c'_{u, 2\%}}{b} \right)^2 = \left( \frac{a}{b \sin \alpha} \right)^2 = \frac{a^2}{b^2 \sin^2 \alpha} \quad 4-4 \]

The coefficient \( c'_{h, 2\%} \) showed a discrepancy of 2.2 between the studies of Schüttrumpf and Van Gent (0.33/0.15 = 2.2). This matches with equation 4-4, because:

\[ \frac{\sin^2 \alpha_1}{\sin^2 \alpha_2} = \frac{\sin^2 (9.46)}{\sin^2 (14.0)} = 2.2 \]

This means that the flow depth coefficient is related to the outer slope angle raised to the square. The flow depth on the outer slope can be drawn for both dike configurations on the moment the maximum (fictive) wave-run up is reached, see Figure 4-5:

**Figure 4-5 Fictive wave run-up on the outer slope for two dike configurations**

The following two assumptions have been made (see Figure 4-5):

- \( h_s = h_p \)
- \( h_q = 2.2h_t \)
- \( h_k = 0 \)
• The flow depth on the outer slope at the position of MSL ($\eta = 0$) will be equal for both dike configurations: $h_p = h_5$.
• The shape of the tongue of water on the outer slope is curved.

The first assumption holds that the volume of the water tongues have to be equal as well, but these volumes will not be distributed the same along the vertical axis; configuration (2) beholds more volume of water above the crest level than configuration (1).

The empirical coefficient $c'_{h,2\%}$ were determined to be 0.33 and 0.15 for both dike configurations (Schüttrumpf, Van Gent 2003). With these values the new coefficient can be determined, related to the outer slope angle, see equation 4-5.

$$c'_{h,2\%} = \frac{9 \cdot 10^{-3}}{\sin^2 \alpha}$$

and: $$\frac{a^2}{b^2} = 9 \cdot 10^{-3}$$

Where:
$c'_{h,2\%}$ = empirical coefficient 2% flow depth on the seaside of the crest

Figure 4-6 gives the relation between coefficient $c'_{h,2\%}$ and the outer slope angle $\alpha$. Extreme values for the outer slope angle $\alpha$ have not been validated.

With the new coefficient $c'_{h,2\%}$ for the maximum flow depth on the seaside of the crest, the coefficient $c'_{u,2\%}$ for maximum velocity on the seaside of the crest can be determined. This can be done with the relation between maximum flow depth and maximum velocity. The relation between the maximum flow depth and maximum velocity.

\[c'_{h,2\%} = \frac{9 \cdot 10^{-3}}{\sin^2 \alpha}\]

\[\frac{a^2}{b^2} = 9 \cdot 10^{-3}\]

\[c'_{h,2\%} = \text{empirical coefficient 2\% flow depth on the seaside of the crest}\]

Figure 4-6 Relation between flow depth coefficient and $\alpha$

* These assumptions cannot be physically proved at this moment. The flow depths in point S and P can also be different and the shape of the overtopping tongue might not be curved as well, however the difference between the flow depths in point S and T must be a factor 2.2.
velocity was shown in equation 4-3. In Figure 4-7 one can see a fit (thick black line) between the 2% velocity and flow depth data of Schüttrumpf and Van Gent.

![Figure 4-7 Relation maximum velocity and maximum flow depth](image)

The fit in Figure 4-7 equals equation 4-6:

\[
\frac{u_{2\%}}{\sqrt{gH_s}} = 3.68 \left( \frac{h_{2\%}}{H_s} \right)^{0.59}
\]

4-6

The theory of maximum flow depth and maximum velocity states that the relation between both unities must be a square root function, equation 4-3. In Figure 4-7 is this relation drawn with a red line, in such a way that it correlates as good as possible with equation 4-6. Thus the relation between maximum flow depth and maximum velocity on the crest can now be described by equation 4-7:

\[
\frac{u_{2\%}}{\sqrt{gH_s}} = 3.1 \left( \frac{h_{2\%}}{H_s} \right)^{0.5} \quad \text{for } 0.03 < \frac{h_{2\%}}{H_s} < 0.25
\]

4-7

With the relation in equation 4-7 the empirical coefficient "a" can be determined using equations 4-4, 4-5. With \( a^2/b^2 = 9 \times 10^{-3} \) and \( b = 3.1 \) makes: \( a = 0.30 \). The empirical coefficient \( c'_{u,2\%} \) becomes:

\[
c'_{u,2\%} = \frac{0.30}{\sin \alpha}
\]

4-8

Where:

\( c'_{u,2\%} = \) empirical coefficient 2% flow velocity on the seaside of the crest  \([-\] \)
Equation 4-8 has been plotted in Figure 4-8.

![Figure 4-8 Relation between velocity coefficient and α](image)

The coefficients on the seaside of the crest for the two different dike models become:

\[ c'_{u,2\%} = \frac{0.30}{\sin \alpha_1} = 1.24 \text{ for a configuration with an outer slope of 1:4.} \]

\[ c'_{u,2\%} = \frac{0.30}{\sin \alpha_2} = 1.83 \text{ for a configuration with an outer slope of 1:6.} \]

These values of the coefficients differ from the ones found by Schüttrumpf and Van Gent and found in the present study (Table 3-4 and Table 3-7). But the values of the coefficient \( c'_{u,2\%} \) listed above, have been based on the flow depth coefficient \( c'_{h,2\%} \) and on the relation between maximum flow depth and maximum velocity. This relation is considered to be more reliable than a determination with only maximum velocity data. Length is an integral of velocity over time, and therefore a more consistent unity.
4.2.2 The influence of the crest

On the crest friction takes place, the maximum velocities and maximum flow depths will therefore decrease over the crest. In the formulae of Schüttrumpf and Van Gent an exponential function has been used, equations 2-27 and 2-28 will be repeated here:

\[
\frac{h_{2\eta}(x_c)}{h_{2\eta}(x_c = 0)} = \exp\left(-c_{n,2\eta} \frac{x_c}{B_c}\right) \quad \text{(Schüttrumpf, Van Gent 2003)}
\]

\[
\frac{u_{2\eta}(x_c)}{u_{2\eta}(x_c = 0)} = \exp\left(-c_{n,2\eta} \frac{x_c \cdot f}{h_{2\eta}(x_c)}\right) \quad \text{(Schüttrumpf, Van Gent 2003)}
\]

The position on the crest \(x_c\) has been made dimensionless by the crest width \(B_c\) in equation 4-9. This means that in this relation the flow depth at the landward side of the crest does not depend on the crest width. In the present study a relation has been found for the flow depth on positions \(x_c\) on the crest made dimensionless by the wave length \(L_0\).

The decrease of the maximum flow depth on the crest has been plotted in Figure 4-9. The maximum flow depth per position \(x_c/L_0\) has been divided through the maximum flow depth on the seaward side of the crest. The first measuring position is on \(x_c = 0.5m\) behind the beginning of the crest. One can see that at this first location behind the beginning of the crest, the maximum flow depth has decreased with approximately 25% in comparison to the seaward side of the crest. This is due to the transition of the water from the outer slope onto the crest. The direction of the water needs to change from the direction parallel to the outer slope into the horizontal direction on the crest. This transition requires some time and space. The relation for the
maximum flow depth on the crest needs to be independent of this transition. The exponential fit in Figure 4-9 therefore only depends on maximum flow depths on the crest behind this transition. Equation 4-9 becomes equation 4-11:

\[
\frac{h_{2\%^{\text{c}}}(x_c)}{h_{2\%(x_c = 0)}} = c_{\text{trans},h} \exp \left( -c_{h,2\%^{\text{c}}} \frac{x_c}{\gamma_c} \right) \tag{4-11}
\]

Where

- \( c_{\text{trans},h} \) = influence due to transition from outer slope to crest [ - ]
- \( \gamma_c \) = friction factor on the crest (=1 for a smooth surface) [ - ]

The maximum velocity depends on the maximum flow depth on the crest, see equation 4-10. The transition factor has already been implemented in the flow depth and therefore does not have to be implemented in the velocity formula. Equation 4-10 will be changed into equation 4-12. In this equation a different friction coefficient is used. The surface of the model in the tests is considered to be smooth (\( \gamma_c = 1 \)).

\[
\frac{u_{2\%^{\text{c}}}(x_c)}{u_{2\%(x_c = 0)}} = \exp \left( -c_{u,2\%^{\text{c}}} \frac{x_c}{\gamma_c h_{2\%(x_c)}} \right) \tag{4-12}
\]

![Figure 4-10 Alteration for maximum velocity on the crest](image)

The coefficients have been determined for regular waves (see Table 4-2). The coefficients for irregular waves are expected to be almost the same.

<table>
<thead>
<tr>
<th>( c_{\text{trans},h} )</th>
<th>( c_{h,2%^{\text{c}}} )</th>
<th>( c_{u,2%^{\text{c}}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.81</td>
<td>15</td>
<td>0.042</td>
</tr>
</tbody>
</table>

*Table 4-2 Empirical coefficients on the crest for regular waves*
4.2.3 The influence of the inner slope

On the inner slope Schüttrumpf uses a Navier-Stokes equation and Van Gent a One-dimensional shallow water equation. The one-dimensional shallow water equation of Van Gent was preferred above the Navier-Stokes equation (Schüttrumpf, Van Gent 2003). Equations have been given in chapter 2 and will not be repeated here.

Although the one-dimensional shallow water equation of Van Gent was preferred above the Navier-Stokes equation of Schüttrumpf both methods will be shown. In Figure 4-11 both equations have been plotted against velocity data on the inner slope of three tests of Schüttrumpf. In Figure 4-12 both equations were plotted together with velocity data of the tests of Van Gent.

![Figure 4-11 Velocities on the inner slope for regular waves Schüttrumpf](image1)

![Figure 4-12 Velocities on the inner slope for irregular waves Van Gent](image2)

The friction coefficients used in the formulae were the values for a smooth surface used by Van Gent and Schüttrumpf in their studies. Thus for the shallow water equation a value of $f_1 = 0.005$ has been used and for the Navier-stokes equation a friction factor of $f=0.02$ has been used. One can see in Figure 4-11 that both formulae give higher velocities than the measurements of Schüttrumpf. This can be caused by the fact that the velocity propellers were located close to the wall. As described in chapter 3; the main body of the wave was passing by and not measured (Figure 3-4). Both figures suggest that the one-dimensional shallow water equation has been closer to all measured values. It is important to realize that the chosen friction coefficient is decisive, so both formulae can always be scaled to the desired values.

Figure 4-13 shows the flow depths of the three irregular waves tests of Van Gent together with both formulae. The flow depth has been determined with a continuity equation: $h(s) = u_0h_0/u(s)$ and thus depends on the velocity on the inner slope. In
Figure 4-13 the formula of the flow depth of Van Gent has been based on the velocity of Van Gent and the flow depth for the formula of Schüttrumpf has been based on the velocity determined with the formula of Schüttrumpf. One can see that again the one-dimensional shallow water equation has been most close to the measured values.

Differences between the results of both equations were small. The one-dimensional shallow water equation has been preferred by Schüttrumpf and Van Gent, as mentioned in chapter 2. The analysis in the present study supports that decision.
4.2.4 Summary of new equations

4.2.4.1 Seaward side of the crest

The formulae for 2% maximum flow depth and flow velocity on the seaside of the crest have been given in section 2.4.3.2. In the present study the angle of the outer slope is implemented in the coefficients, see Table 4-3. The equations are repeated below.

\[ h_{2\%}(x_c = 0) = c'_{h,2\%} (z_{2\%} - R_c) \]  \hspace{1cm} 4-13

\[ u_{2\%}(x_c = 0) = c'_{u,2\%} \sqrt{z_{2\%} - R_c} \]  \hspace{1cm} 4-14

<table>
<thead>
<tr>
<th>Seaward side of the crest</th>
<th>Schüttrumpf (2001b)</th>
<th>Van Gent (2002)</th>
<th>Present study</th>
</tr>
</thead>
<tbody>
<tr>
<td>( c'_{h,2%} )</td>
<td>0.33</td>
<td>0.15</td>
<td>0.010 ( \sin^2 \alpha )</td>
</tr>
<tr>
<td>( c'_{u,2%} )</td>
<td>1.37</td>
<td>1.30</td>
<td>0.30 ( \sin \alpha )</td>
</tr>
</tbody>
</table>

Table 4-3 Coefficients at the seaward side of Schüttrumpf, Van Gent and the present study

4.2.4.2 On the crest

The formulae for maximum flow depth and velocity on the crest have been changed in comparison to Schüttrumpf and Van Gent (Schüttrumpf, Van Gent 2003). The effect of the transition of overtopping water from the outer slope to the crest has been implemented in the equation with the coefficient \( c_{trans} \). Furthermore, a different friction coefficient has been used.

\[ \frac{h_{2\%}(x_c)}{h_{2\%}(x_c = 0)} = c_{trans,h} \exp \left( -c''_{h,2\%} \frac{x_c}{\gamma_c L_0} \right) \]  \hspace{1cm} 4-15

\[ \frac{u_{2\%}(x_c)}{u_{2\%}(x_c = 0)} = \exp \left( -c''_{u,2\%} \frac{x_c}{\gamma_c h_{2\%}(x_c)} \right) \]  \hspace{1cm} 4-16

<table>
<thead>
<tr>
<th>Landward side of the crest</th>
<th>( c_{trans,h} )</th>
<th>0.81</th>
</tr>
</thead>
<tbody>
<tr>
<td>( c''_{h,2%} )</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>( c''_{u,2%} )</td>
<td>0.042</td>
<td></td>
</tr>
</tbody>
</table>

Table 4-4 Coefficients at the landward side of the present study

The coefficients in Table 4-3 and Table 4-4 have been determined with the use of 15 tests from Schüttrumpf (5) and Van Gent (10). All 2% flow depth data on the seaside (position 1) and landside (position 5) on the crest have been plotted in Figure 4-14. The velocity data on both sides of the crest have been plotted in Figure 4-15.
4.2.4.3 Inner slope

On the inner slope the one-dimensional shallow water equation must be used to determine the maximum flow depth and flow velocity. This matches with Schüttrumpf and Van Gent (Schüttrumpf, Van Gent 2003).

![Flow depth on the crest](image1)

*Figure 4-14 2% flow depth values against formula on the crest*

![Velocity on the crest](image2)

*Figure 4-15 2% velocity values against formula on the crest*
4.3 Overtopping time

The maximum overtopping time is useful when one wants to describe the variation of flow depths and velocities in time. In other words; a relation between the maximum flow depth, maximum velocity and the corresponding overtopping time. An assumption of Van der Meer has been that the overtopping time should be in the range of:

\[ T_{ovt} = (0.2 - 0.8)T_p \]  
(Van der Meer, 2006b)  

Where:

- \( T_{ovt} \) = overtopping time  
- \( T_p \) = the peak period in the wave spectrum  

Equation 4-17 adduces that the overtopping time will not be longer than 80% of the peak period of a wave.

![Figure 4-16 measured flow depths as function of time of an overtopping wave group on the crest (Source: Test 1.05A, Van Gent)](image)

With the data in Figure 4-16 overtopping times \( T_{ovt} \) can be derived. \( T_{ovt} \) is the duration of one overtopping event. In Figure 4-16 seven recorded overtopping events and therefore seven overtopping times, indicated with the yellow arrows, are shown. One overtopping period starts with the maximum flow depth and ends when the flow depth, as well as the velocity, is almost zero. \( T_{ovt,2\%} \) is the overtopping time exceeded by 2% of the incoming waves. Not to be confused with the amount of overtopping waves. The larger the volume of an overtopping event has been, the longer the overtopping time \( T_{ovt} \) will be.

A relation for the maximum overtopping time can be based on the remainder of the wave run-up, thus the difference between the fictive wave run-up and the crest freeboard. This can be best visualised with the dike models from the example used in section 4.2.1 above. Figure 4-17 is a combination of Figure 4-2 and Figure 4-3. The remainder of the wave run-up has been indicated with the letter A. The vertical
velocity component of the water is in both dike models the same. Therefore is the
time that the water is flowing between Q-R and T-R the same as well.

