Colorado State University

Design of the US Wave Overtopping Simulator

Project number: vdm09331
Version: 1.0
Date: 21 December 2009
Executive summary

The US Wave Overtopping Simulator has to simulate overtopping discharges up to 2 cfs/s/ft for wave conditions of respectively 8 ft with a peak period of 14 s and 3 ft with a period of 6 s. This requires a Simulator which is in size about three times larger than the existing Dutch one. This report describes the theory of waves overtopping the crest of a levee, the design of the Simulator, how to operate it and possible ways to measure hydraulics during testing.

We know a lot about wave overtopping over levees, but still there are discrepancies between various formulae. First the existing theory is given about wave overtopping discharge and individual wave overtopping volumes. This leads to the distributions of wave overtopping volumes that have to be simulated by the Simulator. Then flow velocities, flow depths and flow times or durations of overtopping wave volumes at the crest of a levee have been discussed, including re-analysis of existing work and some recent research. The conclusion is that using the equations in an integration, to calculate the wave overtopping volume, the volumes are much too large, indicating that at least flow depth and flow time predictions are too large. For this reason the Simulator will mainly be based on flow velocity and the given peak periods of the waves.

Good experience is available with Simulators up to a size of about 6 m³/m width. The majority of all overtopping wave volumes will be limited to this size. It is for this reason that the US Simulator has been designed with an inner Simulator, comparable to the existing sizes, and an outer Simulator, giving the maximum capacity of 16 m³/m. The outer Simulator will only be used for the very large overtopping wave volumes. The mechanical design has been described and this design has been discussed during a visit to CSU, in order to start fabrication of the Simulator.

The Dutch test set-ups at various locations have been described with their improvements every consecutive year. The operation of the Simulator by a steering file and PLC has been given, first by description of the Dutch Simulator and then by possible modifications and improvements for the US Simulator. Finally, experiences to measure flow depth and front velocity of overtopping waves have been described and possible ways of improvements, which will be performed in the Netherlands and tested in February/March 2010. If successful, these kind of measurements can also be developed at CSU.

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1 Introduction

Analysis of the damage caused by Hurricane Katrina demonstrated that the protected side of levee slopes are vulnerable to erosion and, therefore, potential catastrophic breach during large hurricane events. Raising levees to the 1% design crest elevation reduces this risk, however, there still remains a risk of erosion during wave-only overtopping events in excess of the 1% design loading. Industry opinion is that knowledge of resistance to erosion caused by wave only overtopping is under-developed, and knowledge related to designing erosion armoring systems to protect against wave-only overtopping is altogether lacking. Any controlled testing of grass-covered slopes, or any protection system intended to stabilize soil slopes, must be conducted at full scale because of insurmountable scaling effects. Therefore, full-scale overtopping testing under this very dynamic hydraulic loading is essential to provide appropriate levels of confidence in future designs of protected side levee armoring solutions.

Colorado State University (CSU) has received prime funding from the U.S. Dept. of Defense – Army Corps of Engineers for the purpose of performing work under Contract Number W912P8.09.C.0101, entitled "ARRA: Full Scale Wave Overtopping Testing". CSU has to develop a test site and to have testing results by August of 2010. USACE requested to quantify using measurements and document the performance, at full scale and in controlled conditions, of various erosion protection materials under the unsteady, highly turbulent, hydraulic loading associated with wave-only overtopping on the protected side of a levee.

CSU has to construct, calibrate, and deploy a hydraulic Wave Overtopping Simulator device which simulates an overtopping wave by sequential releases of pre-determined volumes of water that have been stored with sufficient potential energy to replicate both the flow depth and flow velocity of the equivalent overtopping wave volume. Each water release has to produce unsteady and turbulent flow conditions across the test levee crest and protected-side slope. The testing will require observation and measurement of the effects of the simulated wave overtopping on a number of erosion protection materials. The materials will be prepared in special trays (including grass growth) under the supervision of the U.S. Army Engineer Research and Development Center (ERDC) and delivered to the Contractor’s site.

The Wave Overtopping Simulator shall be designed to assure that it can reproduce the hydrodynamic conditions for wave-only overtopping determined for all critical reaches of the New Orleans Hurricane Storm Damage Risk Reduction System (HSDRRS) as established for the 100-year up to the 500-year event. This design requirement includes meeting the maximum wave volume determined for all conditions, and being able to supply water to the simulator at the required rate to conduct simulations at near real time.

In summary this means that two wave (overtopping) conditions have to be simulated:

- An 8 ft wave height with peak period of 14 s, giving wave overtopping discharges starting from 0.1 cfs/s/ft up to 2 dfs/s/ft;
- A 3 ft wave height with peak period of 6 s, giving again wave overtopping discharges starting from 0.1 cfs/s/ft up to 2 dfs/s/ft.

In order to develop the Wave Overtopping Simulator and the test site CSU has contracted Van der Meer Consulting bv in the Netherlands for support in design, manufacturing and calibration of the device, measurements of hydraulics and design of test set-up. The work falls under CSU contract number G-2402-1.

A summary of the support from Van der Meer Consulting bv is as follows:
1. Visit of CSU (and Corps) to the Netherlands. Discussion on the Simulator and all items that have to be designed. Preparation in the Netherlands. Study on size and dimensions of facility at CSU (including limits in theory on very large overtopping), proposed improvement, working of valves with long and short periods, pump design. Requirements for the facility. First sketch for the Simulator and after discussion the first design of the Simulator. Design of the surfboards to measure flow depth. *Size of volume based on 2 cft/s/ft.*

2. Visit Colorado State University. Visit facility. Detailed design session on all aspects of the overtopping facility, based on the first design. Discussion on calibration procedure, requirements and measurements. Visit to Vicksburg/New Orleans for a presentation on the Wave Overtopping Simulator – advantages and disadvantages for New Orleans levees. This visit can also be done during one of the other visits, if that is more convenient.

3. Advice and checking in the Netherlands during construction of the overtopping facility.

4. Visit Colorado State University for calibration period at the manufacturer. Developing modifications, based on the calibration tests. Discussion on the test procedures and test programme.

5. Visit Colorado State University after installation of the Simulator, for pre-testing/calibration as installed and for the first week of testing.

6. Visit further tests, by JM only. Exchange of experience in the Netherlands on results with the Wave Overtopping Simulator and state of the art of modelling of failure mechanisms observed.

7. Reporting on hydraulics, designs, etc. over the total period.

The present report is the result of items 1 and 2 above and partly of items 3 and 7. The work in described in these items have been performed by Dr J.W. van der Meer, coastal engineer, and Mr G. van der Meer, mechanical engineer. The report has been written by Dr J.W. van der Meer.
2 Wave overtopping: discharges and overtopping volumes

2.1 Overtopping discharges

A large number of tests has been performed world wide in wave flumes with perpendicular wave attack without currents, on all kind of structures. The most recent work with a summary of all the test and applicable formulae is the Overtopping Manual (2007). This reference is used as guidance. The equation for overtopping on levee slopes, including safety for deterministic design, see the Manual, is given by:

\[
q = \frac{0.067}{\sqrt{g \cdot H_{m0}^4}} \cdot \tan \alpha \cdot \gamma_b \cdot \xi_{m-1,0} \cdot \exp \left( -4.3 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \right)
\]  

with a maximum of:

\[
q = 0.2 \cdot \exp \left( -2.3 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right)
\]

where:

\[q\] = mean overtopping discharge per meter structure width [m³/s/m]
\[g\] = acceleration due to gravity (= 9.81) [m/s²]
\[H_{m0}\] = estimate of significant wave height from spectral analysis = \(4\sqrt{m_{o}}\) [m]
\[\alpha\] = angle between overall structure slope and horizontal [°]
\[\xi_{m-1,0}\] = breaker parameter = \(\tan \alpha / (\xi_{m-1,0})^{0.5}\) [-]
\[s_{m-1,0}\] = wave steepness with \(L_{m-1,0}\), based on \(T_{m-1,0}\):
\[H_{m0}/L_{m-1,0} = 2nH_{m0}/(gT^2_{m-1,0})\] [-]
\[T_{m-1,0}\] = average wave period defined by \(m_{1}/m_{0}\) [s]
\[m_n\] = \(\int f^n S(f) df = n^{th}\) moment of spectral density [m²/sⁿ]
\(f_{1}\)\(\rightarrow\)\(f_{2}\) = lower integration limit = \(f_{1} = \min(1/3.f_{p}, 0.05\) full scale) upper integration limit = \(f_{2} = 3.f_{p}\)
\[m_{n,x}\] = \(x^{th}\) moment of x spectral density [m²/sⁿ]
\(x\) may be: \(i\) for incident spectrum \(\rightarrow\) \(r\) for reflected spectrum
\[R_c\] = crest freeboard of structure [m]
\[\gamma_b\] = correction factor for a berm [-]
\[\gamma_f\] = correction factor for the permeability and roughness of or on the slope [-]
\[\gamma_\beta\] = correction factor for oblique wave attack [-]
\[\gamma_v\] = correction factor for a vertical wall on the slope [-]

Above equations 2.1 and 2.2 give the mean overtopping discharge for all kind of levee slopes and slope configurations, with various roughness for each slope section, oblique wave attack, etc. The best way to calculate wave overtopping for complicated geometries is to use the programme PC-Overtopping or the Neural Network developed under CLASH. Both programmes are free available from the Overtopping Manual (2007) website www.overtopping-manual.com.

In case the Wave Overtopping Simulator has to simulate a certain situation with given wave boundary conditions and geometry of the levee, first the overtopping discharges for various freeboards should be calculated. Equations 2.1 and 2.2 should be applied directly, or by means of given computer programmes. For smooth and straight slopes, however, and under perpendicular wave attack, the equations become more simple and direct application of the equations is then a good option. The equations for such a
smooth and straight slope are:

\[
\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.067 \frac{\xi_{m-1,0}}{\tan \alpha} \cdot \exp \left( -4.3 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0}} \right) \tag{2.3}
\]

with a maximum of:

\[
\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp \left( -2.3 \frac{R_c}{H_{m0}} \right) \tag{2.4}
\]

2.2 Distribution of overtopping waves

Wave overtopping is a dynamic and irregular process and the mean overtopping discharge, \( q \), does not cover this aspect. But by knowing the storm duration, \( t \), and the number of overtopping waves in that period, \( N_{ow} \), it is easy to describe this irregular and dynamic overtopping, if the overtopping discharge, \( q \), is known. Each overtopping wave gives a certain overtopping volume of water, \( V \) and this can be given as a distribution.

