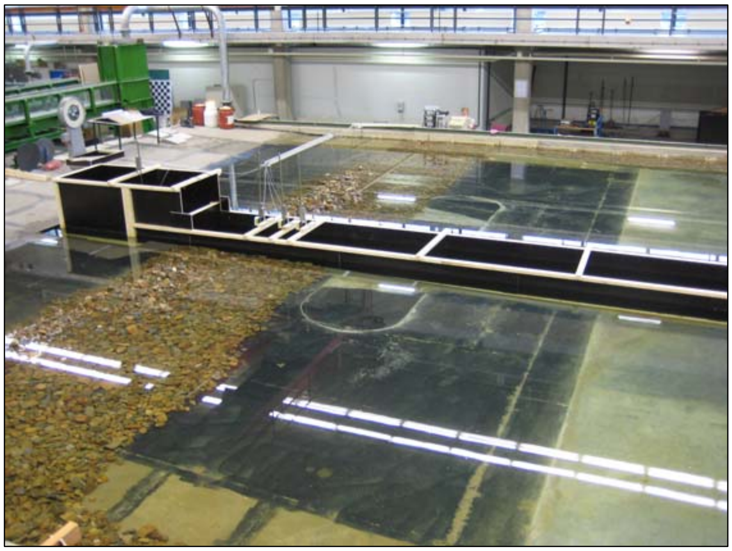
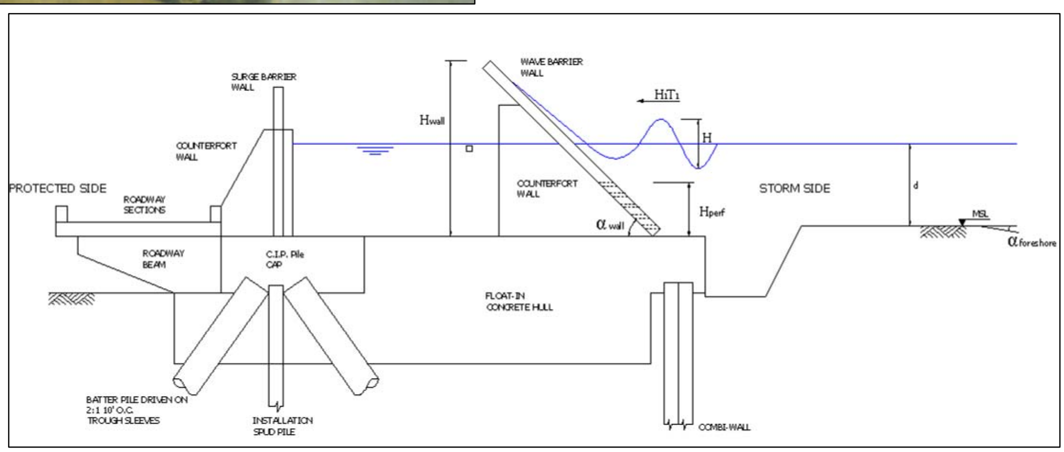
Thesis Master of Science

New Orleans storm surge barrier

An understanding of the hydraulic processes in front of and inside the superstructure





Delft, end February 2009

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Preface

This report is written within the scope of the master of science for the study of Civil Engineering and Geosciences at the Technical University Delft, The Netherlands. The research has been performed under the authority of the section of Hydraulic engineering and is carried out from April 2008 until January 2009. The research has been carried out in close cooperation with Iv-Infra B.V.

The main goal of this report is to obtain a proper insight in the hydraulic processes in front of and inside the superstructure of the New Orleans storm surge barrier consisting out of a perforated inclined wall in front of a vertical backwall.

I would like to thank the members of my graduation committee ir. H.J. Verhagen, ir. R.J. Labeur and ir. J van Spengen for their daily support and I would like to thank Prof. drs. Ir. J.K. Vrijling for his supervision during the research.
I also want to thank the people of the Fluid Mechanics Laboratory for all their support during the physical model testing.

Not in the least I also want to thank my friends and family from Heemskerk and in special my good friend Ruud; my friends Tranqui'lo and Koenis; Sara and my parents, who always supported me during my study at the university and made this financially possible.

Tom Sikkema
Heemskerk, January 2009

Summary

After New Orleans was struck by hurricane Katrina on the 29th of August a consortium including Iv-Infra was in a tender, which involved the design of a storm surge barrier at the east side of the city of New Orleans. This consortium designed a piled-supported structure including a concrete superstructure. The superstructure consists of a perforated inclined wall in front of a vertical backwall. The inclined wall has an energy dissipating function and the vertical backwall does have a water retaining function.

However, there is no standard theory to determine the optimal configuration of the superstructure having such a configuration. To find the optimal configuration total insight in the occurring hydraulic processes in front of and inside the superstructure is a requisite. In this research an analytical analysis has been performed for the water movement in the basin of the superstructure; for the wave run-up and wave overtopping over the perforated inclined wall; for the emptying process of the basin of the superstructure; for the wave induced inflow through the gap at the bottom of the inclined wall and for the length spreading effect, which is the spreading of the extreme overtopping rates in longitudinal direction of the basin. In this analytical analysis a basic model has been used to describe the above mentioned hydraulic processes for simplifications of the reference design and for the reference design itself. This model encloses a perforated rectangular basin, wherein a standing wave is present excited by a discharge through the gap which is the result of an incoming wave. In the basic model the Long Wave Theory has been used in order to have a simple mathematical description.

In the analytical research a reduction factor was found for the influence of the gap in the inclined wall on the wave run-up and wave overtopping. The reduction factor is equal to the reflection coefficient and has to be implemented in the formula for a non-perforated inclined wall. In this analytical research it was also stated that the inclined wall did not influence the water movement in a different way than a vertical wall should do. The water level fluctuation would resonate for a basin width of 0,5L. The flow through the gap can be described with the basic model and is based on a discharge relation and the preservation of volume over the gap.

Physical model tests have been performed to solve the encountered uncertainties, which were encountered during the analytical research. Besides solving the encountered uncertainties verifying the analytical results was an objective of the physical model testing.

From the physical model testing it has become clear that the gap has no influence on the wave run-up, however it influences the wave overtopping. An explanation for this phenomenon is that the overtopping time decreases when a gap is implemented in the inclined wall.

By comparing and combining the results obtained by the analytical research and the physical model testing the hydraulic processes have become clear and a numerical model combining these different hydraulic processes has been constructed.

For determining the wave run-up for a perforated inclined wall the formula for a non-perforated inclined wall has to be used. For determining the wave overtopping over a perforated inclined wall the formula for a non-perforated inclined wall has to be used including a reduction coefficient, which is based on the model testing results and not on the reflection coefficient. From the model testing it can be concluded that the reflection coefficient overestimates the reduction. The flow through the gap, which includes the wave induced inflow and the outflow due to head difference, can be described with the basic model based on the Long Wave Theory, the discharge relation and the preservation of volume.

The numerical model provides one a tool to calculate the overtopping volumes over the vertical backwall; the outflow through the gap and the overtopping volumes over the inclined wall. It is not possible to perform an automatic optimization with the model. One has to check different configurations whether they fulfil the imposed criteria or not. The most important criteria are that the structure has to be as low as possible and that the overtopping discharge over the vertical backwall is not allowed to be larger than 0,1l/s/m. A configuration consisting of an 1:1 sloped inclined wall with

height of 8m and a gap of 1m at the bottom of the wall and a vertical backwall with the same height with a distance of 7m from the inclined wall based on still water level is the result of using the numerical model. The water movement in the basin and the wave induced inflow are not included in the numerical model. However, from the analytical research it is known that the water level fluctuation for a basin width of 7m does not resonate. Therefore the water movement is included in the determination of the configuration.

Content

Chapter 1 Introduction

1.1 Introduction

In the last phase of the study Civil Engineering a master thesis has to be carried out by each student in order to graduate. In this thesis the student has to show all the engineering skills he learned during his study and prove that he can work individually. The work of this thesis can be carried out at the university or under the supervision of a company.
This master thesis will be carried out for the company Iv-Infra and encompasses a subject that will be the basis for the research of two students. In the interest of the results and the size of the subject it is chosen to have two students perform an individual research within the same subject.

1.2 Problem analysis

1.2.1 General information

On the 29th of August 2005 New Orleans was struck by hurricane Katrina. This hurricane, one of the heaviest categories, caused a lot of damage and flooded a large part of New Orleans and the surrounding parishes. To avoid such a financial, emotional and ecological disaster in the future a plan was made to improve the hurricane protection system of the city.

Iv-Infra was part of a consortium in a tender for an order which involves the design of a storm surge barrier at the east side of the city. The main criterion for this storm surge barrier design is the construction time. Certain levels of safety[1] should be reached within a short period and therefore a design is made which has a relative short construction time and consists mainly of prefabricated- and modular elements.

The design was initiated from a type of floodwall that is often used in the U.S.A., the so called T-wall. Using a T-wall type of structure in these conditions would result in huge deformation of the wall itself and the foundation in the subsoil. Therefore the T-wall design was adjusted into a structure shown in the figure below.

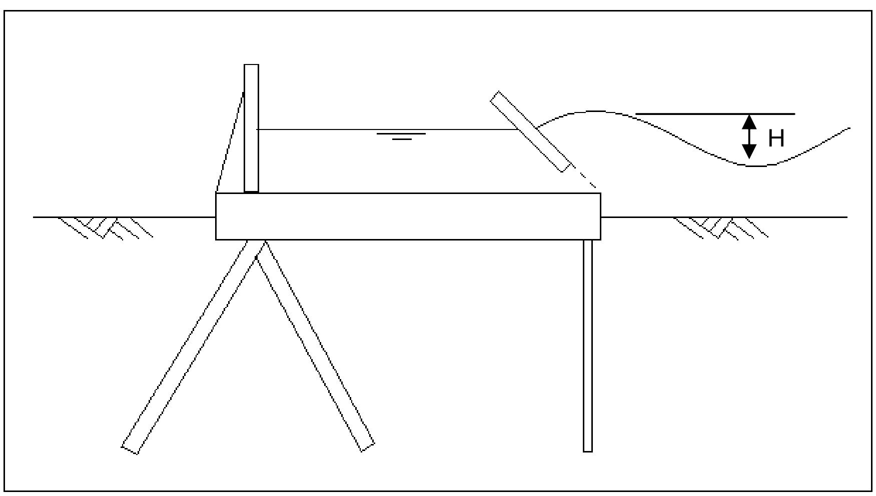


Figure 1-1: Schematized impression of the structure

[1] **First level of safety should be reached in June 2009: 1/50yr storm condition**
Second level of safety should be reached in June 2011: 1/100yr storm condition

Within this design a separation of functions is made. The inclined wall will dissipate the energy of the incident waves under storm conditions. The vertical wall, which has a water retaining function, will prevent inundation. Waves overtopping the inclined wall are allowed in the design and the water collected in the basin between the two walls will be drained back to the seaside through the perforated wall.

The building process starts with the construction of a combi wall after which a floated concrete hull will be attached water tight to the combi wall. From this base a battered pile foundation will be constructed, by hammering the piles through sleeves in the base. When the foundation is completed a vertical wall is constructed over the whole length, which will assure the first required level of safety. After this an inclined concrete wall will be constructed on the base of the structure in front of the vertical wall.

1.2.2 Project area

The storm surge barrier is constructed in Lake Borgne, situated at the east side of the city New Orleans. The storm surge barrier closes off Lake Borgne and the canals northwest and southwest of the lake. Figure 1-2 depicts the project area including Lake Borgne and both canals.
The canal northwest of Lake Borgne is the Gulf Intracoastal Water Way (GIWW) and a moveable gate is constructed in this canal, making navigation possible during normal conditions. The canal southwest of Lake Borgne is the Mississippi River Gulf Outlet (MRGO) and this canal is permanently closed off by the storm surge barrier. The storm surge is constructed in the western part of Lake Borgne. The trace of the storm surge barrier is given in Figure 1-3.

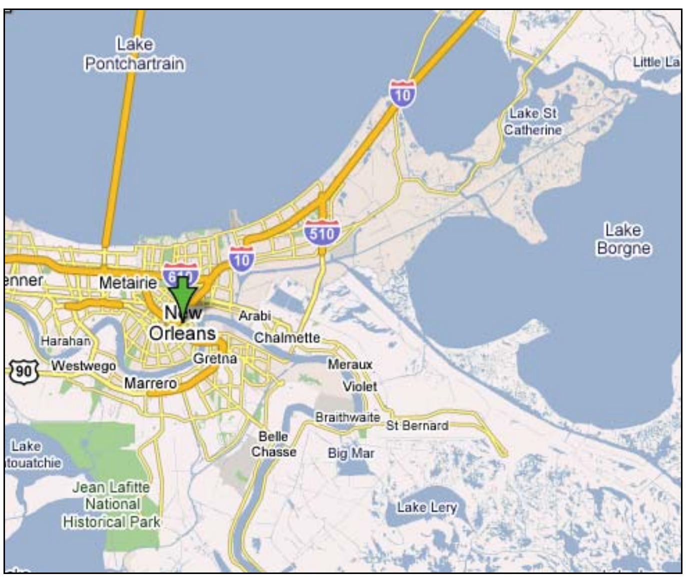


Figure 1-2: Project area

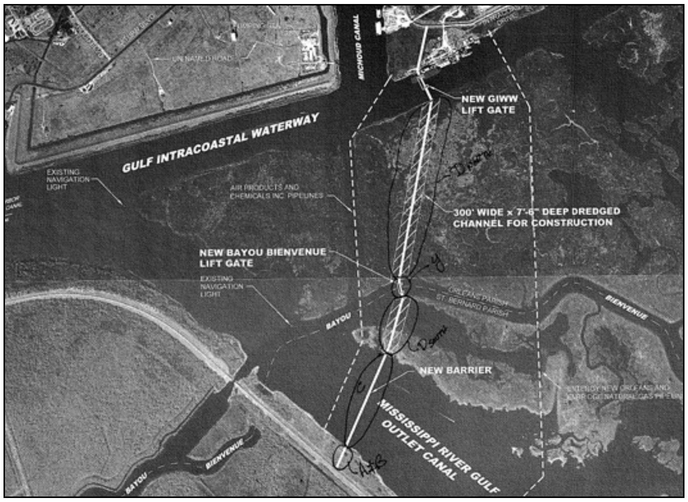


Figure 1-3: Barrier trace

1.2.3 Problem description

The pressure on the planning of the project and the local conditions resulted in the design described above. The dimensions of the elements of the structure were based on a quasi-static calculation with just one loading condition in the form of a storm surge and a single wave condition given by the United States Army Corps of Engineers (USACE). However, in reality the structure will be subjected to a whole wave field and besides a static load a large variable load occurs.

The variable loads on the inclined wall are not trivial and hard to predict. However, the design of the structure has already been made with a basic calculation method and is therefore not yet fully optimized.

A good understanding of the occurring processes inside the superstructure is essential for determining the optimal configuration of this superstructure.
In order to design more of these structures in flood protection works in the future, more insight in the loads on the structure and the processes occurring at the structure are needed.

A lot of research has been done on the subject of wave run-up; wave overtopping and wave loading, but a lot is still uncertain with respect to the coastal structures with inclined and perforated walls. Therefore research on this topic is needed to understand the processes in order to optimize the design for this storm surge barrier.

1.2.4 Problem definition

There is no standard theory to determine the optimal configuration of the superstructure of a permanent storm surge barrier, which consists of a perforated inclined wall having an energy dissipating function and a vertical wall having a water retaining function.

There is not sufficient insight into the (variable) loading on the perforated inclined wall of the superstructure.

1.3 Objectives

1.3.1 Research objectives

The research objectives of this master thesis are to get a better insight into the loads on the superstructure and to get insight in the hydraulic processes in front of and inside the superstructure which determine the optimal configuration of the superstructure. By means of these insights a better design can be made for this type of structures in the future.

1.3.2 Research questions

Main questions

1. What are the hydraulic processes in front of and inside the superstructure and in what manner do they effect the configuration?
2. What are the variable loads on a perforated and inclined wall?

Sub questions

The main question "What are the hydraulic processes in front of and inside the superstructure and in what manner do they effect the configuration" poses the following sub questions:

- Which parameters are of importance for the determination of the optimal configuration of the superstructure?
- What is the influence of the dimension of the different elements of the superstructure on the occurring processes?
- What is the optimal configuration of the superstructure based on minimal height of the superstructure?
- How does the emptying of the basin work?

The main question "What are the variable loads on the inclined wall?" poses the following sub questions:

- Which theories are of importance for the determination of the loads on a perforated and inclined wall?
- Which parameters are of importance for the determination of the loads on a perforated and inclined wall?
- What is the influence of the dimensions of the inclined wall on the loads on the inclined wall?
- What is the influence of variable conditions on the design?

As mentioned earlier two students perform an individual research within the same subject and the two defined main questions form the basis for two individual researches. The main question "What are the hydraulic processes in front of and inside the superstructure and in what manner do they effect the configuration?" forms the basis of the research performed by Tom Sikkema.
The main question "What are the variable loads on a perforated and inclined wall?" forms the basis of the research performed by Ruud Nooij.

1.4 Scope of the research

This research is based on getting insight in the hydraulic processes in front of and inside the superstructure of the New Orleans storm surge barrier in order to can optimize the configuration of the superstructure. These processes are wave run-up, wave overtopping and wave induced inflow for the perforated inclined wall and the emptying process and water movement of the basin of the superstructure.
The configuration of the superstructure forms the starting-point for the analysis of the hydraulic processes and the strength and stiffness of the structure are not taken into account in this research.
As the title already says only the superstructure is analyzed in this research. The foundation of the New Orleans storm surge barrier is very complex and important because of the very weak subsoil, however this is not the subject of the research and therefore not taken into account.

1.5 Outline of report

To find the optimal configuration of the superstructure of the New Orleans storm surge barrier insight in the hydraulic processes in front of and inside the superstructure is necessary. After a description of the environmental boundary conditions in chapter 2 and a description of the reference design including a critical review in chapter 3, an analysis of the hydraulic processes is performed in chapter 4. The different processes are described separately for simplifications of the reference design and thereafter for the reference design itself. In the analytical research assumptions will be made and uncertainties will be encountered. By means of physical model testing these assumptions are verified and the uncertainties tried to be solved. The physical model testing is described in chapter 5.
In chapter 6 the results from the analytical research and from the physical model testing are compared. The processes described in chapter 4 and in chapter 5 form the basis of the numerical model, which is described in chapter 7. Finally in chapter 8 conclusions are drawn and recommendations are given.

Chapter 2 Environmental boundary conditions

In Chapter 1 "Introduction" the research questions and objectives of this thesis have been described. In order to make calculations in the proceeding research the environmental boundary conditions have to be known. The determination of the environmental boundary conditions has been carried out in cooperation with Ruud Nooij, the student who performs the research to get insight in the variable loads on the perforated inclined wall. After the determination of the environmental boundary conditions in this chapter the reference design is described in Chapter 3 "Reference design".

2.1 Hydraulic boundary conditions

In this paragraph the hydraulic boundary conditions for the project area are described, which are the bathymetry; wave conditions; wind climate; water levels; tides and currents.

2.1.1 Bathymetry

A bottom profile from relative deep water to the toe of the structure is required and a length of 150km is assumed to be sufficient. At the 150km distance from the structure the water depth is 25m. In Figure 2-1 the length of the calculation area is presented. The straight line to the flood protection is taken to be representative for the calculation profile. The direction of the straight line is chosen in such a way that the profile is perpendicular to the structure.

From the figure it can be noticed that waves approaching from the Gulf of Mexico will pass barrier islands, a relative shallow area behind the barrier islands, marsh- and wetlands with a level just above MSL, the relative shallow Lake Borgne and again mash- and wetlands with a level at approximately MSL.

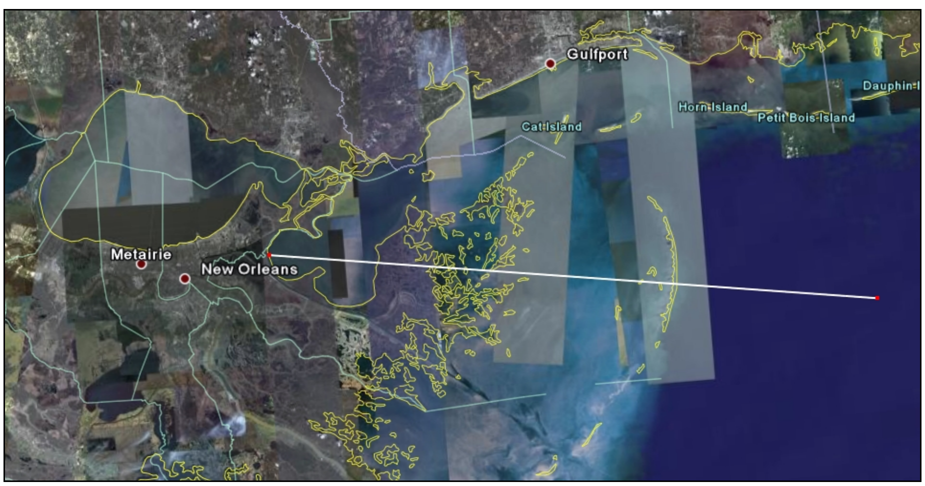


Figure 2-1: Calculation area used in SWAN

From the calculation area presented in Figure 2-1 a bottom profile is determined. For the part of Lake Borgne this is done by considering data from the National Oceanic and Atmospheric Administration (NOAA) database[2], Figure 2-2. For the part eastward of Lake Borgne the depth is estimated with the help of a depth chart used for the calculations with STWAVE, Figure 2-3.

[2] Source: NOAA's Geophysical Data system for Hydrographic Survey Data (CD-ROM data set) v. 3.3

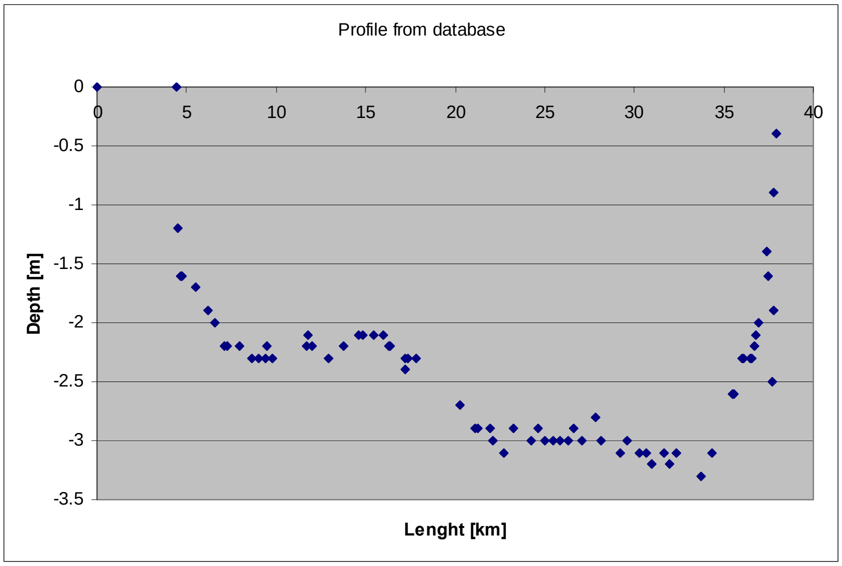


Figure 2-2: Bottom profile of Lake Borgne perpendicular to the structure obtained from NOAA database

In Figure 2-2 it can be seen that the profile between 0-4,5km has a level equal to mean sea level (MSL). In the rest of Lake Borgne an average depth 3m must be expected. In the first part a more shallow area with an average depth of approximately 2m is present, probably due to the existence of wetlands in the past. This can be confirmed by the geometry and the little bar still present around 16km from the structure. Around 25km the average depth of 3m is reached and at the 38km distance the marsh- and wetlands are reached.

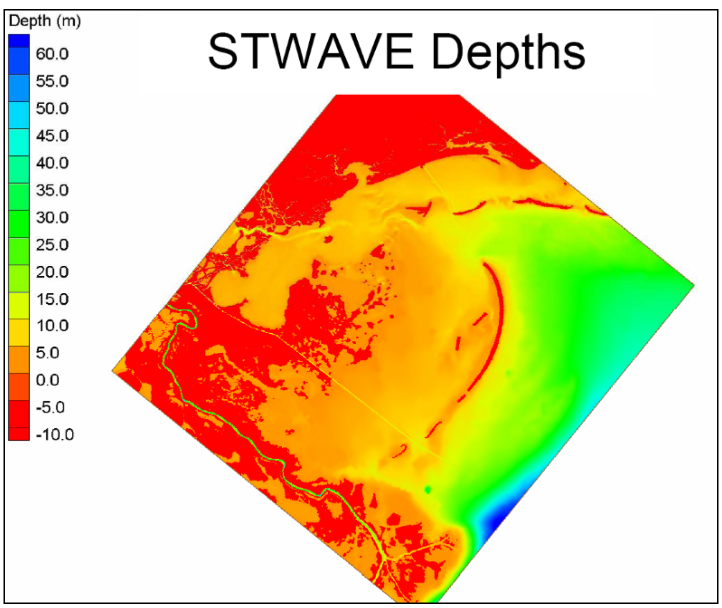


Figure 2-3: Water depth used for STWAVE calculations

The final bottom profile used for the calculations in the SWAN model is presented in Figure 2-4. In this figure the barrier islands can clearly be noticed around 110km from the toe of the structure. Behind the islands the depth increases fast to a relative large value. The mash- and wetlands can also be noticed from 38km to 65km from the toe of the structure and between the areas reaching above MSL also a relative shallow area is present.

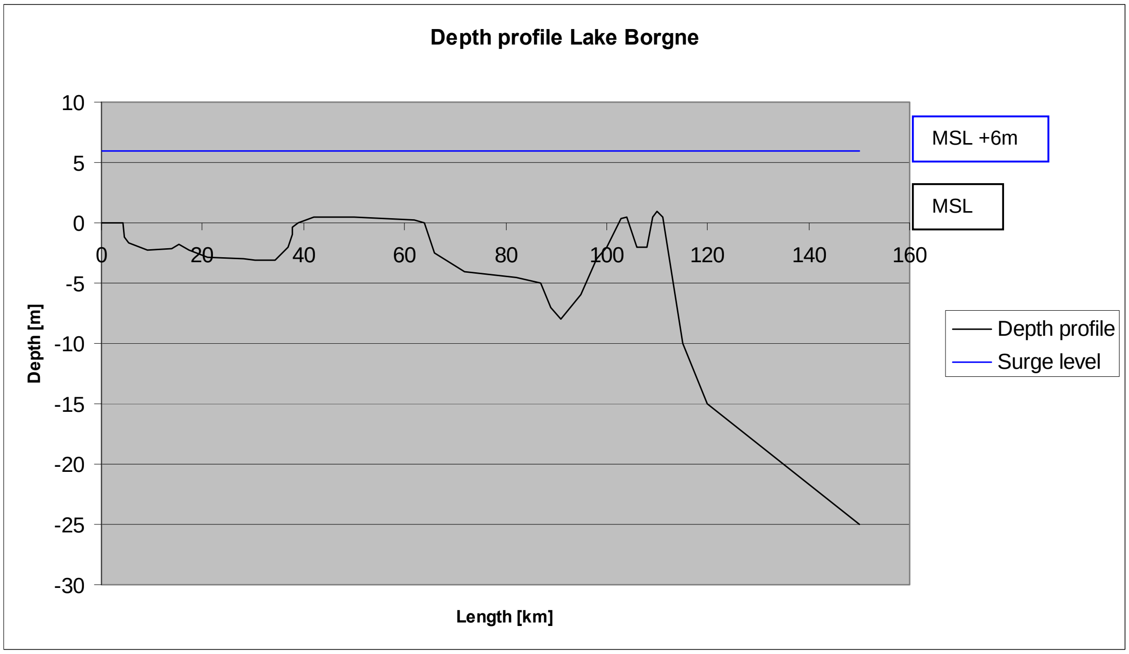


Figure 2-4: Profile Lake Borgne used in SWAN with 1/100year storm surge with respect to MSL indicated

2.1.2 Wave conditions

The USACE has performed calculations to obtain the wave parameters at the toe of the structure. Obviously the toe of the structure is in shallow water because of the wetlands and marsh areas existing in front of the structure at approximately MSL. In such conditions the calculation package STWAVE was used to calculate the near shore waves. The deep water conditions that served as input where based on a Planetary Boundary Layer Model (PBL) to generate wind fields and in addition WAM and ADCIRC where used to generate surge and offshore waves. In the model no friction was taken into account because of the lack of data, which resulted in higher values and thereby later on a conservative design.

However, just one safety level is considered by the USACE, being the design level of safety for a 1/100year storm event. This level seems rather low and the conditions for higher levels of safety are given in probability exceedance curves.

Besides this the USACE only provided a surge, a significant wave height and a wave peak period. These three parameters are not sufficient for the research. Since it is important to have more knowledge about the wave parameters and the wave spectrum at the toe of the structure the calculation package SWAN (Simulating WAves Nearshore) is used to obtain such a spectrum. It is expected that the wave height and wave period calculated by SWAN are approximately the same as the wave height and wave period provided by the USACE. The determination of the wave conditions has been performed in paragraph 2.2.

2.1.3 Wind climate

New Orleans is located in the hurricane belt and therefore hurricanes and other storms can be expected during the year, especially in the hurricane season. A hurricane can cause wind speeds larger than 250km/hr.

Saffir-Simpson

The Saffir-Simpson Hurricane Scale is a scale rating hurricanes from 1 to 5 based on the hurricane's intensity. This scale is set up by Herbert Saffir and Bob Simpson (1969) and used to give an estimate of the potential property damage and flooding expected along the coast when a hurricane makes landfall. Wind speed is the determining factor in the scale, as storm surge values are highly dependent on the parameters of the region. A hurricane with a category 2, 3, 4, 5 is respectively 10, 50, 100, 250 times as destructive as a category 1 hurricane. The Hurricane Scale is presented in Table 2-1.

It has to be noted that all winds presented in this table are based on the U.S.A. 1-minute average and that the storm surge is sensitive to several conditions and therefore just an average.

Table 2-1: Saffir-Simpson Hurricane Scale

Hurricane Scale	Wind speed [km/hr]	Storm surge [m]
Category 1	119 - 153	1,5
Category 2	154 - 177	2,5
Category 3	178 - 209	3,5
Category 4	210 - 249	4,5
Category 5	> 249	> 5,5

A trend over the past years shows an increasing probability of occurrence of high category hurricanes in the future. The accompanying wind speeds are especially important when structures with a large height are designed. Besides probable large forces on protection works also other effects of the wind, like storm surge and waves have to be considered in the design.

2.1.4 Water levels and tides

As already mentioned in paragraph 2.1.1 the bottom profile to 4-5km from the structure has a level equal to MSL. This means no water is present during normal conditions, so water levels and tides are irrelevant for normal conditions. The water levels/surges for extreme conditions are given by Table 2-1. These water levels are dependent of the occurring hurricane or storm. Tides are not taken into account during storm conditions in this research.

2.1.5 Currents

Currents are also irrelevant during normal conditions, because no surge is present. During extreme conditions currents are also not taken into account in this research.

2.2 Wave conditions

In this paragraph the design wave conditions are determined by means of SWAN, because the wave parameters provided by USACE are not sufficient to perform the research.

2.2.1 Safety levels USACE

Besides the design level of safety the USACE also performed calculations for different levels of safety, by considering heavier storms. From these different levels of safety probability exceedance curves can be made and the conditions at the toe of the structure can be determined from these curves. In Figure 2-5 the probability exceedance curves for the surge, significant wave height and wave peak period are given.

In Figure 2-5 also the design level of protection (1,0E-02) and the accompanying design conditions are pointed out.

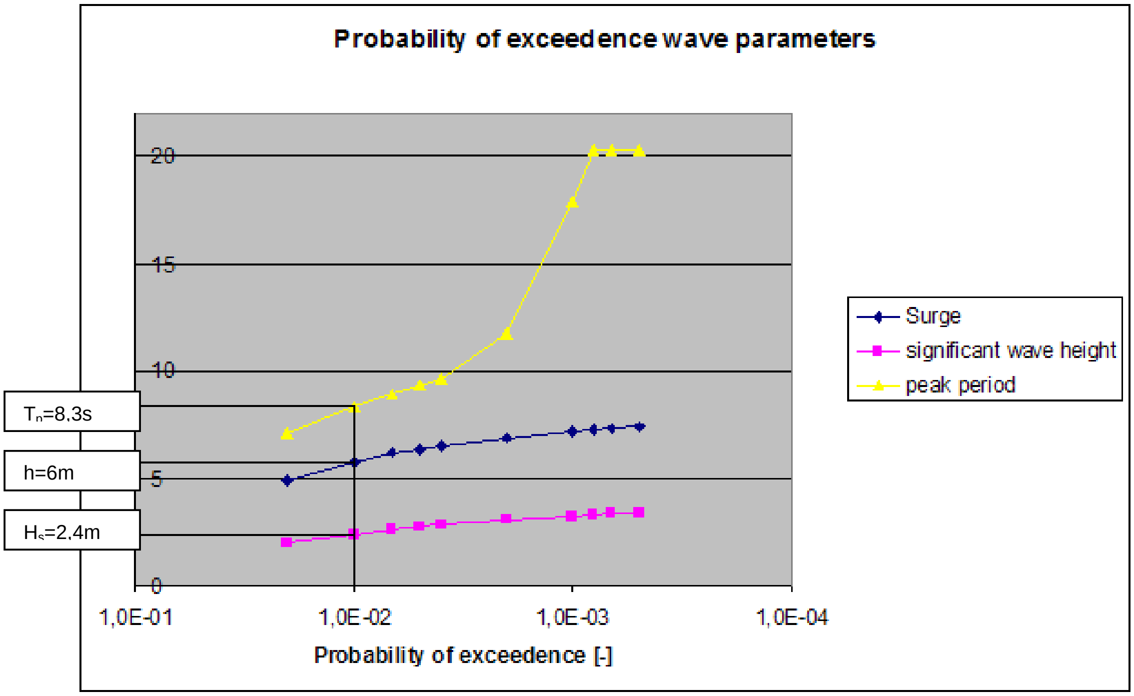


Figure 2-5: Probability of exceedance for different wave parameters given by USACE

In Table 2-2 some different safety levels are presented. If a safety level of an order 10 higher is chosen (1/1000year) the conditions change significantly, especially the peak period.

Table 2-2: Different safety levels

Safety level / Conditions	1/50years	**1/100years**	1/250years	1/1000years
Surge [m]	4,9	**6**	6,7	7,2
Significant wave height [m]	2	**2,4**	2,8	3,3
Wave peak period [s]	7,1	**7,9**	9,6	17,9

The larger values of the peak period can be explained by the fact that the peak period is shifting. The peak period is defined as the period in which the largest amount of energy is present. In the spectrum this is the frequency (f=1/T) with the largest energy. In a double peaked spectrum it can occur that due to wave breaking (wave energy dissipation) the largest peak is decreased and a smaller peak at another frequency becomes the largest. Hereby the peak period is shifted to another period. This phenomenon is also shown in Figure 2-5.
In the particular case presented in Figure 2-5 the shift of the peak period accounts to the larger surge, whereby less (swell) waves break and the largest energy is preserved at larger periods.

2.2.2 About SWAN

SWAN is a wave model for the simulation of waves in waters of deep, intermediate and finite depth. The following physical phenomena can be simulated with SWAN:

- Wave propagation in time and space, shoaling, refraction due to current and depth, frequency shifting due to currents and non-stationary depth.
- Wave generation by wind.
- Nonlinear wave-wave interactions (both quadruplets and triads).
- Whitecapping, bottom friction, and depth-induced breaking.

- Blocking of waves by current.

In order to make a good estimate of the wave characteristics at the toe of the structure by a calculation with SWAN the bottom profile, surge, significant wave height, peak period and wind speed have to be given as input in the model. The normative situation is during storm conditions and therefore storm conditions have to be imposed at the deep water boundary in SWAN.

Calculations with SWAN 1D are made here and it is assumed that these calculations are accurate enough for further analysis. This can be confirmed by the fact that the calculated waves near the toe of the structure in SWAN (and almost each other model) are mainly based on depth-induced breaking. This means that a 2D or 3D model will not give significantly more accurate results.

It has to be taken into account that the considered area is small with respect to the dimensions of the considered storms. This means that in the conditions are relatively the same in the considered area. When for example a large coastline is considered (larger than the dimensions of the storm), differences in conditions can be expected at different locations and therefore the conditions cannot be described by one single parameter for each location.

2.2.3 Input parameters SWAN

The most important boundary conditions that serve as input in SWAN are the bathymetry, the significant wave height, the peak period, the water level (surge) and wind velocity. The mean wave direction and wind direction are taken perpendicular to the structure.

In SWAN a certain average wind velocity, water level, significant wave height and wave period are assumed distinct from each other. This is of course a strong schematization of the situation occurring in reality, because the parameters are correlated. A storm produces wind fields, these wind fields produce waves at deep water and near the coastline this results in a storm surge and near shore waves. Therefore these parameters are related to each other. However, in this analysis the values presented below are used.

Wind speed
An average wind speed of 30m/s is assumed to occur during the normative storm.

Water level
The surge produced by this normative storm is assumed to be 6m.

Significant wave height and wave period
The significant wave height and wave period that serve as input in the model are assumed to occur during a hurricane at deep water. In the model two different combinations of wave heights and wave periods are tested, H_s=8m ; T_p=13s and H_s=10m ; T_p=15s respectively. These combinations are more or less characteristic for waves produced by a hurricane and the output of the model does not produce significant changes (see Table 2-4-Table 2-7). Therefore a H_s=8m and T_p=13s is assumed to be representative for a 1/100year storm and used for further calculations. The wave spectrum used in SWAN is a JONSWAP spectrum.

2.2.4 Comparison SWAN output with USACE values

To check whether the values are more or less the same a comparison is made between the output of SWAN and the values provided by the USACE. The output location is more or less the same; about 200m from the structure and the profile is also considered representative.

The output of SWAN and the parameters provided by USACE are presented in Table 2-3.

Table 2-3: Results of SWAN and USAGE

Wave parameters	SWAN 1D	USACE
H_s [m]	2,24	2,4
T_P [s]	6,45	8,3

The significant wave height closely reflects the parameters provided by the USACE. However, the peak period has a considerable smaller value. This can probably be addressed to the difference in the used models. The USACE used a 2D model instead of the 1D model which is used here. The governing direction for the approaching waves is therefore not necessary the same and that can result in the difference in the peak periods. For example the influence of wind speeds on the generation of waves. For a larger wind speed longer waves are generated as can be seen in Table 2-7.

2.2.5 Analysis of SWAN output

In Figure 2-6 the wave spectra at a distance of 150km (point1), 100km (point4), 50km (point7) and 100m (point9) are given. Figure 2-7 shows the location of the different points in the bottom profile. It can be seen that at point 1 a single peaked spectrum is present. This is due to the fact that point 1 is at the beginning of the calculation area and the spectrum is described by only one (swell) wave.
At point 4 the spectrum shows a second peak. The second peak can be addressed to the breaking of the (swell) waves on the barrier islands and the influence of the wind which generates waves with a different (smaller) period. The total wave energy is also reduced to about 10% of its initial value, which can be addressed to the loss of energy by depth induced breaking of waves.
At point 7, already in Lake Borgne in shallow water a spectrum with a single peak is present again. The peak period is shifted from 12,45s to 6,45s. This is due to the fact that the wind generated waves contain most energy now and therefore the peak period changes. There is also an increase in energy visible of about 30%, which is the result of wind generated waves.
At point 8 the toe of the structure is reached and the water depth is approximately 6m, the peak period stays the same and the wave energy and significant wave height reduces only due to friction and depth induced breaking.

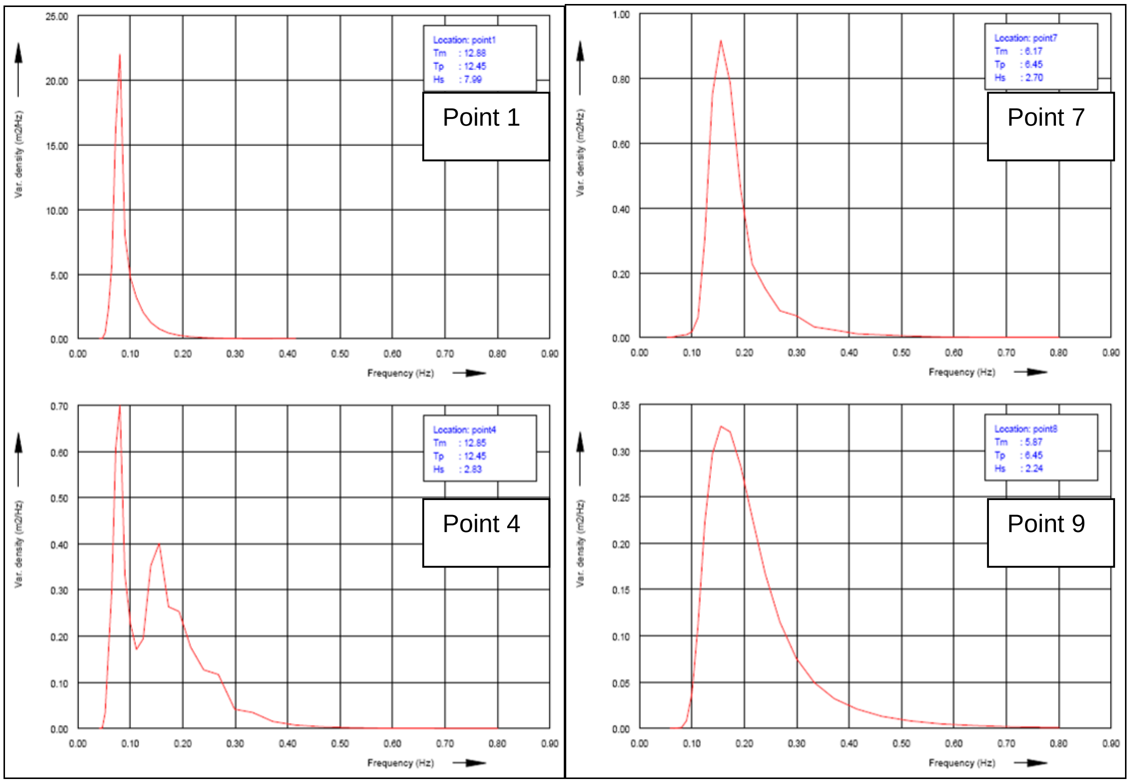


Figure 2-6: Change in wave spectrum along the profile [SWAN]

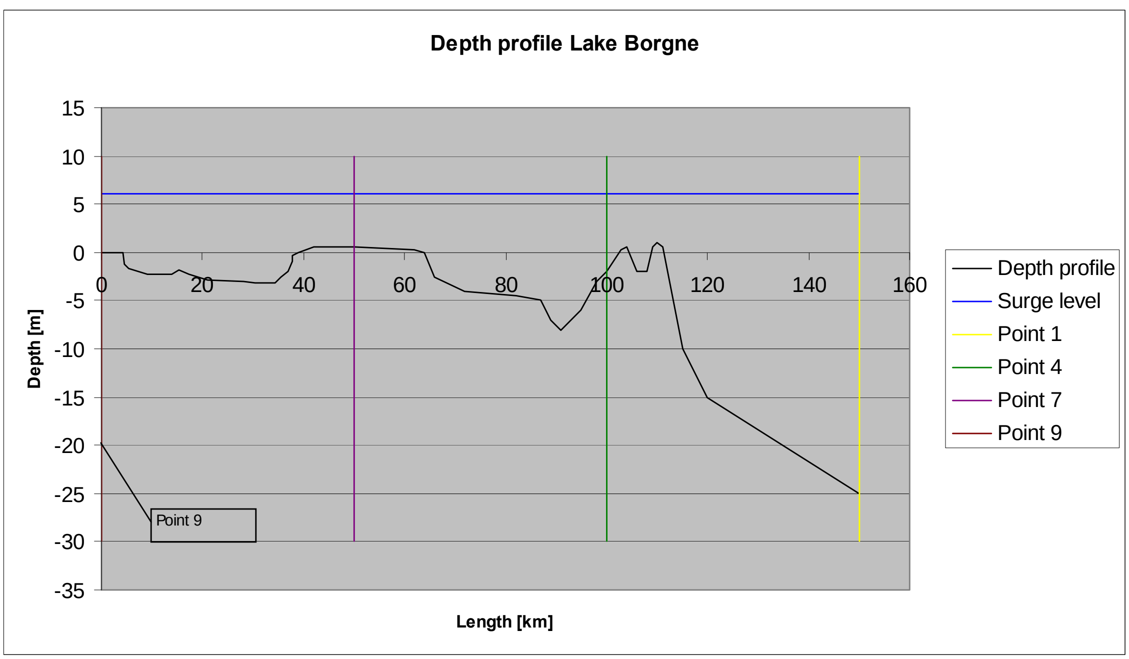


Figure 2-7: Depth profile Lake Borgne including spectrum points

In Figure 2-8 a plot of different parameters along the profile is given. The depth, the significant wave height and wave peak period are shown.
The peak period stays constant at first, but behind the barrier islands the peak period changes as also described above. This is due to the wave energy decrease by depth induced breaking of waves and the generation of wind waves with a different wave period. At the deeper parts of the profile the peak

period increases and at the shallower parts the peak period decreases. This phenomenon can be described by the same processes as described above.
The significant wave height is first increasing due to shoaling (and probable wind effects) where after it decreases due to friction and finally depth induced wave breaking. Furthermore it can be noticed that the wave height increases at the deeper parts along the profile and decreases along the more shallow parts. The larger depth and wind speed allow the waves to grow in height and the smaller depth induces wave breaking and loss of wave energy due to friction.