![Figure 4-17 Velocity components of the example in section 4.2.1](image)

The time water flows between Q and R and T and R can be determined with two
motion equations for velocity and distance, see equation 4-18:

\[
\begin{align*}
A &= u_r t - \frac{1}{2} g t^2 \\
0 &= u_r - g t
\end{align*}
\]

\[t = \frac{A}{2g} = \sqrt{\frac{z_{2\%} - R_c}{2g}} \tag{4-18}

The formula for 2% overtopping time becomes is related to the square root of the
remainder of the wave run-up. With 2g (twice the gravitational force) as a constant
value, the 2% overtopping time can be described by equation 4-19. The overtopping
time has been made dimensionless with the spectral wave period: \(T_{m-1.0} \).

\[
\frac{T_{ovt,2\%}(x_c = 0)}{T_{m-1.0}} = c'_{ovt,2\%} \sqrt{\frac{z_{2\%} - R_c}{\gamma_f H_s}} \tag{4-19}
\]

Where:
\[c'_{ovt,2\%} = \text{empirical coefficient} \quad \text{[-]}\]

![Figure 4-18 2% flow velocity against 2% overtopping time on the seaside of the crest](image)

![Figure 4-19 2% flow depth against 2% overtopping time on the seaside of the crest](image)
If the relation between overtopping time and the remainder of the wave run-up \( (A = z_{2\%} - R_c) \) should physically be a square root function, the relation between velocity and overtopping time should be linear function. This corresponds with Figure 4-18, where a linear relation can be drawn through the 2% values of both unities. The relation between maximum flow depth and overtopping time is presented in Figure 4-19. Here a square root function is expected. Despite the lack of data, equation 4-19 is considered to be a correct approach to determine the overtopping time of an individual overtopping event.

4.3.1 The influence of the crest

The overtopping time on different positions on the crest shows the same transition area which could be seen with the maximum flow depth on the crest. The first part of the crest, just behind the beginning, shows a large difference with the rest of the crest. This difference has been caused by the transition from the outer slope to the crest. The data is fitted without the overtopping time on the seaside of the crest. The logarithmic fit has been drawn in Figure 4-20. The formula for the overtopping time on the crest will become equation 4-20:

\[
\frac{T_{o, 2\%}(x_c)}{T_{o, 2\%}(x_c = 0)} = \left( c_{trans,ovt} + c''_{ovt,2\%} \ln \left( \frac{x_c}{L_0} \right) \right)
\]

4-20

Where:
\( c_{trans,ovt} \) = influence due to transition from outer slope to crest
\( c''_{ovt,2\%} \) = empirical coefficient
4.3.2 The influence of the inner slope

The alteration of the maximum velocity on the inner slope follows a one-dimensional shallow water equation, with respect to the velocity and flow depth on the landward side of the crest. The maximum flow depth on the inner slope can be determined with a continuity equation, see section 4.2.3.

Concerning the continuity equation, the overtopping time should remain the same on the inner slope, because the volume of the overtopping event does not change.

The T_{ovt}'s at the landward side of the crest and on the inner slope have been compared to each other. Results are presented in Figure 4-21 of one test of Van Gent and two tests of Schüttrumpf. The position on the inner slope was for the test of Van Gent s = 0.68m, where s is the distance along the slope. For the tests of Schüttrumpf is the position s = 6.4m taken on the inner slope. The values s/L_0 of the three tests can be found in the legend of the figure.

![Figure 4-21 Overtopping times at the landward side of the crest and on the inner slope](image)

For the test of Van Gent the overtopping time did not change. This coincides with the continuity equation. But for the two tests of Schüttrumpf the overtopping time increased. The overtopping time can only increase if the shapes of the flow depth and velocity graphs change as well. Figure 4-22 shows this transition. Velocities increase and flow depths decrease, conform the one-dimensional shallow water equation. A non-changing shape on the inner slope is presented with the black dotted line in the figure. Increasing overtopping times have been drawn in the figure with a curved shaped line.
The difference between both tests (increasing $T_{	ext{ovt}}$ for Schüttrumpf’s test and the not changing $T_{	ext{ovt}}$ during Van Gent’s test) can be compared with the data of regular wave tests of Schüttrumpf, see Figure 4-23. The value $s/L_0 = 0$ on the horizontal axis is the landward side of the crest.

The overtopping time for regular waves increases to $T_{	ext{ovt}} = T_m$. This means that during these regular wave tests the inner slope was constantly covered with water beyond position $s/L_0 = 0.025$.

The difference between the above mentioned results has probably been caused by the difference between the scales of the tests of Schüttrumpf and Van Gent. The flow depths are very small on the inner slope (millimetres) during small scale tests.
Surface tension plays an important role with such small flow depths and may prevent the shape of the overtopping volume and \( T_{ovt} \) to change.

### 4.3.3 Summary of new equations

#### 4.3.3.1 Seaward side of the crest

Equation 4-21 gives the relation for the overtopping time on the seaward side of the crest:

\[
\frac{T_{ovt,2\%}(x_c = 0)}{T_{m-1.0}} = c_{T_{ovt,2\%}} \sqrt{\frac{L_{2\%} - R_c}{\gamma H}} \tag{4-21}
\]

#### 4.3.3.2 On the crest

The overtopping time increases on the crest. The parameter \( c_{trans,ovt} \) is implemented in the formula. The transition from the outer slope to the crest takes this parameter into account.

\[
\frac{T_{ovt,2\%}(x_c)}{T_{ovt,2\%}(x_c = 0)} = \left( c_{trans,ovt} + c''_{ovt,2\%} \ln \left( \frac{x_c}{L_0} \right) \right) \tag{4-22}
\]

![Graph showing (2%) overtopping time on the crest](image)

**Figure 4-24 2% values maximum overtopping time**

The three coefficients \( c'_{ovt,2\%}, c''_{ovt,2\%} \) and \( c_{trans,ovt} \) for the wave overtopping formulae have been determined for irregular waves with the available data from tests carried out by Schüttrumpf and Van Gent. In Figure 4-24 the 2% values have been plotted...
for the overtopping times of irregular waves. The overtopping time was determined on the seaward side and landward side of the crest. The coefficients have been listed in Table 4-5.

<table>
<thead>
<tr>
<th>Seaward and landward side of the crest</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( C'_{ovt,2%} )</td>
<td>1.15</td>
</tr>
<tr>
<td>( C_{\text{trans,ovt}} )</td>
<td>1.67</td>
</tr>
<tr>
<td>( C''_{ovt,2%} )</td>
<td>0.24</td>
</tr>
</tbody>
</table>

*Table 4-5 Coefficients for the overtopping time formulae*

4.3.3.3 Inner slope

A univocal formula for the overtopping time on the inner slope has not been derived. The tests of Schüttrumpf and Van Gent gave different results for the alteration of the overtopping time on the inner slope. The overtopping time on the inner slope did not change during the small scale tests of Van Gent in comparison to the landward side of the crest. On the other hand: the tests of Schüttrumpf showed an increase of the overtopping time. The scale of the tests might play an important role, due to the very small flow depths in comparison to the crest.
4.4 Velocity and flow depth variation in time

4.4.1 Triangular shape

The variation in time of both the velocity and the flow depth can roughly be estimated as a linear function. As a result the graphs are triangular shaped. The formulae for velocity and flow depth on the crest in time and place are presented in equations 4-23 and 4-24:

\[
\begin{align*}
 u(x_c, t) &= u_2\%(x_c) \left( 1 - c_{u,s} \frac{t}{T_\text{ovt,2}\%(x_c)} \right) \quad 4-23 \\
 h(x_c, t) &= h_2\%(x_c) \left( 1 - c_{h,s} \frac{t}{T_\text{ovt,2}\%(x_c)} \right) \quad 4-24 
\end{align*}
\]

Where:
\[
\begin{align*}
 u &= \text{velocity} \quad [\text{m/s}] \\
 h &= \text{flow depth} \quad [\text{m}] \\
 t &= \text{time per event} \ (0 < t < T_\text{ovt}) \quad [\text{s}] \\
 c_{u,s} / c_{u,h} &= \text{coefficients} \ (s=\text{shape}) \quad [-]
\end{align*}
\]

The assumption of a triangular shape can be validated with the determination of the volume of the overtopping event. The coefficients \(c_{u,s}\) and \(c_{u,h}\) have a value 1 for a pure triangular shape. The integral over \(T_\text{ovt}\) of the product of the maximum velocity and flow depth leads to the volume of an overtopping event. The integral is formulated below, see equation 4-25.

\[
\begin{align*}
 V_2\% &= \int_0^{T_\text{ovt}} u_2\% h_2\% \left( 1 - c_{u,s} \frac{t}{T_\text{ovt,2}\%} - c_{h,s} \frac{t}{T_\text{ovt,2}\%} + c_{u,s} c_{h,s} \frac{t^2}{(T_\text{ovt,2}\%)^2} \right) dt \\
 V_2\% &= u_2\% h_2\% \left. \left( \frac{t - c_{u,s} \frac{t^2}{2}}{T_\text{ovt,2}%} - \frac{c_{h,s} \frac{t^2}{2}}{T_\text{ovt,2}%} + \frac{c_{u,s} c_{h,s} \frac{t^3}{3}}{(T_\text{ovt,2}%)^3} \right) \right|_0^{T_\text{ovt}} \quad 4-25 \\
 V_2\% &= u_2\% h_2\% \left( \frac{T_\text{ovt,2}%}{3} - \frac{c_{u,s} \frac{T_\text{ovt,2}%}{2}}{2} - \frac{c_{h,s} \frac{T_\text{ovt,2}%}{2}}{2} + \frac{c_{u,s} c_{h,s} \frac{T_\text{ovt,2}%}{3}}{3} \right)
\end{align*}
\]

If a purely triangular shape is assumed for the variation in time of both the velocity and flow depth, equation 4-25 leads to equation 4-26:

\[
V_2\% = \frac{1}{3} u_2\% h_2\% T_\text{ovt,2}% \quad \text{with: } c_{u,s} = c_{h,s} = 1 \quad 4-26
\]

Where:
\[
\begin{align*}
 V_2\% &= \text{overtopping volume exceeded by 2% of the incoming waves} \ [\text{m}^3/\text{s/m}]
\end{align*}
\]
With discharge \( q = h u \) \([m^3/s/m]\) equations 4-23 to 4-26 can be visualized in Figure 4-25. One can see that the product of two triangles returns in a curved shape. The surface of this shape is equal to equation 4-26.

![Diagram](image)

*Figure 4-25 The product of velocity and flow depth in time shows the discharge in time*

Equation 4-26 can be rewritten in a dimensionless relation, see equation 4-27:

\[
\frac{V_{2\%}}{H_s} = c_s \frac{u_{2\%}}{\sqrt{gH_s}} \frac{h_{2\%}}{H_s} \frac{T_{overt,2\%}}{T_{m-1.0}}
\]

4-27

The coefficient \( c_s \) can be determined empirically and should be \( c_s = 1/3 \) if the flow depth and velocity in time can be considered to have a triangular shape. In Figure 4-26 the coefficient is determined to be \( c_s = 0.40 \).

![Graph](image)

*Figure 4-26 Measured against determined volumes, with \( c_s = 0.40 \)*

![Graph](image)

*Figure 4-27 Influence of coefficient \( c_s \) on the shape*

The difference in the shape of the graph has been drawn in Figure 4-27. This graph is again the product between the velocity and flow depth variation in time (right hand graph in Figure 4-25). The green line indicates the initial coefficient \( c_s = 0.33 \) and the red line the empirically determined value: \( c_s = 0.40 \).
The difference between 0.33 and 0.40 is very small. The variation of velocity and flow depth in time can be assumed to be triangular shaped. Equation 4-27 can be used for the determination of the volume of an overtopping event.

4.4.2 Cosine shape
Van den Bosch (2006) assumed that the variation of velocity and flow depth in time could be described with a cosine function, see equation 4-28 (repetition of equation 2-34):

\[ u(t) = u_{\max} \cos \left( \frac{\pi}{2T_{ovt}} t \right) \] (Van den Bos 2006) \hspace{1cm} 4-28

Van den Bosch’s research was limited to establish an estimation of the overtopping time; still it is interesting to investigate if the variation of flow depth and flow velocity in time can be assumed to have a cosine shape. The integration of the product of flow depth and velocity over the overtopping time is described below (equation 4-29). The shapes of the different graphs have been drawn in Figure 4-28.

\[
\begin{align*}
V_{2\%} &= \int_0^{T_{ovt}} u_{2\%} h_{2\%} \left( \frac{\pi}{2T_{ovt}} t \right)^2 dt = \frac{u_{2\%} h_{2\%}}{2} \left[ \frac{1}{2} - \frac{1}{2} \cos \left( \frac{\pi}{T_{ovt}} t \right) \right]_0^{T_{ovt}} \\
V_{2\%} &= u_{2\%} h_{2\%} \left( \frac{1}{2} - \frac{T_{ovt}}{2\pi} \sin \left( \frac{\pi}{T_{ovt}} t \right) \right)_0^{T_{ovt}} \\
V_{2\%} &= \frac{1}{2} u_{2\%} h_{2\%} T_{ovt}
\end{align*}
\] \hspace{1cm} 4-29

![Figure 4-28 The product of cosine shaped velocity and flow depth in time](image)

If equation 4-29 will be rewritten to the form of equation 4-26 the coefficient becomes \( c_v = 0.5 \). Coefficient \( c_v \) has been determined empirically and has a value 0.40. The triangular shape coincides much better than the cosine shape with the value of \( c_v \). The linear approach of velocity and flow depth variation in time (equations 4-23 and 4-24) show the best results and is considered as the best approach.
4.4.3 Relation between overtopping volume and wave run-up

Equation 4-26 gives a relation between the overtopping volume \( V_{2\%} \) and the parameters \( h_{2\%} \), \( u_{2\%} \) and \( T_{ovl,2\%} \). Relations for the parameters flow depth, velocity and overtopping time with the wave run-up have been found in the present report. If equations 4-13, 4-14 and 4-19 will be combined with 4-26, equation 4-30 can be derived:

\[
\frac{V_{2\%}}{H^2_s} = c'_{V,2\%} \left( \frac{Z_{2\%} - R_c}{H_s \gamma_c} \right)^2
\]

4-30

Where:
\( c'_{V,2\%} \) = empirical coefficient [–]

The value 2 is determined with the powers of the equations 2-25, 2-26 and 4-19:

\( X \cdot X^{0.5} \cdot X^{0.5} = X^3 \). Van Gent found the same relation for the maximum 2\% volume, see equation 4-31.

\[
\frac{V_{2\%}}{H^2_s} = c'_{V,2\%} \frac{1}{\sqrt{\gamma_c}} \left( \frac{Z_{2\%} - R_c}{H_s \gamma_c} \right)^2 \quad \text{(Van Gent 2002)}
\]

4-31

![Graph showing measured volumes against equation 4-30]( Figure 4-29 Measured volumes against equation 4-30 )

The coefficient is determined by Van Gent to be \( c'_{V,2\%} = 1 \) (Van Gent 2002). In the present report is the coefficient also determined in Figure 4-29 and has the value of \( c'_{V,2\%} = 0.89 \). Unfortunately only data of tests of Van Gent could be used, because the velocity measurements of Schüttrumpf were not reliable and the volumes could therefore not be determined. The coefficient \( c'_{V,2\%} \) should also be related to the outer
slope. From equation 4-13, 4-14 and 4-19 the coefficient should be in the range of \(x/\sin^3{\alpha}\), because:

\[
c_{v,2\%} = c_{h,2\%} \cdot c_{u,2\%} \cdot c_{\text{overt},2\%} = \frac{x}{\sin^3{\alpha}}
\]

4-32

This means that for a dike configuration with an outer slope of 1:6, the 2\% overtopping volume is a factor 3 larger than a dike with an outer slope of 1:4, because \(\sin^3(14)/\sin^3(9.5) = 3.2\).