The two-parameter Weibull distribution describes the behaviour quite well. This equation has a shape parameter, \( c \), and a scale parameter, \( a \). The shape parameter gives a lot of information on the type of distribution. Figure 2.1 gives an overview of some well-known distributions. The horizontal axis gives the probability of exceedance and has been plotted according to the Rayleigh distribution. The reason for this is that waves at deep water have a Rayleigh distribution and every parameter related to the deep water wave conditions, like shallow water waves or wave overtopping, directly show the deviation from such a Rayleigh distribution in the graph. A Rayleigh distribution should be a straight line in Figure 2.1 and a deviation from a straight line means a deviation from the Rayleigh distribution.

![Figure 2.1](image)

Figure 2.1. Various distributions on a Rayleigh scale graph. A straight line (c = 2) is a Rayleigh distribution.

When waves approach shallow water and the highest waves break, the wave distribution turns into a Weibull distribution with \( c > 2 \). An example with \( c=3 \) is shown in Figure 2.1 and this indicates that there are more large waves of similar height. The exponential distribution (often found for extreme wave climates) has \( c = 1 \) and shows that extremes become larger compared to most of the data. Such an exponential distribution would give a straight line in a log-linear graph.

The distribution of overtopping volumes for all kind of structures have average values...
even smaller than \( c = 1 \). Such a distribution is even steeper than an exponential distribution. It means that the wave overtopping process can be described by a lot of fairly small or limited overtopping volumes and a few very large volumes. The \( c \)-values are mostly within the range \( 0.6 < c < 0.9 \). For comparison curves with \( c = 0.65 \) and 0.85 are given in Figure 2.1. The curves are very similar, except that the extremes differ a little. It is for this reason that for smooth slopes an average \( c \)-value of 0.75 has been chosen and not different values for various subsets of data. The same average value has been used for rubble mound structures, which makes smooth and rubble mound structures easy comparable. The exceedance probability, \( P_V \), of an overtopping volume per wave is then similar to:

\[
P_V = P(V \leq V) = 1 - \exp\left(-\left(\frac{V}{a}\right)^{0.75}\right)
\]

(2.5)

with:

\[
a = 0.84 \cdot T_m \cdot \frac{q}{P_{ov}} = 0.84 \cdot T_m \cdot q \cdot N_w / N_{ovw} = 0.84 \cdot q \cdot t / N_{ovw}
\]

(2.6)

Equation 2.6 shows that the scale parameter \( a \), depends on the overtopping discharge, \( q \), but also on the mean period, \( T_m \), and probability of overtopping, \( N_{ov}/N_w \), or which is similar, on the storm duration, \( t \), and the actual number of overtopping waves \( N_w \).

Equations for calculating the overtopping volume per wave for a given probability of exceedance, is given by Equation 2.5. The maximum overtopping during a certain event is fairly uncertain, as most maxima, but depends on the duration of the event. In a 6 hours period one may expect a larger maximum than only during 15 minutes. For the maximum overtopping volume in a storm the following formula can be used, by filling in the number of overtopping waves \( N_{ov} \).

\[
V_{\text{max}} = a \cdot \left[\ln(N_{ov})\right]^{1/3}
\]

(2.7)

The probability of overtopping per wave can be calculated by assuming a Rayleigh-distribution of the wave run-up heights and taking \( R_{u2\%} \) as a basis:

\[
P_{ov} = \exp\left(-\left(\sqrt{-\ln 0.02} \cdot \frac{R_c}{R_{u2\%}}\right)^2\right)
\]

(2.8)

The \( 2\%-\)run-up level, \( R_{u2\%} \), is given in the Overtopping Manual (2007) as:

\[
\frac{R_{u2\%}}{H_{m0}} = 1.75 \cdot \gamma_h \cdot \gamma_f \cdot \gamma_{\beta \cdot \xi_{m-1,0}}
\]

(2.9)

with a maximum of:

\[
\frac{R_{u2\%}}{H_{m0}} = 1.00 \cdot \gamma_f \cdot \gamma_{\beta} \left(4.3 - \frac{1.6}{\sqrt{\xi_{m-1,0}}}\right)
\]

(2.10)
The probability of overtopping per wave $P_{ov}$ is related to the number of incoming ($N_w$) and overtopping waves ($N_{ov}$) by:

$$P_{ov} = \frac{N_{ov}}{N_w} \quad (2.11)$$

In order to design the Wave Overtopping Simulator it is required to calculate the distributions of overtopping volumes for required situations. Based on these distributions the maximum size/capacity of the device can be determined, looking at the maximum overtopping volumes to be simulated. Moreover, the distributions should be simulated as close as possible by the Wave Overtopping Simulator, including the number of overtopping waves in the given test duration.

In summary, to produce realistic overtopping events by the Wave Overtopping Simulator, the distribution of overtopping volumes should be known, together with the number of overtopping waves.

### 2.3 Distribution with fixed number of overtopping waves

Equation 2.6 gives the $a$-coefficient to be used to construct the distribution of overtopping waves. The equation contains the factor 0.84, which actually is only valid for an infinite number of overtopping waves. For a finite number of overtopping waves the factor should be adjusted (increased) by iteration in such a way that the summation of all overtopping volumes is equal to the total discharge over time:

$$\sum_{i=1}^{N_w} V_i = q t \quad (2.12)$$

If this adjustment is not done the summation of volumes will be less than the total overtopping volume calculated from the mean discharge over time. Figure 2.2 shows the adjustment of this factor 0.84 for a large variety of number of overtopping waves. Only if the total number of overtopping waves is in the order of thousand or more the value of 0.84 is reached. For less than 50 overtopping waves the factor is closer to 0.9.

![Figure 2.2](image-url)  
Figure 2.2. Variation of factor $a$ in equation 2.6, depending on total number of overtopping waves.
2.4 Required overtopping for New Orleans testing

Equations in Sections 2.1-2.3 can be used to calculate the wave overtopping distributions to be simulated during testing for New Orleans’ circumstances. Two wave conditions are required, \( H_{m0} = 8 \) ft with \( T_p = 14 \) s and \( H_{m0} = 3 \) ft with \( T_p = 6 \) s. For geometry of the levee a 1V : 4H slope is assumed. Each test duration is 1 hour.

Overtopping discharges to be simulated are from 0.1 cfs/s/ft with increments of 0.1 cfs/s/ft up to 2.0 cfs/s/ft. PC-Overtopping has been used to calculate the required crest freeboard, \( R_c \), for each overtopping discharge and then the 2%-run-up level has been calculated (in case no overtopping on a 1:4 slope), the percentage of overtopping waves and the maximum volume of overtopping wave.

Tables 2.1 and 2.2 give the calculations for each of the wave conditions. PC-Overtopping works with metric units, but a translation is given in the tables.

### Table 2.1. Calculations for \( H_{m0} = 8 \) ft with \( T_p = 14 \) s

| \( H_{m0} \) (ft) | 8 | \( H_{m0} \) (m) | 2.4384 | \( T_p \) (s) | 14 | \( T_m/T_p \) | 1.2 | \( cota \) | 4 | \( R_u \) \( 2\% \) (m) | 8.059 | \( R_u \) (ft) | 26.37 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| \( Ru \) \( 2\% \) (ft) | 38 | 38 | 90 | 90 | 118 | 118 | 136 | 136 | 151 | 151 | 163 | 163 | 174 | 174 | 183 | 183 | 191 | 191 | 198 | 198 | 252 |
| \( Ru \) \( 2\% \) PCO (m) | 1.61 | 1.61 | 4.31 | 4.31 | 6.32 | 6.32 | 8.33 | 8.33 | 10.34 | 10.34 | 12.35 | 12.35 | 14.36 | 14.36 | 16.37 | 16.37 | 18.38 | 18.38 | 24.09 | 24.09 | 30.71 |
| Percentage of overtopping waves PCO (%) | 28.34 | 28.34 | 41.20 | 41.20 | 50.80 | 50.80 | 57.30 | 57.30 | 62.68 | 62.68 | 66.84 | 66.84 | 73.82 | 73.82 | 78.25 | 78.25 | 82.44 | 82.44 | 85.38 | 85.38 | 87.67 |
| Number of overtopping waves | 38 | 38 | 62 | 62 | 89 | 89 | 107 | 107 | 118 | 118 | 136 | 136 | 151 | 151 | 163 | 163 | 174 | 174 | 183 | 183 | 252 |
| Maximum overtopping volume (cft/ft) | 45 | 45 | 64 | 64 | 81 | 81 | 96 | 96 | 110 | 110 | 123 | 123 | 148 | 148 | 172 | 172 | 195 | 195 | 218 | 218 | 239 |
| Maximum overtopping volume PCO (l/m) | 4135 | 4135 | 5980 | 5980 | 7521 | 7521 | 8935 | 8935 | 10225 | 10225 | 11469 | 11469 | 13761 | 13761 | 15998 | 15998 | 18088 | 18088 | 20235 | 20235 | 22233 |
| Coefficient "0.84" in a | 0.915 | 0.915 | 0.872 | 0.872 | 0.868 | 0.868 | 0.869 | 0.869 | 0.865 | 0.865 | 0.866 | 0.866 | 0.864 | 0.864 | 0.863 | 0.863 | 0.861 | 0.861 | 0.860 | 0.860 | 0.860 |