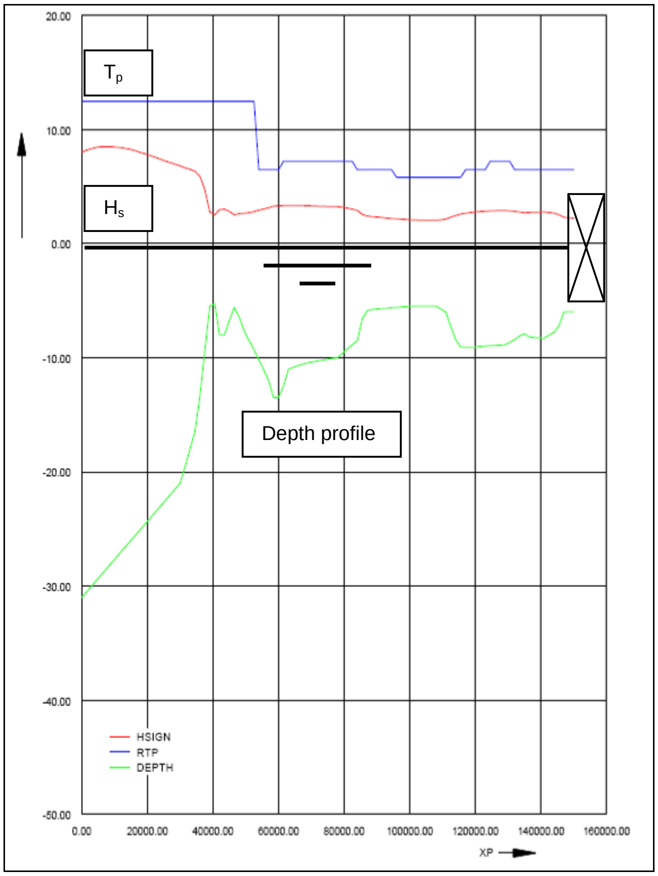


Figure 2-8: Plot of different parameters along the profile

2.2.6 Sensitivity to change of boundary conditions

Besides the wave parameters at the toe of the structure it is also interesting to see what happens if the boundary conditions like surge, wave height, wave period and wind speed are changed. Therefore a small sensitivity analysis is performed to see in what way variation of boundary conditions at deep water (different storm conditions) influences the wave conditions at the toe of the structure.

In this sensitivity analysis a larger and smaller value of the normative value of a certain wave parameter is put in the model and the conditions at the toe of the structure are calculated. By doing this, different conditions can be simulated and analyzed.

The output parameters of the analysis are the significant wave height, the peak period and the mean period at the toe of the structure.
Table 2-4 presents the results of the analysis for the surge; Table 2-5 presents the results of the analysis for the wave height; Table 2-6 presents the results of the analysis for the peak period; and Table 2-7 presents the results for the wind speed.

Table 2-4: Sensitivity analysis for the surge

Input parameter at 150km distance from the structure	Output parameters at the toe of the structure		
Surge [m] **With H_s= 8m; T_p= 13s; U_{wind}= 30m/s**	H_s [m]	T_p [s]	T_m [s]
4	1,68	5,79	5,27
6	2,24	6,45	5,87
8	2,79	7,20	6,46

Table 2-5: Sensitivity analysis for the wave height

Input parameter at 150km distance from the structure	Output parameters at the toe of the structure		
H_s [m] **With surge= 6m; T_p= 13s; U_{wind}= 30m/s**	H_s [m]	T_p [s]	T_m [s]
4	2,24	6,45	5,50
8	2,24	6,45	5,87
12	2,23	6,45	5,85

Table 2-6: Sensitivity analysis for the peak period

Input parameter at 150km distance from the structure	Output parameters at the toe of the structure		
T_p [s] **With surge= 6m; H_s= 8m; U_{wind}= 30m/s**	H_s [m]	T_p [s]	T_m [s]
10	2,23	6,45	5,88
13	2,24	6,45	5,87
16	2,23	6,45	5,88

Table 2-7: Sensitivity analysis for the wind speed

Input parameter at 150km distance from the structure	Output parameters at the toe of the structure		
U_{wind} [m/s] **With surge= 6m; H_s= 8m; T_p= 13s**	H_s [m]	T_p [s]	T_m [s]
10	0,99	4,17	4,07
30	2,24	6,45	5,87
50	2,69	7,20	6,28

From the SWAN output presented in the different tables it can be seen that the wave conditions at the toe of the structure are not sensitive to changes in the significant wave height or wave peak period. However, the wave conditions are sensitive to changes in water depth (surge) and wind velocity.
The reasons for this sensitivity are described earlier and can be addressed to the depth induced wave breaking, bottom friction and the generation of waves by the wind.

From Table 2-4 it can be seen that larger waves occur at larger water depths. When the wind speed is constant the wave parameters are totally determined by the surge (water depth). Therefore it can be expected that the wave height is linear with the surge, because the wave heights are limited by the depth. This can be shown by Figure 2-9, in which the output of SWAN is shown at the toe of the structure for a variable surge. Furthermore it can be noticed that the peak period and mean period are increasing with increasing surge. However, this increase is not linear.

Table 2-7 shows that for situation with larger wind speeds higher waves with larger periods can be expected. This is in line with the results of the nomograms found by Dorresteijn and Groen (Seawaves) for waves generated by wind during storms. In this nomogram the wave conditions are dependent on the storm duration, average wind speed and fetch. Although the wind speed is only varying the same results as presented in the nomograms can be noticed, namely an increase in wave height and wave period with an increase in wind speed. In Figure 2-10 the change of wave parameters to variable wind speeds is presented. It can be seen that the influence of the wind speed is gradually decreasing for the wave height. This can be explained by the fact that the waves at the toe of structure are still depth limited and converge asymptotic to approximately 3m (H=1/2xd).

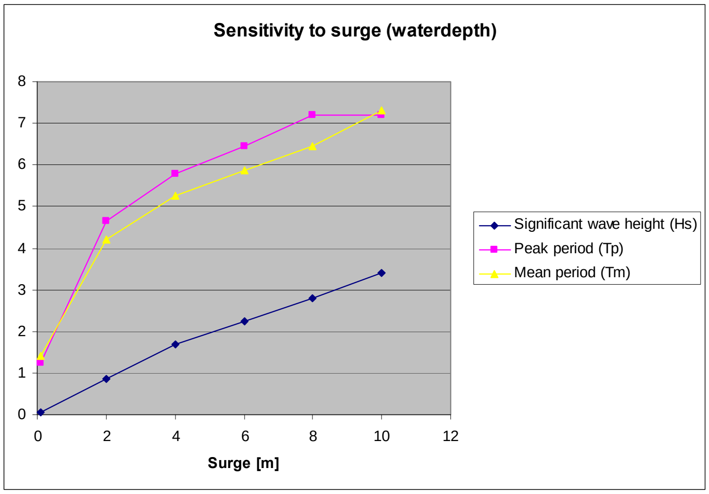


Figure 2-9: Sensitivity of wave parameters to variable surge at the toe of the structure obtained by SWAN

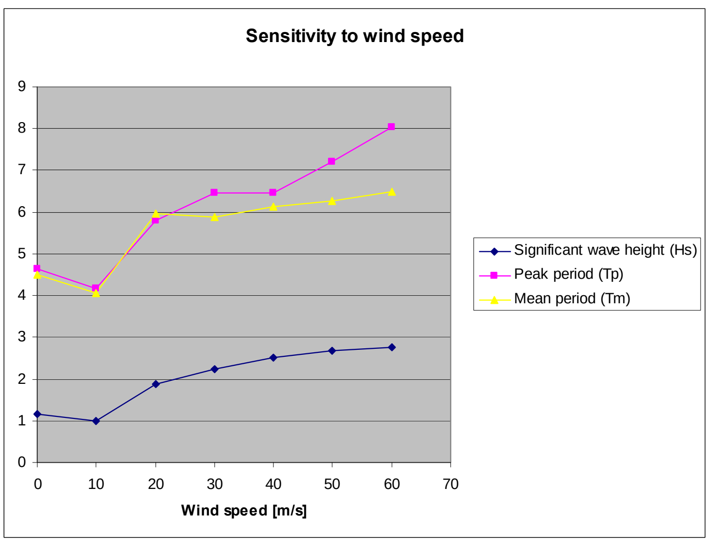


Figure 2-10: Sensitivity of wave parameters to variable wind speeds at the toe of the structure obtained by SWAN

So, from this crude sensitivity analysis it can be concluded that the wind speed and the storm surge are important in the determination of the significant wave height and the spectral peak period. As mentioned before the parameters are correlated. The wave parameters are fully determined by the water depth and thereby the storm surge. The storm surge is fully determined by the wind speed and wind direction. So only the wind speeds from the storm are determining in the production of the near shore conditions. Since the wind speeds are not accurately known their values are assumed. Therefore it seems logical to express the probability of exceedance not in the form of wind speeds, but in the form of storm surge.

2.2.7 Design wave conditions

The design wave conditions are based on the 1/100year safety level. For further research wave parameters are needed to calculate processes like wave overtopping, wave run-up and wave impact.

To design hard structures also the $H_{2\%}$ is required. Therefore the design wave conditions are determined in this paragraph.

Wave conditions from SWAN

The wave conditions at the toe of the structure are determined with the help of SWAN. The following boundary conditions are used as input parameters for the program:

H_s:	8m
T_p:	13s
Surge:	6m
Wind velocity:	30m/s

Three different output datasets are provided by SWAN, namely:

- Table with parameters
- Plot of spectrum with H_s, T_p and T_m
- Table with spectral data

From the spectral data different parameters have been calculated. For each location of the profile the spectral energy has been given in the spectral data and therefore the spectral moments can be determined. The spectral periods T_{m0}, $T_{m-1,0}$ and the spectral wave height H_{m0} are calculated from the moments. The parameters from the spectral data are only calculated for the near structure location. In Table 2-8 the different output parameters are presented. It is clear that the wave heights obtained from the three different datasets are the same.

Table 2-8: Wave conditions from SWAN for 1/100year design condition

Distance from structure [m]	Wave height/ wave period	H_{m0} [m] Table	H_{m0} [m] Plot	H_{m0} [m] Spectral data	T_m [s] Plot	T_p [s] Plot	T_{m02} [s] Spectral data (m_0/m_2)	$T_{m-1,0}$ [s] Spectral data
150000		8,0	8,0	-	12,9	12,5	-	-
103500		2,5	2,8	-	12,9	12,5	-	-
30000		2,7	2,7	-	6,2	6,5	-	-
0		2,2	2,2	2,2	5,9	6,5	4,2	5,3

Distribution of wave heights

During each sea state a certain distribution of wave heights occurs. When the distribution is known, the function of wave heights is known and all the characteristic wave heights can be determined. A very important generalization in this case is whether a deep-water situation or a shallow-water situation is considered.

Deep-water wave height distribution

In deep water waves behave approximately linear. This linear behaviour allows a statistical description of the wave characteristics, based on a Gaussian process of water surface elevation, resulting in a Rayleigh distribution of the individual wave heights that is fully determined by local wave energy.

Shallow-water wave height distribution

In shallow water, the wave height distribution is affected by non-linear effects and by wave breaking. Therefore the wave behaviour is more complicated and the knowledge of the statistical description of wave field characteristics is more limited. In the situation considered here obviously shallow-water is present and therefore the Rayleigh distribution is not valid anymore. This means that the distribution of wave heights is changing due to depth induced wave breaking. The highest waves break to a certain wave height which is based on the local water depth and which results in a larger amount of highest waves. The distribution is therefore not following a Rayleigh anymore, see Figure 2-11.

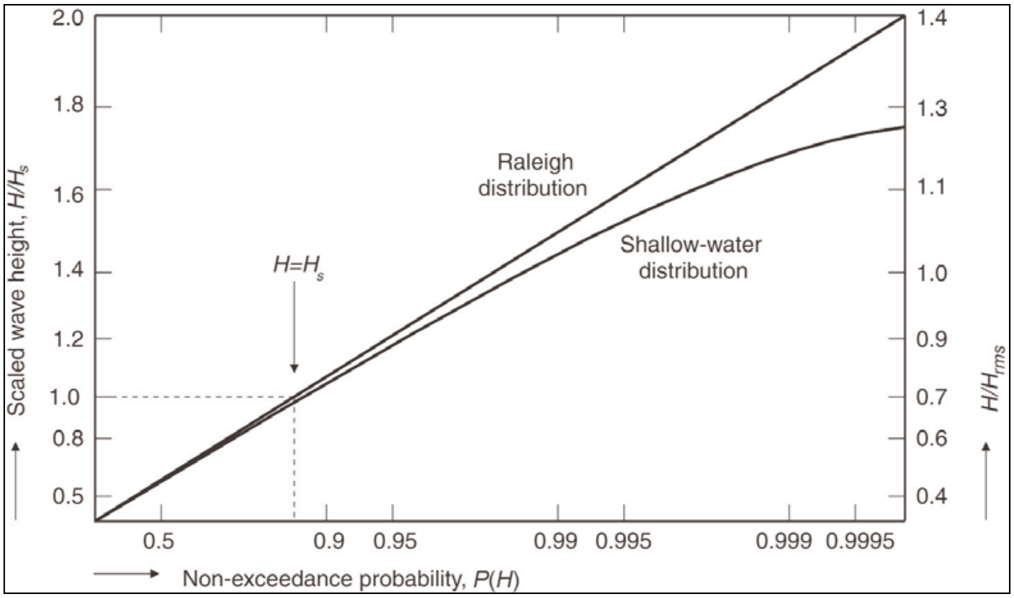


Figure 2-11: Example of a shallow-water observed distribution of wave heights compared to the Rayleigh distribution [Rock manual, chapter 4]

This change in wave height distribution has an important effect on the design. A design wave height for hard structures ($H_{2\%}$) will be somewhat smaller, but also occur significantly more. This has to be taken into account in the design.

Composite Weibull distribution

In shallow-water the wave height distribution differs significantly from the Rayleigh distribution as shown in Figure 2-11. To determine the wave characteristic another wave height distribution describing the proper distribution has to be determined. A wave height distribution that describes the shallow-water wave height closely is the composite Weibull distribution (CWD) [Battjes and Groenendijk, 2000]. This distribution is successfully tested and can be used for engineering practices and also storm conditions, Figure 2-12.

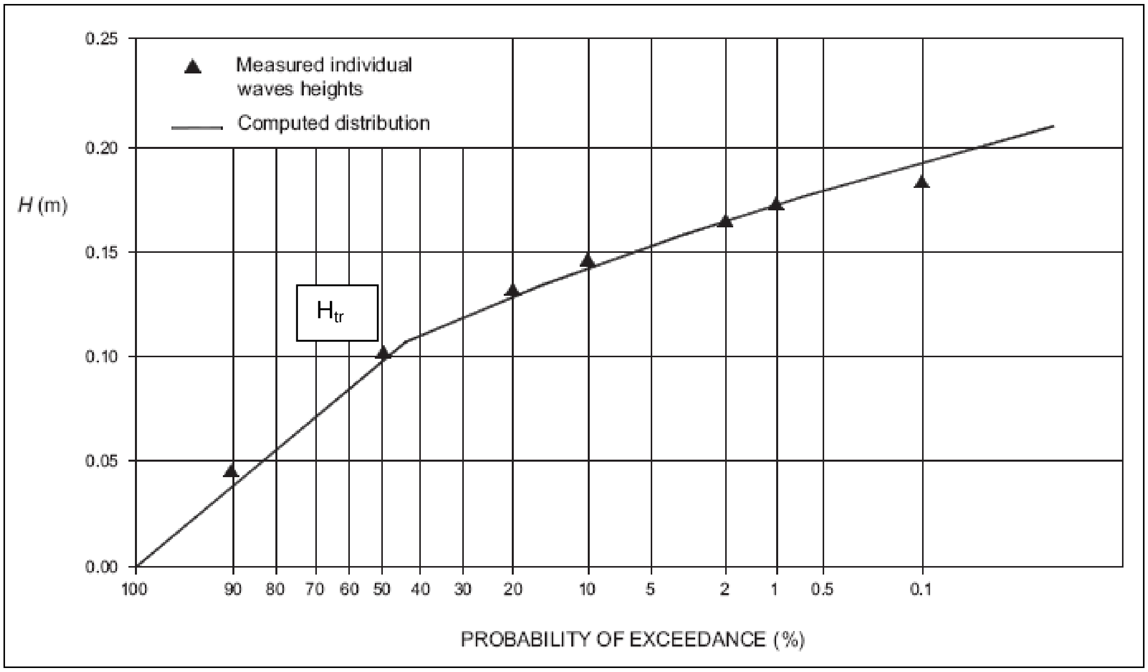


Figure 2-12: Comparison with the measured wave height and calculated wave height by the CWD wave height distribution [Battjes and Groenendijk, 2000]

The CWD is based on two distributions to describe the wave breaking correctly, see Equation 2-1.

$$P(H) = P(\underline{H} < H) = \begin{cases} 1 - \exp(-(H/H_1)^2) & \mathit{for}\ H < H_{tr} \\ 1 - \exp(-(H/H_2)^{3,6}) & \mathit{for}\ H \geq H_{tr} \end{cases} \qquad \text{Equation 2-1}$$

$$H_{tr} = H_{transition}$$

The largest waves break first, while there is no change for the smallest waves. This gives a non-homogeneous dataset of waves: broken waves and non-broken waves. In the CWD this physical effect is reproduced as follows: a Rayleigh distribution for the lowest part of the distribution (as in deep water) and a Weibull distribution for the upper part.

Determination of the characteristic wave heights

To determine the characteristic wave heights the transitional wave height has to be computed with Equation 2-2. In this equation the bed slope α and the local water depth h are the parameters.

$$H_{tr} = (0{,}35 + 5{,}8\tan\alpha)h \qquad \text{Equation 2-2}$$

Besides this transitional wave height the method also requires the knowledge of the root mean square wave height (H_{rms}). In general this wave height is not available and the variance (m_0) or the spectral significant wave height (H_{m0}) is known (from the application of a spectral wave propagation model for instance). So Equation 2-3 has been proposed for H_{m0}.

$$H_{rms} = \left[0{,}6725 + 0{,}2025(H_{m0}/h)\right]H_{mo} \qquad \text{Equation 2-3}$$

The next step is to compute the non-dimensional transitional wave height (H_{tr}/H_{rms}) which is used as input to a table provided by Battjes and Groenendijk (2000) to find the (non-dimensional) characteristic heights: $H_{1/3}/H_{rms}$, $H_{1/10}/H_{rms}$, $H_{2\%}/H_{rms}$, $H_{1\%}/H_{rms}$ and $H_{0,1\%}/H_{rms}$.

Finally the dimensional wave heights from the ratios read in the table and from the value of Hrms can be calculated.

In Table 2-9 the computed values for the characteristic wave heights are presented.

Table 2-9: Characteristic wave heights for 1/100year design condition

Distance from structure [m]	Wave heights	$H_{1/3}$ [m]	$H_{1/10}$ [m]	$H_{2\%}$ [m]	$H_{1\%}$ [m]	$H_{0,1\%}$ [m]
150.000		2,7	3,3	3,6	3,8	4,3
103.500		1,1	1,3	1,4	1,5	1,6
30.000		2,3	2,8	2,9	3,1	3,4
0		1,7	2,0	2,1	2,2	2,5

Interpretation of design conditions

In formulas that are used for the design of (hard) coastal structures frequently the peak wave period and the $H_{2\%}$ are used. The interpretation of these parameters can be important when design such structures as explained below.

Peak period

The spectral peak wave period can shift in magnitude along the profile. The spectrum along the profile has to be considered in order to know what happens with the peak period and to be sure that the design is based on just one peak period. It can occur that near the structure a double peaked spectrum is present and designing on just one peak period can be wrong. A somewhat smaller, but also important peak period can be present and should also be taken into account in the design.

Design wave height

In shallow water the distribution of wave heights change. The highest waves break and more waves with the same wave height are present. When basing a design on the $H_{2\%}$ it should be taken into account that this wave height can occur more often than assumed in the first place. The wave height is probably smaller, but the amount of waves with that wave height increase.

Governing wave parameters

As mentioned earlier the wave heights calculated by the USACE and calculated in SWAN do not differ much, however this is quite the case for the peak periods. The reason for this can be find in the different models used to calculate the parameters.

The USACE did not provide the distribution of the characteristic wave heights. Because this distribution is of importance for the research it is chosen to use the boundary conditions including this distribution determined in this chapter and not the boundary conditions imposed by the USACE.

The hydraulic boundary conditions are the surge level, the different wave heights and the different wave periods.
The surge level near the structure is 6m and is a result of a 1/100yr storm. This surge level is also the water level during the 1/100yr storm because during normal conditions no water is present.
The different wave heights and wave periods are showed in Table 2-10. The distribution of the wave heights is given in Table 2-11.

Table 2-10: Wave heights and wave periods

Distance from structure [m]	Wave height/ wave period	H_{m0} [m] Table	H_{m0} [m] Plot	H_{m0} [m] Spectral data	T_m [s] Plot	T_p [s] Plot	T_{m0} [s] Spectral data (m_0/m_2)	$T_{m-1,0}$ [s] Spectral data
150000		8,0	8,0	-	12,9	12,5	-	-
103500		2,5	2,8	-	12,9	12,5	-	-
30000		2,7	2,7	-	6,2	6,5	-	-
0		2,2	2,2	2,2	5,9	6,5	4,2	5,3

Table 2-11: Distribution of the wave height

Distance from structure [m]	Wave heights	$H_{1/3}$ [m]	$H_{1/10}$ [m]	$H_{2\%}$ [m]	$H_{1\%}$ [m]	$H_{0,1\%}$ [m]
150.000		2,7	3,3	3,6	3,8	4,3
103.500		1,1	1,3	1,4	1,5	1,6
30.000		2,3	2,8	2,9	3,1	3,4
0		1,7	2,0	2,1	2,2	2,5

Gravity surface waves

Now the design wave height and wave period are known, the wave- and water condition can be determined.

Wave condition:

1. Long wave condition: $L/h >> 1 \; or \;\; L >> 2\pi d$

2. Short wave condition: L/h is not much greater than 1

Water condition:

Shallow water: $kh << 1 \qquad h/L < \frac{1}{20}$

Deep water: $kh >> 1 \qquad h/L > \frac{1}{2}$

Intermediate water: $kh = O(1)$

Design conditions:

$$L \approx 35 \; for \; H_{m0} = 2{,}2m \;\; and \; T_{m-1,0} = 5{,}25s \;\; h = 6m$$

During design conditions the short wave condition and the intermediate water condition are valid.

2.3 Geotechnical boundary conditions

The geotechnical boundary conditions are a critical point with respect to the design of the reference structure. Since the accent in this research lays on the analysis of the superstructure and not on the analysis of the substructure the geotechnical boundary conditions do not play a role during further research. However, because of the importance with respect to the reference design some attention is paid to it.

The subsoil of the project area consists of different weak soil layers on top of a Pleistocene clay layer, see Figure 2-13. These weak soil layers are the result of years of deposition of soil particles during high water and flooding. The alignment of the barrier cuts through the western part of Lake Borgne which consists of wetlands. The first layer of the subsoil consists of organic material (mainly peat) followed by some layers of silt, sand and soft clay. These layers are fully saturated and provide very little bearing capacity. The Pleistocene clay layer, which has a significant stiffness compared to the top layers, starts at a depth of approximately 20m below sea level. Figure 2-13 shows a schematic presentation of the subsoil profile of Lake Borgne.

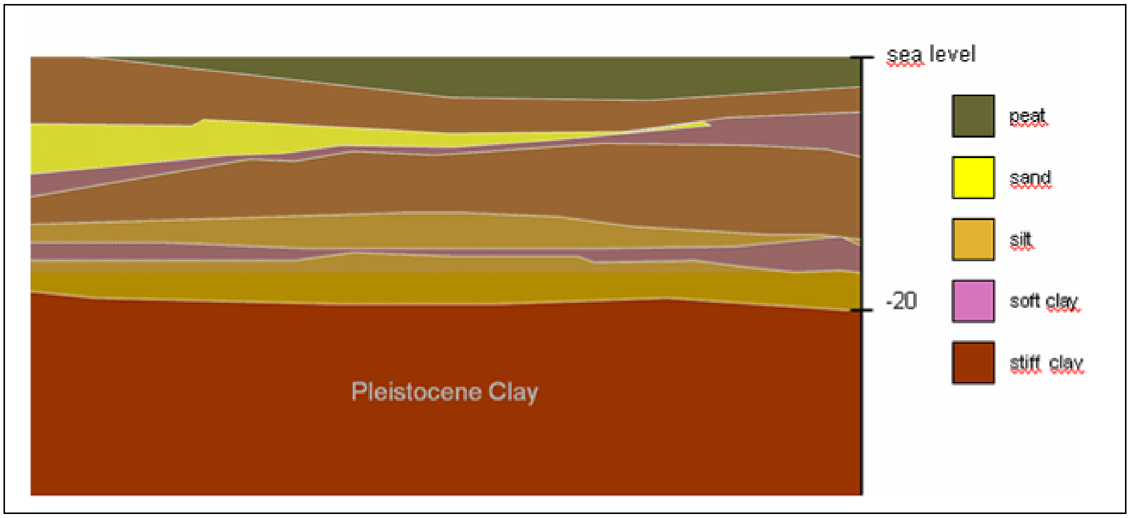


Figure 2-13: Schematized presentation of the subsoil profile along the alignment

Chapter 3 Reference design

In Chapter 2 ”Environmental boundary conditions” the environmental boundary conditions for the project area have been described. In this chapter the requirements of the United States Army Corps of Engineers are given. Then a concept solution is found taking the requirements, imposed by the USACE, not into account. When the best concept solution is found by means of a MCA the requirements of the USACE are analyzed critically. Finally the solution proposed by the consortium of Iv-Infra is described and also a critical reflection on the reference design is included in this chapter.
The critical review on the reference design has been carried out in cooperation with Ruud Nooij, the student who performs the research to get insight in the variable loads on the perforated and inclined wall.
In Chapter 4 ”Hydraulic processes reference design” the hydraulic processes in front of and inside the superstructure of the reference design, which is described in this chapter, are analyzed.

3.1 Introduction

After New Orleans was struck by hurricane Katrina on the 29th of August 2005, the United States Army Corps of Engineers (USACE) started to restore and expand the ‘Hurricane Protection System’ around the city. This flood protection consists of several levees and structures, forming a closed system.

At the east side of the city a new part of this flood protection system has to be designed. In a consortium Iv-Infra made a design for this project.

In Figure 3-1 the project area is presented. In Figure 3-2 this project area is enlarged and the junction of the Inner Harbor Navigation Channel (IHNC) with the Gulf Intracoastal Waterway (GIWW) and Mississippi River Gulf Outlet (MRGO) is visible. The GIWW is coming from the northeast, the MRGO is coming from the southeast and the IHNC is coming from the west. The dead-end channel coming from the north and connects to the GIWW is the Michoud Channel.

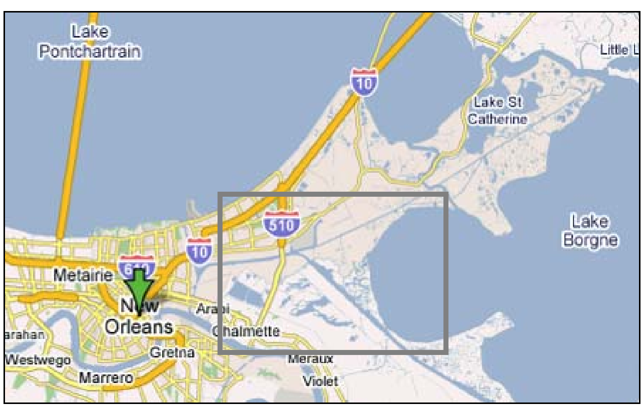


Figure 3-1: Project area.

Figure 3-2: Project area enlarged.

This design of Iv-Infra covered a flood protection in the Gulf Intracoastal Waterway (GIWW), a part of Lake Borgne and the Mississippi River Gulf Outlet.

3.2 Requirements from the USACE

In this paragraph the basic question and project objective posed by the USACE are described. Also the requirements from the USACE are given.

3.2.1 Basic question

A line of defence has to be designed and build that will provide protection from surges produced by a 100-year hurricane coming from the Gulf of Mexico through Lake Borgne. The area to be protected includes the areas along the IHNC and the surrounding parishes. Specifically, the area protected from the Lake Borgne surge must include everything west of the southeast corner of the Michoud Canal (indicated by location 1 in Figure 3-2) and everything northwest of the south point of the Bayou Bienvenue gate structure (indicated by location 2 in Figure 3-2).
A protection against storm surge from Lake Pontchartrain through Seabrook does not have to be included.

3.2.2 Project objective

The project objective is to provide a 100-year level of protection to the IHNC from hurricane induced storm surges by hurricane season 2011 and to have advance measures before June 2009.

3.2.3 Requirements from USACE

The USACE is the client in this project and therefore has some critical demands for the design of the flood protection. The most important design criteria are:

- Construction time
- Hydrostatic loading up to design level
- Level of protection

Construction time
The required level of protection is divided into two phases and the structure must meet a certain level of safety at the end of each phase. Therefore the construction phases are linked to the different levels of safety.

<u>Phase 1: Advance Measures</u>
The advance measures must provide a reduction in still water surge height in the IHNC at the junction with the GIWW of 4 feet for surges from Lake Borgne based on a 1/100-year storm water level at the junction of MRGO and GIWW by 1 June 2009.

Phase 2: 100-year level of protection
The standard is met when a design is made, which entails a hurricane protection system that provides the 100 year level of protection to the IHNC from storm surges coming from the Gulf through Lake Borgne by 1 June 2011. Besides this, a maximum rise of the water level of 1,5ft at the protected side is allowed.

It can be noted that the construction time of the project is an important design condition because of the short time span in which the protection levels have to be reached.

An extra limitation with respect to the time frame for the construction is the existence of the Jones Act. This Jones Act makes it impossible for foreign companies to work in the U.S.A. For example, foreign dredging companies are not allowed to work on U.S.A. projects and thereby the performance of large dredging operations is often difficult because the material and knowledge is not present.

Hydrostatic loading up to design level
A criterion posed on the design by the USACE is that the structure must be able to withstand a hydrostatic loading up to the design level.

Level of protection
The flood protection must meet the 100-year level of protection in June 2011. Therefore the flood protection has to be designed on the hydraulic conditions occurring during a 1/100-year storm.

3.3 Alignment determination

Alignment alternatives
For the determination of the flood protection alignment the USACE proposes four alternatives in the project area. The alternatives are presented in Figure 3-3. These alternatives are not fixed and different variants for the alignment is possible. However, we will only consider these alternatives, because these are the most conceptual.

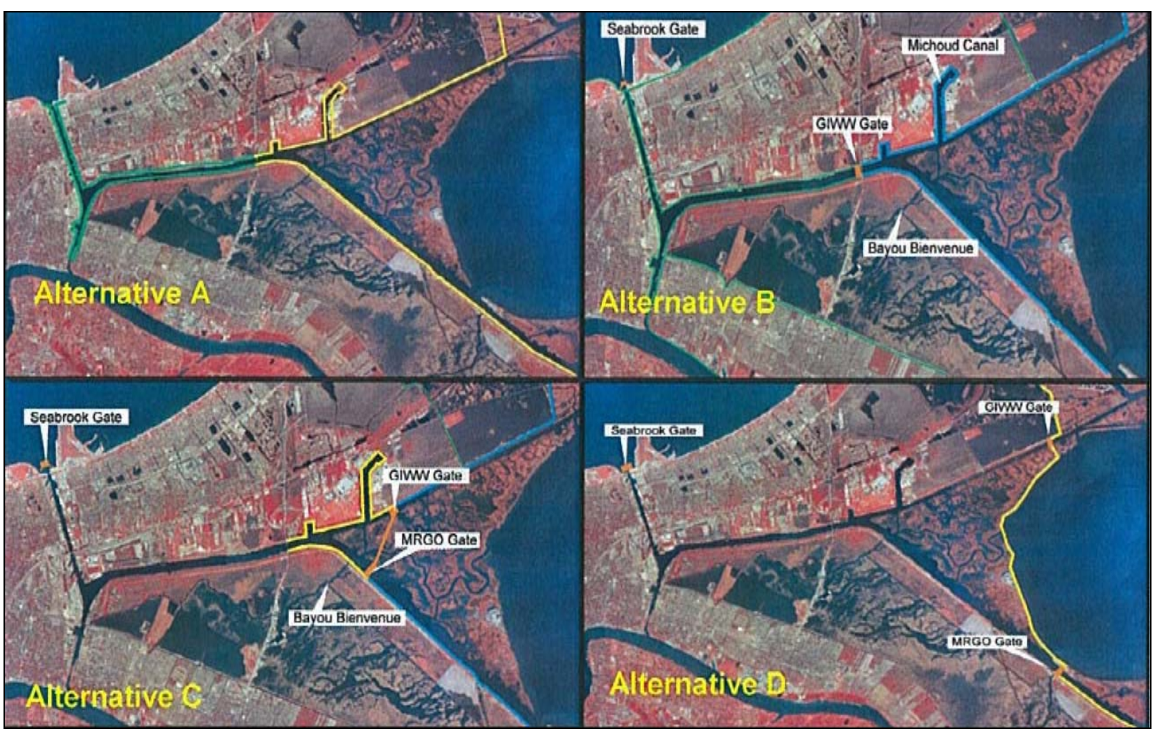


Figure 3-3: Alternatives for the alignment of the flood protection.

Alternative A: Raising floodwalls and levees along the channels and Lake Borgne.

In this alternative only existing floodwalls and levees are raised in order to fulfil the required level of protection. This results in raising a considerable length of floodwalls and levees including the floodwalls and levees along the IHNC Canal.

Alternative B: Raising floodwalls and levees along Lake Borgne and constructing a gate in the GIWW.

In this alternative a moveable gate is constructed in the IHNC Canal. The floodwalls and levees at the protected side of this gate do not have to be raised in this situation. However, the floodwalls and levees still have to be raised over a considerable length.

Alternative C: Raising floodwalls and levees along Lake Borgne and constructing a flood protection in Lake Borgne.

This alternative includes a flood protection starting from the east of the Michoud Canal to a point southeast of the Bayou Bienvenue. The floodwalls and levees in connection with the flood protection still have to be raised as can be seen in Figure 3-3.

Alternative D: Constructing a flood protection in Lake Borgne.

This alternative includes a flood protection situated more eastward compared with alternative C.
In this alternative the smallest amount of kilometres of floodwalls and levees have to be raised; however the protection in Lake Borgne is considerably long.

Determination of the best alternative
The determination of the best alignment should be carried out by following the elementary design cycle [De Ridder, 2002].

Analysis: In this step the criteria are determined which the alignment has to fulfil to.

The following points are carried out:
- Analysis of the important processes
- Analysis of the functions
- Analysis of the mutual relations

Important criteria for alignment determination:

- Length of alignment as short as possible.
- Heightening of levees and floodwalls as less as possible.
- Navigation possibilities.
- No or less funnel effect.
- Overtopping water storage area as large as possible.
- Ecological impact as small as possible.
- No intersections with pipelines, gas lines etc.

Synthesis: In this step the possible alignments are generated.

Possible alignments are generated which fulfil the criteria or most of the criteria determined in the analysis step.

Simulation: In this step the behaviour of the alignments is determined based on a cost-benefit analysis.

Evaluation: In this step the order of the alignments is determined.

Decision: In this step the value-cost relation of the best alignment is tested whether it fulfils the expectation or not.

When the chosen alignment fulfils the expectations it becomes a plausible solution. If not the case a loop back to the synthesis or analysis is necessary.

In finding the best concept solution no full alignment determination is performed. It is assumed that the optimal alignment is alternative C. Alternative D is not plausible because of the long protection in Lake Borgne. Alternatives A and B drop out because of the length of the floodwalls and levees which has to be raised.

Proposed alignment

The proposal of the consortium covered a permanent closure of the MRGO and the Lake Borge part (approximately 2km long), and a moveable closure in the GIWW. The alignment of this proposal is presented in Figure 3-4 and this is a variant of alternative C.

The main criterion for the design of a movable closure in the GIWW was that ship traffic must remain possible. The closure in the GIWW therefore has two main functions, retaining water and allowing passage for ships. For the permanent closure through Lake Borgne and the MRGO a protection with only a water retaining function had to be designed.

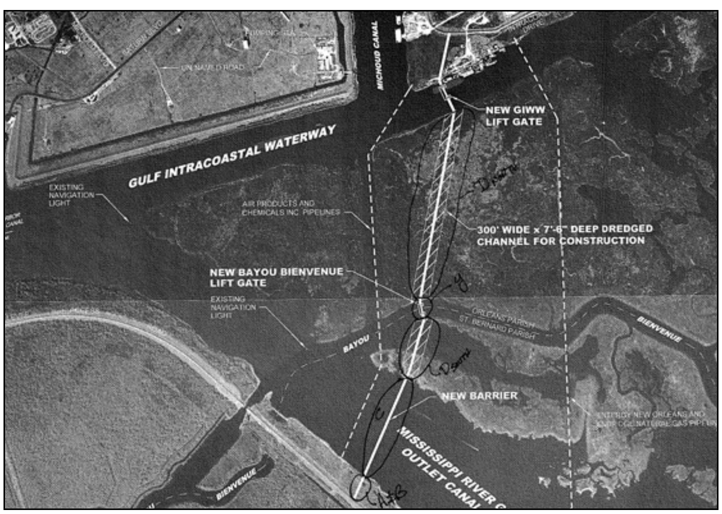


Figure 3-4: Alignment of the flood protection

3.4 Concept solutions and alternatives

Based on the environmental boundary conditions presented in Chapter 2 "Environmental boundary conditions" concept solutions and alternatives have to be determined and assessed. A lot of concept solutions and alternatives can be generated which fulfil the main function of the barrier, retaining water. The solutions can be divided in the following categories:

- **Gravity structures**

  In general a gravity structure retains water by means of gravity or dead weight of the structure. The dead weight results in a compression force and a friction force, acting on the foundation surface. Therefore this type of structure requires soil layers with sufficient bearing capacity direct under the structure.
  Soil improvement of the top layer, by means of dredging or excavation, possibly followed by filling the trench by material with good bearing characteristics is therefore a requisite for this type of structures.

  Within gravity structures the following alternative solutions can be distinguished:
    - Embankment levee
    - Caissons
    - Cellular cofferdam

- **Pile supported structures**

  These structures are supported by a pile foundation which gains it bearing capacity from a deeper soil layer. Since the bearing capacity is obtained by deeper and stronger soil layers it is not necessary to remove the weak top layers. The costs of the foundation depend on the required dimensions and number of foundation piles and the level of the stronger soil layers.

  Within pile supported structures the following alternative solutions can be distinguished
    - Tee wall
    - Delta wall

- **Composite structures**

  Composite structures are a combination of the above mentioned conceptual solutions. When the combined surcharge loads and water pressures being too large for a wall structure, a heavy structure on pile foundations can be a solution.

  Within pile supported structures the following alternative solutions can be distinguished
    - Piled levee
    - Double combi wall

- ***'Weightless' structures***

  This concept is based on the reduction of the surcharge or dead weight of the structure. Structures with a small dead weight can serve as a good solution for constructing on soft subsoil.

  Within pile supported structures the following alternative solutions can be distinguished
    - Inflatable dam
    - Structure with horizontal transfer of loads

In Appendix 1 the concepts are described and some pro's and con's are listed. Then a critical reflection is given for the situation in which the concept is applied. Within the critical reflection of the alternative solutions the boundary conditions are used.

As described in Chapter 2"Environmental boundary conditions" the geotechnical conditions are complex and can therefore be a determining factor for the design of the flood protection. When constructing on soft subsoil, several methods to overcome the problem of soft subsoil can be applied. For example:

- Soil improvement:
    - Applying horizontal or vertical drains to speed up the settlement process
    - Removal of soft layers and filling with material with a good bearing capacity
- Foundation to soil layers with sufficient bearing capacity
- Reducing the dead weight of the structure

3.5 Multi Criteria Analysis

To make a decision which alternative is the best, a Multi Criteria Analysis (MCA) is performed to compare the alternatives. The MCA is given in Appendix 2.

Evaluation

The best alternative based on the performed multi criteria analysis is the embankment levee. This alternative has gained the highest score. After this alternative the most optimal alternatives are respectively the caisson, the delta wall and the piled levee.

However, these alternatives only fulfil the environmental boundary conditions. The USACE imposes also some requirements. These requirements have been described in paragraph 3.2.
Because of the Jones Act soil excavation and dredging are difficult to perform. A result of the requirement imposed by the USACE and the Jones Act is that the embankment levee is not a feasible solution.
Soil excavation and dredging are not possible because of the Jones Act and the short time span of the project. Therefore the gravity structures, such as caissons, drop out as possible solutions.

In paragraph 3.6 a critical review is performed on the requirements imposed by the USACE. In paragraph 3.7 the design of the consortium of Iv-Infra is described. This design is not the best solution taking into account only the boundary conditions. However, the consortium should also take into account the requirements imposed by the USACE and the concept design is forced to adjust. The taken steps to adjust the design are also described in paragraph 3.7.

3.6 Comments on requirements from USACE

In this paragraph a reflection on the requirements imposed by USACE is given. The requirements from the USACE are given in paragraph 3.2 and are based on the following aspects:

- Constructional limitations
- Hydraulic conditions
- Design level

Constructional limitations

A dike or a levee would be the most logic solution in the Netherlands. However, due to national legislation and due to the short time span of the project excavation or dredging of the different weak layers is not possible.

The Jones Act makes it impossible for foreign companies to work in the states. Foreign dredging companies which have the expertise to remove the weak layers cannot perform the work because of this act. American companies don't have the knowledge and the equipment for performing this kind of work.

Besides the Jones Act, the short time span of the project poses difficulties on the removal of the weak soil layers with the determined time span. The first phase of the project with the advance measures has to be finished in June 2009. Removal of soil over a considerable depth (20m) and a considerable width over the entire length (2km) and replacing it by soil with good bearing capacities poses problems with respect to the time frame.

Hydraulic conditions

In paragraph 2.5 the water level set-up (surge) and the occurring waves due to 1/100year storm event are given. The maximum surge during this 1/100year design condition is 6m and the significant wave height amounts 2,4m.

Hydrostatic loading up to design level

A criterion posed on the design by the USACE is that the structure must be able to withstand a hydrostatic loading up to the design level.

A result of this criterion is that the design height of the structure should be kept as low as possible, because a higher structure results in larger hydrostatic pressures and thereby larger horizontal forces.

Another very important aspect is that the storm surge has an effect on the loading of the subsoil and the seepage cutoff screen. The storm surge acts as a surcharge on the subsoil and because of the negligible bearing capacity the subsoil deforms and provides large forces on the seepage cutoff screen.

This hydrostatic loading criterion is in line with the criterion of resiliency. However, in the Netherlands it is not usual to account for hydrostatic loading up to design level, because the design water levels and associated frequencies represent considerably higher levels of safety. Most structures have a structural design level in which an additional height (over height) is incorporated and no hydrostatic loads are considered on this part of the structure.

Design level of safety in New Orleans

The design conditions for the flood protection system around New Orleans are based on a 1/100year level of safety. This level of safety is based on a political choice and is applied throughout the whole state of Louisiana. Besides this level of safety a resiliency condition is imposed that demands a minimum amount of damage of the structure under 1/500year conditions.

Considering the value of New Orleans this level of safety is not an economic optimum.

New Orleans versus the Netherlands

When the level of safety of New Orleans is compared to the levels of safety applied in the Netherlands it can be noted that the level of safety is an order 10 lower. In the Netherlands the dike ring areas have a frequency of exceeding the hydraulic load of 1/1.250 - 1/10.000year. These levels of safety are determined by the Delta Committee in the Delta Plan after a catastrophic flooding in 1953. In New Orleans a similar situation has occurred, but no higher level of safety is chosen for the design of new flood protection structures.

In the “Dutch perspective on coastal Louisiana flood risk reduction and landscape stabilization [Netherlands water partnership Delft, 2007]” a comparison between the New Orleans situation and the Dutch situation is made on several aspects. One of the results of this study was an optimal level of protection for the New Orleans bowl, the central part of the city, based on the economic optimization method as proposed by the Delta Committee [van Dantzig, 1956]. In this method the economical damage, the investment costs (for levee strengthening) and different safety levels are considered. The optimal level of safety for the New Orleans bowl was established at 1/100.000years.

This level of safety is not totally representative for New Orleans. The main reason for this is that the method is based on the largest flood prone area of the Netherlands, South Holland, and the economic importance of this area is not directly comparable to the economic importance of New Orleans for America. However, it gives a good impression on the magnitude of difference between the levels of safety and thereby the perspective of both governments on flood protection.

Design conditions
The design height of the barrier is based on a 1/100year condition, but a criterion posed by the USACE states that the barrier must be able to withstand a 1/500year condition with minimal damage. When the design of a flood protection system is considered it can be noted that this is a rather strange demand. A flood protection system has a main function to prevent flooding and thereby reduce the risk of flooding to an economical optimum considered costs and damage.
When a 1/500year storm occurs a significant larger surge will be present and the hinterland will be flooded. The flood protection in this case will not fail during the storm but will neither provide a protection of the hinterland, because every condition higher than 1/100year will result in a certain amount of flooding.

The choice of a resiliency of the strength of the structure to a higher event than the design event is in principle a good concept. It is not desirable to have a flood protection fails when an event with more extreme conditions with respect to the design conditions occurs. Besides this, after a possible flood of the hinterland under more extreme conditions the structure must be able to retain water from events within the order of the design condition and therefore only little damage is desirable.
In this specific case a demand on the design is also that the structure can be adjusted to a possible higher level of safety.

New Orleans in the future
When the flood protection system finally is completed on the design criteria posed by the USACE the safety of the inhabitants of New Orleans will partly be restored. A sound consideration of the safety of the living conditions with respect to flooding due to storms has to be made. The people living in the Netherlands do not have much of an alternative of leaving the flood plane when the available space in the country is considered, the people living in New Orleans however have a lot more space available which is less vulnerable to flooding.
Perhaps the people evacuated after the flooding should not return to their homes in New Orleans, because the level of safety is not high enough and the change that a similar flood would occur within a relative short time span is too large.

The main question (and decision to be made) is if New Orleans should be protected and to which costs.

3.7 Solution proposed by Iv-Infra

3.7.1 Proposed design

The solution proposed by Iv-Infra is a battered pile supported structure. The superstructure consists of two walls, a vertical and an inclined wall. This design is based on the concept of a Tee wall with battered foundation and a seepage screen. The seepage screen is an extension of the Tee wall and reaches into the Pleistocene clay layer to guarantee water tightness and avoid seepage under the structure.
Because of the weak subsoil the deformations of the seepage screen and of the battered piles under extreme conditions will be considerable. Also the Tee wall itself will deform under the loads during extreme conditions and it will deflect in the direction of Lake Borgne. The battered piles will fail due to too large deflections.
Because of the expected deformations and probable failure of the piles the structure is widened by including a concrete plate. This plate is placed on top of the seepage screen and battered piles will be

hammered through sleeves in the plate. The loads acting on the seepage screen are now partly transferred through the concrete plate to the battered piles.
Since there is enough space on top of the concrete plate it is decided to construct an inclined wall in front of the vertical wall. Both walls have a different function. The inclined wall will dissipate wave energy of the incident waves under storm conditions. The vertical wall has a water retaining function and will prevent inundation of the hinterland.
The configuration of these two walls will finally reduce the total height of the structure, because overtopping waves over the inclined wall are allowed and the water can be drained back through gaps in the inclined wall. This has a positive influence on the hydrostatic pressure at design level. The proposed solution is presented in Figure 3-5.