The difference in overtopping volume of both different dike configurations with the same remainder of the wave run-up \((A = z_{2\%} - R_c)\) can also be determined with the distribution of wave overtopping volumes (equation 2-7). This has been done in Table 4-6

<table>
<thead>
<tr>
<th>Dike configuration</th>
<th>Slope</th>
<th>(H_s) [m]</th>
<th>(T_{m-1.0}) [s]</th>
<th>(\xi) [-]</th>
<th>(z_{2%}) [m]</th>
<th>(R_c) [m]</th>
<th>(A) [m]</th>
<th>(q_{\text{overt}}) [l/s/m]</th>
<th>(P_{\text{overt}}) [%]</th>
<th>(V_{2%}) [l/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>1:4</td>
<td>2.0</td>
<td>5.18</td>
<td>1.14</td>
<td>3.78</td>
<td>2.78</td>
<td>1.0</td>
<td>4.32</td>
<td>12.2</td>
<td>308</td>
</tr>
<tr>
<td>(2)</td>
<td>1:6</td>
<td>2.0</td>
<td>5.18</td>
<td>1.04</td>
<td>2.52</td>
<td>1.52</td>
<td>1.0</td>
<td>9.75</td>
<td>24.0</td>
<td>540</td>
</tr>
</tbody>
</table>

Table 4-6 2\% volumes for two different outer slopes and the same A

In this case the 2\% overtopping volume of a dike with an outer slope of 1:6 a factor 1.8 higher than the volume at a 1:4 dike \((504/308 = 1.8)\). This means that the relation given in equation 4-32 is not correct. The empirical coefficient is related to the outer slope angle, but not as indicated in equation 4-32. A possible equation for the coefficient \(c_{v,2\%}\) can be equation 4-33 below.

\[
c_{v,2\%} = \frac{0.106}{\sin^{1.5}{\alpha}}
\]

4-33

For a dike with an outer slope of 1:4 the coefficient becomes 0.89 and for a 1:6 dike the coefficient becomes 1.59 \((1.59/0.89 = 1.8)\). Unfortunately this equation cannot be compared to the data of Schüttrumpf as mentioned above.

The inconsistence has probably been caused by the fact that the two assumptions made in section 4.2.1 are not correct. The flow depths in point P en S (see Figure 4-2) are not equal and the shape of the overtopping tongue is probably different as well. However, there is a difference in overtopping volume, flow depth and velocity between both dike configurations. The outer slope is the main cause of this difference. More insight in the shape of the overtopping tongue will probably give the solution to this inconsistence.
4.5 Residence time

The total time of water coverage on the dike can be determined with the concept of residence time of the water, in other words the duration that water is present on the crest with a velocity and a flow depth larger than zero. The residence time has been defined in percentage of the total test time. The general formula for residence time is described in equation 4-34:

\[ T_{\text{res}} = \frac{1}{t_{\text{test}}} \sum_{r=1}^{n_{\text{test}}} \Delta t(r) \times 100 \]  

(Bergmann, 1994)  

Where:

\[ T_{\text{res}} \] = total residence time  

[\%]

Figure 4-30 A: velocity data irregular wave test Hannover (31050006). B: calculation of residence time for different threshold values on normal (B1) and logarithmic (B2) scale.

A part of a data record has been given in Figure 4-30-A. The data record is from velocities that have been measured of a test with irregular waves in the GWK. The two graphs in Figure 4-30-B represent the residence time. Both graphs give the same information, only the scales along the horizontal axis differ. The residence time of the water coverage in this test has been around 35%, shown in the extrapolation in B1. The residence time of a certain exceedance level can also be determined. For example: the residence time of water coverage exceeding 1 m/s is approximately
14% in this test, indicated with two red dotted lines in B1. The higher the exceedance level, the smaller the residence time of water coverage will be. The 2% exceedance level in this test $u_{2\%} = 3.45 \, \text{m/s}$, has a residence time of $T_{\text{res,2\%}} = 0.01\%$, thus nearly zero. The residence time of the water coverage is plotted on a logarithmic scale in Figure 4-30—B2. The residence time for different exceedance level shows a Rayleigh shaped distribution conform the red line in Figure 4-31. This matches with the theory that waves in a spectrum can be described by a Rayleigh distribution (in Holthuijsen 2000).

![Figure 4-31 Residence time for different exceedance levels of relative flow velocity ($u/u_{\text{max}}$) of test 3105006. The red-line is a Rayleigh distribution (Weibull distribution to the second power)]](image)

Figure 4-30 has just been an example of one test, but in fact all tests did return the same Rayleigh distribution for the residence time of different exceedance levels. Figures of these tests can be found in appendix C. A detailed description of the numerical input of the Rayleigh distribution can be written in appendix B. The residence time of a flow velocity exceedance value $u_{x\%}$ on the crest can be described with equation 4-35:

$$T_{\text{res,u}} = T_{\text{res}} \exp \left(-k_u \left( \frac{u}{u_{2\%}} \right)^2 \right)$$  \hspace{1cm} 4-35

Where:

$T_{\text{res,u\%}}$ = residence time exceeded by level $u$ \hspace{1cm} [%]

$u$ = flow velocity exceedance level \hspace{1cm} [m]

$k_u$ = coefficient \hspace{1cm} [-]

The residence time of a flow depth exceedance value $h_{x\%}$ on the crest can be described with equation 4-36:

$$T_{\text{res,h}} = T_{\text{res}} \exp \left(-k_h \left( \frac{h}{h_{2\%}} \right)^2 \right)$$  \hspace{1cm} 4-36

Where:

$T_{\text{res,h\%}}$ = residence time exceeded by level $h$ \hspace{1cm} [%]
Equation 4-35 and 4-36 will be valid for exceedance levels larger than the 2% value ($h \geq h_{2\%}$). The different parameters of equations 4-35 and 4-36 have been listed in Table 4-7.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Tests of Schüttrumpf and Van Gent</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_0$</td>
<td>6.14</td>
</tr>
<tr>
<td>$k_u$</td>
<td>2.67</td>
</tr>
<tr>
<td>$T_{res}$</td>
<td>0.12 - 0.53 (depended on $P_{ovt}$)</td>
</tr>
</tbody>
</table>

Table 4-7 Parameters for equations 4-35 and 4-36 for tests Schüttrumpf and Van Gent

Figure 4-32 has been given in order to explain equation 4-36. The 2% flow depth value for this test (31050006) was determined to be $h_{2\%} = 0.25$ m. Residence times of flow depth exceedance levels $h = \{3, 4, 5, \ldots, 23, 24, 25 \text{ cm}\}$ have been determined from the measurements. These values have been plotted against equation 4-36. Three flow depth exceedance levels have been indicated with the different lines in Figure 4-32; $h = 5$, $h = 10$ and $h = 15$ cm. This means that for example the residence time for exceedance level $h = 5$ cm is 9.5%. The crest of the dike model was covered with a flow depth larger than 5 cm for 9.5% of the total test time.

Figure 4-32 Different residence times of different exceedance levels of $h$ (measured and determined values)

Figure 4-33 shows the residence time of the water on the crest for different exceedance levels of the flow depth for seven tests of Schüttrumpf and Van Gent.
The residence time for different exceedance levels of the flow velocity has been given in Figure 4-34 on the next page. Only four tests of Schüttrumpf have been plotted in this figure. The residence times for the different exceedance levels have been determined with the use of the raw velocity data records. The figure shows a large deviation between the formula and the measured values close to the origin of the graph. This coincides with the findings in chapter 3, where was concluded that front velocities should be used instead of the measured velocities for tests of Schüttrumpf. For the highest velocities close to the origin of the graph, thus \( u/u_{2\%} = 1 \), the formula shows much higher residence times than actually was measured.
Figure 4-34 Residence time of different flow velocities exceedance levels against formula of four tests of Schüttrumpf
4.6 Comparison with other datasets

4.6.1 Maximum flow depth on the crest

A comparison with the new theory can be made with data from the small scale tests of Schüttrumpf. Schüttrumpf carried out small scale tests with two different outer slope angles. He found coefficients of 0.054 for a slope of 1:4 and 0.035 for a slope of 1:6 (Schüttrumpf 2001a). In this case he did use \( x_\ast \), which is the remainder of the wave run-up in horizontal direction, equation 3-7. The coefficients should in this case be written as equation 4-37, which is based on a cosine function instead of a sine function:

\[ c = \frac{0.203}{n \cdot \cos^2 \alpha} \quad \text{4-37} \]

The parameters for the different outer slopes can now be determined:

Slope 1:4 \( \rightarrow \) \( c = \frac{0.203}{4 \cdot \cos^2 14.0} = 0.054 \)

Slope 1:6 \( \rightarrow \) \( c = \frac{0.203}{6 \cdot \cos^2 9.46} = 0.035 \)

The values 0.054 and 0.035 are similar to the coefficients found by Schüttrumpf in his small scale tests.

4.6.2 Maximum flow velocity on the crest

4.6.2.1 Wave overtopping test Afsluitdijk (H24)

Within the wave overtopping model tests of the Afsluitdijk (Van der Meer 1987) the maximum front velocities have been obtained by intervals between flow depth gauges. The so-called 5% front velocities had maxima up to 7 m/s. The average outer slope angle was 9.5 degrees. The 5% velocities can be determined with the 5% wave run-up level, see equation 4-38:

\[ \frac{U_{5\%}}{\sqrt{gH_s}} = c'_{u,5\%} \left( \frac{z_{5\%} - R_c}{H_s} \right)^{0.5} \text{ with: } z_{5\%} = \sqrt{\frac{\ln(0.05)}{\ln(0.02)}} z_{2\%} \quad \text{4-38} \]

Where:

\[ z_{5\%} \quad = \text{5\% wave run-up} \quad [\text{m}] \]

\[ c'_{u,5\%} \quad = \text{empirical coefficient} \quad [-] \]

The 5% velocity values of Van Gent have been compared with values from the overtopping tests of the Afsluitdijk in Figure 4-35 and Figure 4-36.
The difference between the coefficient $c'_{u,5\%}$ calculated with tests of Van Gent and the Afsluitdijk model tests can be neglected, see Table 4-8.

### 4.6.2.2 Test of Van Gent with a different crest width

Van Gent carried out small scale tests with different configurations. In the present study only configuration (A) was used, see Figure 3-15 in chapter 3. Configuration (C) has been carried out with a crest width of $B_c = 1.1 \, m$. 

---

**Table 4-8** Velocity coefficient for 5% values of Van Gent and Afsluitdijk

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Van Gent (Figure 4-35)</th>
<th>Afsluitdijk (Figure 4-36)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c'_{u,5%}$</td>
<td>$c_{u,5%} = \frac{0.33}{\sin \alpha}$</td>
<td>$c_{u,5%} = \frac{0.38}{\sin \alpha}$</td>
</tr>
</tbody>
</table>
The 2% velocity values on the seaside and the landside of the crest of configuration (C) have been plotted Figure 4-37. The corresponding coefficients have been listed in Table 4-9.

<table>
<thead>
<tr>
<th>Coefficients</th>
<th>Configuration (C)</th>
<th>Configuration (A)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$B_c = 1.1 \text{ m}$</td>
<td>$B_c = 0.2 \text{ m}$</td>
</tr>
<tr>
<td>$c'_{u,2%}$</td>
<td>0.36/sin$\alpha$</td>
<td>0.30/sin$\alpha$</td>
</tr>
<tr>
<td>$c''_{u,2%}$</td>
<td>0.04</td>
<td>0.05</td>
</tr>
</tbody>
</table>

*Table 4-9 Coefficients determined for two different dike configurations*

4.6.3 Wave-averaged overtopping time

Tuan (2006) uses the residence time to determine the wave-averaged overtopping time, see equation 4-39.

$$T_{avg,ovt} = T_{res} T_m$$  \hspace{1cm} \text{(Tuan 2006)}  \hspace{1cm} 4-39

Where:

- $T_{avg,ovt}$ = wave-averaged overtopping time \hspace{1cm} [-]

Tuan considers a formula for wave-average overtopping time which is based on the theory of average overtopping discharge (equation 2-5). The wave-averaged overtopping time can be determined by equation 4-40.

$$T_{avg,ovt} \frac{s_m}{T_m \tan \alpha} = a_1 \exp \left( a_2 \frac{\sqrt{s_m}}{\tan \alpha \left( \frac{R_c}{H_{m0}} \right)^{3/2}} \right)$$  \hspace{1cm} \text{(Tuan 2006)}  \hspace{1cm} 4-40

Where:

- $a_1$, $a_2$ = shape factors \hspace{1cm} [-]

The shape factors $a_1$ and $a_2$ have been determined by Tuan (Tuan 2006). According to Tuan have the shape factors values of $0.22 < a_1 < 0.24$ and $-2.18 < a_2 < -1.67$. The wave-averaged overtopping times have been given for the tests of Schüttrumpf and Van Gent in Figure 4-38. The shape factors for these tests were $a_1 = 0.11$ and $a_2 = -0.27$. The black line has been found by Tuan in his study.
Although the shape factors differ a lot, the correlation between all studies was very good. In the study of Tuan the shape factors depend on the slope of the dike. With the large difference of the shape factors of Schüttrumpf and Van Gent it seems like there must be another (overtopping) parameter involved. This will not be investigated in the present study.

4.7 Conclusions
The maximum flow depth and maximum velocity depends of the outer slope. The overtopping time is independent of the outer slope. The variation in time of both flow depth and velocity can be approached by linear functions. On the crest a transition parameter was implemented in the formulae for maximum flow depth and velocity. The empirical coefficients found for the different formulae showed good similarities with other datasets. Extensive and detailed conclusions and recommendations have been given in sections 6.1 and 6.2.
5 Case study: The Wave Overtopping Simulator

5.1 Introduction
5.2 Calibration
5.3 Tests Delfzijl
5.4 Inner slope
5.5 Flow depth and flow velocity variation in time
5.6 Conclusion
5.1 Introduction

Up to now the present study has been based on data from tests of Schüttrumpf and Van Gent of wave overtopping on scale models of a dike. The wave overtopping simulator is a device which can simulate overtopping events on full scale on a real dike. With the release of large quantities of water on specific moments in time on the dike, the machine has been able to simulate a storm like it has been acting from the sea. It is important to stress that the principle is not to simulate the waves, the breaking of the waves and the run-up like in a wave flume. The objective is only to simulate the flow depths and velocities on the crest and the inner slope during a heavy storm. The prototype of the simulator had a width of one meter as shown in the picture in Figure 5-1. The picture is taken on a parking place during the calibration session. Figure 5-2 shows the final machine with a width four meter standing on a dike in Delfzijl.