### Table 2.2. Calculations for \( H_{m0} = 3 \) ft with \( T_p = 6 \) s

| \( H_{m0} \) (ft) | 3 | \( H_{m0} \) (m) | 0.9144 | \( T_p \) (s) | 6 | \( T_m/T_p \) | 1.2 | \( cota \) | 4 | \( R_u \) \( 2\% \) (m) | 2.836 | \( R_u \) (ft) | 9.30 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| \( Ru \) \( 2\% \) (ft) | 38 | 38 | 62 | 62 | 89 | 89 | 107 | 107 | 118 | 118 | 136 | 136 | 151 | 151 | 163 | 163 | 174 | 174 | 183 | 183 | 252 |
| \( Ru \) \( 2\% \) PCO (m) | 1.61 | 1.61 | 4.31 | 4.31 | 6.32 | 6.32 | 8.33 | 8.33 | 10.34 | 10.34 | 12.35 | 12.35 | 14.36 | 14.36 | 16.37 | 16.37 | 18.38 | 18.38 | 24.09 | 24.09 | 30.71 |
| Percentage of overtopping waves PCO (%) | 28.34 | 28.34 | 41.20 | 41.20 | 50.80 | 50.80 | 57.30 | 57.30 | 62.68 | 62.68 | 66.84 | 66.84 | 73.82 | 73.82 | 78.25 | 78.25 | 82.44 | 82.44 | 85.38 | 85.38 | 87.67 |
| Number of overtopping waves | 38 | 38 | 62 | 62 | 89 | 89 | 107 | 107 | 118 | 118 | 136 | 136 | 151 | 151 | 163 | 163 | 174 | 174 | 183 | 183 | 252 |
| Maximum overtopping volume (cft/ft) | 45 | 45 | 64 | 64 | 81 | 81 | 96 | 96 | 110 | 110 | 123 | 123 | 148 | 148 | 172 | 172 | 195 | 195 | 218 | 218 | 239 |
| Maximum overtopping volume PCO (l/m) | 1283 | 1283 | 1902 | 1902 | 2483 | 2483 | 2982 | 2982 | 3486 | 3486 | 3953 | 3953 | 4945 | 4945 | 5775 | 5775 | 6777 | 6777 | 7666 | 7666 | 8512 |
| Coefficient "0.84" in a | 0.86 | 0.86 | 0.854 | 0.854 | 0.852 | 0.852 | 0.85 | 0.85 | 0.849 | 0.849 | 0.848 | 0.848 | 0.848 | 0.848 | 0.847 | 0.847 | 0.846 | 0.846 | 0.846 |

2.5 Distributions of volumes

Given the calculations in Tables 2.1 and 2.2 and equations 2.5-2.12, the distributions of overtopping volumes for each test condition can be calculated. The graphs are shown in Figures 2.3 and 2.4 for each wave condition separately. The graph is not give on Rayleigh-scale, but the horizontal axis is simply the number of the overtopping wave in ascending order.

But again it is very clear that wave overtopping is given by a large number of relatively small overtopping volumes and a few large ones.
Figure 2.3. Required distribution of overtopping volumes for $H_{m0} = 8$ ft with $T_p = 14$ s.

Figure 2.4. Required distribution of overtopping volumes for $H_{m0} = 3$ ft with $T_p = 6$ s.
2.6 Maximum size of the Wave Overtopping Simulator

From Figures 2.3 and 2.4 it is clear that the wave conditions with $H_{m0} = 8$ ft and the long peak period of 14 s give the largest overtopping volumes. For this condition maximum volumes even reach 25,000 l/m (25 m$^3$/m) or 270 cft/ft. The maximum volumes for wave conditions with $H_{m0} = 3$ ft and the shorter peak period of 6 s are limited to 10-14 m$^3$/m or 100-150 cft/ft.

It is logical that the condition with the long period gives larger volumes, as there are less waves and overtopping waves in 1 hour, whilst the discharges are similar to the condition with the shorter wave period.

The Dutch Wave Overtopping Simulator has a size of 5.5 m$^3$/m or 60 cft/ft. This means that, if maximum overtopping volumes of 25 m$^3$/m should be realized, the US Simulator should be designed 4 or 5 times larger than the Dutch one. This is practically impossible.

But there is a doubt on the maximum overtopping volumes if they become very large. The physical limit is the water content in an actual wave with the water level close to the crest of a levee. Equations on distributions of overtopping waves do not consider a physical maximum.

Hughes (2009) has performed overtopping tests where the water level was even higher than the crest of the levee. These were tests on continuous overflow combined with wave overtopping. Hughes was able to calculate wave by wave overtopping from integration of the flow depth and flow velocity record and he presented the shape factor of the overtopping distribution for each test. This shape factor has been described in Section 2.2, Equation 2.5.

Figure 2.5 gives Hughes’ (2007) graph, where $b_v$ is equal to $c$ in Equation 2.5. The value of $b_v = c = 0.75$ for positive crest freeboards has been added to this graph. All of Hughes’ shape factors are (much) larger than 0.75, which means that the distributions are much gentler, see also Figure 2.1 Many values are even larger than 2, which means gentler than a Rayleigh distribution. The condition with the water level closest to the crest level ($R_c = -0.29$ m) gives values between 1 and 2 and this is closest to the situation without continuous overflow.

![Figure 2.5](image.png)

Fig. 12. Measured versus predicted Weibull parameter, $b_v$, for wave volume distribution.

Figure 2.5. Shape coefficient for wave volume distribution (from Hughes, 2007).
From Figure 2.5 it can be concluded that if the water level comes close to the crest level, or even exceeds it, the shape factor will increase from $c = 0.75$. The consequence is that the maximum overtopping volume will also decrease. This conclusion can also be validated by Hughes’ (2009) test results.

Figures 2.6-2.8 give individual overtopping wave volumes, first versus the measured individual wave period and then as a rank-ordered distribution. Test conditions have been chosen with the lowest water level (0.29 m above the crest) and with wave heights of respectively 0.79 m; 1.68 m; and 2.48 m. The smallest and largest wave heights are comparable with 3 ft and 8 ft. The wave period in all conditions was the long period of 14 s. Mean discharges were respectively 350; 550; and 700 l/s per m (3.8; 5.9 and 7.5 cfs/s/ft), far beyond the required conditions for testing at CSU.

Red circles give the maximum values. In all these three tests, with conditions far beyond the conditions required for testing at CSU, maximum wave overtopping volumes exceed the volume of 14 m$^3$/m only 8 times, where the absolute maximum is 20 m$^3$/m, followed by 3 maxima of 18 m$^3$/m. These maxima are lower than the calculated maximum of 21 m$^3$/m for the condition with $H_{m0} = 8$ ft and a crest freeboard of 10 ft, with a mean overtopping discharge of 1.4 cfs/s/ft.

**Figure 2.6.** Wave overtopping volumes for combined overflow and overtopping (Hughes 2009). Wave height $H_{m0} = 0.79$ m.

**Figure 2.7.** Wave overtopping volumes for combined overflow and overtopping (Hughes 2009). Wave height $H_{m0} = 1.68$ m.
From above analysis it is clear that if the water level approaches the crest and wave overtopping becomes very large, it is likely that the assumed distribution of overtopping wave volumes with a shape factor of $c=0.75$ is not valid any longer. But it is unknown when this deviation will start.

As the Wave Overtopping Simulator should simulate real wave overtopping conditions, these conditions (the actual distribution of overtopping waves) should be known. It would possibly limit the maximum overtopping volumes to volumes that can indeed by simulated by a practical size of Wave Overtopping Simulator.

For that reason it is strongly recommended to perform tests on wave overtopping, comparable to Hughes (2009), but now with a lower water level, still fairly close to the crest. These tests would validate the decision to limit the size of the US Wave Overtopping Simulator to about 16 m$^3$/m, as described in the next Section.

### 2.7 Decision on maximum size of Simulator

Figure 2.3 gives the distributions of overtopping volumes for the 8 ft wave condition, together with maxima (the horizontal lines) of 11; 14 and 16.5 m$^3$/m size. These sizes are 2; 2.5 and 3 times the size of the Dutch Simulator. A maximum larger than 16 m$^3$/m is not foreseen, as this will not lead to a practical and reliable device.

The graph shows which discharges still can be simulated according to theory, given a certain maximum. For a maximum of 11 m$^3$/m the maximum discharge is about 0.5 cfs/s/ft. For larger discharges the large overtopping volumes have to be cut-off. When we look at the discharge of 2.0 cfs/s/ft then 12 volumes larger than 11 m$^3$/m have to be cut-off.

For a maximum of 14 m$^3$/m the maximum discharge is 0.8 cfs/s/ft and 8 volumes for the discharge of 2.0 m$^3$/m will be cut-off. For the maximum of 16.5 m$^3$/m the values or a maximum discharge of 1.0 cfs/s/ft with only 4 overtopping waves cut-off. As described in Section 2.6 it is unlikely that for overtopping volumes larger than about 15 m$^3$/m the theory is correct. In reality these maximum volumes will be smaller and probably limited to about this maximum around 15 m$^3$/m.

This means that the size of the Simulator should be chosen as large as practically possible, but larger than 3 times the Dutch one (around 16 m$^3$/m) is not necessary.
2.8 Distributions to be simulated

Given a maximum size of the Wave Overtopping Simulator of 16 m³/m, the overtopping distribution which can be simulated, can be calculated. Figures 2.9 and 2.10 give the distributions that can be simulated. There are two reasons why there will be deviations from the theoretical distributions as given in Figures 2.3 and 2.4.

The first one is the maximum size of the Simulator. Overtopping volumes larger than 16 m³/m can not be simulated. All overtopping waves larger than this volume will be simulated as 16 m³/m. In order to keep the same total overtopping water, the volume of water in excess of 16 m³/m in the large waves, will be distributed over the overtopping waves which are just smaller than 16 m³/m. This means that we get more waves of 16 m³/m, with in total the right volume of overtopping water. For example, for the 8 ft wave height and a discharge of 2.0 cfs/ft there are 4 waves larger than 16 m³/m and these will be limited to this maximum (the horizontal maximum in Figure 2.9). In order to keep the same total volume of overtopping water another 11 overtopping waves need a maximum of 16 m³/m. The transition from the theoretical distribution to this maximum is given by the vertical line in Figure 2.9.