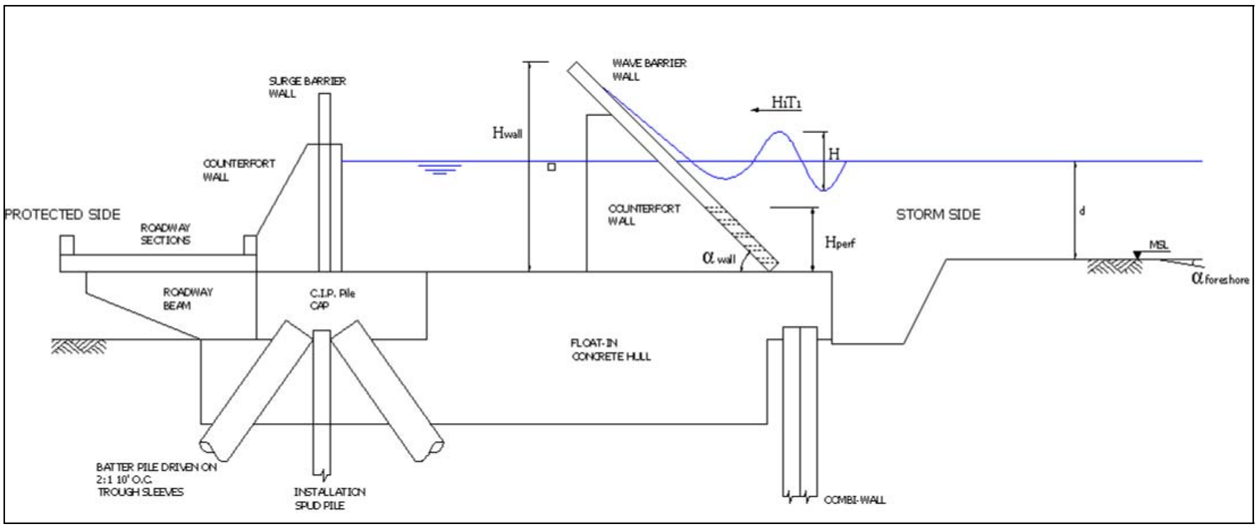


Figure 3-5: Cross section superstructure of the proposed solution by consortium of Iv-Infra

The volume enclosed by the floor, the vertical and the inclined wall will be called basin in the rest of the report. Behind the vertical wall at the protected side a roadway is present as one can see from Figure 3-5. However this roadway is not of importance for this research and is therefore not discussed any further.
In Figure 3-5 the perforation is schematized as a real perforation over a certain height of the inclined wall. The perforation is implemented in the down part of the inclined wall and extends from the bottom to a certain height. First the perforation is schematized as a rectangular gap over the full perforation height. So actually one can then not speak about perforation. The openings/gaps are separated in the length direction of the barrier by counter fort walls.

The structure can be built in the following steps. First a trench has to be dredged before the combi wall will be constructed. The combi wall is constructed from a pontoon where after a floated concrete hull will be attached water tight to the combi wall. From this base a battered pile foundation will be constructed, by hammering the piles through sleeves in the base. When the foundation is completed a vertical wall is constructed over the whole length, which will assure the first required level of safety. After this an inclined concrete wall will be constructed on the base of the structure in front of the vertical wall.

The structure is rather complex, but it is adjusted to the prevailing conditions and the posed requirements. Besides this the barrier can be constructed completely by using prefabricated elements.

3.7.2 Points of attention

Geotechnical conditions

As mentioned in Chapter 2 "Environmental boundary conditions", the subsoil of the project area consists of different weak layers on top of a Pleistocene clay layer. The alignment of the barrier passes the western side of Lake Borgne and at this side of the lake wetlands are present. The first layer of the subsoil consists of organic material followed by some layers of silt, sand and clay. The Pleistocene clay layer starts at a depth of approximately 20m below sea level. Figure 3-6 shows a schematic presentation of the subsoil profile of Lake Borgne.

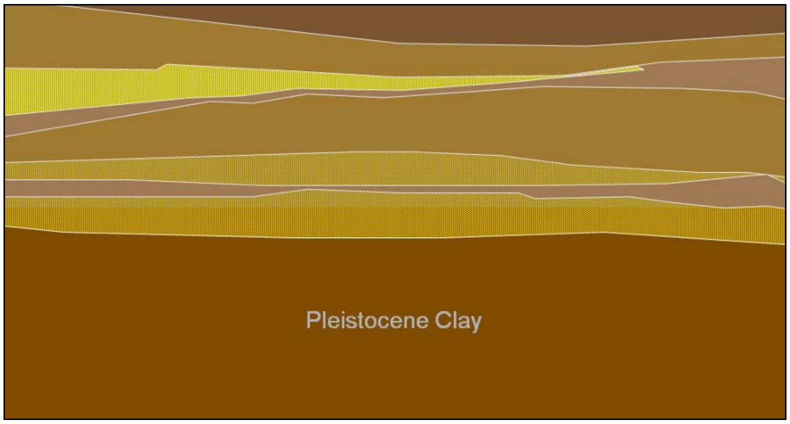


Figure 3-6: Schematized presentation of the subsoil profile along the alignment.

In the Netherlands the most logic solution for this geotechnical problem is applying soil improvement by excavating or dredging the weak soil layers and fill the trench with a material with good bearing properties, like sand.
In this situation the first 20m should be excavated, because the Pleistocene clay layer starts at a depth of approximately 20m below sea level. Excavation or dredging a 20m thick layer is a complex work and knowledge and good equipment is a requisite.
When the sand body is placed an expensive pile supported structure is not necessary any more. A dike or a levee can easily be constructed on top of the improved sand body.

<u>Pile supported structure</u>

Because of the short time span of the project and the Jones Act excavation and dredging are not possible to perform. This is the reason of designing a pile supported structure making it unnecessary to remove the weak subsoil. The piles of the foundation reach the Pleistocene clay layer. It has been confirmed by calculations that the bearing capacity of the Pleistocene clay layer is sufficient.

Different types of soil layers are present in the subsoil, all having a different permeability. Therefore seepage under the structure is a problem and a seepage cutoff screen is necessary. The seepage cutoff screen needs to reach the Pleistocene clay layer to prevent seepage through pervious layers. This results in a seepage cutoff screen with a length of approximately 20m over the entire length of the barrier. When a levee structure on an improved sand body is constructed a seepage screen would not have been necessary because of the larger footprint and thereby an increased seepage distance. So the costs of this screen are additional costs for the pile supported structure. However the placing of the screen can easily be done within the given project time.

Calculations proved that the Pleistocene clay layer provides enough bearing capacity for the occurring loads. Although the calculations are only performed for the shaft resistance there is sufficient cone resistance to prevent horizontal displacements of the cone of the piles. Because of the fact that only calculations are done for the shaft resistance the calculations can be considered conservative.

Hydraulic conditions

Design level of the structure

The design level of the flood protection system depends on the design water level and the amount of wave overtopping allowed. If no wave overtopping is allowed the design height of the flood protection will be larger. This is especially the case when sloping structures are considered. Due to the run-up of waves a higher design level is necessary to reduce the wave overtopping.
To avoid a very high design level a separation of functions in the design is made as described earlier and hereby wave overtopping over the sloping wall is allowed. This means that the vertical wall only has to retain the water up to the surge level and that the total height of the structure is reduced to a minimum. Considering the criterion of loading up to design level this has a favourable effect.

Width of the structure

The separation of functions results in a larger width of the structure, which is also required for the battered pile foundation. This larger width also results in an increasing amount of required concrete for the structure which a negative influence on the costs.
The larger width of the structure also has an effect on the stability of the structure because the surcharge of the surge water reduces the overturning moment. A negative additional effect of this surcharge load is the increasing load on the foundation.

3.8 Conclusion

From the critical reflection of the alternative concept solutions it appears that for the circumstances present in New Orleans, thick layers of weak subsoil, the gravity structures and the composite structures are not feasible. The main cause for this is the insufficient bearing capacity of the weak soil layers and the deep level of the Pleistocene clay layer. Soil improvement is therefore a requisite, but due to the limited construction time and the Jones Act soil improvement is not feasible.

Therefore pile supported structures and 'weightless' structures remain as solutions in this situation.
It can be concluded from the critical review that an inflatable dam is not a very suitable solution; based on the performed MCA it was the least attractive solution. However, relative light weight structures can be a good solution for flood protections on weak soil, but research on the uncertainties is still necessary. Pile supported structures seem feasible, but the mentioned superstructures are not optimal with respect to the present conditions and requirements.

In a consortium Iv-Infra finally designed a pile supported structure based on the concept of a Tee wall. This design is formed by taking into account all the design conditions and criteria. Although long foundation piles are required for stability and the structure is quite complex it seems a feasible solution.
The soft soil only needs to be removed over a small depth to allow the construction of the proper foundation from pontoons. Besides this, prefabricated elements are used and the time schedule is met.

Final conclusion

Constructing dikes or levees for permanent flood protection works is from a technical point of view still the best solution. This is confirmed by looking at flood protection systems all over the world. Even in situations with weak subsoil these solutions in combination with proper soil improvement is the best option.

However, in the New Orleans situation described here, pile supported structures are the best solution. The main reason for this is that soil improvement is not possible and the construction time is very limited.

The design of Iv-Infra is a possible solution to the problem and satisfies the conditions and requirements. This design is taken as the basis for the master thesis.

Chapter 4 Hydraulic processes

4.1 Introduction

In order to determine the optimal configuration of the superstructure it is crucial to understand all the hydraulic processes taking place in and in front of the superstructure. These hydraulic processes depend on the hydraulic boundary conditions described in Chapter 2 'Environmental boundary conditions'; on the configuration of the superstructure and its implementation in the surroundings, described in Chapter 3 'Reference design'.
This chapter describes the hydraulic processes acting on this type of sea defences and the analysis of the hydraulic processes is performed for simplifications of the reference design. After this analytical analysis physical model tests are performed to verify the analytical results and to overcome uncertainties. The physical model testing is described in Chapter 5 'Physical model testing'. In Chapter 6 'Comparison analytical results and model testing results' the results of the analytical and physical research are compared and combined in order to can construct a numerical model, which can calculate the overtopping volumes over the vertical backwall. The numerical model is described in Chapter 7 'A numerical model'.

4.2 Hydraulic processes acting on this type of sea defenses

The reference design is the design of the storm surge barrier proposed by Iv-Infra, described in detail in chapter 3. As mentioned earlier the structure consists of a vertical and an inclined wall. The area enclosed by these two walls is a kind of basin which can be emptied by outflow through perforation in the inclined wall. This type of configuration entails different hydraulic processes for certain boundary conditions. In this paragraph the hydraulic processes are briefly described occurring at this type of sea defences.
During a certain storm, a surge is present and the structure is exposed to waves. Some waves will only give a run-up on the structure, besides an inflow in the basin through the perforation of the inclined wall. Other waves cause a higher run-up than the crest height and will overtop the perforated inclined wall.

The water inflow as a result of the overtopping and of the inflow through the perforation results in a water level rise in the basin. The water level rise causes a water head difference between the level in the basin and the outside water level (unprotected side). Due to this water level difference an outflow through the perforation will be induced.

Five main processes can be distinguished:

1. Water movement in the basin of the superstructure.

2. Wave run-up and wave overtopping over the perforated inclined wall.

3. Emptying of the basin i.e. outflow through the perforation induced by head difference.

4. Wave induced inflow through the perforation of the inclined wall.

5. Length spreading effect; outflow in the longitudinal direction of the basin.

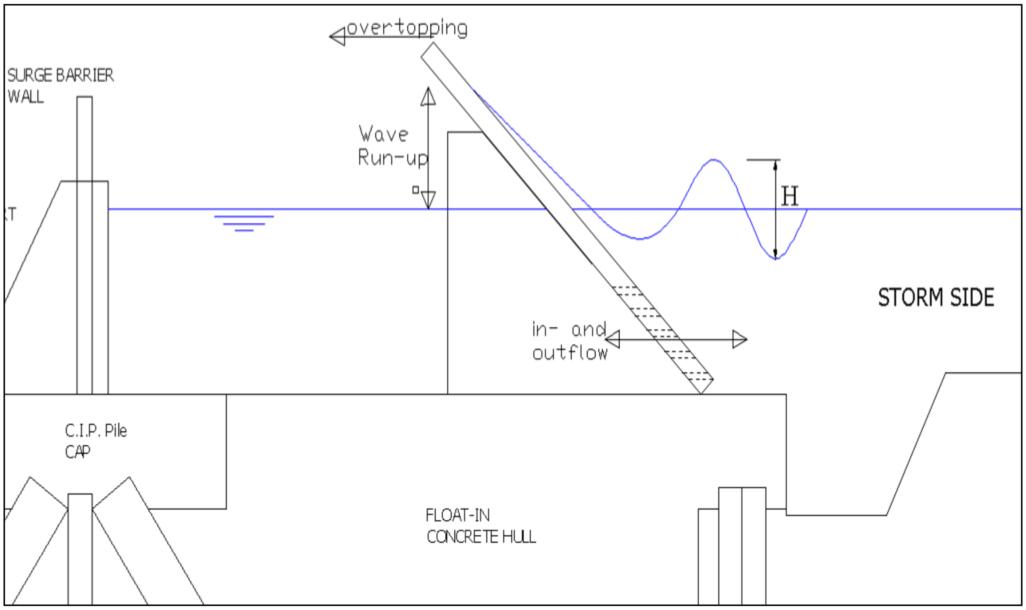


Figure 4-1: processes occurring in and in front of the structure

In this paragraph the different processes will be described briefly and the uncertainties of these processes are given. Because of the uncertainties some simplifications of the structure are made to get a good understanding of the different processes. The hydraulic processes are described separate from each other because the emphasis lies on the understanding of the different processes.

Water movement in the basin of the superstructure

The movement of the water in the basin is important to understand with respect to the emptying process, but also for the foundation of the structure. Large water level fluctuations in the basin result in a higher water retaining backwall. The objective is to keep the vertical backwall as low as possible to avoid large horizontal forces on the foundation and therefore the water movement has to be controlled. The water movement for a perforated rectangular basin is descibed in paragraph 4.4.1. The water movement for a perforated inclined wall in front of a vertical backwall is described in paragraph 4.4.2.

Wave run-up and wave overtopping over an inclined wall

The inclined wall has the function of dissipating energy out of the incoming waves. The approaching waves will run-up the perforated inclined wall and when this run-up is higher than the crest height they will overtop the inclined wall. The wave run-up and wave overtopping over a non-perforated inclined wall are described in respectively paragraph 4.5.1 and 4.5.2. The wave run-up and wave overtopping over a perforated inclined wall are described in paragraph 4.5.3.

Emptying of the basin i.e. outflow through the perforation induced by head difference

The inflow through the perforation and the overtopping volumes cause a water level rise in the basin. This water level rise causes a head difference between the water level inside and outside the structure, which induces an outflow through the perforation in the inclined wall. The outflow, on its part, is influenced by the inflow. The only way to empty the basin is by outflow through the perforation and therefore understanding this process is very important, since the emptying of the basin prevents the water level in the basin rising too high resulting in overtopping the vertical wall. The emptying process is described in paragraph 4.6.

Wave induced inflow through the perforation of the inclined wall

The orbital motion of the waves exerts a horizontal velocity of the water particles under the waves. This velocity is pointed half a wave period in the direction of the structure and half a wave period in opposite direction. This results in an inflow through the perforation in the inclined wall during half a wave period and during half a wave period in an outflow. The head difference between the inside and outside water level induces an outflow through the perforation and this influences the inflow of a new incoming wave. Thus, the inflow not only depends on the incoming waves, but also on the head difference between the inside and outside water level. The wave induced inflow is described in paragraph 4.7. In paragraph 4.8 the total flow through the gap at the bottom of the inclined wall is described, including the emptying process and the wave induced inflow.

Length spreading effect; outflow in the longitudinal direction of the basin.

When short crested waves overtop the inclined wall the occurring length spreading effect increases the basin capacity. The water level rise with a length of the crest of the incoming waves can be discharged through the perforation over a length of the crest, but it also can flow in longitudinal direction of the basin and be discharged through the perforation over there. This effect is called length spreading effect and increases the basin capacity. The length spreading effect is described in paragraph 4.9.

4.3 Basic model

In the following paragraphs the different hydraulic processes are described. For these different processes simplifications have been made in order to understand the processes. When the process for the simplification is understood, the process for the reference design can be determined more easily. To describe the water movement in the basin; the wave run-up and wave overtopping over a perforated inclined wall; the wave induced inflow and the total flow through the gap a common model has been used, which is described in this paragraph.

The frame work of the model consists of a perforated vertical wall in front of a vertical backwall, see Figure 4-2.

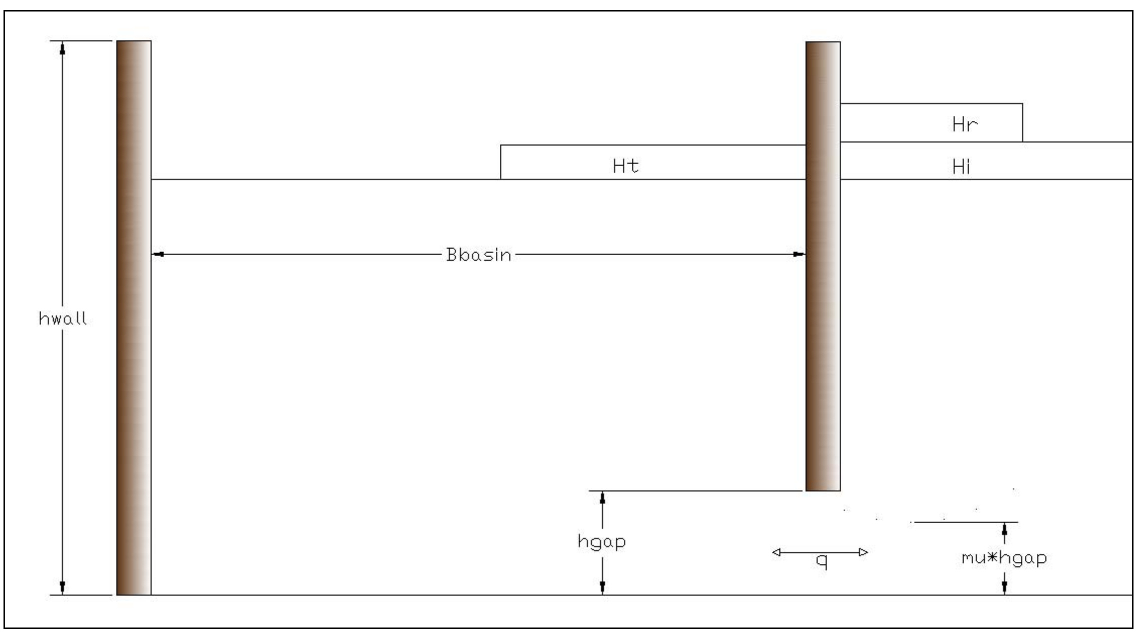


Figure 4-2: Frame work basic model; perforated vertical wall in front of vertical backwall

Schematization

The structure given in the framework is exposed by short waves, which has been determined in paragraph 2.2. However, the Long Wave Theory is used to schematize the incoming, reflected and transmitted waves. Because the Long Wave Theory is used the wave heights can be seen as a water level rise and therefore this schematization results in a simple mathematical description.

The Long Wave Theory makes it possible to use a discharge relation and some equations following from the preservation of volume for determining the flow through the gap and the reflected and transmitted wave height.

Schematization improvement

In the schematization the long wave celerity is used and the pressure difference in the gap is equal to the head difference over the gap, because of the Long Wave Theory. One can improve this schematization by using the short wave celerity and by taking the pressure difference in the gap for short waves into account. In that situation the pressure difference in the gap is not any more equal to the head difference, but it is equal to the head difference times a factor (cosh).

The schematization improvement is not used, since it is assumed that the first schematization is suitable for this situation.

In the basic model the flow q [m³/s/m] through the gap and the different wave height amplitudes are calculated by means of a volume balance (preservation of volume) and a discharge relation over the gap. As already mentioned most of the processes are described by the basic model and therefore the coming calculations in the next paragraphs are based on the following volume balance and discharge relation.

The volume balance implies that discharge left of the perforated vertical wall must be equal to the discharge right of this wall.

Volume balance:	$q = q_{wall;l} = q_{wall;r}$	
With		
q :	discharge under the wall	[m^3/s/m]
$q_{wall;l}$:	discharge based on conditions left of the wall	[m^3/s/m]
$q_{wall;r}$:	discharge based on conditions right of the wall	[m^3/s/m]

The discharge relation is based on Torricelli.

Discharge relation:	$q = \mu a \sqrt{2g(\partial h_{wall;l} - \partial h_{wall;r})}$	
With		
q :	discharge under the wall	[m^3/s/m]
μ:	contraction coefficient	[-]
a:	gap height	[m]
g:	gravitational accelleration	[m/s^2]
$\partial h_{wall;l}$:	water level left of wall	[m]
$\partial h_{wall;r}$:	water level right of the wall	[m]

$$|q|q = (\mu a)^2 2g(\partial h_{wall;l} - \partial h_{wall;r})$$

The discharge relation is for means of simplicity linearized:

$$\frac{8}{3\pi}\hat{q}q = (\mu a)^2 2g(\partial h_{wall;l} - \partial h_{wall;r})$$

However, in this discharge relation the inertia of the gap is neglected. But the assumption that the inertia can be neglected has first to be verified. The relative importance of the friction and the inertia term of the discharge relation of Equation 4-1.

$$\left(\frac{8}{3\pi\left(\mu a\right)^2 2g}\hat{q} + \frac{i\omega L_{gap}}{ga}\right)\tilde{q} = \Delta\tilde{h} \qquad \text{Equation 4-1}$$

Where

ω:	angular velocity	[rad/s]
L_{gap}:	influence length of the inertia and is approximately the gap height	[m]

$$\text{Relative importance: } \frac{\text{friction term}}{\text{inertia term}} = \frac{\left(\frac{8}{3\pi(\mu a)^2 2g}\hat{q}\right)}{\left(\frac{i\omega L_{gap}}{ga}\right)} = \frac{2\hat{q}T}{iL_{gap}3\pi^2\mu^2 a} \qquad \text{Equation 4-2}$$

$$\text{When } \begin{array}{l} \dfrac{\text{friction term}}{\text{inertia term}} \gg 1 \ \text{: inertia can be neglected} \\ \dfrac{\text{friction term}}{\text{inertia term}} \ll 1 \ \text{: friction can be neglected} \end{array} \qquad \text{Equation 4-3}$$

In Figure 4-3 the relative importance is depicted against the gap height for different influence lengths.

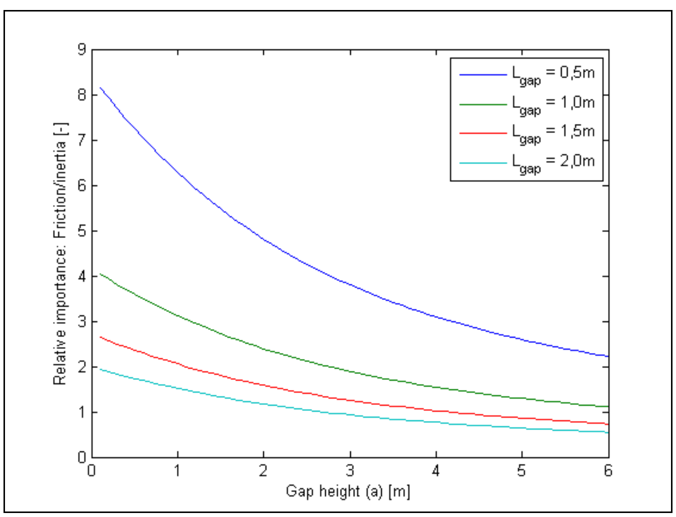


Figure 4-3: Relative importance of friction and inertia term against gap height for H_{m0}=2,2m; h=6m and $T_{m-1,0}$=5,25s

From Figure 4-3 it can be concluded that for a gap height smaller than 1m the inertia term can be neglected because the relative importance is >>1 and that means the friction is significant more of influence than the inertia. It is chosen that for a gap height smaller than 1,5m the inertia term is neglected.

From now on only gap heights smaller than 1,5m are used and therefore the inertia of the gap can be neglected.

4.4 Water movement in the basin

Because of the weak sub soil large horizontal pressures acting on the foundation have to be avoided. Therefore the reference design has to be as low as possible and this means that water fluctuations against the water retaining backwall have to be controlled and avoided as much as possible.
The water movement inside the basin is excited by the overtopped water plunging in the basin and excited by the inflow through the gap at the bottom of the inclined wall. The excitation by the plunging of the overtopped water is neglected, because it is assumed that the contribution to the water movement is insignificant compared to the contribution of the excitation by the inflow through the gap.
It has to be analyzed whether a travelling wave or a standing wave is occurring in the basin (probably both can occur, but resonance has to be avoided and therefore the eigen frequencies of the basin have to be checked in relation to standing waves). Due to transmission through the gap at the bottom of the wall a wave travels in the basin. However, when the basin is small compared to the wave length a standing wave can occur. This means that at the same place two waves travelling in opposite direction are present.
In this paragraph an analysis is performed to check whether a water movement is occurring in the basin or not and what kind of water movement is occurring (a travelling wave or a standing wave). When a standing wave is occurring in the basin resonance of the water level has to be avoided at all times and therefore the eigen frequencies of the basin have to be checked.

Type of occurring wave in the basin

Travelling periodic wave
A travelling periodic wave can be described by Equation 4-4. A travelling periodic wave has a fixed shape whereby the maxima, minima and zero points are travelling with a constant velocity. An observer travelling with a velocity of $dx/dt=c=\omega/k$ (for a wave travelling in positive direction) does not see a change in the phase ($kx-\omega t$) neither in ζ_+ or Q_+ (vertical water displacement respectively discharge). This leads to $c=\omega/k=L/T$.
From Equation 4-4 it can be seen that ζ and Q are in phase. Both parameters are proportional to a cosine with the same phase angle. This means that the maximum of ζ and Q are occurring at the same places.

$$\begin{aligned}
\zeta_+ &= \hat{\zeta}\cos(kx-\omega t)\\
\zeta_- &= \hat{\zeta}\cos(kx+\omega t)\\
Q_+ &= Bc\hat{\zeta}\cos(kx-\omega t)\\
Q_- &= -Bc\hat{\zeta}\cos(kx+\omega t)
\end{aligned}$$

Equation 4-4

Standing periodic wave
When two waves with the same amplitude are travelling in opposite direction and encounter each other and are in phase at $x=0$ a standing periodic wave is the result. Equation 4-5 describes a standing

periodic wave, which is a superposition of two periodic waves travelling in opposite direction. The shape of a standing periodic wave does not change in place, but only fluctuates in time.
In a standing periodic wave there are points where the amplitude of the wave height is always zero. These points are called nodes. There are also points in a standing periodic wave where the amplitude of the wave height is maximum. These points are called antinodes.
The parameters ζ and Q are for x and for t an angle π/2 out of phase.

$$\zeta = \zeta_+ + \zeta_- = 2\hat{\zeta}\cos kx \cos \omega t = \hat{\zeta}_{st}\cos kx \cos \omega t$$
$$Q = 2Bc\hat{\zeta}\sin kx \sin \omega t = Bc\hat{\zeta}_{st}\sin kx \sin \omega t$$

Equation 4-5

Governing situation
As mentioned earlier a travelling periodic wave as well as a standing can occur in the basin. However the governing situation is resonance of the water level fluctuation occurring by a situation with a standing wave. This situation has to be avoided and therefore it has to be checked what kind of standing periodic waves can occur for this type of configurations.

Resonance

Definition: resonance is the tendency of a system to oscillate at maximum amplitude at certain frequencies, known as the system's resonance frequencies (or resonant frequencies).

When the frequency of the excitation is equal to one of the eigen frequencies of the basin resonance is the result. The excitation on the basin is the incident wave at the perforated inclined wall.
The eigen frequencies of the basin have to be determined and it has to be checked whether these frequencies are more or less equal to the wave frequency or not. In the simplifications the eigen frequencies are determined and compared with the wave frequency.

Wave frequency:

$$\omega = ck = \frac{2\pi}{T_{m-1,0}} = 1{,}2 rad/s$$

Equation 4-6

Determined with the following parameters:

h:	6m
H_{m0}:	2,2m
$T_{m-1,0}$:	5,25s
$L(T_{m-1,0})$:	35m
c= $L(T_{m-1,0})/ T_{m-1,0}$:	6,6m/s

The basin of the reference design is enclosed by a concrete bottom plate, a vertical wall and a perforated inclined wall. For this configuration it is difficult to determine the eigen frequency ω. So first the eigen frequencies are determined for a simplification of the reference design. The simplification does have a rectangular configuration. In the simplification a standing wave is occurring in the basin. The water movement for this simplification is described in paragraph 4.4.1. In paragraph 4.4.2 the water movement for the reference design is described.

4.4.1 Perforated rectangular basin

This simplification consists of a perforated vertical wall in front of a vertical backwall, see Figure 4-4. Inside the basin a standing wave is present, which is excited by the flow through the gap at the bottom of the vertical wall.

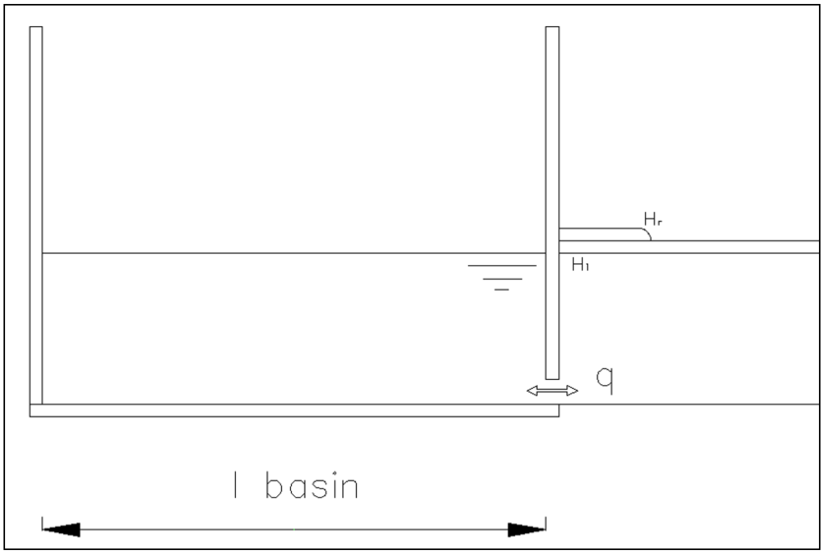


Figure 4-4: Simplification; perforated rectangular basin with a standing wave inside

In the volume balance the flow left of the perforated vertical wall is determined by the standing wave. The wave height amplitude in the discharge relation has to be multiplied by cos(kl) due to the standing wave. This term represents the inertia of the basin as well as the sin(kl) in the flow of the standing wave which is given in Equation 4-7 $\left.\begin{aligned}\tilde{q}_{gate;l} &= -ci\delta\tilde{h}_b\sin(kl)\\ \tilde{q}_{gate;r} &= c(\delta\tilde{h}_r-\delta\tilde{h}_i)\end{aligned}\right\}-ci\delta\tilde{h}_b\sin(kl)=c(\delta\tilde{h}_r-\delta\tilde{h}_i)$

Equation 4-7

.

In paragraph 4.3 it was said the inertia of the gap can be neglected for configurations with a gap height smaller than 1,5m. However, in this simplification the inertia of the gap is not neglected in order to investigate the resonance, because resonance is only possible when the inertia is taken into account. So the inertia of the basin as well the inertia of the gap is taken into account in this simplification.

Volume balance:

$$\left.\begin{aligned}\tilde{q}_{gate;l} &= -ci\delta\tilde{h}_b\sin(kl)\\ \tilde{q}_{gate;r} &= c(\delta\tilde{h}_r-\delta\tilde{h}_i)\end{aligned}\right\}-ci\delta\tilde{h}_b\sin(kl)=c(\delta\tilde{h}_r-\delta\tilde{h}_i) \qquad \text{Equation 4-7}$$

From the volume balance $\partial\tilde{h}_b$ can be derived:

$$\delta\tilde{h}_b=\frac{1}{i\sin(kl)}(\delta\tilde{h}_i-\delta\tilde{h}_r)$$

Discharge relation, based on Equation 4-1:

$$\alpha=\frac{8}{3\pi(\mu a)^2 2g}\hat{q}+\frac{i\omega L_{gap}}{ga}$$

$$\left(\frac{8}{3\pi(\mu a)^2 2g}\hat{q}+\frac{i\omega L_{\text{gap}}}{ga}\right)\tilde{q}=(\partial\tilde{h}_b\cos(kl)-\partial\tilde{h}_i-\partial\tilde{h}_r)$$

$$\left(\frac{8}{3\pi(\mu a)^2 2g}\hat{q}+\frac{i\omega L_{gap}}{ga}\right)c(\delta\tilde{h}_r-\delta\tilde{h}_i)=(-\frac{\cos(kl)}{i\sin(kl)}-1)\delta\tilde{h}_r+(\frac{\cos(kl)}{i\sin(kl)}-1)\delta\tilde{h}_i$$

Factors:

$$\alpha=\frac{8}{3\pi(\mu a)^2 2g}\hat{q}+\frac{i\omega L_{\mathrm{gap}}}{ga}$$

$$\tilde{\gamma}=\frac{\cos(kl)}{i\sin(kl)}$$

$$\tilde{C}_r=\frac{\delta\tilde{h}_r}{\delta\tilde{h}_i}=\frac{\alpha c+\tilde{\gamma}-1}{\alpha c+\tilde{\gamma}+1} \qquad \text{Equation 4-8}$$

δh_r, the δh_b and the $\tilde{q}$ as function of ∂h_i :

$$\delta\tilde{h}_r=\tilde{C}_r\delta\tilde{h}_i=\frac{\alpha c+\tilde{\gamma}-1}{\alpha c+\tilde{\gamma}+1}\delta\tilde{h}_i$$

$$\delta\tilde{h}_b=\frac{1}{i\sin(kl)}(\delta\tilde{h}_i-\delta\tilde{h}_r)=\frac{1}{i\sin(kl)}(1-\tilde{C}_r)\delta\tilde{h}_i \qquad \text{Equation 4-9}$$

$$\tilde{q}=-ci\delta\tilde{h}_b\sin(kl)=c(\delta\tilde{h}_r-\delta\tilde{h}_i)=-c(1-\tilde{C}_r)\delta\tilde{h}_i$$

The resonance calculation has been performed for a wave length of 35m, which is the implied wave length following from the hydraulic boundary conditions.

The wave height amplitudes are now determined and the basin movement can be plotted, which is done in Figure 4-5-Figure 4-7.

In Figure 4-5 the water movement is depicted in relation to the basin length for four different influence lengths of the inertia of the gap.
In Figure 4-6 the water movement is depicted in relation to the multiplication of k_0 and the basin length for four different influence lengths of the inertia of the gap.
In Figure 4-7 the water movement is depicted in relation to the ratio of the basin length and the wave length for four different influence lengths of the inertia of the gap.

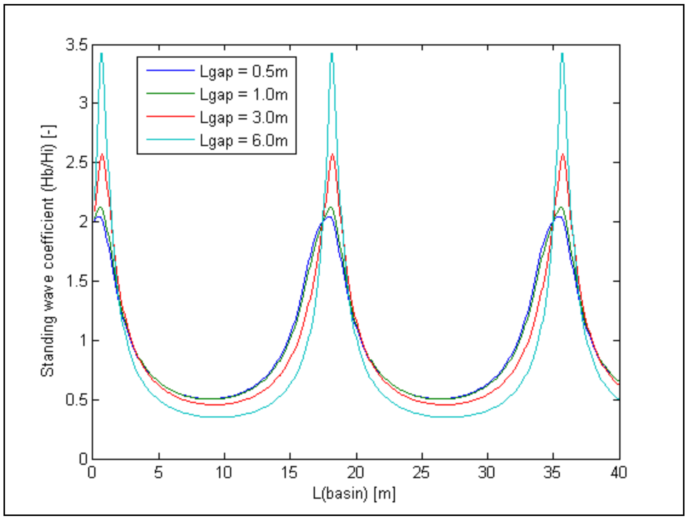


Figure 4-5: Water movement in relation to the basin width, for H_{m0}=2,2m; h=6m; $T_{m-1,0}$=5,25s and h_{gap}=1m.

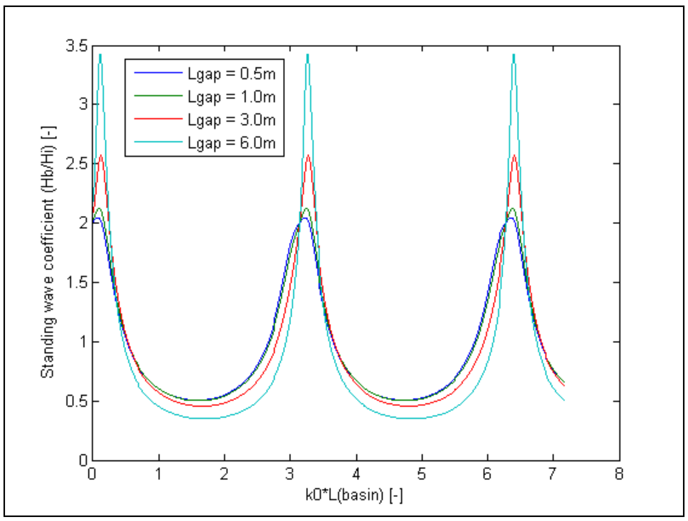


Figure 4-6: Water movement in relation to the multiplication of k_0 and the basin width, for H_{m0}=2,2m; h=6m; $T_{m-1,0}$=5,25s and h_{gap}=1m.

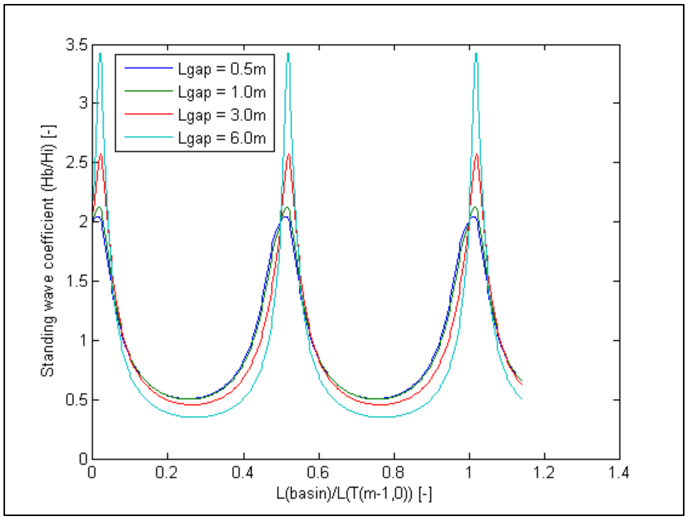


Figure 4-7: Water movement in relation to the ratio of the basin width and the wave length, for H_{m0}=2,2m; h=6m; $T_{m-1,0}$=5,25s and h_{gap}=1m.

From the three figures presented above it can be concluded that resonance is occurring for more than one frequency. From Figure 4-5-Figure 4-7 a basin width can be chosen such that no resonance is occurring. When one chooses a basin width the peaks in the figures have to be avoided because resonance is occurring at those places.

From Figure 4-7 one can see that a basin width of half times the wave length (L=17,5m) or a multiple of one wave length results in resonance.

Intermediate conclusions

Resonance of the water movement is occurring inside a rectangular basin with a gap at the bottom of the vertical wall for a basin width of half or a multiple of half a wave length. The resonance has been determined by using the Long Wave Theory.
The resonance is excited by the incoming waves through the gap at the bottom of the inclined wall. In order to can determine whether resonance is occurring or not the inertia terms of both the basin and the gap have to be taken into account.

4.4.2 Perforated inclined wall in front of vertical backwall

For the perforated rectangular basin resonance was investigated for different basin widths. The result of that analysis is that a basin width can be chosen such that no resonance will occur for the given boundary conditions. In this paragraph the water movement for the reference design is analyzed.

The configuration of the reference design, Figure 4-9, only differs from the perforated rectangular basin described in paragraph 4.4.1 (see Figure 4-8) by a perforated inclined wall instead of a perforated vertical wall. In paragraph 4.4.1 it was also stated that influence of the plunging of the overtopping waves on the water movement in the basin is insignificant compared to the influence of the excitation through the gap by the incoming waves. Therefore the contribution of the overtopping waves to the water movement was neglected.

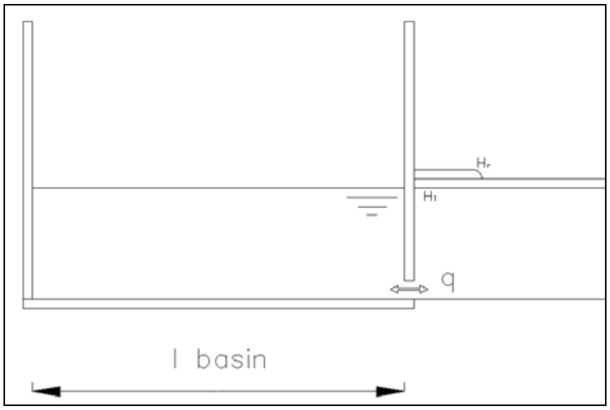


Figure 4-8: Simplification; perforated rectangular basin with a standing wave inside

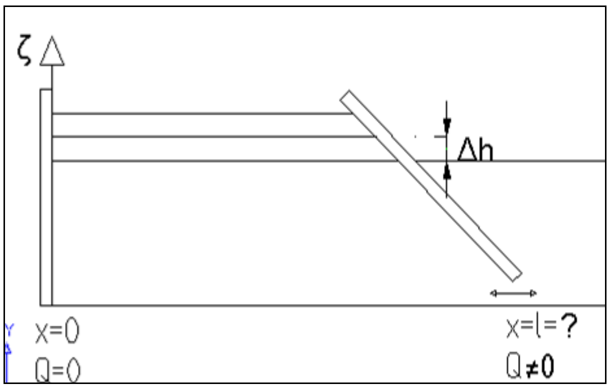


Figure 4-9: Reference design; basin behind perforated inclined wall with standing wave inside

Differences between the perforated rectangular basin of paragraph 4.4.1 and the reference design.
As already mentioned in paragraph 4.4.1 the different wave height amplitudes and the discharge through the gap can be calculated by using the preservation of volume and the discharge relation. Assumptions have to be made when the preservation of volume and the discharge relation are used for the reference design:

- Overtopping causes a water level rise which is not taken into account in the discharge relation.
- It is assumed that the standing wave is not significant influenced by the inclined wall and therefore the discharge for the standing wave $\tilde{q}_{gate;l}$ remains the same as in the case with a vertical wall.
- The inflow $\tilde{q}_{gate;r}$ is for the reference design subject to a phase difference between the incoming and reflected wave. However the influence of this phase difference on the water movement is also assumed to be insignificant and is therefore neglected.
- A standing wave is possible due to the inertia of the basin. This inertia is expressed in the discharge of the standing wave by the sin(kl) term and in the discharge relation by the cos(kl) term. These two inertia terms include the basin width. In the simplifications with a vertical wall the basin width is constant for different water levels inside the basin. However for an inclined wall this basin width is changing. So besides changing the distance between the vertical and inclined wall the basin width is also so a function of the height, because of the inclined wall. The function of the height on the basin width is neglected and only the distance between the vertical and the inclined wall is taken into account in the basin width l. The basin width is the distance between these two walls at water surface.

The volume balance and the discharge relation, which have been used for the rectangular basin described in paragraph 4.4.1 can also be used for the reference design.

The results found in paragraph 4.4.1 for the perforated rectangular basin; perforated rectangular basin with a standing wave inside, are also valid for the reference design when one makes the above assumptions.

Intermediate conclusions
When one makes the assumptions made earlier in this paragraph the water movement in the basin of the reference design does not differ from the water movement in the perforated rectangular basin. It is assumed that the inclined wall does not affect the water movement. Therefore resonance is occurring in the basin when a basin width of 0,5L or a multiple of 0,5L is applied. In order to avoid large water level fluctuations against the vertical backwall these basin widths must not be applied.

4.5 Wave run-up and wave overtopping

As explained in paragraph 4.2 "Hydraulic processes acting on this type of sea defences" the wave run-up and overtopping process are first described for a non-perforated inclined wall. This non-perforated inclined wall has a smooth slope with a certain angle and a certain height. In paragraph 4.5.1 and 4.5.2 it is analysed what the influences are of these two parameters on the run-up and overtopping over the inclined wall.
The simplification of the superstructure for the analysis of the run-up and overtopping process makes it possible to use the EurOtop Manual; "Wave Overtopping of Sea Defences and Related Structures" for the run-up and overtopping calculations. In paragraph 4.5.3 the wave run-up and wave overtopping over a perforated inclined wall are described.

4.5.1 Wave run-up over a non-perforated inclined wall

The wave run-up height is defined as the vertical difference between the highest point of wave run-up and the still water level (SWL). Due to the stochastic nature of the incoming waves, each wave will give a different run-up level. In the Netherlands as well as in Germany many dike heights have been designed to a wave run-up height $R_{u2\%}$.
This is the wave run-up height which is exceeded by 2% of the number of incoming waves at the toe of the structure. The idea behind this was that if only 2% of the waves reach the crest of a dike or embankment during design conditions, the crest and inner slope do not need specific protection measures other than clay with grass. It is for this reason that much research in the past has been focused on the 2%-wave run-up height.
In the past decade the design or safety assessment has been changed to allowable overtopping instead of wave run-up. Still a good prediction of wave run-up is valuable as it is the basic input for calculation of number of overtopping waves over a dike, which is required to calculate overtopping volumes, overtopping velocities and flow depths.