![Figure 5-1 Prototype of the wave overtopping simulator in action during the calibration session](image1)

![Figure 5-2 The overtopping simulator on a dike in Delfzijl](image2)

The wave overtopping simulator was designed to simulate a (design) storm on the Dutch coasts. The average boundary conditions for waves during a storm along the Dutch dike coasts have been listed below:

- Wave height, $H_s = 2 \text{ m}$
- Peak wave period, $T_p = 5.7 \text{ s}$
- Mean wave period, $T_m = 4.7 \text{ s}$
- Outer slope, $\cot \alpha = 4$
- Storm duration, $t_{\text{storm}} = 6 \text{ h}$
- Number of waves, $n_{\text{waves}} = 4600$
The technical design conditions for overtopping discharge along the Dutch coasts were in the order of magnitude of 1 l/s/m, mentioned in chapter 1. With the wave overtopping simulator it was possible to simulate such a condition. The overtopping discharge must be continually pumped into the machine. The valve of the machine opens several times during six hours in order to simulate a storm. The overtopping discharge has been chosen predetermined. The moment of opening the valve specifies the volume of the simulated overtopping event. If the valve is closed, the machine is filled with water. The longer the valve is closed the larger the overtopping event will be after opening the valve. The different volumes during a storm can be described by a Weibull distribution. Equation 5-1 repeats equation 2-8.

\[ P_V = P(V \geq V) = \exp \left( -\left( \frac{V}{a} \right)^{0.75} \right) \], with \( a = \frac{0.84T_m q}{P_{ov}} \) (Van der Meer, 2006a)  

In the test program much larger overtopping discharges than 1 l/s/m have been tested. In fact when testing a larger discharge the crest freeboard has been supposed lower, since the outer slope and the wave conditions were kept constant. It has been assumed that the wave run-up distribution settles with the overtopping distribution. The run-up level, exceeded by probability \( x \), has a relation with the 2\% run-up level, equation 5-2.

\[ R_{u,x} = \frac{\ln(P_{ov})}{\ln(0.02)} Z_{2\%} \] (Van der Meer, 2006b)  

Where:
- \( R_{u,x} \) = run-up level exceeded by \( x \) percent of the incoming waves [m]
- \( P_{ovx} \) = probability of exceedance \( x \) [-]

<table>
<thead>
<tr>
<th>Overtopping discharge [l/s/m]</th>
<th>Corresponding crest freeboard [m]</th>
<th>Overtopping rate [%]</th>
<th>Number of overtopping events [-]</th>
<th>Largest overtopping volume [l/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>5.09</td>
<td>0.19</td>
<td>9</td>
<td>578</td>
</tr>
<tr>
<td>0.73</td>
<td>4.03</td>
<td>2.00</td>
<td>92</td>
<td>1071</td>
</tr>
<tr>
<td>1</td>
<td>3.86</td>
<td>2.77</td>
<td>128</td>
<td>1169</td>
</tr>
<tr>
<td>10</td>
<td>2.62</td>
<td>19.04</td>
<td>876</td>
<td>2659</td>
</tr>
<tr>
<td>20</td>
<td>2.25</td>
<td>29.45</td>
<td>1355</td>
<td>3735</td>
</tr>
<tr>
<td>30</td>
<td>2.03</td>
<td>36.86</td>
<td>1696</td>
<td>4663</td>
</tr>
</tbody>
</table>

*Table 5-1 Overtopping discharges with corresponding parameters for the Dutch dike coasts*

With equation 5-1 and 5-2 the number of overtopping events for different overtopping discharges can be determined, Table 5-1. A crest freeboard which may only be exceeded by 2\% of the incoming waves is 4.03 meter. The overtopping discharge is 0.73 l/s/m and the largest overtopping volume is 1071 l/m. When an overtopping discharge of 30 l/s/m occurs, the crest freeboard should be 2.03 m. The largest overtopping event is now 4663 l/m. The distribution of this overtopping discharge has been shown in Figure 5-3.
This distribution can be simulated with the wave overtopping simulator by releasing the different volumes shown in the text box and indicated with the blue line in Figure 5-3.
5.2 Calibration

The prototype was calibrated in 2006 with the at that time available studies of Van Gent and Schüttrumpf for maximum flow depth and velocity. Because of the discrepancy on the maximum flow depth, the machine was only calibrated on the maximum velocity corresponding with a certain volume, see Figure 5-4. The formula in the figure was the original formula of Van Gent and Schüttrumpf for the seaside of the crest, but with $c'_{u,2\%} = 1.33$ (mean of the initial coefficients of Schüttrumpf and Van Gent). One can see that the maximum flow velocity does not depend on the overtopping discharge. This can be explained by the fact that a higher overtopping discharge just has a lower crest level, or smaller crest freeboard, while the wave and dike parameters will be kept the same. An overtopping event with a certain volume will always have the same maximum velocity, it occurs more often when the overall overtopping discharge is higher.

From Figure 5-4 the ranges of the maximum velocity per overtopping volume could be determined. These velocities were measured with an electric magnetic velocity meter (EMS). The instrument was located on a height of 2 cm above a smooth surface and 2 m behind the overtopping simulator. The rate of measuring the velocity was low with 4 Hz (four measurements per second). Spikes on the data record occurred when the EMS was not submerged, thus during the pause between two waves. At the beginning of the measurement of a wave the EMS needs some adaptation time; this together with the low measuring rate caused the first part of the measurement to be very unreliable. The maximum per overtopping volume was determined with a manually drawn linear fit, Figure 5-5. The figure also shows a good example of the adaptation time of the EMS with the two spikes at $t=1$ of two waves.
During the calibration series different test set-ups were examined, such as the height of the valve, the width of the valve and the transition construction. The final results from the calibration series have been displayed in Table 5-2.

<table>
<thead>
<tr>
<th>Volume in simulator [l/m]</th>
<th>Maximum velocity (measured) [m/s]</th>
<th>Maximum velocity (required) [m/s]</th>
<th>Maximum flow depth (measured) [m]</th>
<th>Flow time $T_{ovt}$ (measured) [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>2.73</td>
<td>1.5 - 3.0</td>
<td>0.04</td>
<td>0.94</td>
</tr>
<tr>
<td>150</td>
<td>2.95</td>
<td>2.5 - 3.5</td>
<td>0.10</td>
<td>1.48</td>
</tr>
<tr>
<td>400</td>
<td>3.43</td>
<td>3.5 - 5.0</td>
<td>0.13</td>
<td>2.05</td>
</tr>
<tr>
<td>700</td>
<td>4.10</td>
<td>4.2 - 5.7</td>
<td>0.15</td>
<td>2.38</td>
</tr>
<tr>
<td>1000</td>
<td>4.62</td>
<td>5.0 - 6.5</td>
<td>0.18</td>
<td>2.78</td>
</tr>
<tr>
<td>1500</td>
<td>4.84</td>
<td>5.5 - 7.0</td>
<td>0.20</td>
<td>3.40</td>
</tr>
<tr>
<td>2500</td>
<td>6.47</td>
<td>6.0 - 8.0</td>
<td>0.26</td>
<td>4.24</td>
</tr>
<tr>
<td>3500</td>
<td>7.07</td>
<td>6.5 - 8.5</td>
<td>0.30</td>
<td>5.48</td>
</tr>
</tbody>
</table>

Table 5-2 Unithes measured at the final set-up of the overtopping simulator

The maximum velocities in column 2 have also been shown in Figure 5-4. One can see that the maximum velocities for the middle large waves were just below the required range. The fits of the remainder of the data has been adequate. Problems occurred with the flow depth measuring instrument. This was an acoustic depth meter. The measurements of the air entrainment flow were not reliable. Therefore the flow depths were read from the wall of the flume, see column 4 in Table 5-2. The acoustic depth meter was used to determine the overtopping time (column five). The maximum measured flow depths have been drawn against the formula found in the present study, see Figure 5-6. One can see that the maxima are close to the
maximum flow depths that have to occur 2 m behind the seaside of the crest (according to the formula). This corresponds with the fact that the measurements took place 2 m behind the wave overtopping simulator.

![Figure 5-6 Overlapping volume against maximum flow depth (formula present study)](image)

The maximum velocity results can also be drawn against the formulae found in the present study, see Figure 5-7.

![Figure 5-7 Overlapping volume against maximum flow velocity (formula present study)](image)
The overtopping time found in the calibration session have also been drawn against the overtopping time formula found in the present study, see Figure 5-8.

![Figure 5-8 Overtopping volume against overtopping time (formula present study)](image)

A higher measuring rate for the EMS would have lead to more accuracy in the calibration series, but as known from the tests of Schüttrumpf and Van Gent velocity measurements have been difficult to carry out. High deviations take place in the vertical distribution, due to the unsteadiness of the flow. It has therefore been advisable to determine the front velocity next time. Nevertheless, the maximum flow depth and maximum velocity do correspond quite well with the formulae found in the present study. The overtopping time has such a large deviation that the measurements or the analysis have to be doubt. In the next section, where the measurements of the tests in Delfzijl will be analysed, a new analysis will be carried out for the overtopping time.
5.3 Tests Delfzijl

After the calibration series a large wave overtopping machine was constructed (Figure 5-2) of four meter wide and a capacity to release a wave of 3500 l/m. The tests were carried out in March 2007 on a dike in Delfzijl (Groningen, Netherlands) in three sessions. The first test session took place on an original grass strip on the inner slope of the dike. The first overtopping discharges, such as the normal boundary condition of 0.1 l/s/m and 1 l/s/m, were to experience what would happen to the dike during storm conditions. The overtopping discharges were enlarged up to 50 l/s/m.

The second test session was carried out on a reinforced grass strip,. The so called “smart grass reinforcement” was placed a year before the tests took place. The grass had been lifted from the dike and geotextile was applied on the clay soil. Subsequently the grass had been replaced and the strip was given a year to grow. The roots of the grass had been able to attach with the geotextile and had therefore improved the strength of the inner slope. The reinforced grass strip should be able to withstand larger overtopping discharges than the original grass. On this dike section the same overtopping discharges were released as was done on the original section.

During the third test session the wave overtopping simulator was placed above a dike section where the grass was removed. The waves were acting on the bare clay. Because of the erosion of the clay and therefore the risk of collapsing of the dike, the largest overtopping discharge was 20 l/s/m.

5.3.1 Measurements

Figure 5-9 Flow depth and velocity meter on the inner slope in dry and wet condition

For the velocity measurements an electric magnetic velocity meter (EMS) was used, the same as during the calibration series. The flow depth was measured with a wave gauge. The knowledge of the calibration series was that an acoustic flow depth meter does not have an accurate output when the flow is high turbulent (with a lot of air entrainment). Problems with a wave gauge existing of two wires occur when the water runs up onto the wires. Therefore especially for these tests, WL-hydraulics has made wave gauges with very thin wires in order to keep this running up as low as
possible. In Delfzijl there was the possibility to measure the velocity and flow depth with a rate of one hundred times per second (100 Hz). The flow depth could be read from the wall of the flume as well. A camera filmed the tests for additional information. The velocity and flow depth measurements only took place during phase one; above the original grass and phase two; the reinforced section. Measuring above the clay was not possible because of the large erosion holes.

For the tests two couples of a wave gauge and a velocity meter were available. During the first test session on the original grass one couple was installed on the crest and the second couple on the inner slope. During the test session on the reinforced grass strip all instruments were located on the inner slope. The wave gauges were in all tests located 3 cm above the surface. The velocity meters have been located on different distances above the surface.

All positions are presented in Table 5-3. In the 2nd and 3rd column of the table is spoken about a 1st and a 2nd couple. The 1st couple is the couple of instruments (a flow depth and velocity meter) positioned on the highest location on the dike and the 2nd couple is positioned below (on the inner slope). The origin was the landward side of the crest, thus -0.40 meter is on the crest, 40 cm before the inner slope. 8.00 meter is on the inner slope and is the distance "s" measured along the slope.

<table>
<thead>
<tr>
<th>Test</th>
<th>1st couple [m]</th>
<th>2nd couple [m]</th>
<th>1 l/m [cm]</th>
<th>10 l/m [cm]</th>
<th>20 l/m [cm]</th>
<th>30 l/m [cm]</th>
<th>50 l/m [cm]</th>
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<td>Original section</td>
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<td>EMS: 2</td>
<td>EMS: 2</td>
<td>EMS: 2</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>WG: 3</td>
<td>WG: 3</td>
<td>WG: 3</td>
<td>WG: 3</td>
<td></td>
</tr>
<tr>
<td>Reinforced section</td>
<td>+2.20</td>
<td>+9.20</td>
<td>-</td>
<td>EMS: 5 / 7</td>
<td>EMS: 5</td>
<td>EMS: 5</td>
<td>EMS: 5</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>WG: 3</td>
<td>WG: 3</td>
<td>WG: 3</td>
<td>WG: 3</td>
</tr>
</tbody>
</table>

*Table 5-3 Positions of the instruments during the tests in Delfzijl*

In the following sections it is important to realize that a 1000 liter wave during a test with an overtopping discharge of 1 l/s/m is the same as a 1000 liter wave during an overtopping test with a discharge of 50 l/s/m. The larger waves just occur more often during the larger overtopping discharges.
5.3.2 Velocity results

First the results of the velocity measurements will be discussed. The prototype of the wave overtopping simulator had been calibrated on the velocity with the same EMS meter; therefore the velocity results from the large wave overtopping simulator can be compared best with the prototype. The measurements were filtered and analysed by WL Delft Hydraulics. Details of this filter method and data-processing have been described in the final report of the wave overtopping tests in Delfzijl (Van der Meer 2007), a summary of that description can be found in appendix D. The results of the velocity measurements have been plotted in Figure 5-10.

![Figure 5-10 Overtopping volumes against maximum velocities (EMS on 2cm, data-processing by WL Delft Hydraulics)](image)

The formula for maximum velocity for different overtopping discharges is drawn with the solid line. The formula from the present study has been used. The flow velocities were measured 1.5 m behind the seaward side of the crest. This means that $C'_{u,2\%} = 1.24$ is reduced with 5% to a value of 1.18.

One can see that for larger volumes the velocities were too low (the maxima of the 3500 liter waves were even lower than the maxima of the 2500 liter waves). This can have several causes:

1. Measurements were incorrect
   - Velocities can be checked by determining the front velocities.
   - Grass might have had a very large reduction on the flow velocity near the bottom and disturbed the flow. Velocities can be checked with measurements with the EMS on a larger distance than 2 cm from the surface, in any case on another position in the vertical distribution.
II. The analysis was incorrect

The analysis of WL Delft Hydraulics can be compared with the moving-average filter technique used in the present study for the analysis of the experiments of Schüttrumpf and Van Gent.

Both probable causes will be treated in the following sections.

5.3.2.1 Measurements validation

The velocities can be verified with the front velocities between the wave gauges. The front velocities were plotted together with the formula (equation 4-14) in Figure 5-11.

![Figure 5-11 Volumes against front velocities (data-processing by WL Delft Hydraulics)](image)

The front velocities for the smaller volumes were even higher than the formula. This has probably been caused by the fact that the second wave gauge was not placed on the crest but on the inner slope. As a consequence the flow will probably have accelerated between the wave gauges. Nevertheless, this gives an indication that the velocity of the water was higher than measured with the EMS on 2 cm above the surface.

The maximum velocities have been drawn with the EMS on 5cm in Figure 5-12. The maximum velocity shows a large scatter per overtopping volume. For example a wave overtopping event with a volume of 1500 l/m has a measured maximum velocity roughly between 3 and 5 m/s. This large deviation is due to the high unsteadiness and turbulence of the flow. Nevertheless the measurements with the EMS on a higher level show that the velocities were closer to the required values. This matches with the results of the analysis of the tests of Schüttrumpf, where only the front velocities have been used.
Figure 5-12 Volumes against maximum velocities with the EMS on 5 cm (data-processing by WL Delft Hydraulics)

The front velocities against the average maximum velocities are presented in Figure 5-13 and Figure 5-14 (definitions in appendix A). In the case for the EMS on 2 cm all front velocities were higher: all data is located at the right side of the black diagonal line. The average maximum velocity does not exceed the 3 m/s, this means that the maximum velocity on the inner slope was even lower than on the crest (hence Figure 5-10, where the maxima were beyond 4 m/s). Large scatter did appear when the EMS was located 5 cm above the surface. Although the front velocities were higher overall, the whole data follows the black diagonal line much better.
5.3.2.2 Analysis validation

During the tests in Delfzijl the overtopping discharge was known. This is rather special, because with irregular wave overtopping tests in a wave flume, the overtopping discharge is difficult to measure. To compare the analysis carried out by WL Delft Hydraulics with the filter technique in the present report, the 30 l/s/m test has been used.

The calculated discharge of the raw data was \( q_{\text{cal,raw}} = 23.23 \text{ l/s/m} \) (see appendix A for a definition). Within this calculation all the flow depth data below 3.5 cm were divined zero. For that reason, the integral has been taken over the period where the flow depth was above 3.5 cm.

