ComCoast

Workpackage 3: Development of Alternative Overtopping-Resistant Sea Defences

Phase 3: Design, construction, calibration and use of the wave overtopping simulator

Interreg North Sea Region

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This report has been prepared by a consortium of Infram and Royal Haskoning

The ComCoast project is carried out in co-operation with ten partners:
- Rijkswaterstaat (NL - leading partner)
- Province of Zeeland (NL)
- Province of Groningen (NL)
- University of Oldenburg (D)
- Environmental Agency (UK)
- Ministry of the Flemish Community (B)
- Danish Coastal Authority (DK)
- Municipality of Hulst (NL)
- Waterboard Zeeuwse Eilanden (NL)
- Waterboard Zeeuws Vlaanderen (NL)

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Workpackage 3: Development of Alternative Overtopping-Resistant Sea Defences

Phase 3: Design, construction, calibration and use of the wave overtopping simulator

Final Report

Acknowledgement:
This report is a deliverable of WP 3 " Smart Grass Reinforcement "
Part of the work was co-financed by the Ministry of Transport, Public Works and Water Management under the SBW programme (Sterkte & Belastingen Waterkeringen)
The work is performed by a consortium of Infram and Royal Haskoning
The report is written by dr J.W. van der Meer of Infram.

PREFACE

Mission Statement of ComCoast

MISSION OF COMCOAST (= COMbined functions in COASTal defence zones)
ComCoast is a European project which develops and demonstrates innovative solutions for flood protection in coastal areas.

ComCoast creates and applies new methodologies to evaluate multifunctional flood defence zones from an economical and social point of view. A more gradual transition from sea to land creates benefits for a wider coastal community and environment whilst offering economically and socially sound options. The aim of ComCoast is to explore the spatial potentials for coastal defence strategies for current and future sites in the North Sea Interreg IIIb region.

ComCoast Goals:
- developing innovative technical flood defence solutions to incorporate the environment and the people and to guarantee the required safety level;
- improving and applying stakeholder engagement strategies with emphasis on public participation;
- applying best practice multifunctional flood management solutions to the ComCoast pilot sites;
- sharing knowledge across the Interreg IIIb North Sea region.

ComCoast Solutions:
Depending on the regional demands, ComCoast develops tailor-made solutions:
- to cope with the future increase of wave overtopping of the embankments;
- to improve the wave breaking effect of the fore shore e.g. by using recharge schemes;
- to create salty wetland conditions with tidal exchange in the primary sea defence using culvert constructions or by realigning the coastal defence system;
- to cope with the increasing salt intrusion
- to influence policy, planning and people
- to gain public support of multifunctional zones.

ComCoast runs from April 1, 2004 to December 31, 2007. The European Union Community Initiative Programme Interreg IIIB North Sea Region and the project partners jointly finance the project costs of 5,8 million.

Information

Information on the ComCoast project can be obtained through the Project Management, located at the Rijkswaterstaat in the Netherlands.

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Project Data

• **Title:**	Design, construction, calibration and use of the wave overtopping simulator
Version:	v3.3
Clients:	CUR on behalf of ComCoast and Rijkswaterstaat
Project number:	04i103
Partners:	Royal Haskoning (partner in the consortium) Noordelijke Hogeschool Leeuwarden G. van der Meer (mechanical engineer) Nijholt Staal & Machinebouw Delft University of Technology
Summary project:	The wave overtopping simulator is a device which is able to simulate overtopping waves at the crest of a dike and at the inner slope. The device has been used in the ComCoast project to test a traditional dike section with grass and the SGR, the Smart Grass Reinforcement, placed in May 2006. This report describes in the first part (Chapters 1-6) the design, construction and calibration of the 1 m wide prototype of the wave overtopping simulator, as it was performed from May – July 2006, under the ComCoast programme. This first part has also been reported as a final version 2.4 in November 2006. The second part continues with the actual construction and testing of the real wave overtopping simulator and the use of it during testing at the dike at Delfzijl, all under the ComCoast programme. Further, the measurements itself and analysis of velocity and flow depth during this testing has been described. Also the work of Bosman, as performed in this project as a MSc-thesis, has been summarised. This second part describes the period from November 2006 to August 2007.
Performed by:	Dr J.W. van der Meer

Contents

1 Introduction

Workpackage 3 of the European project ComCoast deals amongst others with overtopping resistant dikes. A contest has been launched to market parties to come up with overtopping resistant solutions for dikes without increasing the size or geometry of these dikes. The consortium Royal Haskoning and Infram submitted one of the three winning solutions: the Smart Grass Reinforcement, SGR. The next stage in the process was to install a test dike section with this SGR (May 2006). In summer 2006 the grass recovered and the section has been tested in March 2007.

This testing has been performed by the so-called wave overtopping simulator. The idea of an overtopping simulator has been proposed on other occasions and the copy rights have been declared by Infram through the minutes of the TAW Techniek meeting of 13 March 2001.

The classical way of testing dikes is to test a section in a wave flume. As grass and clay can not be scaled, only large scale facilities, like the Delfta flume or GWK, can be used. With the GWK in Germany it might be well possible that extra funds can be found in the European Programme Hydrolab. This is not further explored here. With respect to the cases of Hondsbossche Zeewering and Kapelle it should be noted that even here the scale will not be 1:1, but more 1:3 as the wave heights in reality are in the order of 4-5 m, where the facilities are limited to a significant wave height of around 1.5 m. So, the actual situation cannot be tested on full scale!

Moreover, the constructed solution has to be transferred from its original location to the flume. This has been done before for existing grass dikes, but it is a very costly operation. Finally, only the failure mechanism of erosion of crest and inner slope can be tested, not the geotechnical failure mechanism of water infiltration and sliding of the inner slope. The flume is simply not wide enough to model this mechanism.

The possible failure mechanism of infiltration and sliding can only be tested on the actual dike and for a sufficient width, for example around 30 m. As velocities of overtopping waves are not the predominant load, but simply infiltration of water, the classical way of overflow may be a good test method here. Tests on overflow have been performed at various locations over the past 30 years. The dike in Flevoland has been the most recent example of this. This method can be used if this mechanism has to be tested.

For the failure mechanism of erosion of crest and inner slope, however, the wave overtopping simulator will be the ideal instrument. For this reason ComCoast has chosen to develop the overtopping simulator and to use it for testing in March 2007. This report describes the design, construction and calibration of the prototype of the simulator, the construction and testing of the real wave overtopping simulator and its use during testing on the dike at Delfzijl.

Starting points for the overtopping simulator are:

- everything about wave breaking on slopes and generating overtopping waves is known (TAW 2002: Wave run-up and wave overtopping at dikes – with the programme PC-Overslag);
- everything on individual overtopping waves is known as volumes, distributions, velocities and flow depth of overtopping water on the crest (work of Van Gent and Schüttrumph);
- actual waves are not really required to simulate wave overtopping;
- it is best to test a dike in situ.

2 Boundary conditions and theory

2.1 Wave conditions

The wave boundary conditions (wave height, period and direction, water level) in front of the dike and the geometry of the dike determine whether there will be wave overtopping and how much. With scientific research these conditions are schematised, but should be close to reality.

Tests with the overtopping simulator can be performed to assess the actual safety of a dike or can be used to determine the behaviour of various kinds of inner slopes under wave overtopping (influence of grass maintenance, clay quality, overtopping resistant systems, etc.). In the first situation the boundary conditions for the actual dike should be taken, like the 1 in 10,000 years storm or higher and also the geometry of the actual dike. In the second situation the boundary conditions for the actual dike are not important as the testing is more general of nature. Boundary conditions should be chosen more or less in agreement with the “representative” boundary conditions for a safety assessment in a large area. In the Netherlands this would be the boundary conditions for the 5-yearly safety assessment along the Dutch dikes. These dikes are mainly situated in the north (Groningen, Friesland up to Noord-Holland) and in the south-west in Zeeland. A global analysis is given here.

- For the dike in Delfzijl, where the ComCoast test has been performed (dike ring 6, sections 20-23) a wave height of 1.85 m is given, without a wave period.
- For the design of the new flood defence in Harlingen a wave height of 2.6 m was used with a peak period of 6.4 s. This gives a wave steepness of 0.041. The conditions were including a 100 year sea level rise.
- At the Westerschelde (dike ring 30 – Zuid-Beveland) wave heights are given in the order of 2 – 3 m with a period of 8.4 s. Wave heights up to 2.5 m give mainly perpendicular wave attack on the dikes, larger wave heights give more oblique attack.
- In Zeeuws Vlaanderen wave heights are given around 2 m, sometimes above 3 m. The following combinations of wave heights and periods are given:
    - H_s = 1.75 m; T = 4.4 s
    - H_s = 2.4 m; T = 5.3 s
    - H_s = 3.2 m; T = 5.9 s

  The wave period (peak, mean, etc.) is not given. Wave steepnesses are s = 0.058, 0.055 and 0.059. These values are too high for a peak period. Maximum wave steepness for a peak period is about 0.05 and 0.06-0.07 for a mean period.

Overall, wave heights around 2 m are often found as conditions for the safety assessments. A wave steepness of s_{op} = 0.04 with peak period is found in the Waddensea. Assuming T_p = 1.2 T_m, the wave steepness with the mean period becomes s_{om} = 0.058. This is similar to conditions in Zeeuws Vlaanderen. Of course larger and lower wave heights than 2 m are found, but for testing with the overtopping simulator a wave height of 2 m is accurate enough.

In summary, analysing wave boundary conditions along the Dutch coasts, as used for the 5-yearly safety assessment, gives the following average values which has been used to design the wave overtopping simulator and testing of dike sections

Wave height:	H_s = 2.0 m
Peak period	T_p = 5.7 s (wave steepness s_{op} = 0.04)
Mean period	T_m = 4.7 s (T_p = 1.2 T_m)

2.2 Wave overtopping discharge and volumes

The seaward slope of the dike is taken at 1:4. For further calculations a storm duration of 6 hours is assumed. With these values and the wave conditions from section 2.1 wave overtopping can calculated for a given crest freeboard.

Overtopping equations are given in TAW (2002). They will not be repeated here. The mean overtopping discharges mainly considered during the safety assessments or design of dikes are 0.1, 1 and 10 l/s per m width. For ComCoast also larger overtopping discharges are of interest, such as 20 and 30 l/s per m and even 50 l/s per m. TAW (2002) gives the possibility to calculate for each given mean overtopping discharge: the percentage of overtopping waves and the distribution of overtopping volumes per wave, including the maximum volume of overtopping.

With above chosen wave boundary conditions the 2%-run-up is 4.0 m above still water level. If the crest level is equal to this 2%-run-up level, the overtopping discharge becomes 0.74 l/s per m. This is certainly more than the lowest limit of 0.1 l/s per m, but still below the value of 1 l/s per m. As only 2% of the waves reach the crest, only 93 waves in a period of 6 hours will overtop the crest. This means in average 15 waves per hour or an overtopping event every 4 minutes. With the chosen wave height of 2 m the following values can be calculated:

H_s = 2 m; T_p = 5.7 s; T_m = 4.7 s

	q = **0,1** l/s per m	q = **1** l/s per m	q = **10** l/s per m
Percentage overtopping	0.192	2.744	18.89
Number of waves in 6 hours	4600	4600	4600
Number of overtopping waves	9	126	868
Maximum overtopping volume	580 l/m	1177 l/m	2675 l/m

The main difference is the number of overtopping waves. With 0.1 l/s per m only 9 waves will overtop in 6 hours (in average one every 40 minutes). For 10 l/s per m this is almost 1000 (in average every 25 s an overtopping event).

In order to get an idea of the variation if other boundary conditions would have been chosen, above table has also been produced for a higher and lower wave height than 2 m.

H_s = 1 m; T_p = 4.0 s; T_m = 3.3 s

	q = **0,1** l/s per m	q = **1** l/s per m	q = **10** l/s per m
Percentage overtopping	0,70	7.22	35.7
Number of waves in 6 hours	6545	6545	6545
Number of overtopping waves	46	473	2337
Maximum overtopping volume	236 l/m	433 l/m	1192 l/m

H_s = 4 m; T_p = 8.0 s; T_m = 6.7 s

	q = **0,1** l/s per m	q = **1** l/s per m	q = **10** l/s per m
Percentage overtopping	0,046	0.91	8.71
Number of waves in 6 hours	3224	3224	3224
Number of overtopping waves	1-2	29	280
Maximum overtopping volume	1000-2000 l/m	3132 l/m	6482 l/m

It is clear that a lower wave height of 1 m gives far more overtopping waves to produce the same mean overtopping discharge, but with much lower volumes. The opposite is true for a much larger wave height of 4 m. For 0.1 l/s per m only one or two waves will overtop.

Above tables only give the maximum overtopping volume in a storm. But overtopping volumes in overtopping waves follow a certain distribution. It is this distribution which should be generated by the simulator and preferably schematized to a fixed number of overtopping volumes as it will be difficult, and also not required, to generate exactly each volume individually. An example is given in Figure 2.1 for 1 l/s per m. during 6 hours and for a wave height of 2 m. Around 120 waves will overtop in these 6 hours, which means around 20 per hour. The overtopping volumes could be simulated as follows:

56 waves with 50 l per m
40 waves with 150 l per m
10 waves with 400 l per m
6 waves with 700 l per m
3 waves with 1000 l per m

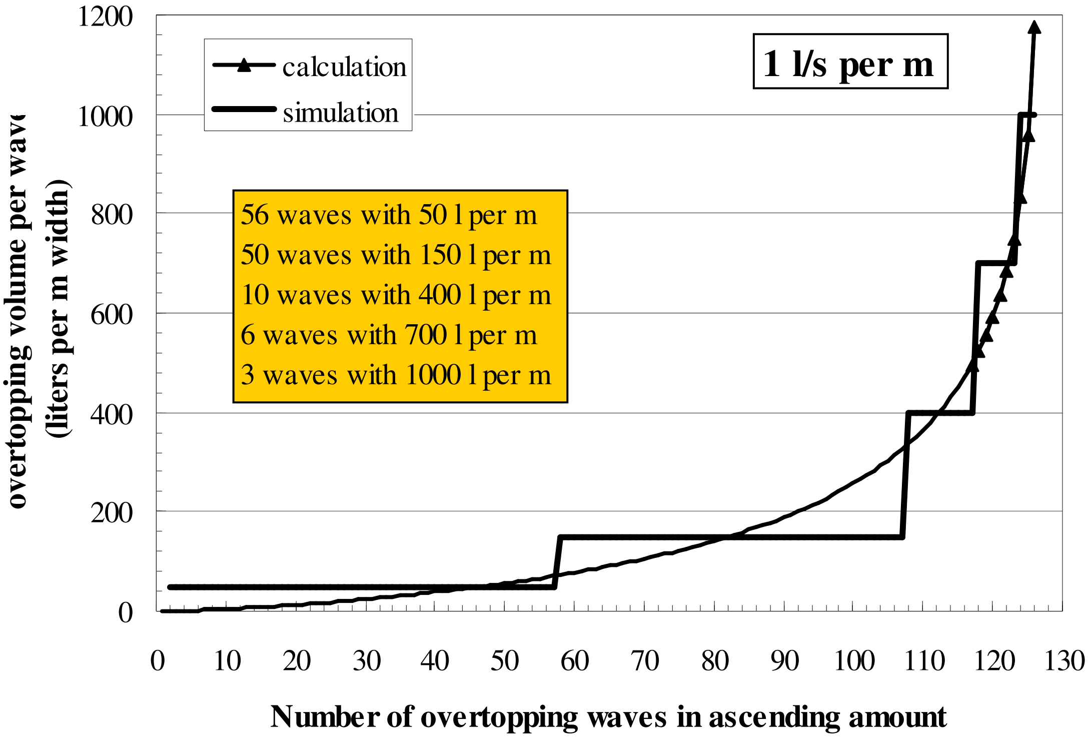


Figure 2.1. Calculated distribution of overtopping volumes and proposal for simulation. Mean discharge q = 1 l/s per m

Of course the volumes should be generated in arbitrary order. Similar graphs can be made for each prescribed overtopping condition. Figure 2.2 gives the small overtopping discharge of 0.1 l/s per m. Actually here every individual volume could be simulated instead of schematised. Figures 2.3 – 2.6 give 10 and 30 l/s per m, with both a detailed graph for the large volumes.

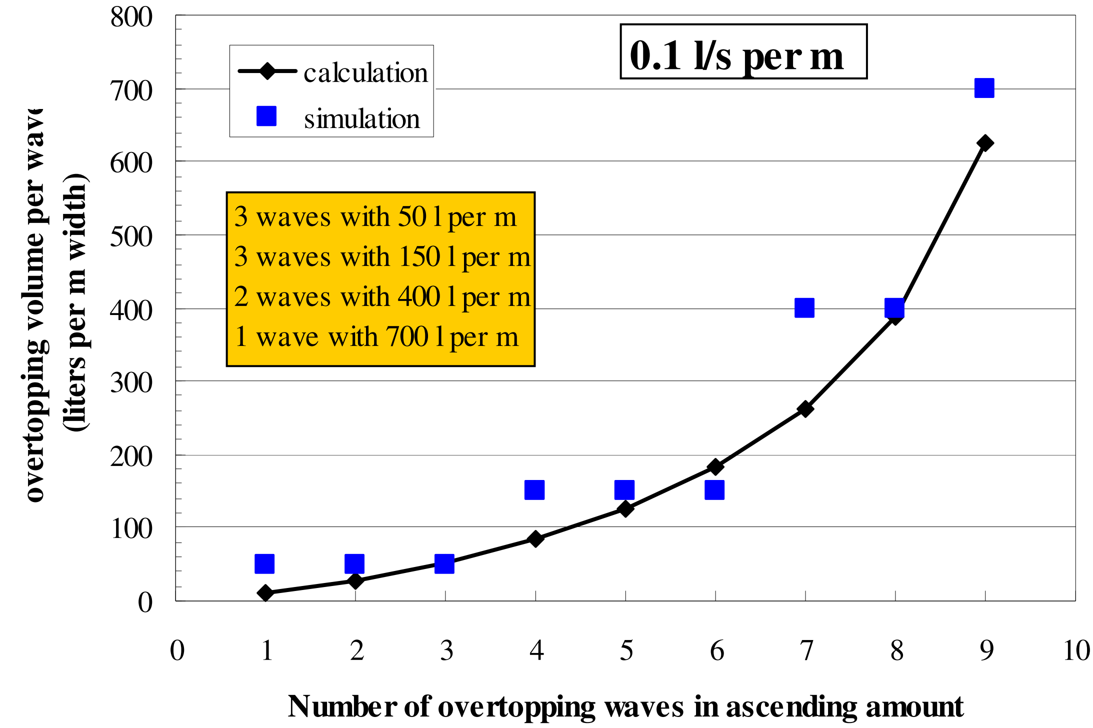


Figure 2.2. Calculated distribution of overtopping volumes and proposal for simulation. Mean discharge q = 0.1 l/s per m

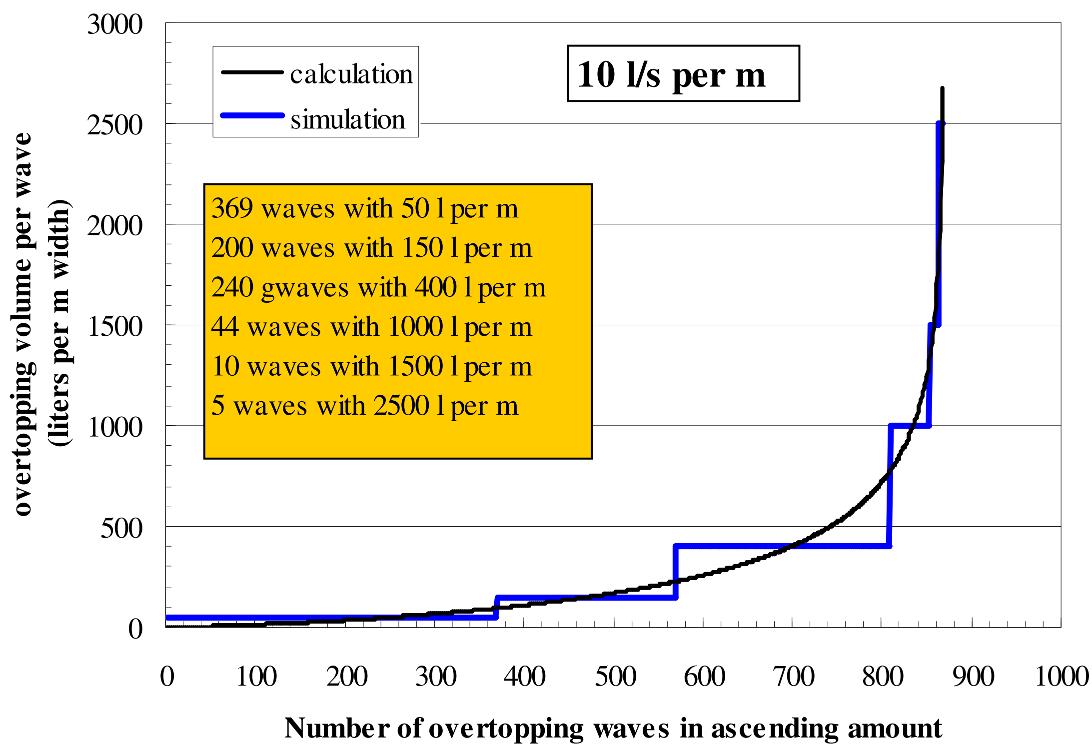


Figure 2.3. Calculated distribution of overtopping volumes and proposal for simulation. Mean discharge q = 10 l/s per m, full distribution

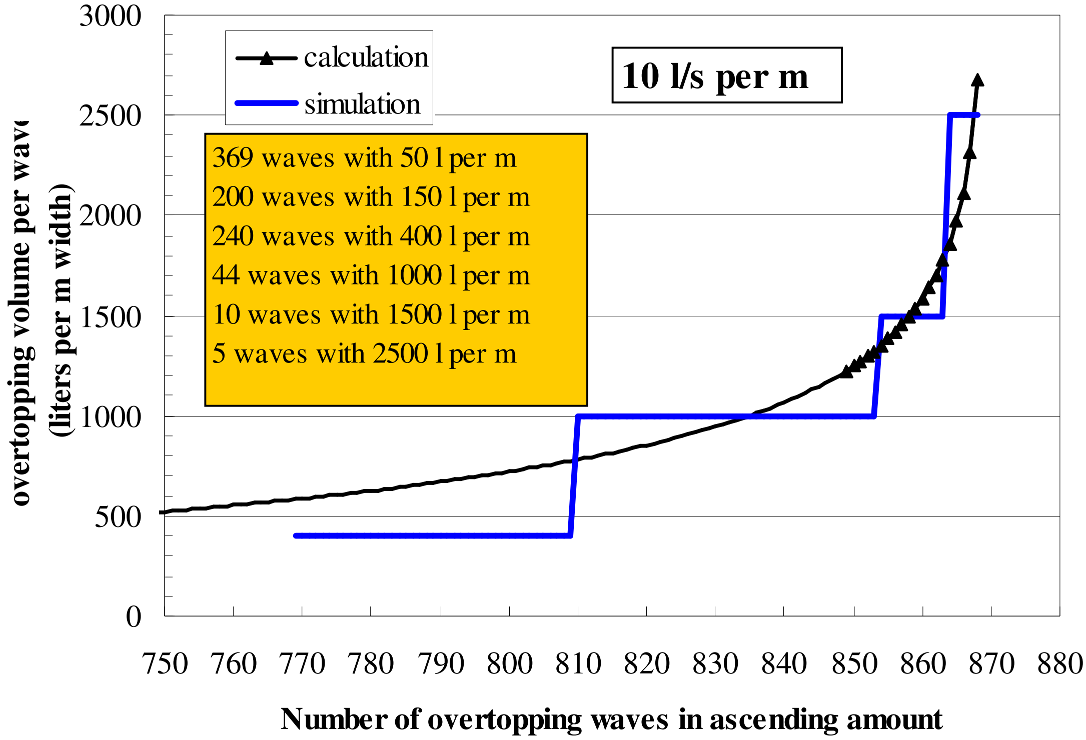


Figure 2.4. Calculated distribution of overtopping volumes and proposal for simulation. Mean discharge q = 10 l/s per m, full distribution of largest volumes

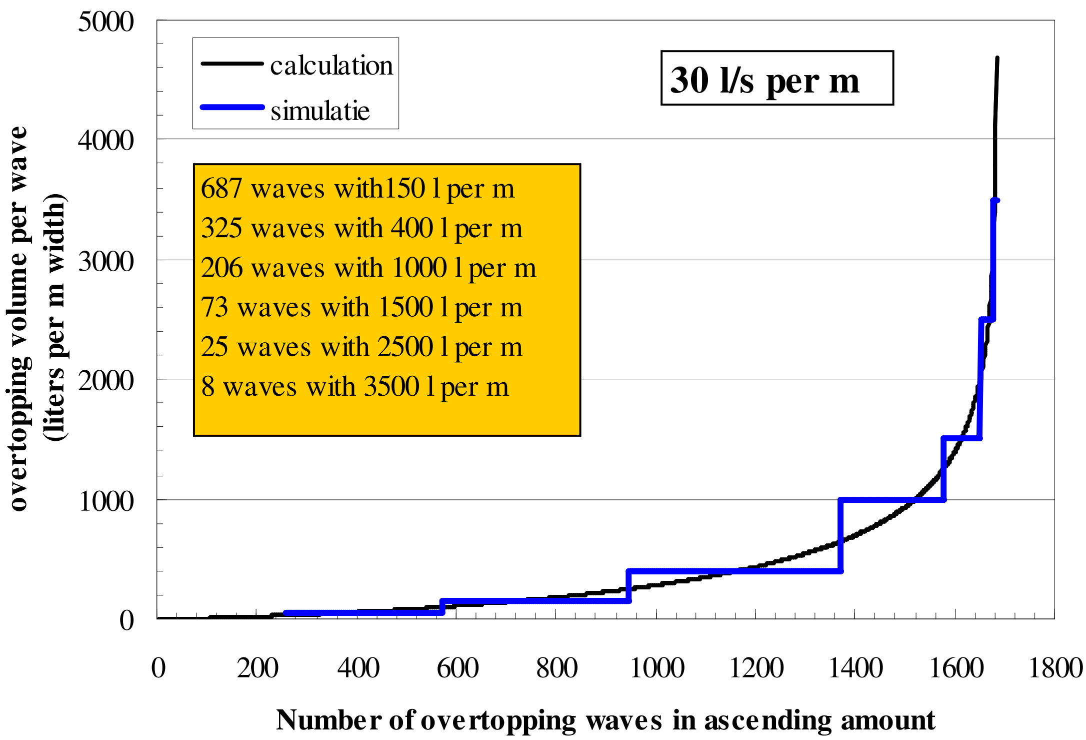


Figure 2.5. Calculated distribution of overtopping volumes and proposal for simulation. Mean discharge q = 30 l/s per m, full distribution

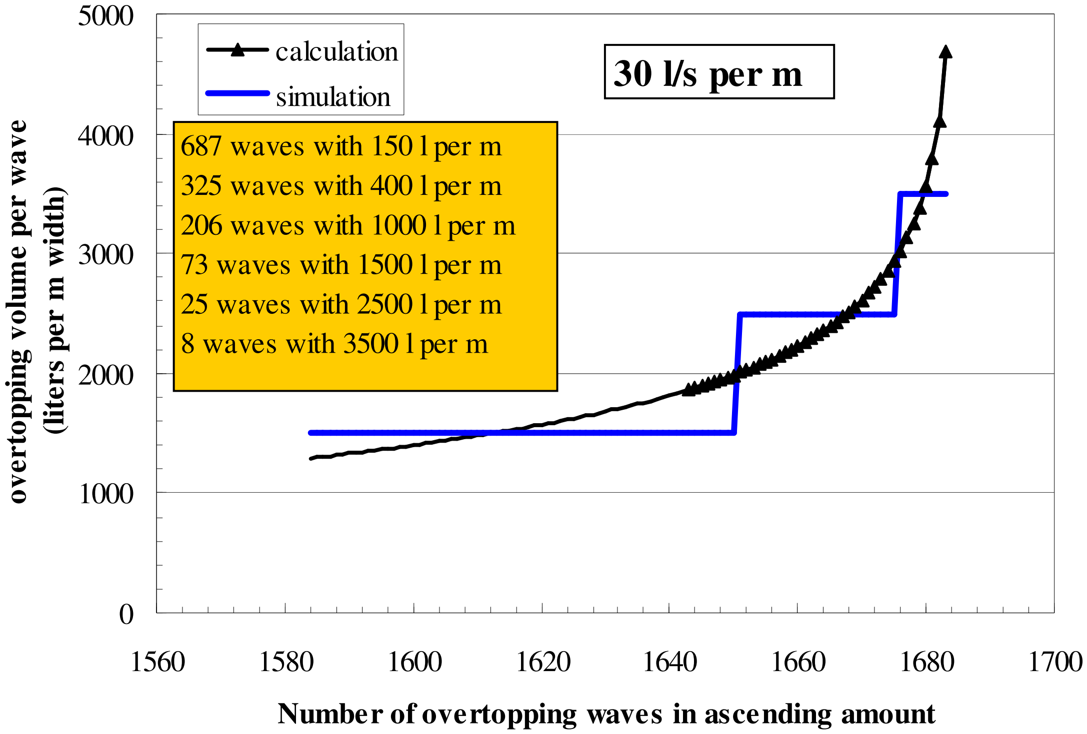


Figure 2.6. Calculated distribution of overtopping volumes and proposal for simulation. Mean discharge q = 30 l/s per m, distribution of largest volumes.

The size of the box of the overtopping simulator determines the maximum overtopping volume which can be generated. Based on the maximum volumes for 30 l/s per m and a transportable size of the simulator, the cross-section has been designed to a surface of 3.5 m^2, giving a maximum overtopping volume of 3.5 m^3 per m with. It is, however, well possible that a larger mean discharge than 30 l/s per m can be generated, may be up to 50 l/s per m, but the maximum overtopping volume exceeding the size of the box can then not be generated. But instead more volumes with the maximum contents of the box could be simulated.

2.3 Velocities and flow depth at the crest

The distribution of overtopping volumes, as described in section 2.2, gives for the overtopping simulator how often it should be filled with a certain volume. These volumes will then be released onto the crest and should give a behaviour close to reality. But what is this reality? The volume of overtopping water flows fast and in a limited time over the crest and inner side of the dike. For the overtopping simulator only the behaviour at the crest is important, as the behaviour on the inner slope will follow automatically. Recent information is available on velocities and flow depth at the crest of a dike. Schüttrumpf and Van Gent, 2003, give formulae for the *maximum* velocity and *maximum* flow depth on a location on the crest.

$$\frac{h_{c,2\%}}{h_{A,2\%(R_c)}} = \exp(-c^*_{c,h}\frac{x_c}{B}) \qquad (2.1)$$

$$\frac{u_{c,2\%}}{u_{A,2\%(R_c)}} = \exp(-c^*_{c,u} \ \frac{x_c . f}{h_{c,2\%}}) \tag{2.2}$$

where:

$h_{c,2\%}$	= flow depth at the crest exceeded by 2% of the incident waves
$h_{A,2\%(Rc)}$	= flow depth at outer slope, reached by 2% of the incident waves
$c^*_{c,h}$	= empirical coefficient
x_c	= location on the crest from the edge of the outer slope
B	= crest width
$u_{c,2\%}$	= velocity at the crest exceeded by 2% of the incident waves
$u_{A,2\%(Rc)}$	= velocity at the outer slope exceeded by 2% of the incident waves
$c^*_{c,u}$	= empirical coefficient
f	= friction coefficient (smooth slope f = 0.01)

The empirical coefficient $c^*_{c,h}$ was different for Schüttrumpf (2002) than for Van Gent (2002) being $c^*_{c,h}$ = 0.89 and 0.40, respectively. For the velocity the same value of $c^*_{c,u}$ = 0.5 was found.

The flow depth and velocity at the outer slope (and transition to the crest) are related to the 2%-run-up level and can be calculated by:

$$\frac{h_{A,2\%}}{H_s} = c^*_{A,h}\left(\frac{R_{u2\%} - R_c}{H_s}\right) \tag{2.3}$$

$$\frac{u_{A,2\%}}{\sqrt{gH_s}} = c^*_{A,u}\sqrt{\left(\frac{R_{u2\%} - R_c}{H_s}\right)} \tag{2.4}$$

where:

$R_{u2\%}$	= 2%-run-up level
R_c	= crest freeboard

The empirical coefficients $c^*_{A,h}$ and $c^*_{A,v}$ were established in two separate investigations: Schüttrumpf (2002) and Van Gent (2002). The coefficients found were $c^*_{A,h}$ = 0.33 and 0.15 and $c^*_{A,u}$ = 1.37 and 1.30, respectively. The coefficients for the maximum velocity are similar, but for the flow depth a factor more than 2 is present. For the time being both values for this coefficient will be used. For the maximum velocity an average value of $c^*_{A,u}$ = 1.33 will be used.

Above equations are only valid for 2%-values. It is possible to find maximum flow depths and velocities for each percentage, and therefore also for each overtopping volume, if the Rayleigh distribution is assumed for wave run-up. The 2%-run-up values can be calculated by TAW (2002) or PC-Overtopping. PC-Overtopping was used here, including the safety margin in the programme (the option “compare with measurements” was not used).

Assuming the Rayleigh distribution for run-up, the following equation gives the run-up level for each exceedance percentage, based on the 2%-run-up level.

$$R_{u,x} = \sqrt{\frac{\ln(P_{ovx})}{\ln(0.02)}} R_{u,2\%} \tag{2.5}$$

where:

$R_{u,x}$ = run-up level exceeded by probability x (between 0 and 1)
P_{ovx} = probability of exceedance x

With the distribution of overtopping volumes (Section 2.2) each overtopping volume for a specific overtopping discharge can be related to its corresponding run-up level $R_{u,x}$ with equation 2.5. This level $R_{u,x}$ should replace $R_{u2\%}$ in equations 2.3 and 2.4 in order to calculate the maximum flow depth and velocity at the "outer crest line", ie the transition from outer slope to the crest. Equations 2.1 and 2.2 can then be used to calculate flow depth and velocity at another location on the crest.

For example, a volume of 2000 l/m in an overtopping discharge of 10 l/s per m, gives a probability of exceedance of P_{ov} = 0.00087 (it is the 4-th largest overtopping wave during 6 hours = 4600 waves). This gives with equation 2.5 a run-up level of 5.37 m. The velocity at the outer crest line becomes 6.92 m/s (equation 2.4) and the flow depth according to Schüttrumpf (20002), equation 2.3, 0.91 m. The velocity and flow depth 3.5 m from the crest (equations 2.1 and 2.2) become 6.79 m/s and 0.37 m. It can be concluded that the maximum velocity decreases only a little over the crest, but the flow depth decreases drastically.

Similar calculations were performed for other overtopping volumes and other mean discharges. Figures 2.7 – 2.9 give the results. Figure 2.7 shows the maximum velocities for overtopping discharges of 1; 10 and 30 l/s per m at the outer crest line. All three mean overtopping discharges give similar velocities for the same volumes per wave overtopping. Also from theoretical reasoning one would expect this, as the same overtopping volume itself is more or less independent from the mean overtopping discharge. An overtopping event with 1000 l/m in a 1 l/s per m discharge should behave similar as a 1000 l/m event in a 30 l/s per m discharge. The difference is of course that the larger discharge will have more of these events, but the event itself should not be too different.

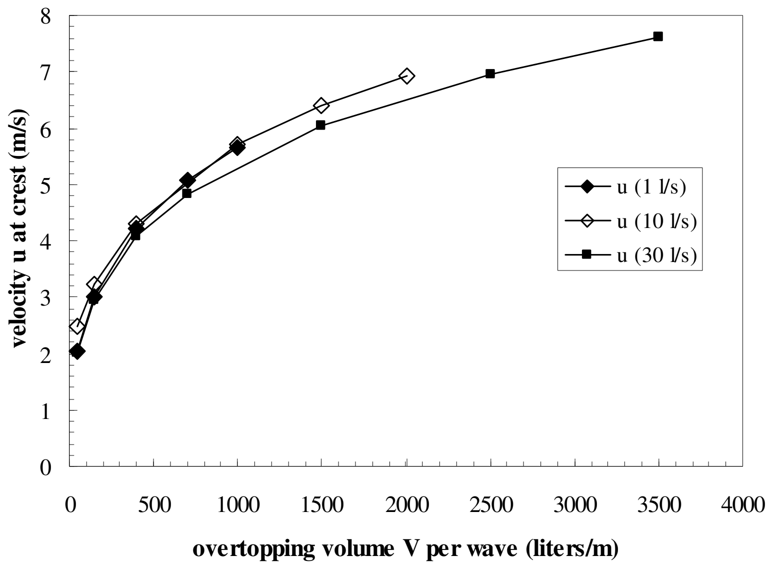


Figure 2.7. Maximum velocities at the outer crest line as a function of the overtopping volume per wave; H_s = 2 m, T_p =5.7 s, tanα = 0.25

Maximum velocities are about 2-2.5 m/s for small volumes of 50 l/m and they increase to velocities of about 5.5 m/s for 1000 l/m and up to a maximum of 7-7.5 m/s for an overtopping volume of 3500 l/m. Figure 2.8 gives the flow depth at the outer crest line, both for Schüttrumpf (2002) and Van Gent (2002). The S and V in the Figure give both authors. As mentioned earlier, the differences are more than a factor 2, which indicates somewhere an error in measurement or analysis. Finally, Figure 2.9 shows the difference in flow depth at the outer crest line and 3.5 m from this line on the crest, using the Van Gent (2002) coefficients.

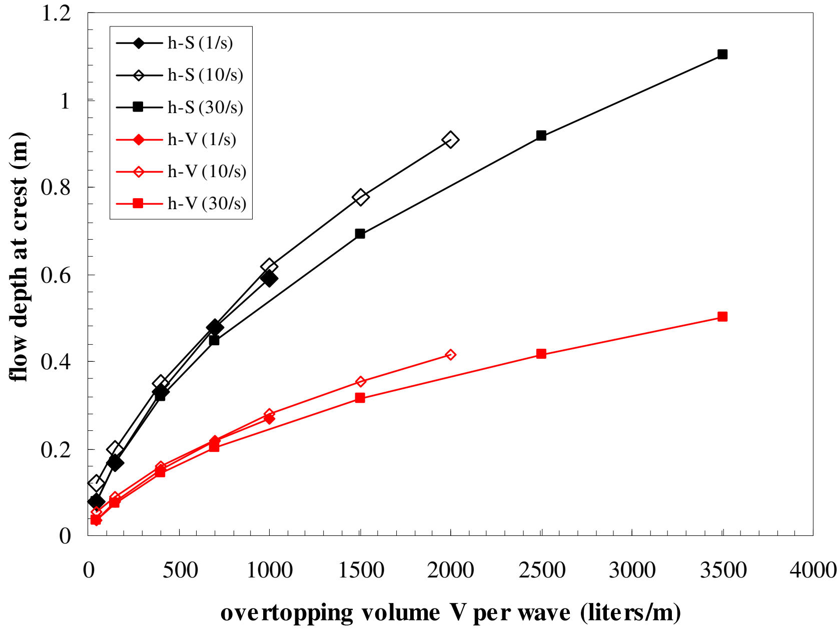


Figure 2.8. Maximum flow depths at outer crest line according to Schüttrumpf (2002) and Van Gent (2002), and applied for H_s = 2 m, T_p =5.7 s, tanα = 0.25 and various overtopping discharges

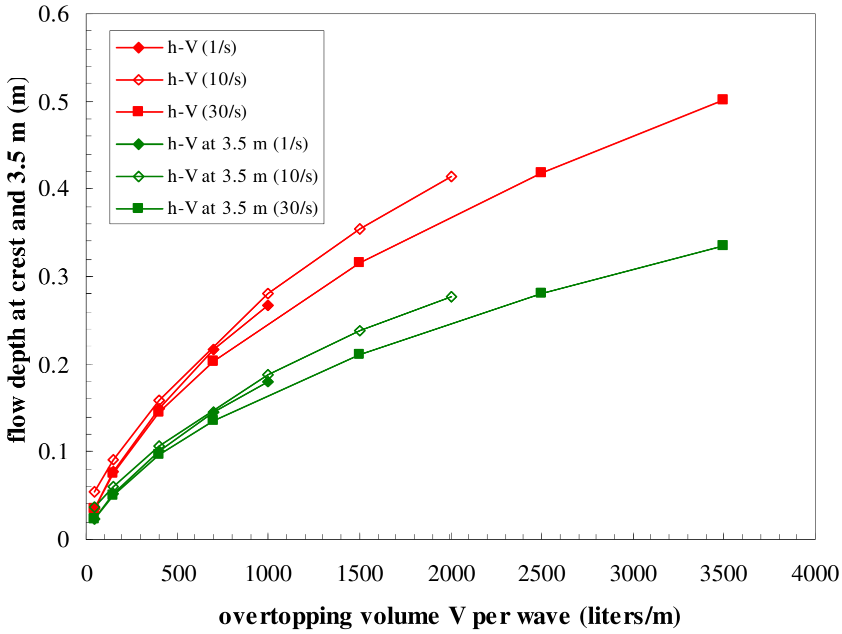


Figure 2.9. Maximum flow depth at the crest and at 3.5 m from the crest, according to Van Gent, 2003; applied for H_s = 2 m, T_p =5.7 s, tanα = 0.25 and various overtopping discharges

It is clear that the maximum flow depth is not a very reliable parameter. The only reason why the flow depth at the crest line is so much higher than a few meters further on the crest is probably that the velocity on the outer slope has an upward component, where the velocity further on the crest becomes horizontal. The wave “jumps” over the outer crest line.

Based on Figure 2.7 the following relationship between overtopping volume and maximum flow velocity can be assumed:

Overtopping volume (l/m)	max. flow velocity (m/s)
50	2.0-2.5
150	2.9-3.2
400	4.1-4.3
700	4.8-5.1
1500	6.2
2500	6.9
3500	7.6

3 Design of wave overtopping simulator

Overtopping waves are random in time and give different overtopping volumes per m per overtopping event. Given such an overtopping volume, the water flows with a certain velocity over the crest and inner slope of the dike, with a certain flow depth and in a certain time. The overtopping simulator should simulate the right velocity and flow depth in time at the crest of the dike, for a given overtopping volume. Overtopping volumes will vary between 50 l/m and 3500 l/m. Roughly the velocities will vary between 2 m/s up to around 8 m/s.

The simulator should be designed in such a way that it will simulate indeed the prescribed velocities and flow depths in time for a given overtopping volume. The following aspects have been considered for the technical design of the device:

- maximum volume: 3.5 m^3/m (4 m wide).
- shape of the box: as high and slender as possible, in order to reach the large velocities for large volumes
- opening of the valve: maximum 0.50 m, to be adjusted during calibration.
- possibility to place the device at different heights with respect to the crest: legs of 2 m length.
- shape of transition slope; to be adjusted during calibration

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:

$$u = \sqrt{2gx} \tag{3.1}$$

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

Overtopping volume (l/m)	**maximum flow velocity (m/s)**	**pressure height (m)**	**average width (m)**
50	2.0-2.5	0.27	0.19
150	2.9-3.2	0.49	0.31
400	4.1-4.3	0.90	0.44
700	4.8-5.1	1.27	0.55
1500	6.2	1.96	0.77
2500	6.9	2.43	1.03
3500	7.6	2.94	1.19

The average shape for each overtopping volume is given in Figure 3.1 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 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. 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 3.1 has been chosen for the design. The box can vary in height by adjustable legs. Figure 3.1 gives two examples at heights of 0.5 m and 1.0 m. Figure 3.2 gives the measures of the box of the overtopping simulator. A butterfly valve of 0.5 m width has been designed in the lowest horizontal part of the box. The box has been designed on four legs and is adjustable in height from 0.5 m to 2.0 above the crest.

The transition slope was made of steel and had dimensions of 2 m long, 1 m wide and 0.6 m high. The rear side had the shape of a quarter circle with a radius of 0.6 m. During the calibration the bottom was enlarged from 2 to 2.5 m, see the analysis of the tests.