In the design of this thesis, however, it is the meaning that waves overtop the inclined wall regularly and therefore the overtopping is not a rare event for design conditions as in the case of the dikes in the Netherlands. Perhaps it is therefore not optimal to use the $R_{u2\%}$ for the wave run-up in this situation because more waves than only 2% of the incoming waves will reach the top of the structure. This point of attention will be discussed later on in this paragraph under overtopping.

In this chapter the run-up and overtopping over a non-perforated inclined wall are described and for the process description for non-perforated situation the $R_{u2\%}$ is used in order to understand the run-up and overtopping process. So to understand the process of run-up and overtopping the (non-perforated) inclined wall is considered as a dike with a smooth impermeable slope.

<u>Relative wave run-up for a sea dike or embankment seawall.</u>

The relative wave run-up height $R_{u,2\%}$ /H_{m0} is related to the surf similarity parameter $\xi_{m-1,0}$. The surf similarity parameter or Iribarren number $\xi_{m-1,0}$ relates the slope steepness tan(α) to the wave steepness $s_{m-1,0} = H_{m0}/L_0$ and is often used to distinguish different breaker types.

Figure 4-10 represents the wave run-up on a smooth impermeable slope.

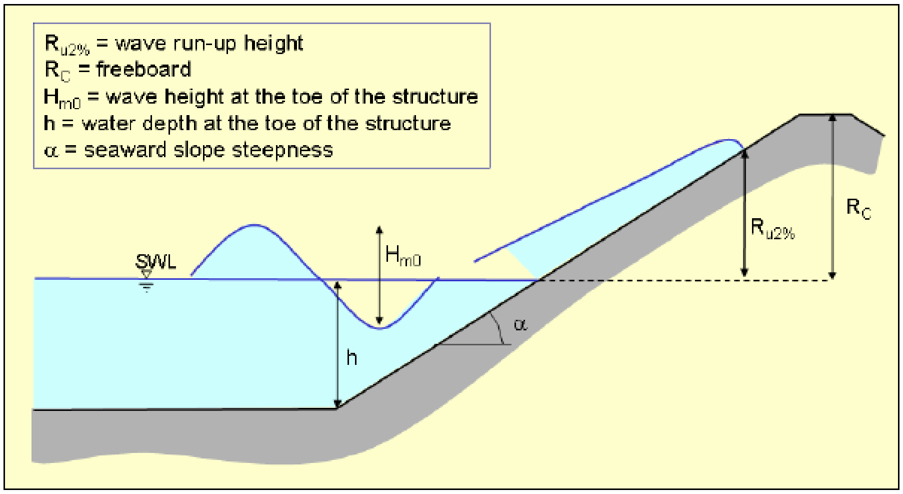


Figure 4-10: Wave run-up schematization. [EurOtop manual]

In the analytical research probabilistic formulas are used instead of deterministic formulas. Deterministic formulas include a safety margin and probabilistic formulas make use of an average and a standard deviation. In this research one is interested in the averages and therefore probabilistic formulas are used.

<u>Probabilistic formula for relative wave run-up height</u> ($0.5< \gamma_b\ \xi_{m-1,0} < 8$ to 10)

$$\frac{R_{u2\%}}{H_{m0}} = 1.65 * \gamma_b \gamma_f \gamma_\beta * \xi_{m-1,0} \text{ for breaking waves with a maximum of} \quad \text{Equation 4-10}$$

$$\frac{R_{u2\%}}{H_{m0}} = 1.0 * \gamma_f \gamma_\beta (4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}) \text{ for non-breaking waves} \quad \text{Equation 4-11}$$

$R_{u2\%}$:	wave run-up height exceeded by 2% of the incoming waves	[m]
H_{m0}:	spectral wave height	[m]
γ_b:	influence factor for a berm	[-]
γ_f:	influence factor for roughness elements on a slope	[-]
γ_β:	influence factor for oblique wave attack	[-]
$\xi_{m-1,0}$:	surf similarity parameter or Iribarren number	[-]
ξ_{tr}:	transition surf similarity parameter between breaking and non-breaking waves.	[-]

The relative wave run-up height increases linearly with increasing $\xi_{m-1,0}$ in the range of breaking waves and small surf similarity parameters less than ξ_{tr}. For non-breaking waves and thus a higher surf similarity parameter than ξ_{tr} the increase is less steep as shown in Figure 4-11 and becomes more or

less horizontal. The relative wave run-up height $R_{u,2\%} / H_{m0}$ is also influenced by: the geometry of the coastal structure; the effect of wind; and the properties of the incoming waves.
From Figure 4-11 it becomes clear that the relative wave run-up increases for an increasing Iribarren number.
However the formula for the wave run-up, which is depicted in Figure 4-11 is valid in the area
$0.5 < \gamma_b \xi_{m-1,0} < 8$ to 10.

The effect of wind on the wave run-up-height for smooth impermeable slopes will mainly be focused on the thin layer in the upper part of the run-up. Very thin layers of wave run-up are not considered and the run-up height is defined where the run-up layer becomes less than 1-2 cm. Wind will not have a lot of effect then.
It is recommended not to consider the influence of wind on wave run-up for coastal dikes or embankment seawalls.
During a hurricane wind velocities are extremely high and their effect on the wave run-up could have significant influence. However, it is stated that the wind only has influence on the thin top layer of the wave run-up and therefore it is also assumed that the effect of wind during a hurricane does not have significant influence on the wave run-up.

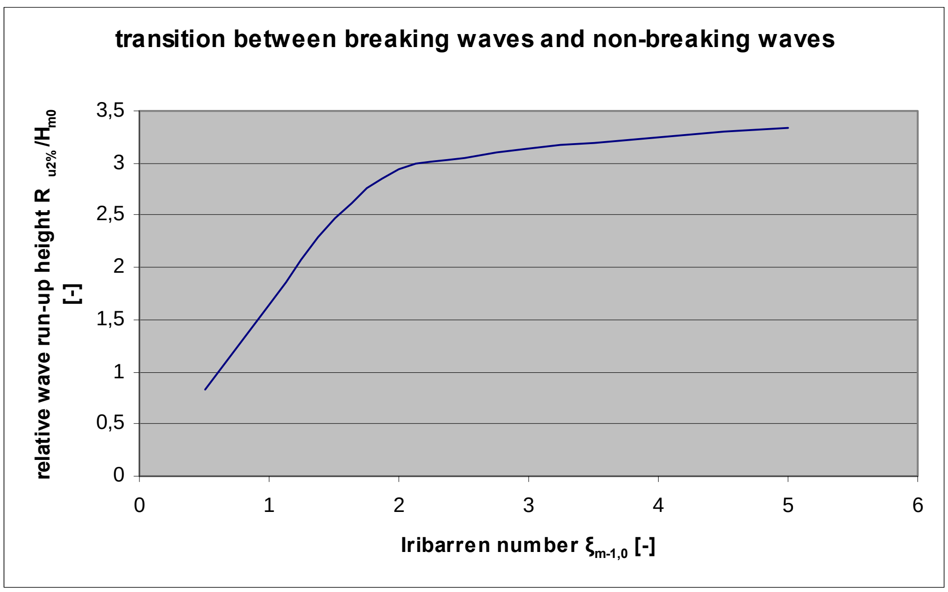


Figure 4-11: relative wave run-up as a function of the Iribarren number for smooth straight slopes

The combination of structure slope and wave steepness gives a certain type of wave breaking and this type of wave breaking can be translated in a parameter, namely the surf similarity parameter or Iribarren number:

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{\dfrac{H_{m0}}{\dfrac{gT^2{}_{m-1,0}}{2\pi}}}} \qquad \text{Equation 4-12}$$

$\xi_{m-1,0}$:	surf similarity parameter or Iribarren number	[-]
α:	slope of the structure	[^{0}]
H_{m0}:	spectral wave height	[m]
g:	gravitational acceleration	[m/s^2]

$T_{m-1,0}$: spectral wave period [s]

Figure 4-12 shows the different types of breaking with their accompanying surf similarity numbers.

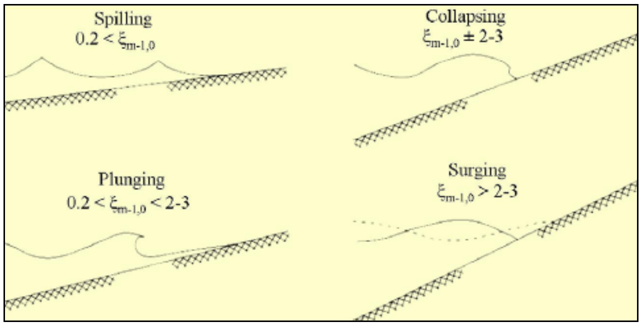


Figure 4-12: breaker types [EurOtop manual]

A steeper slope results in a higher wave run-up, because the distance to be travelled by a wave is shorter than for a configuration with a gentler slope. When the distance to be travelled becomes shorter also the energy dissipation by friction becomes smaller. Besides this, breaking of waves will occur when the slope becomes too gentle. From

Equation 4-12Equation 4-12 and from Figure 4-13 it is visible that a steeper slope results in a larger surf similarity parameter resulting in a higher wave run-up height.
A longer wave contains more water and more energy resulting in a higher wave run-up than for a shorter wave with the same wave height. This can also be concluded from Equation 4-12; a larger wave period results in a larger surf similarity parameter and thus in a higher wave run-up height.

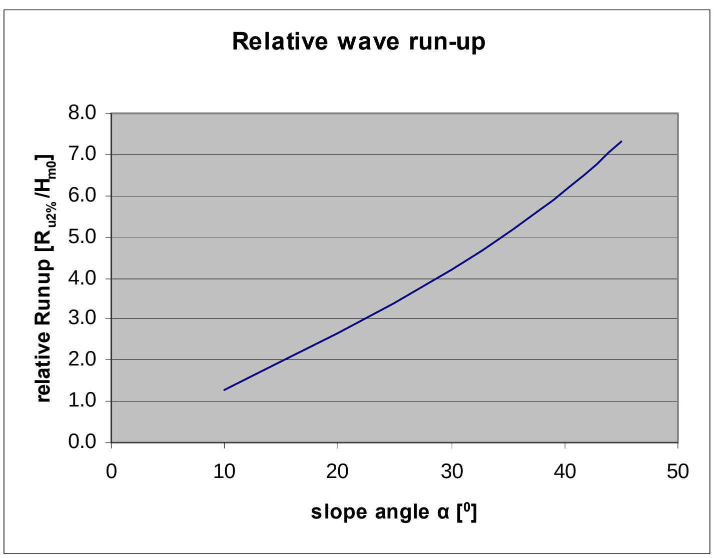


Figure 4-13: Relative wave run-up height as a function of the slope angle for H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m

Intermediate conclusions

The wave run-up is a function of the wave height and the Iribarren number. An increase in the slope angle results in an increase in the wave run-up. However this is valid in the area 0.5< γ_b $\xi_{m-1,0}$ < 8 to 10. The wave run-up also increases when a longer wave or a higher wave is considered. Respectively the wave run-up decreases for a shorter and a lower wave when the non-breaking area is considered.

4.5.2 Wave overtopping over a non-perforated inclined wall

When the wave run-up height becomes larger than the crest height of the structure wave overtopping is the result. Like the wave run-up also the wave overtopping over the non perforated inclined wall can be calculated with the help of the EurOtop manual. The non perforated inclined wall can be seen as the seaward slope of a dike or embankment seawall.
In this paragraph the average overtopping is calculated just as the overtopping volumes per single wave. In this paragraph also the flow depth and the flow velocity of the overtopping wave at the top of the inclined wall are calculated by the method of Bosman.

Wave overtopping is the mean discharge over the top of the structure per linear meter of width, q for example in $m^3/s/m$ or in l/s/m. This discharge is easy to measure in model tests by using an overtopping tank. However, in reality there is no constant discharge over the top of the structure during overtopping. The process of overtopping is very random in time and volume. The highest waves will push a large amount of water over the crest in a short period of time, less than a wave period. The lower waves will not produce any overtopping.
For the emptying process it can be useful to know the average overtopping in order to can calculate the inflow by overtopping over a certain period of time, which is necessary to know for the outflow capacity. But it is also important to know the overtopping volumes per single wave in order to can calculate whether the emptying process is capable of discharging a number of large overtopping volumes following each other.

4.5.2.1 Average overtopping

An average overtopping discharge q can only be calculated for quasi-stationary wave and water level conditions. The average overtopping discharge during a storm has to be calculated for each more or less constant storm water level and constant wave conditions. For the following calculations the water level and wave conditions for a 1/100yr storm are used.

The average wave overtopping depends on the ratio between the freeboard height R_c and the wave run-up height Ru.

$$q = f\left(\frac{R_c}{R_u}\right) \qquad \text{Equation 4-13}$$

q:	average overtopping	[$m^3/s/m$]
R_c:	freeboard height	[m]
R_u:	wave run-up height	[m]

Relative freeboard height:

$$\frac{R_c}{c_{u,1}\xi_{m-1,0}H_{m0}\gamma_b\gamma_f\gamma_\beta\gamma_v} \quad \text{for breaking waves and a maximum of}$$

$$\frac{R_c}{c_{u,2}H_{m0}\gamma_f\gamma_\beta} \quad \text{for non-breaking waves} \qquad \text{Equation 4-14}$$

R_c:	freeboard height	[m]
$c_{u,1}$ and $c_{u,2}$:	coefficients	[-]
$\xi_{m-1,0}$:	surf similarity parameter	[-]
H_{m0}:	spectral wave height	[m]
γ_b:	influence factor for a berm	[-]

γ_f: influence factor for roughness elements on a slope [-]
γ_β: influence factor for oblique wave attack [-]
γ_v: influence factor for wave wall [-]

As one can see from Equation 4-14 there are two formulas for the relative freeboard height. The first formula is for breaking waves, but can never exceed the maximum which is the second formula. For breaking waves always the second formula has to be used to calculate the relative freeboard height.
From Equation 4-14 it can also be seen the relative freeboard height is not any more dependent on the surf similarity parameter.
The above dimensionless relative freeboard height is used by the TAW (Technische Adviescommissie voor Waterkeringen) to derive to following overtopping discharge formulas. These overtopping formulas are valid for a surf similarity parameter $\xi_{m-1,0}< 5$.

<u>Probabilistic design and prediction or comparison of measurements</u> ($\xi_{m-1,0}<5$)
Dimensionless average overtopping discharge for breaking and non-breaking waves.

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = \frac{0,067}{\sqrt{\tan\alpha}}\gamma_b\xi_{m-1,0}\exp(-c_1\frac{R_c}{\xi_{m-1,0}H_{m0}\gamma_b\gamma_f\gamma_\beta\gamma_v})\text{for breaking waves and}$$

with a maximum of: Equation 4-15

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = 0,2\exp(-c_2\frac{R_c}{H_{m0}\gamma_f\gamma_\beta}) \qquad \text{for non-breaking waves}$$

c_1: stochastic coefficient with μ=4,75 and σ=0,5 [-]
c_2: stochastic coefficient with μ=2,6 and σ=0,35 [-]

From Equation 4-13 it became already clear that the average overtopping is a function of the relative freeboard height. Therefore also for the overtopping a distinction is made between breaking waves and non-breaking waves as can be seen from Equation 4-15. The overtopping for breaking waves cannot exceed a maximum and this maximum is also the formula for the average overtopping for non-breaking waves.

For shallow and very shallow foreshores a separate formula is derived for the average overtopping. Underestimation would be the result when using the above formulas for the case of very heavy breaking on shallow foreshores. This kind of breaking on shallow foreshores results in a transformation of the wave spectrum to a flat spectrum with no significant peak period and therefore long waves are present and influencing the surf similarity parameter.

<u>Probabilistic design and prediction or comparison of measurements</u> ($\xi_{m-1,0}>7$)

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = 10^{c3}\exp(-\frac{R_c}{\gamma_f\gamma_\beta H_{m0}(0,33+0,022\xi_{m-1,0})}) \qquad \text{Equation 4-16}$$

c_3: stochastic coefficient with μ=-0,92 and σ=0,24 [-]

<u>Probabilistic design and prediction or comparison of measurements</u> ($5<\xi_{m-1,0}<7$)
Linear interpolation is recommended when one calculates the average overtopping for surf similarity parameters in the range of $5<\xi_{m-1,0}<7$.

To understand the overtopping process it is important to know which parameters are of influence on the average overtopping. Therefore a sensitivity analysis has been performed for the two structural parameters α and h_{wall} and for the two wave parameters H_{mo} and $T_{m-1,0}$ to look for their influence on the average overtopping.

Seaward slope α

The seaward slope α is expressed in the surf similarity parameter ξ as one can see in the previous paragraph. An increasing slope angle of the inclined wall results in an increase of the relative wave run-up, see Figure 4-13.

From Equation 4-15 one can see that the function of the overtopping is an exponent of the relative freeboard height. The surf similarity parameter is expressed in the wave run-up, which is in the denominator of the relative freeboard height. One can imagine that the wave overtopping increases when the wave run-up increases due to an increasing slope angle. As mentioned in paragraph 4.5.1 gentle slopes causes more energy dissipation than steep slopes due to the larger distance which needs to be travelled and due to the breaking of waves.

From Figure 4-14 it can be seen that an increasing slope angle causes an increase in the dimensionless average overtopping. The jump in Figure 4-14 can be explained by the transition zone between Equation 4-15 and Equation 4-16.

Figure 4-15 shows the three different functions. One can see that two exponential functions are separated by a constant function. For non-breaking waves the constant function given by Equation 4-15 is valid. This is also the maximum for breaking waves. Angles which are smaller than 30 degrees result in wave breaking.

For breaking waves the average overtopping increases for an increasing slope angle. When the breaking point is reached the average overtopping is constant for increasing slope angle and at a certain moment when the Iribarren number is larger than 7 the average overtopping is increasing again for increasing slope angle.

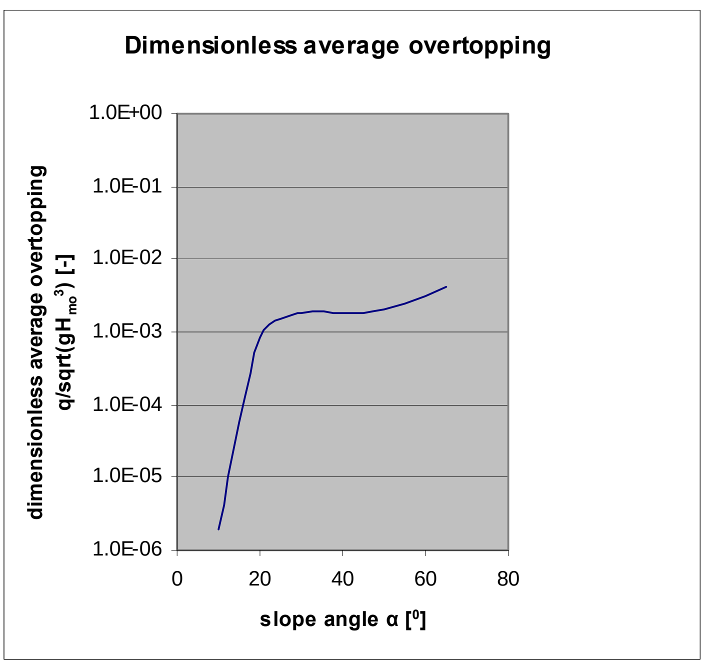


Figure 4-14: Dimensionless average overtopping as a function of the slope angle for h_{wall}=10m, H_{m0}=2,2m, $T_{m-1,0}$=5,25s and h=6m

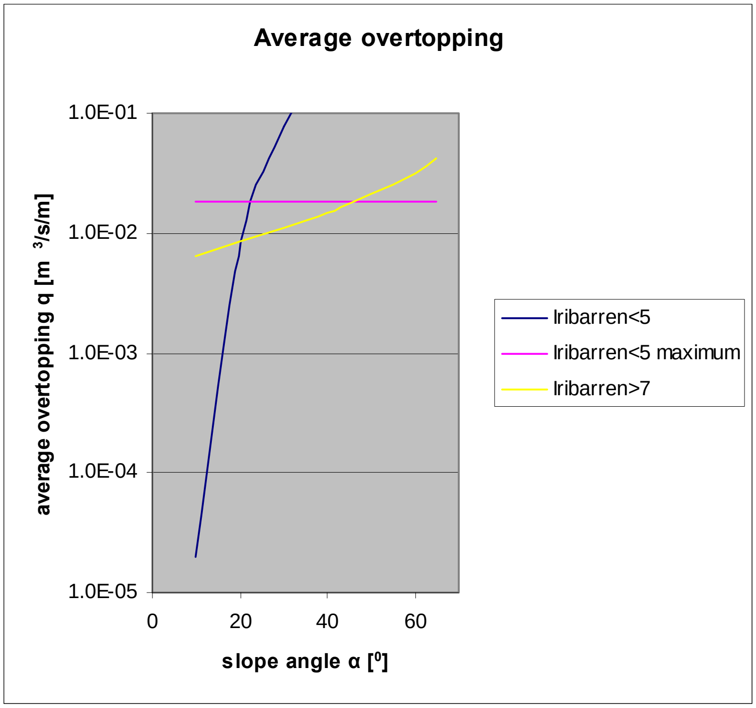


Figure 4-15: Average overtopping curves for the three different Iribarren ranges as a function of the slope angle for h_{wall}=10m, H_{m0}=2,2m, $T_{m-1,0}$=5,25s and h=6m.

The curves in Figure 4-14 and in Figure 4-15 look physical impossible, because of the sudden jumps in the curve. These jumps result from the transition between the different Iribarren ranges. The transitions are determined by the type of wave breaking and therefore the jump in the curve is physical possible.

Height of inclined wall h_{wall}

The height of the inclined wall is expressed in the freeboard height R_c, which can be found in the exponent of the overtopping function. From Equation 4-15 it can be seen that an increasing R_c results in a decrease of overtopping. This can also be seen in Figure 4-16. The higher the wall the less overtopping is possible. The slope angle of the inclined wall is in this sensitivity analysis kept constant.

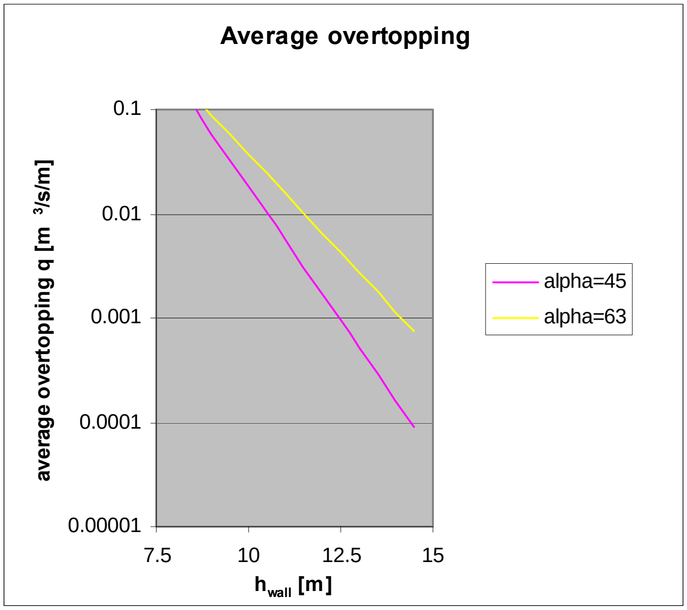


Figure 4-16: Average overtopping as a function of the height of the wall for slope angles 1:1 and 2:1 for H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m

Wave height H_{m0}

The wave height is also expressed in the exponent of the overtopping function, in the denominator of the relative freeboard height. But the wave height is also expressed in the term before the exponent. An increase of the wave height results in an increase of the average overtopping. This can be seen from Equation 4-15 and from Figure 4-17.

In Figure 4-17 a jump can be seen in the curve for α=30^0. This jump represents the transition between breaking and non-breaking waves, for both areas a different formula is used. The other two curves are both in the non-breaking area and therefore no jump is present in these curves.

From physical point of view it is logic that the average wave overtopping increases when the wave height increases. This can be explained by the fact that the freeboard becomes smaller for higher waves, which results in more overtopping.

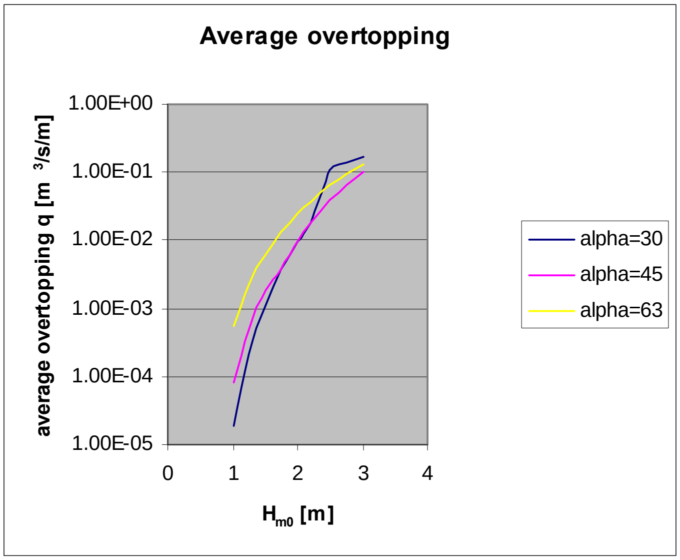


Figure 4-17: Average overtopping as a function of the wave height for slope angles 1:1; 2:1 and alpha=30⁰ for h_{wall}=10m, $T_{m-1,0}$=5,25s and h=6m

Wave period $T_{m-1,0}$

An increase in $T_{m-1,0}$ results in an increase in the surf similarity parameter. However for non-breaking waves the surf similarity parameter is not of influence on the average overtopping and therefore neither $T_{m-1,0}$. This is the reason for q being constant for increasing wave period in the non-breaking area, see Figure 4-18. In the breaking range, q increases with an increasing $T_{m-1,0}$, but in the situation of Figure 4-18 also the maximum value applies and therefore the q is constant for increasing $T_{m-1,0}$. For a larger wave height the breaking waves do not reach the maximum value and then an increase in the breaking range can also be seen in Figure 4-18. In this figure a slope of 30^0 has been used instead of the 1:1 and 2:1 slopes in order to can see the difference between the breaking and non-breaking area.

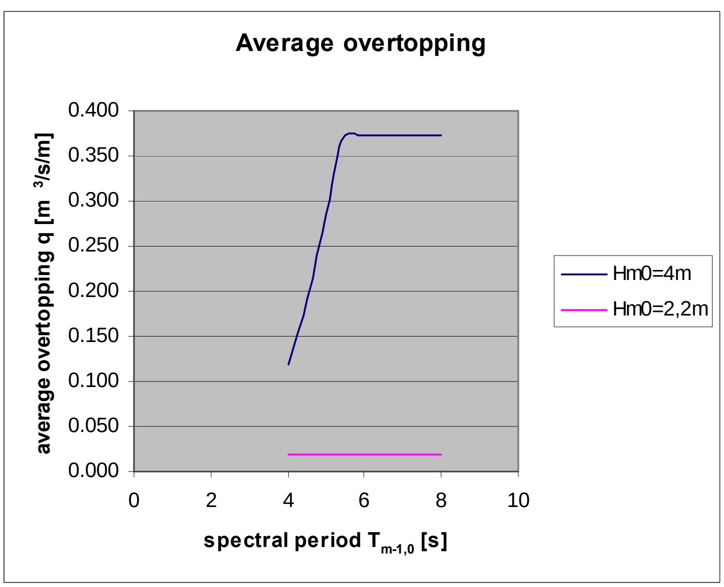


Figure 4-18: Average overtopping as a function of the spectral wave period $T_{m-1,0}$ for slope angle=30^0 and for h_{wall}=10m, H_{m0}=2,2m and h=6m

The amount of allowable overtopping is different between the range of application of the EurOtop manual (dikes and embankments) and the situation of the New Orleans barrier. The manual is actually meant for design conditions with little overtopping and for the New Orleans barrier situation significant overtopping is allowed and overtopping is even a design criterion.
However the formulas from the manual can be used to calculate the overtopping and the upper limit is a zero freeboard situation. Zero freeboard means that the water level comes close to the top of the structure. For this situation the manual gives a different set of formulas, see Equation 4-17.

<u>Zero freeboard</u>

$$\frac{q}{\sqrt{gH_{m0}{}^{3}}} = 0{,}0537\xi_{m-1,0} \qquad for: \xi_{m-1,0} < 2$$

$$\frac{q}{\sqrt{gH_{m0}{}^{3}}} = (0{,}136 - \frac{0{,}226}{\xi_{m-1,0}{}^{3}} \qquad for: \xi_{m-1,0} \geq 2$$

Equation 4-17

From Figure 4-19 one can see that the dimensionless average overtopping increases with an increasing surf similarity parameter.

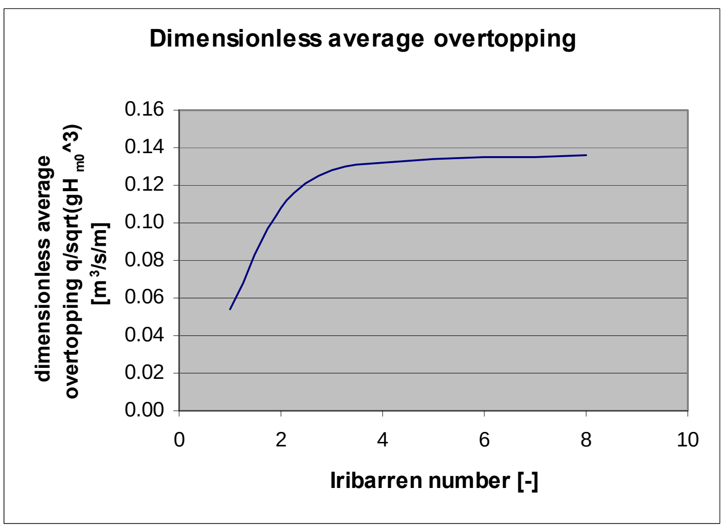


Figure 4-19: Dimensionless average overtopping as a function of the surf similarity parameter for H_{m0}=2,2m

Reduction factors
Besides the wave height and the Iribarren number the wave run-up depends on several reduction factors. These factors take the influence of a berm, the influence of roughness on a slope, the influence of oblique waves and the influence of a wave wall on the wave run-up into account.
The following factors are used in the wave run-up formula and are therefore also important for the wave overtopping:

γ_b:	influence factor for a berm	[-]
γ_f:	influence factor for roughness elements on a slope	[-]
γ_β:	influence factor for oblique wave attack	[-]
γ_v:	influence factor for wave wall	[-]

However in this research it is assumed the waves are perpendicular incident and are encountering a smooth slope without a berm and without a wave wall. Therefore the above described influence factors are set to 1.

4.5.2.2 Overtopping volumes and Vmax

In the introduction of this paragraph it was already mentioned that it is important to know the overtopping volumes per single wave. These volumes can be significant larger than the average overtopping. From Figure 4-20 (EurOtop manual), it can be seen that the ratio q/V_{max} is about 1000 for small q and about 100 for large q. So the maximum volume in the largest wave is about 100-1000 times larger than the mean overtopping discharge. From Figure 4-20 it can also be concluded that for lower wave heights, but for similar mean discharge q, the maximum overtopping volume is smaller.

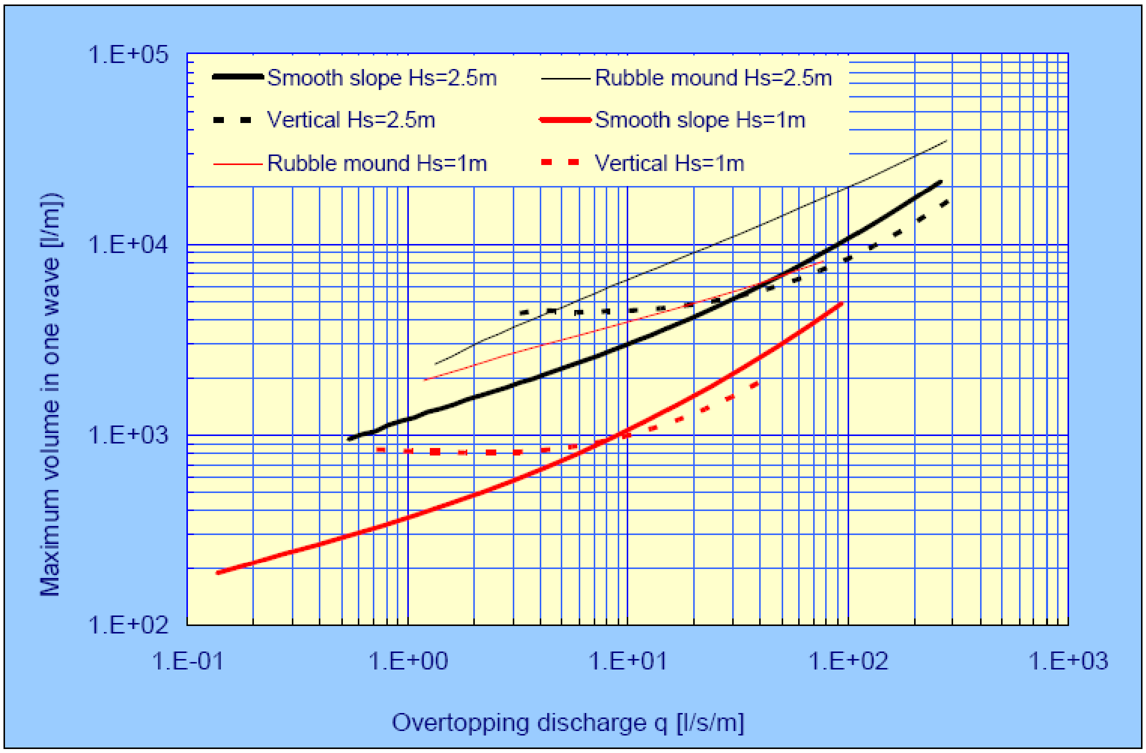


Figure 4-20: relationship between mean discharge and maximum overtopping volume in one wave for smooth, rubble mound and vertical structures for wave heights of 1m and 2,5m [EurOtop manual]

The overtopping volumes per wave can be described by a Weibull distribution with a shape factor of 0.75 and a scale factor a.

The exceedance probability P_V of an overtopping volume per wave V is described as:

$$P_V = P(\underline{V} \leq V) = 1 - \exp\left[-\left(\frac{V}{a}\right)^{0.75}\right] \qquad \text{Equation 4-18}$$

With:

$$a = 0.84 T_{m02} \frac{q}{P_{ov}} = 0.84 T_{m02} q \frac{N_w}{N_{ow}} = 0.84 q \frac{t}{N_{ow}} \qquad \text{Equation 4-19}$$

$$N_w = \frac{\text{storm duration}; t[s]}{T_{m02}[s]}$$ Equation 4-20

Where:

P_V:	exceedance probability of an overtopping volume per wave	[-]
V:	overtopping volume per wave	[m³/m]
a:	scale factor	[-]
T_{m02}:	mean wave period = 4,2s	[s]
P_{ov}:	probability of overtopping per wave	[-]
N_w:	number of incoming waves	[-]
N_{ow}:	number of overtopping waves	[-]
t:	storm duration = 6hr	[s]
q:	average overtopping	[m³/s/m]

In order to calculate the number of waves during a storm, the mean wave period has to be used. As one can see in the above equations T_{m02} has been used.
Also the average overtopping is used, which is described in the previous paragraph. As paragraph 4.5.2.1 describes, the average overtopping does have a maximum and this maximum also counts for the q in the above given formula's.

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

$$P_{ov} = \exp\left[-(\sqrt{-\ln 0{,}02}\,\frac{R_C}{R_{u2\%}})^2\right] = \frac{N_{ow}}{N_w}$$ Equation 4-21

Maximum overtopping volume in a storm:

$$V_{\max} = a\left[\ln(N_{ow})\right]^{4/3}$$ Equation 4-22

a:	scale parameter	[-]
N_{ow}:	number of overtopping waves	[-]

In Figure 4-21; Figure 4-22 and Figure 4-23 the overtopping volumes per wave are depicted as a function of three different wave heights, namely the H_{m0}, the $H_{0.1\%}$ and the $H_{1/3}$. In these figures the volumes are plotted for three different slope angles.

In the non-breaking range and for surf similarity parameters < 7 the average overtopping is constant for increasing slope angles, see Figure 4-15. In this range the maximum overtopping volume is decreasing for increasing slope angle, because the probability of overtopping is increasing. When there are less overtopping waves then the overtopping events that occur are more extreme resulting in larger maximum overtopping volumes.
In the non-breaking range for surf similarity parameters >7 the average overtopping q is increasing again and therefore the overtopping volumes are increasing for increasing slope angles. This can also be seen in the figures below. The lines are in increasing order: α=45, α=30 and α=63 degrees.

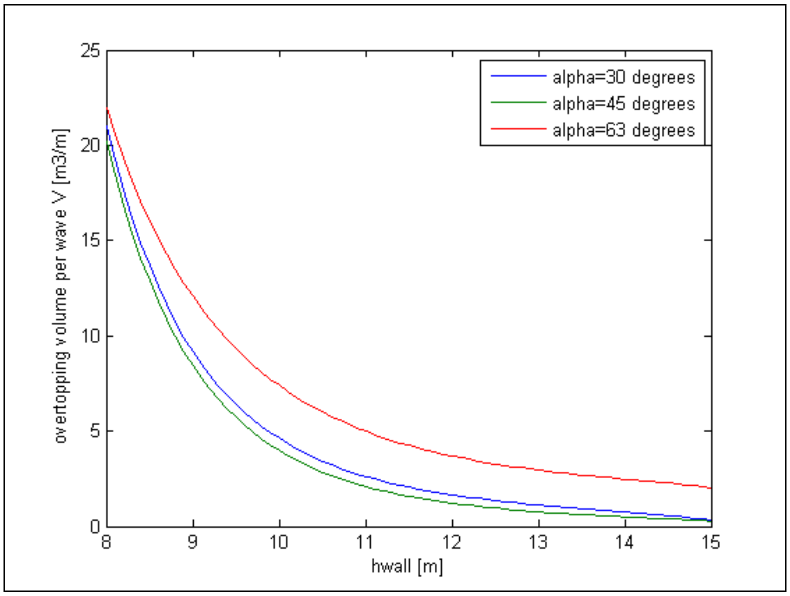


Figure 4-21: Overtopping volume per wave as a function of h_{wall} due to H_{m0}=2,2m and for slope angles 30; 45 and 63 degrees

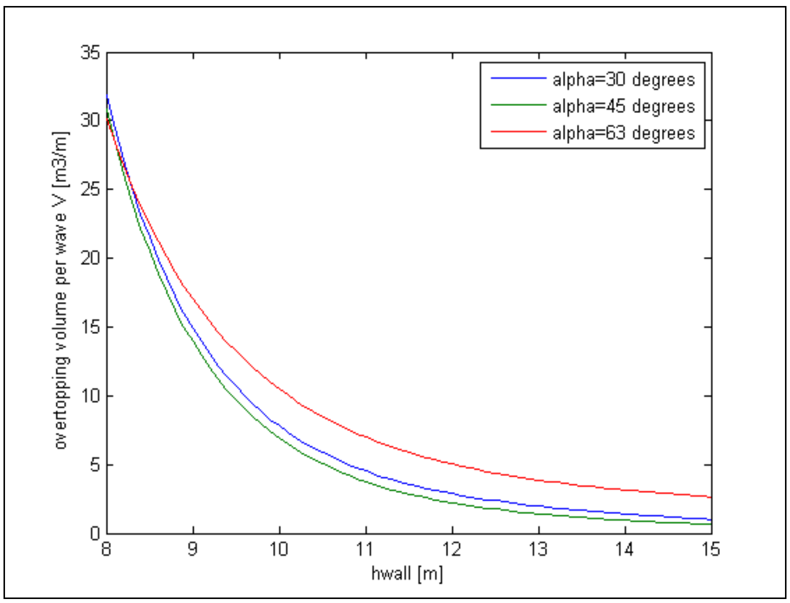


Figure 4-22: Overtopping volume per wave as a function of h_{wall} due to $H_{0.1\%}$=2,5m and for slope angles 30; 45 and 63 degrees

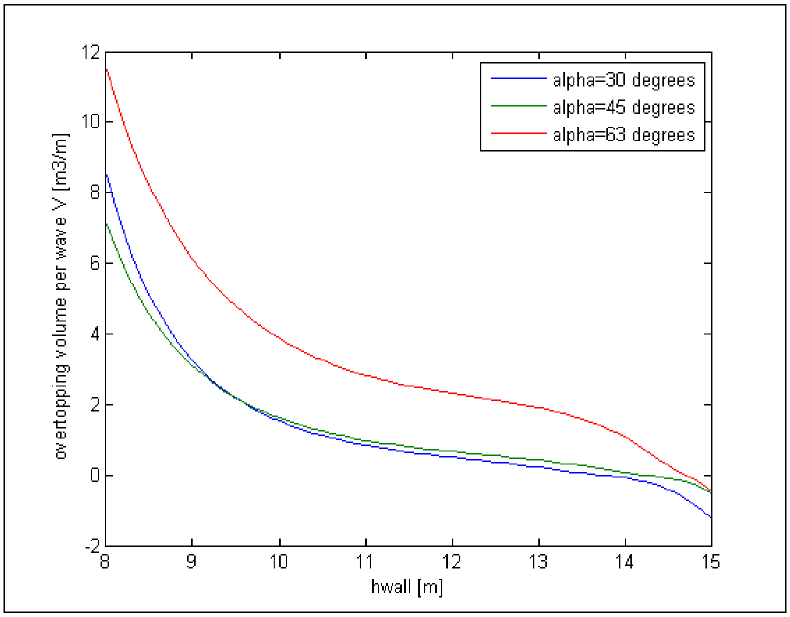


Figure 4-23: Overtopping volume per wave as a function of h_{wall} due to $H_{1/3}$=1,7m and for slope angles 30; 45 and 63 degrees

4.5.2.3 Overtopping flow velocities and overtopping flow depth

The overtopping flow over the inclined wall has a certain velocity and a certain flow depth. The velocity and the slope angle of the inclined wall determine the place of plunging in the basin. It is important to know the path of the overtopping water jet to avoid water jumping over the vertical wall due to a too small width between the inclined and vertical wall.

For dikes and embankments the flow velocity and depth are important parameters for the failure mechanisms. Therefore research has been performed by Schuttrumpf and Van Gent in 2003. Empirical and theoretical functions were derived and verified by experimental data in small and large scale.

Both persons derived formulas for the flow velocity and for the water depth at the top of the dike crest. However the slope angle was missing in both formulas. G. Bosman did some research and derived a new coefficient for the formulas of Schuttrumpf and Van Gent consisting also this slope angle α.

In this paragraph first the overtopping flow depth, velocity and overtopping time are derived and then the path of the water jet is calculated.

Overtopping flow depth, velocity and overtopping time at the top of the inclined wall exceeded by 2% of the incoming waves by Bosman

The formulas for the flow depth, the velocity and the overtopping time are all dependent on the difference between the run-up height and the crest height. For the formulas of the flow depth and the velocity the slope angle is expressed in the coefficient.

Dimensionless wave run-up flow depth:

$$\frac{h_{2\%}(x_c = 0)}{H_{m0}} = \frac{7 * 10^{-3}}{\sin^2 \alpha}\left(\frac{R_{u2\%} - R_c}{\gamma_c H_{m0}}\right) \qquad \text{Equation 4-23}$$

$h_{2\%}$:	overtopping flow depth exceeded by 2% of the incoming waves	[m]
H_{m0}:	spectral wave height	[m]
α:	slope angle of the inclined wall	[^{0}]
$R_{u2\%}$:	wave run-up height exceeded by 2% of the incoming waves	[m]

R_c: crest height [m]
γ_c: influence factor due to friction at the crest [-]

Dimensionless wave run-up velocity:

$$\frac{u_{2\%}(0)}{\sqrt{gH_{m0}}} = \frac{0.25}{\sin\alpha}\sqrt{\frac{R_{u2\%} - R_c}{\gamma_f H_{m0}}}$$ Equation 4-24

$u_{2\%}$: overtopping velocity exceeded by 2% of the incoming waves [m/s]

Dimensionless overtopping time:

$$\frac{T_{ovt,2\%}(x_c = 0)}{T_{m-1,0}} = 1.02\sqrt{\frac{R_{u2\%} - R_c}{\gamma_f H_{m0}}}$$ Equation 4-25

$T_{ovt,2\%}$: overtopping time exceeded by 2% of the incoming waves [s]
$T_{m-1,0}$: spectral wave period [s]

From Figure 4-24 it is clear that the flow depth decreases for an increasing slope angle. The velocity also decreases for an increasing slope angle, see Figure 4-25. Both these figures are based on a height of the wall of 7m instead of the 10m which was used in the wave run-up calculations. In the overtopping calculations it is meant to have some overtopping and therefore the height of the wall is lowered.
Since the run-up increases for an increasing slope angle the overtopping time also increases. The flow depth decreases linearly for an increasing height of the inclined wall, see Figure 4-26. The height of the wall is only expressed in the crest height R_c and not in the coefficient like the slope angle is.
The velocity and the overtopping time both decrease for an increasing height of the inclined wall. At a certain moment the height of the inclined wall is larger than the run-up height and this results in no overtopping, which means an overtopping velocity and an overtopping time of zero. This can be seen in Figure 4-27.