The overtopping discharge calculated by WL Delft Hydraulics after filtering increased a bit: \( q_{\text{cal}} = 23.81 \text{ l/s/m} \). With the moving-average filter technique from the present study the discharge becomes: \( q_{\text{cal}} = 23.17 \text{ l/s/m} \). Figure 5-15 shows an example of a filtered wave.

![Discharge original: 23.23 [l/s/m] and Discharge filter: 23.17 [l/s/m]](image)

*Figure 5-15 Moving-average filtering of one wave on the crest. Flow depth (left) and velocity (right) in time of test Delfzijl011 (30 l/s/m) on the crest.*

The two graphs consist of the flow depth data and the velocity data of a wave on the crest. The volume of the wave was \( V = 700 \text{ liter} \). The spikes of the data disappear with the filter technique. The maximum velocity is 3.86 m/s with the moving-average filter technique, whilst the maximum velocity is only 3.08 m/s with the filter technique of WL Delft-Hydraulics. The front velocity for this wave was even 4.94 m/s, thus quite different.

All the maximum velocities for the 30 l/s/m test, determined with the two different filter techniques, are shown in Figure 5-16. The moving average filter technique was higher for most waves. The main difference between both filter techniques is that the filter technique of WL Delft Hydraulics consists of polynomial functions while the filter technique in the present report is a moving-average filter technique (described in appendix B).
5.3.2.3 Conclusion

The same problems occurred with measuring the velocities as during the large scale tests of Schüttrumpf. It seems that for a non-stationary, high turbulent flow the vertical distribution of the velocity is very diverse. Therefore, it is very difficult to measure the maximum velocity, because different volumes ask for a different height of the velocity meter. Another problem occurred with the tests in Delfzijl; the waves with a small volume and therefore small velocities were hard to measure because of the disturbance of the grass. Front velocities for large overtopping events have shown much more reliability than an EMS velocity meter or a velocity propeller.

Furthermore, it can be concluded that the way of filtering can lead to big differences in the final results. WL Delft Hydraulics has been well experienced in analysing hydraulic measurements. However the difference with the moving-average filter technique is quite large. This technique has proven to be reliable during the analysis of the tests of Schüttrumpf and Van Gent.

An overall view of all the velocity measurements is shown in Table 5-4. One should keep in mind that the front velocities are too high, because they were partly flowing over the inner slope and accelerating. The last column indicated the required velocities. These were the velocities which should be acting on the position on the crest, where the measurements took place (1.5 meter behind the seaside). The maximum velocities should only be 5% lower on this position than on the seaside of the crest. The calibration series fits well with the theory.
<table>
<thead>
<tr>
<th>Volume [l/m]</th>
<th>Max. velocity (Calibration series)</th>
<th>Max. velocity (Delfzijl)</th>
<th>Max. velocity (Delfzijl)</th>
<th>Front velocity (Delfzijl)</th>
<th>Front velocity (Delfzijl)</th>
<th>Required velocity (Delfzijl)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>EMS 2cm 2.73 [m/s]</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.8</td>
</tr>
<tr>
<td>150</td>
<td>-</td>
<td>2.5</td>
<td>3.8</td>
<td>3.8</td>
<td>-</td>
<td>2.5</td>
</tr>
<tr>
<td>400</td>
<td>3.43</td>
<td>2.8</td>
<td>4.1</td>
<td>4.2</td>
<td>-</td>
<td>3.4</td>
</tr>
<tr>
<td>700</td>
<td>4.10</td>
<td>3.3</td>
<td>4.3</td>
<td>5.0</td>
<td>-</td>
<td>4.0</td>
</tr>
<tr>
<td>1000</td>
<td>4.62</td>
<td>3.5</td>
<td>4.4</td>
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<td>-</td>
<td>4.5</td>
</tr>
<tr>
<td>1500</td>
<td>4.84</td>
<td>3.8</td>
<td>4.5</td>
<td>6.4</td>
<td>5.5</td>
<td>5.0</td>
</tr>
<tr>
<td>2500</td>
<td>6.47</td>
<td>4.0</td>
<td>4.7</td>
<td>6.7</td>
<td>6.7</td>
<td>5.7</td>
</tr>
<tr>
<td>3500</td>
<td>7.07</td>
<td>4.2</td>
<td>4.7</td>
<td>7.0</td>
<td>7.1</td>
<td>6.3</td>
</tr>
</tbody>
</table>

Table 5-4 Maximum velocities wave overtopping simulator

One can conclude that the prototype of the overtopping simulator suits the theory quite well. The large wave overtopping simulator shows much more deviations, although it was the same machine as the prototype. Measurements with instruments above grass are difficult. Front velocities can better be used in the future between smaller intervals.

5.3.3 Flow depth results

The measured flow depth maxima per overtopping event have been plotted in Figure 5-17. The solid lines represents the formulae for the maximum flow depth on the crest 1.5 m behind the seaside ($x_c = 1.5m$) for the different overtopping discharges.

![Figure 5-17 Volumes against maximum flow depth measurements (data-processing by WL Delft Hydraulics)](image-url)
The maximum flow depths were not only too low in comparison with the formula; they were also a lot lower than the maximum flow depths measured during the calibration session. This can have several causes, which will be treated in the following sections.

I. **Measurements are incorrect**
   - Wave gauge have not been calibrated right
   - During the calibration session the flow depth were read from the wall and not measured with an instrument.

II. **The analysis is incorrect**
   The analysis can be compared with the moving-average filter technique used in the present report for the analysis of the studies of Schüttrumpf and Van Gent.

#### 5.3.3.1 Measurements validation

The wave gauges have been calibrated during the tests. A large bucket has been used and both wave gauges were placed inside the bucket. The bucket was first filled with a little bit of water and the depth was measured manually and the output of the wave gauges was stored. The procedure was repeated until the bucket of water was filled with 50 cm of water, the total height of the wave gauges. The results have been plotted in Figure 5-18. It can be concluded that the wave gauges worked well in the case of measuring the depth of a stationary volume of water. 1 V on the crest did correspond with 5.18 cm. This coincides with the initial calibration numbers at the beginning of the test program in Delfzijl.

![Figure 5-18 Calibration results of the wave gauges](image)

What catches the eye is that the maximum flow depths were considerably lower than the ones measured during the calibration. A large difference between both test series was that the flow depths during the calibration were read from the wall. It looks like the wave gauge was not able to measure the flow depth correct in Delfzijl.
5.3.3.2 Analysis validation

As well as the velocity data, the flow depth data were also analysed with the filter technique from the present report. The results of the WL Delft Hydraulics filter techniques and the one from the present report are presented in Figure 5-19 for the 30 l/s/m test.

![Figure 5-19 Difference in maximum flow depth for two filter techniques](image)

The moving-average filter technique results in higher maximum flow depths. The scatter of both filter techniques was almost the same.

5.3.3.3 Conclusion

<table>
<thead>
<tr>
<th>Volume [l/m]</th>
<th>Max. flow depth (Calibration series) [m]</th>
<th>Max. flow depth (Delfzijl) WL-filter [m]</th>
<th>Max. flow depth (Delfzijl) Mov-Ave-filter [m]</th>
<th>Required flow depth (Delfzijl) [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0.04</td>
<td>-</td>
<td>-</td>
<td>0.03</td>
</tr>
<tr>
<td>150</td>
<td>0.10</td>
<td>0.07</td>
<td>0.08</td>
<td>0.06</td>
</tr>
<tr>
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<td>0.09</td>
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</tr>
<tr>
<td>700</td>
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<td>0.13</td>
<td>0.13</td>
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<td>0.12</td>
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<td>0.30</td>
<td>0.19</td>
<td>0.20</td>
<td>0.32</td>
</tr>
</tbody>
</table>

*Table 5-5 Maximum flow depth wave overtopping simulator*

Although the moving average filter technique results in higher flow depths than the filter technique of WL Delft Hydraulics, still the results show a large difference with the calibration session. A full overall view is visible in Table 5-5. Comparing the
maximum flow depths with the required maxima of the formula, one can see the same phenomenon as was visible for the maximum velocities. The maxima generated by the simulator for small volumes were too high, the middle size volumes matches with the theory and the maxima generated for the larger volumes were too low.

The reason that the measured flow depths are much smaller than the ones measured during the calibration is probably due to the fact that the wave gauges in Delfzijl could not measure the flow depth correct. That might have been caused by air entrainment in the high turbulent flow.

5.3.4 Overtopping time results

The results of the overtopping time per wave are noted in Figure 5-20.

![Figure 5-20 Overtopping volume against overtopping time per wave (data-processing by WL Delft Hydraulics)](image)

The overtopping times found in Delfzijl fit quite well with the formula. The figure shows a lot of high overtopping time values in the lower volume region. This looks like numerical errors, which can be caused by incorrect determining the beginning and the end of an overtopping event. An overtopping time of 7 seconds can be mistaken with a period between two waves instead of the overtopping time of one wave, certainly when the wave consists of only 400 liter of water. The analysis will be checked in the following section.

5.3.4.1 Analysis validation

The analysis of the overtopping times was carried out by WL Delft Hydraulics as well. The analysis is not a filter technique but a numerical program that recognizes different waves in a data record and determines the length of the individual waves.
For the analysis of the tests of Van Gent and Schüttrumpf in the present report such a numerical program has been written in Matlab (appendix B). The results of the analysis of the 30 l/s/m test are shown in Figure 5-21.

![Figure 5-21 Difference in overtopping time for two analyse techniques](image)

The results of the analysis of the present report look more consistent than the analysis of WL Delft Hydraulics. The high values in the lower volume region have disappeared.

### 5.3.4.2 Conclusion

<table>
<thead>
<tr>
<th>Volume [l/m]</th>
<th>Overtopping time (Calibration series) [s]</th>
<th>Overtopping time (Delfzijl) [s]</th>
<th>Required overtopping time (Delfzijl) [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>2.0</td>
<td>-</td>
<td>1.8</td>
</tr>
<tr>
<td>150</td>
<td>2.0</td>
<td>2.3</td>
<td>2.6</td>
</tr>
<tr>
<td>400</td>
<td>2.5</td>
<td>2.8</td>
<td>3.6</td>
</tr>
<tr>
<td>700</td>
<td>3.0</td>
<td>3.1</td>
<td>4.2</td>
</tr>
<tr>
<td>1000</td>
<td>3.5</td>
<td>3.4</td>
<td>5.1</td>
</tr>
<tr>
<td>1500</td>
<td>3.5</td>
<td>3.8</td>
<td>5.3</td>
</tr>
<tr>
<td>2500</td>
<td>3.5</td>
<td>3.8</td>
<td>5.3</td>
</tr>
<tr>
<td>3500</td>
<td>4.3</td>
<td>5.2</td>
<td>6.6</td>
</tr>
</tbody>
</table>

Table 5-6 Overtopping time results for the wave overtopping simulator

A comparison between the measured overtopping times in Delfzijl and the required values have been listed in Table 5-6. The overtopping time data measured during the calibration were considered to be not correct. The overtopping time was too low for the smaller volumes in Delfzijl.
5.3.4.3 Residence time Delfzijl

The residence time of the water on the dike in Delfzijl has been determined for different exceedance levels of the flow depth. Figure 5-22 shows an example of a 30 l/s/m test, but all tests gave the same results. The data record had a length of 2 hours.

![Figure 5-22 Residence time 30 l/s/m test Delfzijl for a two hour test](image)

If for all tests the residence time is calculated conform equation 4-31; Figure 5-23 can be produced. The shape factor in all cases has been \( k_h = 6.14 \), which was the same in the tests of Schüttrumpf and Van Gent. The total residence time \( D_t \) was different for all tests and is visible in Figure 5-24.

![Figure 5-23 Measured residence times of several tests against formula](image) ![Figure 5-24 Shape factor \( k_h \) against different overtopping discharges](image)

It seems that the total residence time \( D_t \) is linearly dependent on the discharge. This can be explained by the fact that during all the tests the same overtopping events took place, only the frequency increased with increasing discharge. The coefficients determined for equations 4-31 and 4-32 are in resemblance with the tests in Delfzijl. The Rayleigh distribution of the residence time of different exceedance levels seems to be a valid theory (see also appendix D for detailed information).
5.4 Inner slope

5.4.1 Velocity and flow depth

During the tests with original grass and reinforced grass a wave gauge and a velocity meter were installed on the inner slope at 8.00 m and 9.20 m respectively. With the shallow water equation the flow depth and the velocities could be determined on the inner slope per wave volume. The maximum velocities measured on different heights above the surface against the different volumes are plotted in Figure 5-25 and Figure 5-26. The required maximum velocity, determined with the shallow water equation, is drawn with solid lines for both tests. The front velocities and the measured flow depths were used as boundary condition at the landward side of the crest. Friction coefficient is $f = 0.02$.

![Figure 5-25](image1.png)  
*Figure 5-25 Maximum velocities on the inner slope with the EMS on 2 cm above the surface (original grass section, data-processing by WL Delft Hydraulics).*

![Figure 5-26](image2.png)  
*Figure 5-26 Maximum velocities on the inner slope with the EMS on 5 cm above the surface (reinforced grass section, data-processing by WL Delft Hydraulics).*

As described before, the velocities on the original section were measured at 2 cm above the surface, while the velocities on the reinforced section were measured at 5 cm above the surface. This is clearly visible in the two figures. The maximum velocities measured at 2 cm above the surface on the inner slope were even lower than the velocities measured on the crest. For the largest volumes the maxima were even decreasing in comparison to the smaller volumes. This is due to the vertical distribution in combination with the higher flow depths of these largest volumes (described in chapter 3).
Figure 5-27 and Figure 5-28 show the measured and determined flow depths on the inner slope. The boundary conditions at the landward side of the crest in the shallow water equation were the measured flow depths on the crest.

![Graph showing flow depth h at the inner slope vs. overtopping volume V per wave](image)

5.4.2 Overtopping time

Figure 5-29 shows the difference of the overtopping time on the crest and inner slope. Data records were obtained during measurements on the original grass section. The overtopping times were determined with data from the flow depth gauges. All tests of different discharges showed that the overtopping time on the inner slope increased in comparison to the overtopping time on the crest. This coincides with the results from tests of Schüttrumpf, where the same phenomenon happened. The opposite was shown in the tests of Van Gent, where the overtopping time did not change between the crest and the inner slope. In 4.3.3.3 is written that an explanation of this difference might be found in the scales of the tests. The small scale tests of Van Gent may be influenced by the surface tension of the water. The results in Delfzijl match with this explanation; influences of the surface tension can be neglected at this large scale.

Figure 5-30 shows a comparison between the overtopping times on the crest and inner slope of the two different data-processing methods. Both methods show almost the same results. The overtopping times in Figure 5-30 are presented dimensionless. The dimensionless distance from the landward side of the crest to the position on the inner slope is \( s/L_0 = 8/34.49 = 0.23 \). This value is within the range of the results shown in Figure 4-21. The linear fit through the data of Schüttrumpf in Figure 4-21 has been drawn in Figure 5-30 with a dotted line.
The fact that the overtopping times on the inner slope in Delfzijl were larger than during the tests of Schüttrumpf can be explained by the resistance of the grass; the water takes more time to flow down the inner slope.

### 5.5 Flow depth and flow velocity variation in time

During the tests with the wave overtopping simulator the released volumes were known very accurately. The previous chapter described that the variation of flow depth and flow velocity in time can be assumed to be triangular shaped. Equation 4-24 is repeated below with equation 5-3:

\[
\frac{V_{2\%}}{H_s^2} = c_s \frac{u_{2\%}}{\sqrt{gH_s}} \frac{h_{2\%}}{H_s} \frac{T_{ovt,2\%}}{T_{m-1.0}}
\]

For pure triangular shapes \(c_s = 0.33\). The coefficient has also been determined empirically with test data of Van Gent and had a value of \(c_s = 0.40\).