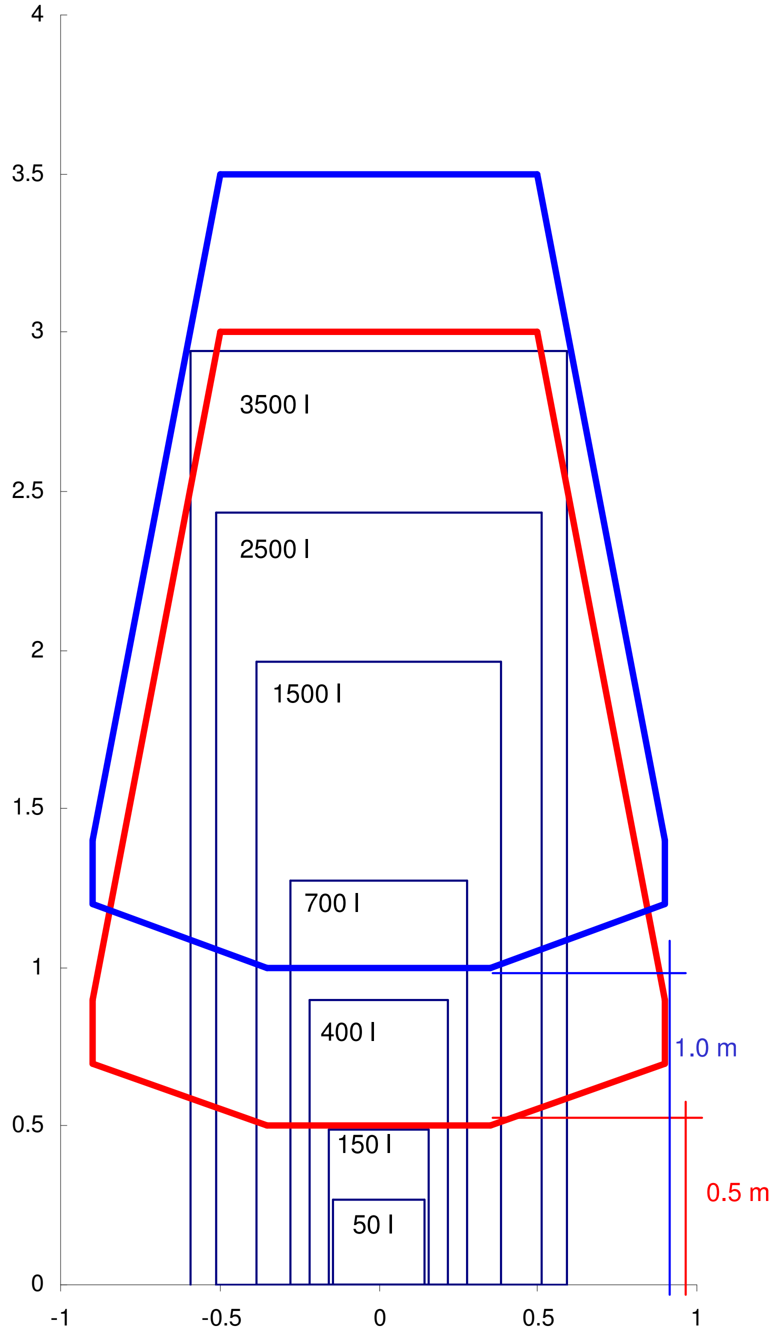


Figure 3.1. Theoretical cross-sections of box of wave overtopping simulator for each overtopping volume and final choice at two heights

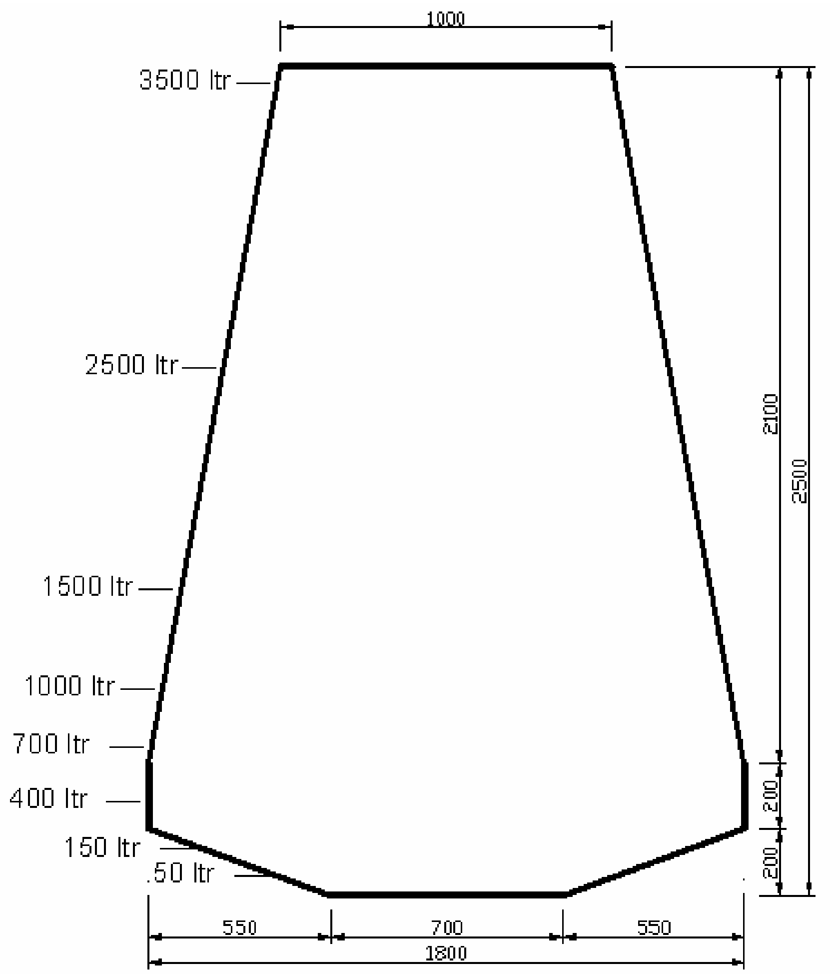


Figure 3.2. Cross-section of the overtopping box

4 Construction of prototype

A prototype of the wave overtopping simulator has been constructed by Nijholt Staal- & Machinebouw at Heerenveen, See Figure 4.1. The mechanical design has been made by G. van der Meer. The prototype is a full cross-section of the wave overtopping simulator, but only 1 m wide (the actual wave overtopping simulator would be 4 m wide). For calibration of the device this 1 m is enough. The prototype has been designed in such a way that it is fairly easy to modify parts of the structure if required during calibration (for example modifying the opening of the valve, modifying the transition slope or changing the height above the crest).

The calibration set-up was constructed by four students of the Noordelijke Hogeschool Leeuwarden with help of Nijholt. The location of the calibration was at a parking place of Nijholt, where water from a ditch is available and used water will disappear into a sewage system. See pictures 4.1 – 4.6 and Figure 4.2 for an overall view of the construction of the prototype and set-up of the calibration site.

Picture 4.1. Construction of prototype

Picture 4.2. Bottom of prototype with valve

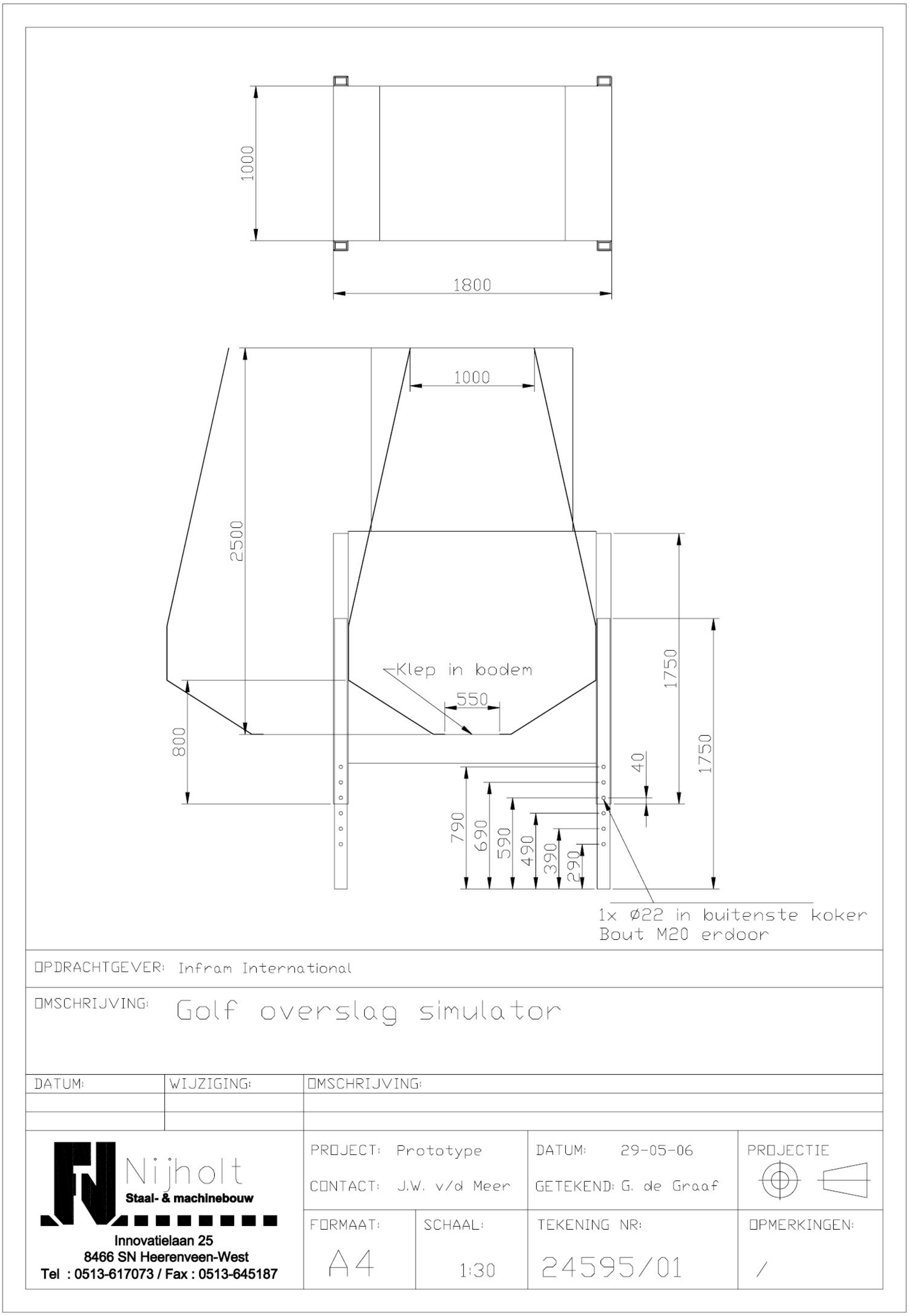
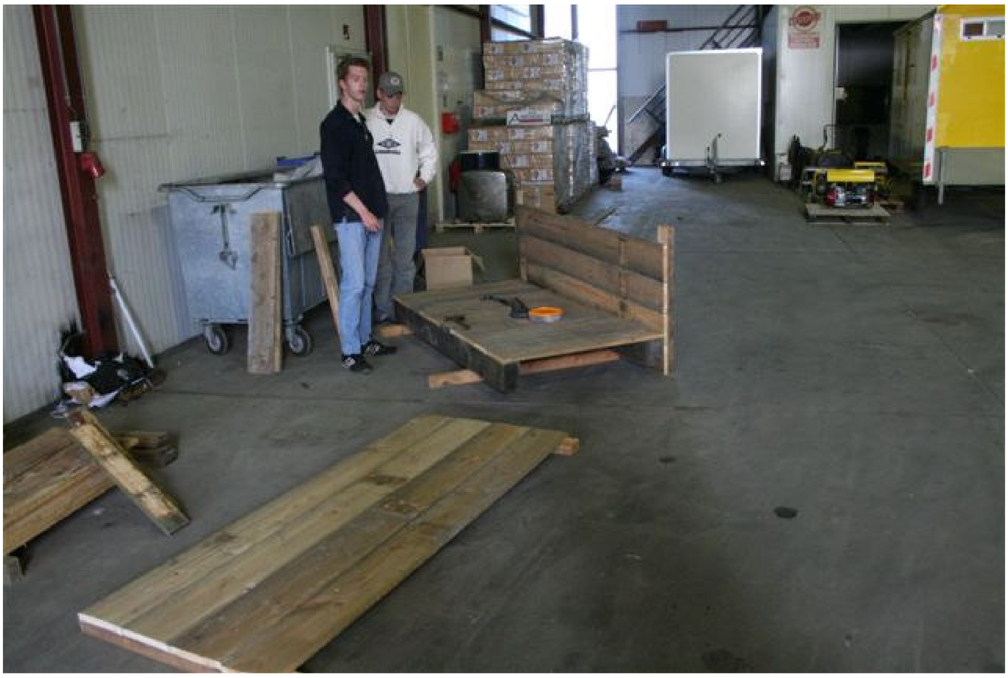


Figure 4.1. Working drawing for mechanical construction



Picture 4.3. Construction of 2 m long and 1 m wide calibration flume

Picture 4.4. Available pumps (3 and 7 l/s)

Picture 4.5. Set-up of calibration flume, surrounded by sand bags

Picture 4.6. Location of calibration site

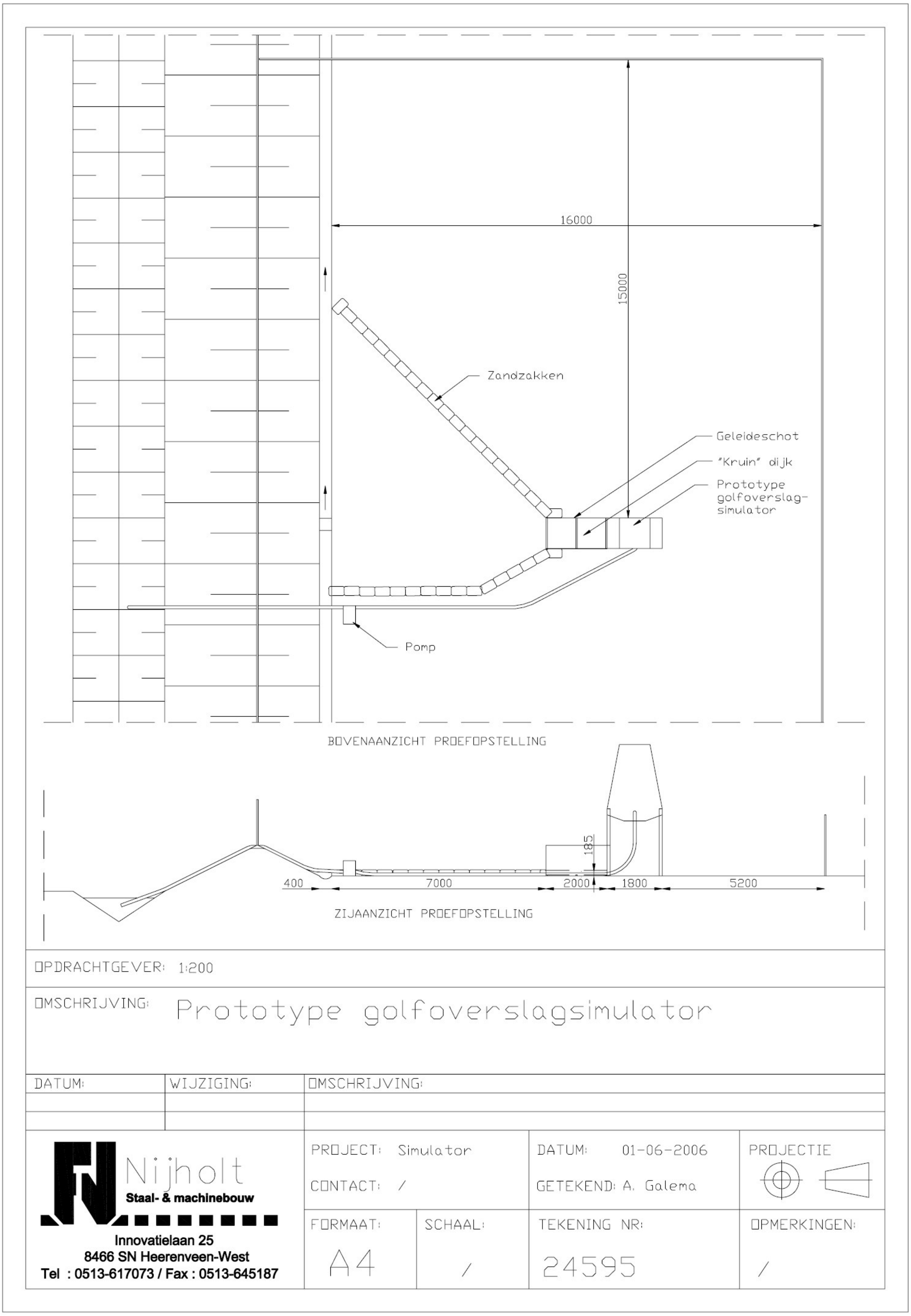


Figure 4.2. Set-up of calibration site.

5 Set-up of calibration

In Chapter 2 formulae are given for flow velocities and flow depths at the crest of a dike, given the overtopping volume in a wave. These are all *maximum* values. There is quite a difference in predicted flow depth between the investigations of Schüttrumpf and of Van Gent (2003). Predicted flow velocities are similar. This means that actually the only fairly well known parameter at the crest will be the maximum velocity.

Besides maximum flow velocities and flow depths, there is also the variation in time. Till now published research has not been focussed on this item. A sea state with a peak period of 5 seconds will give another time record of overtopping waves than a long period of 10 s, both with the same wave height. The wave period (or actually the wave steepness) has influence on the time record of overtopping and this might well be the reason for the small differences in flow velocity in Figure 2.7.

The prototype of the wave overtopping simulator must actually simulate the expected time record of flow velocity and flow depth and not only the maximum values. For this reason a small elaboration was made on existing research. Dr Schüttrumpf was asked to supply the raw test data of a few tests he performed with regular waves. The reason to choose tests with regular waves was that it is easier to measure the actual overtopping volumes per waves for regular waves than for random waves. For regular waves each overtopping wave should give the same overtopping volume and it is easy to average the discharge over a number of waves than to measure the overtopping volume for one wave only. The data with software for processing was submitted by Dr Schüttrumpf.

Four tests were selected from the available set, choosing tests with small to large overtopping and a wave period as close as possible to 5-6 s. Some of the tests, however, were only available with a longer period of 9.5 s. From each test the record for 3 waves was chosen (in order to show the similarity of the regular waves) and the flow velocity and flow depth in time were elaborated. Figures 5.1 – 5.4 show the records, starting with the largest overtopping.

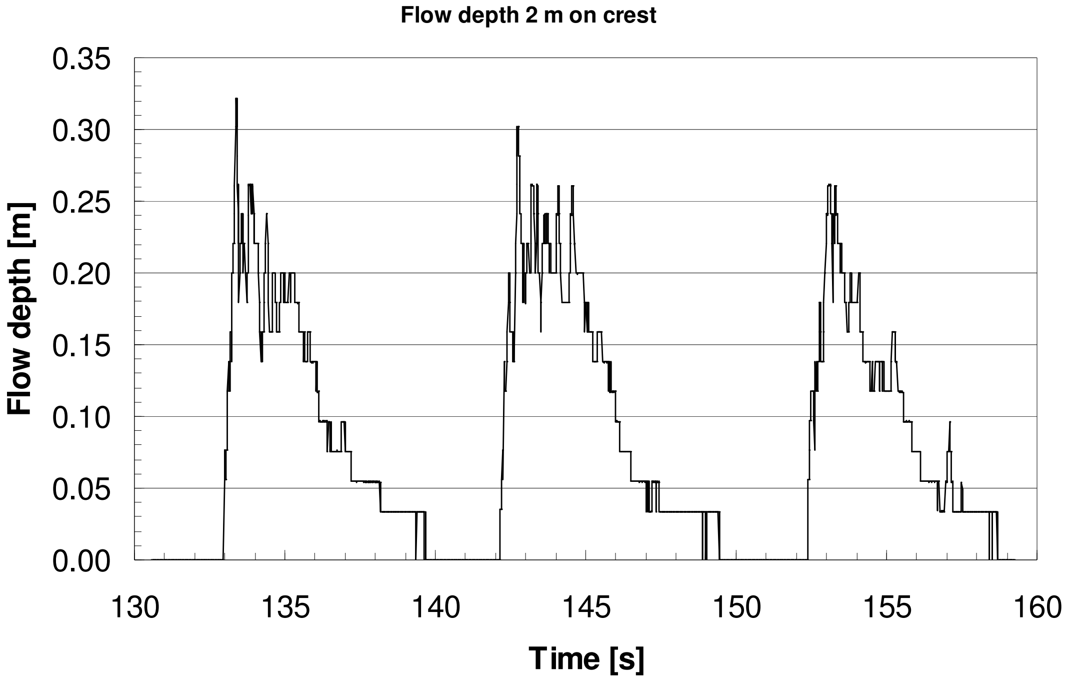
First of all the maxima as given by Schüttrumpf (see legend of figures) are not the true maxima. They are always a little lower. Further, the overtopping duration is longer for larger overtopping, but also longer for longer wave periods:

Test	wave period (s)	mean discharge (l/s/m)	overtopping duration (s)
31050010	9.5	60.0	6.5
31050011	9.5	18.3	5.5
13060010	6.0	0.75	2.4
13060011	5.0	0.50	< 2

Certainly for the largest overtopping the records have more or less a triangular shape. The flow velocity and flow depth in time give also a good idea of the overtopping volume, just by integration. A quick way is to assume a triangular shape and calculate the overtopping volume by:

$V = 1/3\ v_{max}\ d_{max}\ t_1$,
where t_1 = the total time water flows over the crest at a certain location. The four selected tests give the following figures:

Test	v_{max} (m/s)	d_{mx} (m)	t_1 (s)	$V_{calculated}$ (l/m)	$V_{measured}$ (l/m)
31050010	2.7	0.25	6.5	1460	570
31050011	2.1	0.13	5.5	500	174
13060010	1.4	0.06	2.4	67	4.5
13060011	1.2	0.05	2	40	2.5



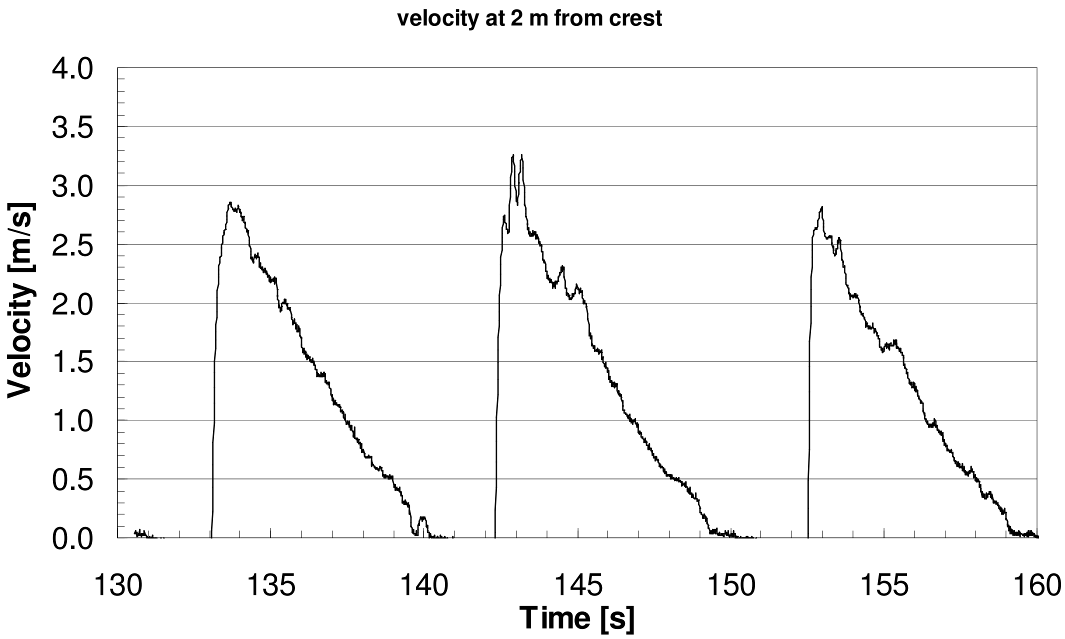
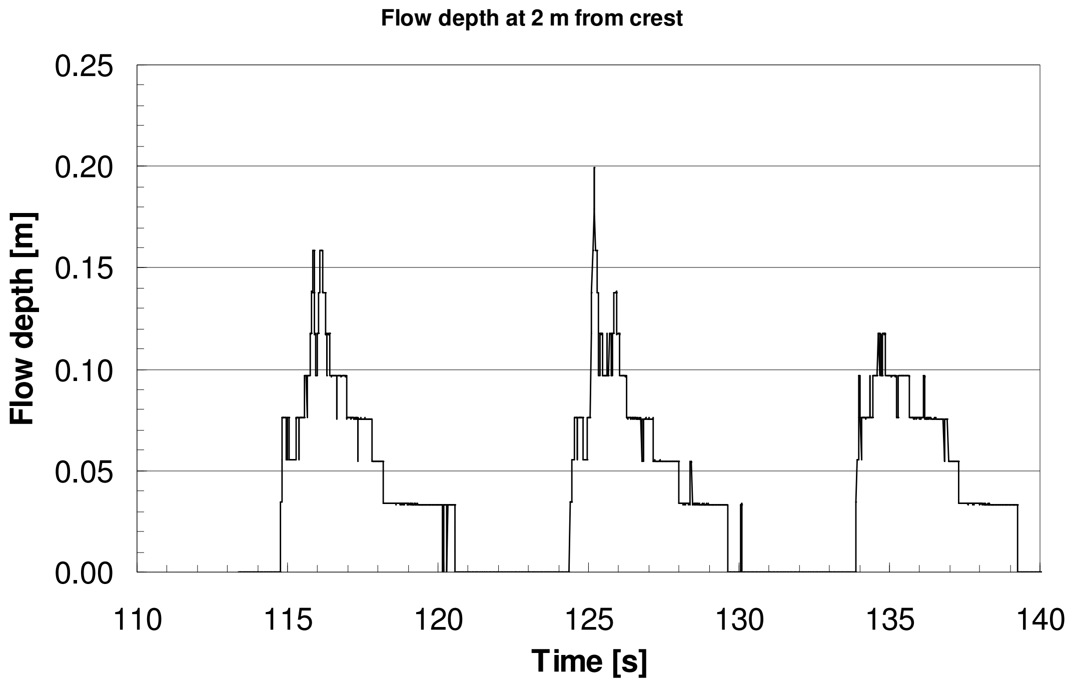


Figure 5.1. Time records of flow depth and flow velocity at 2 m from the crest. Tests by Schüttrumpf. Regular waves. Test 31050010. H = 0.87 m, T = 9.5 s. Measured overtopping discharge was 60.0 l/s per m and maximum flow depth 0.23 m

The difference between calculation from the time record and directly measured overtopping is about a factor 3 for the large overtopping and even much more for the smallest ones! This may be a reason for the large difference in flow depth (about a factor 2) between Schüttrumpf and Van Gent. If flow depths in reality are only half of the flow depths measured by Schüttrumpf, then calculated and measured volumes come much closer. But what can have caused this difference?



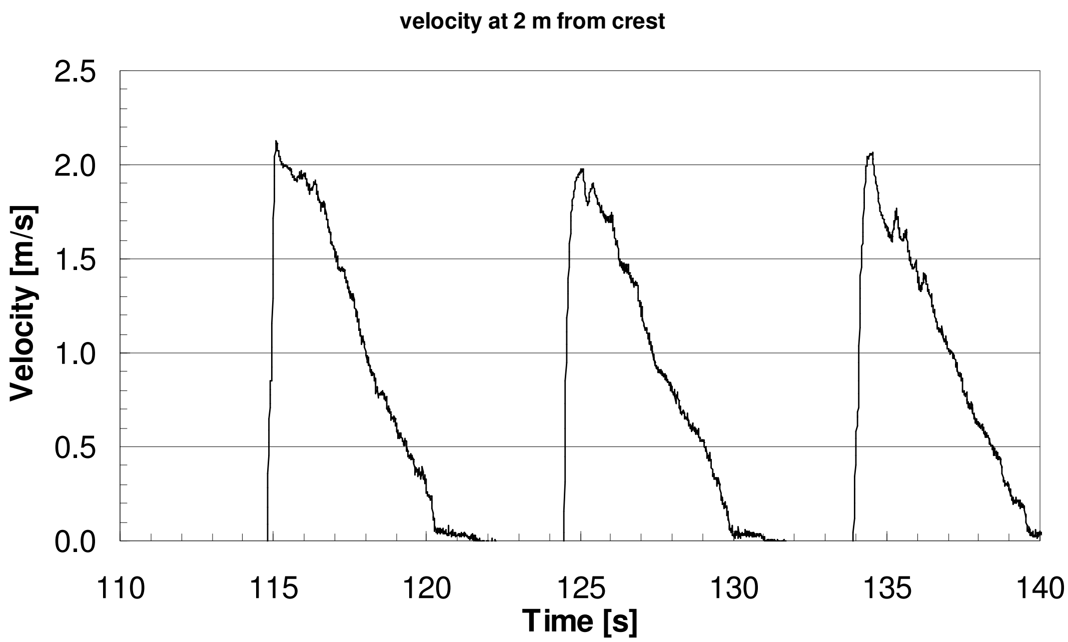
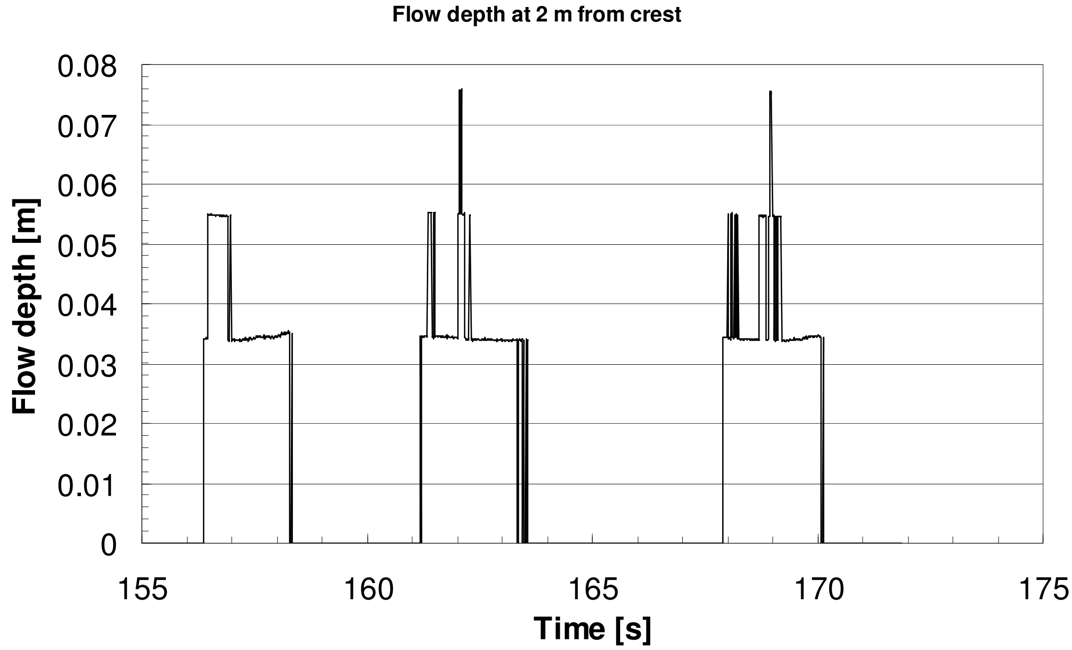


Figure 5.2. Time records of flow depth and flow velocity at 2 m from the crest. Tests by Schüttrumpf. Regular waves. Test 31050011. H = 0.62m, T = 9.5 s. Measured overtopping discharge was 18.3 l/s per m and flow depth 0.10 m

Only contact with Schüttrumph and with Van Gent and more elaboration on this matter can solve this question. The firm conclusion is that measured records of flow velocity and flow depth by Schüttrumpf do not match the direct measurements of overtopping volume or discharge. A tentative conclusion might be that maybe something was wrong with the measurements of flow depth and that the predictions of Van Gent are closer to reality. From this it has been strongly suggested to spend more time on this issue and to solve it. This has indeed been done at a later stage of the project (see Chapter 10).



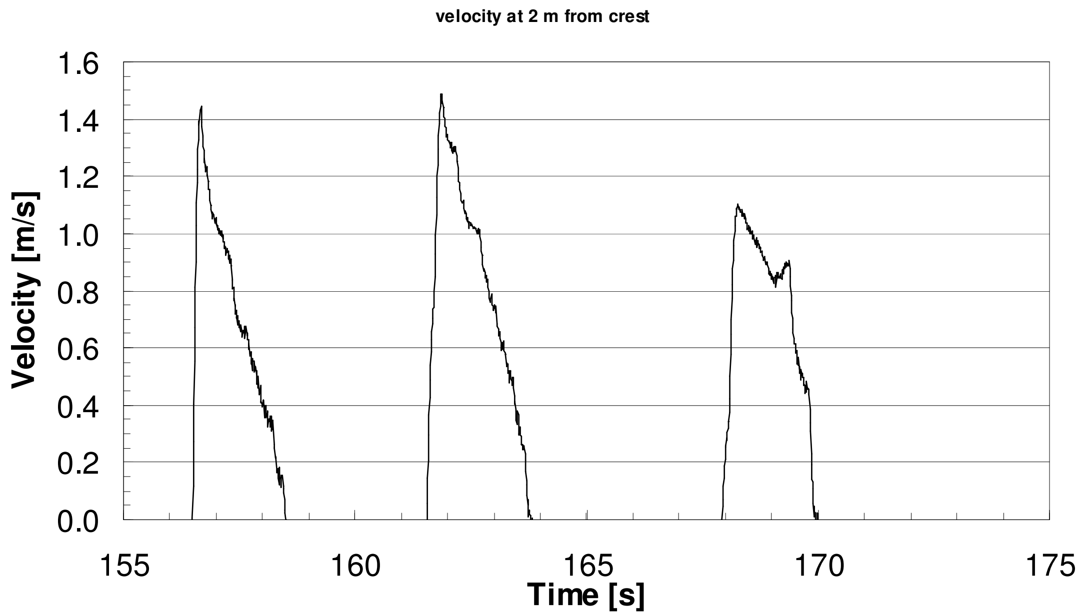
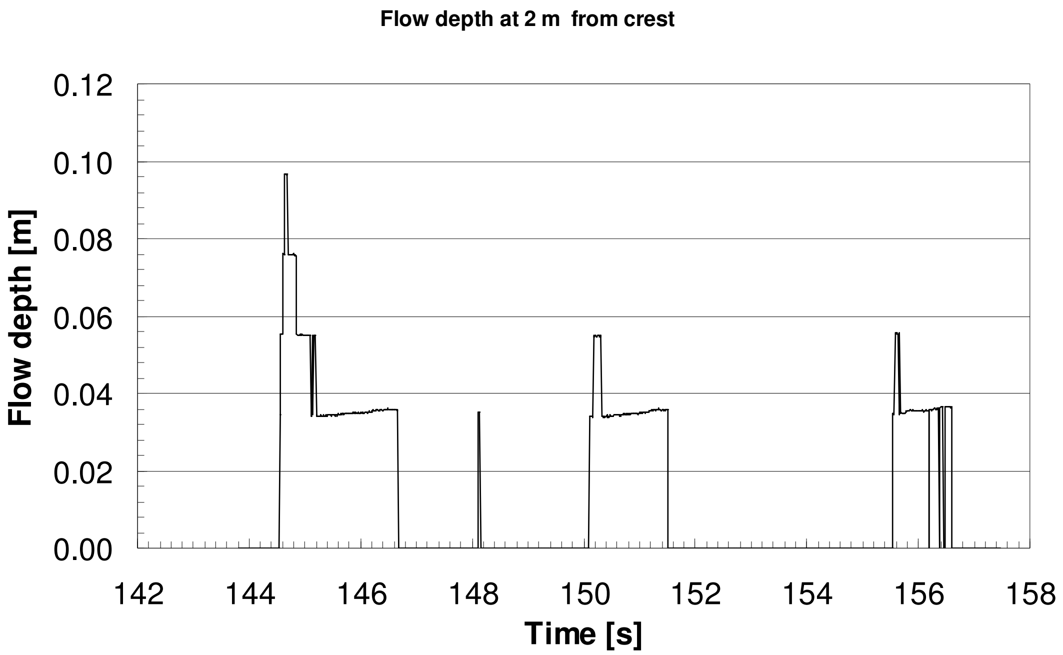


Figure 5.3. Time records of flow depth and flow velocity at 2 m from the crest. Tests by Schüttrumpf. Regular waves. Test 13060010. H = 0.98 m, T = 6.0 s. Measured overtopping discharge was 0.75 l/s per m and flow depth 0.03 m

It is fairly essential for the calibration of the prototype of the wave overtopping simulator to solve the problem and to have correct predictions (but it is outside the scope of this project). For the time being the calibration will be based mainly on the flow velocity and only partly on flow depth (taking predictions by Van Gent) and assuming maximum flow times of about 0.5 – 0.8 T_p for larger overtopping volumes (flow times around 3-5 s) and 0.3 – 0.5 T_p for smaller overtopping volumes (flow times around 2-3 s).



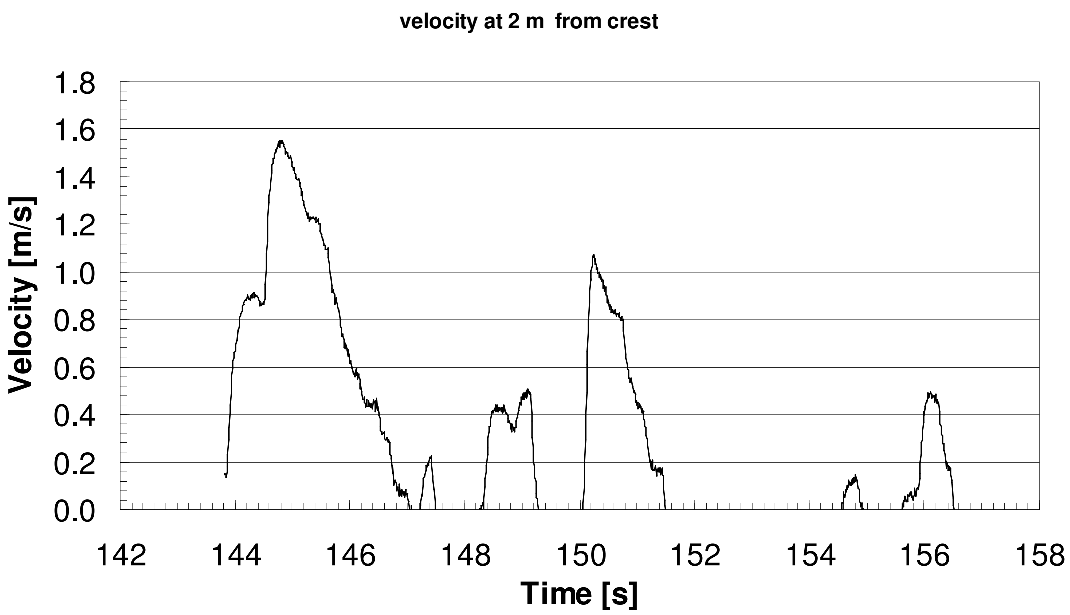


Figure 5.4. Time records of flow depth and flow velocity at 2 m from the crest. Tests by Schüttrumpf. Regular waves. Test 13060011. H = 1.28 m, T = 5.0 s. Measured overtopping discharge was 0.5 l/s per m and flow depth 0.03 m

Volumes to be simulated by the wave overtopping simulator will vary between about 50 l per m and maximum 3500 l per m. Based on Figure 2.7 with the predicted overtopping flow velocities the following targets were set for the simulator to be met, where model uncertainty of the prediction has been taken into account:

Overtopping volume (l/m)	flow velocity (m/s)	range (m/s)	flow time t_1 (s)
50	2.0-2.5	1.5 – 3.0	1.5 – 2.5
150	2.9-3.2	2.5 – 3.5	1.5 – 2.5
400	4.1-4.3	3.5 – 5.0	2.0 – 3.0
700	4.8-5.1	4.2 – 5.7	2.5 – 3.5
1000	5.7	5.0 – 6.5	3.0 – 4.0
1500	6.2	5.5 – 7.0	3.0 – 4.0
2500	6.9	6.0 – 8.0	3.5 – 5.0
3500	7.6	6.5 – 8.5	3.5 – 5.0

The simulator was calibrated at Nijhof Staal- & Machinebouw. An empty part of the parking place was used which was situated along a ditch with clean water, see Figure 4.2. The dike crest was simulated by a 2 m long and 1 m wide flume with a height of 0.6 m. After the 2 m the water could flow freely to the sewage system. Two pumps were available, one of 3 l/s and one of 8 l/s. Both could be used at the same time to fill up the simulator to a required volume. See Pictures 5.1 and 5.2 for an overall view.

Picture 5.1. Overall view of calibration set-up

Picture 5.2. Rear side of wave overtopping simulator

6 Calibration

6.1 Description of test series

After construction of the prototype of the wave overtopping simulator and after set-up of the test site the prototype could be calibrated. The objective of the calibration was to modify the prototype, including the transition, in such a way that each volume would generate the right flow velocity and flow time, as given in Chapter 5. The set-up height (difference between bottom box and crest) started with 1.25 m. The first volume to be tested was V = 1000 l. Then smaller volumes were tested with the same set-up height and subsequently larger volumes. The maximum set-up height of 2 m was tested and subsequently the minimum height of 0.5 m.

Possible modifications were the speed to open the valve, the final opening width of the valve, w, the size of the valve (maximum as constructed was 0.5 m), the shape of the transition slope, l and α, roughness on the transition slope and other set-up heights of the simulator. Figure 6.1 gives an overall view of the parameters that have been changed.

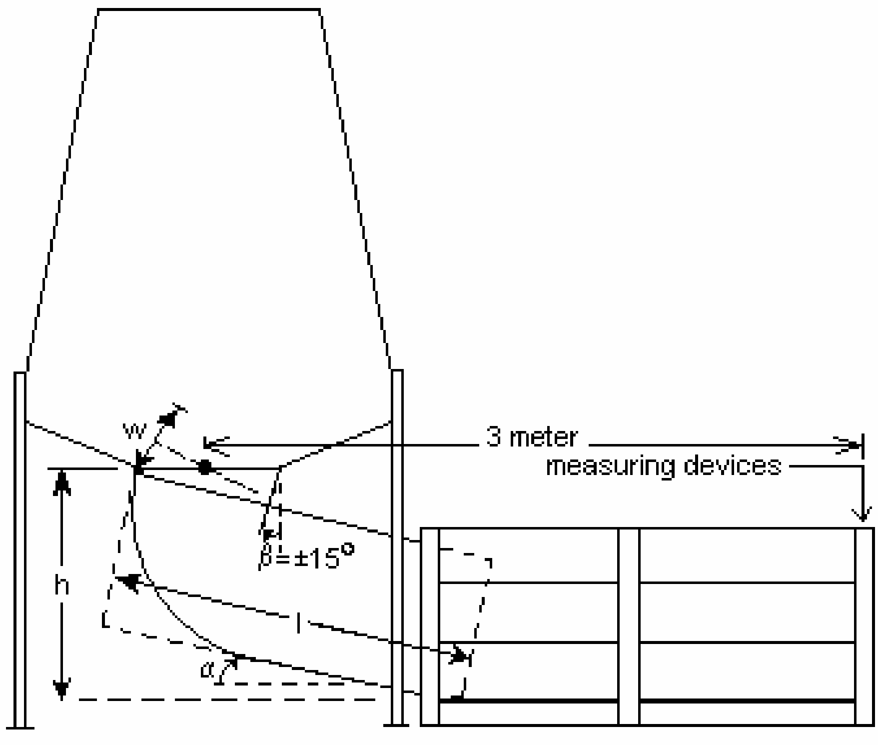


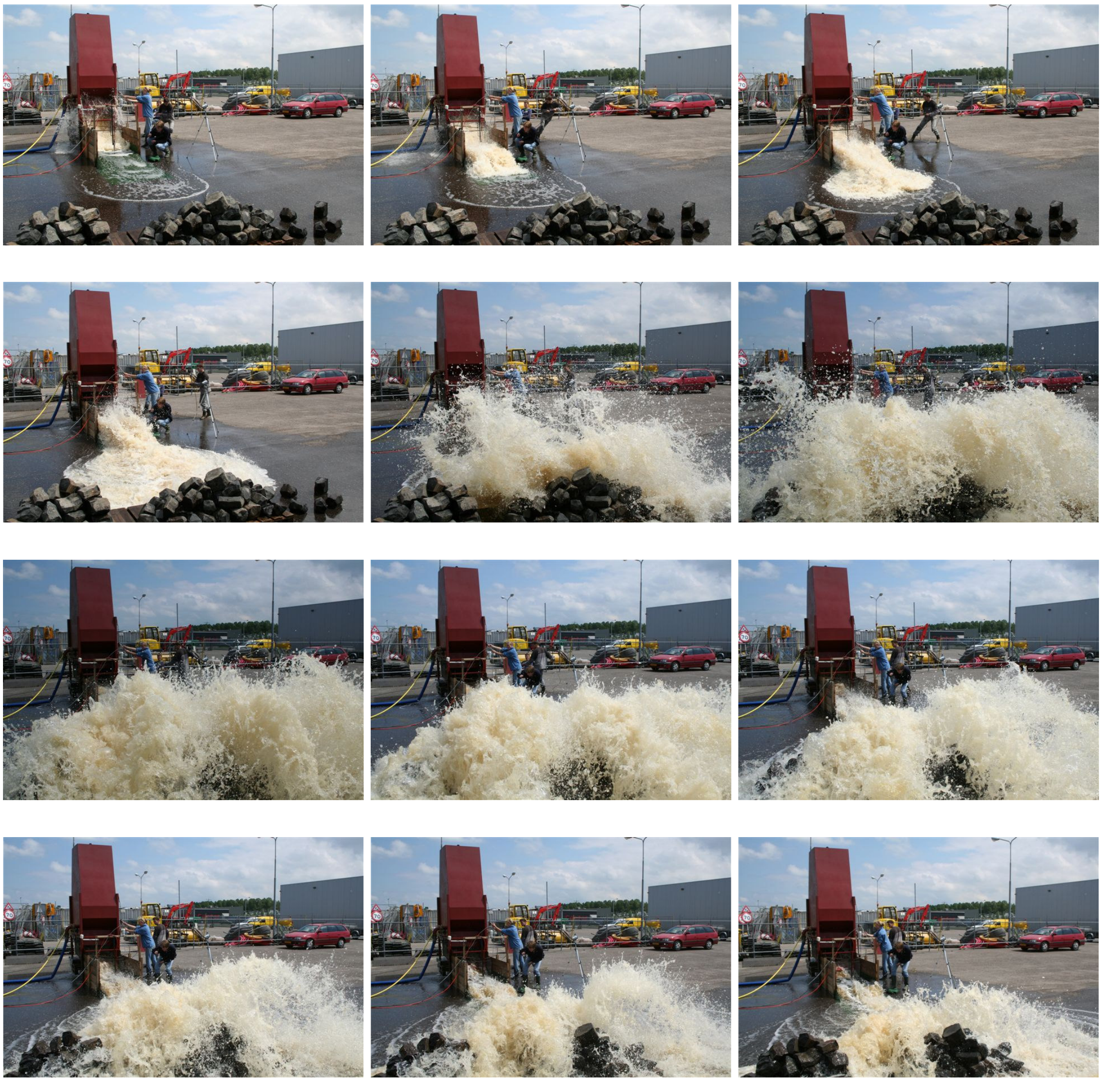
Figure 6.1. Set-up of prototype with measures of parameters changed during calibration

A series of tests consisted of various measurements with different volumes in the box, but with only one unique set-up as given in Figure 6.1. Test series were named C – K (test series A and B being trials which are not reported here). Each test in a series got the extension of the volume in liters in the box and a test number. For example test C1000-2 is a test in series C with 1000 l in the box and test number 2 in the series. An overall view of each test series is given in Table 6.1.

Pictures 6.1 – 6.4 give snapshots of a few tests, both for the very large volumes and the small volumes.

Table 6.1. Overall view of test series

Test series	Height of valve ***h*** w.r.t. crest (m)	Set-up of transition		Opening valve ***w*** (m), with maximum 0,50 m
		Angle α w.r.t. crest (º)	Length ***l*** transition (m)	
C	1,25	10	2,00	0,34
D	1,25	23	2,00	0,42
E	0,84	13	2,00	0,42
G	0,60	7	2,00	0,42
H	0,60	27	2,00	0,42
I	1,83	30	2,50	0,50
J	1,83	30	2,50	0,50
K	1,25	19	2,50	0,50



Picture 6.1. Front view of test with 3500 l

Picture 6.2. Side view of test with 3500 l

Picture 6.3. Front view of test with 150 l

Picture 6.4. Front view of test with 50 l

6.2 Measurements with the instrumentation

An electro magnetic velocity meter (EMS) was rented from WL | Delft Hydraulics, together with an acoustic depth meter (LDM). The EMS was placed 2 cm above the bottom of the flume (or dike crest) and 1.5 -2 m from the end of the transition slope of the simulator. The depth meter was placed about 1.2 m above the bottom of the flume at the same location. A meter scale was placed at the wall of the flume. See pictures 6.5 – 6.8 for an impression.