The second deviation from theory is that very small overtopping waves can not be simulated. The minimum time to open and close the valve of the Simulator will be around a wave period. The minimum overtopping volume that can be simulated is then about the discharge given over one period. A larger discharge will consequently give a larger minimum. For example, a discharge of 1.0 cfs/ft gives in Figure 2.9 a minimum of 1100 l/m (93 l/s per m over about 12 s). These minima are shown in Figures 2.9 and 2.10 as horizontal lines at the lower end.

The total amount of water in all overtopping waves smaller than this minimum is used to calculate the number of overtopping waves with this minimum, but giving the same total amount of water. In fact less overtopping waves will be simulated, but all with at least the minimum volume.

![Figure 2.9. Simulation of distribution of overtopping volumes for Hm₀ = 8 ft with Tp = 14 s.](image-url)

- 0.1 ft³/s/ft
- 0.2 ft³/s/ft
- 0.3 ft³/s/ft
- 0.4 ft³/s/ft
- 0.5 ft³/s/ft
- 0.6 ft³/s/ft
- 0.8 ft³/s/ft
- 1.0 ft³/s/ft
- 1.2 ft³/s/ft
- 1.4 ft³/s/ft
- 1.6 ft³/s/ft
- 1.8 ft³/s/ft
- 2.0 ft³/s/ft

10 l/s/m is about 0.1 cft/s/ft
1000 l/m is about 10 cft/ft
0.9 m is about 3 ft

Hs=8 ft; Tp=14 s
Figure 2.10. Simulation of distribution of overtopping volumes for $H_{m0} = 3$ ft with $T_p = 6$ s.

Figure 2.10 gives the distributions for the smaller wave condition of 3 ft. All maximum waves can be simulated and the minimum values are also smaller than for the 8 ft condition, as the wave period is smaller.

Figures 2.9 and 2.10 give the distributions as they have to be simulated by the Wave Overtopping Simulator, given a maximum size of 16 m$^3$/m. Of course the overtopping volumes will be simulated in random order, as is the case in reality.
3 Flow depth, flow velocity and flow time

3.1 Existing theory

The Wave Overtopping Simulator has to produce the right overtopping volumes according to the distributions described in Chapter 2. But each overtopping volume has to be simulated with a certain maximum flow depth, flow velocity and with a certain flow duration or flow time. This Chapter deals with these flow depths and velocities.

Equations for flow depth and velocity have been based on physical model investigations like by Schüttrumpf (2002) and Van Gent (2002), published as a joined paper in Schüttrumpf and Van Gent (2003). The problem at that time was that the flow depth predicted by Schüttrumpf was twice the one by Van Gent. For this reason the Dutch Simulator in 2006 was designed on flow velocity and not on flow depth. The Overtopping Manual (2007) also gives the equations.

Bosman (2007) investigated this discrepancy and discovered that the difference in predicted flow depth could possibly be explained by the different seaward slopes (1:4 and 1:6) used by the different authors. He used a sinα to combine the equations. Bosman also studied flow depth and flow velocity on the crest of a levee, and finally he looked at the flow time.

The basic equations for (maximum) flow depth and velocity are:

\[
\begin{align*}
\h_{2\%}(x_c=0) &= c_{A,h} \left( R_{u2\%} - R_c \right) \\
u_{2\%}(x_c=0) &= c_{A,u} \left( g(R_{u2\%} - R_c) \right)^0.5
\end{align*}
\]

where:

- \( \h_{2\%} \) = flow depth exceeded by 2% of the incident waves
- \( u_{2\%} \) = flow velocity exceeded by 2% of the incident waves
- \( x_c \) = location on the crest (\( x_c=0 \) is the transition from seaward slope to the crest)
- \( c_{A,h} \) = coefficient for the flow depth
- \( c_{A,u} \) = coefficient for the flow velocity
- \( R_{u2\%} \) = 2% wave run-up level
- \( R_c \) = crest freeboard (vertical distance between crest and still water level)

The following coefficients where found:

<table>
<thead>
<tr>
<th>Author</th>
<th>( c_{A,h} )</th>
<th>( c_{A,u} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schüttrumpf (2002)</td>
<td>0.33</td>
<td>1.37</td>
</tr>
<tr>
<td>Van Gent (2002)</td>
<td>0.15</td>
<td>1.33</td>
</tr>
<tr>
<td>Bosman (2007)</td>
<td>0.010/sin^2\alpha</td>
<td>0.30/sin\alpha</td>
</tr>
<tr>
<td>( Bosman \ (2007) \ 1:4 )</td>
<td>0.17</td>
<td>1.24</td>
</tr>
<tr>
<td>( Bosman \ (2007) \ 1:6 )</td>
<td>0.37</td>
<td>1.82</td>
</tr>
</tbody>
</table>

The Dutch Simulator was designed with \( c_{A,u} = 1.35 \).

Velocity and flow depth change along the crest. The flow depth changes quite suddenly as the vertical velocity component on the seaward slope disappears. On the crest both flow depth and velocity may reduce due to friction, etc., but then the flow time increases (as the total volume does not change). Bosman (2007) gives the following equations for flow depth and velocity along the crest:

\[
\begin{align*}
\h_{2\%}(x)/\h_{2\%}(x_c=0) &= 0.81 \exp(-6 \frac{x_c}{(\gamma_c L_{\text{m}-1,0})}) \\
u_{2\%}(x)/u_{2\%}(x_c=0) &= \exp(-0.042 \frac{x_c}{(\gamma_c \h_{2\%}(x_c))})
\end{align*}
\]
The roughness factor $\gamma_c = 1$ for a smooth slope and $L_{m-1,0} = \text{the wave steepness calculated with the spectral wave period } T_{m-1,0}$. Equation 3.3 shows that the flow depth reduces immediately at the beginning of the crest to 0.81 of its original depth and then shows an exponential decay. Also the flow velocity shows this exponential decay.

Use in recent years of the equations by Bosman (2007) for flow velocities on the crest showed that flow velocities along the crest reduce considerably. As the Simulator has to simulate the correct velocities somewhere on the crest (often near the transition to the landward slope), it is important to know whether these equations are correct or not.

Equations 3.1-3.4 give the 2%-value only. Each overtopping wave volume has a specific probability of occurrence in a certain time period. Coefficients, therefore, are needed for each probability of occurrence, or exceedance, in order to calculate the correct flow depth and velocity for each overtopping wave volume. The easiest way to do this is by assuming a Rayleigh distribution for the wave run-up.

Figure 3.1 shows a graph with overtopping wave volume versus the flow velocity of this overtopping wave. The graph shows curves according to equation 3.1 with $c_{A,u} = 1.35$ (the mean of Schuttrumpf (2002) and Van Gent (2002)) and the small decay in velocity at 3.5 m from the beginning of the crest, according to Van Gent (2002). These curves were used to calibrate the Dutch Simulator and the calibration points agree well with these curves. The lowest curve is according to Bosman (2007), 1.5 m behind the beginning of the crest. The velocity is much smaller, a fact that only recently has been concluded.

Figure 3.1. Flow velocities on the crest according to various equations.

A similar graph is given in Figure 3.2, but now for the flow depth on the crest. Both Schüttrumpf (2002) and Van Gent (2002) show a considerable decrease in flow depth as soon the wave arrives at the crest. Bosman (2007) here is similar to Van Gent (2002) and there is no discrepancy as for the flow velocity. Calibration points during design of the Dutch Simulator agree fairly well with these curves.

Given the discrepancy with the flow velocity it is required to perform a re-analyses of the data. Moreover, the recent Hydralab III Flowdike project makes it possible to introduce more data for validation. This has been described in the next Section.
3.2 Re-analysis of Bosman with Flowdike data

The Flowdike project has been executed under the European Union programme Hydralab III. The objective was to investigate the influence of currents along a levee on wave run-up and wave overtopping. Leading partner was the University of Aachen in Germany. The tests were performed in the wave-current basin of the Danish Hydraulic Institute, DHI, at Hørsholm, see Figure 3.3.

![Figure 3.3. Overview of the Flowdike model with two crest heights and the run-up board.](image)

The experimental investigations were performed for a simple 1:3 slope, typical for river levees. The levee was divided into two separate parts to perform wave run-up and wave overtopping tests at the same time. The overtopping tests were performed on levee sections with crest freeboards of 0.1 m and 0.2 m. The test programme covered...
model tests on wave run-up and wave overtopping, all with and without currents and with and without wind for different wave conditions. Wave conditions cover long crested waves and perpendicular, respectively oblique wave attack.

The crest width was 0.30 m. Flow velocities and flow depths were measure at the transition from seaward slope to the crest and 0.30 m behind this point, at the end of the crest. Figures 3.4 shows a picture of the flow depth meters with micro-propellers on the crest.

Figure 3.4. Measurement of flow depth (wave gauge) and flow velocity (micro-propeller).

Figure 3.5 shows measured flow depths at the seaward side of the crest compared with those at the landward side. The depth seems to decrease by roughly 30%. Figure 3.6 shows a similar graph, but now for the flow velocity. The reduction along the crest is now much smaller.
Figure 3.6. Comparison flow velocities at the crest for the Flowdike project

A re-analysis will be performed on the data of Schüttrumpf (2002) and Van Gent (2002), as used by Bosman (2007), but the data is increased by the Flowdike data and an extra set of data of Van Gent (2002), with a wide crest. This latter data set was not considered by Bosman. The data by Van Gent is given in the graphs by Conf. A and B for the 0.2 m wide crest and Conf. C and D for the 1.1 m wide crest. Conf. D’ is also a wide crest, but now covered with rock.

First a graph is given in Figure 3.7 with only the data of Schüttrumpf (2002) and Van Gent (2002) and with the relationships of Bosman (2007). That graph includes the effect of a wide crest as measured by Van Gent (2002). Then the same graph is given in Figure 3.8, but now with the Flowdike data. These data includes of course the influence of current and oblique wave attack and is therefore only used as a “cloud of points” with a certain trend and large scatter.