In Delfzijl the coefficient \(c_s\) is determined for different overtopping discharges in Figure 5-31. For \(u_{\text{max}}\) the front velocities were used. Only the test with an overtopping discharge of 10 l/s/m return the correct coefficient, see Figure 5-31. In Figure 5-17 one can see that during this test, the averaged measured flow depths were higher than during the other tests. The reason is not clear. The coefficient \(c_s\) has been too high in all the other tests, including the calibration series. This may have several causes. The shape of the overtopping volumes was not correct. One or more of the parameters \(u_{\text{max}}, h_{\text{max}}, T_{ovt,max}\) were not correct.
5.6 Conclusions

Measurements
Measurements with Electro Magnetic Velocity Meters and Wave Gauges of non-stationary turbulent flows were very unreliable. Especially the maximum values from these measurements have to be doubted. The measurements should be carried out in a different manner. The velocity should be determined with time intervals over a short distance. These time intervals can be measured with wave gauges. Flow depths can better be read from the wall directly during the tests or from video footage afterwards. These “old fashion” ways of measuring are recommended for tests with the wave overtopping simulator until other electronic devices are proven to be accurate.

Data-processing
Filter techniques and data-processing methods showed a large influence in the results. Well experienced filter techniques of WL Delft Hydraulics were not proven to be reliable in all cases. A “simple” moving-average filter technique showed good results.

Overtopping time
The overtopping time $T_{ovt}$ showed a correlation with the overtopping volume. The overtopping time increased on the inner slope in comparison to the crest.

Calibration
The wave overtopping simulator can best be calibrated with maximum flow depths in the future. Flow depth measurements show less scatter than velocity measurements, but you have to be able to measure them correctly.
Simulation
It can be concluded that the simulator worked quite accurate. Despite the difficulties in the measurements, the results show good similarities with model tests in wave flumes. The overtopping simulator generates adequate velocities for the different volumes (Table 5-4). The flow depths results were equal to the model tests as well, conform the measurements from the calibration series (Table 5-5). The overtopping times were a little bit to short for the smaller volumes (Table 5-6). This might also be caused by deviations in the measurements.
6 Conclusions and Recommendations

6.1 Conclusions
6.2 Recommendations
6.1 Conclusions

6.1.1 Objectives
The first main objective was to explain the difference between the studies of Schüttrumpf and Van Gent. This difference was found in the tests set-ups of their experiments. Both experiments have been carried out with different outer slope angles. The formulae were based on the vertical difference between the fictive wave run-up and the crest freeboard. The present study proved that the velocity and flow depth at the seaside of the crest should be a function of the outer slope angle as well. The outer slope angle $\alpha$ has been implemented in the empirical coefficients.

The second main objective was to study the duration and variation in time of flow velocities and flow depths on the dike due to overtopping. The present study explains the duration or $T_{overt}$ can be described as a function of the fictive wave run-up and crest freeboard to the power of 0.5. The variation of velocity and flow depth in time can be described as a linear function and is triangular shaped. With the integral of the product of velocity and flow depth over $T_{overt}$ one can determine the volume of an overtopping event. It turns out that the volume of an overtopping event is related to difference between the fictive wave run-up and crest freeboard as well.

The secondary objective was to study the results of the measurements with the wave overtopping simulator. The study demonstrated that velocities and flow depths on a large scale were difficult to measure. In the nearby future the velocity should be measured by the determination of front velocities between small intervals, instead of an EMS-device or velocity propeller. Flow depth can currently better be read from the wall instead of measuring them with an acoustic depth meter or wave gauge. In the long run must be thought of new ways to automatically measure the flow depth and velocity of high non-stationary turbulent flows. Although the measurement results were difficult to analyse, one can say that the wave overtopping simulator worked quite well. The hydraulic conditions due to overtopping during a storm on a dike were simulated adequate.

6.1.2 Seaward side
The maximum flow depth, velocity and overtopping time exceeded by 2% of the incoming waves can be described on the crest with the following formulae.

\[
\frac{h_{2\%}(x_c = 0)}{H_s} = 0.010 \left(\frac{z_{2\%} - R_c}{\gamma_c H_s}\right) \sin^2 \alpha
\]

6-1

\[
\frac{u_{2\%}(0)}{\sqrt{gH_s}} = 0.30 \left(\frac{z_{2\%} - R_c}{\gamma_c H_s}\right) \sin \alpha \sqrt{\frac{z_{2\%} - R_c}{\gamma_c H_s}}
\]

6-2

\[
\frac{T_{overt,2\%}(x_c = 0)}{T_{m-1.0}} = 1.15 \sqrt{\frac{z_{2\%} - R_c}{\gamma_c H_s}}
\]

6-3
Details of Equation 6-1 and 6-2 have been described by Schüttrumpf (Schüttrumpf 2001a).

6.1.3 Crest
A few adaptations to the initial formulae on the crest (Schüttrumpf, Van Gent 2003) have been made in this study; the transition between the outer slope and crest influences the flow depth and velocity at the first part of the crest. Despite a smooth surface, the flow depths decreased with approximately 20% at the first part of the crest. Hereafter the decrease of the flow depth was very small. Rough surfaces have not been analysed in the present study. Formulae on the crest become:

\[
\frac{h_{2\%}(x_c)}{h_{2\%}(x_c = 0)} = 0.81 \exp \left( -6 \frac{x_c}{\gamma_c L_0} \right) \quad 6-4
\]

\[
\frac{u_{2\%}(x_c)}{u_{2\%}(x_c = 0)} = \exp \left( -0.042 \frac{x_c}{\gamma_c h_{2\%}(x_c)} \right) \quad 6-5
\]

\[
\frac{T_{\text{ovt},2\%}(x_c)}{T_{\text{ovt},2\%}(x_c = 0)} = \left( 1.67 + 0.24 \ln \left( \frac{x_c}{L_0} \right) \right) \quad 6-6
\]

6.1.4 Volume
The maximum volume of an overtopping event exceeded by 2% of the incoming waves can be described in two ways. Equation 6-7 is based on the fact that the variation of flow depth and velocity in time is triangular shaped. The relation for the volume can therefore be based on the maximum flow depth, velocity and overtopping time. Equation 6-8 consists of a relation between the volume of an overtopping event and the remainder of the wave run-up.

\[
\frac{V_{2\%}}{H_s^2} = 0.40 \frac{u_{2\%}}{\sqrt{g H_s}} \frac{h_{2\%}}{H_s} \frac{T_{\text{ovt},2\%}}{T_{m-1.0}} \quad 6-7
\]

\[
\frac{V_{2\%}}{H_s^2} = c'_{V,2\%} \left( \frac{Z_{2\%} - R_c}{H_s \gamma_c} \right)^2 \quad 6-8
\]

The coefficient \( c'_{V,2\%} \) has only been determined for a 1:4 dike model and has the value of \( c'_{V,2\%} = 0.89 \). The coefficient for a 1:6 dike model should be \( c'_{V,2\%} = 1.59 \) considering the Weibull distribution of volumes during overtopping.
6.1.5 Wave run-up
The coefficients in equations 6-1, 6-2, 6-3 and 6-8 are slightly different than the ones mentioned before in the present report. The coefficients have been validated with different wave run-up equations. The equations 6-1, 6-2, 6-3 and 6-8 above have been validated with the 2% wave run-up determined with equations 6-9 and 6-10 below out of the TAW 2002 guidelines.

\[
\frac{z_{2\%}}{\gamma_f \gamma_s H_s} = 1.65 \cdot \xi_0 \quad \text{for} \quad \xi_0 \leq 1.75 \quad \text{(TAW 2002)} \quad 6-9
\]

\[
\frac{z_{2\%}}{\gamma_f \gamma_s H_s} = \left( 4.3 - \frac{1.6}{\sqrt{\xi_0}} \right) \quad \text{for} \quad \xi_0 > 1.75 \quad \text{(TAW 2002)} \quad 6-10
\]

6.1.6 Accuracy
The accuracy of above mentioned equations related to the difference of the fictive wave run-up and crest freeboard have known to be (Van Gent 2002):

\[
\frac{(z_{2\%} - R_c)}{H_s} < 1 \quad : \text{Accuracy is good}
\]

\[
1 < \frac{(z_{2\%} - R_c)}{H_s} < 1.6 \quad : \text{Accuracy is unknown}
\]

\[
\frac{(z_{2\%} - R_c)}{H_s} > 1.6 \quad : \text{Accuracy is poor}
\]

In the present study the coefficients have been determined with the coefficients found by Schüttrumpf and Van Gent (Schüttrumpf, Van Gent 2003) and validated with test data from both studies. All available test data had a value \((z_{2\%} - R_c) < 1\).

The coefficients \(c'_{h,2\%}\) and \(c'_{u,2\%}\) related to the outer slope angle have only been validated with outer slope angles \(\alpha = 14^\circ\) and \(\alpha = 9.5^\circ\) (outer slopes of 1:4 and 1:6). The accuracy of the relation beyond or in between these outer slope angles is unknown.

The equations 6-4 to 6-6 for the hydraulic conditions on the crest were validated for data in the range of: \(0.03 < x_c/L_0 < 0.3\). Lower values of \(x_c/L_0\) will be in the area, where the transition between outer slope and crest takes place.

6.1.7 Inner slope
The one-dimensional shallow water equation must be used to determine the flow depths and velocities on the inner slope (equation 6-11).

\[
u \frac{du}{ds} + g(\cos \beta \frac{dh}{ds} - \sin \beta) + \frac{1}{2} f_c \frac{u^2}{h} = 0 \quad \text{(Schüttrumpf, Van Gent 2003)} \quad 6-11
\]

An equation for the overtopping time has not been derived. Overtopping time increases on the inner slope and strongly depends on the friction coefficient of the inner slope.
6.1.8 Comparison with other dataset
The formulae have been compared to other datasets and showed appropriate results. Especially the comparison with the data from the small scale tests of Schüttrumpf showed a good resemblance with the implementation of the outer slope angle in the formulae. The tests, performed on two different outer slopes, showed the same discrepancy between the coefficients which can be solved by implementing the angle in the formula.

6.2 Recommendations

6.2.1 Experiments
In this study new formulae have been derived for the maximum flow depth, velocity and overtopping time on the crest. The report showed an inconsistency between the maximum overtopping volume of an individual wave event at different outer slopes. There are two methods to determine this maximum volume. The first method is through the wave overtopping volume distribution, equation 2-7. This method shows that the 2% exceedance level of an overtopping volume is a factor 2 larger on a dike with an outer slope of 1:6 than on a dike with an outer slope of 1:4. The second method to determine the 2% exceedance level is with the remainder of the wave run-up, equation 6-8. This equation shows a factor 3 difference between both outer slopes. That means that one or more of the new equations for maximum flow depth, velocity and overtopping time are not correct. The formulae for the velocity and overtopping time are physically explained and therefore very reliable. The formula for maximum flow depth on the crest has been derived by means of two assumptions. The tongue of a wave on the outer slope is assumed to have a certain shape. It is highly recommended to investigate the shape of a wave tongue on the outer slope. This might be carried out with a numerical analysis or with physical model scale tests. In addition the influences of the hydraulic conditions due to a berm on the outer slope can be investigated.

6.2.2 Instruments
Large scale model tests in a wave flume as well as tests with the overtopping simulator showed difficulties with measurements of velocities and flow depths. The use of EMS-devices, velocity propellers, wave gauges, and acoustic depth meters showed problems by measuring non-stationary high turbulent flows. It is recommended to study the possibilities of measuring these hydraulic conditions in another way, for example by a stereo photo technique or a laser technique.

6.2.3 Overtopping simulator
The wave overtopping simulator provides insight in the strength and capacity of dikes. Results of tests with the machine can be used for the design of overtopping on coastal structures in the future. Above all it is important that the machine simulates the hydraulic conditions correctly. The characteristics of both the flow velocities and flow depths in time presented in the present study, as well as the analysis of the wave overtopping tests in Delfzijl, can be used to improve the wave overtopping simulator for future experiments.
Despite the unreliable measurement results one can conclude that in Delfzijl the flow velocity, flow depth and overtopping time were simulated accurate on a dike with an outer slope of 1:4. Before the wave overtopping simulator is placed on a dike with an outer slope different than 1:4, the inconsistency described above in 6.2.1 has to be solved first. Not only the velocities and flow depths are related to the outer slope, but the overtopping volumes as well. When the simulator tests, for example, a dike with an outer slope of 1:6, the machine has to produce different volumes for the same storm condition. But it is possible that per individual volume the same velocities and flow depths have to be produced compared to a dike with an outer slope of 1:4. In that case no large adaptations to the simulator have to be made.
Acknowledgements

The development of this work took place with data from large scale tests in the GWK in Hannover and data from small scale model tests by WL Delft Hydraulics. The GWK data were obtained within the project "Loading of the inner slope of seadikes by wave overtopping" (Bundesministerium für Forschung und Technologie KIS 009). The data from WL were obtained within the context of Delft Cluster, project “Processes related to breaching of dikes”. The data from the wave overtopping simulator tests in Delfzijl were obtained within the Comcoast and Rijkswaterstaat SBW project “Strengths and Loads at Flood Defences” (in Dutch: “Sterkte Belasting Waterkeringen”).
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A Definitions

In this appendix a few definitions are listed used in the report.

A.1 Tests Schüttrumpf
When in the text is referred to “tests Schüttrumpf” there is referred to the tests carried out in the large wave flume in Hannover by the University of Braunschweig in 2001.

A.2 Tests Van Gent
When in the text is referred to “tests Van Gent” there is referred to the small scale tests carried out by WL Delft Hydraulics in 2001.

A.3 Overtopping time $T_{ovt}$
It is rather difficult to tell when an overtopping event ends. In fact: the crest and especially the inner slope are constantly wet. To be able to describe the flow velocities and flow depths variations in time, $T_{ovt}$ is divined in the following way:

The overtopping time $T_{ovt}$ is the duration of an overtopping event until the velocities are nearly zero. The flow depths are very small and considered to be negligible.

A.4 Measured discharge $q_{meas}$
The measured discharge, $q_{meas}$, is the discharge which is measured during the execution of the test. This can for example be done by an instrument that measures the water level in the reservoir which collects the overtopping water on the landside of the model. With the dimensions of the reservoir the discharge can be measured.

A.5 Calculated discharge $q_{cal}$
When there is referred to the calculated discharge $q_{cal}$, there is referred to the discharge which can be calculated with the data records of the flow depth and flow velocity, see equation A-1. This is done after filtering has been applied on the dataset.

$$ q_{cal} = \frac{\int_{0}^{t_{test}} h(t)u(t)dt}{1000} $$  \hspace{1cm} A-1

Where:

$q_{cal}$ = calculated overtopping discharge \hspace{1cm} [l/s/m]

$h$ = flow depth \hspace{1cm} [m]

$u$ = flow velocity \hspace{1cm} [m/s]

$t_{test}$ = total test duration \hspace{1cm} [s]

There is also a raw calculated discharge, this is the calculated discharge of the raw measurements, thus without filtering.
A.6 Correlation

A correlation coefficient can be used to determine the relationship between two datasets. The correlation is 1 in the case of an increasing linear relationship, −1 in the case of a decreasing linear relationship, and some value in between in all other cases, indicating the degree of linear dependence between the variables. The closer the coefficient is to either −1 or 1, the stronger the correlation between the variables. The correlation coefficient is described with equation 0-2.

\[
r(P, Q) = \frac{\sum_{i=1}^{n} (P_i - \bar{P})(Q_i - \bar{Q})}{\sqrt{\sum_{i=1}^{n} (P_i - \bar{P})^2 \sum_{i=1}^{n} (Q_i - \bar{Q})^2}}
\]

A-2

Where:
- \( r \) = correlation coefficient [-]
- \( P, Q \) = dataset [-]
- \( n \) = size dataset [-]
- \( \bar{P} \) = mean dataset P [-]
B Matlab scripts

B.1 Filteren

As an example test 31050009 will be filtered in two ways; with a phase-average filter technique and a moving-average filter technique. The CD-rom contains this script in separated files. The master file: "Inlezen_3_Phase_Average.m" contains references to the sub-files. These sub-files are used by several master files. In the script listed below are all files put together. This script can be executed into the matlab editor and should work right away.