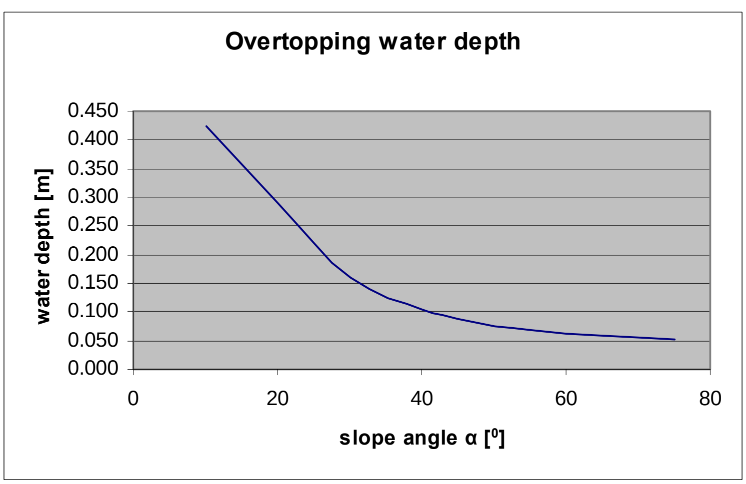


Figure 4-24: Overtopping water depth as a function of the slope angle for h_{wall}=7m; H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m

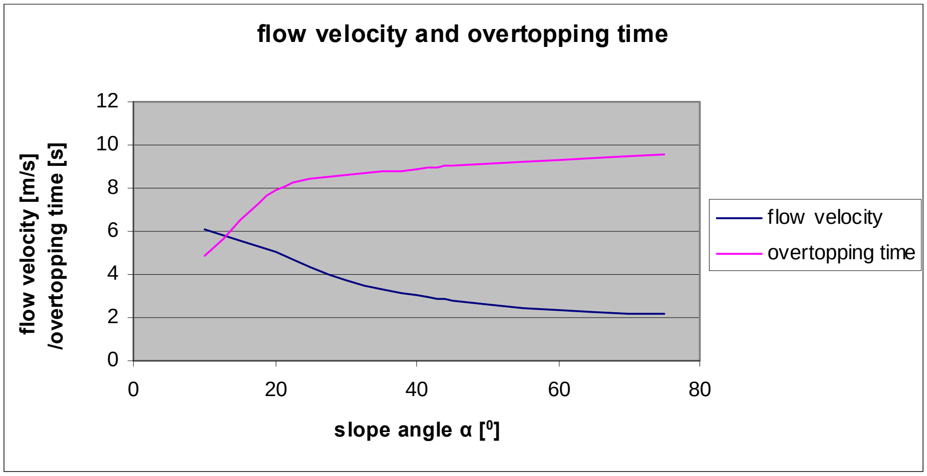


Figure 4-25: Flow velocity and overtopping time as a function of the slope angle for h_{wall}=7m; H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m

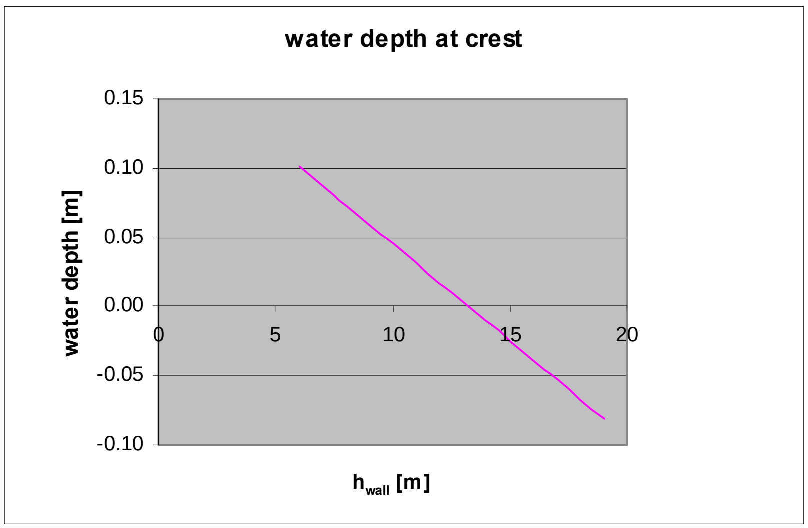


Figure 4-26: Overtopping water depth as a function of the height of the inclined wall for a 1:1 slope; H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m

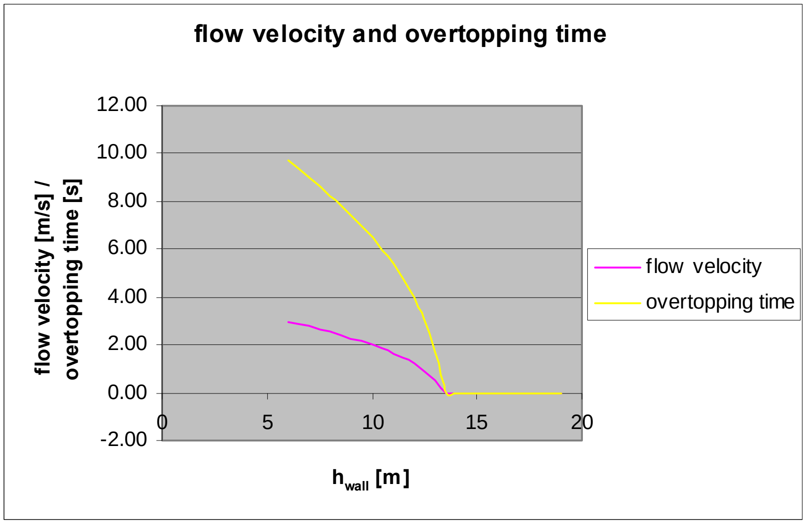


Figure 4-27: Flow velocity and overtopping time as a function of the height of the inclined wall for a 1:1 slope; H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m

Water jet path

The water jet overtopping the inclined wall has a certain velocity and a certain flow depth. It is important to know the path of the jet in order to can apply a sufficient basin width to prevent the water jet jumping over the vertical wall. The path of water jet can be calculated with the following formula [O. Sinniger R., H. Hager W., 1989]:

$$\bar{t} = \tan\alpha * \bar{x} - \frac{g\bar{x}^2}{2u_{2\%}^2(0)\cos^2\alpha} \quad \text{Equation 4-26}$$

t:	vertical distance	[m]
α:	slope angle of inclined wall	[^{0}]
x:	horizontal distance	[m]
g:	gravitational acceleration	[m/s^2]
$u_{2\%}$:	velocity exceeded by 2% of the incoming waves	[m/s]

Figure 4-28 depicts the water jet paths for different slope angles of the inclined wall. From this figure it can be seen that the horizontal distance decreases when the slope angle increases. So the water jet plunges in the basin closer to the inclined wall when the slope angle increases. The inclined wall of the reference design is intended to be steep and the basin width is about 10m. Having this in mind one can conclude from Figure 4-28 that there is no danger the water jet is overtopping the vertical backwall.

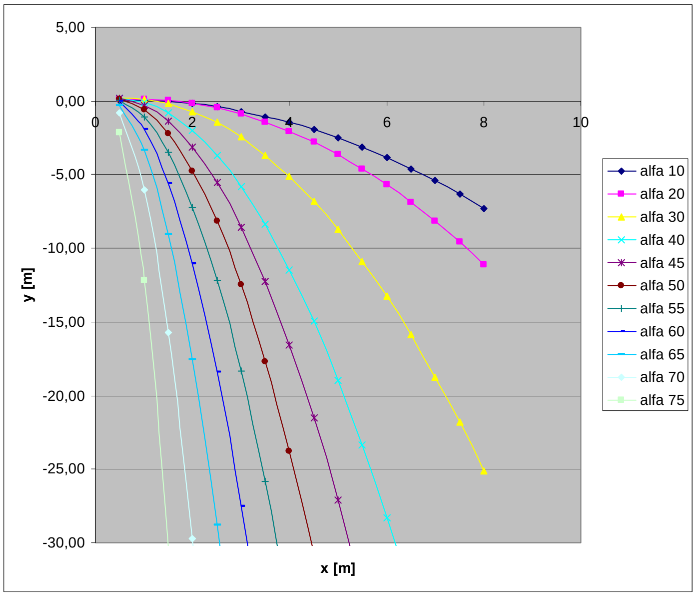


Figure 4-28: Vertical distance of the water jet as a function of the horizontal from the top of the inclined wall

Intermediate conclusions

The average overtopping is a function of the relative freeboard height. There are different overtopping formulas and the one to be used depends on the breaker type. Breaking waves and non-breaking waves overtop differently. For non-breaking waves the average overtopping is constant for increasing slope angle. However, for breaking waves the average overtopping increases for increasing slope angle. This is valid for the area $\xi_{m-1,0}<5$.

The same goes for very have breaking on shallow foreshores $\xi_{m-1,0}>7$, namely the average overtopping increases for increasing slope angle. Between the two Iribarren areas linear interpolation is recommended. The 1:1 sloped wall lies in the $\xi_{m-1,0}<5$ area and the 2:1 sloped wall lies in the $\xi_{m-1,0}>7$ area.

For all the Iribarren areas applies that the overtopping decreases when the height of the wall increases. An increase in wave height results in an increase overtopping. An increase in the wave period $T_{m-1,0}$ results in an increase in the surf similarity parameter. However for non-breaking waves the surf similarity parameter is not of influence on the average overtopping and therefore neither $T_{m-1,0}$.

In the analytical research no reduction factors have to be taken into account for the influence of a berm; the influence of oblique waves; the influence of roughness on a slope or the influence of a wave wall on the wave run-up and consequently on the wave overtopping.

The overtopping volume per single wave is important to know whether the basin can storage these volumes or not. In the non-breaking area a steeper wall results in a larger overtopping volume per single wave.

The thickness of the wave run-up tongue at the top of the wall decreases for an increasing slope angle. The same counts for the flow velocity, namely the flow velocity of the run-up tongue decreases when the slope angle increases. Since the wave run-up increases for an increasing slope angle the overtopping time also increases for an increasing slope angle.

The inclined wall of the reference design is intended to be steep and the basin width is about half a wave length, namely 10-20m. Having this in mind one can conclude from that there is no danger the water jet is overtopping the vertical backwall.

4.5.3 Wave run-up and wave overtopping over a perforated inclined wall

In paragraph 4.5.1 and 4.5.2 the wave run-up and wave overtopping were analyzed for a non perforated inclined wall. The overtopping is dependent on the wave run-up, which is a function of the wave height, the Iribarren number and some reduction factors.
The gap at the bottom of the inclined wall is therefore not easy to implement into the formula for wave run-up. The only way to implement the perforation analytically seems to be to derive an influence factor or to formulate some limits. In paragraph 4.5.3.1 an influence factor for the gap is derived and in paragraph 4.5.3.2 limits are being formulated where in between the wave run-up and wave overtopping are lying.

4.5.3.1 Influence factor for the gap for the wave run-up formula

As discussed in paragraph 4.5.1 the wave run-up depends on the wave height; the surf similarity parameter and some reduction factors. The influence factors are included for an eventual berm; roughness elements on the slope and a certain angle of approach of the incoming waves.
To implement the gap at the bottom of the wall in the run-up formula a derivation of an influence factor seems to be unavoidable, because transforming the Iribarren number is not possible.

A part of the total wave energy is transferred through the gap into the basin and the other part of the total wave energy is reflected by the wall. The amount of wave energy, which passes the gap, can not be used for the wave run-up. The reduction factor which takes the influence of the gap on the wave run-up and wave overtopping into account is assumed to be equal to the reflection coefficient, see Equation 4-27.

$$\gamma_{\mathrm{gap}} = C_{\mathrm{r}} \qquad \text{Equation 4-27}$$

The volume balance and the discharge relation given in paragraph 4.3 result for the rectangular basin with a standing wave inside in a set of equations given in paragraph 4.4.1. The reflection coefficient is given by Equation 4-8. Figure 4-29 depicts the reflection coefficient as a function of the basin width B_{basin} for different gap heights. The reduction factor for the influence of the gap on the wave run-up is equal to the reflection coefficient and therefore can be found from Figure 4-29.

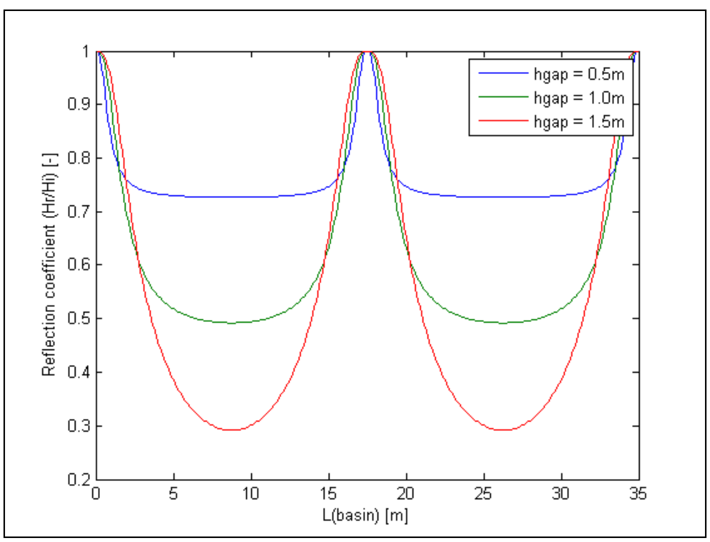


Figure 4-29: Reflection coefficient as function of B_{basin} for different gap heights and L_{gap}=0m

As one can see from the figure above the reflection coefficient is dependent on the basin width. An increase in gap height causes a decrease in reflection. Therefore an increase in gap height results in a decrease of the wave run-up.

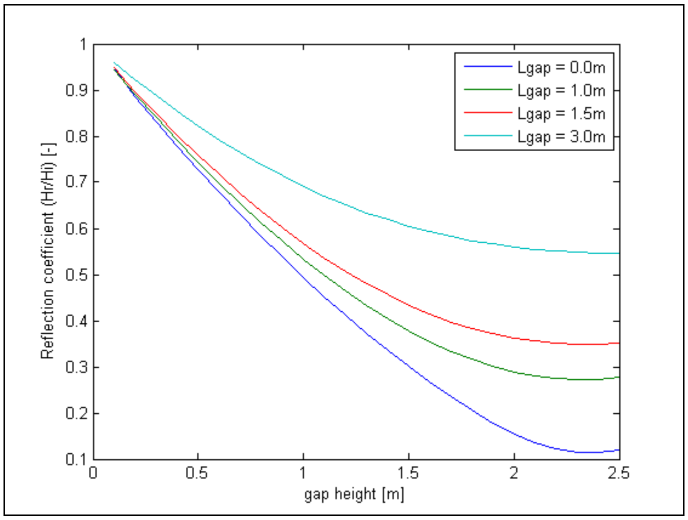


Figure 4-30: Reflection coefficient as a function of the gap height for B_{basin}=10m

Figure 4-30 shows for different gap heights the reflection coefficient/influence factor for a basin width of 10m. The reflection coefficient/influence factor, depicted in Figure 4-29 has to be used in the new run-up formula, see Equation 4-28 and Equation 4-29.

$$\frac{R_{u2\%}}{H_{m0}} = 1,65 * \gamma_{gap}\gamma_b\gamma_f\gamma_\beta * \xi_{m-1,0} \text{ for breaking waves with a maximum of} \qquad \text{Equation 4-28}$$

$$\frac{R_{u2\%}}{H_{m0}} = 1.0 * \gamma_{gap}\gamma_f\gamma_\beta\left(4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}\right) \text{ for non-breaking waves} \qquad \text{Equation 4-29}$$

γ_{gap}: influence factor for a perforation [-]

The overtopping formulas are dependent on the run-up formula and therefore also the overtopping formulas have to be adjusted.

<u>Probabilistic design and prediction or comparison of measurements</u> ($\xi_{m-1,0}$<5)
Dimensionless average overtopping discharge for breaking and non-breaking waves.

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0,067}{\sqrt{\tan\alpha}}\gamma_b\xi_{m-1,0}\exp\left(-c_1\frac{R_c}{\xi_{m-1,0}H_{m0}\gamma_{gap}\gamma_b\gamma_f\gamma_\beta\gamma_v}\right)$$

with a maximum of: Equation 4-30

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0,2\exp\left(-c_2\frac{R_c}{H_{m0}\gamma_{gap}\gamma_f\gamma_\beta}\right)$$

γ_{gap}: influence factor for a perforation [-]
c_1: stochastic coefficient with μ=4,75 and σ=0,5 [-]

c_2: stochastic coefficient with μ=2,6 and σ=0,35 [-]

<u>Probabilistic design and prediction or comparison of measurements</u> ($\xi_{m-1,0}$>7)

$$\frac{q}{\sqrt{gH_{m0}{}^{3}}} = 10^{c3}\exp\left(-\frac{R_c}{\gamma_{gap}\gamma_f\gamma_\beta H_{m0}(0{,}33+0{,}022\xi_{m-1,0})}\right) \qquad \text{Equation 4-31}$$

c_3: stochastic coefficient with μ=-0,92 and σ=0,24 [-]
γ_{gap}: influence factor for a perforation [-]

4.5.3.2 Limits for the overtopping over a perforated inclined wall

For the overtopping discharge different formulas do exist for different types of structures like smooth sloping structures; rubble mound structures and vertical structures. The reference design of this thesis consists out of a perforated smooth inclined wall. At the bottom of this inclined wall a gap is present to make outflow of the basin possible.
However, no overtopping formulas are available for these kinds of structures. Therefore a range is indicated wherein the overtopping discharge of these kinds of structures is lying.

The perforated inclined wall can be split up into two types of structures:

1. smooth sloping structure

   This type of structure has been fully described in chapter 4. Perforation is not taken into account here and therefore the actual overtopping is overestimated and this results in an upper limit for the perforated inclined wall.

2. rubble mound structure

   The gap at the bottom of the inclined wall can be seen as a kind of permeability. A rubble mound structure is also permeable and therefore the perforated inclined wall can be roughly schematized as a rubble mound structure. However a rubble mound structure has also significant friction due to the stones at the slope. The influence factor for the roughness of a rubble mound structure is about γ_f= 0.45.

   So this results in an underestimation for the overtopping resulting in the lower limit for the overtopping over a perforated inclined wall.

The formulas for the smooth sloping structures are known and given in paragraph 4.5.2. The formulas for the rubble mound structures are briefly described below. Some formulas will be the same as for the smooth sloping structures, others are different and the main reason for this is the significant roughness for the rubble mound structures.

Rubble mound structures

The wave run-up and wave overtopping formulas for rubble mound structures are known and well described in the EurOtop manual. As mentioned before a brief description of these formulas is given below.

Wave run-up, $R_{u2\%}$:(probabilistic)

$$\frac{R_{u2\%}}{H_{m0}} = 1.65 * \gamma_b \gamma_f \gamma_\beta * \xi_{m-1,0} \qquad \text{Equation 4-32}$$

$R_{u2\%}$:	wave run-up exceeded by 2% of the incoming waves	[m]
H_{m0}:	spectral wave height	[m]
γ_b:	influence factor for a berm	[-]
γ_f:	influence factor for roughness elements on a slope	[-]
γ_β:	influence factor for oblique wave attack	[-]
$\xi_{m-1,0}$:	surf similarity parameter	[-]

With a maximum of:

$$\frac{R_{u2\%}}{H_{m0}} = 1.0 * \gamma_b \gamma_{fsurging} \gamma_\beta (4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}) \qquad \text{Equation 4-33}$$

$\gamma_{fsurging}$:	influence factor for roughness elements on a slope	[-]

From $\xi_{m-1,0}$=1,8 the roughness factor $\gamma_{fsurging}$ increases linearly up to 1 for $\xi_{m-1,0}$=10, which can be described by:

$$\gamma_{fsurging} = \gamma_f + (\xi_{m-1,0} - 1{,}8)(1-\gamma_f)/8{,}2$$
$$\gamma_{fsurging} = 1{,}0 \; for \; \xi_{m-1,0} > 10 \qquad \text{Equation 4-34}$$

For a permeable core a maximum is reached for $R_{u2\%}/H_{m0}$=1,97

Overtopping discharges, q:

The overtopping discharge for a rubble mound structure is described by only one formula. This formula is the same as for the maximum of the overtopping formula for breaking and non breaking waves for $\xi_{m-1,0} < 5$.

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 * \exp(-2{,}6 \frac{R_C}{H_{m0}\gamma_f \gamma_\beta}) \qquad \text{Equation 4-35}$$

Overtopping volumes per wave:

These formulas are also the same as for the smooth sloping structure. (non-perforated inclined wall)

$$P_V = P(\underline{V} \le V) = 1 - \exp\left[-\left(\frac{V}{a}\right)^{0.75}\right] \qquad \text{Equation 4-36}$$

$$a = 0.84 T_m \frac{q}{P_{ov}} = 0.84 T_m q \frac{N_w}{N_{ow}} = 0.84 q \frac{t}{N_{ow}} \qquad \text{Equation 4-37}$$

From Figure 4-31 and Figure 4-32 it becomes clear that the rubble mound structure is the lower limit for the perforated inclined wall and that the smooth sloping structure is the upper limit. In Figure 4-33 the range of the perforated inclined wall is depicted.

When one now has a perforated inclined wall with a certain relative freeboard one can lookup the range wherein the dimensionless overtopping is lying by means of Figure 4-31-Figure 4-33. However the figures belong to certain slope angles depicted in the figures.

For relative freeboards smaller than 2m a range can be given. This means an upper and a lower limit can be indicated. However for larger relative freeboards the range becomes too large and it is useless to indicate an upper and a lower limit. Therefore for these relative freeboards no limits can be indicated.

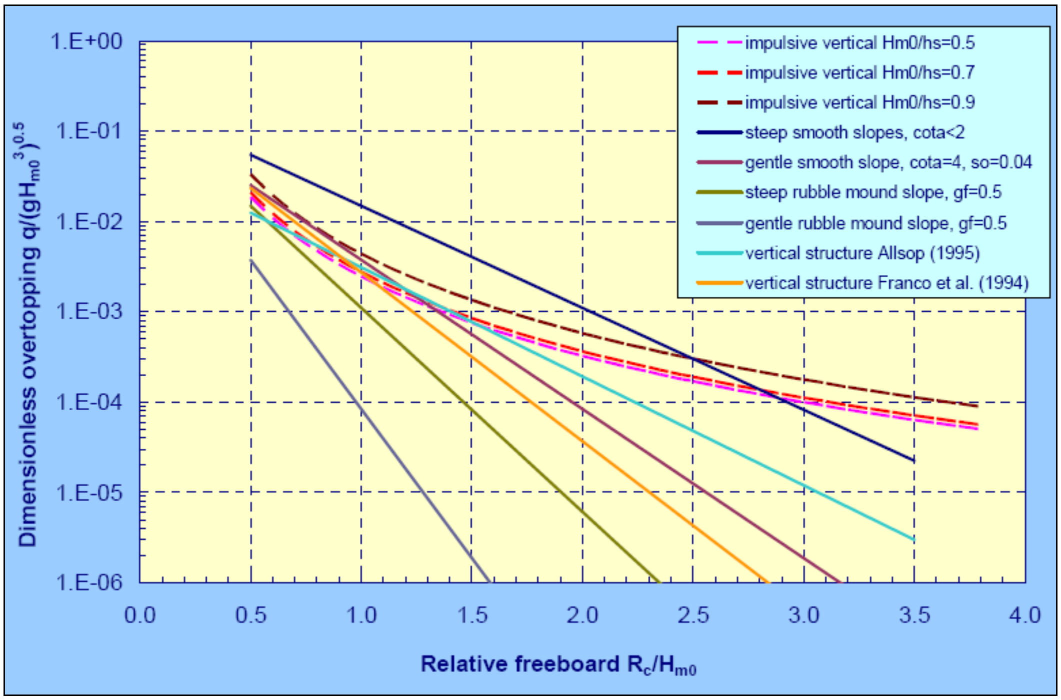


Figure 4-31: Comparison of wave overtopping formulae for various kind of structures [EurOtop manual]

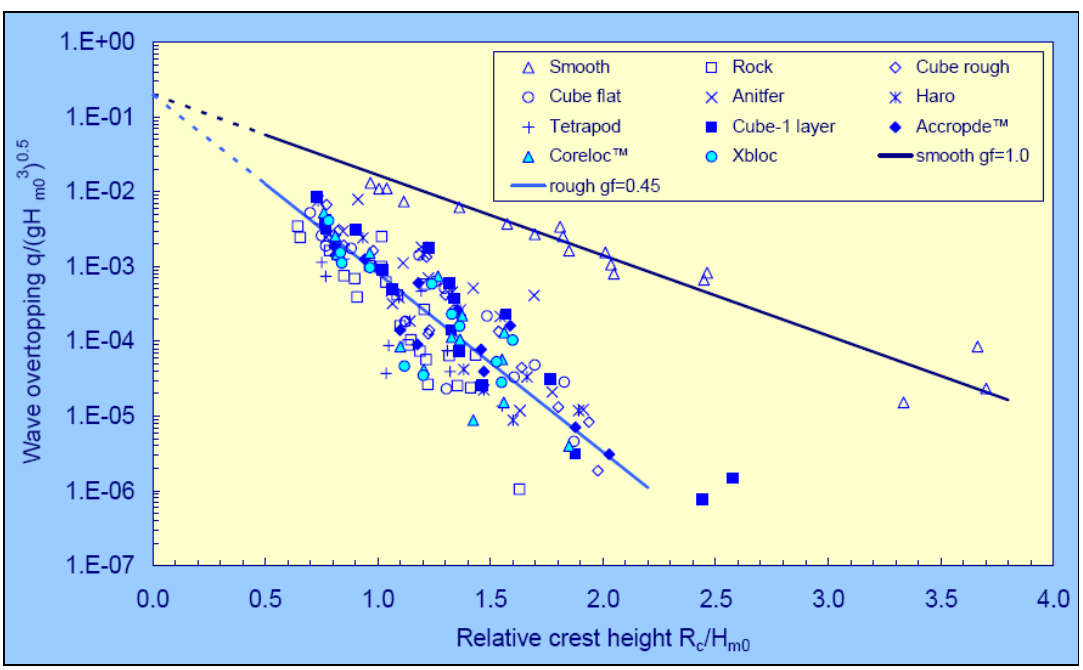


Figure 4-32: Mean overtopping discharge for 1:1,5 smooth and rubble mound slopes [EurOtop manual]

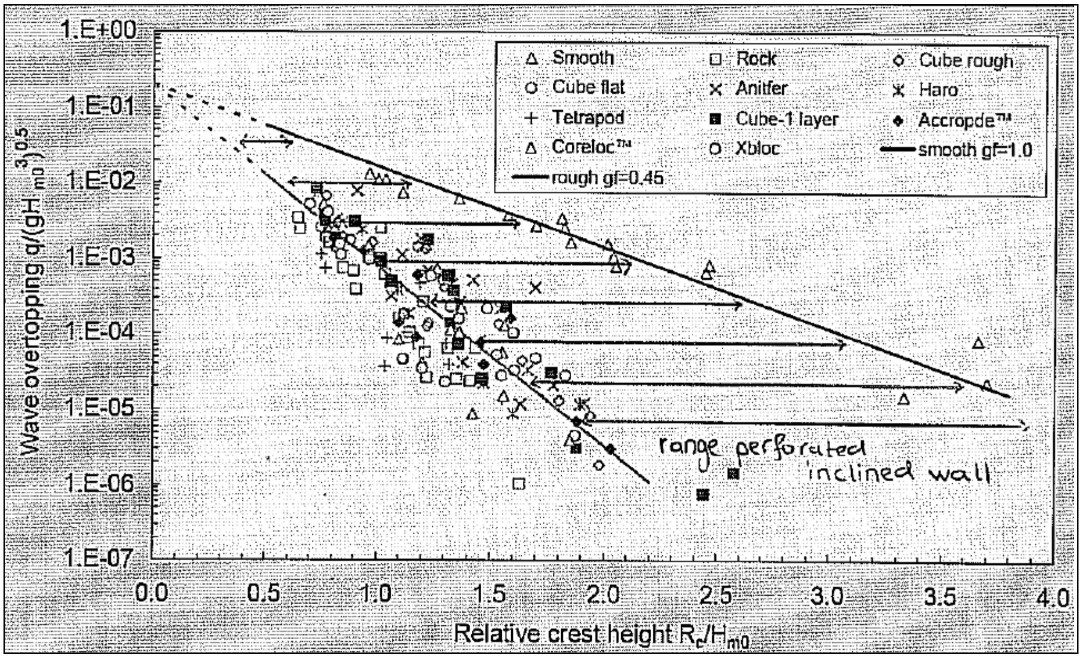


Figure 4-33: Mean overtopping discharge for 1:1,5 smooth; rubble mound slopes and range for perforated smooth slopes

Figure 4-33 depicts the range wherein the average overtopping of a perforated inclined wall is lying. The expectation is that the average overtopping is closer to the upper limit than to the lower limit. The friction loss due to the stones in a rubble mound structure is assumed to have a large impact on the average overtopping. Therefore it is expected that the underestimation of the rubble mound structure is significant more than the overestimation of the non perforated smooth inclined wall.

Intermediate conclusions

The influence of the gap at the bottom of the inclined wall on the wave run-up and on the wave overtopping is taken into account by means of a reduction factor γ_{gap}. This reduction factor is equal to the reflection coefficient, which is based on the loss of energy through the gap.

Also a range has been indicated where in between the wave overtopping discharge is lying. The upper limit represents a non perforated smooth inclined wall and the lower limit represents a rubble mound structure.

4.6 Emptying process

The basin is being filled by overtopping water and by wave induced inflow through the gap at the bottom of the inclined wall. The result is a water level rise in the basin and this causes a head difference between the water level in the basin and the outside water level.

The wave induced inflow is first neglected to describe the emptying process, however the head difference is partly a result of the wave induced inflow.

Thus the outflow depends in this simplification only on the head difference and the Torricelli formula can be used to calculate the outflow. Now the head difference is based on the water level inside the basin and SWL outside the basin, so the pressure differences due to wave motion are not included.

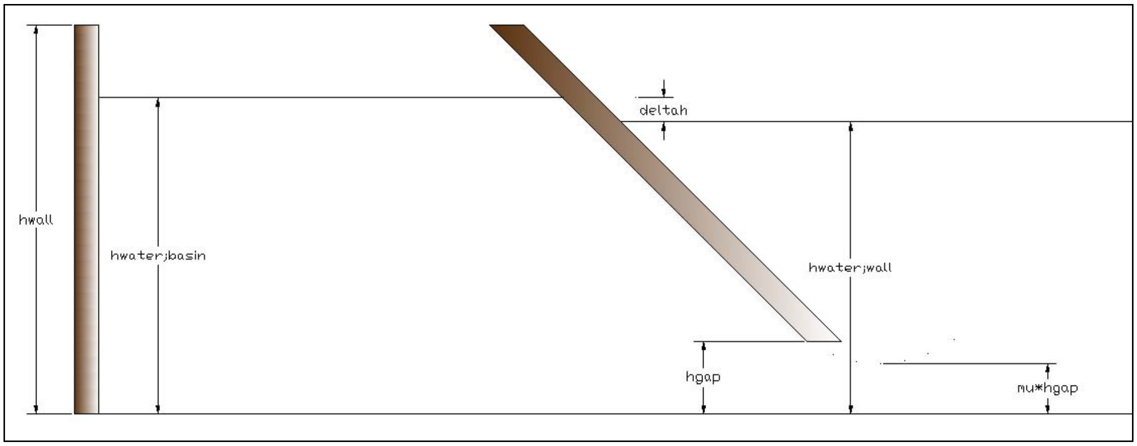


Figure 4-34: Schematized cross-section of the basin of the superstructure

To determine the emptying process first a stationary situation is analyzed. The water level in the basin is not dropping due to the outflow and therefore the head difference remains constant. Therefore the outflow remains also constant which is characterized by a stationary situation. In reality the water level inside the basin is dropping due to the outflow and it is therefore actually a non-stationary situation.

Stationary condition:
When one applies the Bernoulli equation on a streamline starting at the free water surface in the basin to a point in the gap then the velocity and the discharge in the gap can be derived. The energy loss due to the outflow into another water body is proportional to the velocity head.

$$H_{\text{basin}} = h_{\text{water;basin}} + \frac{U_{\text{basin}}^2}{2g}$$

$$H_{\text{wall}} = h_{\text{water;wall}} + \frac{U_{\text{wall}}^2}{2g}$$

Equation 4-38

$$H_{\text{basin}} = H_{\text{wall}} \text{ and } U_{\text{basin}} = 0$$

$$U_{\text{wall}} = \sqrt{2g(h_{\text{water;basin}} - h_{\text{water;wall}})}$$

Where

H_{basin}:	energy level in the basin	[m]
H_{wall}:	energy level in the gap	[m]

This leads to the following set of equations for the velocity and the discharge in the gap at the bottom of the wall, also known as the Torricelli formula:

$$u_{\text{gap}} = \sqrt{2g(h_{\text{water;basin}} - h_{\text{water;wall}})}$$

$$q_{\text{gap}} = \mu h_{\text{gap}} \sqrt{2g\Delta h}$$

Equation 4-39

u_{gap}:	velocity through the gap at the bottom of the inclined wall	[m/s]
g:	gravitational acceleration	[m/s^2]
Δh:	head difference between water level inside and outside the basin	[m]
μ:	contraction coefficient of the gap	[-]

h_{gap}: height of the gap [m]

Non-stationary condition:
As mentioned above the water level in the basin is dropping due to the outflow and therefore the discharge would not be constant. However, when the height of the gap is relatively small compared to the water depth one can assume that the situation is quasi-stationary. When the height of the gap is relatively small, the local acceleration can be neglected compared to the advective acceleration. For the quasi-stationary situation the same formula for the velocity in the gap as for the stationary situation can be used.

For a stationary condition with a constant outflow and constant water volume in the basin the mean residence time of the water particles in the basin would be V/Q. However the head is actually dropping and therefore the outflowing discharge is also decreasing. The emptying duration is therefore longer than V(0)/Q(0) but it is from the same magnitude.
So for a prismatic reservoir with constant surface A_0 the emptying duration is a multiple of the next time scale:

$$\tau = \frac{V(0)}{Q(0)} = \frac{A_0 \Delta h(0)}{\mu A_s \sqrt{2g\Delta h(0)}} = \frac{A_0}{\mu A_s}\sqrt{\frac{\Delta h(0)}{2g}}$$ Equation 4-40

τ:	time scale	[s]
V(0):	initial volume to be discharged	[m³]
Q(0):	initial discharge	[m³/s]
A_0:	area of the water surface	[m²]
A_s:	cross-sectional flow area (perforation area)	[m²]

With the help of the equation of the preservation of volume, Equation 4-41, the emptying duration can be derived, see Equation 4-42.

$$A_0 \frac{dh_0}{dt} = -Q$$
$$A_0 \frac{dZ}{dt} + \mu A_s \sqrt{2gZ}$$ Equation 4-41

The still water surface of the basin A_0 is increasing during the emptying process, because of the implementation of an inclined wall. This increase in basin width has been taken into account, see Equation 4-41. From Equation 4-41 the emptying duration can be derived:

$$t_2 - t_1 = \frac{1}{\mu A_s \sqrt{2g}} \int_{Z_2}^{Z_1} \frac{A_0(Z)}{\sqrt{Z}} dZ$$
$$t_L = -\frac{(z_1 \cot\alpha - 3B_0)}{3\mu A_s}\sqrt{\frac{2z_1}{g}}$$ Equation 4-42

t_L:	duration of emptying the basin	[s]
z_1:	water level in the basin	[m]
z_2:	water level in the basin	[m]
Z:	total head difference	[m]
α:	slope angle	[⁰]

B_0: basin width at water level at t_0 [m]

$\sqrt{\frac{2z_1}{g}}$ time, which a water particle needs to drop over a height Z with an initial velocity of zero.

When an emptying duration is considered z_1 represents the head difference between the inside and outside water level, Δh.

Figure 4-35 depicts the emptying duration as a function of the basin width at water level at t_0 for h_{gap}=0,75m; 1m and 1,5m. When the basin is larger the emptying duration increases, which can be seen from the figure. A larger basin can store a larger volume of water and it takes therefore longer to fill the basin, however for all the basin widths a head difference of 1m has been used.
Besides the basin width and h_{gap}, the head difference Δh and the contraction coefficient μ are of great influence on the emptying duration. On the other hand the slope angle α does not influence t_L significantly.

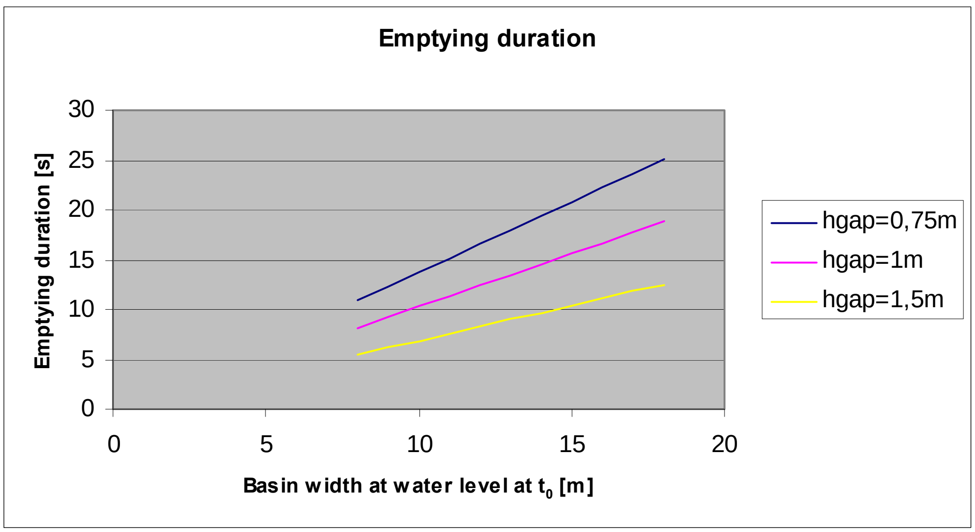


Figure 4-35: Emptying duration as a function of the basin width for Δh=1m; α=45^0 and μ=0,6

Intermediate conclusions
The emptying process is described by the Bernoulli equation, which results in the Torricelli formula. In the schematization wave induced inflow is excluded. The head difference is based on the still water level inside and outside the basin.
The emptying process is significantly influenced by the basin width; the gap height; the head difference and the contraction coefficient. The head difference is a function of the overtopping volumes and the basin capacity, which is not included in the above analysis. For different basin widths the same head difference has been used.

The uncertainties of the emptying process:

- The emptying process has now been schematized as quasi-stationary. Quasi-stationary is the case when the local accelerations in the gap can be neglected compared to the advective accelerations.

4.7 Wave induced inflow without overtopping

The wave induced inflow through the gap at the bottom of the wall is firstly analyzed by considering no overtopping. This can be schematized by considering a vertical wall high enough to prevent overtopping. At the bottom of the wall a gap is implemented where inflow is possible. Behind the perforated wall a basin is present. The inflow is dependent on the gap height and the incoming, reflected and transmitted wave height, which already became clear from paragraph 4.4 'Water movement in the basin'. Therefore it is chosen to use the Long Wave Theory instead of the Linear Wave Theory to determine the wave induced inflow. However the orbital motion can not be considered by this theory and this has to be kept in mind. The result of using this theory is that inflow is present during the entire wave period. Actually half a wave period an inflow through the gap is present and half a wave period an outflow is occurring.

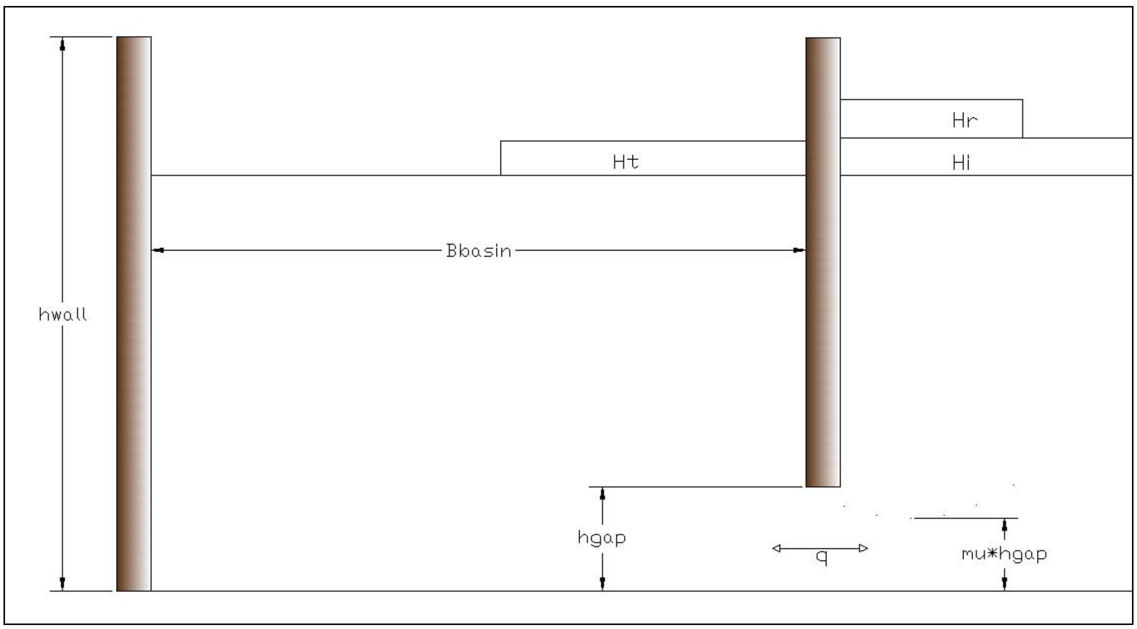


Figure 4-36: Schematization for the wave induced inflow; basic model

For the wave induced inflow also the basic model described in paragraph 4.3 has been used. The flow through the gap can be calculated by Equation 4-43 and Equation 4-44.

$$\tilde{q} = -ci\delta\tilde{h}_b \sin(kl) = c(\delta\tilde{h}_r - \delta\tilde{h}_i) = -c(1-\tilde{C}_r)\delta\tilde{h}_i$$

Where Equation 4-43

$$\delta\tilde{h}_r = \tilde{C}_r\delta\tilde{h}_i = \frac{\alpha c + \tilde{\gamma} - 1}{\alpha c + \tilde{\gamma} + 1}\delta\tilde{h}_i$$

$$\delta\tilde{h}_b = \frac{1}{i\sin(kl)}(\delta\tilde{h}_i - \delta\tilde{h}_r) = \frac{1}{i\sin(kl)}(1-\tilde{C}_r)\delta\tilde{h}_i$$

$$\alpha = \frac{8}{3\pi(\mu a)^2 2g}\hat{q} + \frac{i\omega L_{gap}}{ga}$$

$$\tilde{\gamma} = \frac{\cos(kl)}{i\sin(kl)}$$

$$\tilde{C}_r = \frac{\delta\tilde{h}_r}{\delta\tilde{h}_i} = \frac{\alpha c + \tilde{\gamma} - 1}{\alpha c + \tilde{\gamma} + 1} \qquad \text{Equation 4-44}$$

In Figure 4-37 the wave induced inflow is depicted as a function of the gap height. One can see that the width of the basin does not influence the discharge significantly for small gap heights.

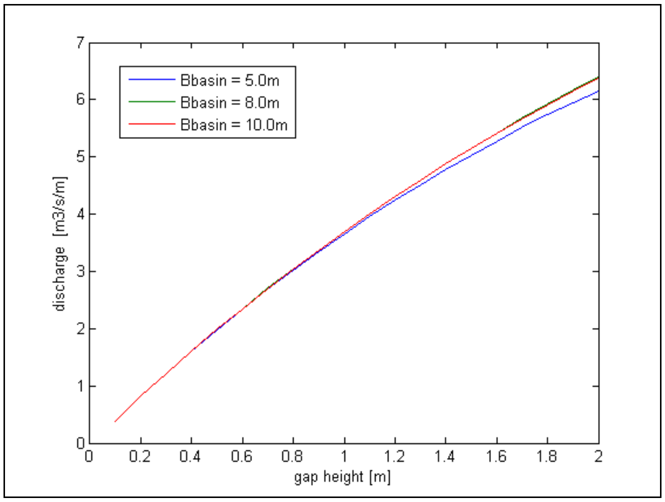


Figure 4-37: Wave induced inflow as a function of the gap height for different basin widths and for Lgap=0m; H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m

Intermediate conclusions

The flow through the gap of the inclined wall can be schematized by means of the Long Wave Theory. The influence of the vertical wall on the inflow is included when one uses this theory. However the orbital motion can not be taken into account by the Long Wave Theory, which has to be kept in mind.

4.8 Flow through perforation

In paragraph 4.6 and 4.7 respectively the emptying process and the wave induced inflow were described separately. In reality the outflow will be counter acted by the wave induced inflow and also the other way around. Therefore in this paragraph the wave induced inflow into the basin and the emptying process are described, taking the influence on each other into account.

The head difference between the inside and outside water level causes an outflow through the gap, which can be counter acted by a wave induced inflow. Both processes were described separately in this chapter, but are combined in this paragraph. Figure 4-38 depicts the head difference between the inside and outside water level and the different wave heights, again based on the basic model.

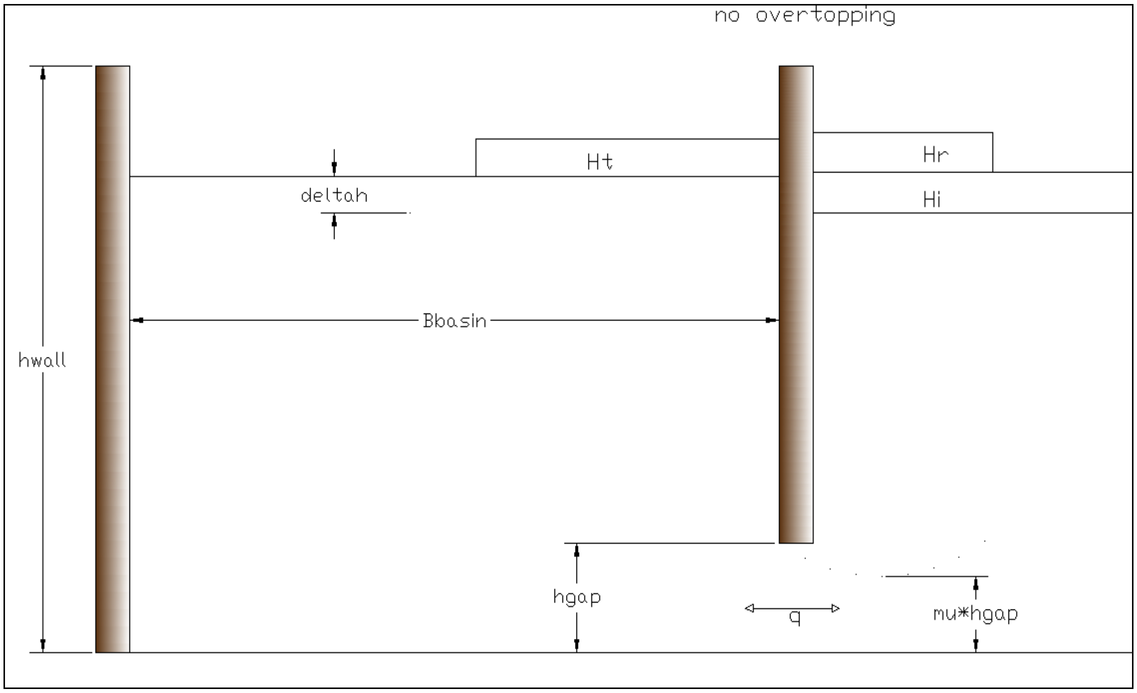


Figure 4-38: Wave induced inflow and the emptying process for the reference design

Outflow due to head difference

The overtopping water volumes over the perforated wall result in a water level rise in the basin. A gap is implemented to empty the basin and this emptying process is excited by the head difference between the inside and outside water level. The emptying process has been described in paragraph 4.6 'Emptying process'. The outflow is described by Equation 4-39. As one can see the formula for the outflow is constant in time. In this formula no water level drop due to outflow is taken into account.