After some trials with the overtopping simulator and first measurements some doubts arose about the correctness of the measurements of the flow depth. Therefore the instruments were calibrated. A small flume was made 0.07 m wide and the largest pump was guided to this flume (about 8 l/s). The flow depth was measured manually and amounted to 0.05 m. The EMS measured a velocity of 2.0 m/s +/- 0.1 m/s. This means a discharge of 0.07*0.05*2.0*1000 = 7 l/s. This is very close to the discharge of the pump. It was concluded that the EMS worked well for this condition.

Picture 6.5. Instruments, away from prototype

Picture 6.6. Instruments, towards prototype

Picture 6.7. Acoustic flow depth meter

Picture 6.8. Electromagnetic velocity meter EMS

The depth meter measures the surface with an acoustic signal which reflects back to the device. Knowing the distance to the bottom of the flume (a constant signal if no water was running), the flow depth was determined if the measured signal was subtracted from this constant distance. Both pumps were directly guided to the flume and the flume was made narrower at the location of the LDM. The flow was laminar with no air entrainment and amounted to 4.0 – 4.5 cm measured manually. The measured flow depth by the LDM was 4.0 – 4.2 cm. It was concluded that for small flow depths without air entrainment the flow depth was measured correctly.

Then flow simulations were performed and both manually and digitally velocities and flow depth were measured. Tests C1000 had water volumes of 1000 l and the simulations were repeated 6 times. Figure 6.2 shows the records of velocity and flow depth of tests C1000-1. The actual simulation takes place from about t = 1 s to 5 or 6 s. After that flow depths are almost zero and the EMS gives spikes as it is not submerged anymore. The maximum velocity measured is 5.3 m/s. The maximum flow depth measured is at least 0.2 m and sometimes more than 0.3 m. The maximum flow depth was also recorded manually at the side wall of the flume at the location of the flow depth meter. In all tests with 1000 l the maximum flow depth was 0.12 – 0.13 m. By no means flow depths of 0.2 or 0.3 m were seen. The flow is highly turbulent with a lot of air entrainment and the surface is not smooth. At the end of the simulation the flow has less air and is less turbulent. It was concluded that the flow depth meter can not measure correctly the flow depth when the flow is so turbulent. Probably the part after 4.5 s was measured correctly.

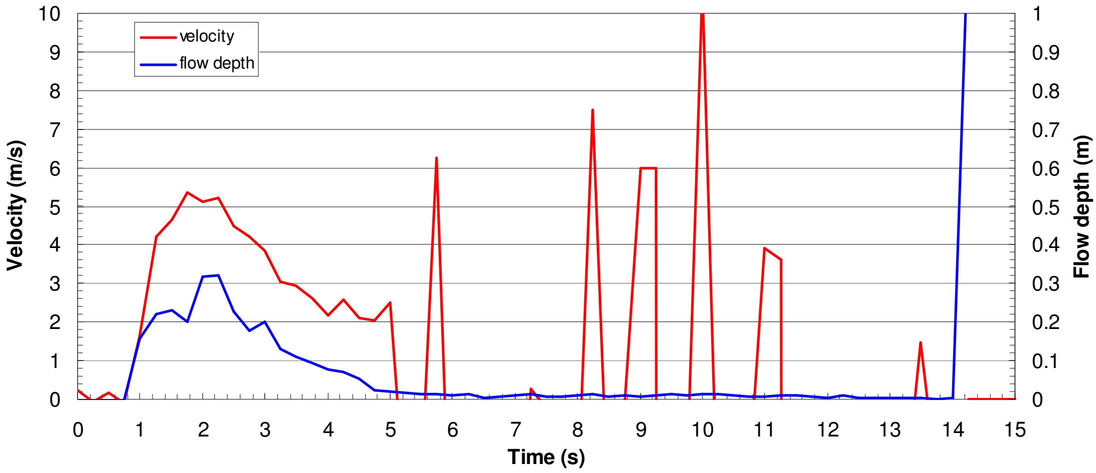


Figure 6.2. Records for test C1000-1

After 5 s the velocity drops suddenly from about 2 m/s to zero. This is the time when the EMS is not longer submerged and the flow depth becomes smaller than about 2-3 cm. The front velocity of the flow was also measured manually. The time was recorded between the point where the water passed the end of the transition slope and a point 3.5 m further. Measured times were 0.6-0.7 s, giving velocities of this front of 5 – 5.8 m/s. The maximum velocity in Figure 6.2 is 5.3 m/s and this is well in the manually measured range. It was concluded again that the EMS was able to measure the velocities correctly.

Another check was done on the measurements. For each of the six C1000 tests the records of velocity and flow depth were multiplied and integrated over time. This was done only over the relevant time period, from start of the flow to the drop of the velocity where the flow depth becomes very small. In Figure 6.2 this was from t = 0.75 – 5.0 s. This integration gives a calculation of the total volume. Calculated volumes for the six tests were: 2.67, 2.17, 2.44, 2.52, 2.2 and 2.18 m^3, where the actual volume in the simulator was 1 m^3. This check shows that the flow depth was measured at least 2 times larger than the actual flow depth. It was concluded that the flow depth could not be measured correctly, but the signal was useful to determine the flow time t_1 in combination with the velocity record.

Another small disadvantage was that the frequency of the measurements was limited to 4 Hz (four measurements per second). But it was concluded that within the total scatter of the measurements and repetition of tests this was acceptable. For the measurements on the dike, however, it is proposed to use a sampling frequency of at least 10 Hz. A final check was made with help of Figure 6.3. The shape of the records are more or less triangular, which means that the following equation can be applied:

$$V = 1/3\ v_{max}\ d_{mas}\ t_1 \qquad (6.1)$$

With V = 1 m^3, v_{max} = 5.3 m/s and t_1 = 4.5 s the maximum flow depth becomes d_{max} = 0.125 m. This is similar to the flow depth of 0.12 – 0.13 m, which was measured manually.

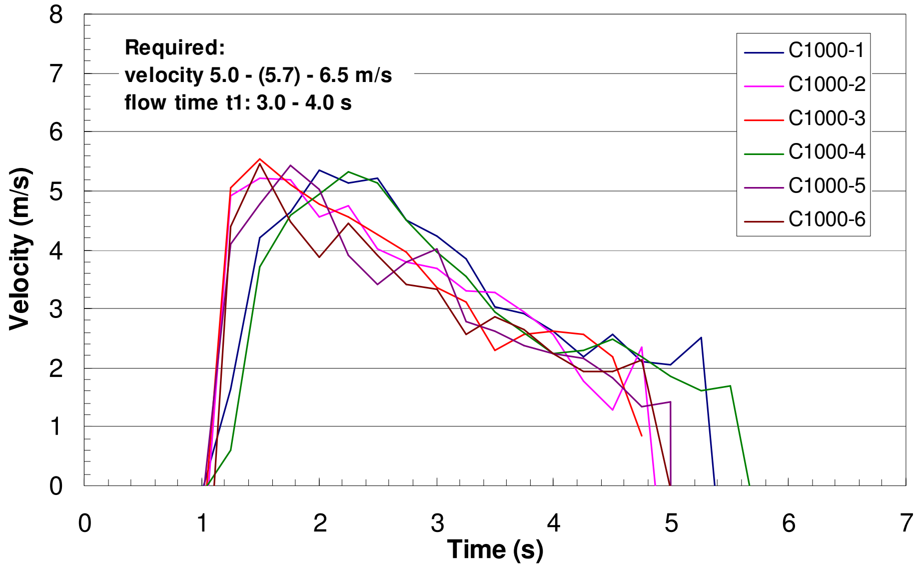


Figure 6.3. Velocity measurements for C1000 (V = 1 m^3)

The relevant parts of the velocity records of tests C1000 were placed in one graph, starting at t = 1 s. This graph is shown in Figure 6.3. The records are quite similar, which means that the flow simulation can be repeated quite well. The required velocity and flow time (see Chapter 5) are mentioned too. The recorded velocity of around 5.7 m/s is well in the required range of 5.0 – 6.5 m/s. The flow time is between 3.75 and 4.75 s, which is slightly longer than the required flow times of 3.0 – 4.0 s. It was concluded that with the height of the simulator of 1.25 m this volume gave more or less the flow simulation that was required, except that the flow time was a little too long.

After this first successful series of calibration tests C it was concluded that:

- The flow depth can not be measured correctly, but the signal is helpful to determine the flow time t_1 in combination with the velocity record.
- The maximum flow depth should be recorded manually.
- The simulator height of 1.25 m gives indeed good simulations for a volume of 1 m^3.
- Flow times in general are too large or a little too large with this set-up C.

6.3 Analysis of calibration tests

An overall view of the test series was given in Table 6.1. From the beginning of the testing it became clear that water under high pressure (certainly with the large volumes) will spread as soon as the valve opens. Side skirts were placed along the transition slope to keep the water within the limits of 1 m wide. From the first measurements C it became also clear that the valve should open as quickly as possible in order to decrease the flow time. A manual system was invented with pulleys and ropes, which made it possible to close the valve quickly and well (in order to prevent leakage) and to open it very quickly. This system worked well, but the real simulator should of course have a hydraulic system to open and close the valve.
Also various openings of the valve were tested, but at the end the conclusion was that the valve should always open to its full width of 0.5 m.

All measurements performed have been tabled in Appendix 1 and the composed velocity measurements have been given in Appendix 2. The maximum flow velocity and the flow time were taken from the measurements and given in Appendix 1. The flow depth in this appendix was measured manually. Finally, the volume calculated with equation 6.1 was given and this figure should be more or less close to the actual volume in the simulator. Appendix 1 gives for each test series with similar volume also the average values. These average values have been accumulated in Table 6.2, which gives a good overall view of all the results of the calibration.

The left half of Table 6.2 gives the set-up of the test series, see also Section 6.1. Then the volume in the simulator is given with next the calculated volume from the records. The next columns give the measured maximum flow velocity with the required range. The maximum flow depth, measured manually, is given then and in the last columns the measured flow time compared with the estimated range.

Based on Table 6.2 a few observations can be made, which were also noticed during calibration. Test series C gave too large flow times and also the velocities were too high for the small volumes. Test series D gave smaller flow times, but still the flow velocity was too high for 150 l and for the large volumes of 2500 l and 3500 l the flow was very turbulent and more or less uncontrolled, which gave a deviation in actual and calculated volume in the simulator.

Test series E gave too large flow times. Test series G and H were performed with a low set-up level of the box. Surprisingly the flow velocities for the small volumes were too high. For the larger volumes the flow was highly turbulent giving large differences between actual and calculated volumes.

Test series I and J had a large set-up height. The height induced a steep and long transition slope and the simulation became less controlled. In test series I for example the volumes of 2500 l gave in average larger velocities, but still shorter flow times, than the larger volumes 3500 l.

For the final test series it was decided to return to the set-up height of 1.25 m and to guide the flow leaving the valve as good as possible by a skirt in front of the valve. The transition slope was also a little longer than in the first series and the angle with the crest was 19 degrees. This final test series will be described more in detail. Figures 6.4 – 6. 11 give all composed velocity measurements.

Table 6.2. Overall view of test results. Values are averaged values of a certain number of tests, see for details Appendix 1 and 2.

Test series	Height of valve above crest h (m)	Angle α of transitions slope (º)	Length of transition slope (m)	Maximum opening of valve w (m)	Volume V in simulator [liter]	Volume V (liter) according to 1/3*v*d*t	Max. velocity v [m/sec]	Max. velocity $v_{required}$ [m/s]	Max. flow depth [m]	Flow time t_1 [s]	Flow time $t_{1;estimated}$ [s]
					150	160	4.48	2.5 – 3.5	0.04	2.70	1.5 – 2.5
					400	350	5.75	3.5 – 5.0	0.06	3.16	2.0 – 3.0
					1000	910	5.40	5.0 – 6.5	0.12	4.13	3.0 – 4.0
					2500	3360	6.40	6.0 – 8.0	0.26	5.96	3.5 – 5.0
C	1.25	10	2	0.34	3500	5080	7.53	6.5 – 8.5	0.30	6.80	3.5 – 5.0
					150	110	4.26	2.5 – 3.5	0.04	1.76	1.5 – 2.5
					400	430	4.65	3.5 – 5.0	0.10	2.88	2.0 – 3.0
					700	690	5.68	4.2 – 5.7	0.13	2.86	2.5 – 3.5
					1000	990	5.18	5.0 – 6.5	0.16	3.58	3.0 – 4.0
					1500	1560	6.20	5.5 – 7.0	0.19	4.04	3.0 – 4.0
					2500	3510	7.17	6.0 – 8.0	0.26	5.57	3.5 – 5.0
D	1.25	23	2	0.42	3500	5320	7.52	6.5 – 8.5	0.42	5.06	3.5 – 5.0
					400	630	3.28	3.5 – 5.0	0.15	3.96	2.0 – 3.0
					700	1310	4.08	4.2 – 5.7	0.15	6.30	2.5 – 3.5
E	0.84	13	2	0.42	1000	1510	4.08	5.0 – 6.5	0.21	5.34	3.0 – 4.0
					50	50	5.40	1.5 – 3.0	0.03	0.89	1.5 – 2.5
					150	170	4.93	2.5 – 3.5	0.05	1.92	1.5 – 2.5
					400	730	4.20	3.5 – 5.0	0.13	4.20	2.0 – 3.0
G	0.6	7	2	0.42	700	840	4.20	4.2 – 5.7	0.16	3.72	2.5 – 3.5
					700	1330	5.04	4.2 – 5.7	0.23	3.52	2.5 – 3.5
					1000	1650	5.48	5.0 – 6.5	0.28	3.24	3.0 – 4.0
					2500	3920	6.97	6.0 – 8.0	0.36	4.70	3.5 – 5.0
H	0.6	27	2	0.42	3500	5760	8.24	6.5 – 8.5	0.40	5.22	3.5 – 5.0
					400	400	5.54	3.5 – 5.0	0.07	3.16	2.0 – 3.0
					1000	820	5.34	5.0 – 6.5	0.16	2.82	3.0 – 4.0
					2500	4050	8.28	6.0 – 8.0	0.33	4.62	3.5 – 5.0
I	1.83	30	2.5	0.5	3500	3690	7.28	6.5 – 8.5	0.27	5.70	3.5 – 5.0
J	1.83	30	2.5	0.5	3500	5960	7.43	6.5 – 8.5	0.43	5.60	3.5 – 5.0
					50	35	2.73	1.5 – 3.0	0.04	0.94	1.5 – 2.5
					150	140	2.95	2.5 – 3.5	0.10	1.48	1.5 – 2.5
					400	290	3.43	3.5 – 5.0	0.13	2.05	2.0 – 3.0
					700	490	4.10	4.2 – 5.7	0.15	2.38	2.5 – 3.5
					1000	790	4.62	5.0 – 6.5	0.18	2.78	3.0 – 4.0
					1500	1100	4.84	5.5 – 7.0	0.20	3.40	3.0 – 4.0
					2500	2370	6.47	6.0 – 8.0	0.26	4.24	3.5 – 5.0
K	1.25	19	2.5	0.5	3500	3760	7.07	6.5 – 8.5	0.30	5.48	3.5 – 5.0

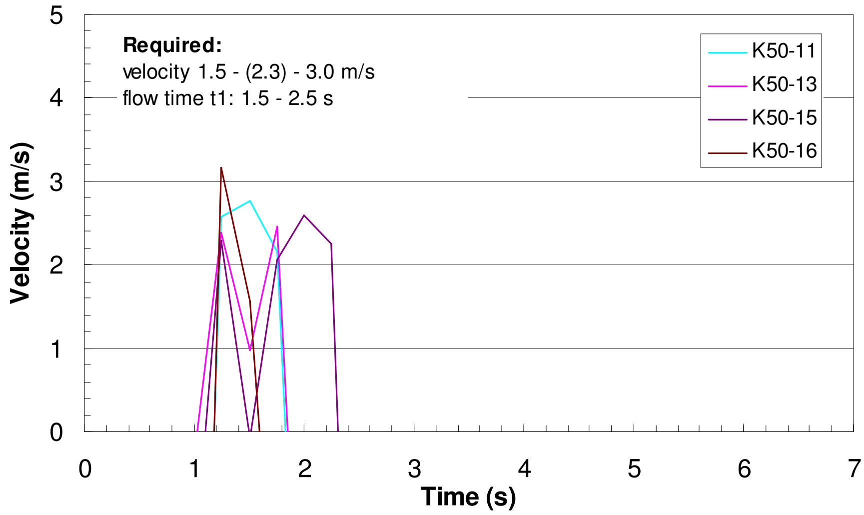


Figure 6.4. Flow velocities for test series K50

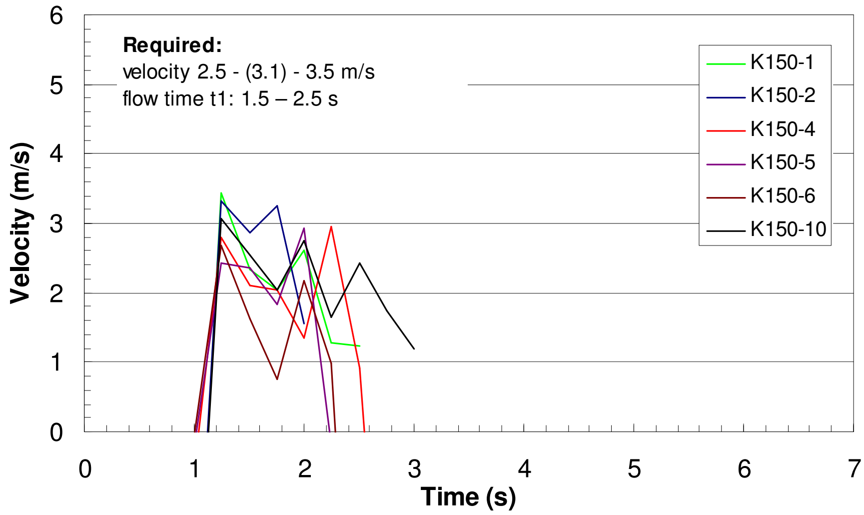


Figure 6.5. Flow velocities for test series K150

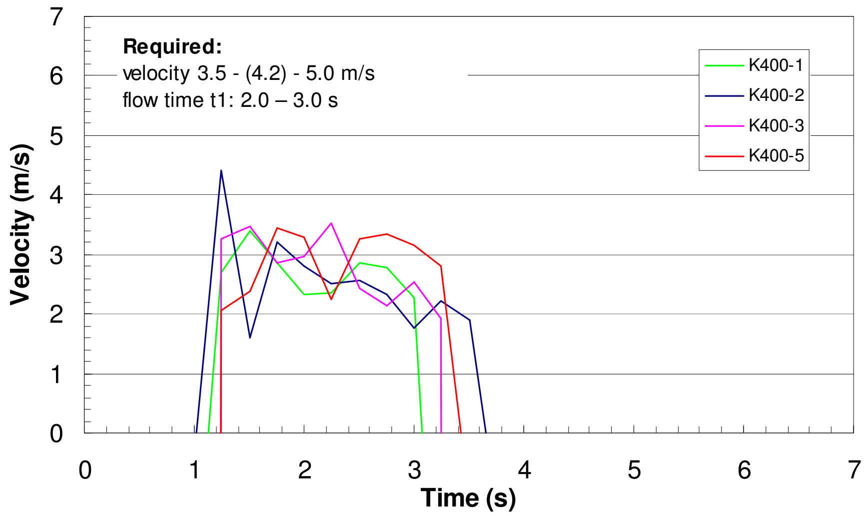


Figure 6.6. Flow velocities for test series K400

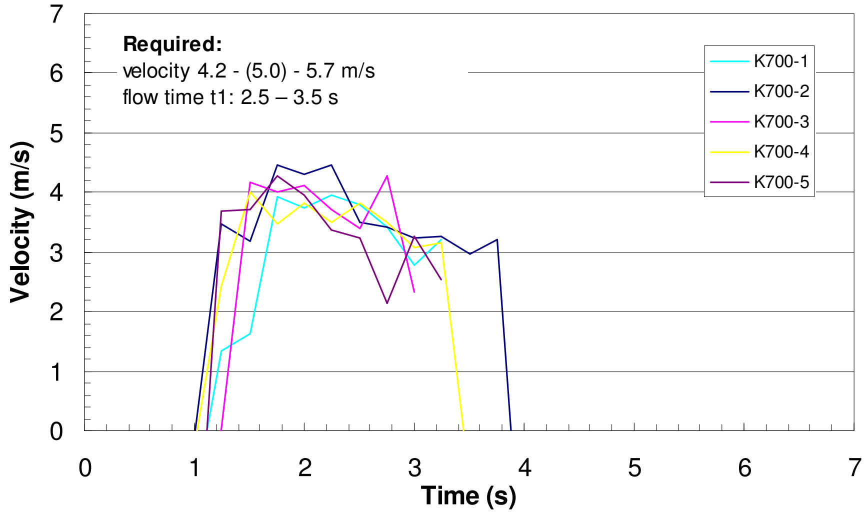


Figure 6.7. Flow velocities for test series K700

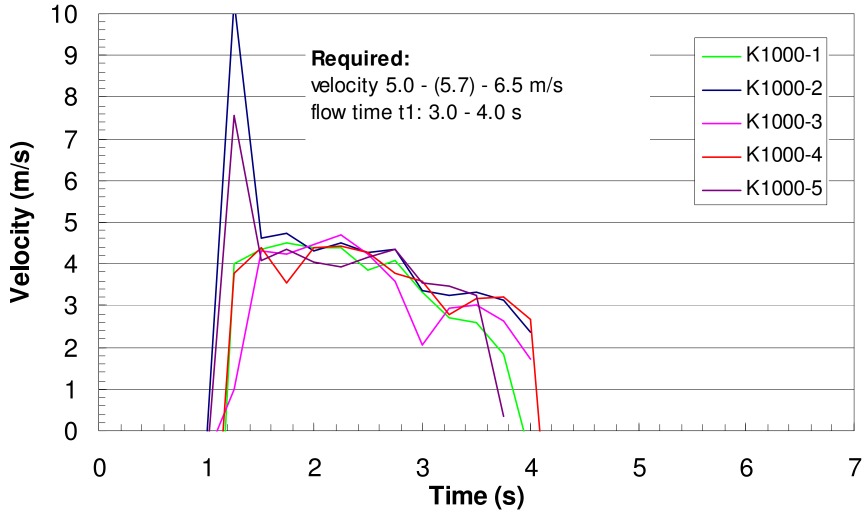


Figure 6.8. Flow velocities for test series K1000

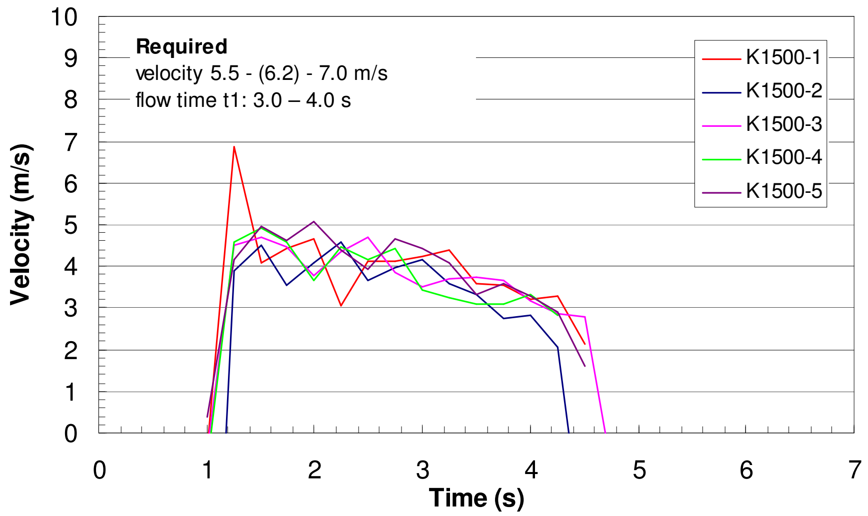


Figure 6.9. Flow velocities for test series K1500

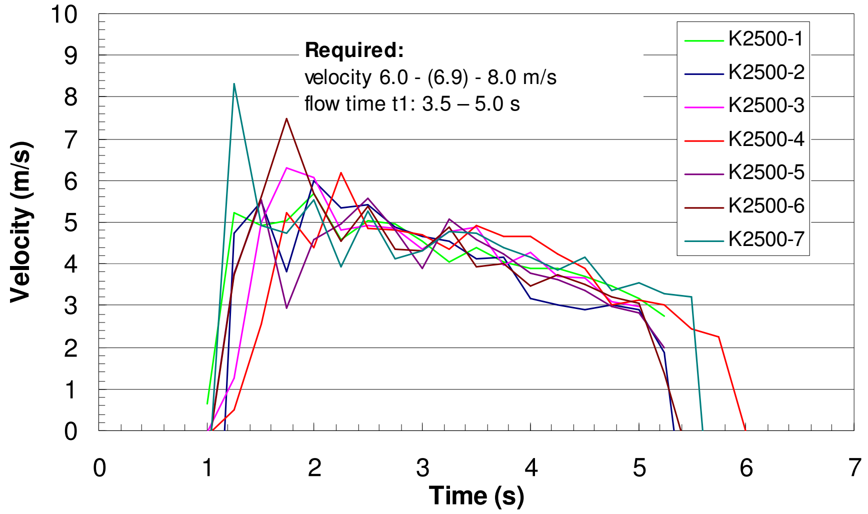


Figure 6.10. Flow velocities for test series K2500

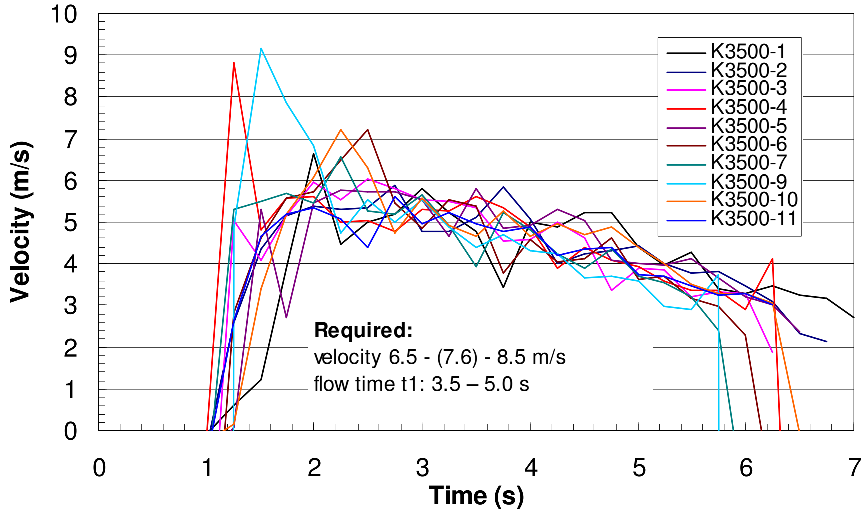


Figure 6.11. Flow velocities for test series K3500

Table 6.3. Final results for test series K

Test	Volume V in simulator [liter]	Volume V (liter) according to 1/3*v*d*t	Max. velocity v [m/sec]	Max. velocity $v_{required}$ [m/s]	Max. flow depth [m]	Flow time t_1 [s]	Flow time $t_{1;estimated}$ [s]
K50	50	35	**2.73**	1.5 – 3.0	0.04	**0.94**	1.5 – 2.5
K150	150	140	**2.95**	2.5 – 3.5	0.10	**1.48**	1.5 – 2.5
K400	400	290	**3.43**	3.5 – 5.0	0.13	**2.05**	2.0 – 3.0
K700	700	490	**4.10**	4.2 – 5.7	0.15	**2.38**	2.5 – 3.5
K1000	1000	790	**4.62**	5.0 – 6.5	0.18	**2.78**	3.0 – 4.0
K1500	1500	1100	**4.84**	5.5 – 7.0	0.20	**3.40**	3.0 – 4.0
K2500	2500	2370	**6.47**	6.0 – 8.0	0.26	**4.24**	3.5 – 5.0
K3500	3500	3760	**7.07**	6.5 – 8.5	0.30	**5.48**	3.5 – 5.0

The comparison of required flow velocity and flow time for this test series K is given in Table 6.3. Flow velocities are within the required range or close to it. Flow times are almost all within the estimated range they should be. This range is, however, only based on physical reasoning and not on research. Also calculated volumes are quite close to the actual volumes. The maximum flow depths, measured visually, show a constant increase with increasing volume.

Overall it can be concluded that the set-up for test series K gives the required performance of the wave overtopping simulator.

6.4 Simulation of wave overtopping in real time

Till now the prototype of the wave overtopping simulator was calibrated with single volumes. Wave overtopping in reality is a certain time period where volumes of water come over the crest in a random way. Section 2.2 describes the distribution of overtopping volumes in a certain period and for a certain overtopping discharge. The volumes should be randomly distributed in a real time simulation.

In reality waves come in wave groups and it is well possible that large overtopping waves are accompanied by other fairly large overtopping waves. This is different for the wave overtopping simulator. A mean discharge is pumped into the simulator and released when the required volume has been reached. It takes longer time to fill up to the large volumes than the small volumes. The mean discharge and the wanted volumes determine the actual time between consecutive overtopping events.

The pumps available had a capacity of 3 l/s and 8 l/s, combined 11 l/s. The water was pumped into the box a little above the valve. With large volumes the water gave a pressure when exceeding the level of the intake. This reduced the pump capacity. Therefore the filling of the box to certain volumes (or levels) was calibrated with time. This calibration was used to create a real time simulation of about 15 minutes. The order of overtopping volumes is given in Table 6.4. The measured record of the flow velocity is given in Figure 6.12 and shows a large number of unwanted spikes, due to bubbles touching the EMS when there is no or hardly water. With help of the time in Table 6.4 the record can be screened, which is given in Figure 6.13.

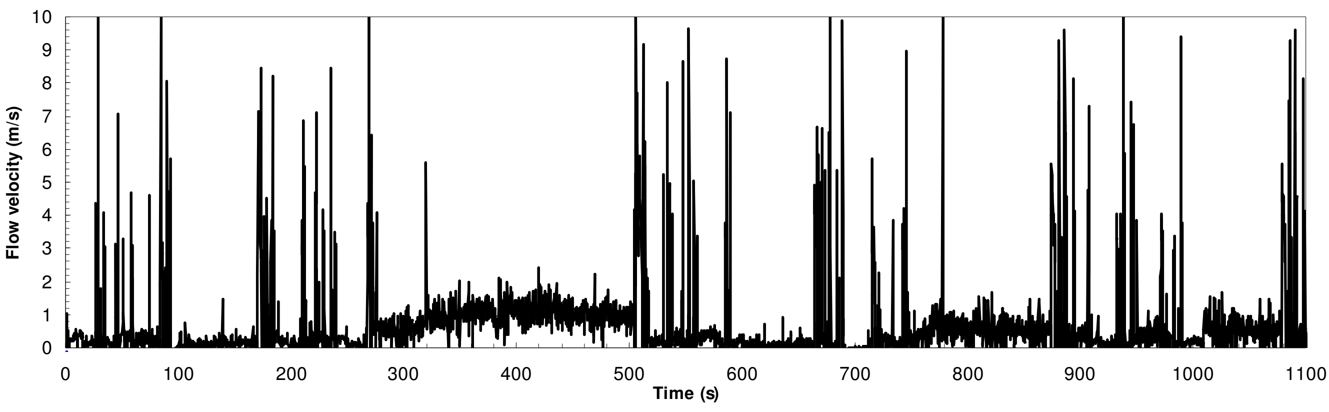


Figure 6.12. Record for 15 minutes real time wave overtopping simulation

Table 6.4. Simulation of real time wave overtopping for 15 minutes

Wave number	Volume (l)	Time (s)	Flow depth (m)	Velocity (m/s)
1	400	0.27	16	4.38
2	50	0.33	10	4.09
3	150	0.45	9	2.65
4	50	0.51	9	3.31
5	50	0.58	9	4.69
6	400	1.25	11	3.92
7	50	1.31	8	4.72
8	1000	2.52	18	4.53
9	150	3.04	8	3.84
10	400	3.31	11	5.48
11	150	3.43	9	4.68
12	50	3.50	10	4.15
13	50	3.56	8	1.75
14	50	4.02	7	0.21
15	400	4.30	12	4.36
16	2500	8.32	28	7.69
17	400	8.59	10	8.00
18	150	9.11	12	0.50
19	50	9.17	9	5.02
20	50	9.23	7	0.34
21	400	9.50	13	7.12
22	1000	11.11	18	6.63
23	150	11.24	11	2.10
24	50	11.30	10	2.58
25	50	11.36	9	4.00
26	400	12.03	12	0.39
27	400	12.30	11	0.49
28	1500	14.44	23	6.49
29	150	14.56	9	4.14
30	50	15.03	9	3.10
31	150	15.15	8	0.96
32	400	15.42	12	7.43
33	50	15.48	9	3.83
34	50	15.55	9	0.57
35	400	16.22	15	2.93
36	150	16.34	10	0.28
37	50	16.40	9	0.32

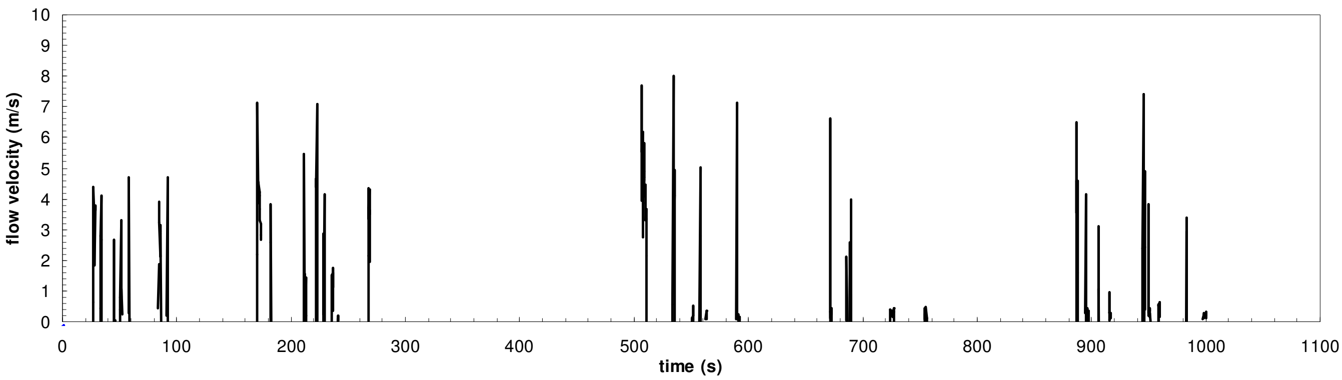


Figure 6.13. Screened record for 15 minutes real time wave overtopping simulation

This screened record looks much better, but it takes a lot of time to create it. The maximum velocities were taken from Figure 6.13 and given in Table 6.4. In Figure 6.14 the measured velocities are compared with the required maximum velocities. Large scatter is found, mainly due to the fact that the EMS measures only with 4 Hz and shows also large peaks in the measurements.

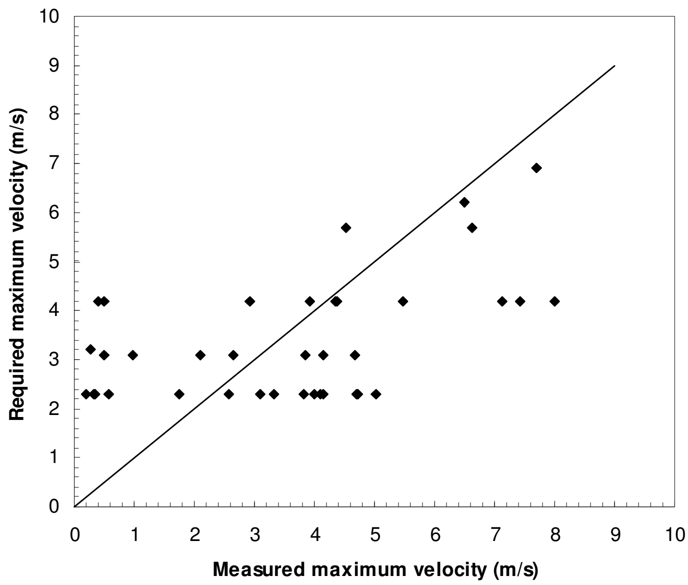


Figure 6.14. Comparison of measured and required maximum velocities for the overtopping simulation in real time

6.5 Conclusions and recommendations

It was well possible to calibrate the prototype wave overtopping simulator for the required targets of maximum flow velocity and flow time. Test set-up K should be the guidance to the development of the real simulator.

Actually, based on present research, good reliable requirements are only available for the maximum flow velocity for each overtopping volume. The flow time was estimated on a physical reasoning with respect to the peak wave period, not on results of research. The flow depth is not easy to measure and present research shows large deviations. Based on the analysis performed in this report it is assumed that the maximum flow depths given by Van Gent (2002) are closer to reality than those given by Schüttrumpf (2002). It is recommended to perform more research on flow depth as well as flow time. The measurements of Van Gent (2002) seem very suitable for this extended research.

Flow velocities can be measured with an EMS, but it is not easy to interpret the records. There are a lot of unwanted spikes if no or hardly water is present and sometimes too large peaks are present in the record. A better sampling rate of around 10-20 Hz will solve the problem partly. The acoustic flow depth meter was not able to measure the flow depths correctly, due to the high turbulence and uneven surface. For the calibration maximum flow depths were measured visually, but this is not the right way for real time wave overtopping which lasts for hours. It can be concluded that if measurements of flow velocity and flow depth are needed on the dike, a study should be undertaken to find the right instruments.

With respect to the design of the real 4 m wide simulator, it was concluded that a valve opening of 0.5 m is sufficient and that it should always be opened to its full width. Furthermore, a hydraulic system has to be installed to open and close the valve. It is also recommended to make the valve as smooth as possible, without disturbing, but supporting steel girders. See Figure 6.14, which gives the final design for the 4 m overtopping simulator. The transition slope has to be well connected to the box near the rear side of the valve. This in order to make a smooth transition for the flowing water. The front side under the valve should have a guidance for the flowing water, see Figure 6.14. The intake of the water by pumps has to be on top of the box. This in order to create a constant discharge for the pumps. An intake at a lower position will influence this discharge, as was the case with the prototype. The gauge glass along the side wall of the simulator should be thicker and should extend to the bottom of the box. A 3D-sketch of the real simulator with adjusted transition slope is given in Figure 6.15 (although still with a width of 1 m).

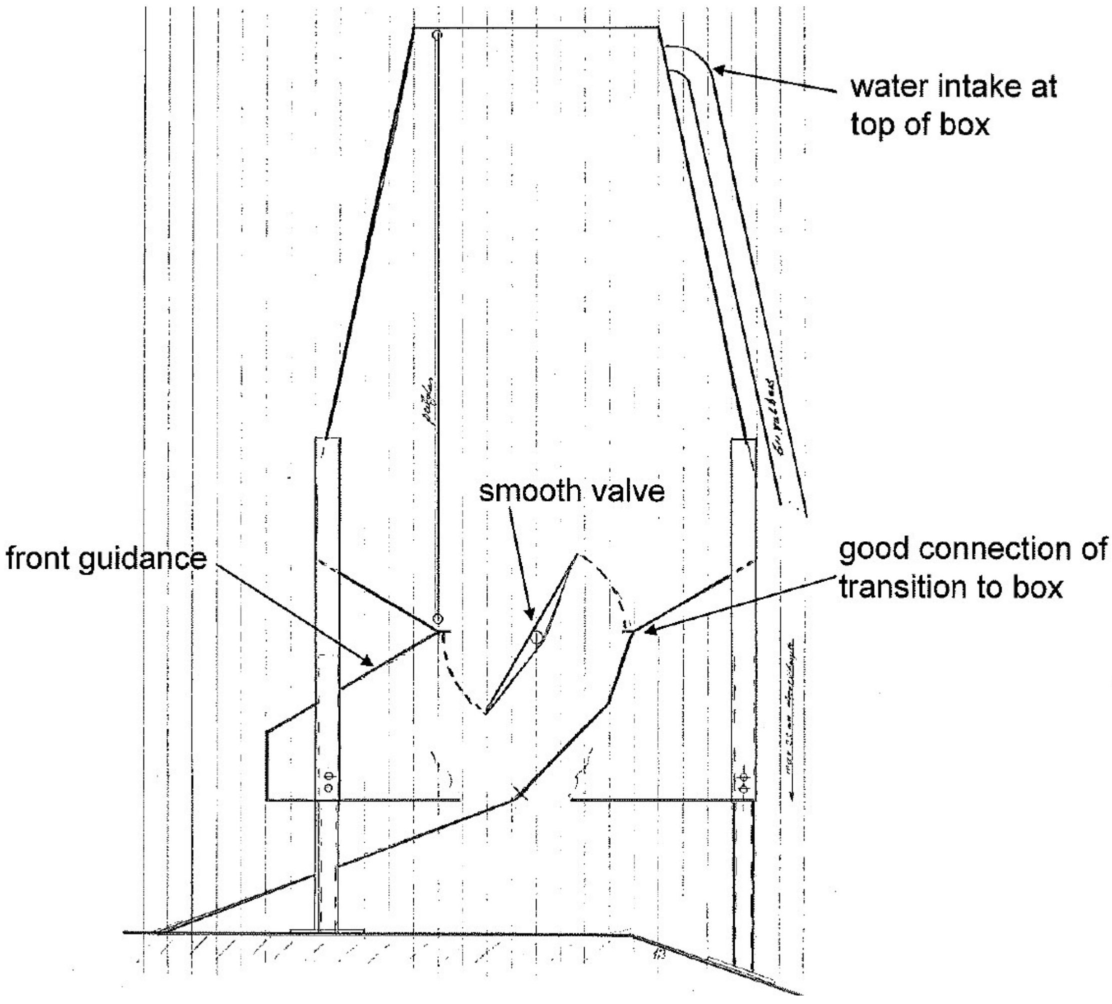


Figure 6.14. Sketch of overtopping simulator with some improvements

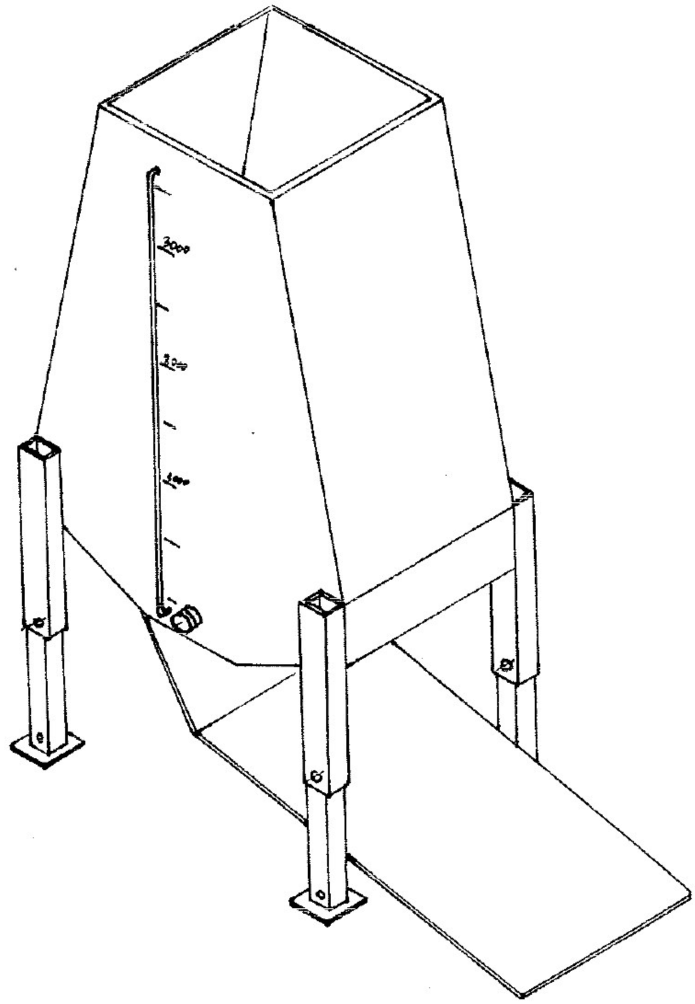


Figure 6.15. 3D-sketch of the final design of the overtopping simulator with adjusted transition

7 Construction and testing of real wave overtopping simulator

7.1 Construction

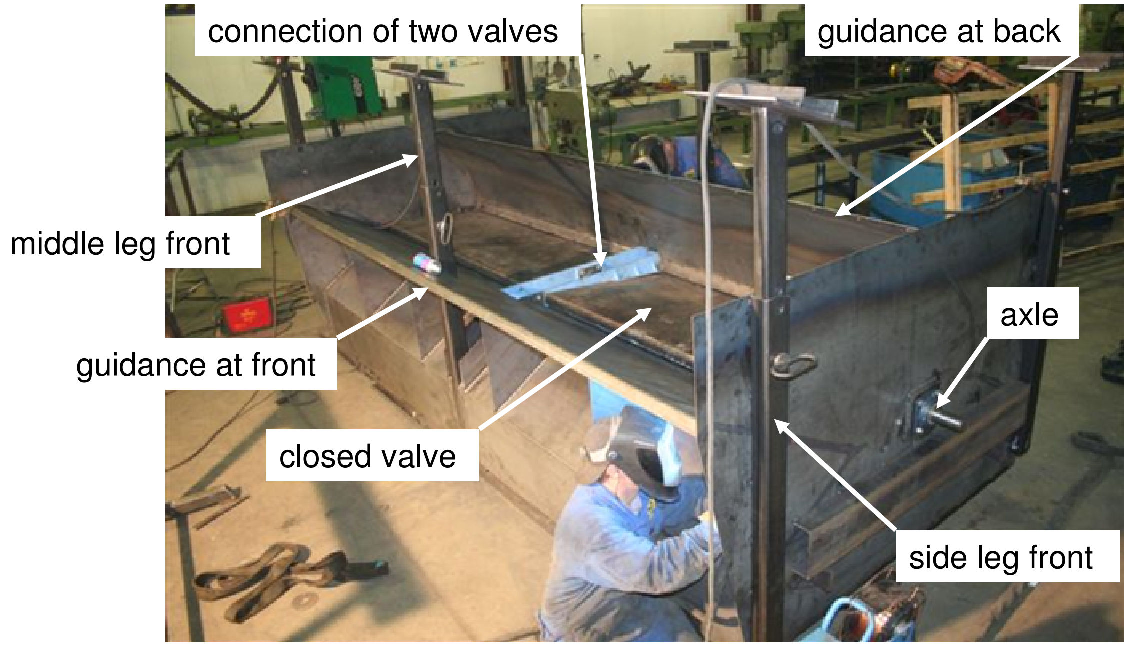
The design of the wave overtopping simulator, the construction of a 1 m wide prototype and the calibration of this prototype was performed in May-June 2006. A full report on these activities has been issued (Infram and Royal Haskoning, 2006). That report has been used and extended for the present report, in order to have one report on the full development and use of the wave overtopping simulator. Chapters 1 - 6 of the present report are similar to the full report from 2006.