B.1.1 Phase-average filter:

```matlab
clear; clc; clf;

%% Load complete dataset middle of the crest (position 3)
Test31050009 = load('data\3-31050009.txt'); % Time line
VX = Test31050009(:,4); % Velocity data
PX = Test31050009(:,2); % Pressure cell data
DX = Test31050009(:,3); % Depth gauge data
Tmean = 7.5; % Mean period
amount = 6; % Amount of overtopping events
n=length(tX); % Numerical time steps

%% Calibration
for k=1:n
    VX(k)=(VX(k))/1000; % Flow depth in cm
    PX(k)=(PX(k))/10; % Flow depth in cm
    DX(k)=((((DX(k))/1000)*2)-1); % Flow velocity in cm
end

%% Delete rustle
for k=1:n
    if VX(k) <= 0.2 % Deleting velocities < 0.2 m/s
        VX(k) = 0;
    end
    if PX(k) <= 2.5 % Deleting flow depth V 2.5 cm
        PX(k) = 0;
    end
    if DX(k) <= 2.5 % Deleting flow depth V 2.5 cm
        DX(k) = 0;
    end
end

%% Calculation discharge of raw data
QXP=0;QXD=0;
for k=1:n
    QXP=QXP+(VX(k)*PX(k)*0.0001); % Discharge with pressure cell
    QXD=QXD+(VX(k)*DX(k)*0.0001); % Discharge with depth gauge
end
QXP=QXP/(Tmean*amount);
QXD=QXD/(Tmean*amount);

%% Plot raw data
subplot(2,2,1);
plot(tX,VX); % Plot raw velocity propeller data
axis([80 125 0 4]);
```

IV
Definitions

xlabel(['Time [s]']);
ylabel(['Flow velocity [m/s]']);
title(['Velocity Pos. (3) 31050009']);
text(97,3.65,['Discharge (PxV): ',num2str(QXP), ' [m3/s/m]']);
text(97,3.15,['Discharge (DxV): ',num2str(QXD), ' [m3/s/m]']);
grid on;

subplot(2,2,2);
plot(tX,PX,'r'); % Plot raw pressure cell data
hold on;
plot(tX,DX,'k'); % Plot raw wave gauge data
axis([80 125 0 20]);
xlabel(['Time [s]']);
ylabel(['Flow depth [cm]']);
title(['Flow Depth Pos. (3) 31050009']);
grid on;

%% Phase-average filter
% Velocity
TintV=zeros(100,1); % Time of begin overtopping events
TintVV=zeros(100,1); % Step of begin overtopping events
m=1;
ConOne = 0;
ConTwo = 0;
for k=31:n-31
    V = [VX(k-2) VX(k-3) ... VX(k-30)]; % Vector V (30 steps in advance)
    ConOne = max(V); % Condition one is the maximum of V
    W = [VX(k+2) VX(k+3) ... VX(k+30)]; % Vector W (30 steps beyond)
    ConTwo = max(W); % Condition two is the maximum of W
    if VX(k-1) <= 0 && VX(k+1) > 0 && ConOne <= 0 && ConTwo > 0
        TintV(m)=tX(k); % Store time begin event
        TintVV(m)=k; % Store step begin event
        if m>1
            check = TintV(m) - TintV(m-1); % Check if it is a new event
            if check < 0.5
                TintV(m) = 0; % Stored values become zero
                TintVV(m) = 0; % if it is a not a new event
            end
        end
        m=m+1;
    end
end
TintV=nonzeros(TintV); % Delete all zero values, only the
TintVV=nonzeros(TintVV); % begin times of real events remain
amount=(size(TintV)); % Amount of overtopping events
steps = Tmean*90;
AIOV=zeros([steps amount]); % Create matrix for velocity data
q=1;
j=1;
w=size(VX);
w=w(1,1);
while q<=amount
    for m=1:amount
        j=TintVV(q)
        for k=1:steps
            if j <= w;
                AIOV(k,m) = VX(j);
            end
            j=j+1;
        end
    end
    q=q+1;
end

%
Definitions

end
q=q+1;
end

GemV=zeros([steps 1]); % Create phase-averaged vector
GemT=zeros([steps 1]); % Create time line for vector
m=0;
for k=1:steps
    GemV(k) = mean(AIOV(k,:)); % Execute phase-averaging
    GemT(k) = m;
    m=m+0.01;
end

% Pressure
...
% Same as the script for velocity

% Flow depth
...
% Same as the script for velocity

%% Maxima
MaxV=zeros([1 amount]); % Maxima of all overtopping events
MaxP=zeros([1 amount]);
MaxD=zeros([1 amount]);

for m=1:amount
    MaxV(m)=max(AIOV(:,m));
    MaxP(m)=max(AIOP(:,m));
    MaxD(m)=max(AIOD(:,m));
end

%% Smoothen
n=length(GemT); % Smoothen phase-averaged data
for k=7:n-7
    GemV(k)=(GemV(k-2)+GemV(k-1)+GemV(k)+GemV(k+1)+GemV(k+2))/5;
    GemP(k)=(GemP(k-2)+GemP(k-1)+GemP(k)+GemP(k+1)+GemP(k+2))/5;
    GemD(k)=(GemD(k-2)+GemD(k-1)+GemD(k)+GemD(k+1)+GemD(k+2))/5;
end

%% Calculating discharge filtered data
QXP=0;QXD=0;
for k=1:stappen
    QXP=QXP+VX(k)*PX(k)*0.0001; % Discharge with pressure cell
    QXD=QXD+VX(k)*DX(k)*0.0001; % Discharge with depth gauge
end
QXP=QXP/(Tmean);
QXD=QXD/(Tmean);

%% Plot phase-averaged data
subplot(2,2,3);
plot(GemT,GemV);
axis([0 Tmean 0 4]);
xlabel(['Time [s]']);
ylabel(['Flow velocity [m/s]']);
text(2.5,3.65,['Discharge (PxV): ',num2str(QXP),' [m3/s/m]']);
text(2.5,3.15,['Discharge (DxV): ',num2str(QXD),' [m3/s/m]']);
grid on;

subplot(2,2,4);
plot(GemT,GemP,'r');
hold on;
plot(GemT,GemD,'k');
Definitions

axis([0 Tmean 0 20]);
xlabel(['Time [s]']);
ylabel(['Flow depth [cm]']);
grid on;

%% Maxima
Max31050009_3=zeros([3 amount]);
Max31050009_3(1,:)=MaxV(1,:);
Max31050009_3(2,:)=MaxP(1,:);
Max31050009_3(3,:)=MaxD(1,:);
Max31050009_3=Max31050009_3';
save('output\Max31050009_3.txt','-ascii','-tabs','Max31050009_3');

Output:

<table>
<thead>
<tr>
<th>Event</th>
<th>Max V [m/s]</th>
<th>Max P [cm]</th>
<th>Max D [cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>3.7975</td>
<td>13.8625</td>
<td>17.5994</td>
</tr>
<tr>
<td>(2)</td>
<td>2.6204</td>
<td>14.6385</td>
<td>9.4153</td>
</tr>
<tr>
<td>(3)</td>
<td>2.8672</td>
<td>13.6246</td>
<td>17.6257</td>
</tr>
<tr>
<td>(4)</td>
<td>2.4474</td>
<td>16.7039</td>
<td>15.5647</td>
</tr>
<tr>
<td>(5)</td>
<td>2.6444</td>
<td>21.5294</td>
<td>17.6272</td>
</tr>
<tr>
<td>(6)</td>
<td>2.5772</td>
<td>14.1754</td>
<td>17.6038</td>
</tr>
</tbody>
</table>

Table B-1 Maximum values stored in the output file: "Max31050009_3.txt"

Figure B-1 Matlab output of the phase-average script
B.1.2 Moving-average filter

The moving-average filter exists of a rather simple principle. The Matlab script for this filter technique is almost equal to the phase-average filter technique described in the previous section. Only the part “Phase-average filter” until “Calculating discharge filtered data” must be replaced with the script data below.

```matlab
%% Moving-average filter
for k=7:n-7
    VX(k)=(VX(k-6)+VX(k-5)+VX(k-4)+VX(k-3)+VX(k-2)+VX(k-1)+VX(k)
    +VX(k+1)+VX(k+2)+VX(k+3)+VX(k+4)+VX(k+5)+VX(k+6))/13;
    PX(k)=(PX(k-6)+PX(k-5)+PX(k-4)+PX(k-3)+PX(k-2)+PX(k-1)+PX(k)
    +PX(k+1)+PX(k+2)+PX(k+3)+PX(k+4)+PX(k+5)+PX(k+6))/13;
    DX(k)=(DX(k-6)+DX(k-5)+DX(k-4)+DX(k-3)+DX(k-2)+DX(k-1)+DX(k)
    +DX(k+1)+DX(k+2)+DX(k+3)+DX(k+4)+DX(k+5)+DX(k+6))/13;
end
```

Output:

![Figure B-2 Matlab output of the moving-average script](image)
B.2 Data-processing overtopping time

The data processing script filters the data with a moving-average filter technique, subsequently the maximum flow depth, velocity and overtopping time will be calculated and placed in a text file. In the end the residence times of different exceedance levels will be determined as well. As an example wave overtopping test 1.07 of Van Gent will be used.

```matlab
% Calibratie + filteren + plotten TEST A 1.07
clear; clc; clf;

TestA107 = load('data\A107a.asc');
tX = TestA107(:,1);  \$ Time line
VX = TestA107(:,5);  \$ Velocities
DX = TestA107(:,3);  \$ Depth
QX = TestA107(:,2);  \$ Discharge
Tmean=2.14;
TestDuration = 2100;

% Delete rustle
% Moving average filter
rustle

% Plot filtered data
subplot(2,2,1);
plot(tX,VX,'linewidth',1);
axis([890 915 0 2]);
xlabel('Time [s]');
ylabel('Flow velocity [m/s]');
title('Flow velocity Pos. (P2) A 1.07');
grid on;

subplot(2,2,2);
plot(tX,DX,'k','linewidth',1);
axis([890 915 0 3]);
xlabel('Time [s]');
ylabel('Flow depth [cm]');
title('Flow depth Pos. (P2) A 1.07');
grid on;

% Residence Calculation

% Velocity
steps = size(VX);
ResV_X = [0.2 0; 0.3 0; ... 4.4 0; 4.5 0];  \$ Create output matrix
for k=1:steps
    if VX(k)>0.11 && VX(k)<=0.2
        ResV_X(1,2) = ResV_X(1,2) + 1;
    elseif VX(k)>0.2 && VX(k)<=0.3
        ResV_X(2,2) = ResV_X(2,2) + 1;
    elseif VX(k)>0.3 && VX(k)<=0.4
        ResV_X(3,2) = ResV_X(3,2) + 1;
    elseif VX(k)>4.4 && VX(k)<=4.5
        ResV_X(44,2) = ResV_X(44,2) +1;
    end
end
```
ResV_X(43,2) = ResV_X(43,2) + ResV_X(44,2); % Fill in the output matrix
ResV_X(42,2) = ResV_X(42,2) + ResV_X(43,2);
...
ResV_X(2,2) = ResV_X(2,2) + ResV_X(3,2);
ResV_X(1,2) = ResV_X(1,2) + ResV_X(2,2);

ResV_X(:,2) = ResV_X(:,2)/100; % Change time steps into seconds
ResV_X(:,2) = ((ResV_X(:,2))/TestDuration)*100; % Percentage of test time

% Depth
...

%% Residence plot
% Velocity
Y = 0;
YY = 0;

X = ResV_X(:,2);
X = nonzeros(X);
p = size(X);
MaxV_X = ResV_X((p+1),1);
for k=1:p
  Y(k) = (ResV_X(k,1))/(MaxV_X);
end
Y = Y';
X(p+1) = 0;
Y(p+1) = 1;

% Weibull distribution
A=-0.95; % Location parameter
B=0.51; % Scaling parameter
C=2; % Shape parameter
p=p+1;
Wb = [1 p];
for k=1:p
  Wb(k) = exp(-(((Y(k)-A)/B)^C))*1000; % Weibull distribution
end

subplot(2,2,3);
semilogx(0);
scatter(X,Y,'b');
hold on;
plot(Wb,Y,'r'); % Weibull distribution
axis([0 100 0 1]);
xlabel('Residence time [%]');
ylabel('Relative flow velocity [-]');
grid on;

% Flow depth
...

%% interval calculation
% Velocity
n=length(tX);
TintV=zeros(100,6);
TintVV=zeros(100,2);
m=1;
ConOne = 0;
for k=6:n-6
V = [VX(k-1) VX(k-2)];
ConOne = max(V);
if ConOne <= 0.001 && VX(k) > 0.001
    TintV(m,1)=tX(k);
    TintVV(m,1)=k;
    m=m+1;
end
end
m=1;
ConTwo = 0;
for k=6:n-6
W = [VX(k+1) VX(k+2)];
ConTwo = max(W);
if ConTwo <= 0.001 && VX(k) > 0.001
    TintV(m,2)=tX(k);
    TintVV(m,2)=k;
    m=m+1;
end
end
kolom1 = TintVV(:,1);
kolom2 = TintVV(:,2);
kolom1 = nonzeros(kolom1);
kolom2 = nonzeros(kolom2);
TintVV = {[kolom1 kolom2]};
amount=size(TintVV);

% Flow depth
% same script as above for velocity

% Hmax Umax en V
MaxD = 0;
MaxV = 0;
Volume = 0;
for j=1:amount
    p=TintVV(j,1);
    q=TintVV(j,2);
    for k=p:q
        Volume = Volume + (VX(k)*DX(k)*0.02); % Calculating volume per wave
        if VX(k) > MaxV
            MaxV = VX(k); % Calculating maximum velocity
        end
        if DX(k) > MaxD
            MaxD = DX(k); % Calculating maximum flow depth
        end
    end
    TintV(j,3)=TintV(j,2)-TintV(j,1); % Create output table
    TintV(j,4)=MaxD;
    TintV(j,5)=MaxV;
    TintV(j,6)=Volume;
    MaxD = 0;
    MaxV = 0;
    Volume = 0;
end
save('output\Table107_P2.txt','-ascii','-tabs','TintV'); % Save table
Definitions

Output:

![Figure B-3 Matlab output of the residence time script](image)

<table>
<thead>
<tr>
<th>$t_{\text{begin}}$</th>
<th>$t_{\text{eind}}$</th>
<th>$T_{\text{ovt}}$</th>
<th>$h_{\text{max}}$</th>
<th>$u_{\text{max}}$</th>
<th>$V$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9386</td>
<td>2.6874</td>
<td>1.7488</td>
<td>0.0047</td>
<td>0.7270</td>
<td>0.0013</td>
</tr>
<tr>
<td>6.3726</td>
<td>7.5286</td>
<td>1.1560</td>
<td>0.0073</td>
<td>1.1083</td>
<td>0.0049</td>
</tr>
<tr>
<td>9.8010</td>
<td>11.4806</td>
<td>1.6796</td>
<td>0.0062</td>
<td>0.9082</td>
<td>0.0043</td>
</tr>
<tr>
<td>18.0903</td>
<td>19.2858</td>
<td>1.1955</td>
<td>0.0032</td>
<td>0.8719</td>
<td>0.0018</td>
</tr>
<tr>
<td>54.5079</td>
<td>55.328</td>
<td>0.8200</td>
<td>0.0014</td>
<td>0.3443</td>
<td>0.0001</td>
</tr>
<tr>
<td>60.9793</td>
<td>62.0563</td>
<td>1.0769</td>
<td>0.001</td>
<td>0.4932</td>
<td>0.0002</td>
</tr>
<tr>
<td>68.7154</td>
<td>69.5354</td>
<td>0.8200</td>
<td>0.0035</td>
<td>0.7041</td>
<td>0.0007</td>
</tr>
<tr>
<td>...</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1459.789</td>
<td>1460.491</td>
<td>0.7015</td>
<td>0.0019</td>
<td>0.1801</td>
<td>0.0001</td>
</tr>
<tr>
<td>1462.912</td>
<td>1464.127</td>
<td>1.2152</td>
<td>0.0056</td>
<td>0.6373</td>
<td>0.0016</td>
</tr>
</tbody>
</table>