Figure 3.7 shows the flow depth along the crest. The points on the left side of the graphs were considered by Bosman (2007), who gives the exponential decay with equation 3.3. With the data of the wide crest of Van Gent (2002), however, it is clear that there is hardly any decay of flow depth over the crest. Van Gent (2002) gives a direct reduction to two-third of the value at the seaward crest, but no further reduction.

Figure 3.8 is similar to 3.7, but now the Flowdike data have been added. The general trend is clear: after an initial drop, due to the transition from seaward slope to a horizontal crest, there is no further reduction in flow depth along the crest. Actually, this can be expected for a smooth crest, where energy dissipation hardly occurs.

Figures 3.9 is comparable with Figure 3.8, but now for the flow velocity instead of flow depth. Although a few data points of Van Gent (2002) suggest an increase of velocity (part of the data set of Conf. A), the general trend is that velocity reduces along the crest. The Flowdike data points fall nicely between the other points, giving a relationship for decay of flow velocity over the crest:

\[ \frac{u_{2\%}(x_c)}{u_{2\%}(x_c=0)} = \exp(-1.4 \frac{x_c}{L_{m-1,0}}) \]  \hspace{1cm} (3.5)
Figure 3.7. Flow depth along the crest of a levee.

Figure 3.8. Flow depth along the crest of a levee, with Flowdike data.

Figure 3.9. Flow velocity along the crest, including all data.
Figure 3.10 shows the flow depth at the seaward crest for Schüttrumpf (2002) as well as Van Gent (2002), both with their fit of coefficient $c_{A,h}$ of 0.33 and 0.15, respectively, see Equation 3.1. There is a clear difference, which was explained by Bosman (2007) by taking the seaward slope into account, receiving a coefficient $c_{A,h} = 0.010/\sin^2 \alpha$. Figure 3.11 gives the relationship of Bosman (2007), although the sinus has been replaced by the almost similar but more often used cotangents. Now the two data sets indeed fall together.

Figure 3.10. Flow depth at the seaward crest by Schüttrumpf (2007) and Van Gent (2007).

Figure 3.11. Flow depth at the seaward crest by Schüttrumpf (2007) and Van Gent (2007), including the seaward slope (Bosman 2007)).

Figure 3.12 is similar to Figure 3.10, but now the Flowdike data on a slope 1:3 have been added. The data points of Flowdike fall in between the slopes 1:6 and 1:4, which is not logical. The expectation was that the data had to fall below the 1:4 slope. No explanation is available and therefore the only conclusion can be that there is no valid ground to include the seaward slope in the equation.
In Figures 3.7 and 3.8 it was shown that flow depth decreases quickly behind the seaward crest and then remains more or less constant. For flow depth on a smooth crest there is no decay along the crest. For this reason Figure 3.12 was repeated in Figure 3.13, but now for the flow depth along or at the landward side of the crest. Now the three data sets are more in line with each other, although there is still a considerable scatter. A fair fit through all the points gives:

\[ h_{2\%}(x=0) = 0.13 \left( R_{u2\%} - R_c \right) \]  

(3.6)

The coefficient 0.20 fits the Flowdike data, is a little too large for Van Gent’s (2002) data and a little too small for Schüttrumpf’s (2002) data. But again, there is no explanation to bring the data in line and the scatter has to be accepted.

Figure 3.14 shows the relative flow velocity on the seaward crest versus the relative freeboard. Data of all three sets have been given. Now the data of Schüttrumpf (2002)
are on the left side of the graph (1:6), the data of Van Gent (2002) in the middle (1:4) and the data of Flowdike (1:3) on the right side. It seems now that there is a clear influence of the seaward slope, as given by Bosman (2007) in Equation 3.2. For this reason \( \cot \alpha \) has been introduced (instead of \( \sin \alpha \)) in Figure 3.15. Now the data fall together.

![Figure 3.14. Flow velocity at the seaward crest with all data sets.](image)

![Figure 3.15. Flow velocity at the seaward crest, including the seaward slope \( \cot \alpha \).](image)

A fair fit through the data points in Figure 3.15 gives:

\[
    u_{2\%}(x_c=0) = 0.35 \cot \alpha \ (g(R_{u2\%} - R_c))^{0.5}
\]  

(3.7)

### 3.3 Final equations for flow depth and flow velocity

The analysis in Section 3.3 leads to the following summary of equations for flow velocity and flow depth on and along the crest of a levee, with a smooth slope.

The flow depth reduces directly behind the seaward crest and remains then almost
constant along the crest. This flow depth along the crest is given in Figure 3.13 and Equation 3.6:

\[ h_{2\%}(x_c=0) = 0.13 \left(R_{u_{2\%}} - R_c\right) \quad (3.6) \]

The flow depth at the seaward crest is 50% larger than given in Equation 3.6.

The flow velocity on the seaward crest is given in Figure 3.15 and can be described by Equation 3.7:

\[ u_{2\%}(x_c=0) = 0.35 \cot \alpha \left(g(R_{u_{2\%}} - R_c)\right)^{0.5} \quad (3.7) \]

The decay of flow velocity along the crest is given in Figure 3.9 and Equation 3.5:

\[ \frac{u_{2\%}(x_c)}{u_{2\%}(x_c=0)} = \exp(-1.4 \frac{x_c}{L_{m-1,0}}) \quad (3.5) \]

### 3.4 Velocity and depth versus overtopping volume

Chapter 2 gives the equations to derive distributions of overtopping volumes for each possible wave condition and levee geometry. The specific requirements for new Orleans have been elaborated in that Chapter, resulting in the distributions that have to be simulated by the Wave Overtopping Simulator, see Figures 2.9 and 2.10. Sections 3.1-3.3 give the derivation of equations to calculate flow depth and flow velocity along the crest, but for the 2%-value only.

By assuming a Rayleigh distribution for the flow velocity (Equation 3.7) and flow depth (Equation 3.6) the velocity and flow depth can be calculated for each overtopping wave volume with a certain probability of exceedance. Such calculations lead to graphs of flow velocity or flow depth versus overtopping wave volume. Figure 3.16 gives such a graph for the 8 ft wave condition and Figure 3.17 for the 3 ft condition.
Design of the US Wave Overtopping Simulator; version 1.0

Figure 3.17. Flow velocity at the seaward crest versus overtopping wave volume for the condition of $H_m = 3$ ft and $T_p = 6$ s.

Figures 3.16 and 3.17 show that each overtopping discharge leads to a little different curve for the flow velocity. This can not really be explained as one single wave in an overtopping event does not know to what mean overtopping discharge it belongs. It can be expected that similar overtopping volumes result in similar crest velocities, but that is not the case. As long as this discrepancy has not been solved, we have to cope with the scatter introduced by the curves. Figure 3.18 shows the two outer boundaries of both Figures 3.16 and 3.17 and also proposes a mean curve in between.

Figure 3.18. Scatter of flow velocity with a proposal for a mean curve.
Figure 3.19. Proposed flow velocity versus measurements with the Dutch Simulator.

Figure 3.19 is similar to Figure 3.18, but now measured data points have been added, which were established by the Dutch Simulator. This Simulator is limited to a size of 5.5 m$^3$/m, about one-third of the proposed US Simulator. The red points give the data points during design and calibration of the Simulator, the blue triangles give measured points on a landward slope of 1:3. The data fall more or less around the proposed mean curve.

Figure 3.20. Flow depth along the crest versus overtopping wave volume for the condition of $H_m = 8$ ft and $T_p = 14$ s.
Design of the US Wave Overtopping Simulator; version 1.0

With Equation 3.6 it is possible to calculate the flow depth on the crest and to give the relationship with the volume of the overtopping wave. These kind of graphs have been shown in Figure 3.20 and 3.21 for both wave conditions at New Orleans. Figure 3.22 gives the lower and upper boundaries and again it is clear that there is quite some scatter. Figure 3.22 also includes measurements on the landward slope of a levee and these measurements coincide with the lower boundary of the 3 ft wave condition. The overtopping was simulated for a wave condition with $H_{m0} = 2$ m and a peak period of $T_p = 5.7$ s.

Figure 3.21. Flow depth along the crest versus overtopping wave volume for the condition of $H_{m0} = 3$ ft and $T_p = 6$ s.

Figure 3.22. Scatter of flow depth along the crest and comparison with measurements.
The scatter in the graphs is large, certainly for the flow depth. It was also for this reason that the Dutch Simulator was designed on flow velocity only and for a peak period around 6 s, assuming that wave overtopping volumes of around 5 m³/m would have a flow time around this peak period.

There is another contradiction in the prediction equations of flow velocity and flow depth. The figures show that flow velocity and flow depth will be larger for the 8 ft wave condition than for the 3 ft condition, but for the same overtopping volume, see Figures 3.18 and 3.22 for a direct comparison. But the wave period is much longer for the 8 ft condition (14 s versus 6 s). Having the same volume for these two conditions and a more than two times longer period for the 8 ft wave height must result in lower flow velocity and smaller flow depth than for the 3 ft wave height. The integral of flow velocity, multiplied by flow depth, over the flow time gives the volume of the overtopping wave.

It must be concluded that present knowledge and prediction formulae do not yet give consistent answers, certainly not when the wave period changes a lot. It is for this reason that also the prediction formulae for flow time will be considered.

### 3.5 Flow times

Bosman (2007) gives the following prediction of flow times, $T_{ovt\,2\%}$, based on 5 tests of Van Gent (2002).

$$T_{ovt\,2\%}/T_{m-1,0} = 1.15 \left( (R_{u2\%} - R_c)/H_{m0} \right)^{0.5} \tag{3.6}$$

The flow time is related to the spectral period $T_{m-1,0}$. As soon as $R_{u2\%} - R_c$ becomes equal to the wave height, the flow time will be larger than the peak period. With Equation 3.6 it is possible to calculate the flow time for each overtopping volume in a similar way as for the flow velocity and flow depth. Graphs are shown in Figures 3.23 and 3.24 for the two wave conditions.