The final goal of the prototype calibration was to come to a final design of the 4 m wide overtopping simulator to be used for testing on the dike at Delfzijl. Based on the experience with the prototype, some improvements were suggested for the final simulator. These conclusions and recommendations have been described in Section 6.5 and gave the start for the construction of the final simulator.

The final 4 m wide simulator was constructed in November and December 2006 at Nijholt Staal & Machinebouw at Heerenveen. The mechanical design was optimised by G. van der Meer.

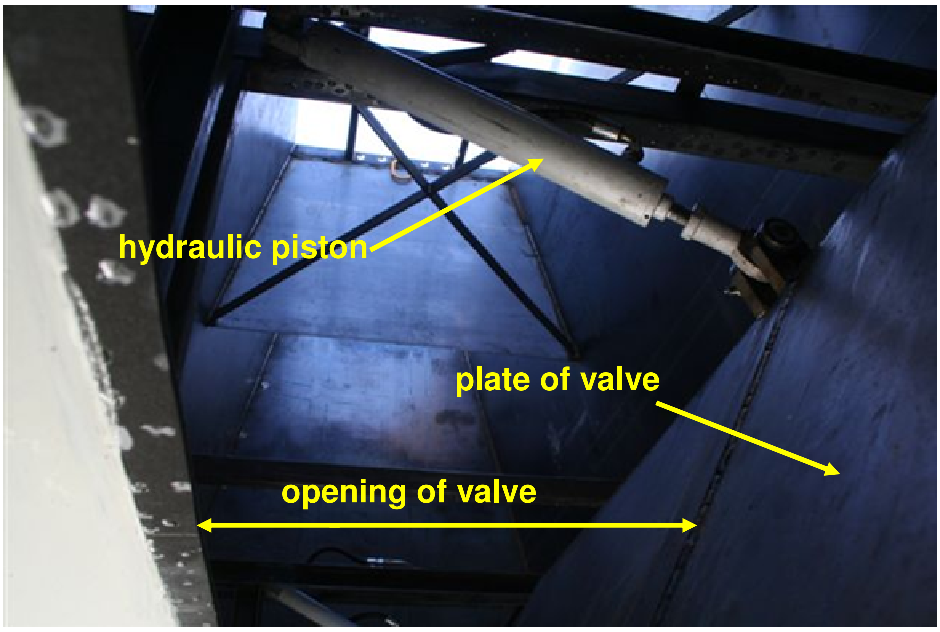
The butterfly valve is an important part of the simulator and it should function well during the whole testing period. It was made of a 2 m long construction frame with in the middle an axle (see Picture 7.1). The construction frame was covered by steel plates. Two of these valves were placed in the simulator, connected in the middle. Picture 7.2 shows the closed valve, the connection in the middle and the axle (for construction reasons the simulator was put upside down). The connection of the axle to the side wall of the simulator is shown at Picture 7.3. The valve was operated by hand during calibration of the prototype. The final simulator had two hydraulic pistons inside the simulator, which were operated by the hydraulic pressure of a tractor. Picture 7.4 shows the inner side (from below) of the simulator and one of the hydraulic pistons.

Picture 7.1. Construction frame with axle for butterfly valve, without cover plates



Picture 7.2. Construction of box with valve and legs. Box was place upside down.

Picture 7.3. Connection of axle to the side wall of the simulator.



Picture 7.4. Inner side of simulator, looking through the opening of the valve upwards.

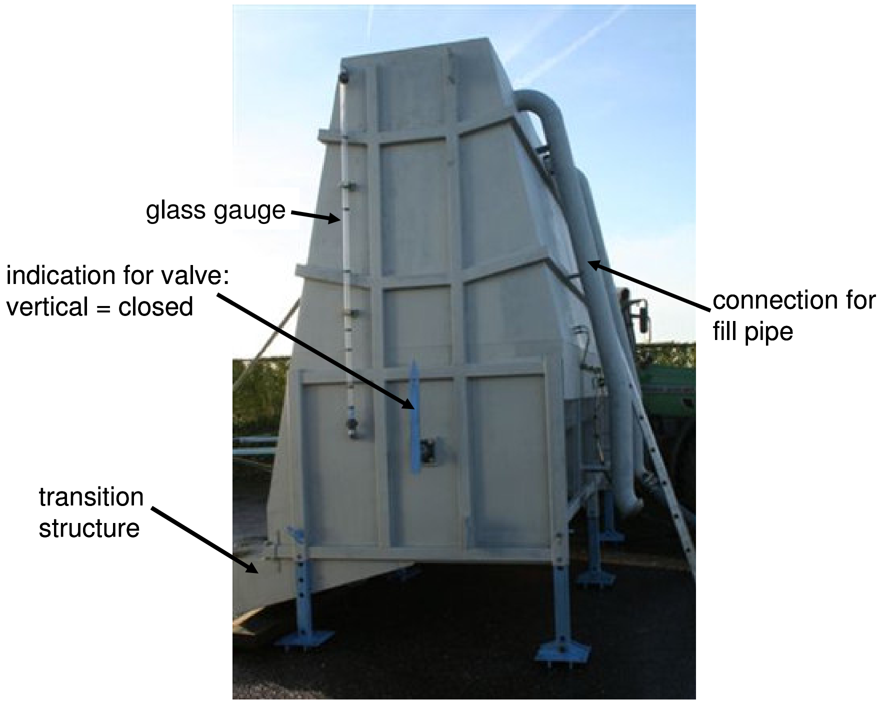
Picture 7.5. Adjustable leg front side (upside down

Picture 7.6. Adjustable leg rear side (upside down). Plate was later made rotational.

The 4 m wide simulator had six adjustable legs. The front legs had a horizontal base plate, see Picture 7.5, as they were to be put on the crest of the dike. The legs at the rear side first had plates under an angle (angle of seaward slope of the dike), see Picture 7.6. Later the connection to the leg was made in such a way that the plate could rotate and adjust to the slope of the dike.

Part of the transition or guidance structure, which was a separate structure during calibration, was now connected to the box of the simulator. Picture 7.2 shows the plates for guidance on both the back and front side of the box. The remainder of the transition slope was mounted to the simulator before testing, see Picture 7.7, and had to be dismantled before transport on a truck. Picture 7.7 shows the simulator after construction and painted with primer. The glass gauge to monitor the fill level of the box is given, an indicator to monitor whether the valve is open or closed, the remaining transition structure and the fill pipes which have their entrance to the box at the very top in order to guarantee a constant discharge by the pumps.

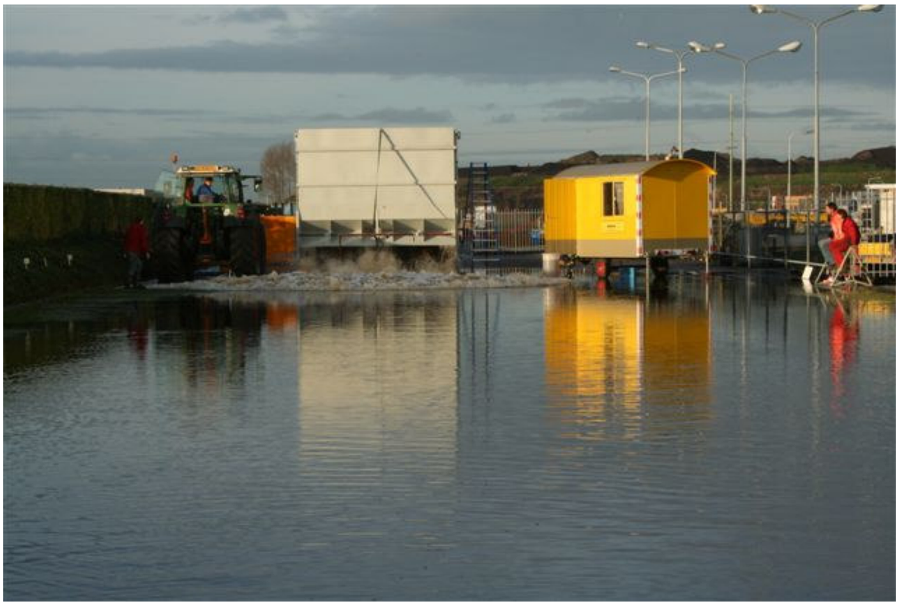
Picture 7.8 shows the rear side of the simulator after construction and just before first testing it. The two fill pipes are shown with one connected to a small pump. The tractor is required to operate the valve by hydraulic pressure. Picture 7.9 shows the simulator from the top with the box almost completely filled. Picture 7.10 shows the testing or working of the final wave overtopping simulator. After some improvements on leakage of water at the valve, the simulator was considered to be ready for testing on the dike.



Picture 7.7. Side view of the simulator after construction, painted by primer.

Picture 7.8. View from the rear, after construction and before testing the simulator.

Picture 7.9. The simulator from the top, filled with 14 m^3 of water.



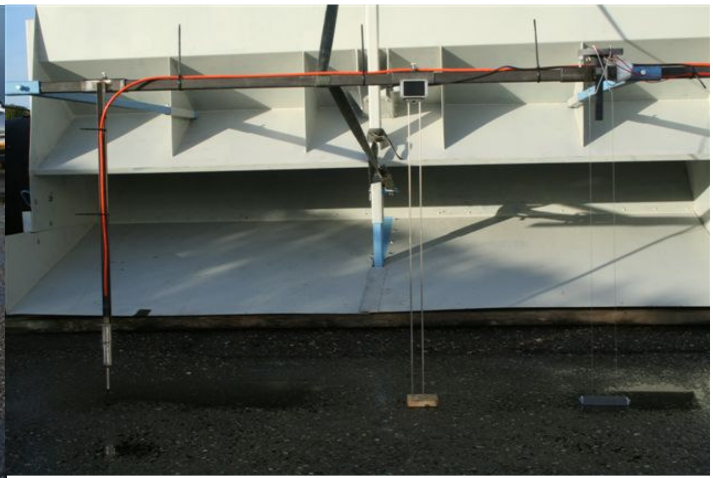
Picture 7.10. First testing of the simulator at the parking place of Nijholt Staal & Machinebouw.

7.2 Testing new instruments

During the calibration of the prototype (Section 6.2) it became clear that flow depth could not be measured with the acoustic flow depth meter. Moreover, the 4 Hz sampling frequency of the electromagnetic velocity meter was seen as insufficient. For this reason WL | Delft Hydraulics was asked to come with a possible solution.

At 18 December 2006 the wave overtopping simulator was tested (see Picture 7.10) and at the same time also adapted instruments of WL | Delft Hydraulics. The calibration of one the instruments is shown on Picture 7.11. The three instruments tested are shown on Picture 7.12. From left to right this is an EMS (electromagnetic velocity meter), but now with 25 Hz sampling frequency, a conventional wave gauge and a wire flow depth gauge. This wire flow depth gauge was specially developed for this kind of tests. The instruments were placed directly in front of the simulator. The main objective was to obtain a measured record for further analysis and to estimate the reliability under test conditions

Figures 7.1 and 7.2 show examples of records of the EMS and the conventional wave gauge. Unfortunately, for some unknown reason, the signal of the wire flow depth gauge could not be measured. It should have given a record comparable to the wave gauge. Another pity was that the velocity meter was calibrated at the office at a maximum of 2.5 m/s. Actual velocities were often larger than 5 m/s and then the record was clipped at maximum voltage, giving 2.5 m/s. This was also the case in Figure 7.1, although this was for a released volume of water that did not generate much higher velocities than 2.5 m/s. Although the “spiky” signal remains as during the calibration tests, there are now more data points per second and with some filtering or smoothing it should be possible to generate a signal giving more or less a continuous line. It was concluded, therefore, that this EMS would be sufficient for testing at the dike.



Picture 7.12. From right to left in front of the simulator: electromagnetic velocity meter EMS, conventional wave gauge and new wire flow depth meter.

Picture 7.11. Calibration of instrument

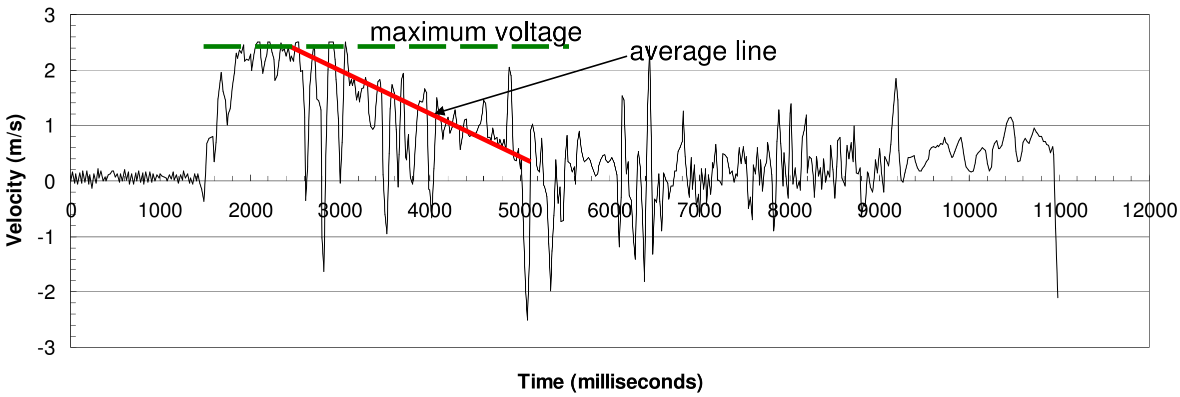


Figure 7.1. Record of electromagnetic velocity meter.

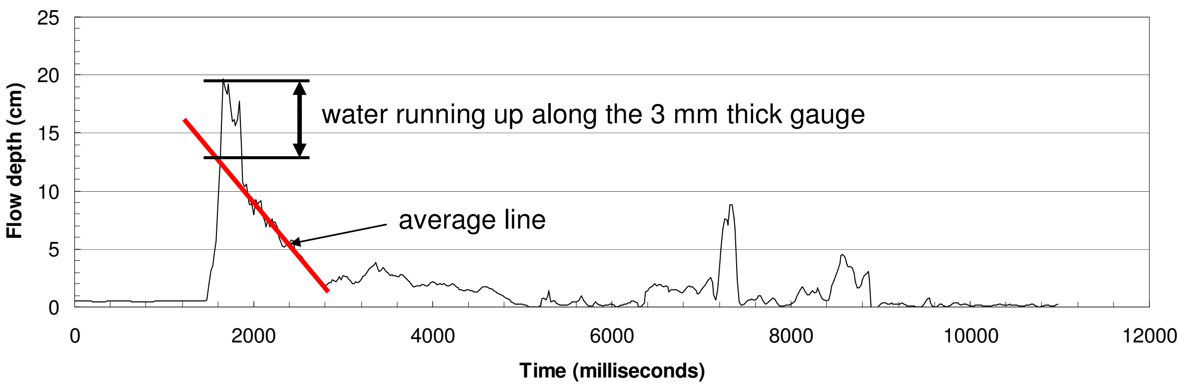


Figure 7.2. Record of conventional wave gauge.

The conventional wave gauge (Figure 7.2) gave a nice signal. The problem, however, was that water with a large velocity bounced against the 3 mm thick gauge and went vertically up along the gauge for many centimetres. This was also noticed by eye and the water could go up for 10 – 20 cm or even more! This can also be seen in Figure 7.2 where the first part is higher than suggested by the average line. It was concluded that the conventional wave gauge could not be used to measure flow depths accurately.

Although the wire flow depth meter did not record any signal, it was witnessed by eye that water was not running up along the wire and, therefore, this wire gauge would be a much better instrument than the wave gauge to measure flow depth. It was assumed that the recorded signal would be similar to the wave gauge, this means without spikes. It was, therefore, concluded that this wire flow depth meter should be used at the dike.

7.3 Calibration day at dike

On 19 December 2006 the wave overtopping simulator was transported to the dike at Delfzijl and installed on the crest and partly on the seaward slope of this dike. Picture 7.13 shows the installation and the mounting of the transition or guiding slope to the simulator. For this calibration test day the pump was hung directly into the box of the simulator.

Picture 7.13. Installation on dike for calibration day and mounting of transition slope.

The objectives of the calibration day on 20 December were three-fold:

a) the behaviour of the simulator on the dike should be evaluated, specifically when filled with 14 m^3 or 14 tons of water.
b) the main objective was to test the pumps, the pump capacity, and reliability in getting a constant required discharge;
c) all the testing would be recorded on digital video. The situation early in the morning with lack of day light should be tested and a good location for the video should be found.

The working of the simulator itself was not an objective, as the simulator was tested sufficiently directly after construction. The situation just before the first water would flow over the dike is shown in Picture 7.14. Guiding walls were provisionally made, only to half of the inner slope. On 20 December the first water flowed over the dike, see Picture 7.15.

Picture 7.14. Wave overtopping simulator installed on dike and ready for action on the calibration day.

Picture 7.15. The first water flowed over the dike on 20 December 2006.

ad a). When installing the simulator on the dike it became clear that a dike is not nicely flat and equal. Some of the six legs hardly touched the ground before water was pumped into the simulator. But after filling the box a few times to its maximum of 14 m^3 the simulator settled and the plates of the legs were pushed into the clay for sometimes 5 – 10 cm, see Picture 7.16. It was clear that the plates were too small to survive many loads of a full simulator and would slowly, but steadily, subside into the clay.

The only remaining issue for the simulator was to modify the plates of the legs and give them more bearing capacity. With the legs adjustable in height it was fairly easy to install the simulator vertically. The opening and closing of the valve occurred on the tractor and worked sufficiently.

Picture 7.16. Subsidence of the legs into the clay after being loaded a few times with 14 m^3 of water.

ad b). The main objective was to test the pump system. An immersed pump with a capacity of around 700 m^3/hour was rented from the company Buitenkamp, together with a power generator. The pump and hoses to the wave overtopping simulator are shown on Picture 7.17. The required pump capacity could be adjusted and maintained by setting a frequency. For real testing on the dike the following pump capacities or discharges were required and foreseen:

1 l/s per m width, or 14.4 m^3/hour (4 m wide simulator)
10 l/s per m width, or 144 m^3/hour
20 l/s per m width, or 288 m^3/hour
30 l/s per m width, or 432 m^3/hour

All these discharges were tested on 20 December. It appeared that the larger discharges could be adjusted very well and also stayed constant during pumping. The smallest discharge of only 1 l/s per m width was a little difficult as discharges with an accuracy of 0.1 m^3/hour could not be adjusted as the frequency step was not small enough. The accuracy was about 1 m^3/hour. Overall it was concluded that only one pump was sufficient to reach the required discharges and that a constant discharge could be generated over the period of a test (6 hours).

Picture 7.17. The pumping system during the calibration day on 20 December.

ad c). The experience with video during minimal daylight and the right position of the camera, was another objective. This experience has been described in the report on testing and will not be described here.

Finally, a specific observation has been noted during this calibration day. When the simulator was filled for the first time and released, this water flowed over a dry inner slope and could penetrate into this slope. The simulator was filled with 400 l per m width, or 1200 l in total. When this volume was released *all water penetrated into the slope*. No water reached the road at the toe of the slope. A second volume of 1200 l penetrated partly into the slope, but roughly half of the volume indeed reached the road. For later volumes it could not be noticed that water penetrated into the slope and the impression was that all water then went along the slope.

8 Wave overtopping simulator in action at the dike

8.1 Installation at the dike

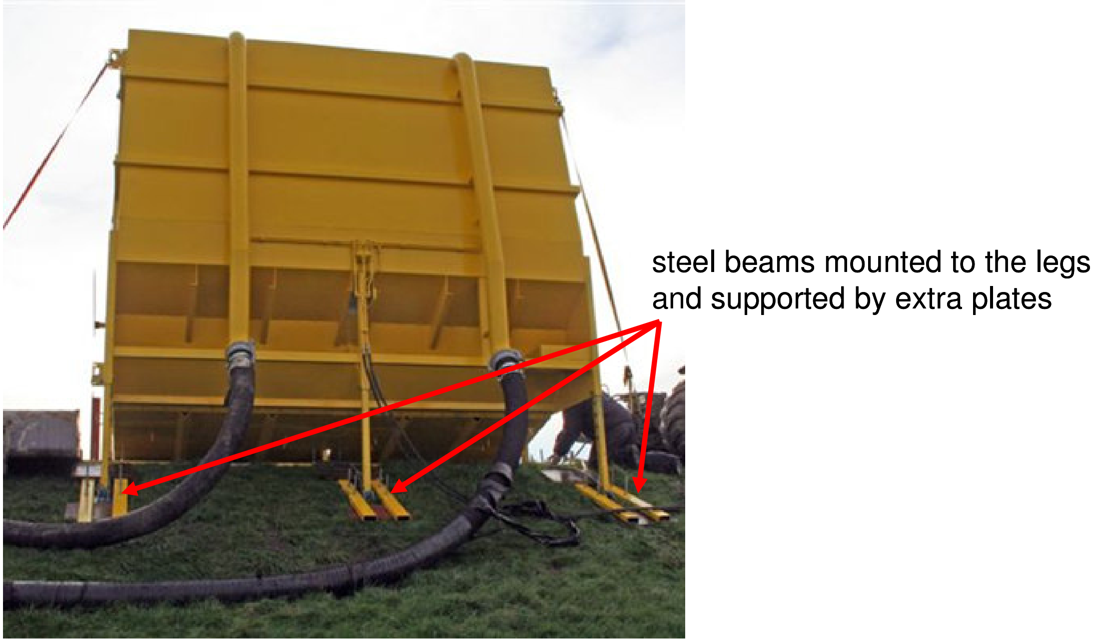
During January and February 2007 the wave overtopping simulator was further prepared to be used for real testing end of February. First of all the simulator was painted in its final colour, the yellow of the Rijkswaterstaat. Also the prototype was painted yellow and both were transported on 16 February to the location at Delfzijl, see Picture 8.1. The simulator has a height of 3.3 m, without legs being installed. In order to be transported as normal traffic on the road, the simulator has to be placed on a low loader.

A large crane took the simulator from the truck and put it on the crest of the dike. But before placing it there, the transition or guiding slope was mounted to the simulator, see Picture 8.2. The placing of the simulator, hanging in the large crane, is shown on Picture 8.3.

Picture 8.1. Transport of simulator and prototype arriving at test location at Delfzijl.

Picture 8.2. Mounting the transition or guiding slope onto the simulator while hanging in the crane.

Picture 8.3. Placing the simulator on the test section by a large crane.



Picture 8.4. Adjustments to the plates of the legs by extra steel girders and plates. Two hoses of the pumping system have been attached to the simulator.

Picture 8.5. Detail of adjusted leg with extra steel girders and wooden plates.

As result of the calibration day in December (see Section 7.3) the plates of the legs had been adjusted, see Pictures 8.4 and 8.5. After first settlement of the original plates by filling the simulator with water, additional steel girders were mounted to the plates. Two extra wooden plates were then placed underneath these girders. In this way the total load of a full simulator (about 16 tons) could be reduced to acceptable pressures on the soil, where the girders could still be adjusted to local unevenness of the slope (the system was adjustable, not fixed).

Another item that was adjusted during the preparation for the real testing was the operation of the valve to open and close the box of the simulator. First the operation of the system was the original system on the tractor to operate the hydraulic system of this tractor. A modification was made in such a way that the operation could take place in the measuring cabin by means of a joy stick, see Picture 8.6.

Picture 8.6. Operating the wave overtopping simulator by means of a joystick and list of times when to open the valve.

8.2 Circulation of water

A large pump with a capacity of 700 m^3/hour was rented from the company Buitenkamp. The discharge could be adjusted by a frequency processor. Picture 7.17 gives the set-up during the calibration day at 20 December and this was similar for the real testing. A large hose contained the water and close to the simulator this hose was split into two smaller hoses, which were connected to the rear side of the simulator, see Picture 8.4

This system worked well for all test conditions foreseen. The maximum overtopping discharge which was foreseen and where the wave overtopping simulator was designed for, was 30 l/s per m, or 432 m^3/hour. The pump capacity of 700 m^3/hour was enough to get this discharge indeed 10 m higher in the box of the simulator. But after testing with 30 l/s per m there was still no damage to the test section and it was decided to upgrade the simulator to 50 l/s per m.

In order to get this discharge a larger pump with a capacity of 1000 m^3/hour was ordered, together with a heavier power generator. But the frequency processor remained in place. It appeared that this frequency processor did not really match with the new pump and only a maximum discharge of 40 l/s per m could be reached. After little time this frequency processor collapsed. It was replaced by another one, but then the engine of the pump broke down. This pump was replaced by a similar one, but finally it was decided not to go beyond the discharge of 40 l/s per m (576 m^3/hour).

In order to increase the pump capacity it was decided to place the large hose of the pump system directly into the box of the simulator and not to split this large hose in two smaller ones (as on Picture 8.4). In this way part of the energy loss by friction was overcome. Picture 8.7 gives an impression a large released volume during testing with 50 l/s per m wave overtopping.

Overall it can be concluded that the foreseen discharges could be generated as wanted and that problems started after requiring almost the double of the maximum foreseen discharge. Problems with the frequency processor and pump were fixed by the company Buitenkamp as soon as possible and they are gratefully acknowledged for their support.

Picture 8.7. Release of a large volume during test with 50 l/s per m. Hose of pump was put directly into the box of the simulator.

8.3 Operation during testing

A list of exact times when to open the valve during testing was provided for each test. The procedure how to calculate the number of overtopping waves during 6 hours and the distribution of overtopping volumes has been described in Section 2.2. This procedure was used to make a list for each test condition and for 1 or 2 hours, depending on the number of overtopping waves. This list was then repeated to complete the six hours of test duration.

Table 8.1. Distribution of volumes for two hours of 10 l/s per m wave overtopping

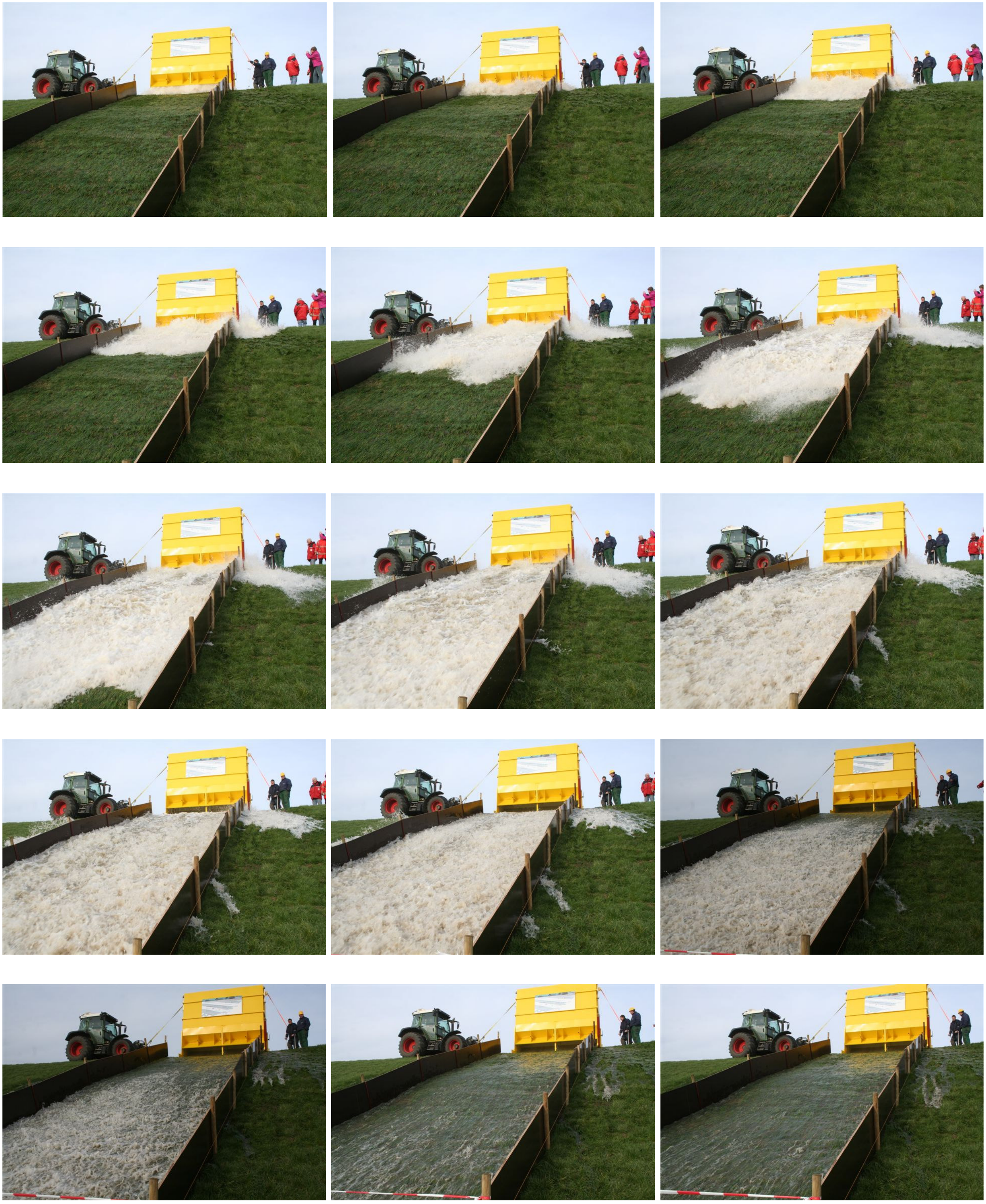
number of events	volume (l/m)
2	2500
3	1500
11	1000
19	700
49	400
84	150
128	50

Table 8.2. Part of operating list of times to open the valve of the simulator

Test 10 l/s per m; 2 hours

Wave number	Volume (liters/m)	Open hr.min.s		Wave number	Volume (liters/m)	Open hr.min.s
1	150	0.00.13		51	50	0.19.42
2	150	0.00.28		52	700	0.20.52
3	50	0.00.33		53	400	0.21.32
4	50	0.00.38		54	150	0.21.48
5	400	0.01.17		55	50	0.21.53
6	50	0.01.23		56	50	0.21.57
7	150	0.01.38		57	150	0.22.12
8	150	0.01.53		58	400	0.22.52
9	50	0.01.58		59	50	0.22.57
10	400	0.02.37		60	150	0.23.12
11	700	0.03.47		61	50	0.23.18
12	150	0.04.02		62	50	0.23.23
13	50	0.04.08		63	400	0.24.02
14	150	0.04.23		64	150	0.24.18
15	50	0.04.28		65	700	0.25.27
16	50	0.04.33		66	50	0.25.33
17	150	0.04.48		67	400	0.26.12
18	400	0.05.27		68	50	0.26.18
19	150	0.05.43		69	50	0.26.23
20	1000	0.07.21		70	50	0.26.27
21	50	0.07.28		71	150	0.26.42
22	400	0.08.07		72	1000	0.28.21
23	150	0.08.23		73	400	0.29.02
24	50	0.08.28		74	50	0.29.07
25	50	0.08.33		75	150	0.29.23
26	50	0.08.38		76	50	0.29.27
27	400	0.09.17		77	50	0.29.33
28	150	0.09.33		78	400	0.30.12
29	50	0.09.38		79	50	0.30.18
30	400	0.10.17		80	150	0.30.33
31	150	0.10.33		81	150	0.30.48
32	50	0.10.38		82	400	0.31.27
33	150	0.10.53		83	50	0.31.33
34	150	0.11.07		84	50	0.31.38
35	50	0.11.13		85	700	0.32.46
36	400	0.11.52		86	400	0.33.27
37	700	0.13.01		87	50	0.33.32
38	50	0.13.07		88	150	0.33.47
39	50	0.13.13		89	50	0.33.53
40	50	0.13.18		90	50	0.33.58
41	400	0.13.57		91	400	0.34.37
42	50	0.14.03		92	150	0.34.53
43	150	0.14.18		93	50	0.34.58
44	50	0.14.23		94	50	0.35.02
45	50	0.14.28		95	400	0.35.42
46	400	0.15.07		96	400	0.36.22
47	1000	0.16.45		97	50	0.36.28
48	150	0.17.03		98	150	0.36.43
49	50	0.17.07		99	50	0.36.47
50	1500	0.19.36		100	2500	0.40.54

In order to simulate the distribution of volumes of overtopping waves in an easy manner, fixed volumes were used: 50; 150; 400; 700; 1000; 1500; 2500 and 3500 l per m width. Each distribution resulted in a number to be simulated for each of these fixed volumes. An example for 2 hours of simulation of 10 l/s per m wave overtopping discharge resulted in Table 8.1.



Picture 8.8. First waves over the dike after installation of the wave overtopping simulator

The number of events with according volumes as in Table 8.1 were then randomly listed. By using the constant discharge it was then possible to calculate for every event when this volume would be reached in the box of the simulator and when to open the valve. An example of such a list, for the first 100 overtopping waves during 10 l/s per m wave overtopping discharge is given in Table 8.2. These kind of tables were provided for all wave overtopping conditions. With such a list the real testing of the dike at Delfzijl could start. First waves went over the dike during the set-up of the whole installation on 16 February, see Picture 8.8 for an impression. Real tests started after the opening event on 27 February.

In general the procedure was to start with a small overtopping discharge and to increase it in steps to a maximum. The first step was an overtopping discharge of 0.1 l/s per m. As this small discharge means only 9 overtopping waves in 6 hours (see Figure 2.2), with in average an interval between overtopping events of about 40 minutes, it was decided to speed up the test. Actually, a discharge of 1 l/s per m was used to fill the overtopping simulator. But the volumes were released according to the distribution for 0.1 l/s per m (Figure 2). Table 8.3 gives the operating list and shows that a “six hours test duration” could be speeded up to only 35 minutes.

Table 8.3. Operating list for 0.1 l/s per m, speeded op to 1 l/s per m filling discharge

Test 0.1 l/s per m		
Wave number	**Volume (liters/m)**	**Open min.s**
1	150	2.28
2	50	3.18
3	400	9.57
4	50	10.48
5	700	22.27
6	150	24.57
7	50	25.48
8	400	32.27
9	150	34.58

The steps of 1; 10; 20 and 30 l/s per m were performed in steps of 6 hours each. It appeared that even after all these steps the original dike section showed hardly any damage. The design of the wave overtopping simulator, however, was based on a maximum discharge of 30 l/s per m. It was decided to upgrade the simulator to 50 l/s per m, although this gave limitations due to the maximum size of the box (3.5 m^3/m) and the maximum pump discharge possible (40 l/s per m).

The limitation in size of the box of the simulator will be discussed first. As in Sections 2.1 and 2.2, the boundary conditions for overtopping calculations are a wave height of 2 m at the toe of the dike, a peak period of 5.7 s and a mean period of 4.7 s. The 2% run-up level on a slope of 1:4 is then 4.0 m with a mean overtopping discharge of 0.74 l/s per m. In order to get a wave overtopping discharge of 50 l/s for these wave conditions, the free crest board should be decreased to only 1.76 m. For a storm duration of 6 hours the following characteristics are found:

H_s = 2 m; T_p = 5.7 s; T_m = 4.7 s

	q = **50** l/s per m
Percentage overtopping	46.98
Number of waves in 6 hours	4600
Number of overtopping waves	2159
Maximum overtopping volume	6364 l/m

It appears that almost half of the waves will reach the crest and will overtop for this condition (47%). The maximum volume in 2159 waves is 6.3 m^3 per m and this is 80% more than the capacity of the box of the simulator (3.5 m^3 per m). The limitation of the simulator is that the maximum volume in one wave can not be more than 3.5 m^3 per m. This will elaborated more in depth below.

Figure 8.1 gives the complete distribution of overtopping volumes for 50 l/s per m overtopping and a proposal for simulation. With a mean discharge of 50 l/s per m it is not possible to simulate small volumes like 50 or 150 l per m, as this means that the simulator should open every second or 3 seconds. In reality it is possible to have many small volumes, as then the mean overtopping discharge is not constant as for the pump discharge. The only possibility for simulation is that all the small overtopping volumes are simulated by a smaller number of volumes with for instance 400 l per m. The distribution gives 1335 waves with a volume smaller than 400 l per m and 1524 waves with a volume smaller than 550 l per m (the middle between 400 and 700 l per m). All 1524 waves with a volume smaller than 550 l per m were modelled by 648 waves with a volume of 400 l per m. These 648 waves contain the same total volume of water as the 1524 waves in the distribution (274 m^3 per m in total).

Another problem in Figure 8.1 is that the maximum volume can not be simulated. In order to look more in detail, Figure 8.2 has been made.

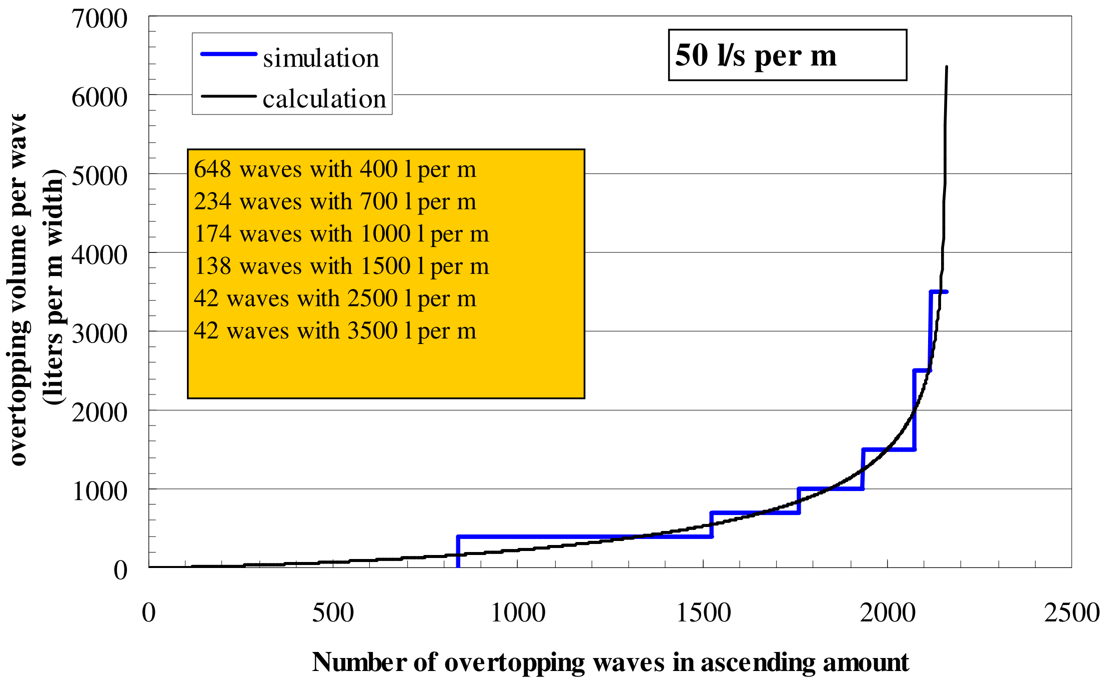


Figure 8.1. Calculated distribution of overtopping volumes and proposal for simulation. Mean discharge q = 50 l/s per m; full distribution

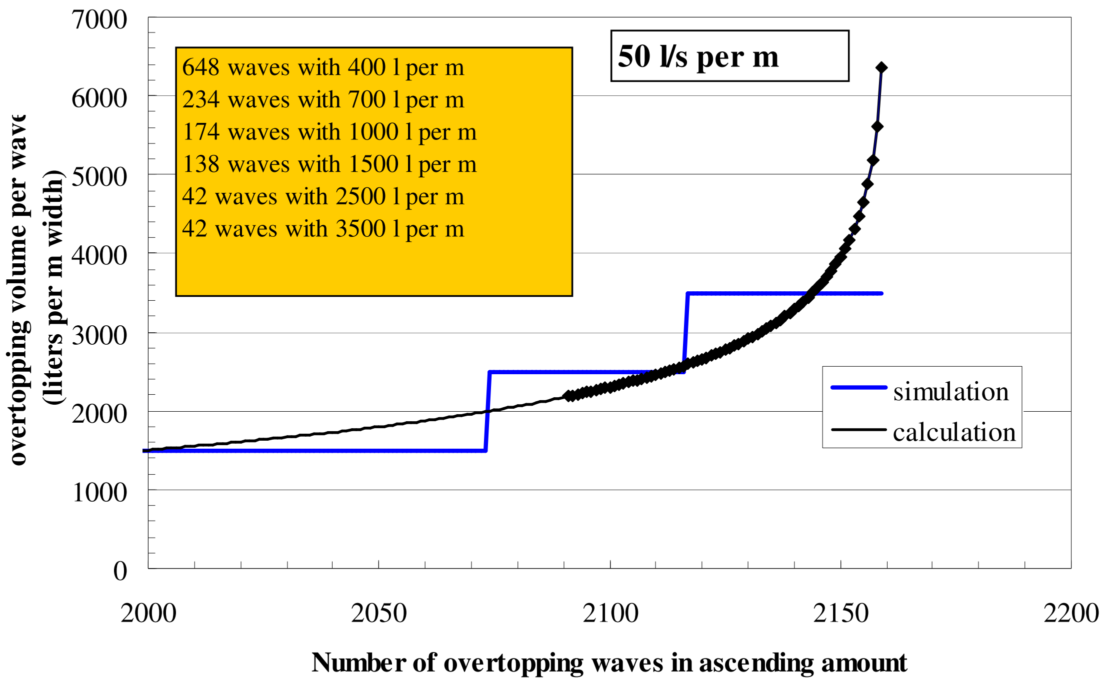


Figure 8.2. Calculated distribution of overtopping volumes and proposal for simulation. Mean discharge q = 50 l/s per m; full distribution of largest volumes

In total only 15 overtopping waves have a volume larger than 3.5 m^3 per m and only 5 overtopping waves have a volume larger than 4.5 m^3 per m. These waves can not be simulated by the wave overtopping simulator due to the capacity of the box of the simulator. In order to compensate for missing these large volumes, a fairly large number of 3.5 m^3 per m volumes was taken. Roughly all volumes between 2.5 m^3 and 3.5 m^3 per m were modelled as 3.5 m^3 per m. In total 27 volumes of 3.5 m^3 per m were taken to simulate volumes between 2.5 m^3 and 3.5 m^3 and 15 volumes of 3.5 m^3 per m were taken to simulate the largest volumes between 3.5 m^3 and 6.3 m^3 per m. Also 42 volumes of 2.5 m^3 per m were taken to simulate volumes between 2 m^3 and 2.5 m^3 per m, see Figure 8.2. Overall it can be concluded that missing the real large overtopping volumes in the distribution was compensated by simulating in excess a number of 3.5 m^3 and 2.5 m^3 per m volumes. Therefore, the maximum size of the box of 3.5 m^3 per m was not considered as a real restriction to perform a 50 l/s per m test adequately.

Another problem was that the pump could not realize a discharge over 40 l/s per m. A solution to cope with this is similar to the solution to “speed up” the small overtopping discharge of only 0.1 l/s per m, see Table 8.3. There the time between overtopping events was shortened by increasing the pump discharge. In that way a duration of 6 hours could be reduced to only 35 minutes. A similar approach was followed for the large overtopping discharge of 50 l/s per m. The distribution of overtopping volumes has been given in Figures 8.1 and 8.2 and this distribution was modelled. But the time to fill the box to the required volume took a little longer due to the smaller pump discharge. Actually, the test was “slowed down” by 25% (50 : 40). The total test took, therefore, not 6 hours, but 7.5 hours and the damaged was measured after each 2.5 hours instead of 2 hours.

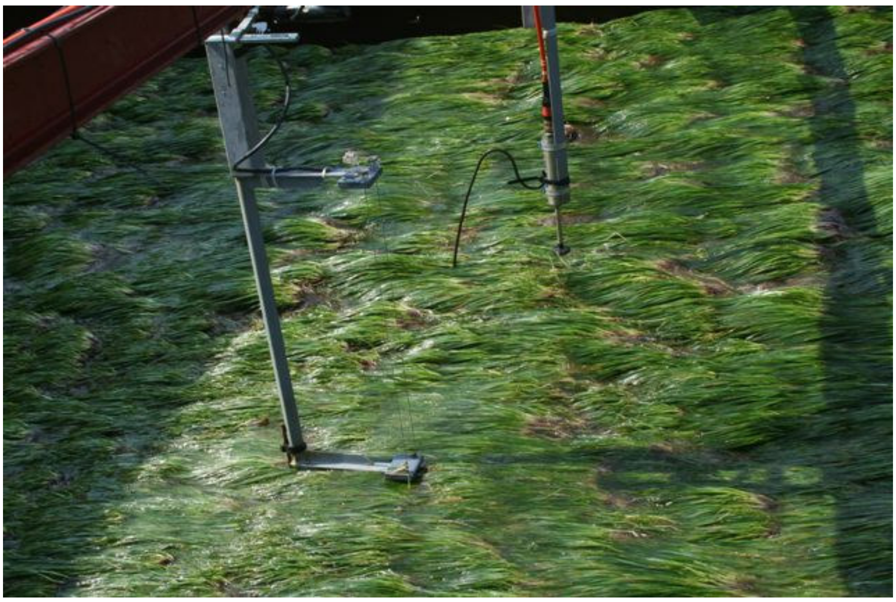
With the method described in this section operating lists were made for testing conditions of 0.1; 1; 5; 10; 20; 30; and 50 l/s per m. Each list had a duration of 1 or 2 hours and this was repeated to come to a total test duration of 6 hours.

9 Wave overtopping measurements and analysis

9.1 Measurements

WL | Delft Hydraulics developed two instruments specifically for the tests at Delfzijl, in order to measure flow velocity and flow depth on crest and inner slope of the dike. They were calibrated/validated on 18 December when the construction of the simulator was finished and the simulator was tested with water, see Section 7.2. Pictures 9.1 and 9.2 show the instruments at the test section. The first picture shows a steel beam that was placed over the full width of the 4 m wide test section and the instruments were mounted to this steel beam. Two beams were placed, one at or close to the crest of the dike and one half way the inner slope (Picture 9.2).

Picture 9.1. Steel beam over test section with instruments mounted to it.



Picture 9.2. A wire flow depth meter (foreground) with an electromagnetic velocity meter. Flow of water goes from right to left in the picture.

The EMS (electromagnetic velocity meter) was placed 2 cm centimetres above the soil, and after the first overtopping waves, also above the grass. The wires of the flow depth meter were mounted in a U-frame in order not to disturb the water before the flow depth was measured. The U-frame was mounted 3 cm above the soil in all tests, which means that the actual flow depth was 3 cm more than measured. Later, the height of the EMS was changed to 5 centimetres and in one situation even to 7 cm above the soil.