Wave induced inflow

The wave induced inflow has been described in paragraph 4.7 'Wave induced inflow without overtopping'. As already mentioned the orbital motion is not included, because the Long Wave Theory has been used. The inflow is described by Equation 4-43 and Equation 4-44.

Flow through gap, considering the outflow and the wave induced inflow

To determine the flow through the gap when both processes are being considered the Long Wave Theory is used making it possible to implement the water level rise in the discharge relation, which is given by Equation 4-1. When implementing the water level rise the following discharge relation is the result:

$$\left(\frac{8}{3\pi(\mu a)^2 2g} \hat{q} + \frac{i\omega L}{ga} \right) \tilde{q} = \delta\tilde{h}_t + \Delta h - \delta\tilde{h}_i - \delta\tilde{h}_r \qquad \text{Equation 4-45}$$

When using the above given discharge relation and the preservation of volume given by

$$\left. \begin{aligned} \tilde{q}_{gate;l} &= -ci\delta\tilde{h}_b \sin(kl) \\ \tilde{q}_{gate;r} &= c(\delta\tilde{h}_r - \delta\tilde{h}_i) \end{aligned} \right\} -ci\delta\tilde{h}_b \sin(kl) = c(\delta\tilde{h}_r - \delta\tilde{h}_i) \qquad \text{Equation 4-7}$$

Equation 4-7 the following set of equations is derived.

$$\alpha = \frac{8}{3\pi(\mu a)^2 2g}\hat{q} + \frac{i\omega L_{gap}}{ga}$$

$$\tilde{\gamma} = \frac{\cos(kl)}{i\sin(kl)}$$

$$\tilde{C}_r = \frac{\delta\tilde{h}_r}{\delta\tilde{h}_i} = \frac{\alpha c + \tilde{\gamma} - 1}{\alpha c + \tilde{\gamma} + 1}$$

δh_r, the δh_b and the $\tilde{q}$ as function of ∂h_i :

$$\delta\tilde{h}_r = \tilde{C}_r\delta\tilde{h}_i + \frac{\Delta h}{\alpha c + \tilde{\gamma} + 1} = \frac{\alpha c + \tilde{\gamma} - 1}{\alpha c + \tilde{\gamma} + 1}\delta\tilde{h}_i + \frac{\Delta h}{\alpha c + \tilde{\gamma} + 1}$$

$$\delta\tilde{h}_b = \frac{1}{i\sin(kl)}(\delta\tilde{h}_i - \delta\tilde{h}_r) = \frac{1}{i\sin(kl)}((1-\tilde{C}_r)\delta\tilde{h}_i - \frac{\Delta h}{\alpha c + \tilde{\gamma} + 1})$$

$$\tilde{q} = -ci\delta\tilde{h}_b\sin(kl) = c(\delta\tilde{h}_r - \delta\tilde{h}_i) = -c((1-\tilde{C}_r)\delta\tilde{h}_i - \frac{\Delta h}{\alpha c + \tilde{\gamma} + 1})$$

Figure 4-39 depicts the discharge through the gap at the bottom of the wall as a function of the gap height. In the figure the discharge is depicted for different water level rises. The figure is based on a basin width of 10m. When one compares Figure 4-39 with Figure 4-37 in which the wave induced inflow is depicted one can conclude that the wave induced inflow is decreased by implementing the water level rise in the discharge relation.

Because the Long Wave Theory has been used the orbital motion has not been included. In Figure 4-39 time dependency is missing and it has to be kept in mind that in reality the inflow is influenced by the orbital motion.

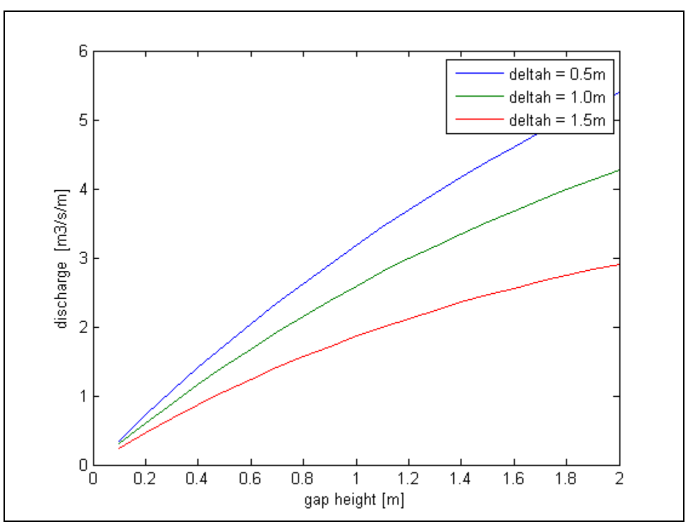


Figure 4-39: Discharge through the gap as a function of the gap height for different water level rises and for B_{basin}=10m; L_{gap}=0m; H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m

In paragraph 4.6 'Emptying process' the emptying duration has been determined, excluding wave overtopping and wave induced inflow. The determined emptying duration gives a good insight of the process and the parameters which are of influence on this process.

It becomes very complex to determine the emptying duration of the basin when wave induced inflow is included and therefore this calculation has not been performed.
For insight into the emptying process one is referred to paragraph 4.6 'Emptying process'.

Intermediate conclusions
The flow through the gap at the bottom of the inclined wall consists of a constant component, which is the outflow due to the head difference and an orbital component, namely the wave induced inflow. Both processes were described separately in chapter 4, but in paragraph 4.8 they are combined.
The discharge through the gap, when both processes are being considered, has been described by the Long Wave Theory. The reason for this is that the Long Wave Theory makes it possible to include the influence of the vertical wall on the flow. The water level rise due to overtopping water volumes is implemented in the discharge relation. Because the Long Wave Theory has been used the orbital motion can not be included.

The emptying duration has not been determined for this situation, because of the complexity. On the other hand insight in the emptying duration has already been obtained in paragraph 4.6 'Emptying process' and therefore the complex determination of the emptying duration for the outflow including the wave induced inflow is not necessary.

4.9 Length spreading effect

The emptying process has only been described for the outflow in the cross-section where the overtopping also takes place. However, since wind-generated waves are short crested and therefore also hurricane waves, the maximum instantaneous overtopping rate does not occur simultaneously along the entire length of the superstructure of the barrier. This gives the superstructure basin a chance to spread extreme overtopping rates in longitudinal directions of the barrier. The length spreading effect will be discussed in this paragraph.

Figure 4-40 depicts the top view of the superstructure of the barrier. This figure shows the length spreading effect. It can be seen from this figure that the overtopped water can flow in the longitudinal direction through the basin and the water can thus drain back in the sea at a different place than the inflow location. In this paragraph an analysis of the length spreading effect for the New Orleans situation is given, just like an estimation for the length spreading effect factor.

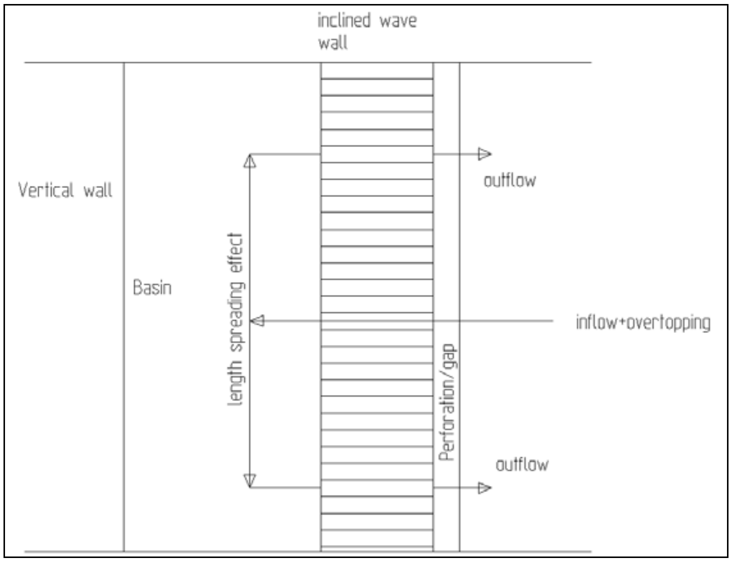


Figure 4-40: Top view superstructure including length spreading effect

4.9.1 Analysis of the length spreading effect for the New Orleans situation

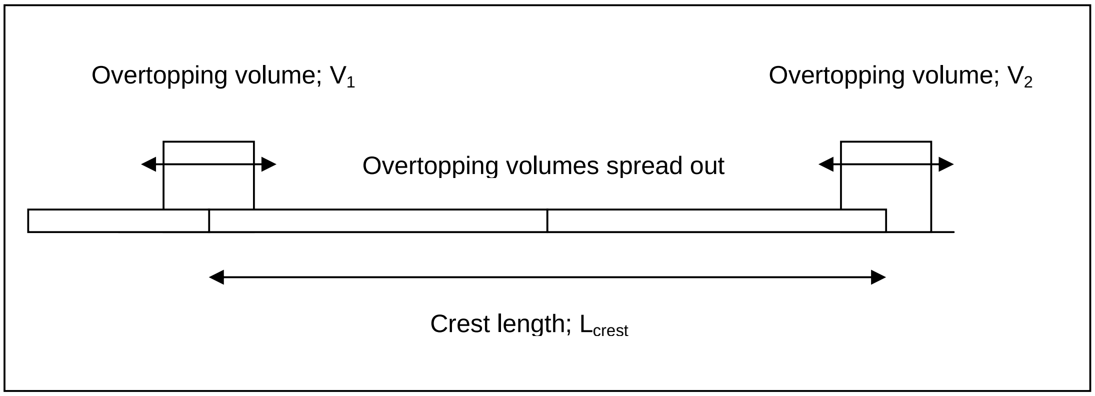
Overtopping volumes of water are entering the basin. These overtopping volumes cause a water level rise in the basin, which induces an outflow through the gap at the bottom of the inclined wall. The length spreading effect is the spread of extreme overtopping rates in longitudinal directions of the barrier. This spread of extreme overtopping rates can increase the buffer capacity of the superstructure.

Before one determines the length spreading effect one has to know whether this effect is important or not for the situation. When the overtopping volumes can be drained back in time, no spreading out of the volumes is necessary.

The maximum length spreading effect can be found when one applies multiple overtopping volumes in the basin with intermediate distances equal to the crest length. These volumes have to be spread out to an equal volume over the entire basin length, see the figure below. The maximum reduction factor is the ratio of the initial water level rise and the water level rise after spreading out the overtopping volume.

Maximum reduction factor

$$\text{length spreading effect}_{\max} = \frac{h_{\text{overtopping volume; before spreading}}\ [\mathrm{m}]}{h_{\text{overtopping volume; after spreading}}\ [\mathrm{m}]}$$



For the length spreading effect three time scales are of importance, namely:

- Time scale of overtopping; intermediate time between two overtopping events
- Time scale of the spreading of the overtopping volumes
- Time scale of the emptying of the basin

A requirement for the validity of this maximum reduction is that the time which is necessary for spreading out the volumes is equal or smaller than the intermediate overtopping time, because the water level in the basin needs time to adapt. So the ratio of $\tau_{spreading}$ and $\tau_{overtopping}$ is important. Further the ratio of $\tau_{spreading}$ and $\tau_{emptying}$ is important. As stated earlier there is no spreading necessary when the emptying duration is very short. There is no time to spread the extreme overtopping rates, because the overtopping volumes are drained back in a short time span.

The three time scales:

$$\tau_{\text{spreading}} = \frac{L_{\text{crest}}}{2c}$$

$$\tau_{\text{overtopping}} = \frac{t - N_{\text{ow}} T_{\text{m02}}}{N_{\text{ow}} - 1} \qquad \text{Equation 4-46}$$

$$\tau_{\text{emptying}} = \frac{V}{Q} = \frac{B_{\text{basin}} \Delta h}{\mu h_{\text{gap}} \sqrt{g \Delta h}}$$

Where

$\tau_{spreading}$:	time scale of the spreading in the basin of the overtopping volumes	[s]
$\tau_{overtopping}$:	time scale of the overtopping time; intermediate time between two overtopping volumes	[s]
$\tau_{emptying}$:	time scale of the emptying process; time which is necessary to drain back the overtopping volume	[s]
L_{crest}:	crest length of the incoming waves	[m]
c:	velocity of the spreading wave	[m/s]
N_{ow}:	number of overtopping waves	[-]
T_{m02}:	mean wave period	[s]
V:	overtopping volume per unit meter	[m³/m]
Q:	discharge through gap per unit meter	[m³/s/m]
B_{basin}:	width of the basin	[m]
Δh:	water level difference between inside and outside water level	[m]
μ:	contraction coefficient of the gap	[-]
h_{gap}:	height of the gap	[m]

When

$$\tau_{\text{overtopping}} \ll \tau_{\text{spreading}} \qquad \text{Equation 4-47}$$

The extreme volume rates can not be spread in one overtopping intermediate time. While the spreading of one overtopping volume is taken place, the next overtopping event is occurring. When the time scale of the overtopping events is far smaller than the time scale of the spreading, no length spreading effect can be taken into account.

When

$$\tau_{\text{emptying}} \ll \tau_{\text{spreading}} \qquad \text{Equation 4-48}$$

The emptying time is far smaller than the spreading time. Before the spreading of the extreme overtopping volume rates is finished, this volume is already drained back through the gap and no length spreading effect can be taken into account.

New Orleans situation

When one uses the next values the result is given in

Table 4-1.

B_{basin}:	width of the basin	[m]	10
α:	angle of the slope	[⁰]	45
h_{wall}:	height of the inclined wall	[m]	9
t:	storm duration	[s]	21600
h_{gap}:	height of the gap	[m]	1,0

Table 4-1: Time scales for configuration with h_{gap}=1m; V_{max}=10m³/s/m; B_{basin}=10m; α=45⁰

Time scale	[s]
$\tau_{spreading}$	6
$\tau_{overtopping}$	4
$\tau_{emptying}$	5,5

From the table one can conclude that both criteria given by Equation 4-47 and Equation 4-48 are not valid. One can see that the time scales are in the same range and therefore the length spreading can be taken into account. This means that the length spreading effect does have a positive influence on the basin capacity.

4.9.2 Estimation of the length spreading effect

Engineering company DHV derived in 2005 a concept, called the Crest Drainage Dike which could be a successful alternative for crest heightening of dikes. The basic concept of the Crest Drainage Dike is a basin, integrated in the crest of a dike, which collects overtopping water and thus reduces the load on the inner slope of a dike. The collected water in a crest basin is drained landward or seaward through pipes.
In this concept also the length spreading effect is included. DHV assumes that the length spreading effect enlarges the buffer capacity to twice the initial buffer capacity. This assumption has not been verified by model tests and an analytical verification is also missing.
In the thesis "Wave overtopping aspects of The Crest Drainage Dike, a theoretical, analytical and experimental research" [Van Steeg, 2007] also the above assumption has been followed.
So for both researches an analytical derivation for the length spreading effect is missing. The length spreading effect has not yet been subject of sufficient research in the past.

The length spreading effect factor is a factor which increases the buffer capacity of the basin due to the length spreading effect. In this thesis also the assumption of the length spreading effect factor of 1.5 is made.

Intermediate conclusions

From the analysis it can be concluded that the length spreading effect can be taken into account and does have a positive influence on the basin capacity of the structure. The spreading time scale; the intermediate overtopping time scale and the emptying time scale are in the same range for the New Orleans situation. A length spreading effect factor of 1,5 is assumed to be a first value. However, further research is necessary.

4.10 Analytical results

The main question of the research was: "What kinds of hydraulic processes occur inside the superstructure and in what manner do they effect the configuration?" In order to can say in what manner the processes effect the configuration firstly good insight in these processes is necessary. In this chapter after the inventory of the occurring hydraulic processes an analysis of these hydraulic processes has been performed for simplifications of the superstructure and for the reference design. Before a numerical model can be constructed wherein the processes are combined and the overtopping over the vertical backwall can be calculated physical model tests are performed. The objective of the physical model tests are to verify the analytical results and to overcome the uncertainties. Chapter 5 'Physical model testing' describes the physical model testing.

Now the analytical results for the hydraulic processes are described briefly.

Water movement in the basin

To describe the water movement in the basin the Long Wave Theory has been used. By applying this theory the different wave heights, by this theory schematized as water level rises, and the flow through the gap can be calculated by means of a discharge relation and equations following from the preservation of volume. The wave heights and the discharge are given by Equation 4-9.

It is stated that the water movement in a basin consisting of a vertical and an inclined wall and the water movement in a rectangular basin are the acting the same. Therefore resonance of the water level in the basin of the reference design is excited for basin widths of n*(0,5L_{wave}) with n=1,2,3etc. To avoid resonance these basin widths can not be applied.

Wave run-up

The wave run-up is influenced by the gap, which is present at the bottom of the inclined wall. It is found that the wave run-up decreases when a gap is implemented at the bottom of the inclined wall. It is also found that when the gap height increases the wave run-up decreases. The influence of the gap on the wave run-up is translated in a reduction factor, which is equal to the reflection coefficient, see Equation 4-27. This reduction factor is based on the loss of energy through the gap. The wave run-up for the perforated inclined wall is given by Equation 4-28 and by Equation 4-29.

Wave overtopping

The wave overtopping is influenced by the gap, which is present at the bottom of the inclined wall. It is found that the same reduction factor for the influence of the gap on the wave run-up is valid for the influence of the gap on the wave overtopping.

The wave overtopping for the perforated inclined wall is given by Equation 4-30 and by Equation 4-31.

Emptying process

A water level rise in the basin is the result of the overtopping water volumes over the perforated inclined wall. The water level rise causes a head difference between the inside and outside water level, which induces an outflow through the gap. The emptying process is described by the Torricelli formula and the outflow through the gap is given by Equation 4-39.

Wave induced inflow

The wave induced inflow through the gap has been calculated by using the Long Wave Theory and is given by Equation 4-43. The orbital motion is not included in this formula, because the Long Wave Theory instead of the Linear Wave Theory has been used. By assuming long waves are present the influence of the vertical wall on the flow can be included, which is not possible when the Linear Wave Theory would be used.

Length spreading effect

The draining in longitudinal direction of the basin of the overtopping water volumes is called length spreading effect. From the analysis it can be concluded that the length spreading effect can be taken into account and does have a positive influence on the basin capacity of the structure. The spreading time scale; the intermediate overtopping time scale and the emptying time scale are in the same range for the New Orleans situation. A length spreading effect factor of 1,5 is assumed to be a first value. However, further research is necessary.

Chapter 5 Physical model testing

5.1 Introduction

Chapter 7 'A numerical model' describes a numerical model based on the hydraulic processes which have been described and analyzed in Chapter 4 'Hydraulic processes'. Because during the analysis of the hydraulic processes uncertainties became clear model tests have been performed to solve these uncertainties. Besides solving the encountered uncertainties verifying the analytical results is an objective of the model testing. In this chapter the model testing is described, just like the results of the model testing. In Chapter 6 'Comparison of analytical results and model testing results' the results of the model testing are compared with the analytical results.
Because this research does have the same reference design as the research performed by Ruud Nooij and therefore have a lot interfaces with that research, the model tests for both researches are performed simultaneously. However, the analysis of the results obtained by the model testing has been performed individually.

5.2 Uncertainties and processes to perform model tests on

Within the analytical description of the hydraulic processes some uncertainties are encountered which create a lack of insight. By performing model tests the uncertainties within the occurring processes are tried to be expelled. In this paragraph the processes and additional uncertainties are described.

Wave run-up
Summary analytical part
When a wave encounters a sloping wall it runs up the wall until a certain height. This process is called wave run-up and the vertical distance measured from SWL to the maximum wave run-up height is defined as the wave run-up height.
When considering a smooth and straight inclined wall the wave run-up is influenced by the wall inclination and the wave parameters, like wave height and wave period, combined in the surf similarity parameter. The wave run-up process of such a structure can be described by the present design formulas.
However, in the reference design the inclined wall has a gap at the bottom. It is expected that the change in geometry with respect to an impermeable wall has an effect on the wave run-up process. This effect is described in the analytical part by the implementation of a reduction factor considering the wave energy. It is assumed that a part of the total wave energy is transmitted under the wall into the basin that cannot be used for wave run-up and therefore has a reducing effect on the wave run-up. The reduction factor is equal to the reflection coefficient.

Uncertainties
The uncertainties that arise within the analytical description are the following:
- The influence of the gap on the wave run-up.
- The reduction coefficient describes the reduction of wave run-up by the influence of the gap properly.

Wave overtopping
Summary analytical part
Wave overtopping is very much related to wave run-up. The process of wave overtopping occurs when the crest height of the structure is smaller than the maximum wave run-up height.
When considering a smooth and straight slope the amount of wave overtopping depends on the ratio between the wave run-up height and the crest freeboard height. If the crest freeboard height is smaller

than the wave run-up height wave overtopping occurs and the amount of wave overtopping can be calculated using present design formulas.
When the inclined wall of the reference design is considered it can be expected the gap at the bottom has the same effect on the wave overtopping as on the wave run-up. Therefore the same reduction factor can be used.
Besides a reduction factor also a range is given wherein the overtopping discharge of this kind of structures is lying. The upper limit of this range is a smooth non-perforated inclined wall and the lower limit of this range is a rubble mound breakwater.

Uncertainties
The uncertainties that arise within the analytical description are the following:
- The reduction coefficient describes the reduction of wave run-up and thereby also the reduction of wave overtopping properly in reality.
- The range which is indicated wherein the overtopping discharge of this kind of structures is lying is correct.

Water movement in the basin
Summary analytical part
Water movement in the basin is an important process because large water level fluctuations in the basin are not allowed in order to avoid large variable pressures against the vertical water retaining wall. In the analytical part resonance is discussed and for a rectangular basin the dimensions are known which causes resonance. The dimensions, which cause resonance, are related to the wave length. In the analytical research it is found that resonance is occurring for basin widths of 0,5L or multiples of this length.
However, the reference design consists out of an inclined wall and a vertical wall instead of two vertical walls in front of each other. It is expected that the angle of the wall does not influence the process of resonance much and the same basin widths that cause resonance are valid for the reference design as for the situation with two vertical walls.

Uncertainties
- The configuration of the reference design does act in the same way as the configuration with two vertical walls for the resonance process.

Outflow
Summary analytical part
After wave overtopping of the inclined wall and the inflow through the gap of the wall the water level in the basin rises. This water level rise results in an outflow of water to the seaside due to a water head difference between the water inside the basin and outside the basin. The intensity of this water flow has a certain effect on the incident waves and thereby influences the hydraulic processes.
There are no formulas present to determine the magnitude of the effect of the outflow of water on the incident waves, only a qualitative description can be given.

Uncertainties
- The effect and magnitude of an outflow of water at the bottom of the inclined wall upon the wave run-up and wave overtopping.

5.3 Problem analysis

Problem description
The reference design of the storm surge barrier contains a gap at the bottom of the inclined wall. The inclined wall has to dissipate energy of the approaching waves and overtopping over this wall is

allowed. The overtopped water has to be discharged back into the lake and therefore a perforation in the inclined wall is necessary. The perforation is schematised as a gap at the bottom of the inclined wall.
In the introduction of this chapter it is mentioned that not all the occurring processes in and around the superstructure can be determined analytically. This is because there are no existing formulas for perforated inclined walls. The referred processes are: Wave run-up and wave overtopping over the inclined wall; the influence of the outflow on the approaching waves and water movement in the basin.
The influence of the gap on these different processes is unknown.

Problem definition
The influence of the gap at the bottom of the inclined wall on the wave run-up and wave overtopping is unknown. Also the water movement in the basin of a configuration like the reference design is unknown just like the influence of the outflow on the incoming waves.

Objective
The objective of the model tests is to get insight in the influence of the gap at the bottom of the inclined wall on the above mentioned processes.

Research questions and expectations
In order to comply with the above mentioned objective it is important to have some research questions answered. A division in main and sub questions is made here. For the main questions expectations are included.

<u>Main questions</u>

1. What is the influence of the gap in the inclined wall on the wave run-up?

Expectation 1: It is expected that just like the influence of roughness on a slope; the influence of a berm and the influence of oblique waves that the gap in the inclined wall influences the wave run-up in a form of a reduction on this run-up. It is also expected that the higher the gap the higher the reduction will be.

2. What is the influence of the gap in the inclined wall on the wave overtopping?

Expectation 2: It is expected that just like the wave run-up the overtopping will be reduced when a gap is present in the inclined wall. After all, the wave overtopping is a function of the wave run-up and the crest height. Therefore, influence of the gap in the inclined wall on the wave overtopping is expected when it was already expected for the wave run-up. It is also expected that the higher the gap the higher the reduction will be.

3. What is the influence of a vertical backwall behind the perforated inclined wall on the incoming waves?

Expectation 3: It is expected that the wave, which has been reflected by the vertical backwall has a reducing effect on the wave run-up and wave overtopping.

4. What is the influence of the outflow on the incoming waves?

Expectation 4: It is expected that the outflow through the gap in the inclined wall has a reducing effect on the wave run-up and therefore also a reducing effect on the wave overtopping. It is expected that the larger the outflow the higher the reducing effect on the wave run-up and wave overtopping will be.

5. What is the influence of the gap and the inclined wall on the water movement in the basin? (Whether resonance is occurring or not)

Expectation 5: For the configuration consisting out of a vertical and an inclined wall like the reference design it is unknown what happens inside the basin and for which basin width resonance is occurring. However, it is expected that just like for a rectangular basin for a basin width of 0.5L resonance is occurring and that for 0.75L no water fluctuations are occurring in the basin.

Sub questions

The main question "What is the influence of the gap in the inclined wall on the wave run-up" poses the following sub questions:

- What is the wave run-up height on a *non-perforated inclined wall* with slopes of 1:1 and 2:1 for the given boundary conditions?
- What is the wave run-up height on a *perforated inclined wall* with slopes of 1:1 and 2:1 with a gap (h_{gap}=0,04m-0,10m) at the bottom of the inclined wall for the given boundary conditions?

The main question "What is the influence of the gap in the inclined wall on the wave overtopping" poses the following sub questions:

- What is the wave overtopping discharge for a *non-perforated inclined wall* with slopes of 1:1 and 2:1 for the given boundary conditions?
- What is the wave overtopping discharge for a *perforated inclined wall* with slopes of 1:1 and 2:1 with a gap (h_{gap}=0,04m) at the bottom of the inclined wall for the given boundary conditions?

The main question "What is the influence of a vertical backwall behind the perforated inclined wall on the wave run-up and wave overtopping" poses the following sub questions:

- What is the wave run-up height on a *perforated inclined wall* with a slope of 1:1 with a gap (h_{gap}=0,10m) at the bottom of the inclined wall and a backwall on a distance of 0,5L behind the inclined wall for the given boundary conditions?
- What is the wave run-up height on a *perforated inclined wall* with a slope of 1:1 with a gap (h_{gap}=0,10m) at the bottom of the inclined wall and a backwall on a distance of 0,75L behind the inclined wall for the given boundary conditions?

The main question "What is the influence of the outflow on the incoming waves" poses the following sub questions:

- What is the wave run-up height on a *perforated inclined wall* with a slope of 1:1 with a gap (h_{gap}=0,04m and 0,10m) at the bottom of the inclined wall when an *outflow* is present for the given boundary conditions?
- What is the wave overtopping discharge for a *perforated inclined wall* with a slope of 1:1 with a gap (h_{gap}=0,04m) at the bottom of the inclined wall when an *outflow* is present for the given boundary conditions?

The main question "What is the influence of the gap and the inclined wall on the water movement in the basin" poses the following sub questions:

- What is the standing wave height against the vertical wall behind a *perforated inclined wall* with a slope of 1:1 with a gap (h_{gap}=0,10m) at the bottom of the inclined wall when *no overtopping* is allowed for the given boundary conditions?

In order to calibrate the model additional tests are performed for a non-perforated inclined wall and for a perforated vertical wall.
In order to simulate the outflow induced by a water head difference, which occurs in reality due to the overtopped water a constant outflow is generated in some tests. This outflow is based on the collected overtopped water in the preceding performed tests.

5.4 Experimental set-ups

In order to find the uncertainties described in paragraph 5.2 the model tests have to be prepared and model set-ups have to be determined. In order to get a good notion about the processes to be tested and to get a grip on the uncertainties the model tests have to be simplified as much as possible. This simplification can be done by isolating the processes as much as possible to get a good grip on the uncertainties and to prevent noise in the results and thereby difficulties in analysis of the results. Therefore it is not necessary to make a set-up which is exactly the same as the design structure, but simple configurations already meet the requirements. This paragraph describes the configuration of the test set-ups and the materials and equipment needed.

Wave basin
Since no wave flume is available at this time the model tests are performed in the large wave basin of the Fluid Mechanics Laboratory of the Delft University of Technology. To allow a proper simulation and a good insight in the processes only a part of the wave basin is used. A flume with a width of 1m is created by constructing a confined relative small area in longitudinal direction through which the waves can propagate, see Figure 5-1. Since the accent of the model tests is to obtain an insight in influence of the gap on the processes unidirectional waves can be used and creating a wave flume will satisfy.
The characteristic length of the wave basin is 30m, including a sloped part and the wave flume is constructed over a length of 9,5m. The toe of the inclined wall is constructed at 16,75m from the wave board. At the end of the wave basin a gentle slope is present which should not be a part of the flume, because it influences the transmitted wave characteristics. The sloping part of the wave basin is starting 4m in front of the inclined wall and therefore a part of the sloped area of the basin has to be removed. Stones are present at the sloping part of the basin acting as wave absorbers. The water depth in the basin is 0,4m.

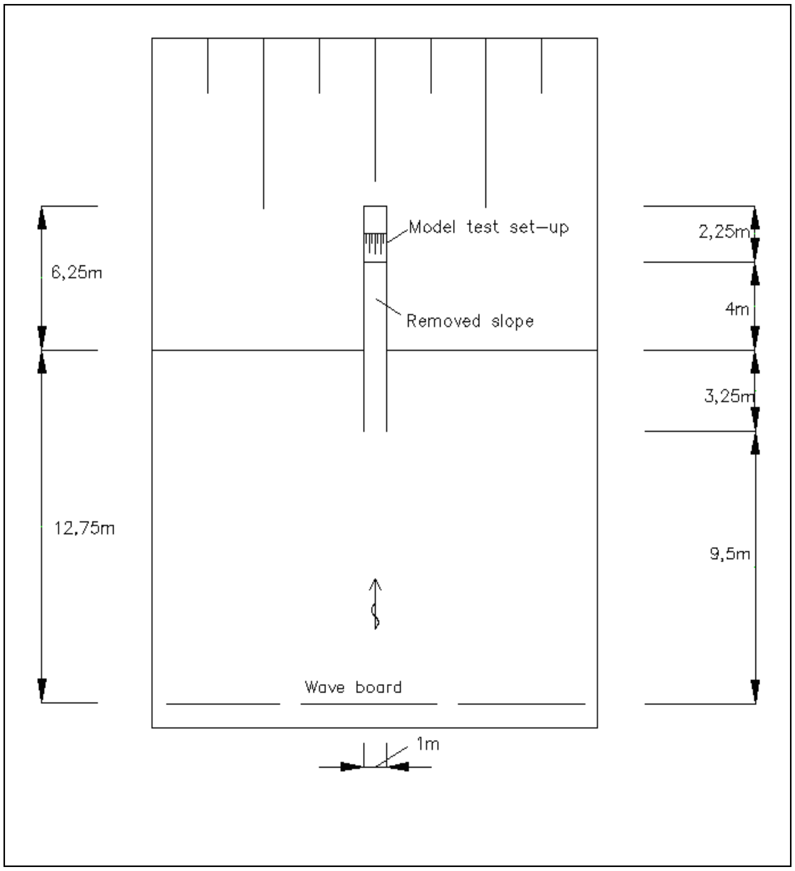


Figure 5-1: Schematized set-up of the model tests in the wave basin (top view) with h=0,4m

Wave board

The wave board can produce regular as well as irregular waves. In the scope of the model tests and results it is good enough to perform tests with regular waves. The waves have to be similar to the design waves in order to get the best results.

A draw-back of the wave board in the wave basin is that the wave board has no correction option for reflected waves. Working with large measurement times can therefore result in unsatisfactory results because the occurrence of standing wave patterns. A possibility to avoid this problem is to perform measurements in relative short time intervals in order to get no wave reflection from the wave board within the measurement time. Therefore the distance from the wave board to the model set-up and period of the produced waves have to be taken into account.

Furthermore, a certain minimum distance (in the order of several meters) from the wave board to the constructed flume has to be taken into account to develop the proper waves.

Set-ups for different tests

For each process an individual model test set-up is thought of to obtain the best results and these are described here. There are seven set-ups required to perform the tests on the different processes. All the model set-ups consist out of an inclined wall with certain characteristics, which is supported by counter forts for the sake of stability and to transfer the horizontal loads from the waves, except for one, which is a vertical wall. Counter forts are used, because they have only limited influence on the transmitted waves, which also should be measured in some cases. The walls that confine the measurement area for the model tests can also serve as counter forts.

Scale rules

When performing model tests in coastal engineering two scale rules are primary used, the scale rule according to Froude and to Reynolds. With the Froude-scale rule, the Froude number in the prototype and the model should be equal. This scale rule is used when both inertia and gravity are dominant. With the Reynolds-scale rule the Reynolds number in the prototype and the model should be equal and the viscosity is dominant over the inertia.

Geometric similarity

With the Froude-scale rule geometric similarity is taken into account. This means that the length scales of the model are presented with the same scale as in the prototype, which results in a so-called undistorted model. For the scale factor or scale ratio N_x the following holds:

$$N_x = \frac{X_p}{X_m} \qquad \text{Equation 5-1}$$

N_x = scale factor for value X
X_p = value of X in prototype
X_m = value of X in model

Kinematic similarity

The kinematic similarity indicates similarity of motion between particles in model and prototype. In geometric similar model, kinematic similarity gives particle paths that are geometrical similar to the prototype.

Geometric similarity with respect to length scales can therefore also be applied to wave length:

$$N_L = \frac{\left[L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi h}{L}\right)\right]_P}{\left[L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi h}{L}\right)\right]_M} \qquad \text{Equation 5-2}$$

in which:

L = wave length [m]
T = wave period [s]
g = gravitational acceleration [m/s^2]
h = water depth [m]

Because the term $2\pi h/L$ is dimensionless, this is the same in the prototype and the model and as a result the hyperbolic tangent also is the same. This results in:

$$N_L = N_g \cdot N_T^{\,2} \qquad \text{Equation 5-3}$$

Because the gravity in the prototype is equal to the gravity in the model and therefore the scale factor is unity, he Froude-time scale for the wave period becomes equal to:

$$N_T = \sqrt{\frac{N_L}{N_g}} = \sqrt{N_L}$$ Equation 5-4

Scaling of hydraulic parameters

When the scale rules presented above are used the hydraulic parameters can be scaled from the prototype to the model. When scaling these parameters the ratios of wave height to water depth (H/h) and water depth to the wave length (h/L) should also be kept equal. When this is satisfied the following wave parameters for the model are obtained:

h = 0,40m
H = 0,15m
T = 1,36s
L = 2,30m

The scaling is based on the scale factor between the water depth in the prototype and the model, N_h=15. For the scaling of the dimensions of the model test set-ups the same scale factor is used in order to comply with geometric similarity of an undistorted model.

Dimensions model test set-ups

In order to come up with the right dimensions for the model test set-ups the dimensions of the prototype have to be known. In the analytical part of the researches these are more or less determined and have to be scaled in order to obtain the dimensions of the models. The dimensions that are analytically derived and known at this moment are presented in Appendix 3.

Material dimensions for model tests

The wave characteristics and the structural parameters are already scaled in the beginning of this paragraph. The average overtopping volumes have to be known to determine the dimensions of the tank, which collects the overtopped water. The tank has to be large enough to store the overtopped water, but not too large in order to can measure the water level inside the tank. Equation 5-5 gives the scale factors for the average overtopping and the wave period.

$$n_q = n_H^{3/2} = (15)^{3/2} \approx 1/58$$
$$n_T = \sqrt{n_H} = \sqrt{15} \approx 3,87$$ Equation 5-5

To determine the dimensions of the tank the overtopped volume of water has to be known. To determine the overtopping volume for a certain wall height also the model width and the measuring time is needed. The model width as mentioned before is assumed to be 1m. The measuring time is assumed to be 25s. This is based on a wave length of about 2,3m; a wave period of 1,36s and a distance from the wave board to the toe of the wall of about 17m. One measuring cycle contains about 15 waves and results in a measuring time of about 25s.

Average overtopping q in m3/s/m:

$$\frac{q}{\sqrt{gH_{m0}^{3}}} = \frac{0,067}{\sqrt{\tan\alpha}}\gamma_b\xi_{m-1,0}\exp(-c_1\frac{R_c}{\xi_{m-1,0}H_{m0}\gamma_b\gamma_f\gamma_\beta\gamma_v})$$ for breaking waves and

with a maximum of: Equation 5-6

$$\frac{q}{\sqrt{gH_{m0}^{3}}} = 0,2\exp(-c_2\frac{R_c}{H_{m0}\gamma_f\gamma_\beta})$$ for non-breaking waves

By means of n_q; the overtopping formula given in Equation 5-6; the measuring time and the model width the overtopping volumes are calculated for different wall heights and are given in Table 5-1.

Table 5-1: Overtopped volumes for a measuring period of 20s and for different wall heights

h_{wall} [m]		q [m3/s/m]		q [l/s/m]		V [l]
prototype	model	prototype	model	prototype	model	model
11	0,73	0,01	1,72*10^-4	10	0,172	4,3
10	0,67	0,02	3,45*10^-4	20	0,345	8,6
9,5	0,63	0,03	5,17*10^-4	30	0,517	12,9
9	0,6	0,06	0,001	60	1,03	25,8
8	0,53	0,19	0,0033	190	3,28	82

It is chosen to take a wall height of the prototype of 15m for the set-ups which exclude overtopping, resulting in a wall height of the model of 1m.

It is chosen to take a wall height of the prototype of 9m for the set-ups which include overtopping, resulting in a wall height of the model of 0,6m. Since it is expected that a gap in the inclined wall would reduce the overtopping the wall must not be too low to avoid a situation that no overtopping occurs when a gap is implemented in the inclined wall.

It is chosen to use gap heights of 0,6m; 1,2m and 1,5m for the prototype instead of 0,5m and 1m resulting in gap heights of 0,04m; 0,08m and 0,1m for the model.

Appendix 3 gives the model dimensions for the different set-ups, just as the dimensions of the plates to be used in the model set-ups.

Set-up 1

This first set-up consists of a vertical wall with a gap at the bottom. With this set-up the water velocity in the gap can be measured. The set-up is schematically shown in Figure 5-2.

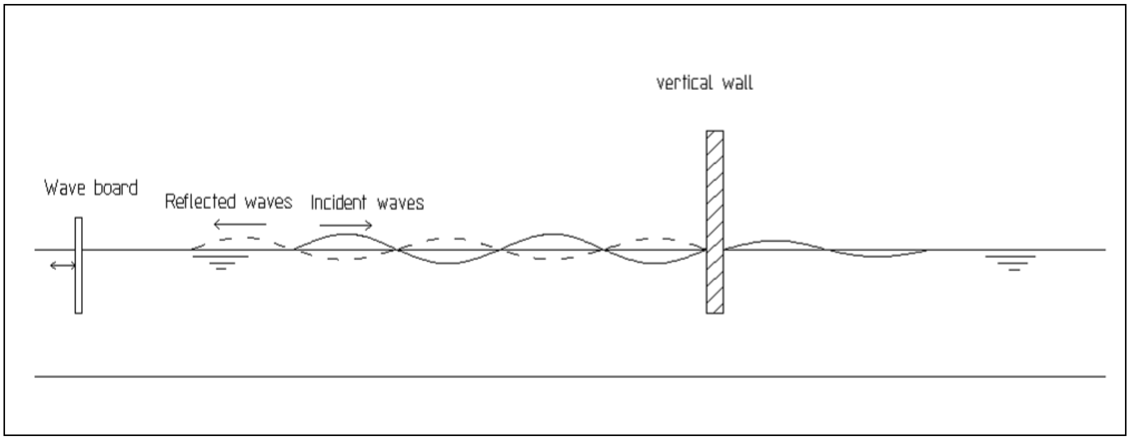


Figure 5-2: Model set-up 1

Set-up 2

This set-up consists of a smooth and straight inclined wall that is long enough to prevent wave overtopping. With this set-up it is intended to measure the wave run-up along the wall.
For this set-up two different slopes are tested, namely 1:1 and 2:1. The set-up is schematically shown in Figure 5-3.

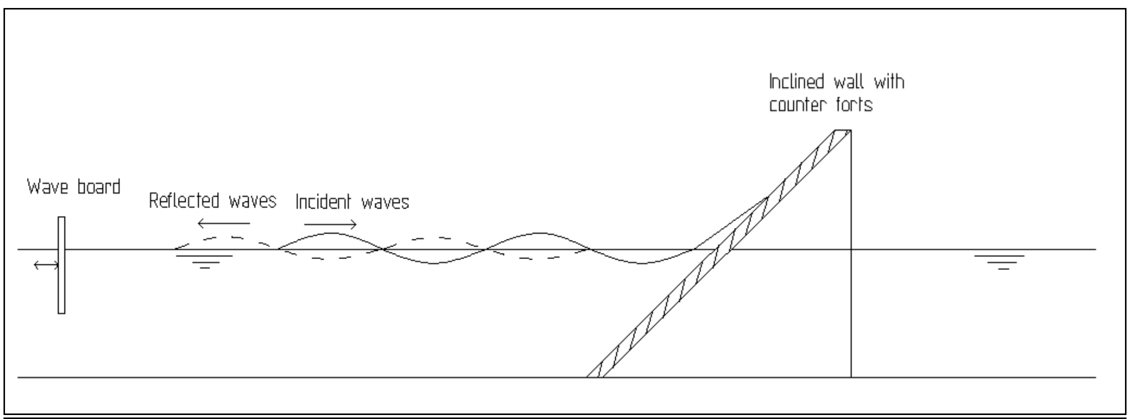


Figure 5-3: Model set-up 2

Table 5-2: Set-up 2

	Test series 1	Test series 2
Angle [^{0}]	45	63,4
Height wall [m]	1	1
Gap height [m]	0	0
Wave period [s]	1,36	1,36
Number of tests [#]	3	3

Set-up 3

This set-up is quite similar to the first set-up. However, a smaller wall height is taken to allow wave overtopping which can be measured by collecting the overtopped water in a tank. For this set-up two different slopes are tested, namely 1:1 and 2:1. The set-up is schematically shown in Figure 5-4.

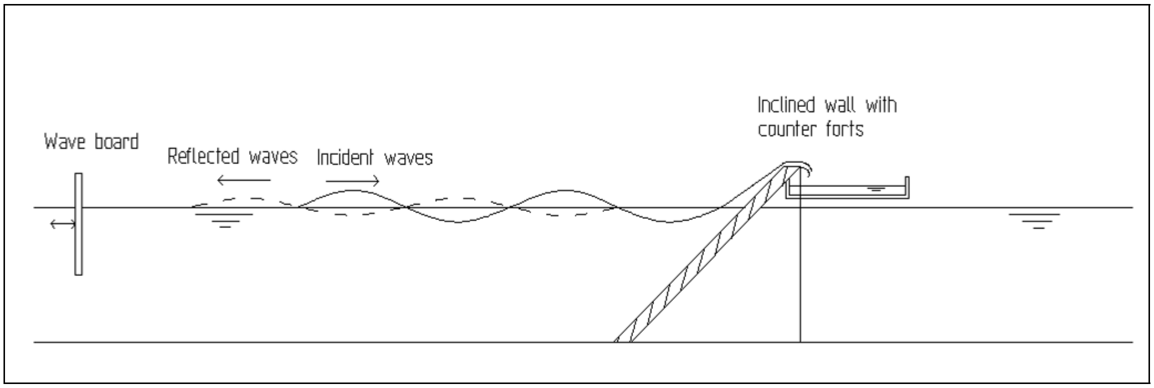


Figure 5-4: Model set-up 3

Table 5-3: Set-up 3

	Test series 1	Test series 2
Angle [^{0}]	45	63,4
Height wall [m]	0,6	0,6
Gap height [m]	0	0
Wave period [s]	1,36	1,36
Number of tests [#]	3	3

Set-up 4

In this set-up a gap at the bottom of the inclined wall is present and the wall is sufficiently long to prevent wave overtopping. In this way the effect of the gap on the wave run-up can be measured. Besides this process also the water velocity in the gap is measured. For this set-up two different slopes are tested, namely 1:1 and 2:1. For the slope 1:1 are besides the design wave also a longer and a shorter wave tested. The set-up is schematically shown in Figure 5-5.