![Figure 3.23](image-url)  
**Figure 3.23.** Flow time at the crest versus overtopping wave volume for the condition of $H_{m0} = 8$ ft and $T_p = 14$ s.
3.6 Velocities, flow depths and flow times to be simulated

Prediction formulae do not give consistent answers, but the Simulator has to be designed on what we know. In summary Figure 3.25 shows the relationship between distributions of overtopping volumes and the relationship with $R_u \% - R_c$, which height is used in all prediction formulae for velocity, flow depth and flow time.

Figure 3.25 first shows distributions of overtopping wave volumes for discharges of 0.1; 0.6; 1.2 and 2.0 cfs/s/ft for the 8 ft wave condition, comparable to Figure 2.3. The only difference is that the horizontal scale is given as a Rayleigh scale and the basis for the distribution is the number of incident waves and not the actual number of overtopping waves.
For instance, the 0.1 cfs/s/ft discharge has 12.3% overtopping waves (Table 2.1) and therefore the curve starts at this percentage. The much larger discharge of 2.0 cfs/s/ft has 64.1% overtopping waves and starts much more to the left in the graph. The curves give the theoretical distribution as a line, but the 15 largest individual overtopping volumes are given by markers. The 2% line has also been highlighted. There are only a few volumes (actually 6) that are larger than the 2%-value.

The 2% run-up level is calculated on basis of the seaward slope and the wave conditions. This leads to a 2% run-up level of 8.04 m and is given by the large green dot. We assume a Rayleigh distribution for the run-up levels. This means that the distribution of run-up levels is given in Figure 3.25 by a straight line through the 2% level.

Each given discharge is reached by a certain crest freeboard, \( R_c \). The larger the overtopping discharge, the lower the crest freeboard will be. Crest freeboards corresponding with the four overtopping discharges are given by the horizontal dotted lines. The 0.1 cfs/s/ft discharge needs a crest freeboard of 5.88 m, the 2.0 cfs/s/ft discharge a freeboard of 2.71 m (see also Table 2.1).

These crest freeboards coincide with the distribution of run-up levels where the distribution of wave overtopping volumes start. This is logical, as there will be no overtopping for wave run-up levels lower than the crest freeboard.

Suppose that we want to compare the vertical distance \( R_{u2\%} - R_c \) for a fixed volume of wave overtopping of say 4 \( m^3/m \). Each of the four discharges have such a volume in their distribution. The level is given by the black horizontal line. The vertical distance \( R_{u2\%} - R_c \) is now the given by the difference in run-up level (green line) and the crest freeboard (dashed horizontal line) at the location where the distribution of overtopping wave volumes coincides with the horizontal line at 4 \( m^3/m \). These vertical distances, used in the prediction formulae, are given in Figure 3.25 by double arrows.

The vertical distances \( R_{u2\%} - R_c \) in Figure 3.25 decrease a little from small discharges...
(at the right of the graph) to large discharges. This is the reason why flow velocity, flow depth and flow time decrease if the overtopping discharge increases. Physically this may not be totally correct, but the graph explains how it this occurs, given the present assumption on hydraulics at the crest of a levee.

The mean curve in Figure 3.18 has more or less to be simulated by the Wave Overtopping Simulator and has also been used to design the shape of this Simulator. Some values are given in Table 3.3.

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</tr>
<tr>
<td>17000</td>
<td>10.4</td>
</tr>
</tbody>
</table>

Table 3.3. Proposed mean values for flow velocity at the crest versus overtopping volume.

There is too much doubt on the prediction formulae and graphs for the flow depth and flow time. Flow times should be around the peak period for the largest overtopping wave volumes only and smaller for smaller discharges.

More and very dedicated research is required to solve the inconsistencies in the present prediction formulae.
4 Design of the Wave Overtopping Simulator

4.1 Dutch and Vietnamese design

The Wave Overtopping Simulator is not more than a box with a special shape that is constantly filled with water (the constant required overtopping discharge), a butterfly valve that is opened at prescribed times to release all the water in the box, and a transition from valve opening to a horizontal flow of water at the crest of a levee.

The shape of the box depends on the flow velocities required at the crest of the levee. The pressure height above the valve determines the flow velocity at the valve (Bernoulli). As first approximation of the velocity at the valve the following equation can be used:

$$u = \sqrt{2gx}$$

where \(x\) = the pressure height. Given the overtopping volumes and required velocities at the crest from Chapters 2 and 3, the required pressure height and corresponding average width of the box can be calculated.

The average shape for each overtopping volume, as used to design the first (Dutch) Wave Overtopping Simulator, is given in Figure 4.1 by thin lines. Small overtopping volumes need small pressure heights and small widths. The large overtopping volumes need a fairly large height and still a fairly small width. These theoretical considerations give a slender type of box. The first design was limited to a maximum volume of the box of 3.5 m³/m (the US Simulator will be 16 m³/m).

The velocities have been calculated directly at the crest. In reality space is required for a transition slope in order to change the direction of the velocity from vertical to horizontal. For this reason the valve will be a certain distance above the crest of the dike. Certainly small volumes will increase in velocity from valve opening to the crest due to gravity. Larger volumes may have more friction at the opening.

![Figure 4.1. Design of shape of Dutch Simulator in 2006.](image-url)
In order to have the small volumes as close as possible to the crest and still have enough pressure height for the large volumes, the shape in Figure 4.1 has been chosen for the design: wide at the bottom and high and slender for large volumes. The box can vary in height by adjustable legs. From calibration it appeared that a fixed height of the valve above the crest of 1.25 m gave the required velocities. The net valve opening was 0.6 m. Figure 4.2 shows the 4 m wide Dutch Simulator in action in 2007.

![Figure 4.2](image)

**Figure 4.2.** The 4 m wide and 3.5 m³/m large Dutch Simulator in 2007 testing a levee.

The Dutch Simulator was increased in volume in 2007 by adding 2 m³/m more on top of the existing box, see Figure 4.3. The Simulator became even more slender. Experience with the use of the Simulator during testing led to a few improvements, like a special platform for the adjustable legs and a special power generator for the hydraulic cylinders (instead of a tractor, see Figure 4.2).

![Figure 4.3](image)

**Figure 4.3.** Improved Dutch Simulator, 2008. Volume increased to 5.5 m³/m, special platform with adjustable legs and separate power generator.

Other improvements were made when a Simulator was designed for Vietnam in 2008. The transition slope and the platform became integral parts of the box, instead of separate parts. The hydraulic cylinders to open and close the valve were put outside the box, which gave more power to close the valve completely (which is required for large volumes, i.e. large water pressures on the valve causing leakage). Another modification was that the shape of the Simulator changed according to the given theory above. Figure 4.4 gives the theoretical sizes for each volume and the chosen shape. The net valve opening became 0.6 m. Figure 4.5 shows the Vietnamese Simulator in action during May 2009.

![Figure 4.4](image)

**Figure 4.4.** Theoretical sizes for each volume and the chosen shape.

![Figure 4.5](image)

**Figure 4.5.** Vietnamese Simulator in action during May 2009.
Figure 4.4. Design of shape of Vietnamese Simulator, 2008.

Figure 4.5. Vietnamese Simulator in action in 2009.
4.2 US design
Given the hydraulic boundary conditions to be tested for New Orleans' situations, as described in Chapters 2 and 3, the US Wave Overtopping Simulator will have a maximum size of 16 m$^3$/m. This is almost three times the size of the Dutch or Vietnamese Simulator.

Figure 4.6 gives the theoretical shapes of the box for various volumes up to 16 m$^3$/m. It also gives the shapes of the first Dutch 3.5 m$^3$/m Simulator and the 5.5 m$^3$/m Vietnamese Simulator. The US Simulator needs to be higher and wider.

![Figure 4.6. Theoretical shape of the US Simulator for various volumes and the original Dutch and modified Vietnamese Simulator.](image)

The maximum size of the US Simulator is based on maximum test conditions only. This means that in most cases the released volumes during testing will be much smaller than the maximum size. In probably more than 95% of all overtopping volumes to be released for testing the volume will be less than 6-8 m$^3$/m. It is therefore required that this majority of volumes simulate the flow velocity in a proper way and that the design and working of the Simulator is not focussed on the large volumes only.

The Dutch and Vietnamese Simulators have proven that volumes up to 5.5 m$^3$/m can be generated with their designed shape. It is for this reason that this proven concept was used again for the US Simulator. Figure 4.7 gives the principal idea. An inner Simulator has been designed more or less similar in size and shape as the Dutch or Vietnamese design. The maximum size will be about 6 m$^3$/m. In order to realize the volumes in excess of 6 m$^3$/m an outer Simulator was designed around the inner one. This outer Simulator has a shape according to the theoretical shape of the larger "water boxes", see Figure 4.7.
The Simulator will be filled from above, directly into the inner Simulator. It a larger volume than 6 m$^3$/m is required, the inner Simulator will overflow and fill the outer Simulator as well, till the required volume. All the water will be released through the butterfly valve at the inner Simulator. The size of this valve has been enlarged to a net opening of 1.0 m. Water should also flow from the outer Simulator into the inner one. For this reason two valves have been designed at the side of the inner Simulator, see Figure 4.7. These valves will close when the inner Simulator is filled, due to the water pressure. If the outer Simulator has to be filled to some extend and the butterfly valve is opened, the pressure in the inner Simulator will drop and the valves at the side will automatically open, releasing also the water from the outer Simulator. These valves at the side of the inner Simulator should automatically work without steering by hydraulic cylinders.

### 4.3 Mechanical design

Conceptual design drawings were made of the US Simulator and its mechanical parts. As CSU has a well equipped mechanical shop with experienced people it was agreed that conceptual mechanical drawings would be enough, together with explanation and discussion during a visit. Mechanical design drawings were made and the visit took place 21 and 22 October 2009. Detailed design, ordering of steel and the actual construction started at CSU from that meeting. Contacts between CSU and the mechanical engineer, Gerben van der Meer, were maintained via email.

This Section gives the conceptual design drawings with some explanation. Figure 4.8 shows a 3D-sketch of the Simulator. It shows the inner and outer Simulator and the integrated platform with adjustable legs and the transition slope. A safety floor is suggested, as well as a measure ("water splitter") to prevent falling water damaging the valve.
Figure 4.8. 3D-sketch of the US Simulator.