During most tests the instruments were present, but at the end of the test and certainly for the 50 l/s per m tests it was decided to remove or not to place the instruments as damage to the instruments was to be expected.

The beams with instruments were placed at or in the neighbourhood of the crest and half way the slope (at different locations for the natural grass section and the SGR-section). Measurements were performed for the test section with normal grass (un-reinforced section) and for the SGR, the reinforced section. No measurements were performed for the test on bare clay. An overall view of measurements, locations of instruments and processed measurements is given in Table 9.1.

Table 9.1. Overall view of velocity and flow depth measurements

Normal grass; un-reinforced section					
Processed record	Measured records	Mean overtopping discharge q (l/s per m)	Height of EMS above soil (m)	Location crest (m from inner crest line)	Location slope (m from inner crest line)
03	03	0.1	0.02	0.40	8.00
05	04 - 06	1	0.02	0.40	8.00
07	07	10	0.02	0.40	8.00
08	08 – 10	20	0.02	0.40	8.00
11	11	30	0.02	0.40	8.00
SGR; reinforced section					
Processed record	Measured records	Mean overtopping discharge q (l/s per m)	Height of EMS above soil (m)	Location crest (m from inner crest line)	Location slope (m from inner crest line)
16	14 – 16	10	0.07	2.20	9.20
19	17 – 19	20	0.05	2.20	9.20
20; 23	20 – 23	30	0.05	2.20	9.20
24	24	50	0.05	2.20	9.20

9.2 Analysis of measurements

9.2.1 Processing by WL | Delft Hydraulics

Records of flow velocity and flow depth at crest and inner slope were sent to WL | Delft Hydraulics for further processing. After some processing a meeting was held, results of the processing discussed and the final way of processing decided.

A part of a raw record of flow depth is given in Figure 9.1. The horizontal axis is given in sample number, which means that with a sampling frequency of 25 Hz every 100 samples is equal to

4 s. The vertical axis gives the flow depth in cm. In reality the flow depth meter was placed 3 cm above the soil and, therefore, this 3 cm was added to the signal to give the correct flow depth.

In order to eliminate the short peaks the signal was filtered with a Finite Impulse Response filter (FIR), which is a kind of smoothing average, but more accurate. The upper signal in Figure 9.1 gives the filtered signal, which was used for further processing.

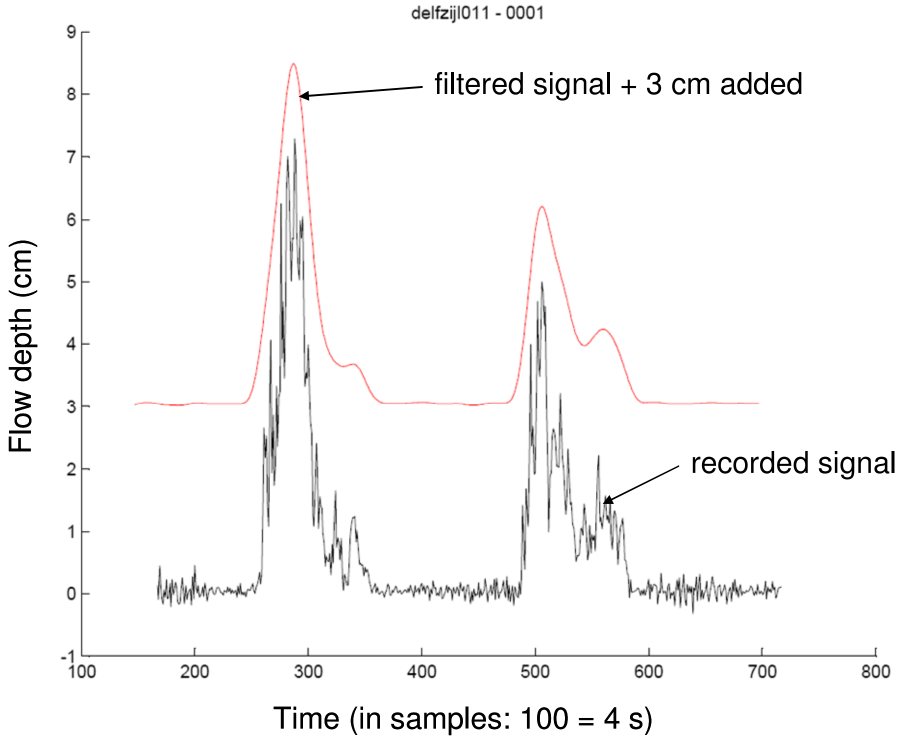


Figure 9.1. Example of a raw signal and filtered with a FIR filter.

The duration of each overtopping event, T_{ovt}, was calculated from the filtered record of the flow depth. This is the duration that the flow depth is larger than 3.5 cm. For flow depths between 3.0 cm and 3.4 cm the total number of events was not constant. Between 3.4 cm and 3.8 cm the same number of overtopping events was found and for this reason the value of 3.5 cm was chosen.

Then the maximum flow depth and maximum velocity were determined for each overtopping event. Finally, an integration was made over the period of the overtopping event and the product of flow depth and velocity, giving the total volume of the overtopping event:

$$V = \int_0^{T_{ovt}} v \cdot h \cdot \Delta t$$

A table was made with the overtopping events in chronological order and for each event the start time of the event, the duration, the maximum flow depth, the maximum velocity and the calculated volume by integration. This was done for the measurements near the crest and half way the inner slope. Table 9.2 gives an example of the first 500 s of the test record 11 with 30 l/s per m. The actual released volumes are shown in the last column. It should be noted that in this 500 s, two volumes of 150 l were *not* detected during processing.

Table 9.2. Part of processed record 11. First 500 s of test.

At crest					At inner slope					
Start time	Duration (T_{ovt})	Max. flow depth	Max velocity	Integral $\int_0^{T_{ovt}} v \cdot h \cdot \Delta t$	Start time	Duration (T_{ovt})	Max. flow depth	Max velocity	Integral $\int_0^{T_{ovt}} v \cdot h \cdot \Delta t$	Volume produced V
[s]	[s]	[m]	[m/s]	[m^3/m]	[s]	[s]	[m]	[m/s]	[m^3/m]	[m^3/m]
4.94	6.82	0.085	3.24	0.432	7.26	4.68	0.046	1.93	0.208	0.150
21.78	2.42	0.080	3.17	0.297	23.86	5.16	0.053	2.15	0.301	0.400
44.60	2.90	0.102	3.08	0.504	46.30	2.16	0.068	2.12	0.218	0.700
58.74	2.66	0.105	3.22	0.444	60.66	7.56	0.059	2.28	0.394	0.400
90.58	3.44	0.097	3.66	0.555	92.28	6.98	0.079	2.11	0.541	1.000
97.78	2.42	0.076	2.87	0.247	99.92	5.46	0.052	1.75	0.260	0.150
103.04	2.04	0.072	3.02	0.209	105.40	5.56	0.044	1.76	0.225	0.150
124.68	2.84	0.108	3.11	0.424	126.44	7.20	0.070	2.21	0.467	0.700
138.80	2.92	0.098	2.96	0.443	140.76	7.20	0.061	2.35	0.410	0.400
161.84	3.12	0.119	3.43	0.572	163.56	6.82	0.067	2.41	0.432	0.700
168.00	2.24	0.073	2.49	0.218	170.40	5.52	0.046	2.08	0.209	0.150
173.46	2.22	0.073	1.90	0.206	175.94	6.12	0.043	1.20	0.169	0.150
221.26	3.54	0.133	3.87	0.957	222.74	8.98	0.104	2.17	0.745	1.500
244.76	2.86	0.127	3.25	0.597	246.42	9.42	0.066	2.05	0.503	0.700
277.96	3.30	0.120	3.56	0.659	279.56	8.62	0.085	2.50	0.616	1.000
296.48	3.02	0.109	3.13	0.447	298.30	7.00	0.064	2.16	0.397	0.400
302.94	2.28	0.066	2.48	0.196	305.32	6.82	0.045	1.73	0.267	0.150
333.80	3.28	0.105	3.70	0.681	335.36	10.00	0.082	2.27	0.643	1.000
358.78	3.26	0.118	3.47	0.681	360.46	9.84	0.074	1.96	0.518	0.700
371.92	2.90	0.112	3.05	0.404	373.96	6.30	0.055	2.02	0.357	0.400
377.76	2.26	0.070	2.54	0.201	380.28	8.50	0.047	1.42	0.280	0.150
390.10	2.74	0.102	3.17	0.387	392.14	7.00	0.053	2.01	0.325	0.400
398.78	2.54	0.068	2.48	0.305	400.96	8.22	0.046	1.46	0.311	0.150
417.58	3.02	0.131	3.33	0.557	419.40	8.90	0.066	2.55	0.457	0.700
432.42	3.14	0.096	3.20	0.474	434.34	6.06	0.056	2.26	0.340	0.400
437.94	2.74	0.065	2.57	0.232	440.42	8.58	0.045	1.65	0.298	0.150
485.20	3.78	0.138	3.36	1.030	486.72	0.00	0.034	0.26	0.000	1.500
499.34	2.78	0.097	2.60	0.361	501.34	6.82	0.056	1.88	0.304	0.400

In total 10 records were processed in this way, see Table 9.1. The analysis is given in this Section and Figures 9.2 – 9.23 give all data graphically. First actual overtopping volumes are compared with volumes determined from the integration of velocity and flow depth, as described above. Then maximum flow depths are given, first as a function of the actual overtopping volume which was released from the simulator. Then maximum velocities are described. An alternative way of measuring velocities is to look at the time difference in start of an overtopping record at the crest and at the inner slope. This gives an average velocity of the front of an overtopping wave over the inner slope. Finally, the overtopping flow time T_{ovt} has been given.

The next pages give the graphs with data, with two graphs on each page. At the bottom of the page some observations or conclusions have been made on these graphs.

In graphs where the actual overtopping volumes are used for the horizontal axis, the data groups for each mean overtopping discharge have been shifted a little in order to make a clear division between the overtopping discharges. For example, data for 1000 l per m have been given at 920 l/m for 0.1 l/s per m, at 940 l/m for 1 l/s per m, at 970 l/m for 10 l/s per m, at 1000 l/m for 20 l/s per m and at 1030 l/m for 30 l/s per m.

In some cases data have been compared with prediction curves. These have not been taken from Van Gent (2002) or Schüttrumpf (2202), see Chapter 2, but from the final result of the analysis by Gijs Bosman, see Chapter 10.

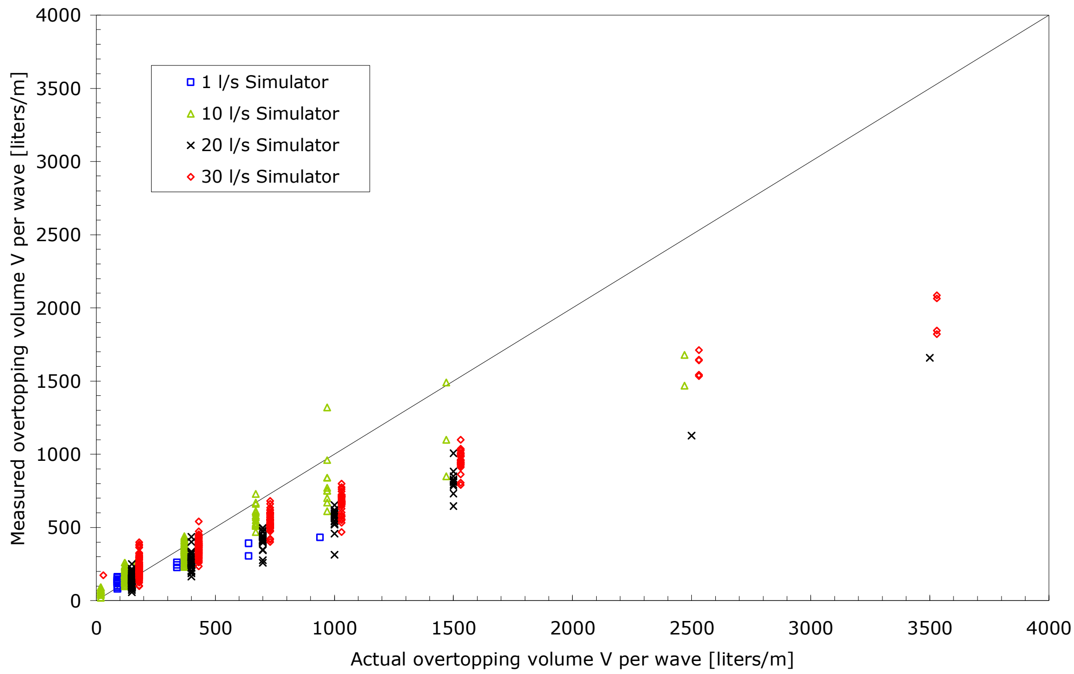


Figure 9.2. Actual overtopping volumes versus calculated from integration of velocity and flow depth – natural grass section

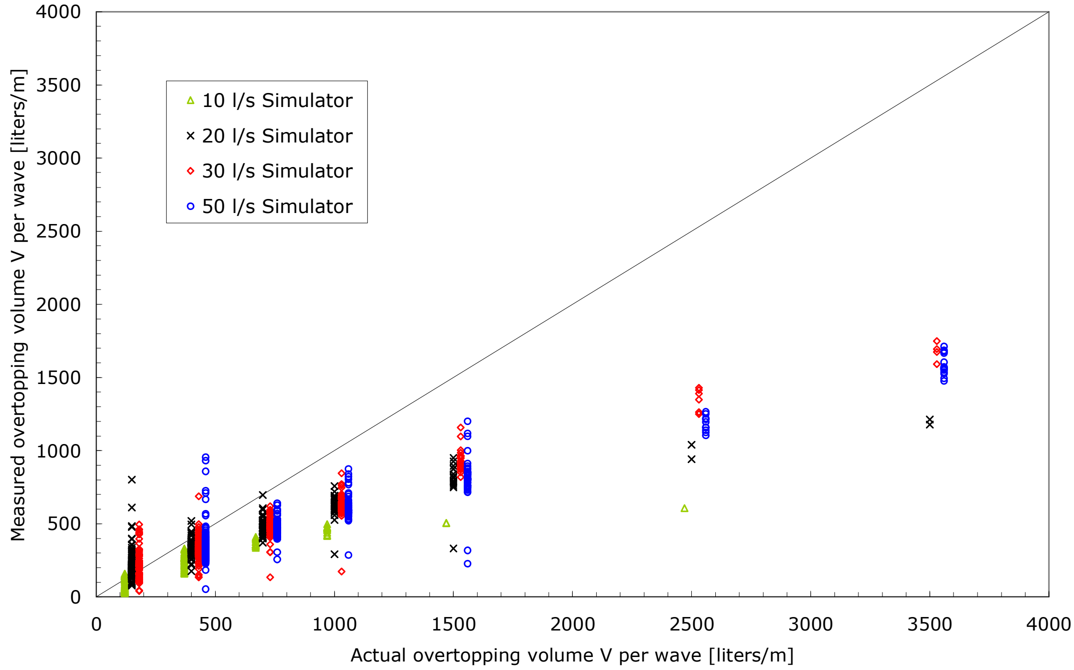


Figure 9.3. Actual overtopping volumes versus calculated from integration of velocity and flow depth – reinforced SGR section

Calculated and actual volumes are more or less equal for volumes between 150 and 400 l/m. For larger volumes, the integration gives a too low value. For the large overtopping volumes of 2500 l/m and 3500 l/m the volume by integration is about half of the actual volume.

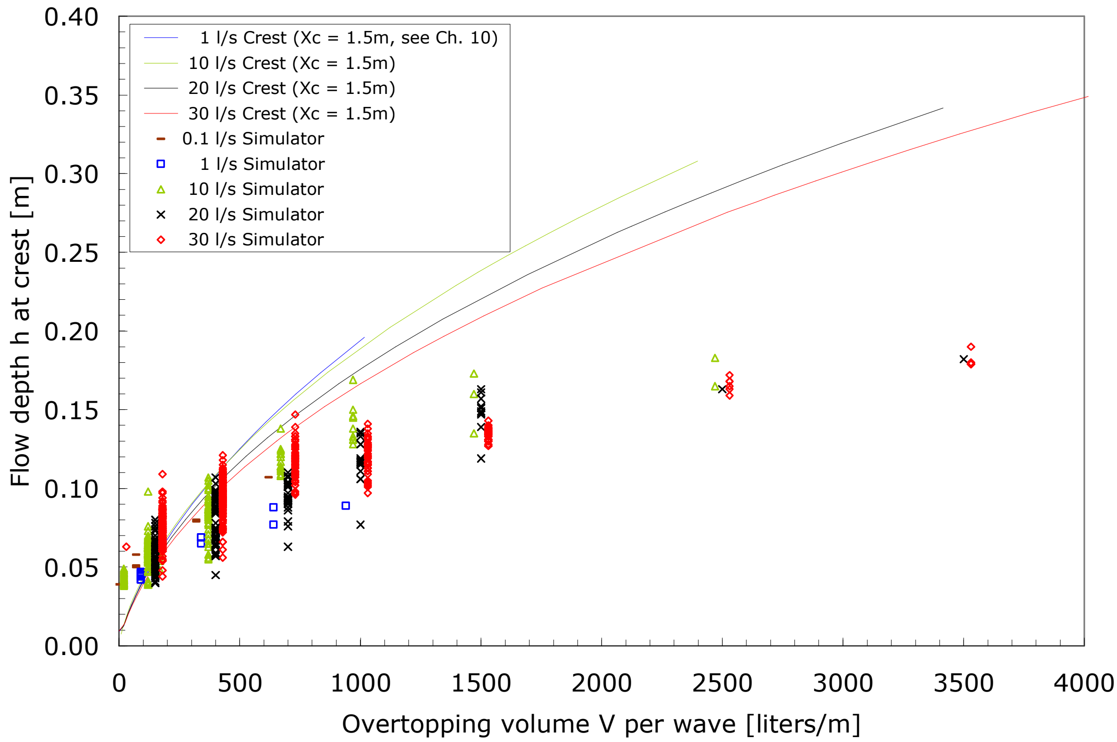


Figure 9.4. Flow depths h at the crest – natural grass section; Curves, see Ch. 10.

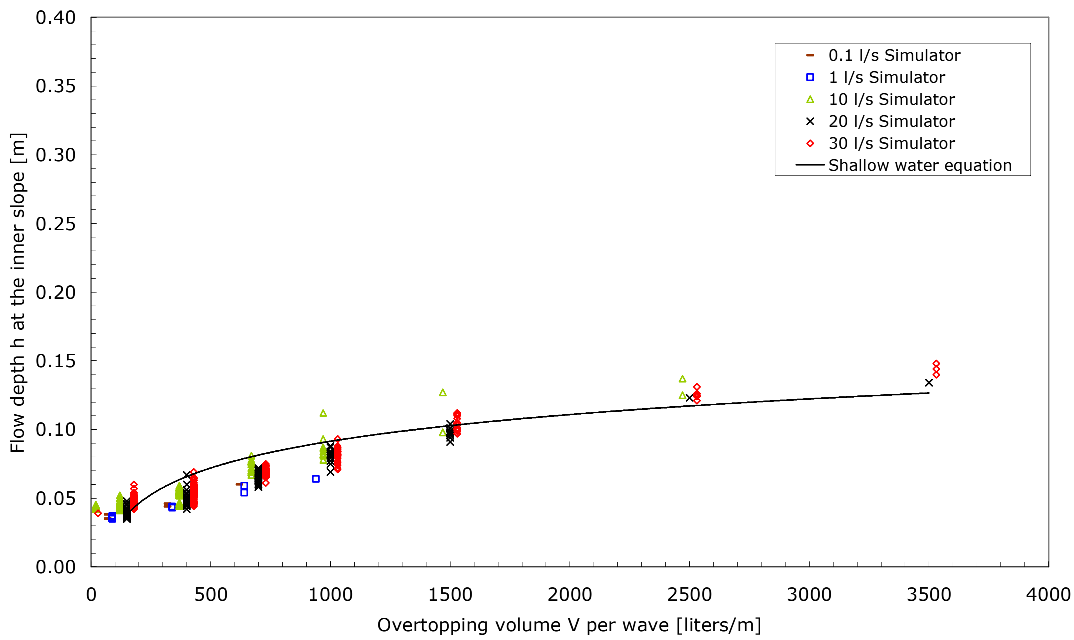


Figure 9.5. Flow depths h at inner slope – natural grass section; Curve, see Ch. 10.

Flow depths at the crest are smaller than predicted (see Chapter 10 for the prediction curves), certainly for the larger volumes. Only up to 400 l/m flow depths are more or less equal to the prediction line. The flow depths at the inner slope are consistently smaller than at the crest. The scatter of data is larger at the crest than at the inner slope. This is consistent with the shallow water equation (see Chapter 10).

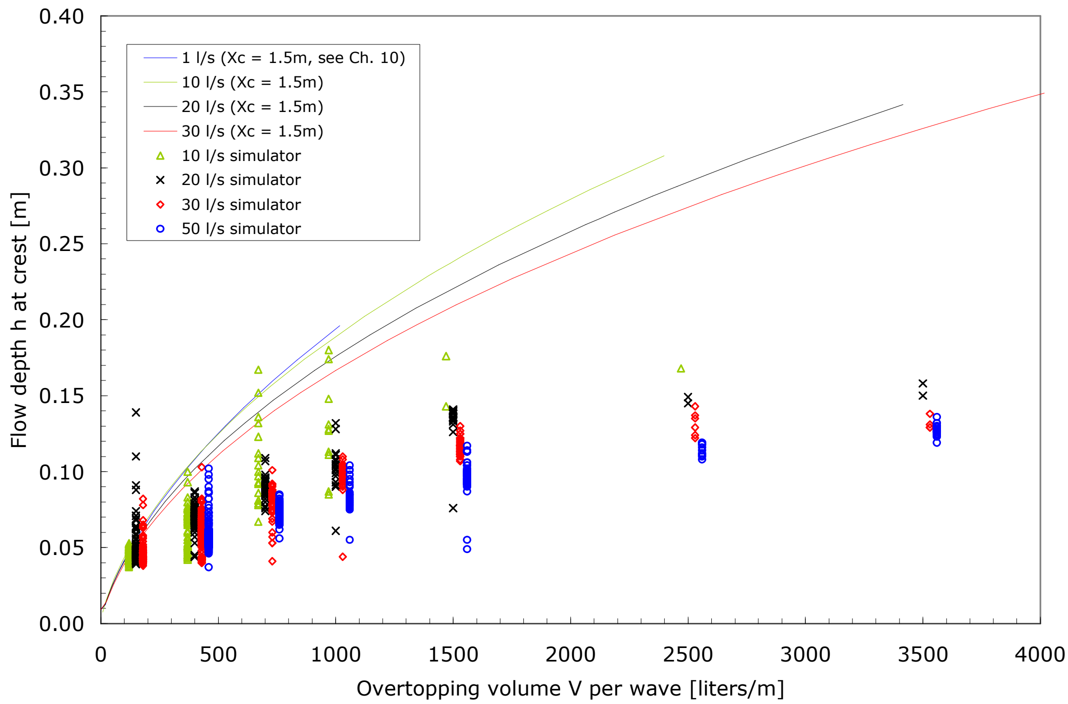


Figure 9.6. Flow depths h 2.2 m from the crest – reinforced SGR section; for curves see Ch. 10.

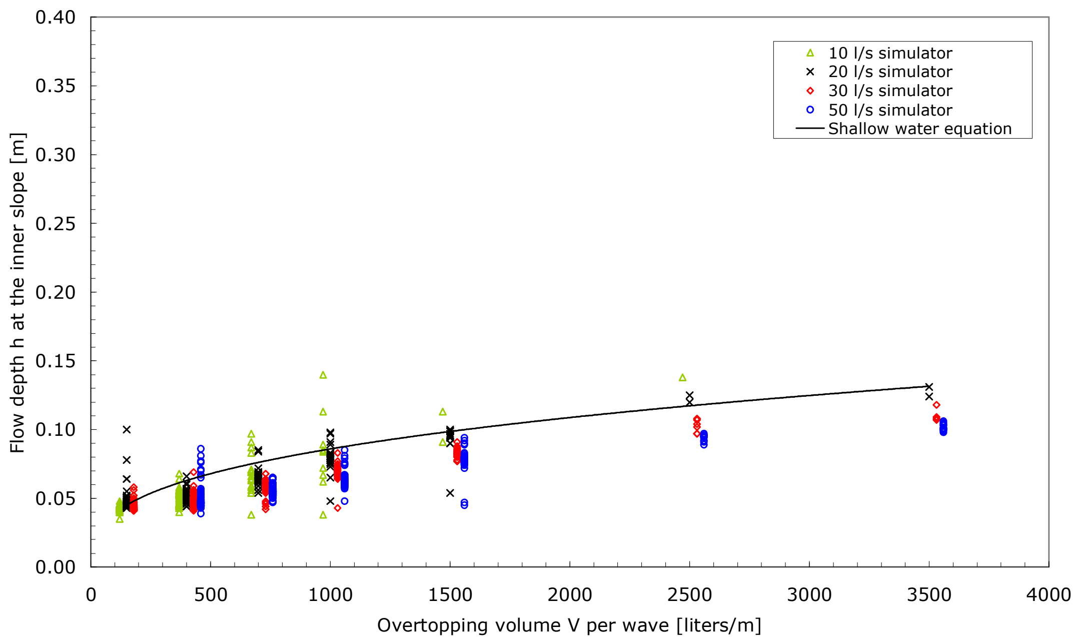


Figure 9.7. Flow depths h at inner slope – reinforced SGR section; for curve see Ch. 10.

Similar conclusions as for Figures 9.4 and 9.5. The measured flow depths for 3500 l/m are very small at the inner slope (only about 0.10 - 0.12 m), but more or less as predicted by the shallow water equation (see Chapter 10).

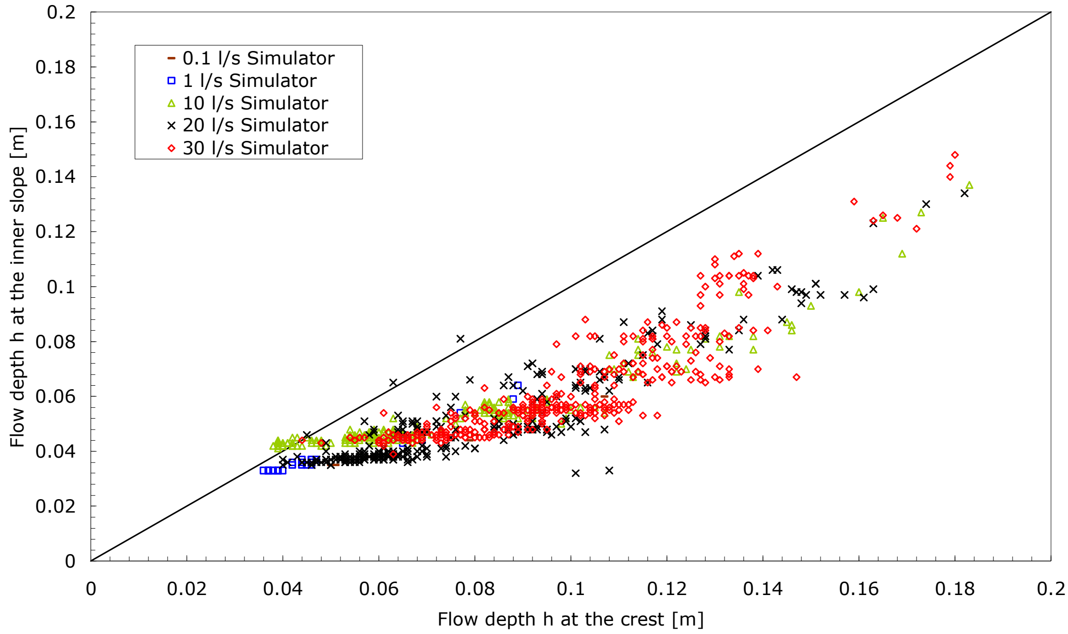


Figure 9.8. Comparison flow depths at crest and inner slope – natural grass section

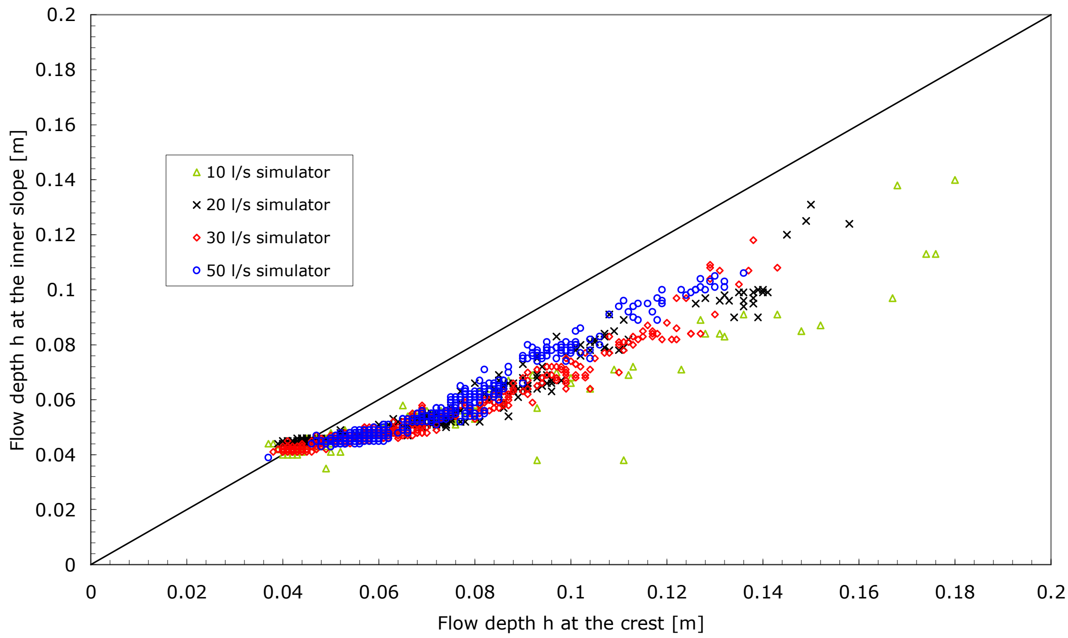


Figure 9.9. Comparison flow depths at 2.2 m from the crest and at inner slope – reinforced SGR section

The flow depth is constantly smaller at the inner slope, except for very small flow depths of 0.04 m. Then flow depths at crest and inner slope are equal. Figure 9.8 shows more scatter, maybe caused by the difference in location of the instruments at the crest: in Figure 9.8 the instruments were placed at the horizontal crest and close to the simulator. In the series for the SGR section the instruments were placed on the down slope, about 2.2 m from the crest.

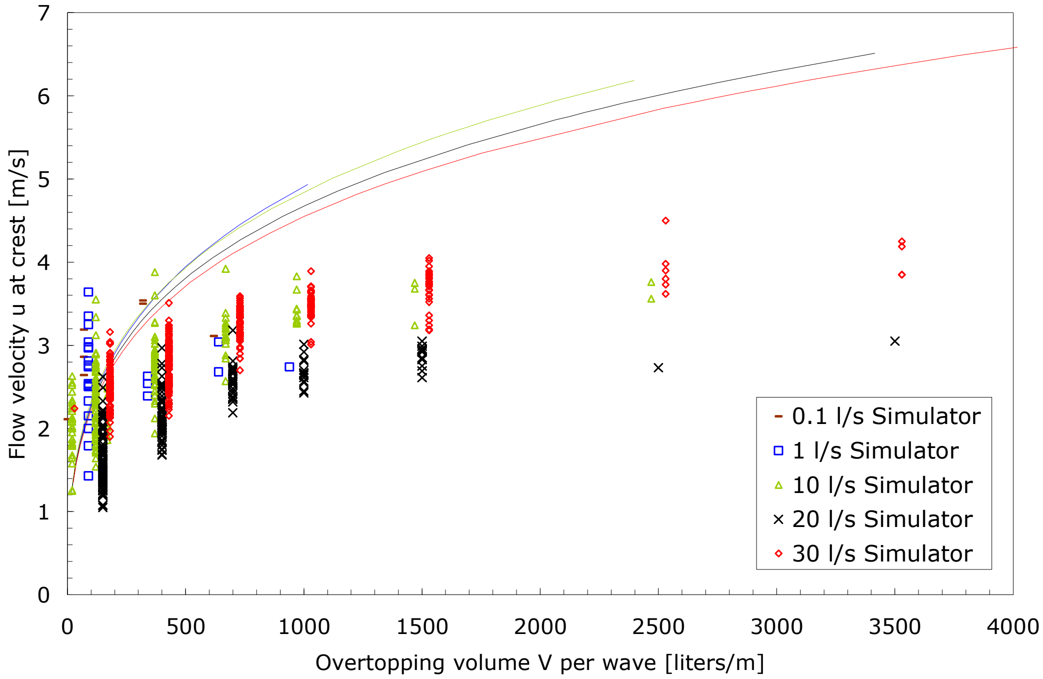


Figure 9.10. Maximum flow velocities u at the crest – natural grass section (2 cm height); for curves see Ch. 10 – data points and corresponding curves have the same colour.

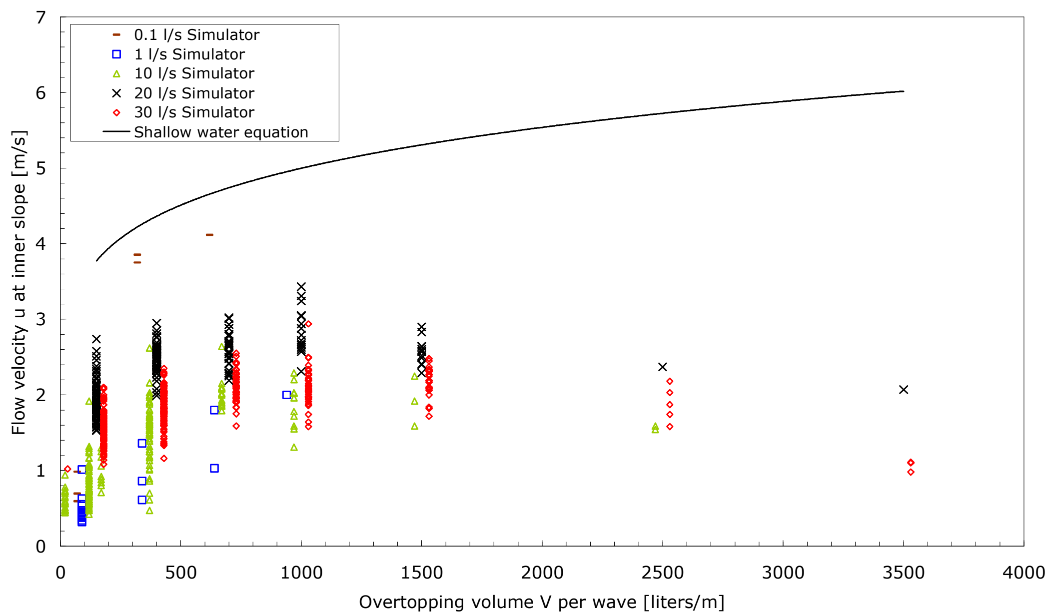


Figure 9.11. Maximum flow velocities u at inner slope – natural grass section (2 cm height); for curve see Ch. 10.

Only for volumes of 150 l/m or smaller the velocity is as expected, see comparison with the curve in Figure 9.10. For larger volumes the measured velocities are almost a factor 2 smaller than expected. In Figure 9.10 velocity increases with volume, but in Figure 9.11 the largest volumes show a decrease in velocity! Only velocities of 1 – 2 m/s were measured for volumes above 2500 l/m. This is strange and not according to reality. It seems that the 20 l/s per m test is consistently lower at the crest and higher at the inner slope.

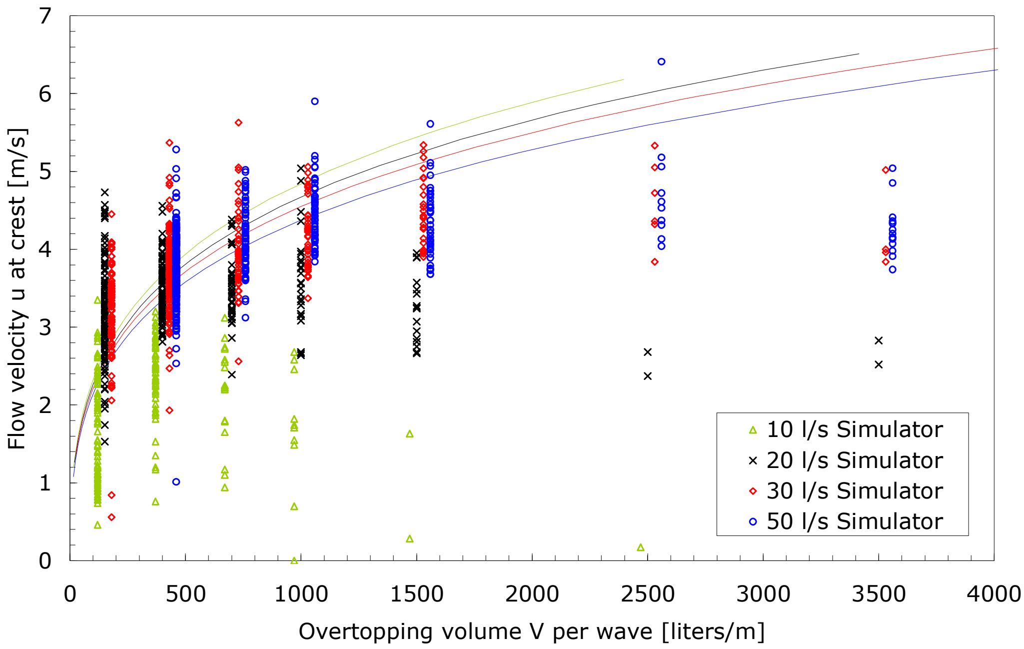


Figure 9.12. Maximum flow velocities u at 2.2 m from the crest – reinforced SGR section (5 cm height). For curves see Ch. 10 – data points and corresponding curves have the same colour.

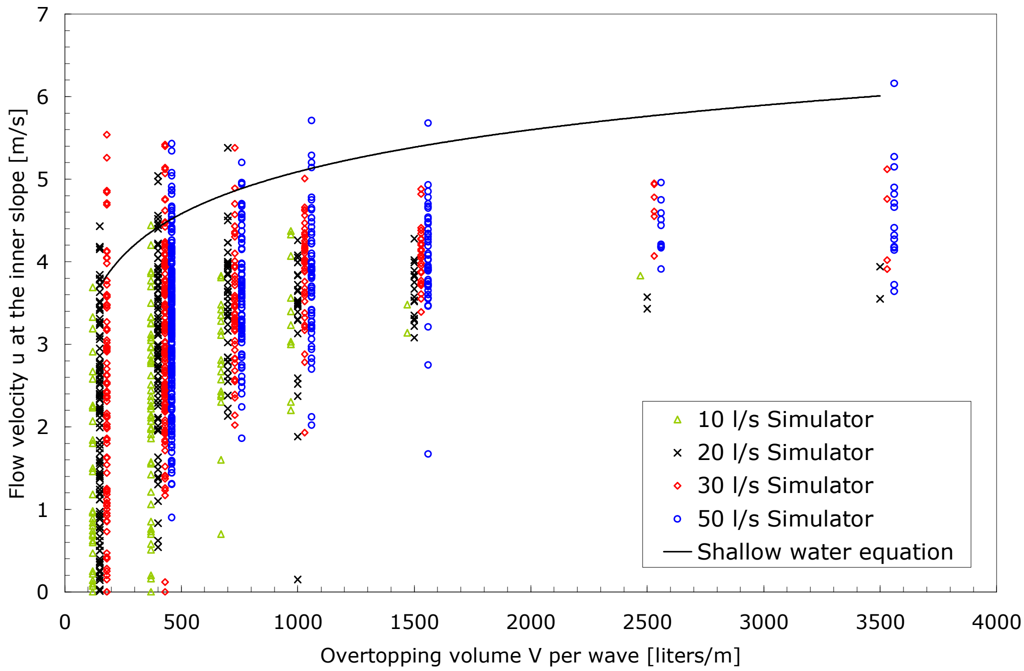


Figure 9.13. Maximum flow velocities u at inner slope – reinforced SGR section (5 cm height). For curve, see Ch. 10.

Velocities are now larger than in Figures 9.10 and 9.11, probably because the EMS was placed higher above the soil. But even here velocities stay below the predicted line for larger overtopping volumes. More or less similar velocities are found for volumes larger than about 700 l/m.

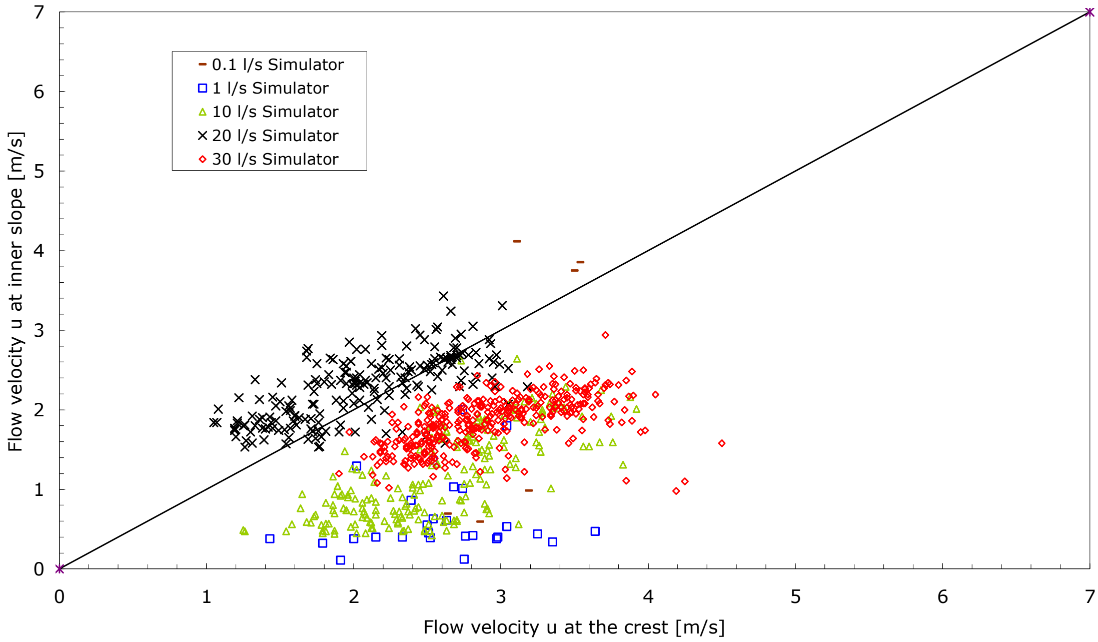


Figure 9.14. Comparison flow velocities at crest and inner slope – natural grass section (2 cm height)

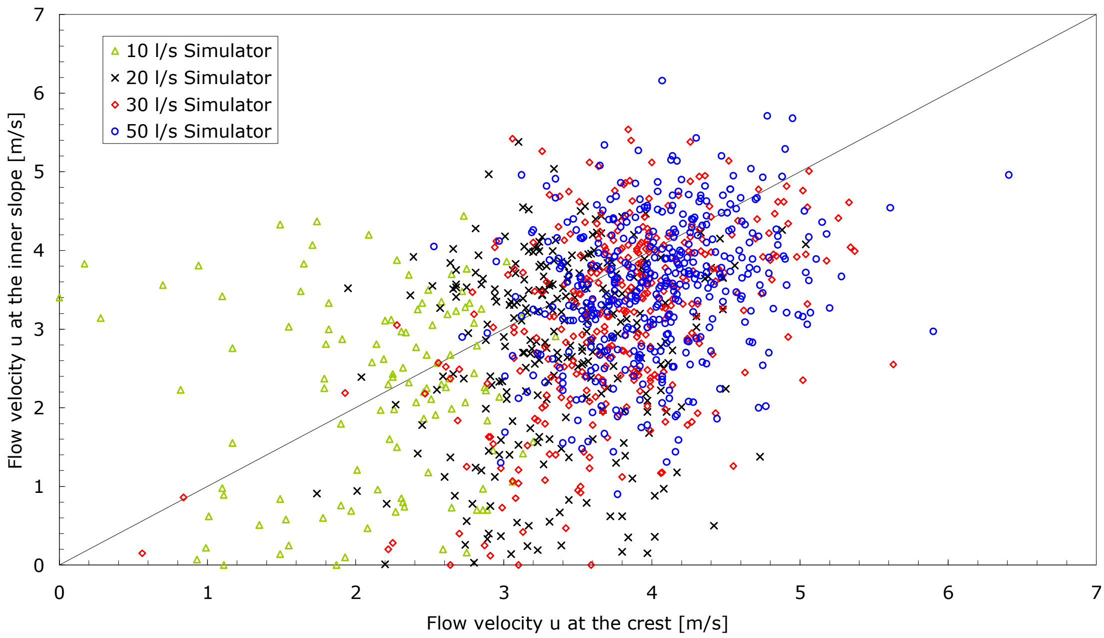


Figure 9.15. Comparison flow velocities at 2.2 m from the crest and inner slope – reinforced SGR section (5 cm height)

Figure 9.14 shows strange inconsistencies between the various overtopping discharges. Certainly the 20 l/s per m discharge shows a large deviation from the rest. In Figure 9.15 the various overtopping discharges are mixed. Here the velocities were measured at a large distance from the soil. In average, velocities at the inner slope are a little smaller than at the crest. The scatter is very large.

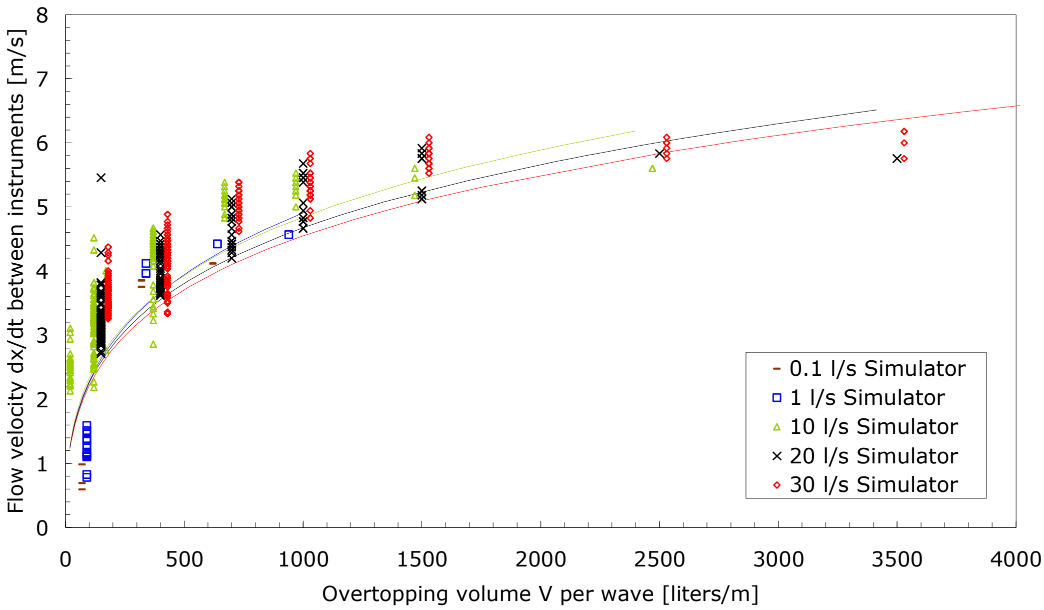


Figure 9.16. Flow velocity of overtopping front between instruments at crest and inner slope – natural grass section; see for curves Ch. 10 – data points and corresponding curves have the same colour.