Table B-2 The output file: “table107_P2..txt”

More output files of tests of Schüttrumpf and Van Gent have been given in the next appendix C.
C  Data of tests Schüttrumpf and Van Gent

C.1 Available data from Schüttrumpf and Van gent in the present study

| Test | $d_{toe}$ | $R_c$ | $H_s$ | $T_{m-1.0}$ | $T_p$ | $q$ | $q_{ct}$ | $P_{ovt}$ | $L_0$ | $\xi$ | $z_{2\%}$ | $h_{2\%}/H_s$ | $h_{2\%}/H_s$ | $u_{2\%}/gH_s^{0.5}$ | $u_{2\%}/gH_s^{0.5}$ | $T_{ovt2\%}/T_{m-1.0}$ | $T_{ovt2\%}/T_{m-1.0}$ | $V_{2\%}/H_s^2$ |
|------|----------|-------|-------|-------------|-------|-----|--------|----------|-------|-------|----------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|
| 1.01 | 0.30     | 0.30  | 0.139 | 2.21       | 2.51  | 0.270 | -      | 19       | 7.626 | 1.852 | 0.373    | 0.060          | 0.047          | 0.865          | 0.951          | 0.738          | 0.724          | 0.305          |
| 1.02 | 0.30     | 0.30  | 0.133 | 1.88       | 2.02  | 0.050 | -      | 5        | 5.518 | 1.610 | 0.311    | 0.020          | 0.011          | -              | -              | -              | -              | -              |
| 1.03 | 0.35     | 0.25  | 0.147 | 2.16       | 2.50  | 0.760 | -      | 30       | 7.284 | 1.760 | 0.375    | 0.102          | 0.086          | 1.199          | 1.166          | -              | -              | 0.504          |
| 1.04 | 0.35     | 0.25  | 0.137 | 1.84       | 2.00  | 0.170 | -      | 13       | 5.286 | 1.553 | 0.308    | 0.045          | 0.036          | 0.578          | 0.768          | -              | -              | 0.233          |
| 1.05 | 0.40     | 0.20  | 0.150 | 2.14       | 2.49  | 2.190 | 2.380  | 51       | 7.150 | 1.726 | 0.375    | 0.174          | 0.139          | 1.467          | 1.294          | -              | 0.919          | 0.904          |
| 1.06 | 0.40     | 0.20  | 0.140 | 1.79       | 1.99  | 0.520 | 0.443  | 27       | 5.003 | 1.494 | 0.303    | 0.092          | 0.068          | 0.973          | 1.041          | -              | 0.828          | 0.405          |
| 1.07 | 0.40     | 0.20  | 0.135 | 1.51       | 1.59  | 0.140 | 0.107  | 14       | 3.560 | 1.284 | 0.251    | 0.047          | 0.033          | 0.469          | 0.695          | -              | 0.772          | 0.199          |
| 1.08 | 0.45     | 0.15  | 0.143 | 1.78       | 1.98  | 1.370 | -      | 50       | 4.947 | 1.470 | 0.305    | 0.144          | 0.110          | 1.326          | 1.216          | -              | -              | 0.626          |
| 1.09 | 0.45     | 0.15  | 0.138 | 1.51       | 1.59  | 0.590 | -      | 40       | 3.560 | 1.270 | 0.254    | 0.092          | 0.062          | 1.031          | 1.057          | -              | -              | 0.336          |
| 1.10 | 0.50     | 0.10  | 0.140 | 1.49       | 1.59  | 2.280 | -      | 63       | 3.466 | 1.244 | 0.253    | 0.182          | 0.114          | 1.416          | 1.263          | 1.041          | 1.081          | 0.818          |

Table C-1 Tests Van Gent with configuration A used in the present study

| Test | $d_{toe}$ | $R_c$ | $H_s$ | $T_{m-1.0}$ | $T_p$ | $q$ | $q_{ct}$ | $P_{ovt}$ | $L_0$ | $\xi$ | $z_{2\%}$ | $h_{2\%}/H_s$ | $h_{2\%}/H_s$ | $u_{2\%}/gH_s^{0.5}$ | $u_{2\%}/gH_s^{0.5}$ | $T_{ovt2\%}/T_{m-1.0}$ | $T_{ovt2\%}/T_{m-1.0}$ | $V_{2\%}/H_s^2$ |
|------|----------|-------|-------|-------------|-------|-----|--------|----------|-------|-------|----------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|
| 31050006 | 5.001 | 0.000 | 0.728 | 6.71       | 6.17  | 11.20 | 9.776  | 16      | 70.36 | 6.388 | 1.720    | 0.246          | 0.146          | 0.953          | 0.900          | 0.565          | 0.657          | 2.36           |
| 09050006 | 5.010 | 0.990 | 0.872 | 3.90       | 4.37  | -    | 4.843  | 22      | 23.70 | 0.869 | 1.338    | 0.201          | 0.139          | 1.036          | 0.932          | 0.855          | 0.826          | 1.04           |
| 25050003 | 5.010 | 0.990 | 0.880 | 4.20       | 2.98  | 7.98  | 7.727  | 25      | 27.56 | 0.933 | 1.429    | 0.224          | 0.155          | 0.727          | 0.385          | 0.843          | 0.890          | 1.70           |
| 16060003 | 3.701 | 0.300 | 0.921 | 4.16       | 3.59  | 1.50  | 1.358  | 7.3     | 27.01 | 0.903 | 1.457    | 0.115          | 0.041          | 0.815          | 0.563          | 0.548          | 0.442          | 0.25           |
| 24050009 | 5.010 | 0.990 | 0.903 | 4.14       | 4.02  | 3.66  | 4.754  | 9.1     | 26.73 | 0.907 | 1.434    | 0.171          | 0.077          | 0.892          | 0.786          | 0.696          | 0.694          | 0.57           |

Table C-2 Tests Schüttrumpf used in the present study
C.2 Output graphs

C.2.1 Filtered flow depths and velocities

The three figures below show samples from filtered data of tests carried out by Van Gent. The data has been filtered with a moving-average filter technique, described in the appendix B.

![Flow velocity and depth graphs for tests 1.05, 1.06, and 1.07](image)

*Figure C-1 Test 1.05 Van Gent, landside of the crest*

*Figure C-2 Test 1.06 van Gent, landside of the crest*

*Figure C-3 Test 1.07 Van Gent, landside of the crest*
Data of tests Schüttrumpf and Van Gent

**Figure C-4** Data on the seaside of the crest of test 09050006 of Schüttrumpf

**Figure C-5** Data on the landside of the crest of test 09050006 of Schüttrumpf
The flow depth, pressure and velocity data have been given for test 31050010 in Figure C-6 below. This is a test with regular waves, $H = 1m$ and $T_m = 9.5s$. 

Position 1 (seaside crest: $x_c = 0m$)  Position 2 (crest: $x_c = 0.50m$)  Position 3 (middle crest: $x_c = 1.00m$)  Position 4 (crest: $x_c = 1.50m$)
One can see that the flow depth measurements on the inner slope are unreliable from the wave gauge. This has been described in chapter 3. The data from the pressure cells can better be used instead.
Data of tests Schüttrumpf and Van Gent

C.2.2 Residence times

Residence times of three tests carried out by Van Gent. The red continues lines are Rayleigh distributions.

Figure C-7 Flow depth residence time of different exceedance level of test 1.05 (Van gent)

Figure C-8 Flow depth residence time of different exceedance level of test 1.06 (Van gent)
Data of tests Schüttrumpf and Van Gent

Figure C-9 Flow depth residence time of different exceedance level of test 1.07 (Van gent)

Figure C-10 Residence time of different exceedance levels of test 09050006. 
A: seaside of the crest and B: landside of the crest.
C.3 Comparison
In the present report has been described that the difference of the flow depth and velocity between the studies of Van Gent and Schüttrumpf has been caused by the difference in the angle of the outer slope. In this section some other parameters will be compared to give the reader more insight in both experiment set-ups.

- Difference in wave flume
- Difference in water depth
- Difference in breaker parameters
- Difference in overtopping rate

C.3.1 Wave flume
A difference between the wave two flumes is that the Scheldt-flume of WL used by Van Gent is equipped with an active wave absorption system to minimise reflection off the wave board. This means that wave reflection from the model is measured and is accounted by the motion of the wave board. The wave board absorbs these reflecting waves and prevents these waves from re-reflecting and disturbing the measurements. The GWK wave flume in Hannover does not have such equipment installed. Second order long-periodic resonance waves can disturb the measurements.

C.3.2 Water depth
The water depth might have an influence in the flow depth and flow velocities on the dike. Deep water surface profiles does not much deviate from a cosine shape, while in shallow water the troughs are flatter and the peaks are steeper. This is called the dissimilarity of the waves. A way to quantify this dissimilarity is with the number of Ursell, equation C-1.

\[ U = \frac{H L^2}{d^3} \]  
(Battjes, 2001) C-1

Where:
- \( U \) = number of Ursell  [-]
- \( d \) = water depth  [m]
- \( L \) = wave length  [m]

In Figure C-11 the numbers of Ursell have been plotted against the maximum flow depths. In this case there is no correlation. In Figure C-12 the dimensionless water depth are presented against the maximum flow depth. In this case it looks like there seems to be a correlation. It attracts the attention that the ratio \( d/H_i \) is twice as high in the tests carried out by Schüttrumpf.
The dimensionless water depth can be implemented in the formula, see Figure C-13. The values of Schüttrumpf for the maximum flow depth come very close to Van Gent's values, but the flow depths of Schüttrumpf are still larger. The correlation is decreasing as well, thus with the original formula with the different coefficients (0.15 and 0.33) a higher correlation was found than when the water depth is implemented.
C.3.3 Breaker parameter
The breaker parameter is already participating in the formulae of Schüttrumpf and Van Gent, since the wave run-up height depends on the breaker parameter and the wave height.

![Figure C-14 Breaker parameter against flow depth](image)

In Figure C-14 the breaker parameter values have been given against the maximum flow depth values. One can see that for four tests of Schüttrumpf the breaker parameters were almost the same (around 0.9) while the maximum flow depths were completely different. This data does not show an extra, underlying correlation between the maximum flow depth and the breaker parameter.

C.3.4 Overtopping rate
The overtopping rate can also have an influence on the 2% values of the flow depth and flow velocity. The 2% values are dependent on the amount of incoming waves. Thus if 1000 waves are generated in the flume, the 20th highest overtopping event determines the 2% value. If the overtopping rate is very high, a lot of overtopping events take place. This generates a higher probability that the 20th event will be larger, because there are more events.

In Figure C-15 the overtopping rate has been given against the maximum flow depth. It looks like there is a (linear) correlation between the maximum flow depth and the overtopping rate: the higher the overtopping rate the higher the flow depth, especially for the data of Van Gent this correlation looks quite well. The correlation is a logical result if one takes a look to the relation between wave heights, wave period, flow depth, velocity and overtopping rates. But the flow depths measured by Schüttrumpf were still higher than Van Gent’s, despite the smaller overtopping rates in his tests.
C.3.5 Conclusion

Besides the outer slope as a cause for the discrepancy between the flow depths and velocities in the studies of Schüttrumpf and Van Gent, there seems not to be another (wave overtopping) parameter involved.
D Extra data Delfzijl

D.1 Filtering and data processing by WL Delft Hydraulics

Data of the tests carried out in Delfzijl with the wave overtopping simulator have been filtered by WL-Delft hydraulics. WL used a Finite-Impulse-Response Filter Technique (FIR). The results from such a filter technique are presented in Figure D-1 and Figure D-2 below.

![Figure D-1 Flow depth after filtering with a FIR filter technique](image1)

![Figure D-2 Velocity after filtering with a FIR filter technique](image2)
Upon the filtered flow depth 3 cm has been added (Figure D-1), because the wave gauge was located 3 cm above the surface during the tests. In Figure D-2 have the measured, thus raw velocity, been moved 0.5 m/s downwards and the filtered velocity 0.5 m/s upwards, in order to give a clear view of both data records.

Furthermore WL-Delft Hydraulics has processed output tables for all tests carried out in Delfzijl. The layout of such is shown in Figure D-3. Columns 1-5 are data from the instruments on the crest and columns 6-10 are data from the inner slope. The integral in the 5th column and 10th column is the integral over time of the product of velocity and flow depth. In this way one is able to calculate the volume of the wave. An advantage of the wave overtopping simulator is that one knows the exact volume of water that was released during the test per wave. These “produces” volumes are presented in the last column (column 11). The volume calculated by the integral is, especially for larger wave, too small in comparison to the produced waves. This is due to the fact that a part of the water flows beneath the instruments and therefore has not been measured. Figure D-4 shows the result of a comparison between the calculated and produced wave volumes of several tests on the crest. The smaller volumes show some deviation, but the larger volumes show very large deviations.

Is this deviation only due to water that flows beneath the instruments or are the measurements unreliable? The measurements are probably unreliable as was concluded in chapter 5. The figure shows that for the larger volumes only the half has been measured. From video footage it is clear that not the half of the volume flew beneath the instruments. This is another indication that instruments were not able to measure correctly.
D.2 Comparison crest and inner slope

In this section a few extra figures which show the difference between the measurements on the crest and inner slope. All figures are from the test with the original grass section and all data have been processed by WL-Delft Hydraulics.
**Figure D-6** Comparison between the velocity on the crest and inner slope

**Figure D-7** Comparison between the overtopping time on the crest and inner slope
D.3 Residence time

Delfzijl test005, discharge of 1 l/s/m, residence time is 0.6%.

Delfzijl test015, discharge of 10 l/s/m, residence time is 2%.

Delfzijl test009, discharge of 20 l/s/m, residence time is 5%.

Delfzijl test011, discharge of 30 l/s/m, residence time is 7%.

Delfzijl test023, discharge of 30 l/s/m, residence time is 7%

Figure D-8 Residence time for different exceedance levels of five tests in Delfzijl
The formula for the residence time of different exceedance levels is repeated below:

\[ T_{\text{res},h} = T_{\text{res}} \exp\left(-k_h \left( \frac{h}{h_{50\%}} \right)^2 \right) \]  

D-1

The residence times for different exceedance levels of test005 and test011 have been given in Table D-1 below. The 4th and 8th column show the residence times determined with equation D-1.

<table>
<thead>
<tr>
<th>Test</th>
<th>011</th>
<th>( k_h )</th>
<th>6.138</th>
<th>Test</th>
<th>005</th>
<th>( k_h )</th>
<th>6.138</th>
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<tr>
<td>( h_{\text{max}} )</td>
<td>0.21 cm</td>
<td>( T_{\text{res}} )</td>
<td>0.07</td>
<td>( h_{\text{max}} )</td>
<td>0.12 cm</td>
<td>( T_{\text{res}} )</td>
<td>0.0057</td>
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<td>residence</td>
<td>time</td>
<td>per</td>
<td>exc.</td>
<td>level</td>
<td>( h/ h_{\text{max}} )</td>
<td>Equation</td>
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<td>0.05</td>
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<td>0.049429</td>
<td>0.003976</td>
<td>0.03</td>
<td>0.2500</td>
<td>0.003884</td>
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Table D-1: Residence time values of 2 test inclusive the values determined with eq. D-1

The calculations in Table D-1 above have been carried out for five tests of Delfzijl. The results are presented in Figure D-9.
Figure D-9 The residence times of different exceedance levels for the five tests in Figure D-8.

Figure D-10 The total residence time against the overtopping discharge.