Figure 4.9 gives the cross-section of the Simulator. Sizes can be read from the left drawing where every square is equal to 0.20 m. The right drawing gives the actual cross-section with water splitter and safety floor. As some test conditions have long wave periods it might be a solution to decrease the total release opening in order to lengthen the flow of water. For this reason an adjustable front plate has been designed, which can be brought and held in position by hydraulic cylinders.

Figure 4.10 gives the details of the adjustable legs, where an inner square profile can slide through an outer square profile.

Figure 4.11 shows details of the platform with adjustable legs. It also shows the level gauges (see Figure 4.12 as well). These level gauges are equipped with a floater and enables a visual check of the level of the water in the inner and outer Simulator. A further idea is to measure water pressures (= water level) by means of pressure transducers, which enables a direct check of the filled volume on the steering pc.

Figure 4.12 gives a front view of the Simulator. The total width of the Simulator is 6 ft, 1.83 m, which is less than half of the Dutch Simulator with 4 m.

Figure 4.13 gives a top view with the location of the adjustable legs.

Finally, Figure 4.14 shows the transition slope from valve opening to the horizontal crest. It gives details of the sealing at the valve and the location of the cylinder.
Measures: each square is 0.20 m

Figure 4.9. Cross-section of the Simulator.

Figure 4.10. Details of the adjustable legs.
Design of the US Wave Overtopping Simulator; version 1.0

Figure 4.11. Details of the platform with adjustable legs.

Figure 4.12. Front view of the Simulator
Figure 4.13. Top view of the Simulator.

Figure 4.14. Cross-section and details of the transition slope, the butterfly valve with rubber sealing and the location of the hydraulic cylinder.
4.4 Test set-up
Initially the Wave Overtopping Simulator was designed to simulate wave overtopping at the crest and landward slope of levees and to test the strength of such levees. The objective of the tests for New Orleans' situations is to test reinforcement systems on landward sides of levees. Reinforcement systems have been tested at CSU for long, but then for constant overflow only, like for dams. Reinforcement systems are placed in 6 ft wide trays of steel and then brought to a test location. The US Wave Overtopping Simulator will also be situated at such a test location. The design of the test location, water supply, trays, etc. is the responsibility of CSU.

But there will be similarities with a test set-up at a levee and therefore the various test set-ups that were made in the Netherlands will be described. Figure 4.15 shows the test set-up for the first testing in 2007. The Simulator stands partly on the crest, which means that only the landward slope was tested, not the crest itself. The water was taken from the inner ditch and pumped into the Simulator. The pump was frequency controlled and could be set at any discharge, which then remained constant. The power generator for the pump actually gave power for the whole test site.

The water has to be guided along the slope by installing guiding walls. In this first set-up the lower part of the down slope was protected by rock as it was seen as a weak spot. If we had to test this levee again, this part would of course be part of the testing, but at that stage the focus was on the landward slope only. Figure 4.15 shows that the measuring cabin was placed onto two storage containers. The water flowed between these containers. In this way there was a direct view on the test section. The power for the hydraulic cylinders at the valve came from a tractor at the dike crest.

Figure 4.15. First set-up of the Dutch Simulator in 2007.

Figure 4.16 shows the set-up in 2008, where a 30 m long landward slope was tested. The Simulator was enlarged to 5.5 m$^3$/m. A special cabin was hired for visitors (the first cabin on the picture. The measuring cabin was now placed on a carriage which made displacement to another test section very easy. The distance between the wheels was large enough to let the water flow between these wheels and underneath the carriage. Hydraulic power for the cylinders was provided by a hired power generator instead of the tractor.
Figure 4.16. Set-up in 2009, Boonweg.

Figure 4.17. Set-up in 2009, St Philipsland

Figure 4.17 shows a second set-up in 2009. Now the Simulator is placed more on the seaward slope and in this way the crest could be tested too. At this location water had to be brought to the inner ditch as the natural supply of water was not enough. The picture on the right hand side shows that stability of the Simulator during storms was provided by fixing it to 4 ton concrete blocks.

The set-up in 2009 is shown in Figure 4.18. Here there was not water between the landward slope and the highway and for this reason a 100 m long ditch was made and covered/sealed with heavy plastic fabrics. The site had also to be visually protected from the highway in order not to attract attention of drivers on this highway.
Figure 4.18. Test set-up in 2009.
5 Steering of the valve of the Wave Overtopping Simulator

5.1 Steering file

One of the main details of the Wave Overtopping Simulator is steering of the valve to release the water in the box. If a required volume in the box is reached, the valve should be opened and closed again to receive the next volume. In the first two years of testing with the Dutch Simulator the steering was done manually by a joy stick and a steering file on paper. The operator got a precise time to open the valve and by experience he knew when to close it (earlier for small volumes, later for very large volumes). The main advantage of manual operation is that the operator is always present and he/she can stop immediately in important situations, for example if damage progresses rapidly or instruments/parts break down. With manual operation it is also possible to deviate from the steering list, and create more time between overtopping volumes, in order to repair quickly or remove dirt from instruments.

But operating a joy stick for 6 hours a day is also tiring. It is for this reason that automatic steering was designed for the Dutch Simulator in 2008, but keeping the joy stick option. First the steering file will be discussed and then the modifications required for the US Simulator. In a next section the device itself will be described.

Each hydraulic condition and required overtopping discharge gives a distribution of overtopping wave volumes that have to be simulated. Figures 2.9 and 2.10 give the distributions for the 8 ft and 3 ft wave conditions, respectively. These distributions are given in ascending order of volume. In reality wave overtopping is random and the first that has to be done is to randomise the list of volumes. The same number of overtopping volumes is present as well as the volume for each wave, but only the order of executing the volumes is randomised.

Figure 5.1 shows the total test sequence as performed for a full test in the Netherlands. Each test condition lasted for 6 hours and these conditions were 0.1; 1; 10; 30; 50; and 75 l/s per m. Figure 5.1 shows indeed the random simulation of overtopping wave volumes. It shows too that most overtopping volumes are small, only a few very large overtopping wave volumes are present.

![Figure 5.1](image_url)

**Figure 5.1.** Overtopping wave volumes in time for a full sequence of testing in the Netherlands, each condition for 6 hours with increasing discharge.
When the volumes are randomised, a list exists with the wave number and the volume of that overtopping wave. The time duration, $t_i$, to fill the Simulator to the required volume, $V_i$, is simply:

$$t_i = \frac{V_i}{q} \quad (5.1)$$

These durations are marked from “valve just closed” to “valve just closed again”, see also Figure 5.2. Also during the time that the valve is opened and the water is released, the filling of the box continues (although the water is released at the same time). Applying Equation 5.1 for each volume gives a cumulative list of times when the valve should be closed again.

The time to calculate then is the time when the valve should open. This is a few seconds before the “valve closed” time, depending on the size of the volume, see Figure 5.2.

![Figure 5.2. Calculation of steering file](image)

The speed of the valve to open and close depends on the hydraulic power on the cylinder. These time durations must be established by calibration. It can also be set in some way by adjusting the hydraulic power. Suppose the time to open the valve completely is $t_o = 2$ s and to close it is $t_c = 3$ s. The time to empty a full (Dutch) Simulator is then a duration that has to be established by calibration. Suppose that this time is 5.5 s for a volume of 5.5 m$^3$/m. This gives a factor of $f_e = 1.0$ m$^3$/s per m.

The release time for each volume is then, assuming a linear relationship, the required volume divided by this factor $f_e$. We also want to be sure that all water indeed has left the Simulator before the valve has completely closed. For this reason 1 s extra is taken for each volume. The total duration that the valve is open is than given by:

$$\text{Duration for valve to be open} = \frac{V_i}{f_e} + 1 \quad (5.2)$$

The calculated duration should be subtracted from the time that the valve should be just closed (see Figure 5.2) and gives the time to open the valve. With the durations and speeds of the valve to open and close, $t_o$ and $t_c$, it is then also possible to calculate the time when the valve should start to close. For a small volume the valve does not fully open, as the signal “close it” is given earlier. For a large volume it is possible that the valve opens fully and stays open for some time, before the valve has to close again. The steering file is then simply composed by four values, the actual number of the overtopping wave volume in chronological order, the volume of this wave, the time when to open the valve and a duration in 0.1 s after which the valve should be closed. A part of such a steering file is shown in Table 5.1.

<table>
<thead>
<tr>
<th>Number of wave</th>
<th>Volume</th>
<th>Time to open</th>
<th>Time to close</th>
<th>0.1 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>...</td>
<td>......</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
<tr>
<td>232</td>
<td>2436</td>
<td>3235</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>233</td>
<td>620</td>
<td>3256</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>234</td>
<td>5500</td>
<td>3439</td>
<td>35</td>
<td></td>
</tr>
<tr>
<td>...</td>
<td>......</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
</tbody>
</table>

Table 5.1. Part of a steering file for the Dutch Simulator.
The steering of the US Simulator has one complication more than the Dutch one. The wave overtopping conditions to be simulated for New Orleans’ conditions have two completely different wave periods, a very long one with a peak period of 14 s and a much shorter one of 6 s. The latter one is comparable with what the Dutch Simulator can do.

The valve opening of the US Simulator is also larger than for the Dutch Simulator, in order to release the very large volumes quickly enough. This all means that the size of valve opening becomes to play a role. It should not open too much for small volumes and also not for required long wave periods. The valve opening should be made adjustable.

In principal there are two systems to control the size of valve opening. Knowing the speed of opening of the valve (depending on the hydraulic power), a duration can be calculated to open the valve and then it should stop after such a duration. The disadvantage of this system is that the opening depends also on the hydraulic power of the system and if this changes a little, this should be recognised and the steering file should be adjusted.

A second method is to place a rotational encoder on the axis which measures the rotation of this axis and valve. The rotation is directly linked to the size of valve opening. The steering file can then contain the maximum rotation (= valve opening) for each overtopping wave volume with a certain period.