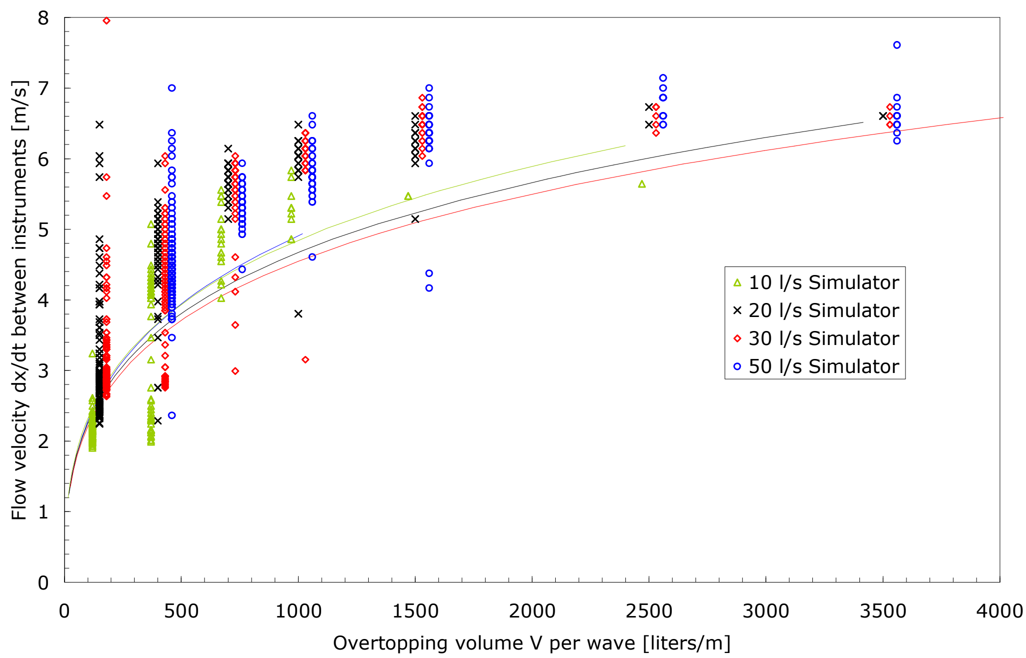


Figure 9.17. Flow velocity of overtopping front between instruments at crest and inner slope – reinforced SGR section; see for curves Ch. 10 – data points and corresponding curves have the same colour.

The front velocity of the overtopping waves is at least as high as the prediciton. The velocities are (much) higher than the directly measured velocities. Here velocities up to 7 m/s were measured. Velocity increased nicely with increasing volume, but seems to become more or less constant for the large overtopping volumes. Velocities increase at the inner slope, which has not been taken into account for the prediction lines.

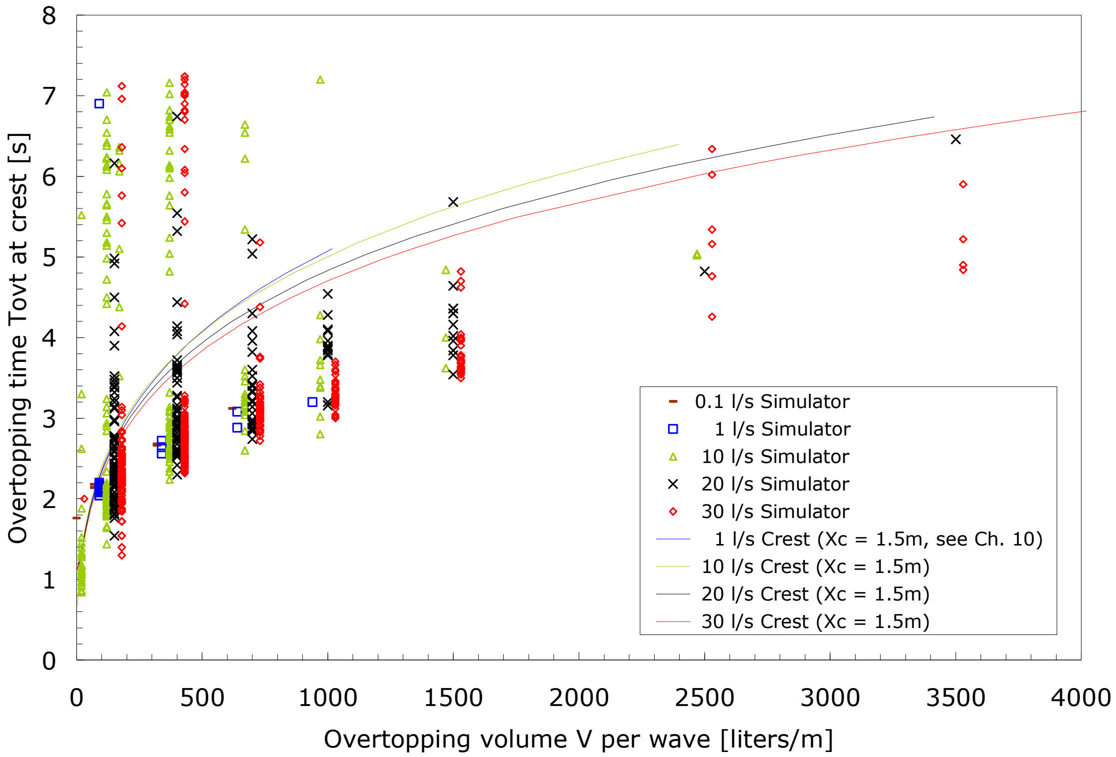


Figure 9.18. Overtopping time T_{ovt} at the crest – natural grass section; see for curves Ch. 10.

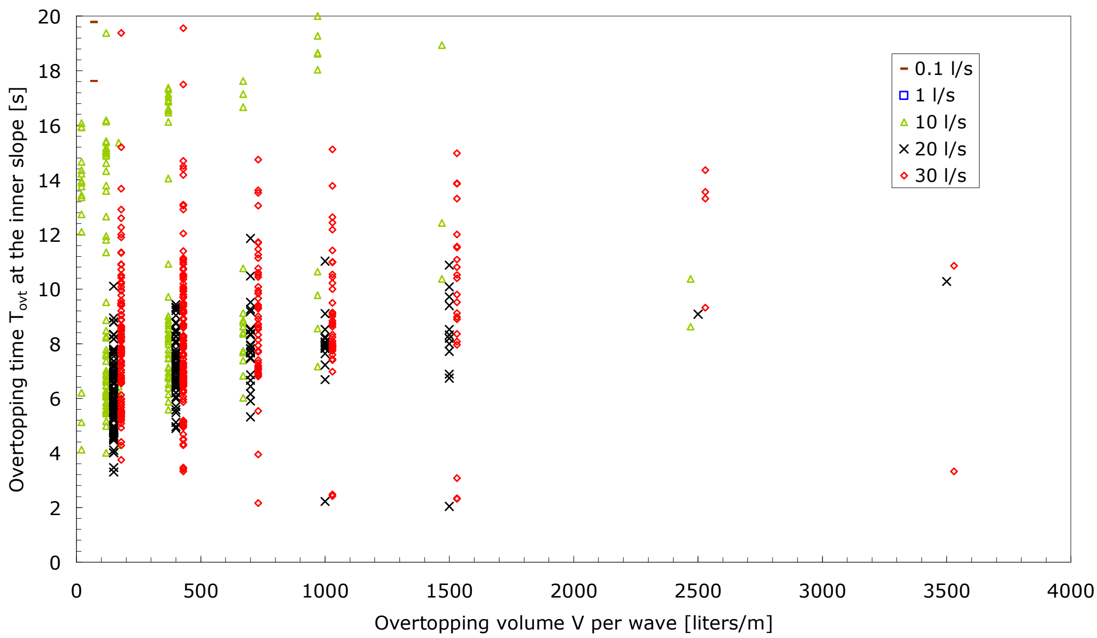


Figure 9.19. Overtopping time T_{ovt} at inner slope – natural grass section

The general trend in Figure 9.18 is that the overtopping time increases gradually from about 2 – 5.5 s if the volume increases from 150 l/m to 3500 l/m. This is more or less what was expected, although the smaller overtopping times should have been a little larger, see Chapter 10. There is a cloud of data points for smaller volumes that is about twice as expected. The situation at the inner slope is more confusing. Very large overtopping times are found sometimes.

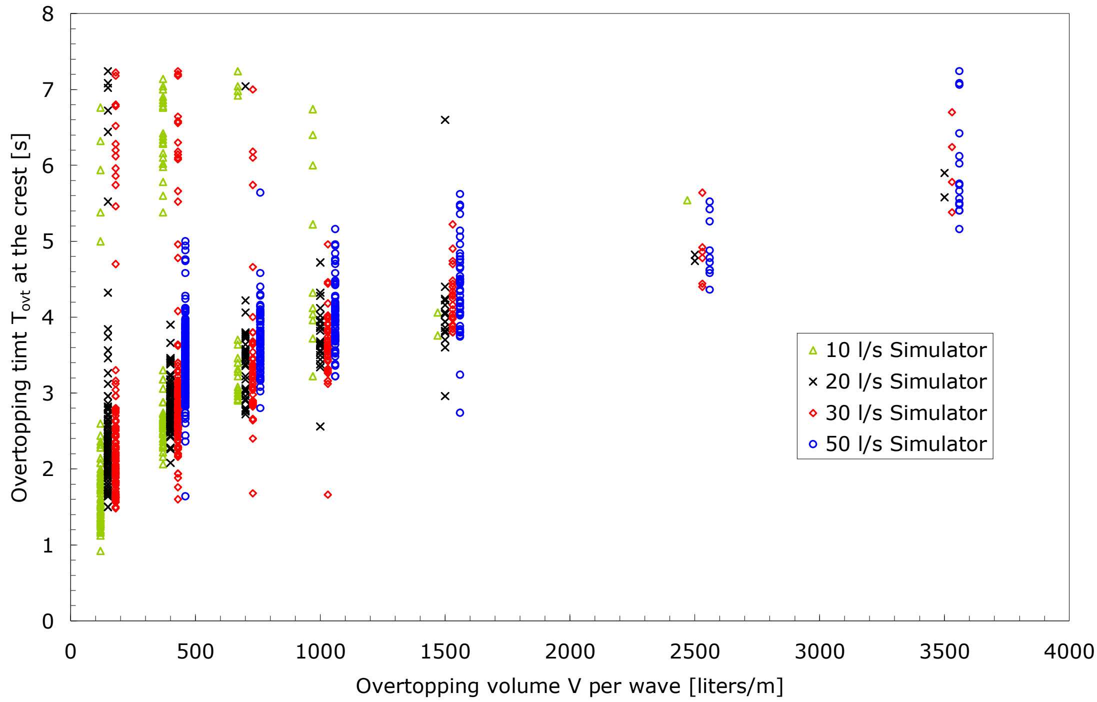


Figure 9.20. Overtopping time T_{ovt} at 2.2 m from the crest – reinforced SGR section

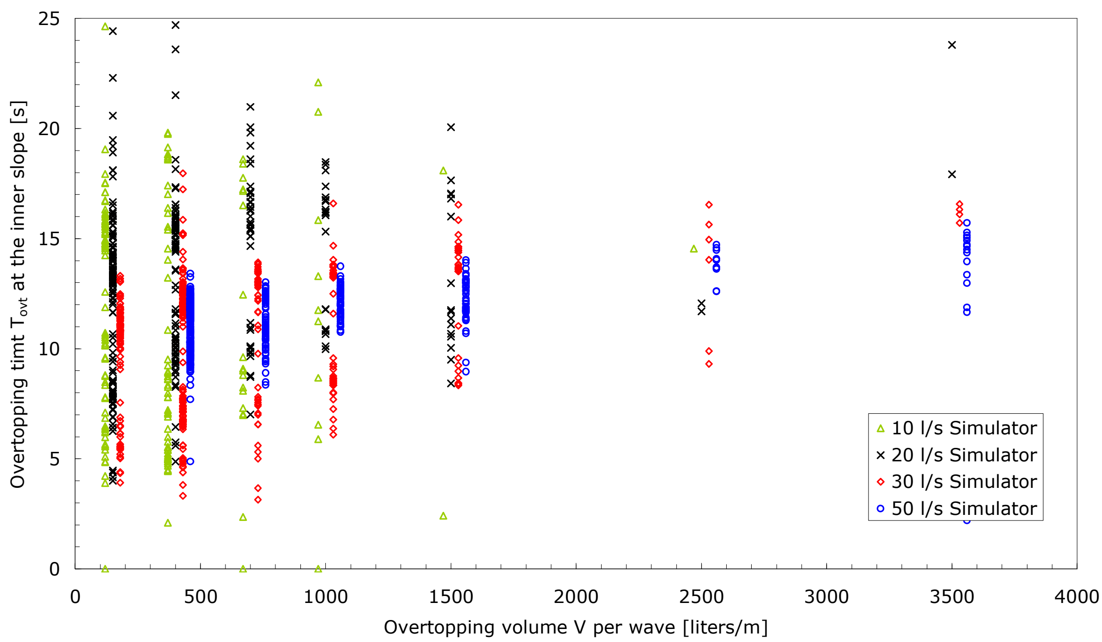


Figure 9.21. Overtopping time T_{ovt} at inner slope – reinforced SGR section

Similar conclusions as for the results on the natural grass section (Figures 9.17 – 9.18).

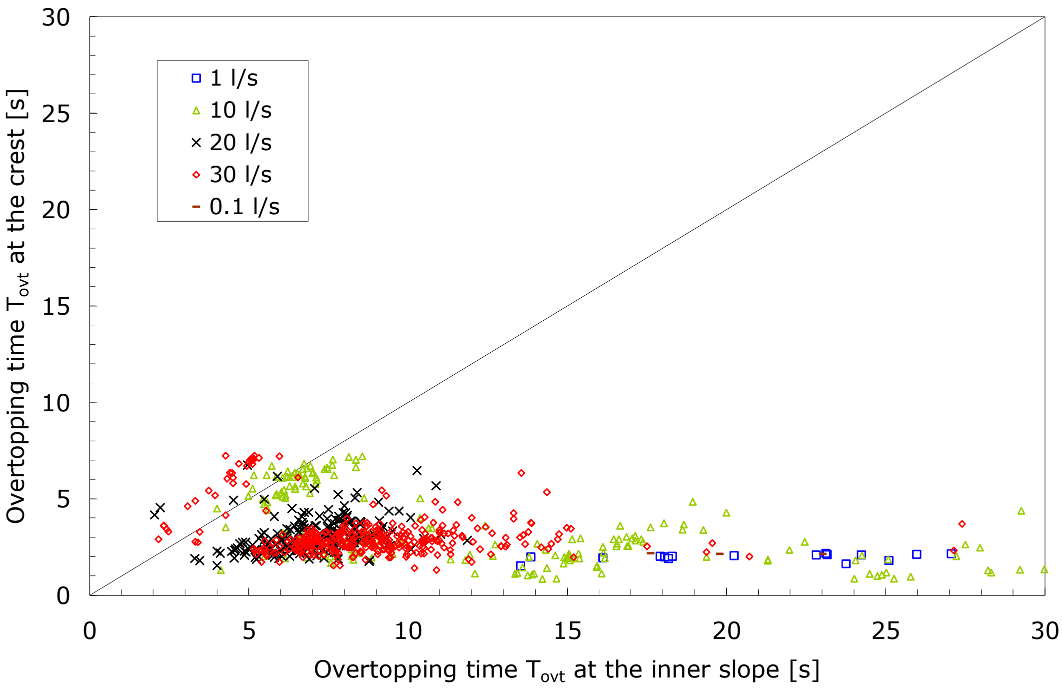


Figure 9.22. Comparison overtopping time T_{ovt} at crest and inner slope – natural grass section

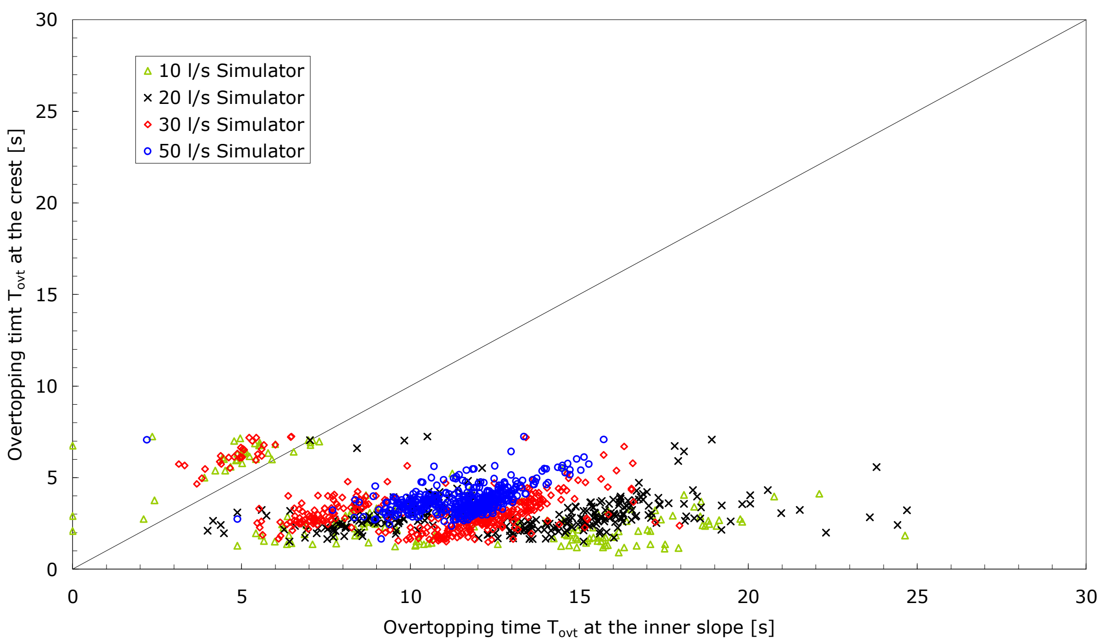


Figure 9.23. Comparison overtopping time T_{ovt} at 2.2 m from the crest and inner slope – reinforced SGR section

The conclusion is clear that in general the overtopping times at the inner slope are much larger than at or close to the crest. In combination with also decreasing flow depth and velocity along the slope, it seems that the whole process of overtopping slows down along this slope, maybe due to friction, roughness or that always a little water remains on the slope.

9.2.2 Filtering by moving average

Instead of the FIR-filtering technique as applied by WL | Delft Hydraulics in the previous section, a moving average filter technique can be used. The following was used as filter: the average of 9 time steps was calculated, being the average over 0.18 s. The analysis has been performed by Gijs Bosman as part of his MSc-thesis (Bosman, 2007). Figure 9.24 shows the results for one overtopping event, for flow depth and velocity and at the crest and half way the slope.

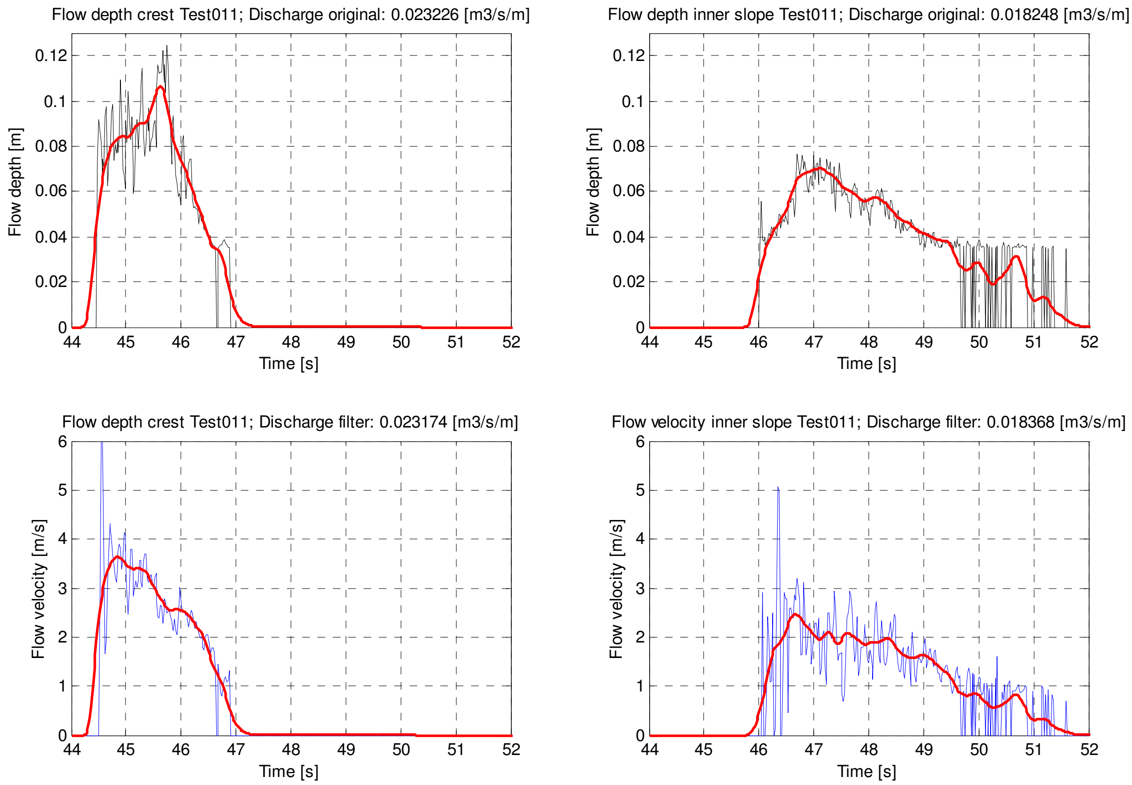


Figure 9.24. Records filtered with a moving average filter.

In order to compare the differences between the FIR-filter technique by Delft Hydraulics and the moving average filter, test 11 (Table 9.1) with 30 l/s per m for the natural grass section was elaborated. This was done for the velocity (Figure 9.25), flow depth (Figure 9.26) and the overtopping time (Figure 9.27), all at the crest.

Figure 9.25 shows that the moving average filter technique resulted in larger velocities. Now the velocities at the smaller overtopping volumes, say smaller than 700 l/m are more or less according to the predictions. But the velocities for the larger volumes are still too small.

Also the flow depth increased a little with the moving average technique, see Figure 9.28. But the difference between measurements and predictions, certainly for the larger volumes, is still large.

The moving average filter technique gives much nicer results for the overtopping time, see Figure 9.27. The cloud of large overtopping times for small volumes was not found and a nice relationship is present, showing increasing overtopping time with increasing volume. Still the measured overtopping times are a little smaller than predicted, but for the smaller volumes only.

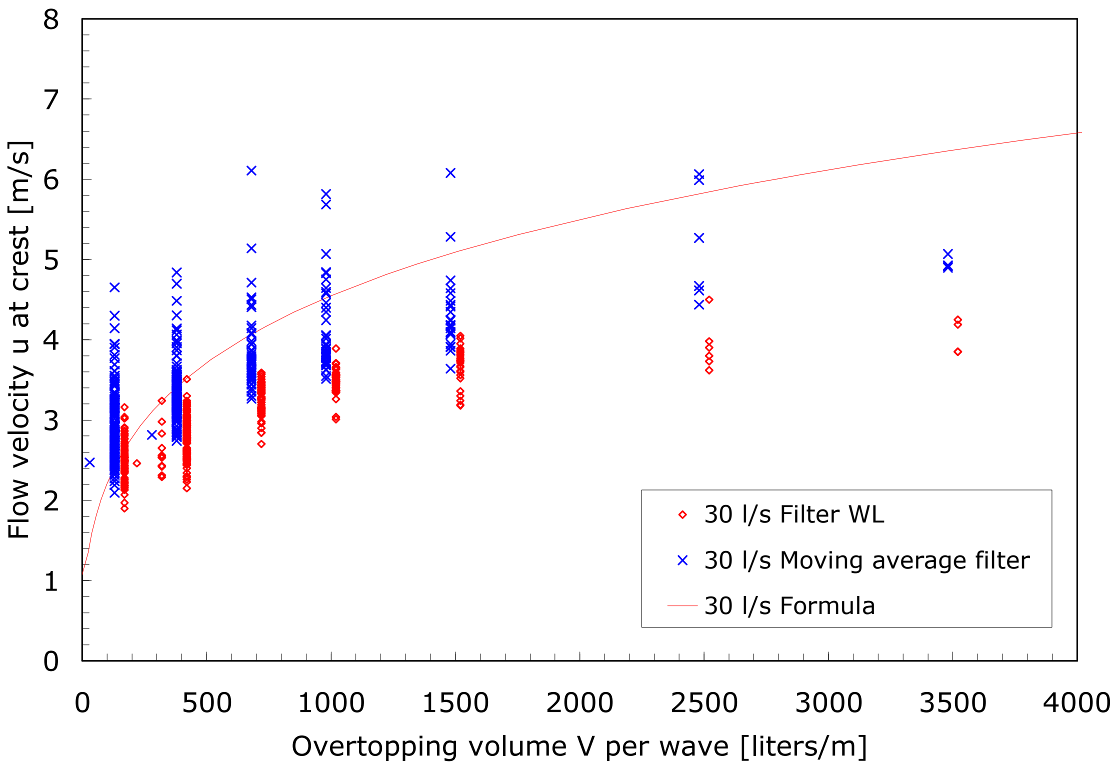


Figure 9.25. Test 11, 30 l/s per m, natural grass. Flow velocity at the crest for two filter techniques; for curve see Ch. 10.

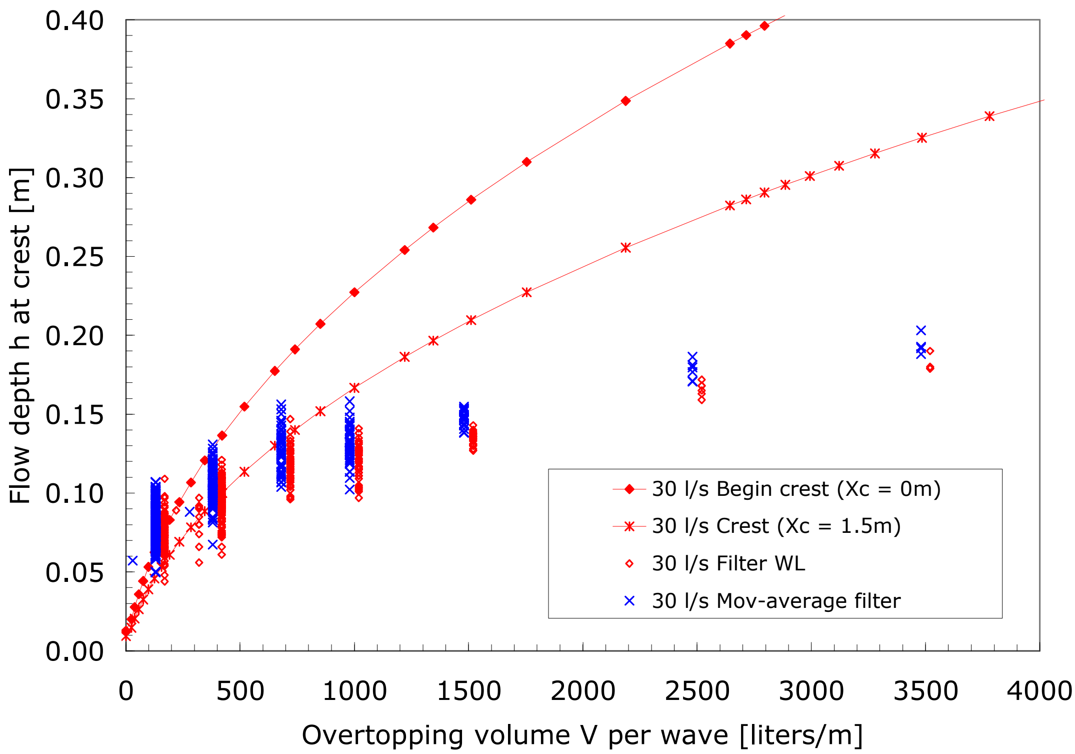


Figure 9.26. Test 11, 30 l/s per m, natural grass. Flow depth at the crest for two filter techniques; for curves see Ch. 10.

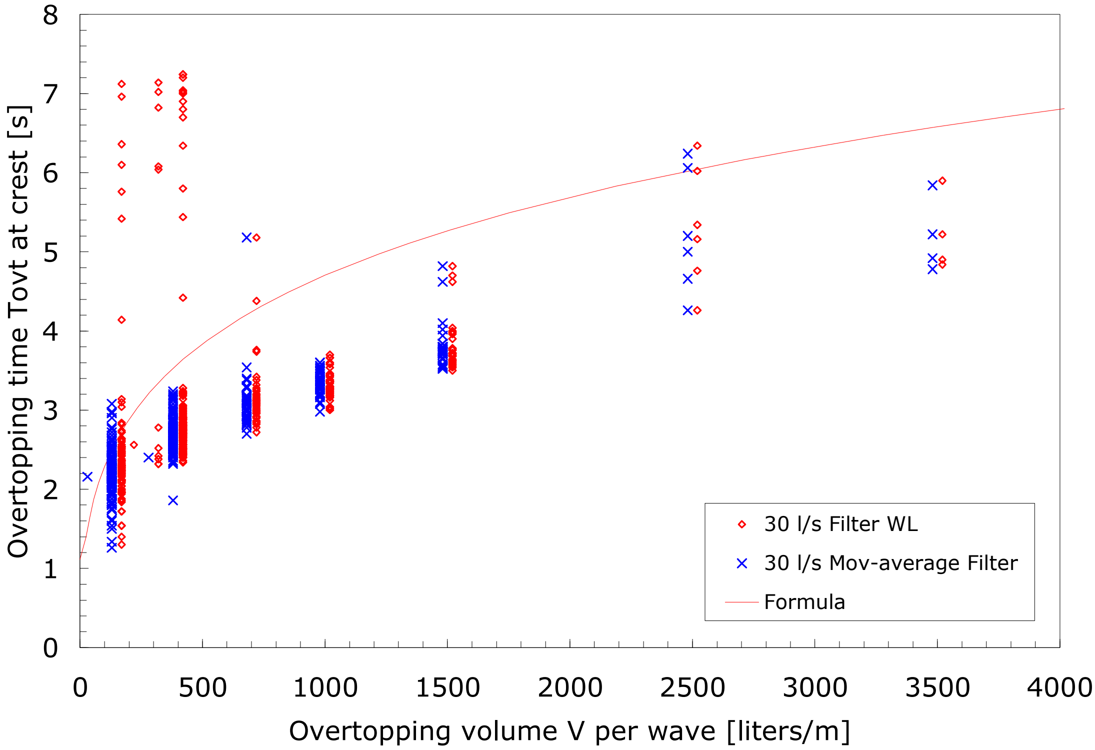


Figure 9.27. Test 11, 30 l/s per m, natural grass. Overtopping times at the crest for two filter techniques; for curve see Ch. 10.

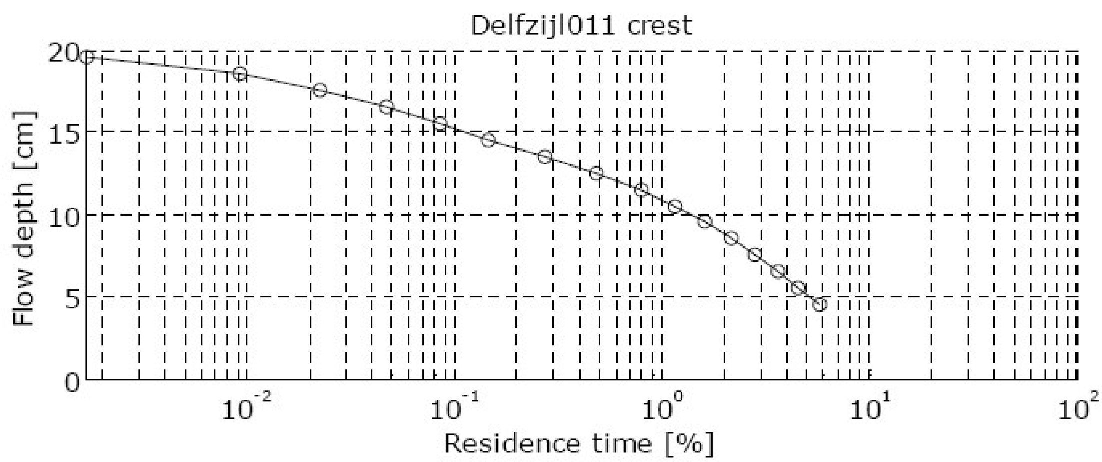


Figure 9.28. Residence time of water on the crest; test with 30 l/s per m.

Another analysis was performed on the data processed with the moving average filter. Figure 9.28 gives an example of the residence time of water exceeding a certain flow depth. The time record of flow depth was analysed for various discrete values (every cm from 5 – 20 cm) and the time that this value was exceeded was taken for every overtopping event. The summation of all these time in the record, divided by the total test duration, gives the residence time.

For example, the test with 30 l/s per m is shown in Figure 9.28 and shows that a flow depth of 5 cm was exceeded for 6% of the time, a flow depth of 10 cm was exceeded only 1.5% of the time and a flow depth of 15 cm gave an exceedance percentage of about 0.1% of the time.

9.3 Conclusions on measurements

9.3.1 Velocities

The instruments used were specially developed by Delft Hydraulics and partly calibrated when the final simulator had been constructed, see Section 7.2. At that time velocities larger than 2.5 m/s were not measured (due to a wrong maximum voltage) and also the actual flow depth meter was not working, but the conventional wave gauge system worked.

The first conclusion on analysis of measurements is that the electromagnetic velocity meter (EMS) was not able to measure large velocities correctly. The cause may be the very high turbulence and air inclusion, partly caused by the fact that a grassed slope is not perfectly smooth. That the velocities were indeed not measured correctly follows from the fact that front velocities were more or less according to prediction. An EMS as has been used in the tests is not applicable for this kind of tests and preferably another instrument has to be deployed or developed.

One option might be to measure the front velocity at various places, with shorter intervals than now between the crest and half way the slope. An interval of roughly 1 m would be sufficient as increase in velocity over such a short distance at the slope would not be significant. It should be noted that in such a way the front velocity will be measured only, not the full time record of the velocity.

The analysis also showed that the location where the velocity is measured is important. Velocities at 2 cm and 5 cm above the slope gave different results. Finally, it was shown that data processing on records with fast fluctuations, as in Figure 9.24, is important and may give significant differences depending on the filter technique used. The moving average technique gave larger velocities than the FIR technique which was used by Delft Hydraulics.

9.3.2 Flow depths

Thin wires were used to measure flow depths, as the conventional 2-3 mm thick wave gauges showed up-rushing water along the gauges (see Section 7.2 and Figure 7.2).

The measured flow depths were smaller than expected, certainly for the larger volumes. The moving average filter gave slightly larger flow depth than the FIR filtering. Flow depths larger than 0.2 m were never measured, even not for released volumes of 3500 l/m. The impression from video and witness of the testing was that larger flow depths were present. But flow depths were never recorded visually during testing. We recommend that visual observation of flow depths should be done in future as well.

There is suspicion that the flow depth meter did not work well for the large released volumes, due to the large turbulence and air inclusion, but hard evidence can not be given. Nevertheless, it is recommended to check this and develop a better instrument, if possible.

9.3.3 Overtopping times and residence times

Even if (maximum) velocities and flow depths can not be measured accurately, it is still possible to use the records to give overtopping times. An overtopping time starts when the instrument gives a signal for the first time and finishes if there is not signal anymore. Overtopping times were not known at the time of calibration of the wave overtopping simulator, and therefore, these times were estimated. Further analysis (Chapter 10) gave predictions for these overtopping times and these were compared with measured overtopping times. Figure 9.27 gives an example. This graph shows that the type of filter has influence on results. It shows also that overtopping times for large overtopping volumes were according to predictions and that overtopping times were a little too small for smaller volumes. Visual observation of and hand-timing of the residence times should be encouraged in future.

Actually, this could also be expected. The design of the wave overtopping simulator has been described in Chapter 3. The shape of the cross-section has been developed according to the theory of fall velocity, see Figure 3.1. Small volumes should be released close to the crest to give the correct velocity. But vertical space is required at the crest, due to the fact that the vertical velocity has to be transformed to a horizontal velocity. For this reason the simulator was designed on adjustable legs and the cross-section was made wider to keep small volumes of water close to the crest. But still it can be expected that small volumes would give a little too large velocities (which was indeed the case during calibration) and that overtopping times, therefore, would decrease. This latter aspect could not be measured correctly during calibration as the sampling frequency of the EMS was not sufficient. But the measurements at the dike show it clearly.

If flow depths are measured correctly, they can also be used to look at residence times, or actually percentage of time that a certain flow depth during a test has been exceeded. Figure 9.28 gives such an example. But the main criterion is that flow depths have to be measured correctly.

9.3.4 Wave overtopping simulator

Calibration of the prototype of the wave overtopping simulator gave an idea how well the simulator produced required flow velocities (Chapter 6). Although not all instruments worked correctly, it can be concluded that the wave overtopping simulator:

- produced the correct velocities for each volume, certainly for the larger volumes;
- produced the correct overtopping times for the larger volumes and a little too small for the smaller volumes;
- produced probably correct flow depths, but proof can not be given due to some doubt about the correct working of the flow depth meter.

10 Further elaboration of velocity, flow depth and duration of overtopping events (Bosman, 2007)

10.1 Introduction

When the wave overtopping simulator was designed (May – June 2006), it was discovered that not all relevant parameters had been described sufficiently in earlier scientific work. Maximum flow velocities and flow depths have been described in Section 2.3. It was concluded that maximum flow velocities were described consistently by equations 2.2, 2.4 and 2.5. But maximum flow depths could differ a factor 2, depending on which method was taken (Van Gent, 2002, or Schüttrumpf et al., 2002).

Also the full time record of an overtopping event was not known sufficiently, as only equations for maximum velocity and maximum flow depth were available. The shape of the overtopping time record is probably more or less triangular. But the duration of overtopping for each event was not know at all. For this reason an estimation of this duration was made, based on the actual peak period of the waves and the volume in the overtopping event. The calibration of the overtopping simulator, as described in Chapter 5, was then based on maximum velocities for each overtopping volume and on *estimated* durations of flow times, see Table 10.1, which was taken from Chapter 5.

Table 10.1. Maximum velocities and estimated flow times as function of overtopping volumes, used for the calibration of the wave overtopping simulator

	Theory	*For calibration purposes*	
Overtopping volume (l/m)	**flow velocity (m/s)**	**range (m/s)**	**flow time t_1 (s)**
50	2.0-2.5	1.5 – 3.0	1.5 – 2.5
150	2.9-3.2	2.5 – 3.5	1.5 – 2.5
400	4.1-4.3	3.5 – 5.0	2.0 – 3.0
700	4.8-5.1	4.2 – 5.7	2.5 – 3.5
1000	5.7	5.0 – 6.5	3.0 – 4.0
1500	6.2	5.5 – 7.0	3.0 – 4.0
2500	6.9	6.0 – 8.0	3.5 – 5.0
3500	7.6	6.5 – 8.5	3.5 – 5.0

In Chapter 5 it was strongly suggested to spend more time on the issue of flow depth and duration of overtopping events. It was under the SBW-programme of Rijkswaterstaat (Sterkte & Belastingen Waterkeringen) that this suggestion was indeed followed and in January 2007 Gijs Bosman of Delft University started his MSc-work on this subject. This chapter describes the main results of his MSc-work, Bosman (2007).

10.2 Experiments by Van Gent and Schüttrumpf

Bosman visited Schüttrumpf for a week and discussed all details on test set-up and data processing. Five tests were elaborated in depth. Data of three test were retrieved from Delft Hydraulics on tests of Van Gent (2002). Bosman analysed the test set-up and performed the data processing using his moving average filter technique. He came to the following conclusions on the measurements.

Experiments Schüttrumpf

Schüttrumpf used various instruments. 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 m/s. 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. The flow depth measurements from the depth gauges show good results as well.

Analysis

The analysis in both studies has been carried out correctly, but the 2%-values of velocity of the tests of Schüttrumpf are about 35% higher if the front velocities are used. This means that both velocities and flow depths were larger during the tests of Schüttrumpf in comparison to Van Gent, not only the flow depths.

Theory

Taking the high correlation in both studies of Schüttrumpf and Van Gent into consideration, the conclusion can be drawn that the principles of the equations on flow velocity and flow depth are correct. The fictive wave run-up minus the crest freeboard, $Ru_{2\%}$ - R_c, 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 discrepancy may be due to differences in the model set-up, see the next section.

10.3 Flow depths and velocities

Both flow depth and flow velocity are related to the "shortage in crest height" related to the 2%-run-up value, or A = $Ru_{2\%}$ - R_c. The equations assume that if A is equal for both slopes, it will give similar flow depth and velocity. But the situation for different slopes is quite different, see Figure 10.1. A steeper slope gives a larger run-up than a gentler slope, for similar wave conditions. And the difference or influence of the slope has not been taken into account by Van Gent (2002) or Schüttrumpf (2002). Van Gent used a 1:4 slope and Schüttrumpf used a 1:6 slope in the large wave flume and 1:4 and 1:6 slopes in small scale investigations.

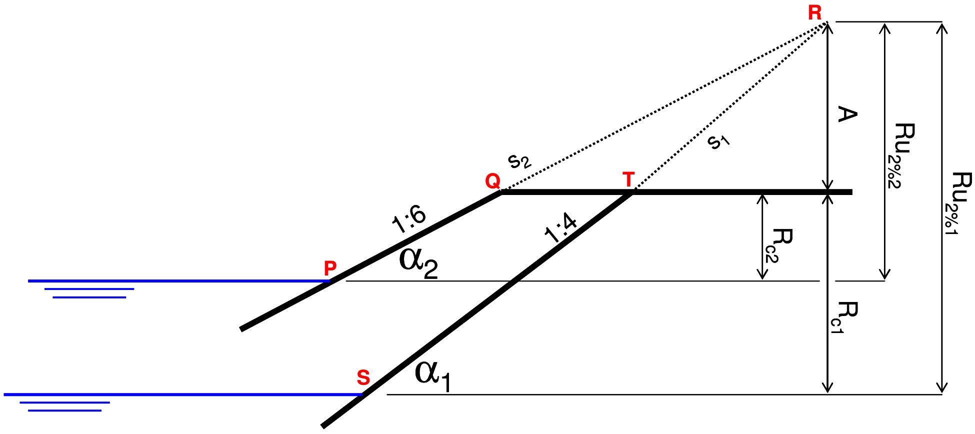


Figure 10.1. Schematization of run-up and overtopping on a 1:4 and a 1:6 slope.

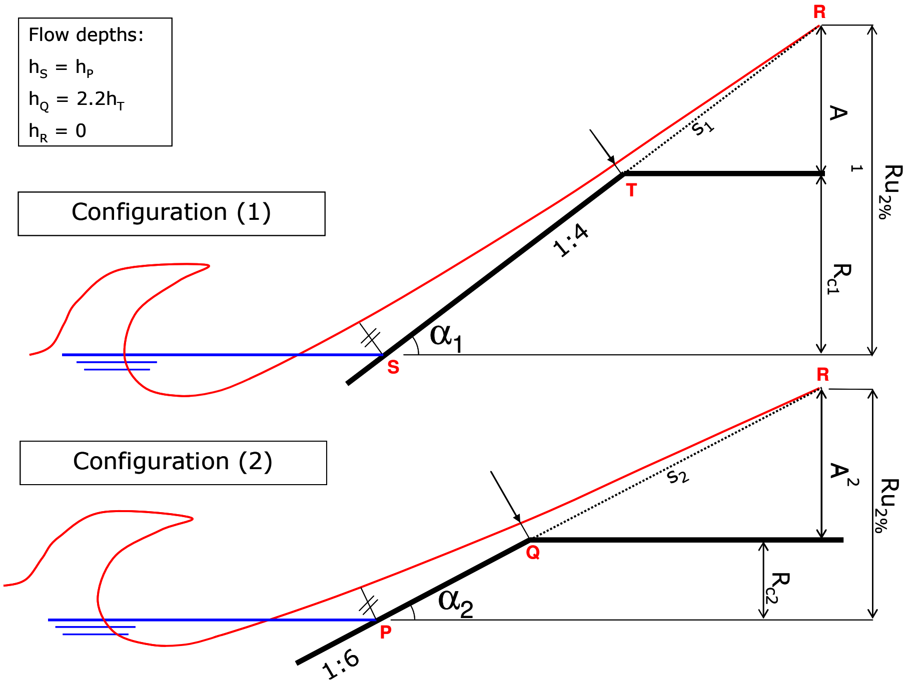


Figure 10.2. Fictive wave run-up on two different slopes

The vertical velocity component should be the same at points Q and T in Figure 10.1 to reach point R. This means that the velocities along the slope should be different, actually according to sinα!

Bosman used the following two assumptions (see Figure 10.2):

- The flow depth on the outer slope at the position of still water line will be equal for both dike configurations: $h_P = h_S$.
- The shape of the tongue of water on the outer slope is curved: $h_Q = 2.2h_T$.

Based on fitting between flow depths and flow velocities of a few tests from Van Gent (2002) as well as from Schüttrumpf (2002), Bosman finds the following relationships:

Flow depth coefficient: $c_{h,2\%} = 0.009/\sin^2\alpha$ (10.1)

Flow velocity coefficient: $c_{u,2\%} = 0.30/\sin\alpha$ (10.2)

Equation 10.1 is shown in Figure 10.3,where the horizontal axis is taken as cotα. Also the values of Van Gent and Schüttrumpf are given in the graph. For slopes 1:4 and 1:6 the coefficient $c_{h,2\%}$ gets values of 0.15 and 0.33, respectively, and these are similar to the coefficients of Van Gent (2002) and Schüttrumpf (2002). With above analysis and assumptions the difference in flow depth coefficient between the two researchers has been explained.

Also the velocity will be dependent on the slope angle, in contrast to what was concluded in Chapter 2. Figure 10.4 shows equation 10.2, together with the data points. Initially Van Gent and Schüttrumpf had almost the same coefficients for velocity, $c_{u,2\%}$ = 1.30 and 1.37, respectively. But due to the wrong measurements of large velocities by Schüttrumpf, his coefficient actually is 1.8. Also these coefficients fit nicely with the developed equation.