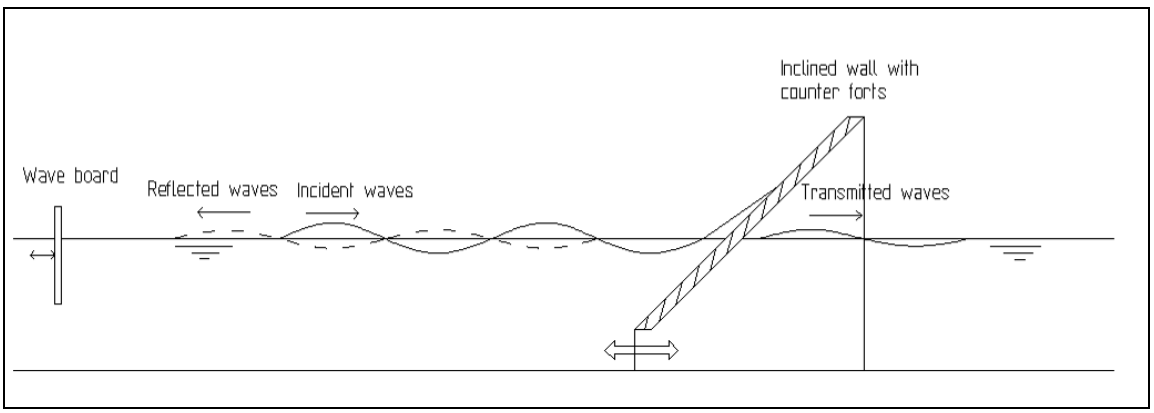


Figure 5-5: Model set-up 4

Table 5-4: Set-up 4

	Test series 1	Test series 2	Test series 3	Test series 4	Test series 5	Test series 6	Test series 7	Test series 8
Angle [^{0}]	45	45	45	45	45	63,4	63,4	63,4
Height wall [m]	1	1	1	1	1	1	1	1
Gap height [m]	0,04	0,08	0,08	0,08	0,10	0,04	0,08	0,10
Wave period [s]	1,36	1,36	2,07	0,77	1,36	1,36	1,36	1,36
Number of tests [#]	3	3	3	3	3	3	3	3

Set-up 5

The wall height in this set-up is limited to allow wave overtopping and to measure the effect of the gap on the wave overtopping process. Again the overtopping water can be collected in a tank to determine the amount of wave overtopping. Besides this process also the water velocity in the gap is measured. For this set-up two different slopes are tested, namely 1:1 and 2:1. The set-up is schematically shown in Figure 5-6.

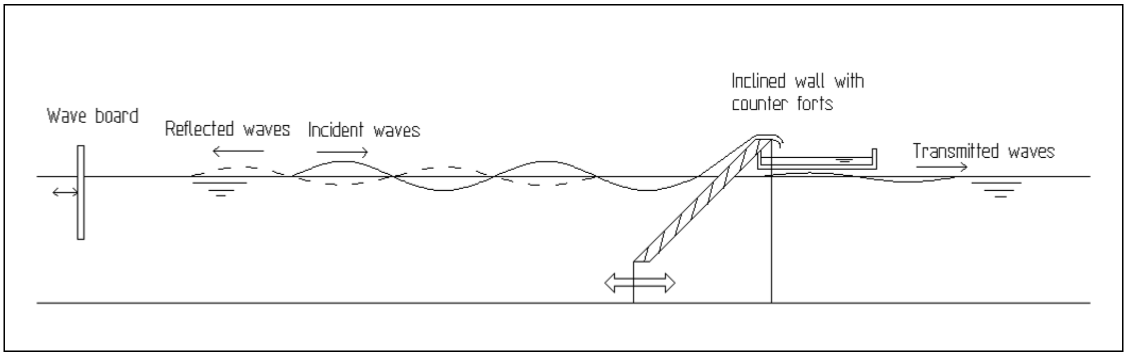


Figure 5-6: Model set-up 5

Table 5-5: Set-up 5

	Test series 1	Test series 2
Angle [^{0}]	45	63,4
Height wall [m]	0,6	0,6
Gap height [m]	0,04	0,04
Wave period [s]	1,36	1,36
Number of tests [#]	3	3

Set-up 6

In this set-up an inclined wall with a gap at the bottom is used and the effect of the outflow of water through the gap on the incident waves can be measured. The combination of gap and outflow of water has a certain effect on the wave run-up process. Besides this process also the velocity in the gap is measured. No wave overtopping is allowed here by the extension of the wall above the maximum wave run-up height. For this set-up one slope is tested, namely 1:1. The set-up is schematically shown in Figure 5-7.

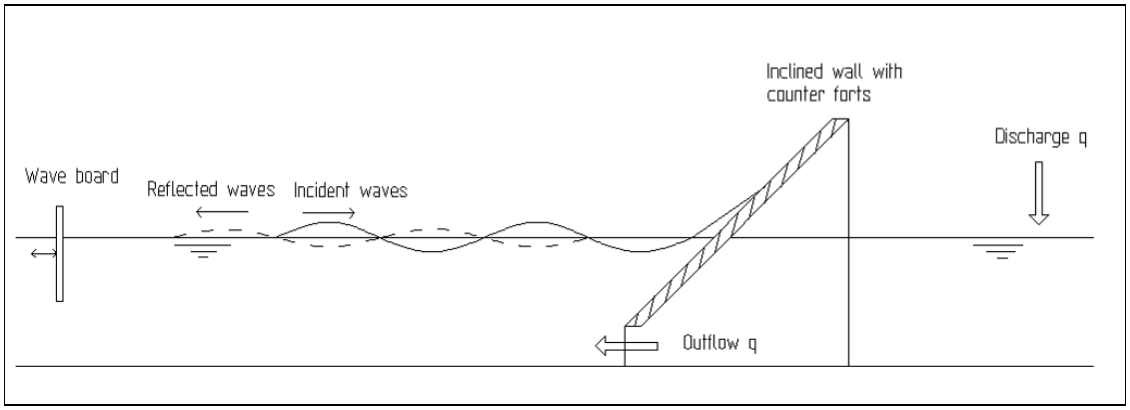


Figure 5-7: Model set-up 6

Table 5-6: Set-up 6

	Test series 1	Test series 2	Test series 3
Angle [^{0}]	45	45	45
Height wall [m]	1	1	1
Gap height [m]	0,04	0,04	0,10
Wave period [s]	1,36	1,36	1,36
Discharge [l/s/m]	1,5	10	10
Number of tests [#]	3	3	3

Set-up 7

In this set-up the effect of the gap and outflow of water on the wave overtopping is measured by a wall height that allows wave overtopping. Besides this process also the water velocity in the gap is measured. The overtopping water is again collected in a tank to measure the volume. For this set-up one slope is tested, namely 1:1. The set-up is schematically shown in Figure 5-8.

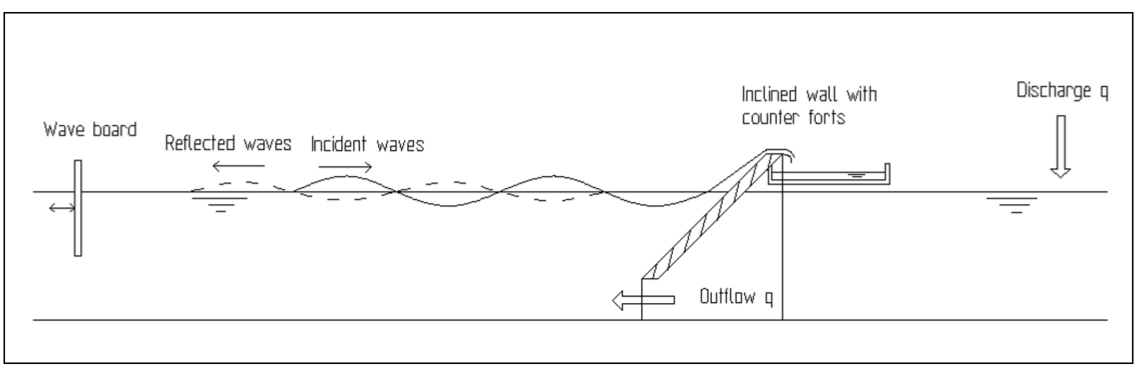


Figure 5-8: Model set-up 7

Table 5-7: Set-up 7

	Test series 1
Angle [^{0}]	45
Height wall [m]	0,6
Gap height [m]	0,04
Wave period [s]	1,36
Discharge [l/s/m]	10
Number of tests [#]	3

Set-up 8

In this set-up a vertical wall behind the inclined wall is constructed in order to test whether resonance occurs for the imposed gap height and hydraulic conditions. Due to the backwall a standing wave is formed in the basin and the occurrence of resonance can be observed just by looking at the wave pattern, or by measuring the standing wave height with the available devices for transmitted waves. Also in this set-up the water velocity in the gap is measured. For this set-up one slope is tested, namely 1:1. The set-up is schematically shown in Figure 5-9.

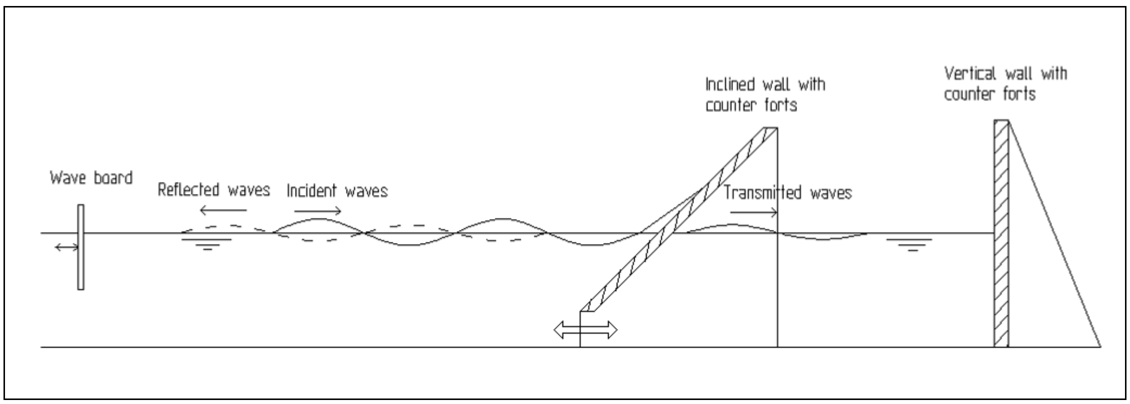


Figure 5-9: Model set-up 8

Table 5-8: Set-up 8

	Test series 1	Test series 2	Test series 3	Test series 4
Angle [^{0}]	45	45	90	90
Height wall [m]	1	1	1	1
Gap height [m]	0,10	0,10	0,10	0,10
Wave period [s]	1,36	1,36	1,36	1,36
Basin width [m]	1,17 (0,5L)	1,75 (0,75L)	1,17	1,75
Discharge [l/s/m]	0	0	0	0
Number of tests [#]	3	3	3	3

In order to get liable results three tests for one test situation have to be performed. In the following paragraphs the term test series is used and this means a test situation for a certain set-up. For each test series three tests are performed with exact the same conditions.

Materials and equipment needed
For each set-up in combination with the process that is tested materials and equipment are needed.

Materials
Overall it can be said that the materials for each set-up are more or less common and exist of a smooth inclined wall with counter forts and the timbering to realise a flume.

Equipment
The instruments to measure the processes are different for each process and are summed up below.

- Wave run-up
  Photographic techniques are performed to record the wave run-up. The wave run-up is recorded on camera and snap shots of the wave run-up are analysed by means of computer software.

- Wave overtopping
  The overtopping water is collected in a tank and by means of the weight of the collected water the volume of the overtopping water is determined.

- Water velocity
  The water velocity in the gap is measured by means of a (EMS) velocity probe.

- Water movement in the basin
  A water level gauge placed in the basin is used to measure the water level fluctuations.

The measurements are digitally converted to ASCI file on a personal computer, except for the photographic and water volume measurements.

Locations
The locations of the measurement equipment have an important influence on the measurement results and therefore have to be determined in advance.

5.5 Results

The results for the wave run-up; wave overtopping; outflow through the gap at the bottom of the inclined wall and the water movement in the basin are treated separately in this paragraph. In this paragraph the results are given, just like an analysis of the results.

5.5.1 Wave run-up

This paragraph gives the results for the wave run-up found by the model tests. Besides an overview of the results also an analysis of these results is given.
First the way of determining the wave run-up is explained briefly below.

Way of determining the wave run-up
Perpendicular to the inclined wall a camera is set up and is recording during the tests. As mentioned before a certain measurement period is used in order to avoid multi reflection by the wave board.
The first waves generated by the wave generator cannot be used for the analysis, because the first waves are not yet fully developed. For this reason the waves 6-10 are used for the run-up analysis. For waves 6-10 of each test snap shots are made of the maximum run-up level. This has been performed with the help of a computer. With the program 'scan_xy' these snapshots are transferred to data representing the wave run-up. Each wave does have different wave run-up points along the meter width of the inclined wall. These points are averaged and this can be done because the run-up is quite horizontal over the width of the inclined wall. Thereafter the wave run-up for the waves 6-10 is

averaged to get ultimately one wave run-up for each test. One can average waves 6-10 because regular waves are used for these tests and therefore each wave should give approximately the same wave run-up. Three tests have been performed for each test series and therefore the wave run-up heights of these three tests are averaged to get one wave run-up height for each test series.

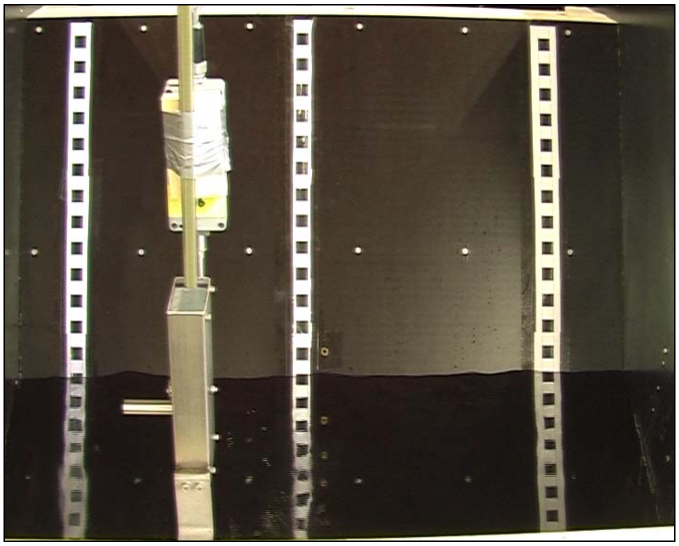


Figure 5-10: Snap shot of wave run-up for set-up 4 test series 7

Results
In paragraph 5.3 'Problem analysis' the main and sub questions for the different processes were given. The main question for the wave run-up was: "What is the influence of the gap in the inclined wall on the wave run-up".

In order to find out the influence of the gap on the wave run-up set-ups 2 and 4 have been used. Set-up 2 is the non- perforated inclined wall and set-up 4 is the perforated inclined wall.
Besides the influence of the gap also the influence of the angle of the slope on the wave run-up has been tested. Both set-ups are tested for the slopes 1:1 and 2:1. The hydraulic boundary conditions in combination with these slopes result in non-breaking waves on the structure. Also the influence of the wave length has been tested. A larger wave should result in a higher wave run-up, because of the larger water volume in the wave. A shorter wave results therefore in a lower wave run-up.

The main question for the vertical backwall was: "What is the influence of a vertical backwall behind the perforated inclined wall on the incoming waves"? In set-up 8 the effect of the vertical backwall on the wave run-up has been tested for a perforated 1:1 sloped wall.

The main question for the outflow was: "What is the influence of the outflow on the incoming waves". In set-up 6 the effect of the outflow through the gap on the wave run-up has been tested for different discharges, for approximately 1,5l/s/m and 10l/s/m. The discharge 1,5l/s/m is based on the overtopping discharge for an inclined wall with a height of 0,6m; see Table 5-1 (1:1 slope and gap height = 0,04m). However it is expected that the influence of this discharge on the incoming waves is almost zero and therefore also a larger discharge is tested to look whether there is some influence on the incoming waves or not.
Table 5-9 gives the wave run-up results for all the test series for the different set-ups. This has also been depicted in Figure 5-11. In Table 5-9 the mean and the standard deviation are given in order to make clear the variation of the measurements.

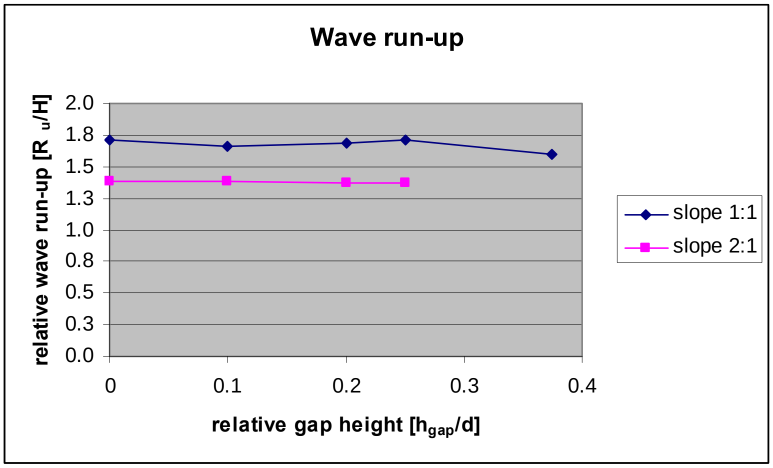


Figure 5-11: Relative wave run-up inclined walls without any discharge for H=0,15m; T=1,36s and d=0,4m

Table 5-9: Wave run-up [m] for inclined walls for H=0,15m; T=1,36s and d=0,4m

Wall angle = 45^0 (1:1)		Wall angle = $63{,}4^0$ (2:1)	
Gap height [m]	Wave run-up [cm]	Gap height [m]	Wave run-up [cm]
0	μ=25,6; σ=1,02	0	μ=20,8; σ=1,00
0,04	μ=24,9; σ=1,15	0,04	μ=20,8; σ=0,72
0,08	μ=25,2; σ=1,68	0,08	μ=20,5; σ=0,65
0,10	μ=25,7; σ=0,38	0,10	μ=20,5; σ=0,82
0,15	μ=23,9; σ=0,91	0,15	-
Wall angle = 45^0 (1:1); gap height = 0,08m		Wall angle = 45^0 (1:1); gap height = 0,10m	
Wave period [s]	Wave run-up [cm]	Basin width [m]	Wave run-up [cm]
1,16	μ=40,0; σ=4,71	0,5L	μ=26,0; σ=0,53
1,36	μ=25,2; σ=1,68	0,75L	μ=25,9; σ=0,80
1,55	μ=29,8; σ=1,28		
Wall angle = 45^0 (1:1); gap height = 0,04m			
Discharge [l/s/m]	Wave run-up [cm]		
0	μ=24,9; σ=1,15		
1,5	μ=26,0; σ=0,88		
10	μ=25,9; σ=1,30		
Wall angle = 45^0 (1:1); gap height = 0,10m			
Discharge [l/s/m]	Wave run-up [cm]		
0	μ=25,7; σ=0,38		
10	μ=26,1; σ=0,91		

Analysis test results:

1. The gap in the inclined wall has no effect on the wave run-up. The wave run-up for a non-perforated inclined wall and for the perforated inclined walls are the same. The wave run-up for a non-perforated 1:1 sloped wall is 26cm and this is also found for the 1:1 sloped perforated walls (25cm; 25cm and 26cm for respectively gap heights of 0,04m; 0,08m and 0,10m)
2. The wave run-up is higher for a slope of 1:1 than for a slope of 2:1. The theory tells that the relative wave run-up increases for an increasing Iribarren number. The Iribarren number for the steeper wall is higher, however the run-up decreases. The Iribarren number for the

given boundary conditions in combination with the 2:1 sloped wall is about 9 and this is outside the area wherein the above assumption is valid.
When the angle is shifting to 90^0 the wave run-up has to decrease to H, namely the amplitude of a standing wave against a vertical wall. Therefore the wave run-up has to decrease to 15cm, when shifting to 90^0.
From set-up 2 it is clear that the 2:1 sloped wall results in less wave run-up (21cm) than the 1:1 sloped wall (26cm).

3. The wave run-up for a longer wave becomes higher, as expected. The wave with a wave period of 1,55 gives a wave run-up of 30cm for an 1:1 slope.
4. The wave with a wave period of 1,16s results in breaking which gives large splashes. Because of these splashes the wave run-up cannot be determined.
5. The wave run-up is not affected by the presence of a vertical backwall behind the perforated inclined wall. The basin width does also not affect the wave run-up.
6. The outflow through the gap has no effect on the wave run-up. An outflow of 1,5l/s/m and an outflow of 10l/s/m through a gap of 0,04m and an outflow of 10l/s/m through a gap of 0,10m in an 1:1 sloped inclined wall do not affect the wave run-up and it remains the same as in the situation without any outflow.
7. Set-up 4 additional is a set-up which consists out of an 1:1 sloped inclined with a gap height of 0,15m. This test is performed after analysing the results of the other tests. It was interesting to find out whether the wave run-up would reduce when the gap height was further increased to 0,15m. One can see that the wave run-up is not significantly reduced by the larger gap; it is reduced from 26cm to 24cm. Therefore it is not possible to can say from this test that the wave run-up is reduced by an increasing gap height.
8. The wave run-up height found by the tests is far smaller than the wave run-up height found by the analytical calculation. However the wave run-up calculated in the analytical research is based on irregular waves and cannot be compared with the results from the tests. To compare the test results the simple Hunt formula for regular waves can be used.

   Hunt: $R_u/H<3$

   The test results still differ largely with the analytical results. A wave run-up of 0,26m is found for the non-perforated 1:1 sloped wall. When one scales this wave run-up a wave run-up for the prototype of 3,9m is found. The formula of Hunt provides a maximum wave run-up of 6,6m, which is a factor 1,7 larger than the tested wave run-up. Ruud Nooij concluded that the pressures found by the model tests were a factor 1,5 smaller than the pressures found by the analytical calculation.

5.5.2 Wave overtopping

This paragraph gives the results for the wave overtopping found by the model tests. Besides an overview of the results also an analysis of these results is given. First the way of determining the wave overtopping is explained briefly below.

Way of determining the wave overtopping

Behind the inclined wall a tank is placed which has to collect the overtopping water. The dimensions of this tank are given in paragraph 5.4 'Experimental set-ups'. The dimensions of the tank are determined so that the overtopping water for both sloped walls (1:1 and 2:1) can be collected in the tank for one measurement period.
Just like for the wave run-up the first waves generated by the wave board cannot be used for the analysis. Waves 6-14 are collected in the tank to determine the wave overtopping discharge. After each test the tank is weighted and the volume of water inside the tank is divided by 9 wave periods to

get the overtopping discharge. For each test series 5 tests are performed instead of 3 for the wave run-up. The reason for this is to obtain more liable results. The discharges of these 5 tests are averaged to get for each test series one overtopping discharge.

Results

In paragraph 5.3 'Problem analysis' the main and sub questions for the different processes were given. The main question for the wave overtopping was: "What is the influence of the gap in the inclined wall on the wave overtopping".

In order to find out the influence of the gap on the wave overtopping set-ups 3 and 5 have been used. Set-up 3 is the non- perforated inclined wall and set-up 5 is the perforated inclined wall. However only a gap of 0,04m has been used, because the expectation was that larger gap heights would result in too much reduction leading to no overtopping.

Besides the influence of the gap also the influence of the angle of the slope on the wave run-up has been tested. Both set-ups are tested for the slopes 1:1 and 2:1, which in combination with the wave conditions both are in the non-breaking area.

The main question for the outflow was: "What is the influence of the outflow on the incoming waves". In set-up 7 the effect of the outflow through the gap on the wave overtopping has been tested for one discharge, namely 10l/s/m.

Table 5-10 gives the wave overtopping results for all the test series for the different set-ups. This has also been depicted in Figure 5-12. In Table 5-10 the mean and the standard deviation are given in order to make clear the variation of the measurements.

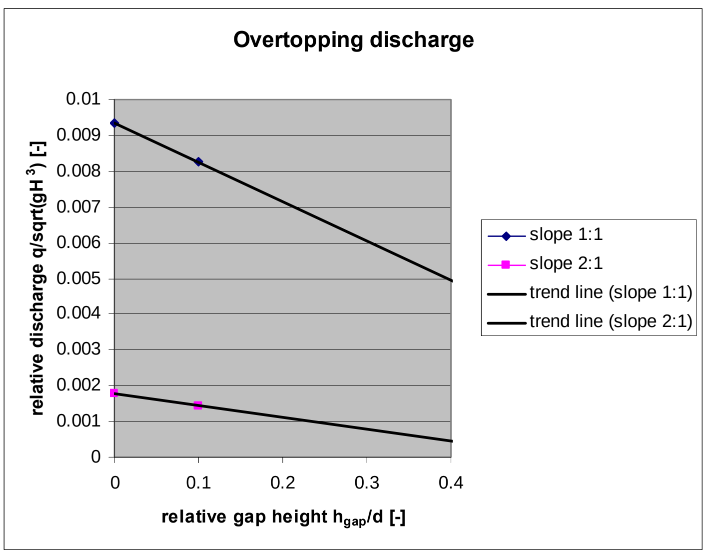


Figure 5-12: Overtopping discharge without discharge through gap for h_{wall}=9m; H=0,15m; T=1,36s and d=0,4m

Table 5-10: Wave overtopping for inclined walls for h_{wall}=9m; H=0,15m; T=1,36s and d=0,4m

Wall angle = 45^0 (1:1)			Wall angle = $63{,}4^0$ (2:1)		
Gap height [m]	Wave overtopping discharge [l/s/m]		Gap height [m]	Wave overtopping discharge [l/s/m]	
	Data of 5 tests	Data excluding first test		Data of 5 tests	Data excluding first test
0	μ=1,74; σ=0,12	μ=1,70; σ=0,10	0	μ=0,31; σ=0,02	μ=0,32; σ=0,02
0,04	μ=1,44; σ=0,10	μ=1,49; σ=0,06	0,04	μ=0,24; σ=0,04	μ=0,26; σ=0,02

Wall angle = 45^0 (1:1); gap height = 0,04m		
Discharge [l/s/m]	Wave overtopping discharge [l/s/m]	
	Data of 5 tests	Data excluding first test
0	μ=1,44; σ=0,10	μ=1,49; σ=0,06
10	μ=1,56; σ=0,09	μ=1,57; σ=0,10

Analysis test results:

1. The gap in the inclined wall has an effect on the wave overtopping. The wave overtopping for a non-perforated inclined wall and for the perforated inclined wall are different. The wave overtopping for a non-perforated 1:1 sloped wall is 1,7l/s/m and the wave overtopping for a 1:1 sloped perforated wall is 1,4l/s/m (for a gap height of 0,04m).
   The wave overtopping for a non-perforated 2:1 sloped wall is 0,3l/s/m and the wave overtopping for a 2:1 sloped perforated wall is 0,25l/s/m (for a gap height of 0,04m).
   Both sloped walls do have a reduction of about 20% in wave overtopping when a gap of 0,04m in the inclined is situated. When one assumes a linear relation between the overtopping discharge q and the gap height, Figure 5-12 is the result.
2. The wave overtopping is larger for a slope of 1:1 than for a slope of 2:1. This is in line with the wave run-up, which is explained in paragraph 7.6.1. From set-ups 3 and 5 it is clear that the 2:1 sloped wall results in less wave overtopping (0,3l/s/m) than the 1:1 sloped wall (1,7l/s/m).
3. From set-up 7 it becomes clear that the wave overtopping increases 8% by a discharge of 10l/s/m through the gap in the inclined wall compared to the situation without any outflow through the gap, namely set-up 5.

5.5.3 Outflow

This paragraph gives the results for the outflow through the gap in the inclined/vertical wall found by the model tests. Besides an overview of the results also an analysis of these results is given. First the way of determining the outflow through the gap is explained briefly below.

Way of determining the outflow through the gap in the inclined/vertical wall
The velocity in the gap at the bottom of the inclined wall has been measured by means of an EMS velocity probe. The velocity probe has been placed for all the different set-ups at half the gap height. The output of this probe is in volts and the computer converts this to a velocity.

Results
In paragraph 5.3 'Problem analysis' the main and sub questions for the different processes were given. The main question for the outflow through the gap was: "What is the influence of the outflow on the incoming waves"? In paragraph 5.5.1 and 5.5.2 the influence of the outflow on respectively the wave run-up and wave overtopping has been described.
The influence of the outflow on these two processes is therefore known, but it is also interesting to find out what happens in the gap for the following situations:

- The influence of the angle of the wall on the velocity in the gap at the bottom of the inclined wall. The difference in velocity between walls of 90^0; $63{,}4^0$ and 45^0.
- The influence of the gap height on the velocity in the gap at the bottom of the inclined wall.
- The influence of overtopping on the velocity in the gap at the bottom of the inclined wall.
- The influence of the outflow on the velocity in the gap at the bottom of the inclined wall.

Nothing can be said about the velocity profile in the gap, because the velocity is measured at only one place in the gap. Therefore in the following figures the surface area of the positive amplitude does not have to be equal to the surface area of the negative amplitude of the water velocity. Thus the following plots represent the water velocity as function of the time at one place in the gap.

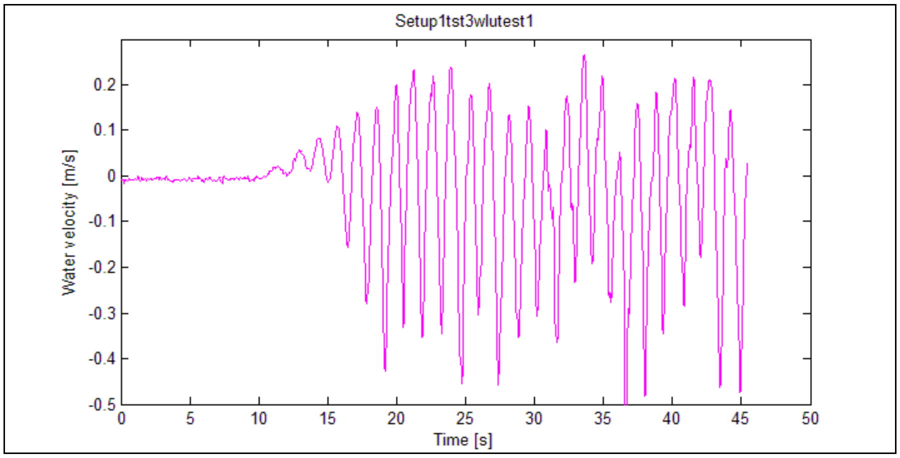


Figure 5-13: Water velocity in gap for vertical wall with h_{gap}=0,10m; H=0,15m; T=1,36s and d=0,4m

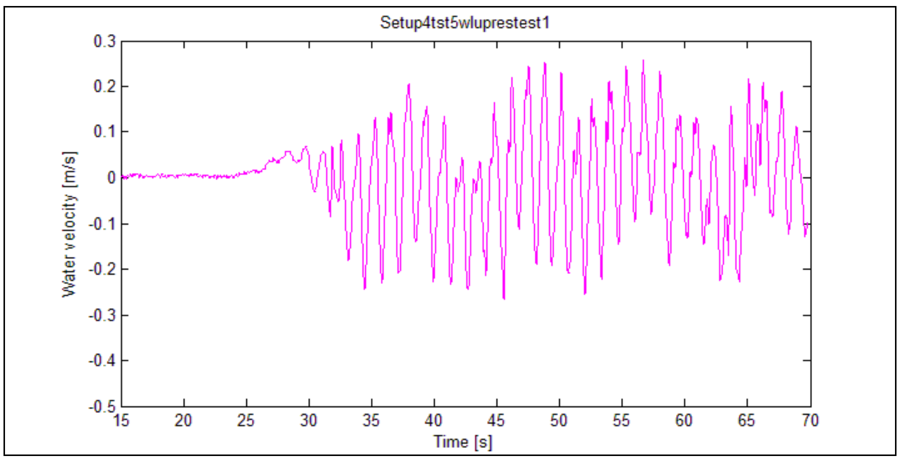


Figure 5-14: Water velocity in gap for 1:1 sloped wall with h_{gap}=0,10m; H=0,15m; T=1,36s and d=0,4m

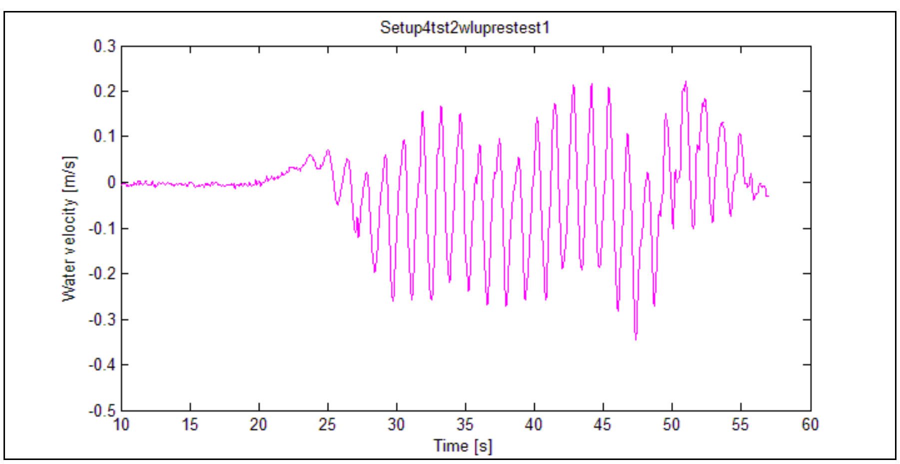


Figure 5-15: Water velocity in gap for 1:1 sloped wall with h_{gap}=0,08m; H=0,15m; T=1,36s and d=0,4m

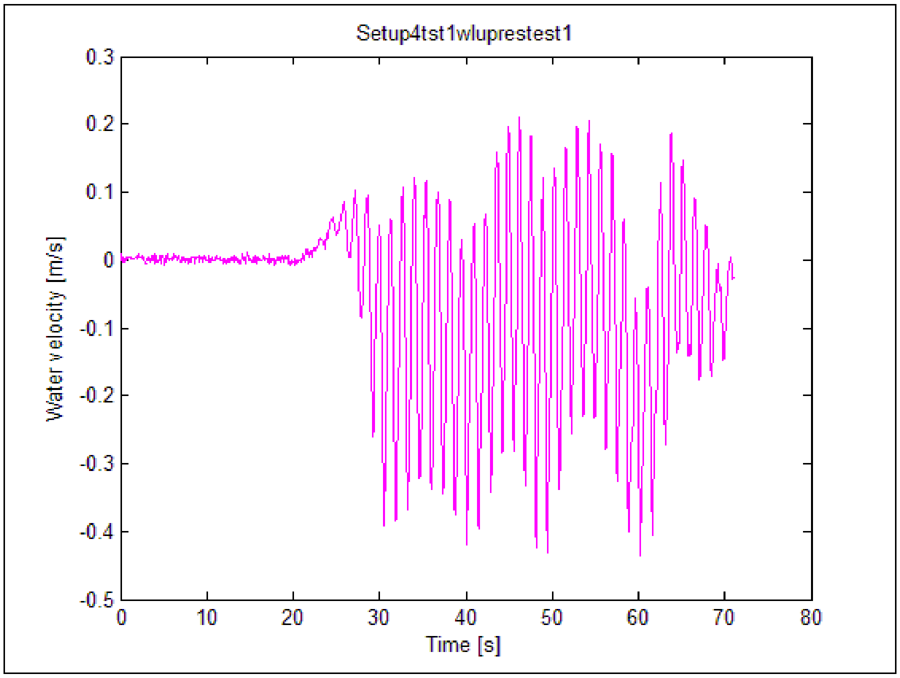


Figure 5-16: Water velocity in gap for 1:1 sloped wall with h_{gap}=0,04m; H=0,15m; T=1,36s and d=0,4m

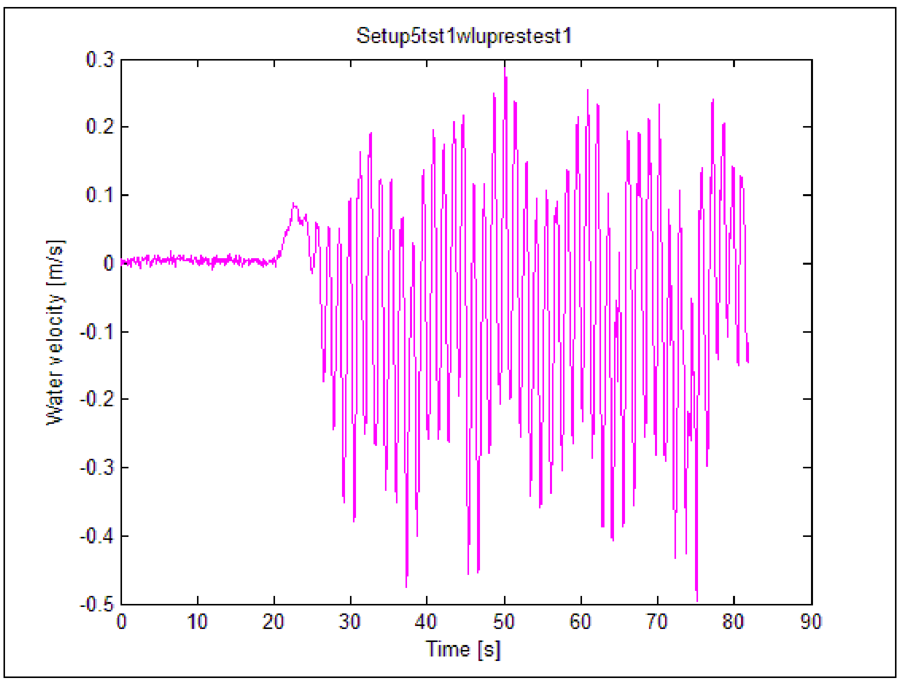


Figure 5-17: Water velocity in gap for 1:1 sloped wall when overtopping is present with h_{gap}=0,04m; H=0,15m; T=1,36s and d=0,4m

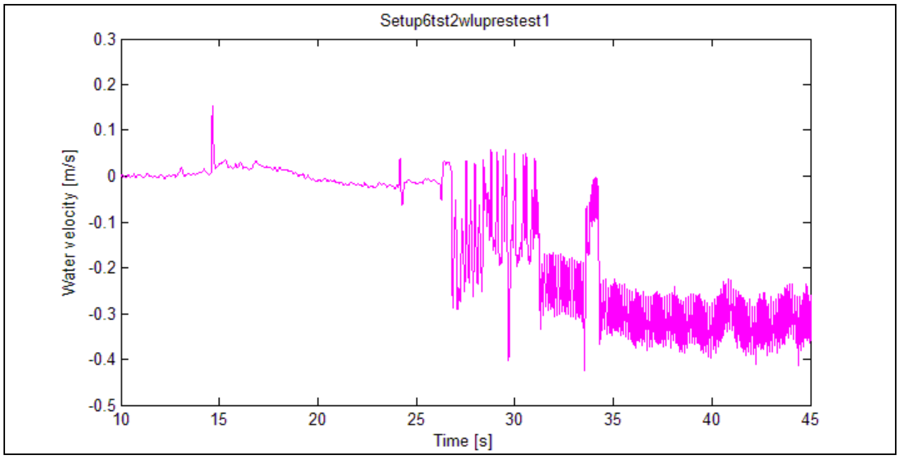


Figure 5-18: Water velocity in gap for 1:1 sloped wall with h_{gap}=0,04m; q=10l/s/m; H=0,15m; T=1,36s and d=0,4m

Analysis test results:

1. The negative amplitude of the velocity, representing the velocity pointing in opposite direction in relation to the incoming waves, decreases when the angle becomes smaller than 90^0. This is in line with the theory, because for a decreasing angle the reflection coefficient also decreases. When the wave height decreases the velocity also decreases, since the velocity is a function of the wave height (linear wave theory). This can be concluded from Figure 5-13 and Figure 5-14.
2. It can be seen that the negative amplitude of the velocity decreases when the gap height increases. This can also be explained by the fact that the reflection coefficient decreases for increasing gap height. This can be concluded from Figure 5-14; Figure 5-15 and Figure 5-16.
3. The velocity in the gap seems not to be influenced by the fact whether overtopping over the inclined wall is present or not. The negative amplitude remains the same for both situations. The positive amplitude seems a little bit larger for the case with overtopping. Perhaps this can be explained by the decrease of resistance due to the decreasing wave run-down as a result of the wave overtopping. This can be concluded from Figure 5-16 and Figure 5-17.
4. When a discharge through the gap is present the average shifts due to the presence of this discharge. This can be seen from Figure 5-18. The velocity shifts from 0 to -0,3m/s due to the discharge through the gap, excluding orbital motion. The discharge does have an influence on the velocity amplitudes, which decrease when a discharge through the gap is present. However, this influence cannot be found from Figure 5-16 and Figure 5-18. From this figure only the shift of the average is clearly visible, which cannot be said about the orbital motion.

5.5.4 Water movement in basin

This paragraph gives the results for the water movement in the basin found by the model tests. Besides an overview of the results also an analysis of these results is given.
First the way of determining the water movement in the basin is explained briefly below.

Way of determining the water movement in the basin

The water movement in the basin is measured by a water level gauge, which is placed inside the basin just in front of the backwall. This water level gauge is connected to the computer and on this computer the volt output is transferred to a water level fluctuation. Besides the digital output also visual measurements have been performed. It is easy to observe whether resonance is occurring or not. The water movement in the basin is therefore also recorded on camera.

Results

In paragraph 5.3 'Problem analysis' the main and sub questions for the different processes were given. The main question for the water movement in the basin was: "What is the influence of the gap and the inclined wall on the water movement in the basin (whether resonance is occurring or not)".

The expectation was that the water movement of the configuration consisting out of a perforated inclined wall in front of a vertical backwall is acting the same as for the configuration consisting out of two vertical walls.

This means that resonance is occurring for a basin width of 0,5L and multiples of this length and that for a basin width of 0,75L almost no water fluctuation is present in the basin. In Set-up 8 this has been tested for only the 1:1 sloped inclined wall. Also the rectangular basin has been tested, which consists out of two vertical walls. For both configurations two basin widths has been tested, namely 0,5L and 0.75L. The results obtained by the water level gauges are presented in figures 7-19-7-22. The visual measurements recorded on camera give the same results.

In all the test series of set-up 8 a gap height of 0,10m has been used. The reference level of the basin width is still water level.

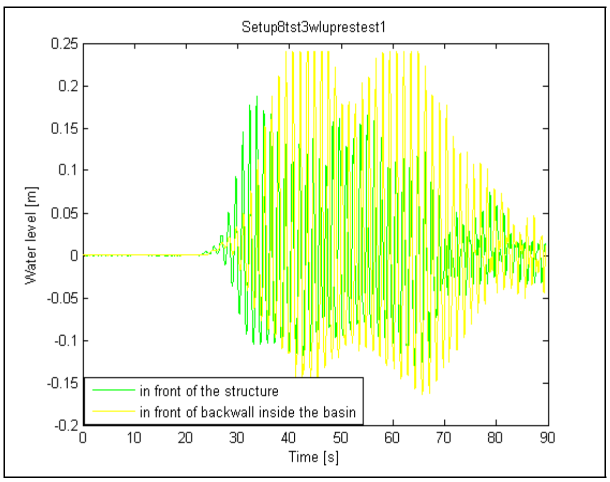


Figure 5-19: Water movement for a configuration consisting out of perforated vertical wall in front of a vertical backwall and a basin width of 0,5L for H=0,15m; T=1,36s and d=0,4m

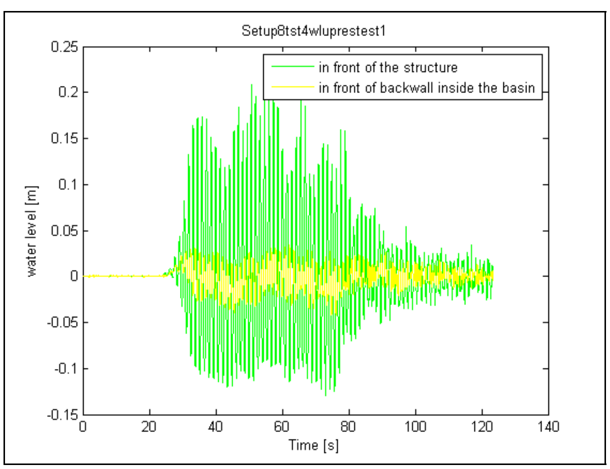


Figure 5-20: Water movement for a configuration consisting out of perforated vertical wall in front of a vertical backwall and a basin width of 0,75L for H=0,15m; T=1,36s and d=0,4m

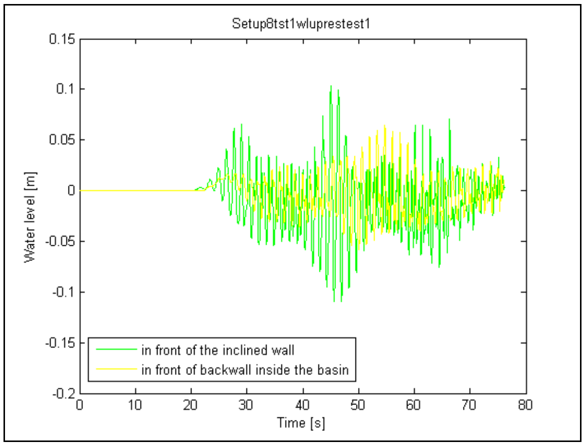


Figure 5-21: Water movement for a configuration consisting out of a 1:1 sloped perforated inclined wall in front of a vertical backwall and a basin width of 0,5L for H=0,15m; T=1,36s and d=0,4m

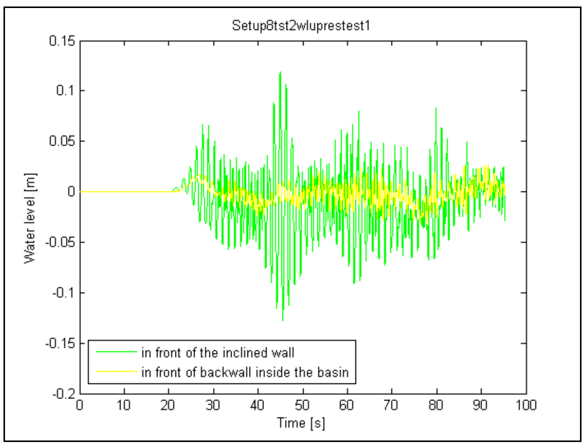


Figure 5-22: Water movement for a configuration consisting out of a 1:1 sloped perforated inclined wall in front of a vertical backwall and a basin width of 0,75L for H=0,15m; T=1,36s and d=0,4m

Analysis tests results:

1. The results of test series 3 and 4, which represent the rectangular configuration, confirm the theory of paragraph 4.9.2 about resonance in a basin. One can see from Figure 5-19 that resonance is occurring in a rectangular basin for a width of 0,5L. In addition, the water level gauge was not able to record the top of the water level fluctuations and the water level is therefore actual higher, see Figure 5-19. From Figure 5-20 almost no water level fluctuation is visible for a basin width of 0,75L.
2. From figures 113 and 114 it can be seen that the expectation about the water movement in the basin behind an 1:1 sloped perforated inclined wall was wrong. The water movement does not act the same as in the case for a configuration consisting out of two vertical walls. There is no resonance occurring for a basin width of 0,75L, but neither for 0,5L what was expected. However, it can be seen from Figure 5-21 and Figure 5-22 that the water movement in the basin is larger in the case with a basin width of 0,5L than for a basin width of 0,75L.