The required maximum rotation for each overtopping condition has to be established during a calibration period, when the Simulator has been fabricated. The Simulator should be filled with different volumes and released with different valve openings or degree of rotation. This will give a matrix of volumes, valve openings and realised flow velocities, flow depths and flow times and is the input for the steering file.

The steering file will then have one extra column, for example the maximum opening of the valve, given by the maximum rotation, see Table 5.2.

<table>
<thead>
<tr>
<th>Number of wave of wave</th>
<th>Volume l/m</th>
<th>Time to open s</th>
<th>Max opening ° rotation</th>
<th>Time to close 0.1 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>...</td>
<td>.....</td>
<td>.....</td>
<td>.....</td>
<td>...</td>
</tr>
<tr>
<td>232</td>
<td>2436</td>
<td>3235</td>
<td>6.25</td>
<td>18</td>
</tr>
<tr>
<td>233</td>
<td>620</td>
<td>3256</td>
<td>2.13</td>
<td>9</td>
</tr>
<tr>
<td>234</td>
<td>5500</td>
<td>3439</td>
<td>14.63</td>
<td>35</td>
</tr>
<tr>
<td>...</td>
<td>.....</td>
<td>.....</td>
<td>.....</td>
<td>...</td>
</tr>
</tbody>
</table>

Table 5.2. Example of a possible steering file for the US Simulator.

5.2 PLC

The steering file is input for the system that operates the butterfly valve of the Simulator. The operation is done by hydraulic power to cylinders attached to the Simulator and to the valve. The steering itself of the Dutch Simulator is performed by a programmed PLC. That system will be described first and then possible options for the US Simulator.

The Dutch PLC is placed in a steel box and has a touch screen, see Figure 5.3. The inner side of the box with this touch screen is given in Figure 5.4. Figures 5.5 and 5.6 show the PLC itself with its inner parts, including a memory card with steering files. As the original joy stick was still available, this could also by attached to the PLC in case the steering system would not work, see Figures 5.7 and 5.8.
Figure 5.3. Dutch PLC with touch screen
Figure 5.4. Rear side of touch screen.

Figure 5.5. Inner parts of PLC
Figure 5.6. Details with memory card.

Figure 5.7. Joy stick connected to PLC
Figure 5.8. Original joy stick.

Figure 5.9. Touch screen to set speed of hydraulic power and to load steering file.
The first screen of the system is given in Figure 5.9. The durations to open and close the valve completely have to be measured and given to the system. Actually, they are not use in the steering system, but should be used in preparing the steering files. The screen also enables to load a specific steering file.

The main screen is shown in Figure 5.10. The left side displays the next ten wave overtopping volumes that will be generated (in Figure 5.10 not a random file, but repeated and then increased volumes) and in the second column the time at which the valve will be opened. There is also a digital clock, so it is easy to see for the operator when the next volume will be released and what size it will have. In Figure 5.10 for example the time after start of testing is 606 s, where the next volume will be released at 616 s, 10 seconds later.

The buttons in the middle and right down part of the screen give the possibility for automatic operation or interrupted manual operation. With “discharge active” the test starts with automatic operation.

There are two ways to interrupt the manual operation and steer the valve like a joy stick. With pushing “manual” the steering time continues and volumes, that were foreseen during the time that the Simulator is operated manually, will be discarded. This original steering file continues when the “discharge active” is used, but at the elapsed time. This kind of operation is useful when one needs a little more time between overtopping volumes, in order to do small repairs or clean the instruments from dirt.

The second way of manual operation is by pushing “interrupt file” and then “manual”. The steering time remains then at the time where it was interrupted. This can be used if the test has to be interrupted due to damage, bad functioning instruments, or other unforeseen accidents and with the intention to continue testing after repair. The pumping can even be stopped and the test later continued.

The manual operation can simply be done by pushing the "open" and "close" buttons, just like using a joy stick.

If “discharge active” is pushing during operation, the test will stop and can not be continued where it has been stopped.

![Main screen of operating system.](image)

In fact a similar system can be made for the US Simulator. The PLC will be a little different with more components (there is an extra steering to stop opening the valve at some size of opening), but the working itself with the touch screens could be similar.
Technique progresses, however, and it is possible to make a system based on a pc with touch screen. In such a case the PLC itself will be a closed steel box without a touch screen, but will still look similar as in Figures 5.5 and 5.6. This box can be placed close to the hydraulic system of the Simulator. It will then be connected with the pc, which replaces the touch screen as in Figure 5.3 and the steering file is sent directly from the pc (instead of the memory card in Figure 5.6). In this way the actual (hydraulic, mechanical and electronic) system to operate the valve is disconnected from the steering of the valve, which may be an advantage.

But also for the US Simulator it is proposed to make a joy stick, which enables a direct replacement of the pc if this system is not working.
6 Hydraulic measurements

6.1 Flow depth: surfboard

In the first years of testing it appeared to be very difficult to measure any hydraulic parameter on the landward slope, like flow velocity or flow depth. The velocities can approach 8 m/s and the water is very turbulent with a lot of air entrainment. Laboratory instruments have not been designed for this kind of conditions. In 2009 a lot of attention was focussed on improving the measurements in the Dutch testing.

Amongst them a floating device to measure the flow depth and a high speed camera to measure front velocities of an overtopping wave. The floating device is a curved board which has been hinged about 1 m above the slope and which floats on top of the flowing water. The rotation at the hinge is measured and gives the flow depth. Shape and measures of this surfboard are given in Figure 6.1.

![Figure 6.1. Plan view and cross-section of surfboard to measure flow depth (measures in mm).](image)

Figure 6.2 shows the record of this floating device for three consecutive overtopping volumes of 3.0 m$^3$/m each. Recording started exactly when the signal was given to open the valve. The overtopping volumes and the records of flow depth reproduce very nicely. The maximum flow depth is about 0.25 m.

![Figure 6.2. Record of flow depth with floating device for 3 overtopping waves of 3 m$^3$/m](image)
Hydraulic measurements in 2009 were performed by starting with small overtopping wave volumes, repeating them three times and then increasing the volume. Flow depth measurements of consecutive overtopping wave volumes are shown in Figure 6.3. The smallest overtopping volume is 200 l/m and the largest 4 m$^3$/m. There is a very nice trend in increasing flow depth with increasing overtopping volume. Also the records start earlier in time for increasing volumes, indicating that the velocity of these volumes is larger (the time between opening the valve and reaching the device is smaller). Measured flow depths have already been discussed in Figure 3.22.

![Figure 6.3. Flow depth measurements of consecutive overtopping wave volumes with the surfboard.](image)

### 6.2 Front velocity: high speed camera

It is extremely difficult to measure flow velocities in the turbulent and air contained flow. Beside this difficulty, a measurement of velocity is always a point measurement, somewhere above the slope. Velocities close to the slope will be smaller than further above the slope. The actual interesting parameter is the average flow velocity, which can not be measured. It is for this reason that the front velocity may give a fair estimation of the maximum average velocity. The front velocity is the velocity of the front of the wave over the slope.

![Figure 6.4. Two pictures of the high speed camera, where the front of the wave travelled 1.0 m in 0.18 s, giving a front velocity of 5.5 m/s](image)
One way to measure front velocity is to use a high speed camera. Such camera was used in 2009, which took 50 frames per second. Contours with a distance of 1.0 m had been drawn on the grass of the inner slope, see Figure 6.4. This figure gives two pictures by this camera, where the wave front travelled 1 m. The duration over a distance of 1 m or 2 m gives the front velocity. Measured front velocities with this high speed camera have already been discussed in Figure 3.19.

6.3 Integration of flow depth and velocity over time
The released volume from the Simulator is predefined. For the hydraulic measurements volumes from 200 l/m up to 4 m$^3$/m were released and each volume three times. A recalculation of the released volume can be derived by integrating the flow depth record over time and multiplying it with the velocity. It is assumed that the front velocity is similar to the maximum depth averaged velocity and that flow velocity and flow depth have the same shape of record in time.

Figure 6.5 gives the released volumes versus the volumes found by integration. Measurements were performed with fresh water as well as with salt water, the main difference being that the size of air bubbles in salt water is much smaller than in fresh water. The integrated volumes are about 20-25% bigger than the released ones (except for 3000 l per m) showing that the air entrainment might by about 20-25%.

![Graph showing released volumes versus volumes found by integration](image)

Figure 6.5. Released volumes from the Dutch Simulator versus volumes found by integration over flow depth and front velocity.

6.4 New developments
Although the high speed camera works well to determine front velocities, it is also time consuming in data analysis. Other methods with wire gauges placed apart 1 or 2 m did not yet give reliable results. But the method of “front detectors” (as with the high speed camera) must be reliable if the instruments give reliable records. The surfboard is such a reliable instrument, see the measured records in Figures 6.2 and 6.3. An idea is to place more surfboards on a levee slope, preferably about 2 m apart. Depending on the length of the slope about 5 surfboards must be sufficient to measure flow depth as well as front velocity along the slope. For the next tests in the Netherlands in February/March 2010, five of these surfboards will be made and used.
Another new development is on measuring velocities. A well proven method to measure the speed of a small boat is to use a small paddle wheel on the hull. Such a paddle wheel is about 2 cm in diameter and 1 cm thick. The whole system is fairly light and it must be possible to place it on the surfboard without making it too heavy. It is not known what the effect of the turbulence and air entrainment will be, but it seems feasible to place such a paddle wheel through the surfboard and measure velocities. This system will be made on three of the Dutch surfboards.

A paddle wheel on the surfboard will measure velocities in the most upper part of the flow. It is also possible to measure flow velocities directly near the slope if such a paddle wheel is mounted on a plate and this plate is fixed to the bottom of the slope. One such device will be developed for the next tests in the Netherlands.

If the tests in February/March 2010 show good results, similar or improved systems can be made at CSU. There is enough time make these instruments after the experience in the Dutch testing becomes clear.
References
(Relevant to the Wave Overtopping Simulator, not all cited).

CLASH, Crest Level Assessment of coastal Structures by full scale monitoring, neural network prediction and Hazard analysis on permissible wave overtopping. Fifth Framework Programme of the EU, Contract n. EVK3-CT-2001-00058. www.clash-eu.org