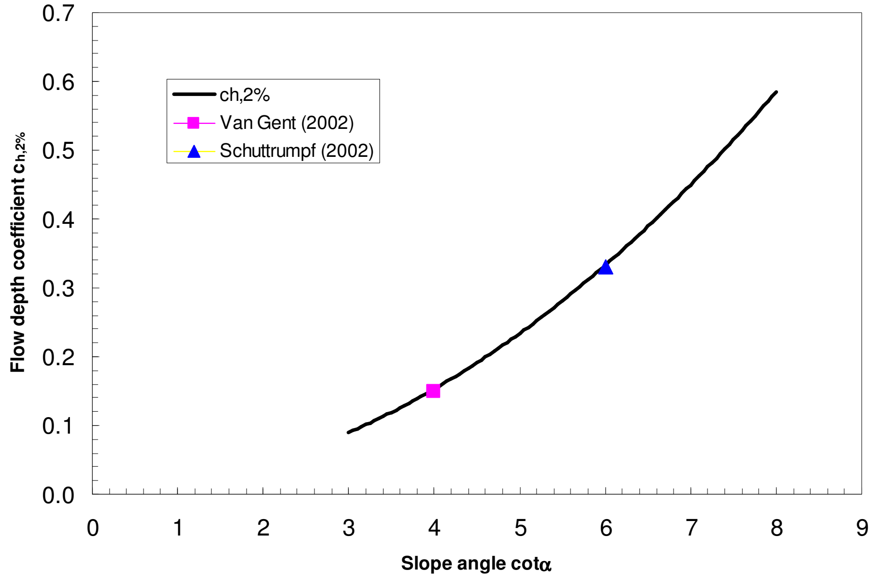


Figure 10.3. Flow depth coefficient as function of the slope angle cotα

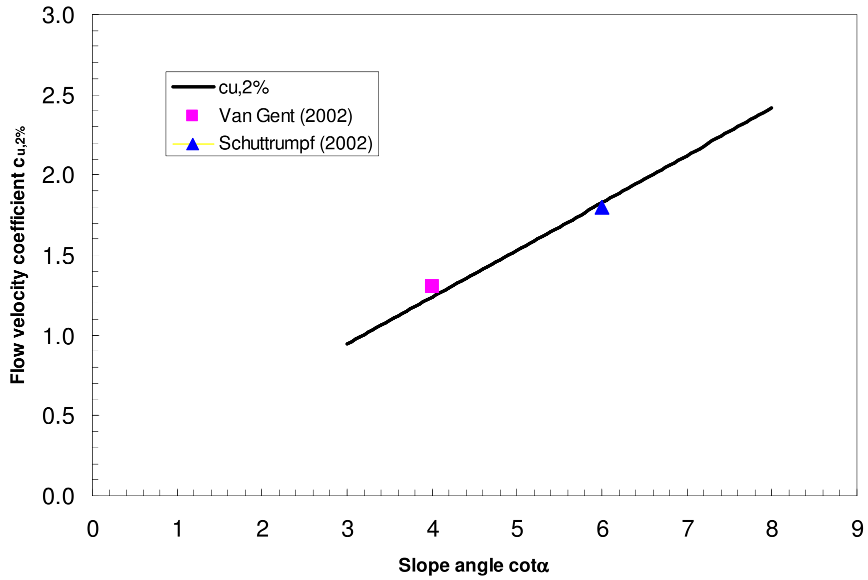


Figure 10.4. Flow velocity coefficient as function of the slope angle cotα

Figures 10.3 and 10.4 have been given for slope angles between 1:3 and 1:8. For steeper slopes the wave breaking may change to collapsing or surging and then the linear relationship between run-up and breaker parameter starts to change, which may also give a deviation in flow depth or velocity. Slopes gentler than 1:8 are hardly slopes, but act more like a steep foreshore. The method has been developed for slopes of 1:4 and 1:6 and therefore its range of application should be limited to about 1:3 to 1:8.

The new formulae for maximum flow velocity and flow depth at the beginning of the crest now become:

$$h_{2\%}(x_c = 0) = 0.009/\sin^2\alpha * (Ru_{2\%} - R_c) \qquad (10.3)$$

$u_{2\%}(x_c = 0) = 0.30/\sin\alpha * (Ru_{2\%} - R_c)^{0.5}$ (10.4)

The equations for flow velocity and flow depth on the crest give an exponential decay, see Section 2.3 and equations 2.1 and 2.2. What has not been taken into account is that the velocity on the outer slope has an upward component, due to the slope and that the direction changes to horizontal on the crest itself. This change of flow causes differences between what happens right at the edge with the outer slope and the crest and at the horizontal crest itself. This transition has not been included in the analysis of Van Gent or Schüttrumpf.

Figure 10.5 gives the relationship between the relative flow depth (maximum flow depth at the edge of the crest with the outer slope and along the crest) as function of the location on the crest. It is clear that the trend does not start in 1, which is the edge to the outer slope. There is a fast drop in maximum flow depth directly behind the crest, due to the transition in flow direction.

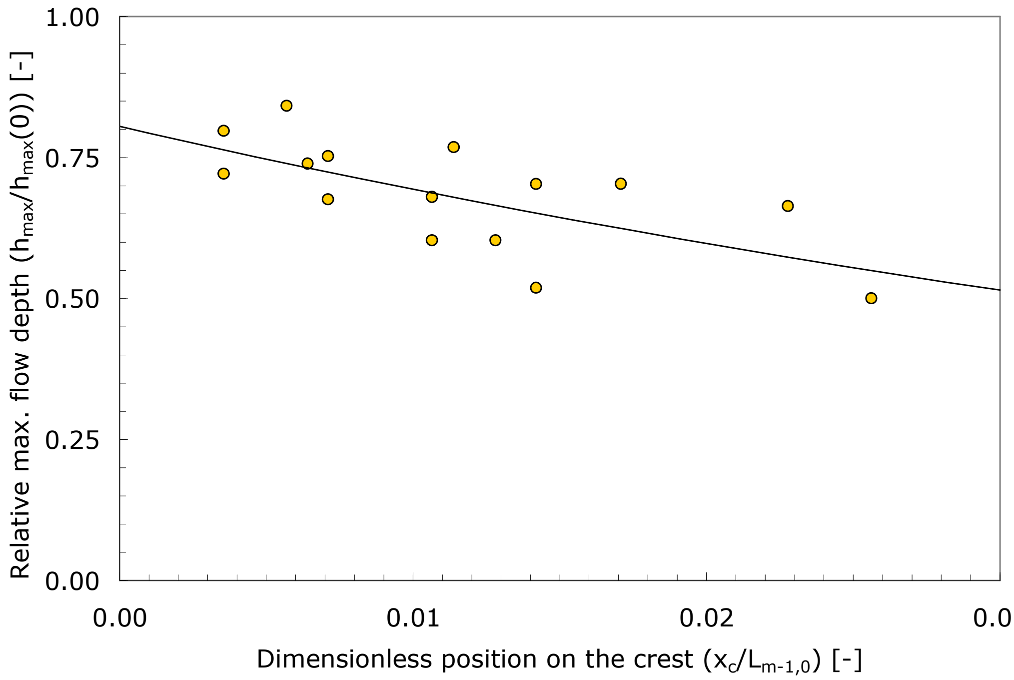


Figure 10.5. Relative flow depth along the crest (regular waves)

Bosman included an extra coefficient in the formulae for maximum flow depth along the crest and found, based on Figure 10.5:

$h_{2\%}(x_c)/h_{2\%}(x_c=0) = 0.81 \exp(-6\ x_c / (\gamma_c\ L_{m-1,0}))$ (10.5)

With γ_c = 1 for a smooth crest. As the velocity along the slope is based on the flow depth, the extra coefficient 0.81 should become 1 again for the velocity. This is indeed the case (see Figure 10.6) and the following equation was found for the velocity along the slope:

$u_{2\%}(x_c)/u_{2\%}(x_c=0) = \exp(-0.042\ x_c / (\gamma_c\ h_{2\%}(x_c)))$ (10.6)

It are actually equations 10.3 – 10.6 which should be used for the wave overtopping simulator and with a value of x_c = 1-2 m (just after the beginning of the crest). It is for this reason that a value of x_c = 1.5 m has been chosen in the graphs of Chapter 9.

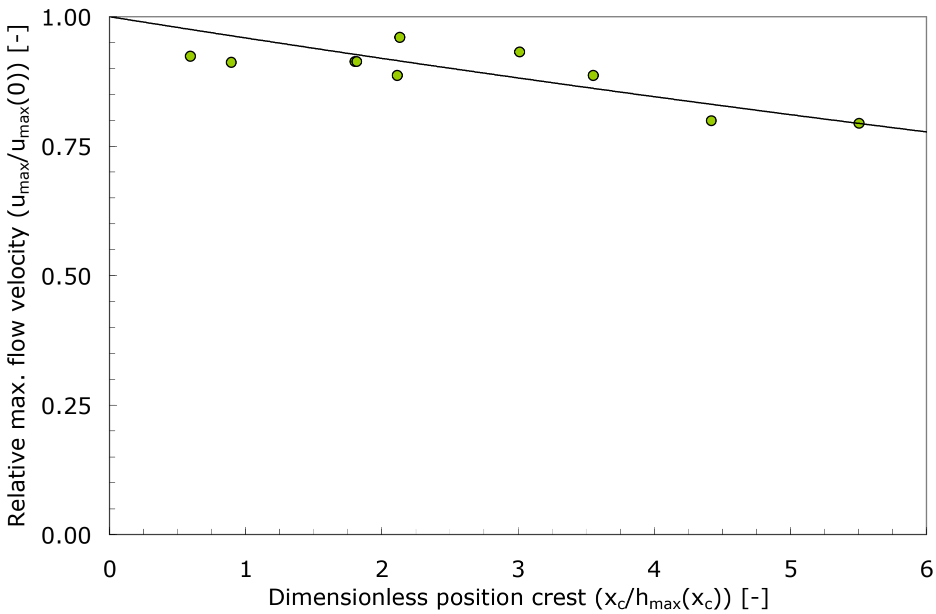


Figure 10.6. Relative flow velocity along the crest

10.4 Overtopping times

The rough assumption on overtopping times, used to calibrate the wave overtopping simulator, was T_{ovt} = 0.3 to 0.8 times T_p (see Chapter 5). No explicit formulae were available from literature. Bosman analysed the overtopping time in depth and came to the conclusion that also overtopping time could be described as a function of "shortage of crest height", $Ru_{2\%}$ - R_c. He found the following equation:

$$T_{ovt,\,2\%}(0)/T_{m-1,0} = 1.15\,((Ru_{2\%} - R_c) / (\gamma_f\,H_{mo}))^{0.5} \quad (10.7)$$

The overtopping time is independent of the outer slope.

The overtopping time becomes longer when the overtopping water travels along the crest. This is given by the following equation, which also includes a jump or transition directly behind the crest:

$$T_{ovt,\,2\%}(x_c)/T_{ovt,\,2\%}(x_c{=}0) = 1.67 + 0.24\,\ln(x_c / L_{m-1,0}) \quad (10.8)$$

For simulation with the wave overtopping equations 10.7 and 10.8 should be used with a value of x_c = 1-2 m.

Bosman concluded furthermore that the overtopping time on the inner slope for the small scale tests of Van Gent (2002) remained the same, where the overtopping times in the large scale tests of Schüttrumpf increased drastically on the long slope. Figure 10.7 gives the comparison. It was concluded that small scale tests could include scale effects for overtopping times at the inner slope and that in large scale (and in reality) these overtopping times may increase, as also the shape of the overtopping record changes (no triangle anymore, see Section 10.5).

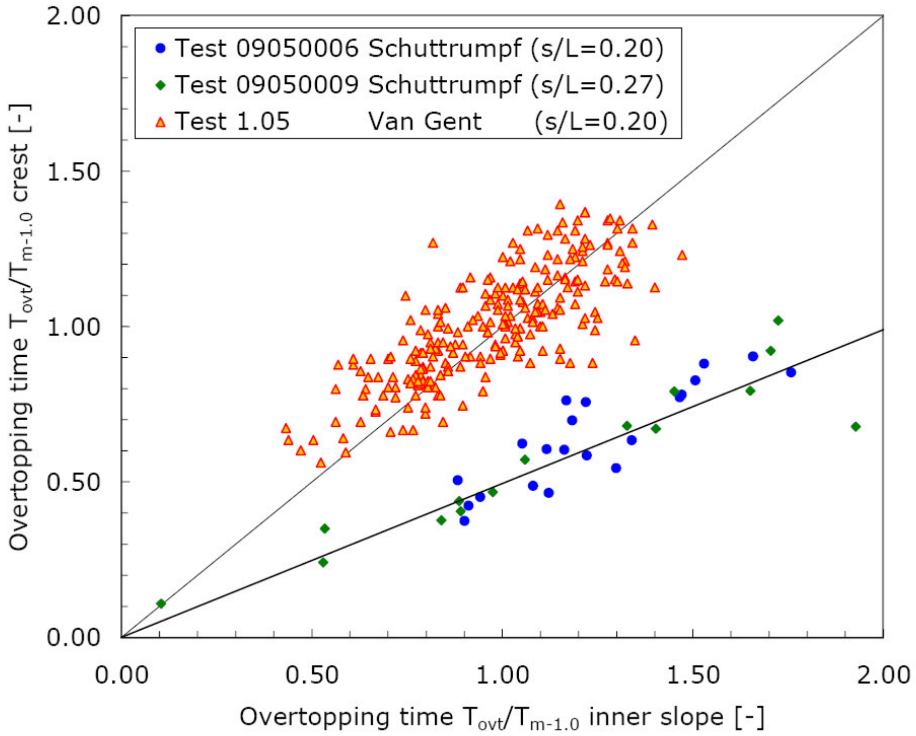


Figure 10.7. Difference in overtopping time on inner slope for small and large scale tests

10.5 Shape of overtopping event

Bosman concluded that the shape of an overtopping event could best be simulated by a triangle and not by a cosine function. He concluded too that the shape changes when an overtopping wave travels over the inner slope, while the overtopping time becomes longer. Figure 10.8 gives this phenomenon in a schematic way.

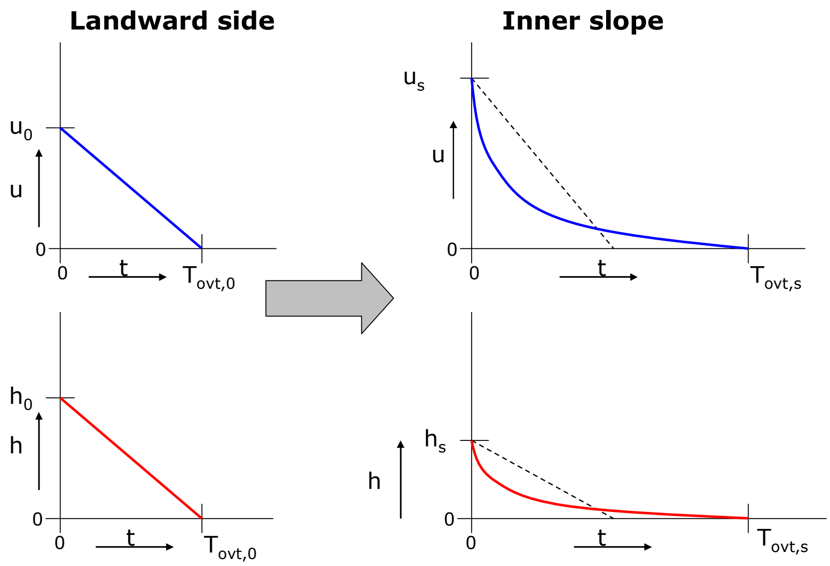


Figure 10.8. Transition of velocity and flow depth along inner slope

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Appendix 1. Tables with individual measurements

C	16-06-2006 (day 167) Height of valve above the crest: 1,25 m Maximum opening of valve: 0,34 m Observation: Measurement device for flow depth not connected (broken cable)					
	Moment of the day	**flow velocity v (m/s)**	**Flow depth d (m)**	**flow time t (s)**	**Observation**	**Volume V (m3) according to 1/3*v*d*t**
C400-1	13.45	4.20	0.06	3.00	-	0.23
C400-2	13.48	5.00	0.07	3.50	-	0.38
C400-3	13.50	3.00	0.06	3.00	-	0.17
C400-4	13.52	4.20	0.05	2.50	-	0.16
C400-5	13.53	3.80	0.06	3.30	-	0.23
C400-6	13.54	10.00	0.06	3.50	-	0.64
C400-7	13.55	7.80	0.07	3.50	-	0.59
C400-8	13.56	8.00	0.06	3.00	-	0.44
Average values for test C400:		**5.75**	**0.06**	**3.16**		**0.35**
C150-1	14.15	3.50	0.04	2.50	-	0.12
C150-2	14.17	4.80	0.04	3.00	-	0.19
C150-3	14.18	4.80	0.04	2.50	-	0.16
C150-4	14.19	4.20	0.04	3.00	-	0.17
C150-5	14.09	5.10	0.04	2.50	-	0.17
Average values for test C150:		**4.48**	**0.04**	**2.70**		**0.16**
C1000-1	13.59	5.40	0.10	4.10	-	0.74
C1000-2	14.02	5.20	0.11	4.00	-	0.76
C1000-3	14.04	5.50	0.12	4.00	-	0.88
C1000-4	14.07	5.30	0.13	4.20	-	0.96
C1000-5	14.09	5.50	0.13	4.30	-	1.02
C1000-6	14.12	5.50	0.14	4.20	-	1.08
Average values for test C1000:		**5.40**	**0.12**	**4.13**		**0.91**
C2500-1	14.49	6.00	0.25	5.80	-	2.90
C2500-2	14.54	6.50	0.30	6.20	-	4.03
C2500-3	14.59	6.50	0.25	6.20	-	3.36
C2500-4	15.03	6.20	0.27	5.60	-	3.12
C2500-5	15.08	6.80	0.25	6.00	-	3.40
Average values for test C2500:		**6.40**	**0.26**	**5.96**		**3.36**
C3500-1	15.15	The valve doesn't open properly				
C3500-2	15.25	7.70	0.35	6.00	-	5.39
C3500-3	15.29	6.50	0.28	7.00	-	4.25
C3500-4	15.37	7.00	0.30	7.00	-	4.90
C3500-5	15.44	8.90	0.27	7.20	-	5.77
Average values for test C3500:		**7.53**	**0.30**	**6.80**		**5.08**

D

19-06-2006 (day 170)
Height of valve above the crest: 1,25 m
Maximum opening of valve: 0,42 m
Observation:

	Moment of the day	**flow velocity v (m/s)**	**Flow depth d (m)**	**flow time t (s)**	**Observation**	**volume V (m3) according to 1/3*v*d*t**
D1000-1	8.56	5.10	0.16	4.00		1.09
D1000-2	8.58	5.10	0.16	3.50		0.95
D1000-3	9.00	5.60	0.17	3.00		0.95
D1000-4	9.02	5.20	0.15	3.50		0.91
D1000-5	9.04	4.80	0.16	4.20		1.08
D1000-6	9.07	5.30	0.16	3.30		0.93
Average values for test D1000:		**5.18**	**0.16**	**3.58**		**0.99**
D1500-1	9.11	6.40	0.18	4.00		1.54
D1500-2	9.14	5.30	0.19	4.20		1.41
D1500-3	9.18	6.10	0.19	4.00		1.55
D1500-4	9.21	7.30	0.18	4.00		1.75
D1500-5	9.25	5.90	0.20	4.00		1.57
Average values for test D1500:		**6.20**	**0.19**	**4.04**		**1.56**
D700-1	9.29	5.30	0.12	3.00		0.64
D700-2	9.31	6.40	0.13	2.30		0.64
D700-3	9.33	5.50	0.14	3.00		0.77
D700-4	9.34	5.20	0.13	3.00		0.68
D700-5	9.36	6.00	0.12	3.00		0.72
Average values for test D700:		**5.68**	**0.13**	**2.86**		**0.69**
D400-1	10.11	wrong measurement				
D400-2	10.12	4.30	0.11	4.00		0.63
D400-3	10.13	4.20	0.09	2.50		0.32
D400-4	10.13	5.00	0.09	2.50		0.38
D400-5	10.14	5.10	0.09	2.50		0.38
Average values for test D400:		**4.65**	**0.10**	**2.88**		**0.43**
D150-1	10.24	3.80	0.04	2.00		0.10
D150-2	10.25	2.90	0.04	2.00		0.08
D150-3	10.26	5.50	0.05	1.80		0.17
D150-4	10.27	6.10	0.04	2.00		0.16
D150-5	10.29	3.00	0.04	1.00		0.04
Average values for test D150:		**4.26**	**0.04**	**1.76**		**0.11**
D2500-1	10.47	6.20	0.22	5.50	valve opened too slow	2.50
D2500-2	10.52	7.50	0.29	4.50		3.26
D2500-3	11.23	6.80	0.27	5.00		3.06
D2500-4	11.28	6.20	0.27	7.50	valve opened too slow	4.19
D2500-5	11.34	7.00	0.26	5.50		3.34
D2500-6	11.40	9.00	0.27	6.00	valve opened too slow	4.86
D2500-7	11.46	7.50	0.27	5.00		3.38
Average values for test D2500:		**7.17**	**0.26**	**5.57**		**3.51**
D3500-1	11.54	7.20	0.34	4.80		3.92
D3500-2	12.03	7.60	0.40	5.50		5.57
D3500-3	12.12	8.50	0.44	5.40		6.73
D3500-4	12.20	6.80	0.45	5.00		5.10
D3500-5	12.28	7.50	0.46	4.60		5.29
Average values for test D3500:		**7.52**	**0.42**	**5.06**		**5.32**

E	19-06-2006 (day 170) Height of valve above the crest: 0,84 m Maximum opening of valve: 0,42 m Observation:					
	Moment of the day	**flow velocity v (m/s)**	**Flow depth d (m)**	**flow time t (s)**	**Observation**	**volume V (m3) according to 1/3*v*d*t**
E700-1	14.49	3.50	0.13	7.50		1.14
E700-2	14.51	3.80	0.15	6.00		1.14
E700-3	14.53	5.60	0.16	5.80		1.73
E700-4	14.54	3.50	0.16	6.20		1.16
E700-5	14.56	4.00	0.17	6.00		1.36
Average values for test E700:		**4.08**	**0.15**	**6.30**		**1.31**
E1000-2	15.00	3.80	0.22	3.80		1.06
E1000-3	15.03	3.60	0.21	5.60		1.41
E1000-4	15.05	5.00	0.20	6.00		2.00
E1000-5	15.07	3.80	0.20	5.50		1.39
E1000-6	15.10	4.20	0.21	5.80		1.71
Average values for test E1000:		**4.08**	**0.21**	**5.34**		**1.51**
E400-1	15.29	3.20	0.16	4.00	Opening valve: 26 cm	0.68
E400-2	15.30	3.50	0.13	3.50	Opening valve: 26 cm	0.53
E400-3	15.31	3.10	0.17	4.80	Opening valve: 26 cm	0.84
E400-4	15.32	3.60	0.12	3.50	Opening valve: 26 cm	0.50
E400-5	15.33	3.00	0.15	4.00	Opening valve: 26 cm	0.60
Average values for test E400:		**3.28**	**0.15**	**3.96**		**0.63**

G	21-06-2006 (day 172) Height of valve above the crest: 0,60 m Maximum opening of valve: 0,42 m Observation: Smaller transition					
	Moment of the day	**flow velocity v (m/s)**	**Flow depth d (m)**	**flow time t (s)**	**Observation**	**volume V (m3) according to 1/3*v*d*t**
G400-1	11.49	3.40	0.13	3.80		0.56
G400-2	11.50	4.00	0.11	4.20		0.62
G400-3	11.50	6.80	0.12	4.00		1.09
G400-4	11.51	3.80	0.13	4.20		0.69
G400-5	11.52	3.00	0.14	4.80		0.67
Average values for test G400:		**4.20**	**0.13**	**4.20**		**0.73**
G150-1	11.54	4.10	0.05	2.20		0.15
G150-2	11.55	4.25	0.06	1.40		0.12
G150-3	11.56	2.40	0.05	2.20		0.09
G150-4	11.57	6.00	0.06	1.80		0.22
G150-5	11.58	7.90	0.05	2.00		0.26
Average values for test G150:		**4.93**	**0.05**	**1.92**		**0.17**
G50-1	12.08	4.90	0.03	0.50		0.02
G50-2	12.09	3.00	0.02	1.60		0.03
G50-3	12.09	3.70	0.03	0.60		0.02
G50-4	12.10	5.00	0.02	0.50		0.02
G50-5	12.10	6.20	0.04	0.90		0.07
G50-6	12.11	7.00	0.03	1.20		0.08
G50-7	12.11	7.20	0.03	1.10		0.08
G50-8	12.12	4.20	0.03	1.00		0.04
G50-9	12.12	8.70	0.03	1.00		0.09
G50-10	12.13	4.10	0.03	0.50		0.02
Average values for test G50:		**5.40**	**0.03**	**0.89**		**0.05**
G700-1	12.25	5.00	0.15	3.80		0.95
G700-2	12.26	3.60	0.16	3.80		0.73
G700-3	12.27	3.70	0.17	4.00		0.84
G700-4	12.28	3.70	0.16	3.00		0.59
G700-5	12.30	5.00	0.16	4.00		1.07
Average values for test G700:		**4.20**	**0.16**	**3.72**		**0.84**

H	21-06-2006 (day 172) Height of valve above the crest: 0,60 m Maximum opening of valve: 0,42 m Observation:					
	Moment of the day	**flow velocity v (m/s)**	**Flow depth d (m)**	**flow time t (s)**	**Observation**	**volume V (m3) according to 1/3*v*d*t**
H3500-1	14.12	7.60	0.39	5.50		5.43
H3500-2	14.44	6.90	0.41	4.80		4.53
H3500-3	14.53	9.80	0.41	5.00		6.70
H3500-4	15.03	9.70	0.39	5.50		6.94
H3500-5	15.49	7.20	0.41	5.30	opening valve: 50 cm	5.22
Average values for test H3500:		**8.24**	**0.40**	**5.22**		**5.76**
H700-1	15.09	5.20	0.22	3.00		1.14
H700-2	15.11	5.30	0.21	3.80		1.41
H700-3	15.13	4.70	0.23	3.00		1.08
H700-4	15.14	5.30	0.24	3.50		1.48
H700-5	15.16	4.70	0.23	4.30		1.55
Average values for test H700:		**5.04**	**0.23**	**3.52**		**1.33**
H1000-1	15.18	5.60	0.28	3.00		1.57
H1000-2	15.21	5.00	0.27	3.00		1.35
H1000-3	15.23	5.60	0.27	3.30		1.66
H1000-4	15.26	5.40	0.28	3.30		1.66
H1000-5	15.29	5.80	0.29	3.60		2.02
Average values for test H1000:		**5.48**	**0.28**	**3.24**		**1.65**
H2500-1	15.35	6.30	0.34	4.30		3.07
H2500-2	15.41	6.60	0.37	5.00		4.07
H2500-3	15.54	8.00	0.36	4.80	opening valve: 50 cm	4.61
Average values for test H2500:		**6.97**	**0.36**	**4.70**		**3.92**

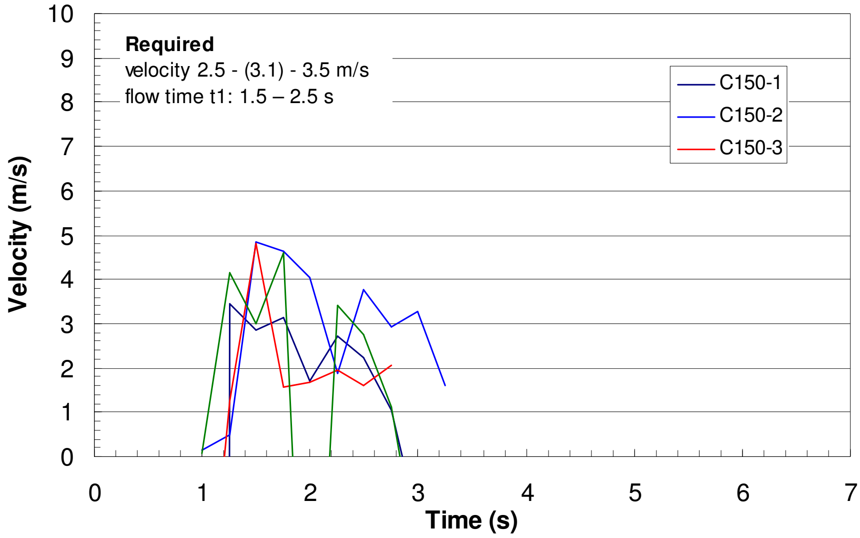
I	22-06-2006 (day 173) Height of valve above the crest: 1,83 m Maximum opening of valve: 0,50 m Observation: length of transition slope enlarged					
	Moment of the day	**flow velocity v (m/s)**	**Flow depth d (m)**	**flow time t (s)**	**Observation**	**volume V (m3) according to 1/3*v*d*t**
I2500-1	09.19					
I2500-2	09.24					
I2500-3	09.32	9.20	0.27	4.20	Front guidance completely open	3.48
I2500-4	09.40	7.80	0.28	5.10	Front guidance completely open	3.71
I2500-5	10.20	7.70	0.24	6.00	Front guidance completely open	3.70
Average values for test I2500:		**8.28**	**0.33**	**4.62**		**4.05**
I3500-1	10.27	6.50	0.26	6.00		3.38
I3500-2	10.34	7.50	0.27	6.00		4.05
I3500-3	10.42	7.60	0.27	5.80		3.97
I3500-4	10.50	7.50	0.27	5.00		3.38
Average values for test I3500:		**7.28**	**0.27**	**5.70**		**3.69**
I400-1	11.39	4.40	0.08	2.90		0.34
I400-2	11.41	4.30	0.06	3.50		0.30
I400-3	11.42	6.00	0.06	3.20		0.38
I400-4	11.43	8.50	0.07	3.00		0.60
I400-5	11.44	4.50	0.08	3.20		0.38
Average values for test I400:		**5.54**	**0.07**	**3.16**		**0.40**
I1000-1	11.27	5.20	0.18	3.00		0.94
I1000-2	11.29	5.40	0.17	2.50		0.77
I1000-3	11.31	6.00	0.16	3.00	Front guidance completely open	0.96
I1000-4	11.33	5.10	0.15	2.80	Front guidance completely open	0.71
I1000-5	11.35	5.00	0.16	2.80	Front guidance completely open	0.75
Average values for test I1000:		**5.34**	**0.16**	**2.82**		**0.82**

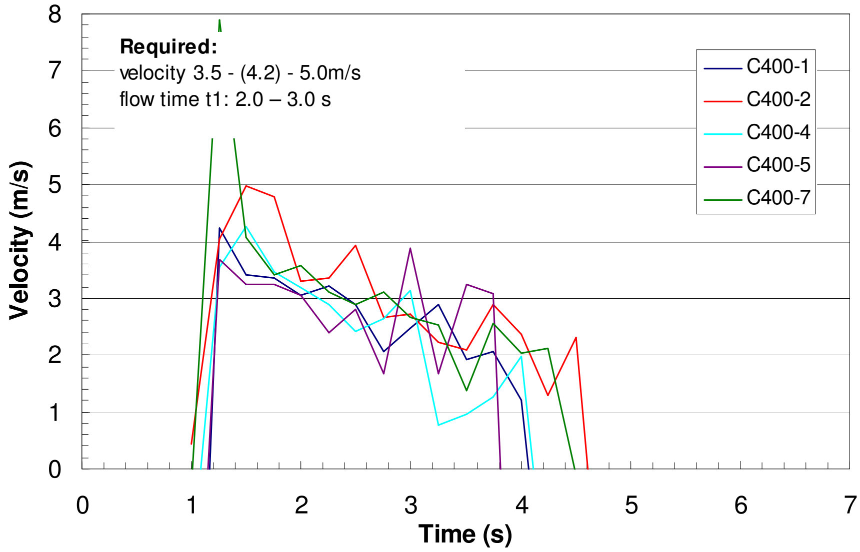
J	22-06-2006 (day 173) Height of valve above the crest: 1,83 m Maximum opening of valve: 0,50 m Observation: length of transition slope enlarged					
	Moment of the day	**flow velocity v (m/s)**	**Flow depth d (m)**	**flow time t (s)**	**Observation**	**volume V (m3) according to 1/3*v*d*t**
J3500-1	10.57	7.00	0.44	5.80		5.95
J3500-2	11.04	6.60	0.43	6.00		5.68
J3500-3	11.12	9.00	0.43	5.60		7.22
J3500-4	11.23	7.10	0.42	5.00		4.97
Average values for test J3500:		**7.43**	**0.43**	**5.60**		**5.96**

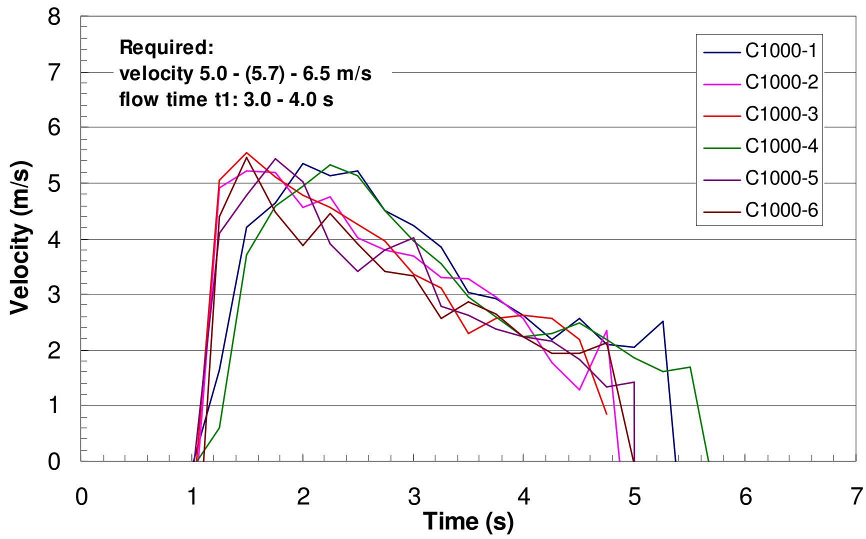
K	28-06-2006 (day 179) Height of valve above the crest: 1,25 m Maximum opening of valve: 0,50 m Observation: length of transition slope enlarged					
	Moment of the day	**flow velocity v (m/s)**	**Flow depth d (m)**	**flow time t (s)**	**Observation**	**volume V (m3) according to 1/3*v*d*t**
K3500-1	08.57	6.00	0.28	6.20		3.47
K3500-2	09.03	5.50	0.27	6.40		3.17
K3500-3	09.10	6.00	0.29	5.50		3.19
K3500-4	09.16	6.00	0.30	5.00		3.00
K3500-5	09.22	5.80	0.31	5.50		3.30
K3500-6	09.28	6.00	0.30	5.00		3.00
K3500-7	14.16	7.00	0.30	5.20		3.64
K3500-8	14.21	6.50	0.30	5.50		3.58
K3500-9	14.27	6.80	0.32	5.00		3.63
K3500-10	14.33	7.00	0.34	5.20		4.13
K3500-11	14.39	5.50	0.34	5.80		3.62
Average values for test K3500:		**6.19**	**0.30**	**5.48**		**3.43**
K2500-1	10.01	5.20	0.25	4.50		1.95
K2500-2	10.05	6.00	0.24	4.10		1.97
K2500-3	10.09	6.00	0.24	4.10		1.97
K2500-4	10.14	5.50	0.24	4.00		1.76
K2500-5	10.18	5.50	0.26	4.20		2.00
K2500-6	14.43	7.00	0.27	4.40		2.77
K2500-7	14.47	7.00	0.29	4.40		2.98
Average values for test K2500:		**6.03**	**0.26**	**4.24**		**2.20**
K1500-1	10.21	5.00	0.19	3.70		1.17
K1500-2	10.23	4.50	0.20	3.00		0.90
K1500-3	10.26	4.70	0.19	3.10		0.92
K1500-4	10.28	5.00	0.21	3.20		1.12
K1500-5	10.31	5.00	0.21	4.00		1.40
Average values for test K1500:		**4.84**	**0.20**	**3.40**		**1.10**
K1000-1	10.32	4.40	0.17	2.80		0.70
K1000-2	10.34	5.00	0.19	2.80		0.89
K1000-3	10.36	4.70	0.19	3.00		0.89
K1000-4	10.37	4.50	0.18	2.80		0.76
K1000-5	10.39	4.50	0.19	2.50		0.71
Average values for test K1000:		**4.62**	**0.18**	**2.78**		**0.79**
K700-1	10.45	3.70	0.14	2.50		0.43
K700-2	10.46	4.50	0.15	2.50		0.56
K700-3	10.47	4.10	0.15	2.20		0.45
K700-4	10.48	4.00	0.16	2.20		0.47
K700-5	10.49	4.20	0.16	2.50		0.56
Average values for test K700:		**4.10**	**0.15**	**2.38**		**0.49**
K400-1	10.55	3.40	0.12	2.00		0.27
K400-2	10.56	3.50	0.12	2.20		0.31
K400-3	10.56	3.40	0.12	2.00		0.27
K400-4	10.57	3.60	0.14	2.00		0.34
K400-5	10.58	3.20	0.12	2.00		0.26
Average values for test K400:		**3.42**	**0.12**	**2.04**		**0.29**

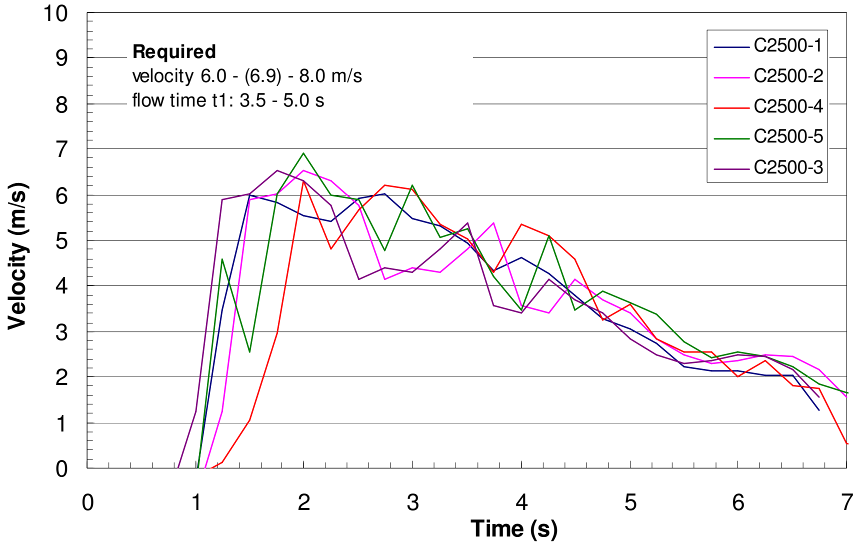
K	28-06-2006 (day 179) Height of valve above the crest: 1,25 m Maximum opening of valve: 0,50 m Observation: length of transition slope enlarged					**continued**
K150-1	11.09	3.30	0.10	1.80		0.20
K150-2	11.11	3.30	0.09	1.80		0.18
K150-3	11.12	-	0.09	-		
K150-4	11.12	2.80	0.10	1.50		0.14
K150-5	11.13	2.40	0.10	1.30		0.10
K150-6	11.15	2.70	0.10	1.20		0.11
K150-7	11.16	2.40	0.11	1.20		0.11
K150-8	11.17	3.70	-	1.50		
K150-9	11.18	-	0.09	-		
K150-10	11.19	3.00	0.11	1.50		0.17
Average values for test K150:		**2.95**	**0.10**	**1.48**		**0.14**
K50-1	11.23		0.03			
K50-2	11.24		0.04			
K50-3	11.25		0.04			
K50-4	11.25		0.03			
K50-5	11.26		0.05			
K50-6	11.27		0.03			
K50-7	11.27		0.04			
K50-8	11.28		0.05			
K50-9	11.28		0.04			
K50-10	11.29		0.05			
K50-11	14.53	2.80	0.04	0.70		0.037
K50-12	14.53		0.04	-		
K50-13	14.54	2.40	0.05	0.60		0.040
K50-14	14.55		0.04	-		
K50-15	14.55	2.60	0.04	0.80		0.043
K50-16	14.56	3.10	0.04	0.50		0.021
Average values for test K50:		**2.73**	**0.04**	**0.94**		**0.035**

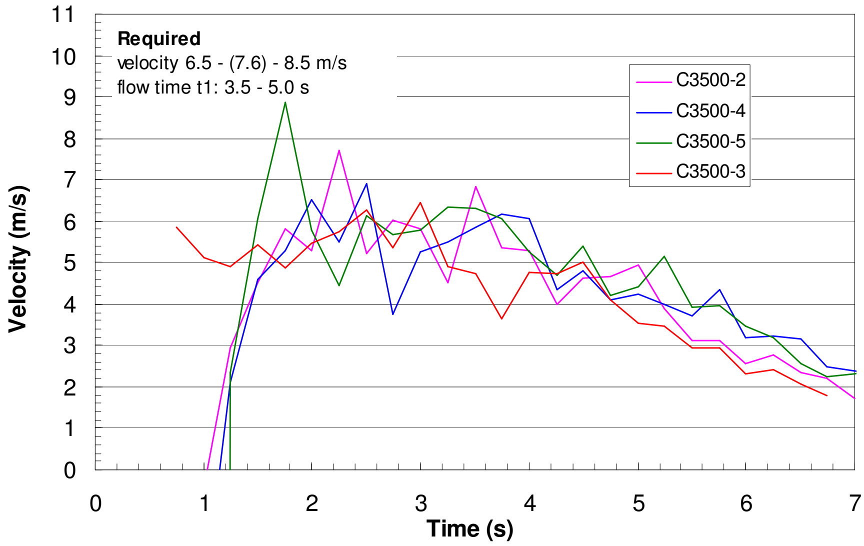
Appendix 2. Figures with composed velocity measurements

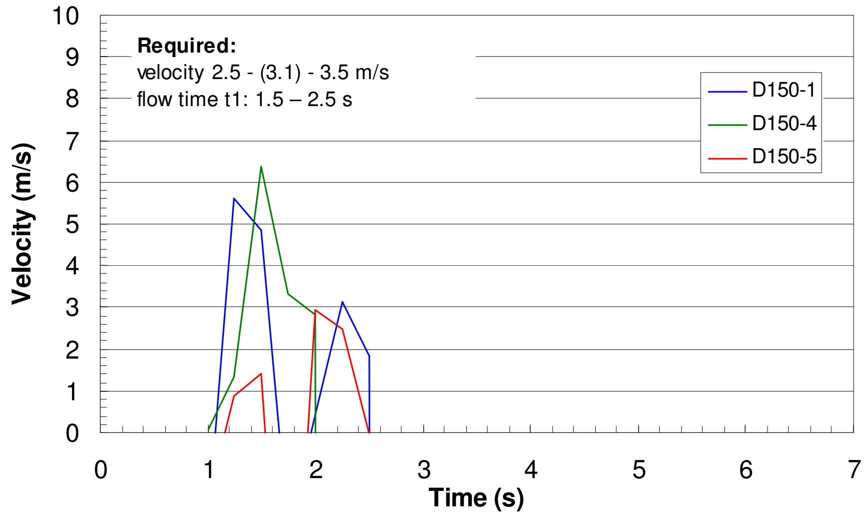


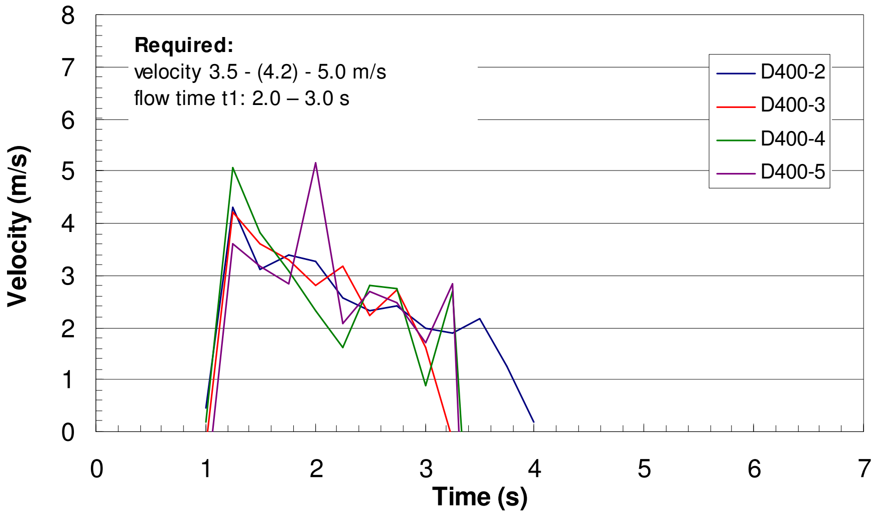


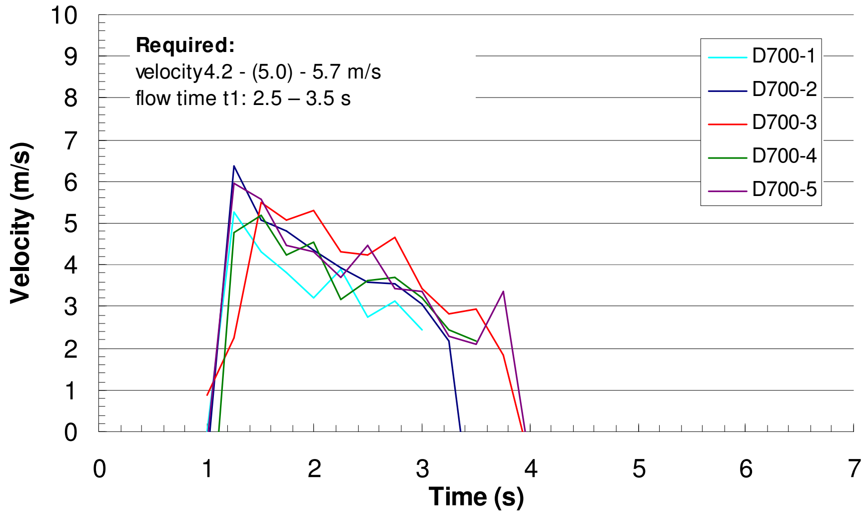


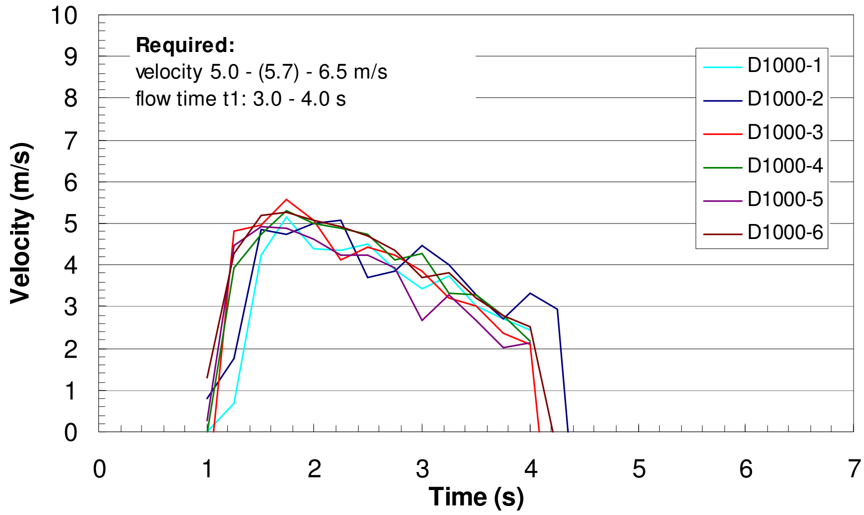

Required
velocity 6.0 - (6.9) - 8.0 m/s
flow time t1: 3.5 - 5.0 s
C2500-1
C2500-2
C2500-4
C2500-5
C2500-3
Velocity (m/s)
Time (s)