5.6 Conclusions of physical model testing

In this paragraph first the conclusions for the separate processes are given, which were found by the model tests. Then conclusions and possible explanations for the processes combined are given.

5.6.1 Conclusions for the separate hydraulic processes

Influence of gap height on the wave run-up

The wave run-up is not decreasing by the presence of a gap at the bottom of an inclined wall. When the gap height is increased, it is not noticeable in the wave run-up.

Influence of the gap on the wave overtopping

The wave overtopping is significantly reduced by the presence of a gap. A gap of 0,04m reduces the wave overtopping with 20%. This is the case for both the 1:1 and 2:1 sloped walls.

Influence of a vertical backwall on the incoming waves

From the tests it can be concluded that the wave run-up is not influenced by the presence of a vertical backwall. The influence of a vertical backwall on the wave overtopping has not been tested.

Influence of an outflow on the incoming waves

From the tests it can be concluded that the presence of an outflow through the gap at the bottom of the inclined wall has no effect on the wave run-up.
However, the wave overtopping is increased by the presence of an outflow. For an outflow of 10l/s/m through a gap of 0,04m the wave overtopping increases with 8%.

Influence of gap and inclined wall on water movement in the basin

The configuration which consists of a perforated inclined wall in front of a vertical backwall does not act the same as a configuration consisting out of a perforated vertical wall in front of a vertical backwall when the water movement is considered. There is no resonance occurring for a basin width of 0,5L and neither for a width of 0,75L which was expected.

5.6.2 Conclusions and possible explanations for the combined hydraulic processes

An explanation for the phenomenon that the wave run-up is not influenced by the presence of a gap and the wave overtopping is influenced by the presence of a gap could be that the wave run-up height remains the same, but that the wave run-up tongue becomes thinner. This would result in less overtopping. However, this should be visible in the pressures measured along the wall, which is not the case. The pressures are analyzed in the research of Ruud Nooij and were tested simultaneously with the model tests performed for this research. Ruud Nooij found that the pressures are not influenced by the presence of a gap at the bottom in the inclined wall.
Therefore it is assumed that the overtopping time decreases when a gap at the bottom of the inclined wall is implemented. The wave run-up height remains the same just like the thickness of the tongue, but the wave overtopping time is shorter. It is not possible to verify this assumption by means of the obtained data.

From the model tests it can be concluded that the presence of a backwall, indicating a basin is present behind the perforated inclined wall, is not influencing the incoming waves at all. The same can be concluded for the presence of an outflow through the gap at the bottom of the inclined wall. The discharge 10l/s/m was increasing the wave overtopping with 8%, however this discharge was more than 5 times larger then the actual discharge occurring. The actual wave overtopping discharge for a wall with a height of 0,6m was 1,5l/s/m. However, it was expected that the outflow would had a reducing effect on the wave overtopping, but this is not the case. The discharge of 10l/s/m is increasing the wave overtopping. The outflow has no influence on the wave run-up at all.
For the design no reducing effect by an outflow on the wave run-up and the wave overtopping has to be taken into account. The gap can fulfil its main function, namely emptying the basin, without affecting the wave run-up and wave overtopping.

The model tests show that the inclined wall largely affects the water movement in the basin. For a basin width of 0,5L resonance is not occurring, neither for 0,75L. It is expected that the inclined wall is the reason the standing wave cannot develop in the basin. The reference level for the basin width in the model tests is still water level. Perhaps it would be better to take half the gap height as reference level, since the water movement is excited through the gap.
From the model tests one can conclude that in the design a basin width of 0,5L can be used, knowing resonance is not occurring. The water movement is only tested for a 1:1 sloped wall and this has to be kept in mind.

Chapter 6 Comparison of analytical results and model testing results

6.1 Introduction

In Chapter 5 'Physical model testing' the encountered uncertainties in the analytical research have tried to been solved. Besides solving the encountered uncertainties verifying the analytical results was also an objective of the model testing. In this chapter the results of the model testing are compared with the results obtained by the analytical research. The results are compared mainly qualitative, however for some processes also a quantitative comparison is performed. The model testing was based on the understanding of the separate processes and the comparison of the analytical results and the results of the model testing is therefore also based on the hydraulic processes. In this chapter the hydraulic processes are described separately. In paragraph 6.2 the comparison is performed for all processes in a qualitative way and where necessary also in a quantitative way.
In Chapter 7 'A numerical model' a numerical model is constructed to calculate the overtopping over the vertical backwall for a certain configuration. The model has been constructed based on the analytical results and the results obtained by the model testing.

6.2 Comparison results for the separate hydraulic processes

In this paragraph the comparison is performed for the separate hydraulic processes. The schematization steps followed in the model testing are also the basis for the comparison of the analytical results with the results of the model testing.

6.2.1 Wave run-up

Non-perforated inclined wall
The wave run-up height found by the tests is far smaller than the wave run-up height found by the analytical calculation. However the wave run-up calculated in the analytical research is based on irregular waves and can not be compared with the results from the tests. To compare the test results the simple Hunt formula for regular waves can be used.

Hunt: $R_u/H<3$

The test results still differ largely with the analytical results. A wave run-up of 0,26m is found for the non-perforated 1:1 sloped wall. When one scales this wave run-up a wave run-up for the prototype of 3,9m is found. The formula of Hunt provides a maximum wave run-up of 6,6m, which is a factor 1,7 larger than the tested wave run-up.

Perforated inclined wall
In the analytical part of the report it is stated that the wave run-up is influenced by the presence of the gap at the bottom of the inclined wall. A reduction factor has been derived to take the reduction of the gap on the wave run-up into account. However, from the model testing it becomes clear that the gap at the bottom of the inclined wall does not influence the wave run-up. In the numerical model no reduction for the wave run-un is taken into account.

Perforated inclined wall in front of a vertical backwall

In the analytical part it has not been analyzed whether the vertical backwall influences the wave run-up or not. From the model testing it became clear the vertical backwall does not influence the wave run-up.

Perforated inclined wall in front of a backwall, when a discharge through the gap is present

The discharge through the gap represents the outflow, which is induced by the head difference between the inside and outside water level. In the analytical part it has also not been analyzed whether an outflow influences the wave run-up or not. From the model testing it can be concluded that the presence of an outflow through the gap at the bottom of the inclined wall has no effect on the wave run-up.

6.2.2 Wave overtopping

Non-perforated inclined wall

The wave overtopping found by the model testing is far larger than the wave overtopping found by the analytical calculation. However the overtopping calculated in the analytical research is based on irregular waves and cannot be compared with the results from the tests.

Perforated inclined wall

In the analytical part of the report it is stated that the wave overtopping is influenced by the presence of the gap at the bottom of the inclined wall. A reduction factor has been derived to take the reduction of the gap on the wave overtopping into account. This reduction factor is the same for the wave run-up.

From the model testing it also becomes clear that the gap at the bottom of the inclined wall does influence the wave overtopping. A gap of 0,04m reduces the wave overtopping with about 20%, which means a reduction factor of 0,8. This is the case for both the 1:1 and 2:1 sloped walls. The reduction factor obtained by the analytical research for a gap of 0,6m, which represents 0,04m in the model set-up, is approximately 0,75 according to Figure 4-29.

Therefore reduction for the wave overtopping in the numerical model is taken into account. The reduction factor is based on the results of the model testing. For determining the factor Figure 5-12 is used. For small gaps the reduction factor of the analytical results is quite similar to the reduction factor obtained by Figure 5-12. However, for an increasing gap height the difference between the analytical results and the results from the model testing becomes larger. For those cases the results from the model testing are used, so Figure 5-12 is used to determine the reduction factor for the influence of the gap on the wave overtopping. The reduction factor is given in Figure 6-1.

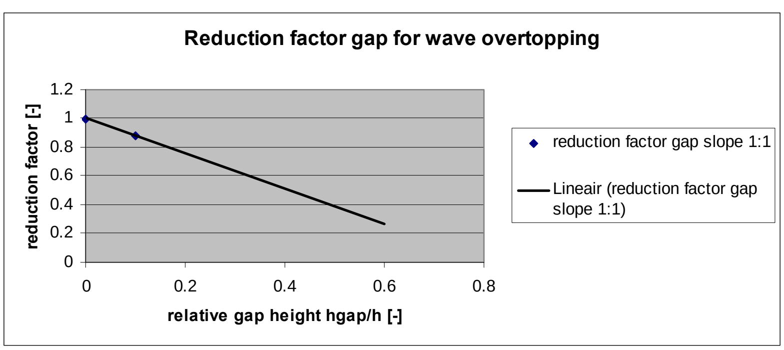


Figure 6-1: Reduction factor influence gap on wave overtopping for R_c=3m; α=45^0; H_{m0}=2,2m and $T_{m-1,0}$=5,25s

In the analytical research also an upper and a lower limit were given, where in between the overtopping over the perforated inclined is lying. One can verify these limits by means of the results obtained by the model testing. The height of the inclined wall of the overtopping set-ups was 0,6m.

The relative crest height R_c/H is therefore 1,33. The overtopping resulting from the model testing for 1:1 sloped perforated wall is in between 1*10^-2 and 5*10^-3, see Figure 5-12. This is valid for a relative crest height of 1,33. One can see that this is in the range given by Figure 4-33. The results are lying closer to the non-perforated inclined wall than to the permeable rubble mound structure. This seems logic, because the permeable rubble mound structure includes a lot of extra resistance. However, it can not be concluded that the overtopping over a perforated inclined wall is always closer to the non-perforated inclined wall than to the rubble mound structure, because only one point (relative crest height) is compared.

Perforated inclined wall in front of a vertical backwall
In the analytical part it has not been analyzed whether the vertical backwall influences the wave run-up or not, neither it has been analyzed in the model testing.

Perforated inclined wall in front of a backwall, when a discharge through the gap is present
It the analytical part it has also not been analyzed whether an outflow influences the wave overtopping or not. From the model testing it has become clear that the outflow 1l/s/m, which were based on the overtopping discharge, does not influence the wave overtopping. However, when the outflow was increased significantly to 10l/s/m an increase of 8% in wave overtopping was the result.

6.2.3 Water movement

Perforated vertical wall in front of vertical backwall
In the analytical research it is determined that resonance of the water level fluctuation in the basin occurs for basin widths of n*(0,5L_{wave}) with n=1,2,3etc. The same is found by the model testing. In the analytical research it is also determined that damping of the water level fluctuation in the basin occurs for basin widths of n*(0,75L_{wave}) with n=1,2,3etc. The same is also found by the model testing.

Perforated inclined wall in front of vertical backwall
In the analytical research it is stated that the water movement in a basin consisting of an inclined wall in front of a vertical backwall and the water movement in a rectangular basin are the acting the same. Therefore resonance of the water level in the basin of the reference design is excited for basin widths of n*(0,5L_{wave}) with n=1,2,3etc. However, from the model testing it has to be concluded that resonance is not occurring for basin widths of n*(0,5L_{wave}) with n=1,2,3etc. Because the water movement of the reference design acts the same as the water movement in the rectangular basin as stated in the analytical research damping of the water level fluctuation should occur for basin widths equal to n*(0,75L_{wave}) with n=1,2,3etc. The same is also found by the model testing.
The reference level for the basin width in the model tests is still water level. Perhaps it would be better to take half the gap height as reference level, since the water movement is excited through the gap.

6.3 Uncertainties

In this paragraph the uncertainties are described which are not solved by means of the analytical research and the physical model testing.

Wave run-up and wave overtopping
It is uncertain why the wave run-up height found by the tests is far smaller (factor 1,7) than the wave run-up height found by the analytical calculation.
It remains uncertain why the wave run-up is not influenced by the gap and the wave overtopping is influenced by the gap. As already said in paragraph 5.6.2 the overtopping time could be the reason for this phenomenon. However, this assumption can not be verified by the obtained data and therefore this remains an uncertainty.

Water movement

It is not known for which basin width the water movement in the reference design is resonating. The reference level for the basin width in the model tests is still water level. Perhaps it would be better to take half the gap height as reference level, since the water movement is excited through the gap.

So it is not known whether the inclined wall is the reason that no resonance is occurring or the wrong chosen reference level is the reason that resonance is not occurring.

Chapter 7 A numerical model

7.1 Introduction

In Chapter 4 the occurring processes have been described separately for simplifications of the reference design and for the reference design. However, in reality the processes are linked and do influence each other. In this chapter the processes are combined and a numerical model is constructed to check possible configurations, whether the water is overtopping the backwall during a storm or not.
In paragraph 7.2 'Numerical model' the numerical model is described, which calculates the overtopping over the vertical backwall during a storm, just as the overtopping over the perforated inclined wall and the outflow through the gap due to head difference.
In paragraph 7.3 'Optimization of the configuration' an optimization of the configuration of the superstructure by means of the numerical model is performed. The overtopping over the backwall is for a number of configurations calculated by the model.

7.2 Numerical model

In this paragraph a numerical model is created, which is based on the numerical model of the Crest Drainage Dike (CDD) [Van Steeg, 2007]. The numerical model for the CDD has been used, because of resemblance of the configuration and the occurring processes of the CDD with the reference design. In paragraph 7.2.1 the model framework for the reference design is described excluding any drainage of the basin. The emptying process of the basin is included in the model in paragraph 7.2.2. In both paragraphs the model is described by one single wave. However, during a storm the structure is exposed to a wave field and attention needs to be paid to this. The model is adapted for the wave field in paragraph 7.2.3.

In the numerical model the following processes are included:

- Wave run-up
- Wave overtopping
- Outflow through the gap at the bottom of the inclined wall due to head difference between the inside and outside water level
- Length spreading effect
- Overtopping over the vertical backwall

In the numerical model the following processes are excluded:

- Wave induced inflow
- Water movement in the basin
- Wave grouping

As mentioned above the model is based on the numerical model of the Crest Drainage Dike. The differences between the CDD and the reference design of the New Orleans storm surge barrier are:

- The geometry of the superstructure; the CDD only has a basin in the crest, which is empty in the initial state. The basin of the reference design is bounded by an inclined wall and a vertical wall in stead of two vertical walls for the CDD configuration.
- The draining system; the emptying process of the reference design is based on outflow through the gap at the bottom of the inclined wall and the emptying process of the CDD is based on drainage by means of drains. However, the emptying process for both models is based on head difference.

7.2.1 Model framework

In this paragraph the set-up of the numerical model is described, just as the processes which are taken into account.

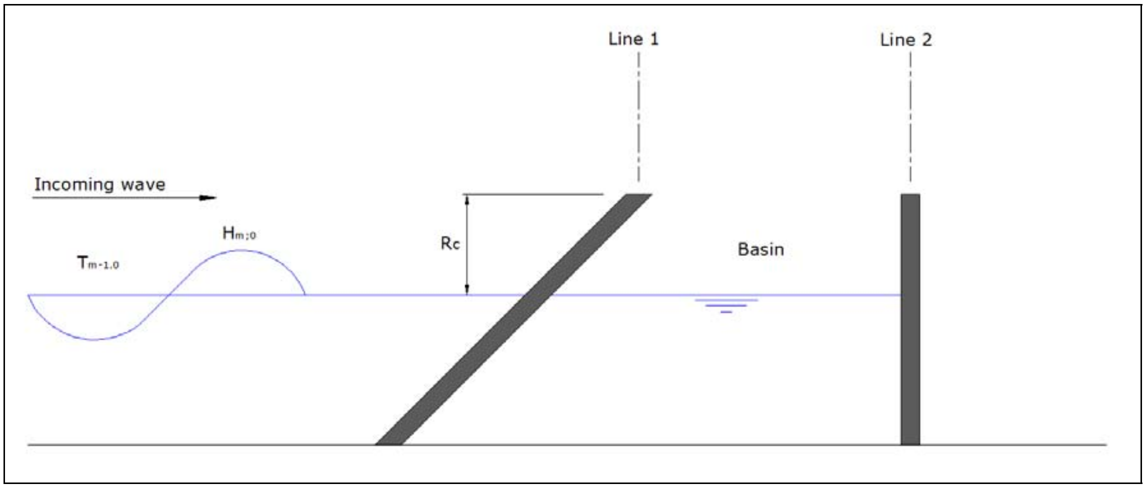
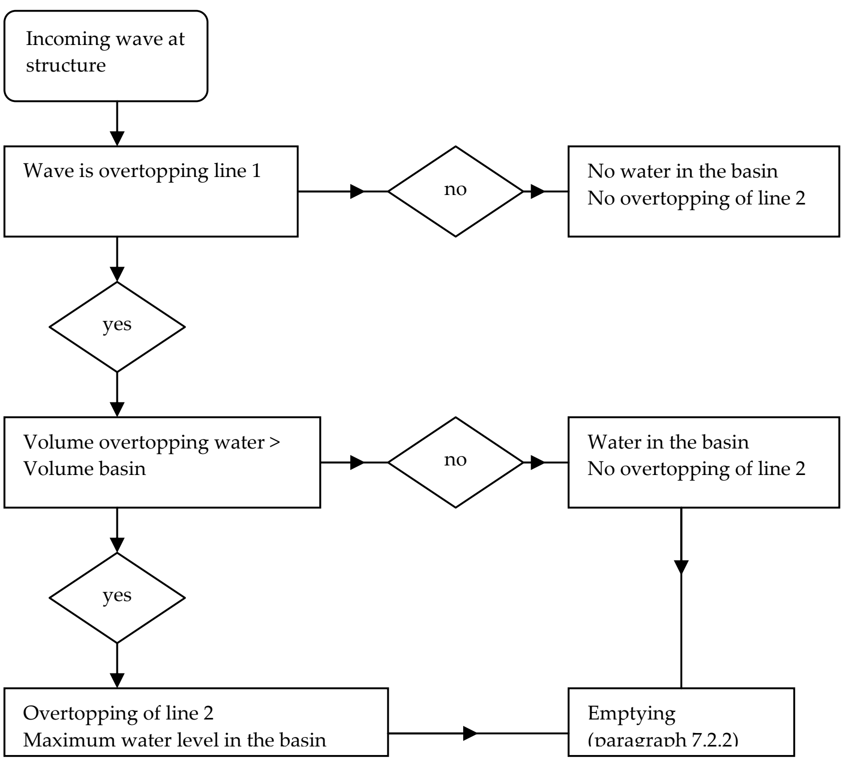


Figure 7-1: Schematization of the superstructure of the New Orleans storm surge barrier

The superstructure of the reference design is bounded by a perforated inclined wall, a vertical backwall and a bottom plate. In the superstructure a water level of 6m is present. In the numerical model the basin represents the part of the superstructure above the still water level of 6m. In other words the basin represents the part of the superstructure which can collect the overtopping water.
As can be seen in Figure 7-1 the basin of the superstructure has a certain volume that can work as a buffer for the wave, which is passing line 1. If the volume of the wave, which is passing line 1, is larger than the volume in the crest, a part of the wave will also pass line 2. In this schematization no outflow through the gap at the bottom of the inclined wall has been applied. A schematization of the above described model is shown in the following flow chart.



Incoming wave at inclined wall

In the model the average overtopping discharge can not be used, because this parameter can not say something about whether the basin can store the overtopping volumes, which are random in time, or not. Therefore the single overtopping volumes are used and as already mentioned in paragraph 4.5.2.2 these volumes are Weibull distributed [Van der Meer, 2002].

In the model it is determined whether $P_{ov}>0$ or $P_{ov}=0$. When the probability of overtopping per wave is zero, no overtopping volume is passing line 1. The result of $P_{ov}=0$ is that the basin is not being filled and therefore no overtopping over the backwall is taking place.

Water that passed line 1

In the case $P_{ov}>0$, water is passing line 1 and the basin is being filled with the possibility that a part of the overtopping volume is passing line 2, see the above given flow chart. It should be determined whether water is also passing line 2. In this model, it is assumed that whether the water is passing line 2 or not depends on the following parameters:

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

The *volume of the solitary* wave can be determined using the theory explained in paragraph 4.5.2.2 (Equation 4-10 and Equation 4-12, page 65) .

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

The *volume of the basin* is determined by three parameters: angle of the inclined wall; width of the basin and the height of the basin, see Equation 7-1.

$$V_{basin} = B_{basin} * h_{basin} - \frac{h^2_{basin}}{2\tan(\alpha)} \qquad \text{Equation 7-1}$$

Where

V_{basin}:	volume of the basin	[m³/m]
B_{basin}:	width of the basin at still water level	[m]
h_{basin}:	height of the basin; similar to the crest height	[m]
α:	angle of the inclined wall	[⁰]

The *length spreading effect* has been explained in paragraph 4.9. It is assumed that due to this length spreading effect, the buffer capacity is 1,5 as much as the volume of this buffer.

By means of increasing the basin capacity by the LSE factor the length spreading effect is included in the model.

Equation 7-2 represents the maximum basin capacity, wherein also the effectiveness of the basin and the length spreading effect are included.

$$V_{buffer} = \theta * l.s.e * V_{basin} \qquad \text{Equation 7-2}$$

Where

V_{buffer}:	maximum basin capacity	[m³/m]

θ:	effectiveness of the basin	[-]
l.s.e.:	length spreading effect	[-]
V_{basin}:	volume of the basin	[m³/m]

Whether the water is passing line 2 or not depends on two parameters, namely the volume of the overtopping water and V_{buffer}.

If $V_{\text{water passed line 1}} > V_{\text{buffer}}$ water is overtopping line 2, the backwall

$$V_{\text{water passed line 2}} = V_{\text{water passed line 1}} - V_{\text{buffer}}$$

$$V_{\text{basin}} = V_{\text{buffer}}$$

If $V_{\text{water passed line 1}} \leq V_{\text{buffer}}$ no water is passing line 2, the backwall

$$V_{\text{basin}} = V_{\text{water passed line 1}}$$

In this model framework the emptying process is not yet being considered. One can see from Figure 7-1 that the gap at the bottom of the inclined wall is missing. In the next paragraph the emptying process by means of an outflow through the gap due to head difference is included in the model.

7.2.2 Outflow through the gap due to head difference

In this paragraph the emptying process is included in the model. Figure 7-2 shows the schematization of the superstructure with the gap in the inclined wall.

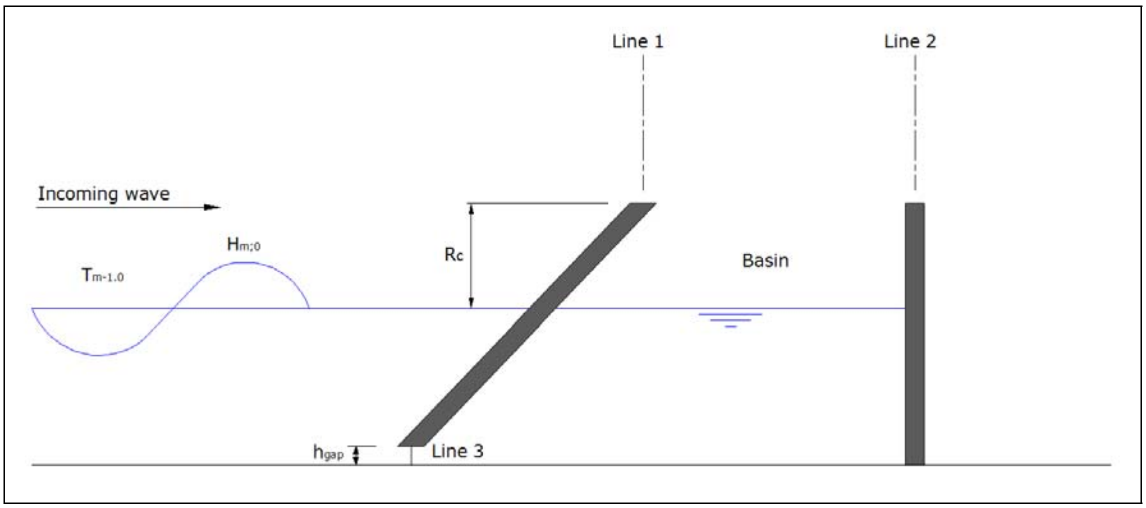


Figure 7-2: Schematization of the superstructure including the gap at the bottom of the inclined wall

In paragraph 7.2 it is mentioned that wave induced inflow is not being considered in the numerical model, therefore only an outflow through the gap is present. The outflow is excited by head difference between the inside and outside water level. In the model the water level rise is a result of the overtopping volumes over the inclined wall. The outflow is defined as the water passing line 3.

The emptying capacity of the gap has been explained in paragraph 4.6 and the discharge through the gap (Equation 4-39) and the emptying duration (Equation 4-42) are described.

The emptying duration which is given by (Equation 4-42) is non-linear and because linearity is preferred this emptying duration has been linearized, see Appendix 4.

$$t_{\text{E,linear}} = \frac{V_{\text{basin}}}{Q_{\text{gap}}} = \frac{B_{\text{basin}} * h_{\text{basin}} - \dfrac{h_{\text{basin}}^2}{2\tan(\alpha)}}{\mu A_{\text{gap}}\sqrt{2g\kappa h_{\text{basin}}}}$$ Equation 7-3

Where

κ: linearization factor [-]

The emptying volume per meter length in one wave period can be derived from Equation 7-3, see Equation 7-4.

$$V_{\text{emptying}} = \frac{Q_{\text{gap}}}{T} = \frac{\mu A_{\text{gap}}\sqrt{2g\kappa h_{\text{basin}}}}{T}$$ Equation 7-4

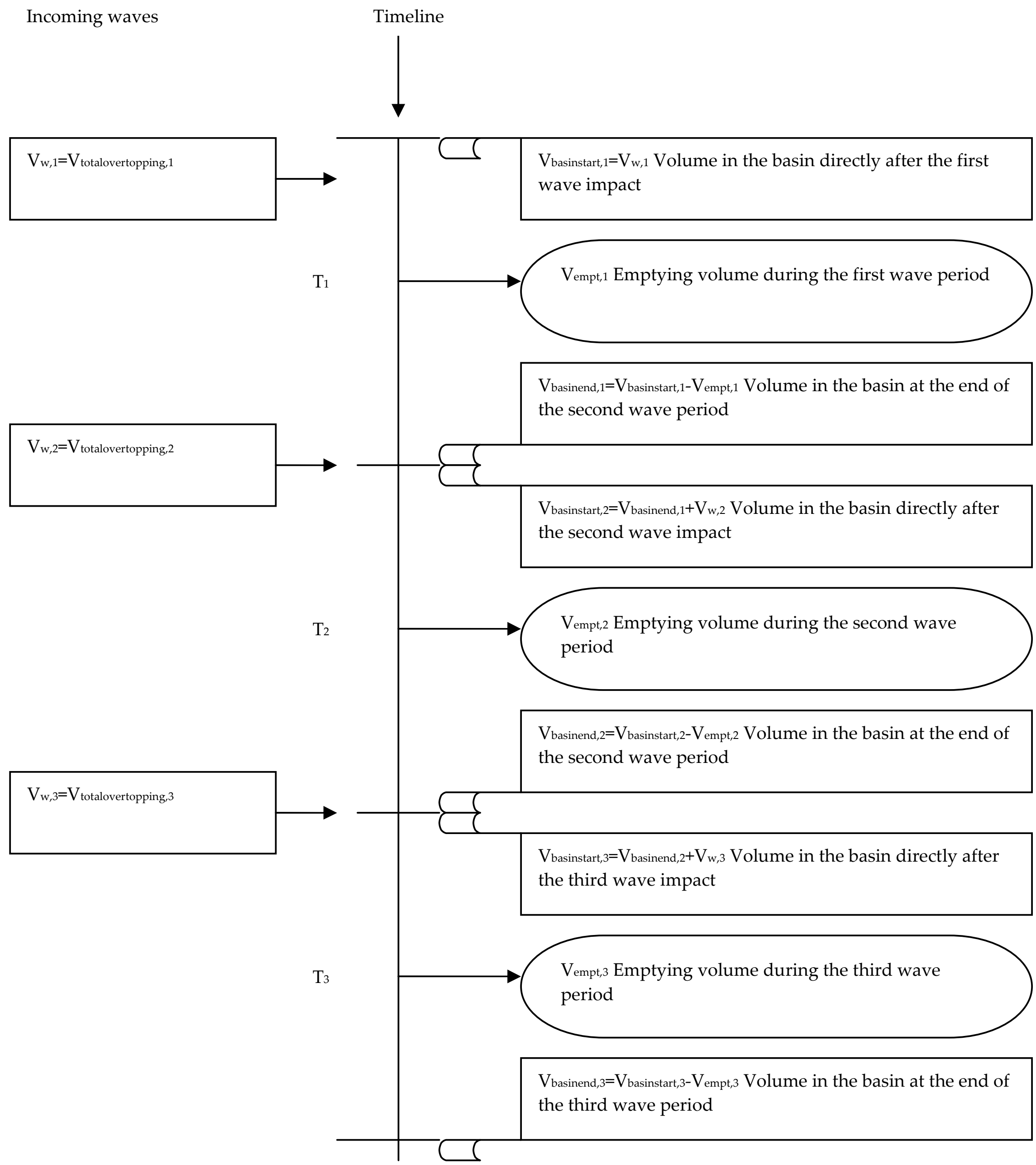
Where

$V_{emptying}$:	emptying volume per meter length	[m³/m]
Q_{gap}:	linearized emptying discharge per meter length	[m³/s/m]
T:	wave period	[s]

7.2.3 Theory regarding a wave field

In the model described so far only a single wave has been considered. When a volume of water was passing line 1 the basin was empty at all times. However, when a storm is considered, a wave field is present. Waves are constantly overtopping and the basin is not necessarily empty, because water can be present from the previous overtopped water volume. The effect of the wave field is included in the model by taking account the basin volumes of the previous waves and the emptying volumes during these previous waves. The change in the basin volume due to overtopping volumes and the emptying process is described during three wave periods in the following flow chart.

From the flow chart it becomes clear that the basin volume at the end of each wave period is the difference between the basin volume at the start of the wave period and the emptying volume of that wave period.



Wave grouping

Wave grouping is the phenomenon that waves do not approach the structure with constant intermediate times. The structure is being exposed to wave groups, which means that the intermediate time for two waves is almost zero. One can imagine that this has a negative effect on the buffer capacity. The basin does not have time to empty after a wave impact, since the next wave is immediately following. As already mentioned in paragraph 7.2 wave grouping is not included in the numerical model. The reason for this lies in the lack of insight in this process.

7.2.4 Numerical model

In the previous paragraphs the model frame has been described. This model frame is the basis for the numerical model. In Appendix 5 the program code is given.
As mentioned in paragraph 7.2 the numerical model of the Crest Drainage Dike has been used. As stated before the geometry en the empting process of the New Orleans storm surge barrier differs from the CDD. For these two things adjustments in the model have been performed. Also the comparison with physical tests, which is present in the initial model, has been removed.
The initial model of the CDD is based on the top-down model described in [Mesman, 1991]. One of the reasons it was chosen for this top-down model was that it is easy to adapt, which now has been done for the New Orleans storm surge barrier.

7.3 Optimization of the configuration

In this paragraph an optimization of the configuration of the superstructure by means of the numerical model, derived in the previous paragraphs, is performed. Since the numerical model does not optimize the configuration by itself, the overtopping over the vertical backwall is calculated for different configurations by means of the model. Then a conclusion can be drawn for the best functioning configuration according to the water retaining function of the vertical backwall. In paragraph 7.3.1 'Configurations and output' the configurations are described, which are being tested by the model. In this paragraph also the output for the different configurations is given. The next configuration is being achieved by the output of the previous configuration. For example, when a certain configuration leads to significant overtopping over the vertical backwall one has to adjust to configuration. One can enlarge the gap, resulting in a decrease in emptying time. The basin capacity can also be enlarged to decrease the overtopping over the vertical backwall.

7.3.1 Configurations and Output

The configurations are calculated by the model. Due to the fact the overtopping volumes are created by a random generator, based on a Weibull distribution, the outcome of the model is for every run different. Therefore it is chosen to run the model for 10 storms resulting in 10 outcomes for each configuration. So besides the description of the configurations also the mean and the deviation of the outcomes of these configurations are given in Table 7-1. The full output of the configurations is given in Appendix 6. In this appendix the output for all the 10 runs for each configuration is given.
The discharges presented in Table 7-1 are the mean of the cumulative discharges during a storm. For the configurations listed in Table 7-1 the hydraulic boundary conditions described in Chapter 2 'Environmental boundary conditions' are used: H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m.

Table 7-1: Description of configurations for H_{m0}=2,2m; $T_{m-1,0}$=5,25s; t_s=6hr and h=6m, including output numerical model

	Configuration 1	Configuration 2	Configuration 3	Configuration 4
Angle inclined wall [^{0}]	45	45	63,4	63,4
h_{wall} [m]	10	8	8	8
h_{gap} [m]	0,75	0,5	0,5	0,75
B_{basin} [m]	10	8	8	8
Output numerical model				
$q_{overtopmean}$ [l/s/m]	μ=0 σ=0	μ= 0,106 σ= 0,141	μ = 0,291 σ= 0,293	μ = 0,084 σ= 0,195
$q_{emptyingmean}$ [l/s/m]	μ= 10,7 σ= 0,4	μ = 163,1 σ= 3,5	μ = 193,1 σ= 5,4	μ = 179,8 σ= 3,1
$q_{totalovertopmean}$ [l/s/m]	μ= 10,7 σ= 0,4	μ = 163,3 σ= 3,5	μ= 193,4 σ=5,4	μ= 181,8 σ= 4,7

7.3.2 Optimal configuration

From Table 7-1 it can be seen that configuration 1 results in no overtopping over the vertical backwall. This means that the combination of water buffer and emptying the basin works properly. The other three configurations result in overtopping over the vertical backwall. The question is whether this overtopping is allowed or not.
It is assumed that an overtopping discharge over the vertical backwall is not undesirable. However, the discharge is not allowed to be too large. It is assumed an overtopping discharge of 0,1l/s/m gives no problems at the protected side of the New Orleans storm surge barrier. In reality there is a roadway present behind the vertical backwall. During storm conditions it is still possible to use this roadway when a q of 0,1l/s/m is present. However, it has to be noted that the q is an average overtopping, but the single overtopping volumes can be large and can endanger the roadway users. The roadway will probably not be used during storm conditions except by people from the hurricane protection system.

When an overtopping discharge of 0,1l/s/m is allowed configuration 2 and configuration 4 are both suitable. Configuration 4 consists out of an inclined wall with an angle of $63{,}4^0$ in stead of 45^0 , which is the angle of the wall of configuration 2. This means that less concrete is used for the inclined wall of configuration 4, having in mind that h_{wall} for both configurations is similar.

The main criterion for the optimization process:
The structure has to be as low as possible, having in mind that the width of the structure can not be too large.

Because of the main criterion it is tried to lower the structure further. This has only been performed for the 1:1 sloped wall. The results are shown in Table 7-2. As can be seen from the table configurations 5 and 6 do not fulfil the overtopping criterion of max 0,1l/s/m. Configuration 7 does meet the overtopping criterion. This configuration consists of an 1:1 sloped wall with a height of 7,5m and a gap with a height of 1,25m. The full output of configurations 5, 6 and 7 are also given in Appendix 6.

Table 7-2: Optimal configuration for H_{m0}=2,2m; $T_{m-1,0}$=5,25s; t_s=6hr and h=6m

	Configuration 5	Configuration 6	Configuration 7
Angle inclined wall [^{0}]	45	45	45
h_{wall} [m]	7	7	7,5
h_{gap} [m]	0,75	1	1,25
B_{basin} [m]	8	9	9
Output numerical model			
$q_{overtopmean}$ [l/s/m]	μ=106,4 σ=8,7	μ= 2 σ= 0,9	μ= 0,096 σ= 0,13
$q_{emptyingmean}$ [l/s/m]	μ= 441,4 σ= 7,4	μ = 267,2 σ= 5,9	μ = 187,3 σ= 3,2
$q_{totalovertopmean}$ [l/s/m]	μ= 547,9 σ= 14,2	μ = 269,2 σ= 6,3	μ = 187,4 σ= 3,3

A numerical model has been constructed excluding the water movement in the basin and the wave induced inflow. By means of this numerical model a configuration has been determined which fulfills the criterion that the overtopping over the vertical backwall is equal or smaller than 0,1l/s/m. The superstructure has to be as low as possible. Configuration 7 is a configuration which fulfills the above given overtopping criterion and the above given requirement.

For finding this configuration by means of the numerical model the water movement and the wave induced inflow were not included. However during the search for the “optimal” design these processes have been kept in mind. The basin is chosen such that it has not a width of n*(0,5L_{wave}) with n=1,2,3etc, so resonance would not be a problem. Plunging over the vertical backwall of the overtopping volumes for an empty basin is also not a problem, see Figure 4-28.

From Figure 4-37 one can see that a basin width of 9m and a gap height of 1,25m result in a wave induced inflow of about 4,5m^3/s/m. Compared to the overtopping over the inclined wall the inflow is quite large and the outflow would be influenced negatively by this wave induced inflow. However one has to keep in mind that the orbital motion is not included in the determination of the discharge and that the wave induced inflow actually is pointed half a wave period in the opposite direction, the same direction of the outflow.

Intermediate conclusions

The numerical model provides one a tool to calculate the overtopping volumes over the vertical backwall; the outflow through the gap and the overtopping volumes over the inclined wall. It is not possible to perform an automatic optimization with the model. One has to check different configurations whether they fulfil the imposed criteria or not. When the height of the structure has to be as low as possible configuration 7 is feasible. This configuration consists of an 1:1 sloped wall, with a height of 7,5m. The gap at the bottom of the inclined wall has a height of 1,25m and the width of the basin at still water level is 9m.

Although configuration 7 fulfils the criteria it is based on the hydraulic design conditions (H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m). When these design conditions are underestimated, the structure is completely overtopped. Therefore it is recommended to increase the height of the inclined wall.

Chapter 8 Conclusions and recommendations

The main objective of this research was to get insight in the hydraulic processes in front of and inside the superstructure which determine the optimal configuration of the superstructure. In this research a numerical has been constructed to check whether a configuration fulfils overtopping criteria or not. This model is based on insight in the hydraulic processes in front of and inside the superstructure. The insight in the hydraulic processes has been obtained by an analytical research and by physical model testing. In paragraph 8.1.1 the conclusions following from the analytical research are given. In paragraph 8.1.2 the conclusions following from the model testing are given. The final conclusions for the hydraulic processes, which form the basis for the numerical model, are given in paragraph 8.1.3. In paragraph 8.1.4 the conclusion for the optimal configuration is given. Finally the recommendations are given in paragraph 8.2.

8.1 Conclusions

8.1.1 Conclusions analytical research

In this paragraph the conclusions from the analytical research are given for the separate hydraulic processes.

Water movement in the basin

- The water movement in the basin depends on the gap height and on the basin width. For an increasing gap height the transmitted wave height increases resulting in a larger water level movement. The water movement in a basin consisting of a vertical and an inclined wall and the water movement in a rectangular basin are the acting the same. Therefore resonance of the water level in the basin of the reference design is excited for basin widths of n*(0,5L_{wave}) with n=1,2,3etc.

Wave run-up

- The wave run-up is influenced by the gap, which is present at the bottom of the inclined wall. The wave run-up decreases for increasing gap height. The influence of the gap on the wave run-up is translated into a reduction factor, which is equal to the reflection coefficient. This reduction factor is based on the loss of energy through the gap. The reflection coefficient depends on the gap height and on the basin width.
  So, to account for the influence of a gap at the bottom of an inclined wall on the wave run-up, the wave reflection coefficient is used as a reduction factor in the formula for non-perforated inclined walls.

Wave overtopping

- The wave overtopping is influenced by the gap, which is present at the bottom of the inclined wall. The reduction factor for the influence of the gap on the wave run-up is also valid for the influence of the gap on the wave overtopping. So, to account for the influence of a gap at the bottom of an inclined wall on the average wave overtopping, the wave reflection coefficient is used as a reduction factor in the formula for non-perforated inclined walls.
- The formula for maximum single overtopping volumes during a storm for non-perforated inclined walls can also be used for perforated inclined walls. The maximum single overtopping volume is a function of the average overtopping and the reduction factor for the influence of the gap is implemented in the formula of the average overtopping.
- The water jet jumping from the inclined wall is not overtopping the vertical backwall for steep slopes and basin widths $\leq$4m.

Emptying process

- The water level rise in the basin, as a result of overtopping water volumes, causes a head difference between the inside and outside water level. This water level rise induces an outflow through the gap. The emptying process is described by the Torricelli formula.

Wave induced inflow

- Besides overtopping water the wave induced inflow determines the water level rise in the basin. The wave induced inflow can be calculated by means of the used basic model. This model is based on the Long Wave Theory, a discharge relation and the preservation of volume.

Length spreading effect

- The length spreading effect can be taken into account and does have a positive influence on the basin capacity of the structure. The spreading time scale; the intermediate overtopping time scale and the emptying time scale are in the same range for the New Orleans situation. A length spreading effect factor of 1,5 is taken for the reference design.

8.1.2 Conclusions physical model testing

Water movement in the basin

- The configuration which consists of a perforated inclined wall in front of a vertical backwall does not act the same as a configuration consisting of a perforated vertical wall in front of a vertical backwall when the water movement is considered. There is no resonance of the water level occurring for a basin width of 0,5L and neither for a width of 0,75L which.

Wave run-up

- The wave run-up is not decreasing by the presence of a gap at the bottom of an inclined wall. When the gap height is increased, it is not noticeable in the wave run-up.
- The wave run-up is not influenced by the presence of a vertical backwall.
- From the tests it can be concluded that the presence of an outflow through the gap at the bottom of the inclined wall has no effect on the wave run-up.

Wave overtopping

- The wave overtopping is significantly reduced by the presence of a gap. A gap of 0,04m reduces the wave overtopping with 20%. This is the case for both the 1:1 and 2:1 sloped walls.
- An outflow 1l/s/m, which is based on the overtopping discharge, does not influence the wave overtopping. However, when the outflow is increased significantly to 10l/s/m an increase of 8% in wave overtopping is the result. The increase of inflow is not a realistic value for the situation and therefore it can be concluded that the outflow has no influence on the wave overtopping.

8.1.3 Final conclusions hydraulic processes

In this paragraph the final conclusions are given for the hydraulic processes, which form the basis for the numerical model.

Water movement in the basin

- The water movement in the basin is increased by an increasing gap height.
- The water movement in a basin consisting of a vertical and an inclined wall and the water movement in a rectangular basin are not the acting the same. The inclined wall influences the water movement in the basin.

Wave run-up

- The wave run-up is not influenced by the gap, which is present at the bottom of the inclined wall. No reduction factor has to be taken into account for the influence of the gap on the wave run-up. The formula for non-perforated inclined walls has to be used to calculate the wave run-up for perforated inclined walls.
- The wave run-up height for non-perforated inclined walls obtained by the analytical research differs from the model testing results with a factor 1,7.

Wave overtopping

- The wave overtopping is influenced by the gap, which is present at the bottom of the inclined wall. A reduction factor has to be taken into account for the influence of the gap on the wave overtopping. The reduction factor based on the results from the model tests has to be implemented in the formula for non-perforated inclined walls to account for the influence of the gap.

Emptying process

- The water level rise in the basin, as a result of overtopping water volumes, causes a head difference between the inside and outside water level. The water level rise induces an outflow through the gap and can be described by the Torricelli formula.

Wave induced inflow

- Besides overtopping water the wave induced inflow determines the water level rise in the basin. The wave induced inflow can be calculated by means of the used basic model. This model is based on the Long Wave Theory, a discharge relation and the preservation of volume.

Length spreading effect

- The length spreading effect can be taken into account and does have a positive influence on the basin capacity of the structure. The spreading time scale; the intermediate overtopping time scale and the emptying time scale are in the same range for the New Orleans situation. A length spreading effect factor of 1,5 is taken for the reference design.

8.1.4 Final conclusion optimal configuration

The numerical model provides one a tool to calculate the overtopping volumes over the vertical backwall; the outflow through the gap and the overtopping volumes over the inclined wall. It is not possible to perform an automatic optimization with the model. The numerical model includes the wave run-up, wave overtopping over the inclined wall, wave overtopping over the vertical backwall and the emptying process. The model does not take wave induced inflow, wave grouping and the water movement into account.

One has to check different configurations whether they fulfil the imposed criteria or not. When the height of the structure has to be as low as possible and the maximum discharge over the vertical backwall is 0,1l/s/m, the following configuration seems to be the most optimal:

angle of the inclined wall, α:	45^0
height of the inclined wall, $h_{wall:}$:	7,5m
height of the gap, h_{gap}:	1,25m
width of the basin (at SWL), B_{basin}:	9m

Although this configuration fulfils the criteria it is based on the hydraulic design conditions (H_{m0}=2,2m; $T_{m-1,0}$=5,25s and h=6m). When these design conditions are underestimated, the structure is completely overtopped. Therefore it is recommended to increase the height of the inclined wall.

8.2 Recommendations

8.2.1 Recommendations insight hydraulic processes

- It has to be investigated for which basin width resonance of the water level fluctuation is occurring in the basin of the reference design. It also has to be investigated which depth has to be considered as reference level for the basin width.
- Since it can not be explained by this research why the gap influences the wave overtopping and not the wave run-up further research is necessary to find the reason for this phenomenon. The expectation is that the overtopping time decreases when a gap is implemented. This has to be checked by physical model testing.
- It has to be investigated why the analytical results for the wave run-up for a non-perforated inclined wall differ with a factor 1,7 from the results obtained by the physical model testing.
- The reduction coefficient is now based on two gap heights, which means extrapolation has been used for other gap heights. Since only two points are used for this extrapolation, the reduction coefficient is not very liable. So the reduction coefficient for the influence of the gap on the wave overtopping has to be further investigated by physical model testing.

8.2.2 Recommendations numerical model

- The numerical model excludes the wave induced inflow, the water movement and wave grouping. Including each of these processes would result in an improvement of the model. The most important improvement would be the including of the wave induced inflow, because this hydraulic process causes filling of the basin and it influences the emptying of the basin negatively.

8.2.3 General recommendations

- In this study the near shore wave conditions are determined with SWAN 1D. Since the water line and wind have a large influence on the wave conditions a 2D model should be used.
- When it is decided to perform more experimental research on this topic, the physical model tests should be performed in a wave flume for more accurate results.
- Only one wave condition is used in this thesis. This wave condition describes a non-linear wave. It would be better to have an understanding of the influences of a whole wave field, because other wave conditions can result in a different behaviour of the processes and wave pressures. Therefore also model tests with irregular waves have to be performed.

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