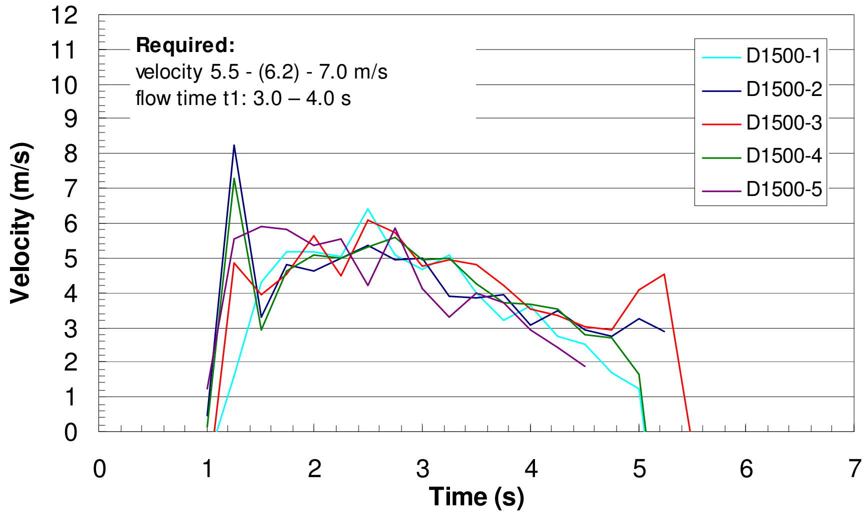

Required
velocity 6.5 - (7.6) - 8.5 m/s
flow time t1: 3.5 - 5.0 s
C3500-2
C3500-4
C3500-5
C3500-3
Velocity (m/s)
Time (s)

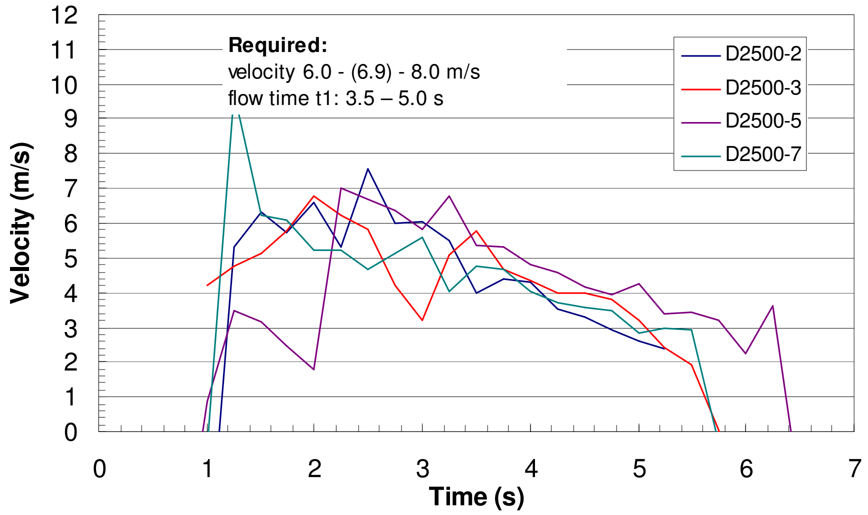

Required:
velocity 2.5 - (3.1) - 3.5 m/s
flow time t1: 1.5 – 2.5 s
D150-1
D150-4
D150-5
Velocity (m/s)
Time (s)

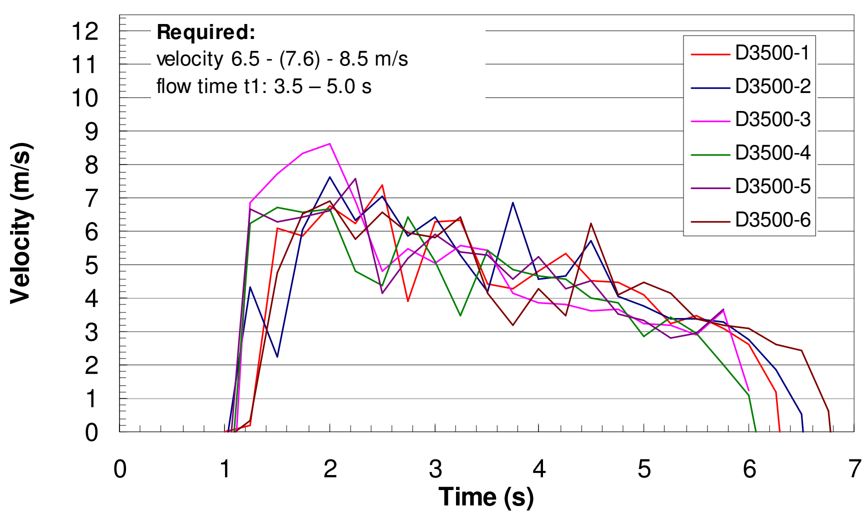

Required:
velocity 3.5 - (4.2) - 5.0 m/s
flow time t1: 2.0 – 3.0 s
D400-2
D400-3
D400-4
D400-5
Velocity (m/s)
Time (s)

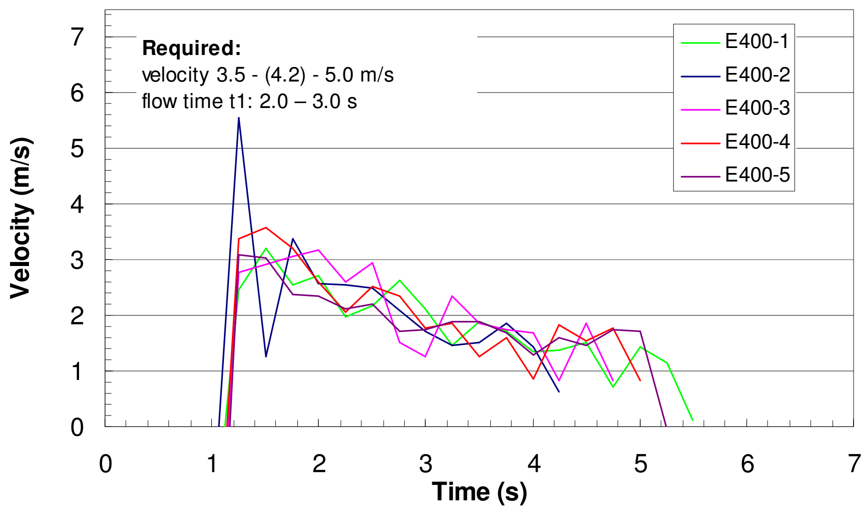

Required:
velocity4.2 - (5.0) - 5.7 m/s
flow time t1: 2.5 – 3.5 s
D700-1
D700-2
D700-3
D700-4
D700-5
Velocity (m/s)
Time (s)

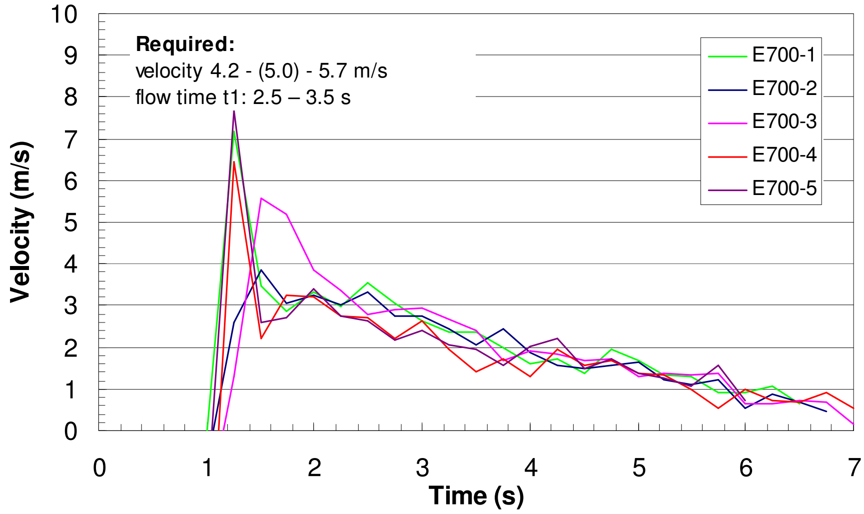

Required:
velocity 5.0 - (5.7) - 6.5 m/s
flow time t1: 3.0 - 4.0 s
D1000-1
D1000-2
D1000-3
D1000-4
D1000-5
D1000-6
Velocity (m/s)
Time (s)

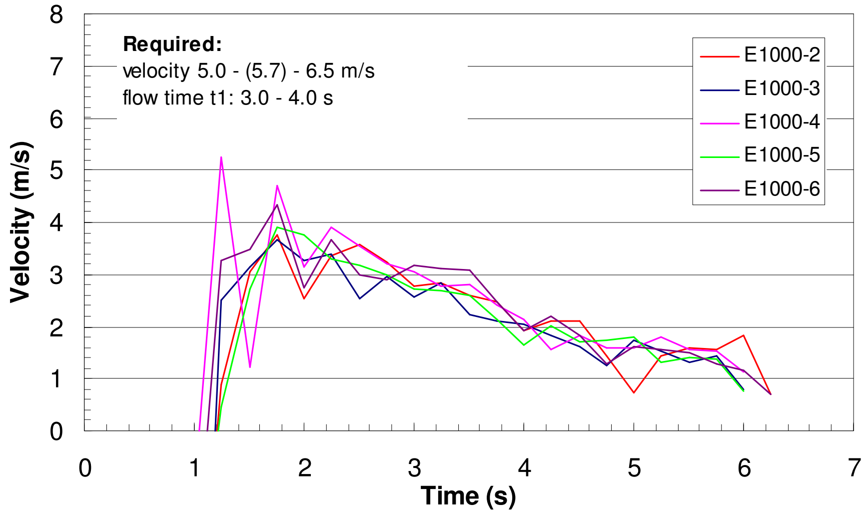

Required:
velocity 5.5 - (6.2) - 7.0 m/s
flow time t1: 3.0 – 4.0 s
D1500-1
D1500-2
D1500-3
D1500-4
D1500-5
Velocity (m/s)
Time (s)

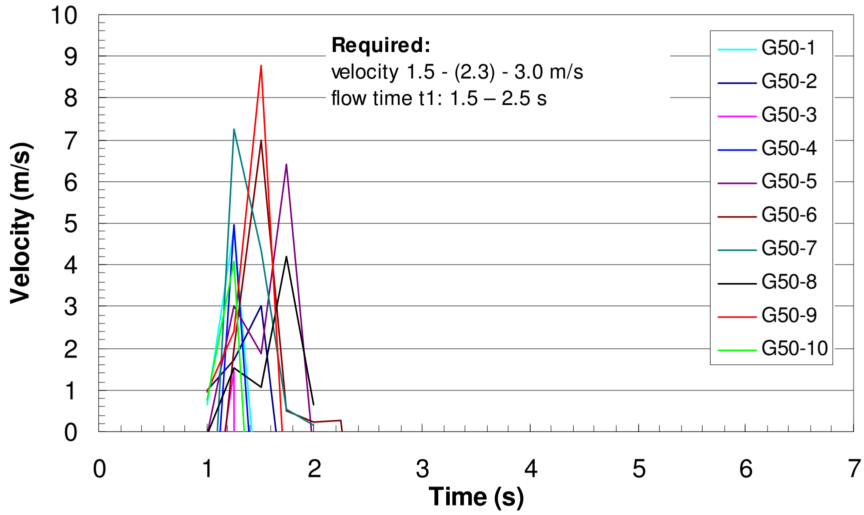

Required:
velocity 6.0 - (6.9) - 8.0 m/s
flow time t1: 3.5 – 5.0 s
D2500-2
D2500-3
D2500-5
D2500-7
Velocity (m/s)
Time (s)

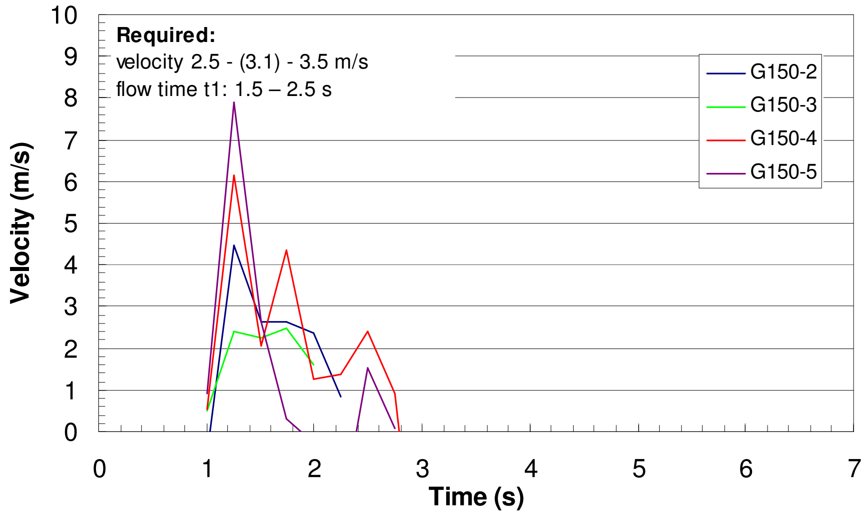

Required:
velocity 6.5 - (7.6) - 8.5 m/s
flow time t1: 3.5 – 5.0 s
D3500-1
D3500-2
D3500-3
D3500-4
D3500-5
D3500-6
Velocity (m/s)
Time (s)

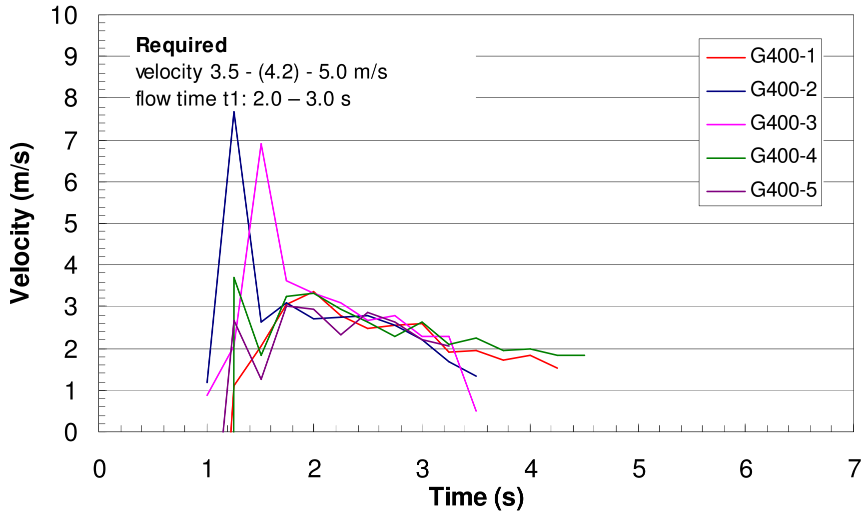

Required:
velocity 3.5 - (4.2) - 5.0 m/s
flow time t1: 2.0 – 3.0 s
E400-1
E400-2
E400-3
E400-4
E400-5
Velocity (m/s)
Time (s)

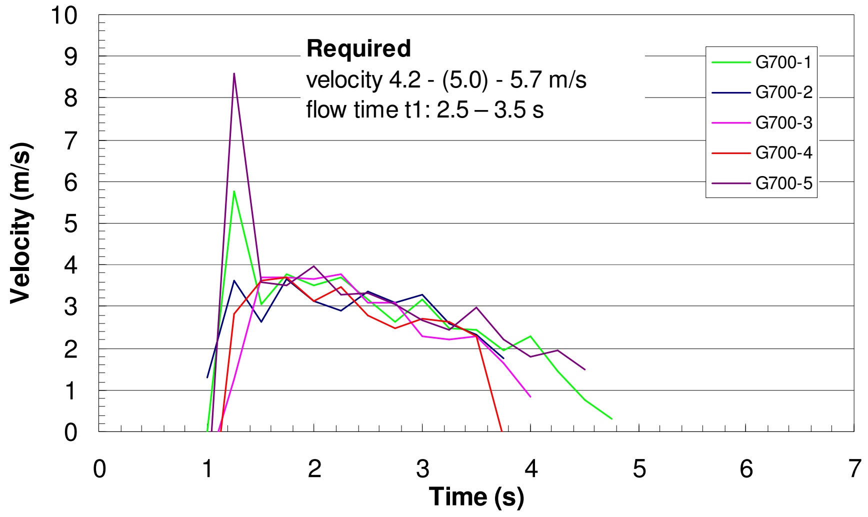

Required:
velocity 4.2 - (5.0) - 5.7 m/s
flow time t1: 2.5 – 3.5 s
E700-1
E700-2
E700-3
E700-4
E700-5
Velocity (m/s)
Time (s)

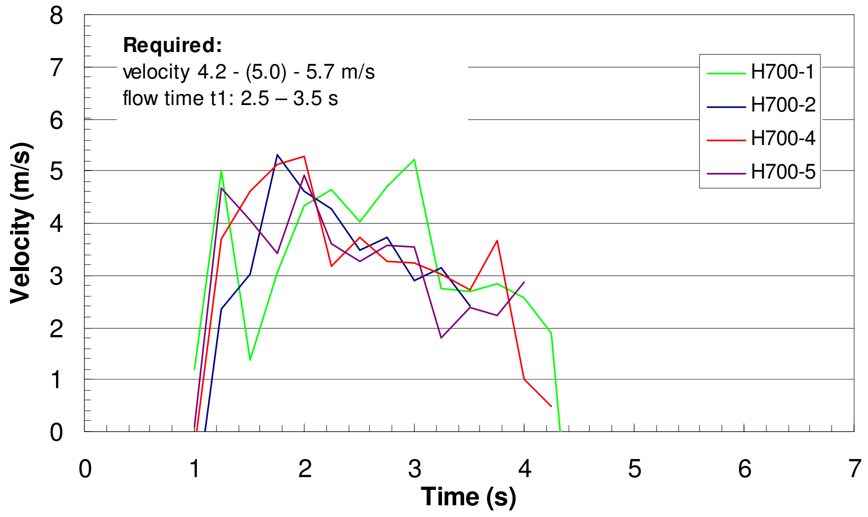

Required:
velocity 5.0 - (5.7) - 6.5 m/s
flow time t1: 3.0 - 4.0 s
E1000-2
E1000-3
E1000-4
E1000-5
E1000-6
Velocity (m/s)
Time (s)

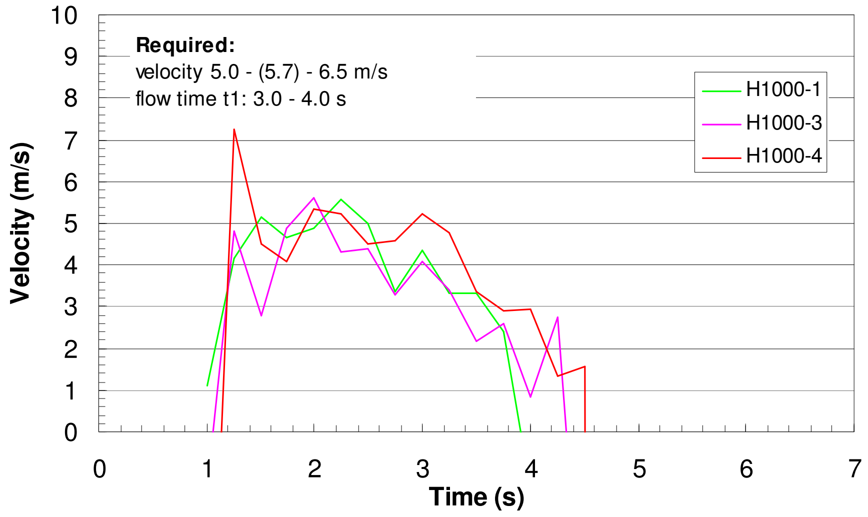

Required:
velocity 1.5 - (2.3) - 3.0 m/s
flow time t1: 1.5 – 2.5 s
G50-1
G50-2
G50-3
G50-4
G50-5
G50-6
G50-7
G50-8
G50-9
G50-10
Velocity (m/s)
Time (s)

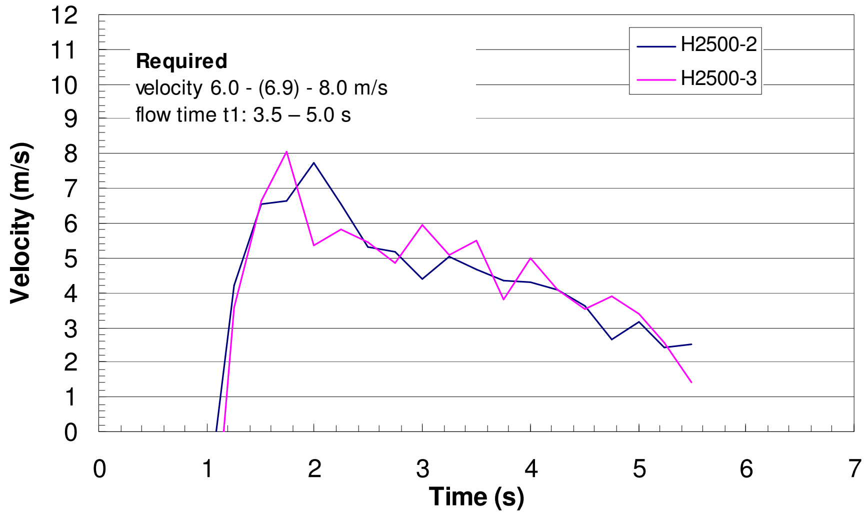

Required:
velocity 2.5 - (3.1) - 3.5 m/s
flow time t1: 1.5 – 2.5 s
G150-2
G150-3
G150-4
G150-5
Velocity (m/s)
Time (s)

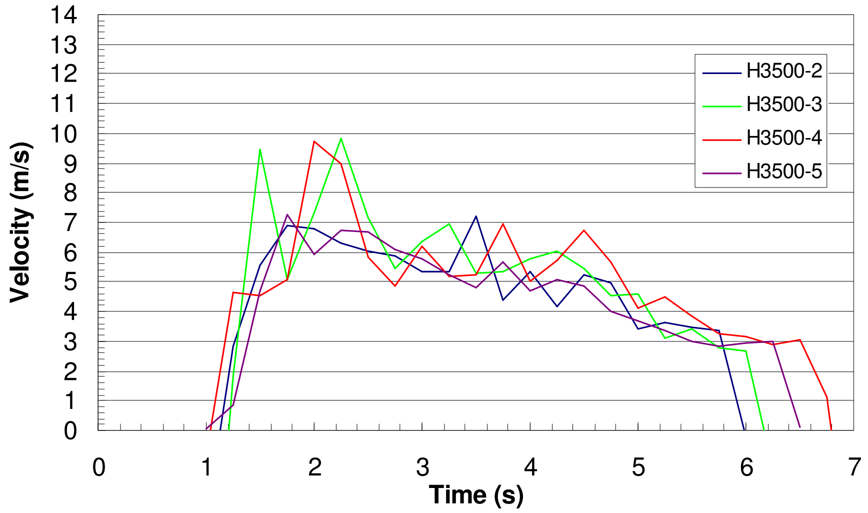

Required
velocity 3.5 - (4.2) - 5.0 m/s
flow time t1: 2.0 – 3.0 s
G400-1
G400-2
G400-3
G400-4
G400-5
Velocity (m/s)
Time (s)

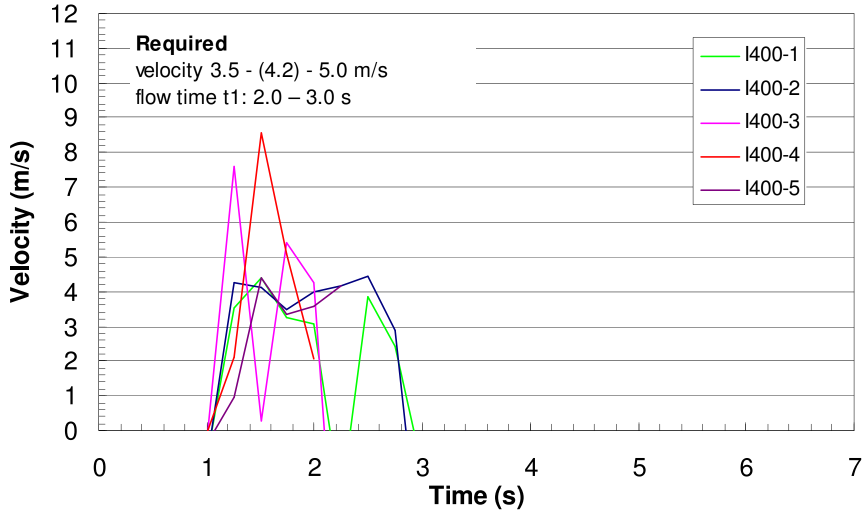

Required
velocity 4.2 - (5.0) - 5.7 m/s
flow time t1: 2.5 – 3.5 s
G700-1
G700-2
G700-3
G700-4
G700-5
Velocity (m/s)
Time (s)

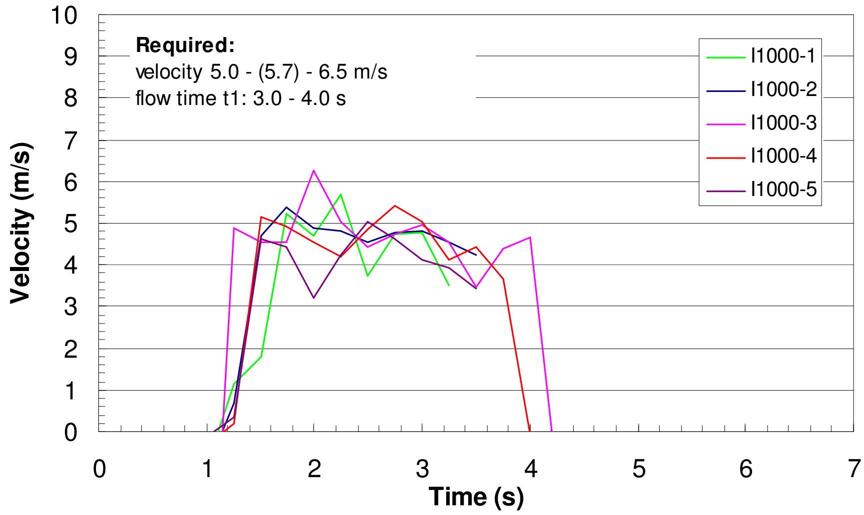

Required:
velocity 4.2 - (5.0) - 5.7 m/s
flow time t1: 2.5 – 3.5 s
H700-1
H700-2
H700-4
H700-5
Velocity (m/s)
Time (s)

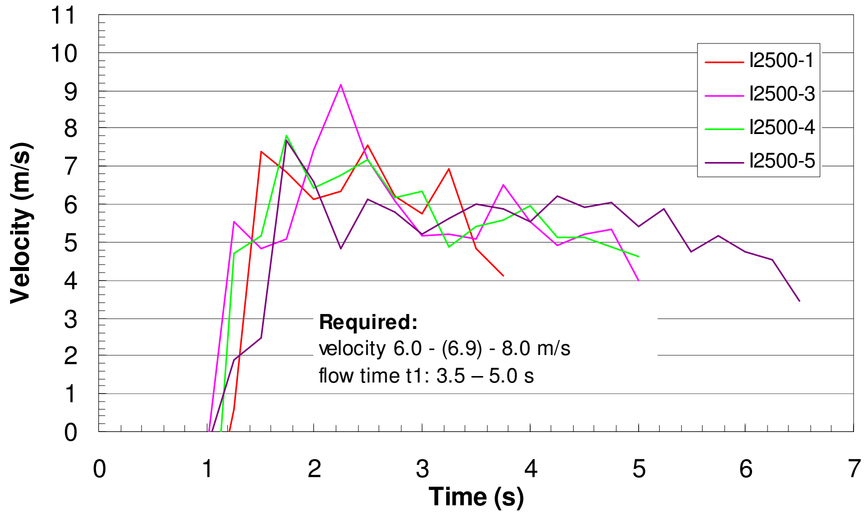

Required:
velocity 5.0 - (5.7) - 6.5 m/s
flow time t1: 3.0 - 4.0 s
H1000-1
H1000-3
H1000-4
Velocity (m/s)
Time (s)

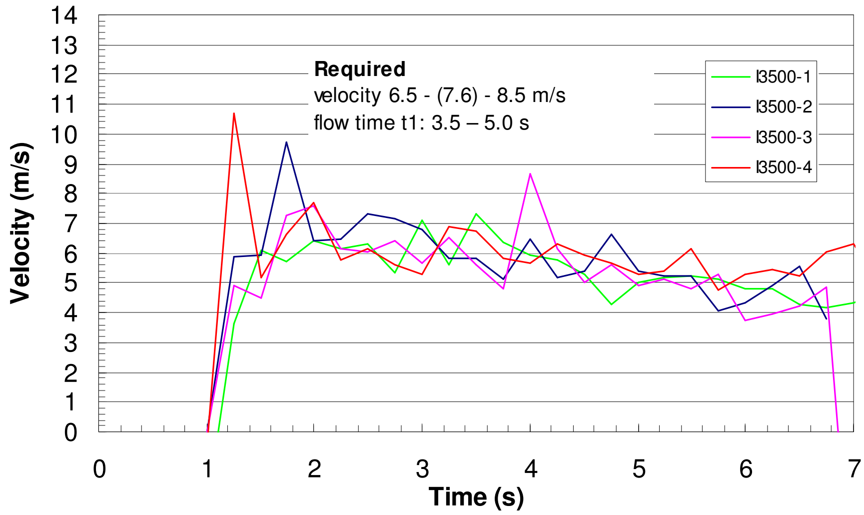

Required
velocity 6.0 - (6.9) - 8.0 m/s
flow time t1: 3.5 – 5.0 s
H2500-2
H2500-3
Velocity (m/s)
Time (s)

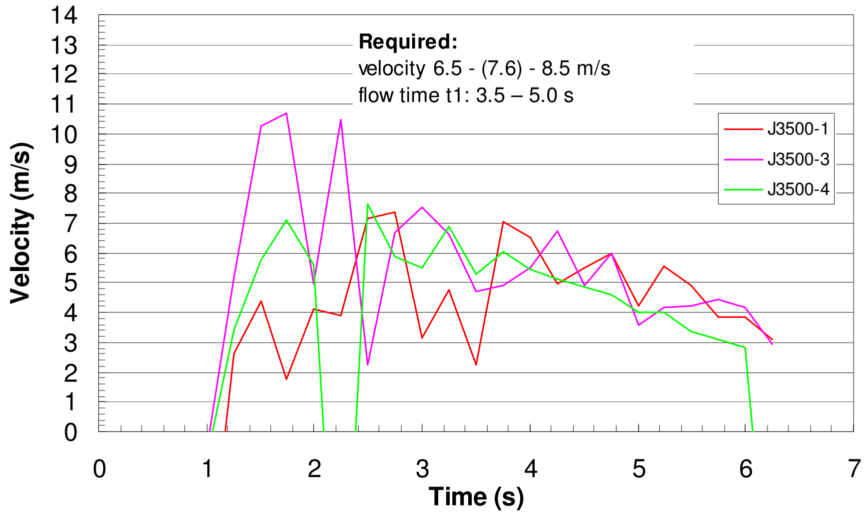

H3500-2
H3500-3
H3500-4
H3500-5
Velocity (m/s)
Time (s)

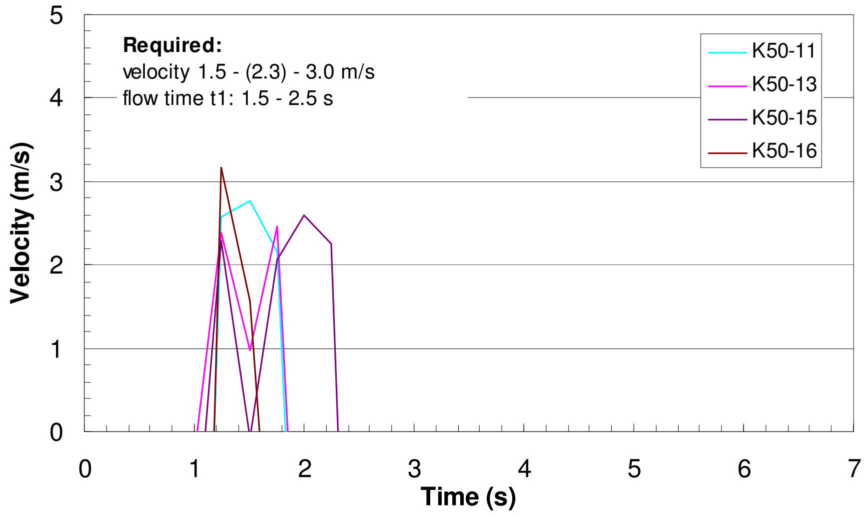

Required
velocity 3.5 - (4.2) - 5.0 m/s
flow time t1: 2.0 – 3.0 s
I400-1
I400-2
I400-3
I400-4
I400-5
Velocity (m/s)
Time (s)

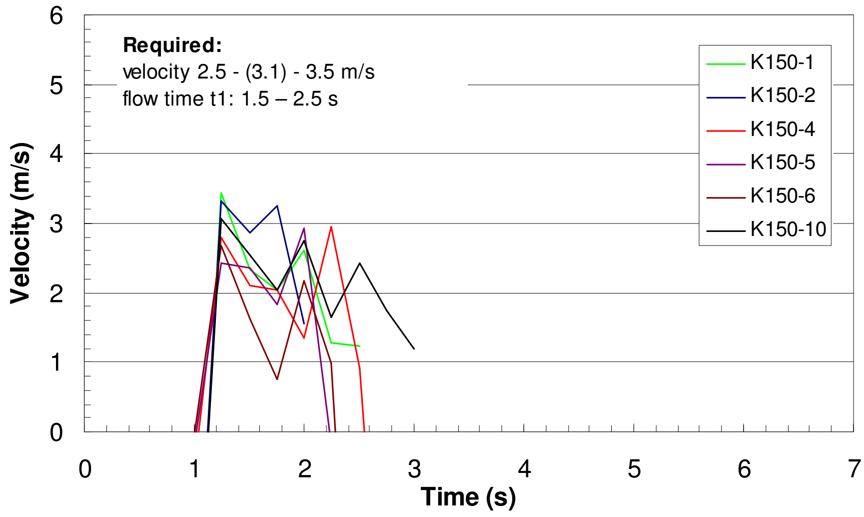

Required:
velocity 5.0 - (5.7) - 6.5 m/s
flow time t1: 3.0 - 4.0 s
I1000-1
I1000-2
I1000-3
I1000-4
I1000-5
Velocity (m/s)
Time (s)

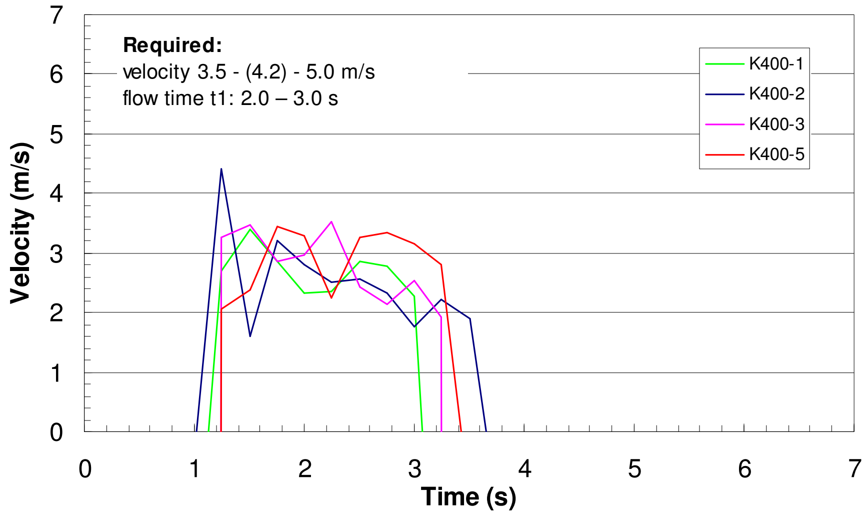

I2500-1
I2500-3
I2500-4
I2500-5
Required:
velocity 6.0 - (6.9) - 8.0 m/s
flow time t1: 3.5 – 5.0 s
Velocity (m/s)
Time (s)

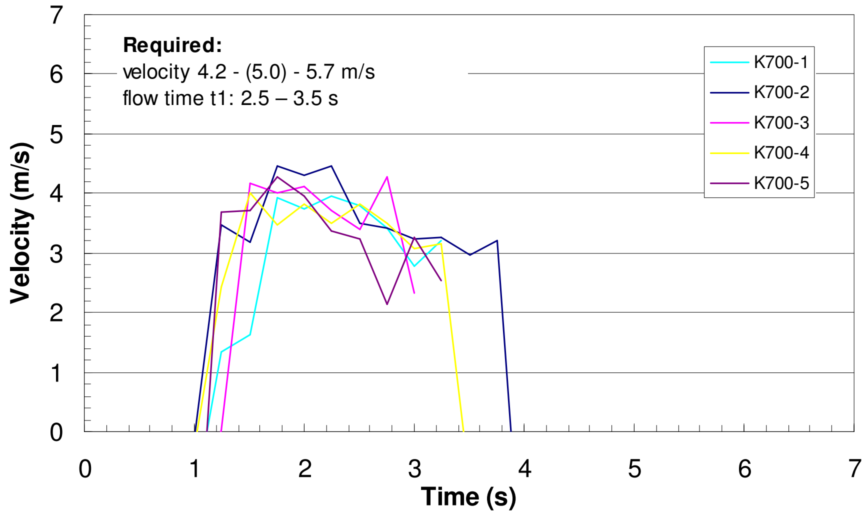

Required
velocity 6.5 - (7.6) - 8.5 m/s
flow time t1: 3.5 – 5.0 s
I3500-1
I3500-2
I3500-3
I3500-4
Velocity (m/s)
Time (s)

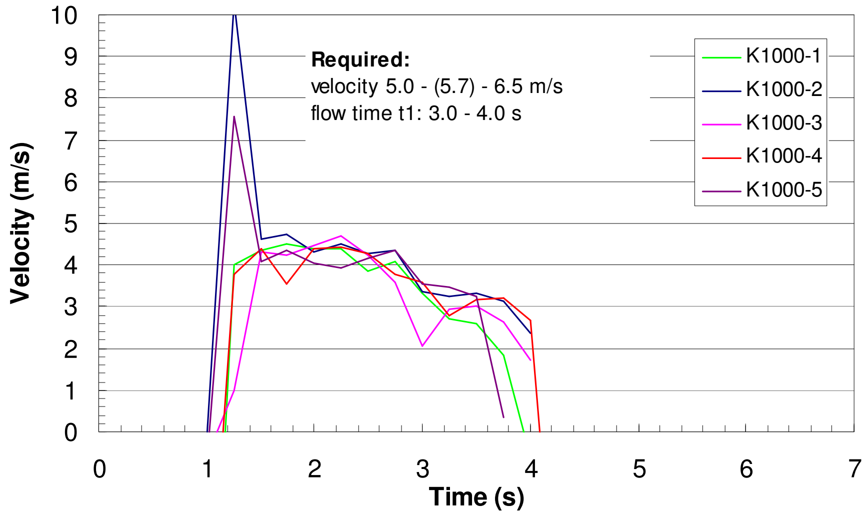

Required:
velocity 6.5 - (7.6) - 8.5 m/s
flow time t1: 3.5 – 5.0 s
J3500-1
J3500-3
J3500-4
Velocity (m/s)
Time (s)

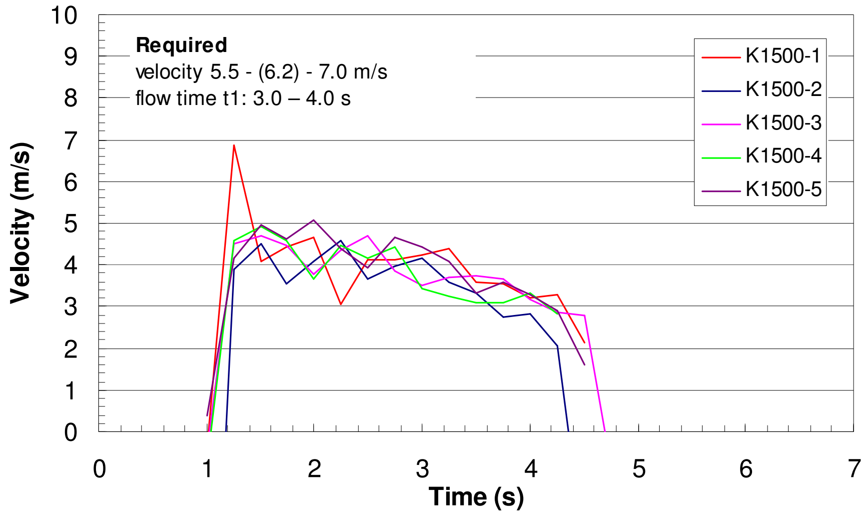

Required:
velocity 1.5 - (2.3) - 3.0 m/s
flow time t1: 1.5 - 2.5 s
K50-11
K50-13
K50-15
K50-16
Velocity (m/s)
Time (s)

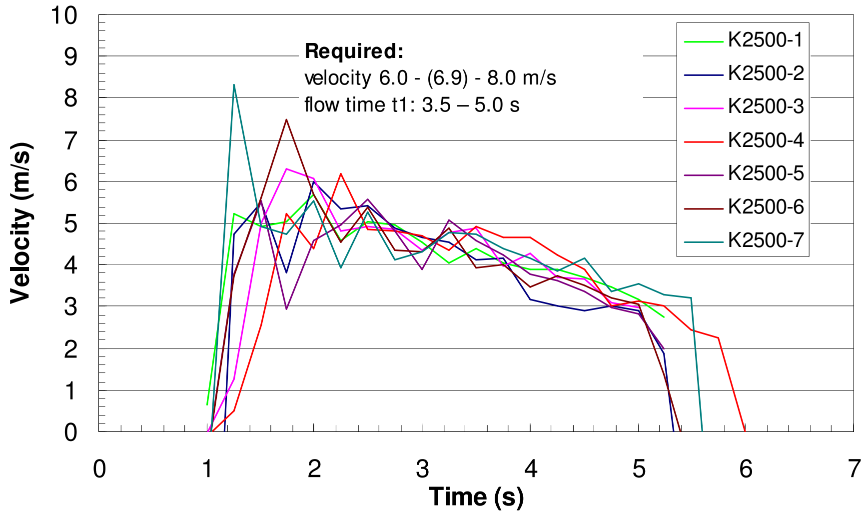

Required:
velocity 2.5 - (3.1) - 3.5 m/s
flow time t1: 1.5 – 2.5 s
K150-1
K150-2
K150-4
K150-5
K150-6
K150-10
Velocity (m/s)
Time (s)

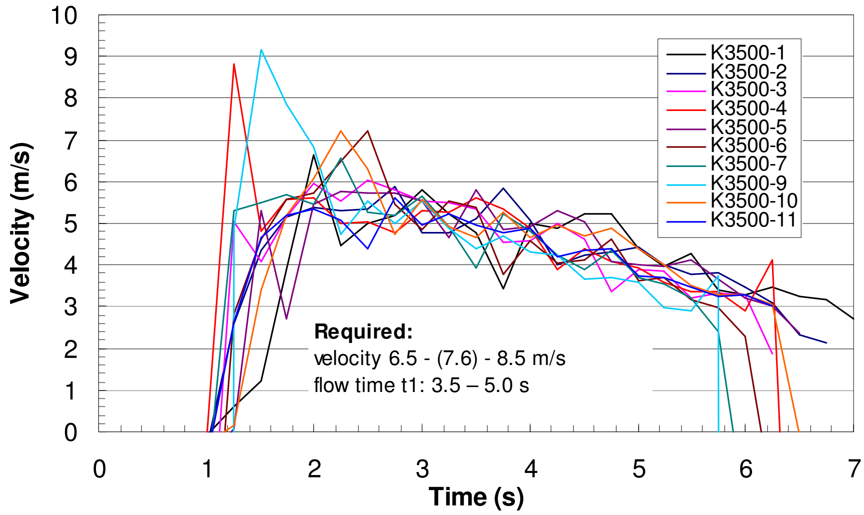

Required:
velocity 3.5 - (4.2) - 5.0 m/s
flow time t1: 2.0 – 3.0 s
K400-1
K400-2
K400-3
K400-5
Velocity (m/s)
Time (s)


Required:
velocity 4.2 - (5.0) - 5.7 m/s
flow time t1: 2.5 – 3.5 s
K700-1
K700-2
K700-3
K700-4
K700-5
Velocity (m/s)
Time (s)


Required:
velocity 5.0 - (5.7) - 6.5 m/s
flow time t1: 3.0 - 4.0 s
K1000-1
K1000-2
K1000-3
K1000-4
K1000-5
Velocity (m/s)
Time (s)


Required
velocity 5.5 - (6.2) - 7.0 m/s
flow time t1: 3.0 – 4.0 s
K1500-1
K1500-2
K1500-3
K1500-4
K1500-5
Velocity (m/s)
Time (s)


Required:
velocity 6.0 - (6.9) - 8.0 m/s
flow time t1: 3.5 – 5.0 s
K2500-1
K2500-2
K2500-3
K2500-4
K2500-5
K2500-6
K2500-7
Velocity (m/s)
Time (s)


K3500-1
K3500-2
K3500-3
K3500-4
K3500-5
K3500-6
K3500-7
K3500-9
K3500-10
K3500-11
Required:
velocity 6.5 - (7.6) - 8.5 m/s
flow time t1: 3.5 – 5.0 s
Velocity (m/s)
Time (s)