Simulating wave penetration in an inlet using the numerical models SWAN and SWASH

Dimensioning of an inlet and the comparison of two numerical models

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PREFACE

This research was conducted in partial fulfillment of the Master degree of Civil Engineering at Delft University of technology. The research was proposed by Ingenieursbureau Gemeente Amsterdam and most of the thesis work has been carried out at Ingenieursbureau Gemeente Amsterdam.

I would like to thank all the members of my graduation committee for their supervision and constructive criticism. First of all I would like to thank Dr. Ir. Bas Hofland for the overall supervision, guiding me in the right direction and beholding me of bringing up more and more things that could also been done. Then I would like to thank Dr. Alessandro Antonini for being critical at the right moment and especially for helping me out when I could not figure out how to detect reflected waves from a wave signal. I would also like to thank Dr. Ir. Gerbrant van Vledder for helping me out with the use of SWAN and SWASH. Furthermore I would like to thank Ir. Franco Pantano for helping me getting around at the Gemeente Amsterdam and for providing me all the necessary information to succeed with this research. Finally I would like to thank my fellow students, friends and family who helped me through this process.

Rutger R. Komen
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SUMMARY

To cope with the ever growing demand of living area in Amsterdam, the municipality is building a new island in the IJmeer, named Strandeiland. In the eastern part of Strandeiland an inlet will be situated. This inlet will be protected by means of a breakwater from incoming waves that can develop all over the Markermeer and IJmeer. The walls surrounding the inlet, as proposed in a provisional design by the municipality of Amsterdam, are a combination of a vertical wall with a berm in front of it. In normal conditions the berm will be above the water level and people will be able to walk on this berm. The future use of the inlet is not yet decided but there is a (big) chance that the inlet will function as a marina.

The objective of this research is to determine the normative wave conditions in the inlet for different situations. These wave conditions can then be translated to useable advice for the municipality about different subjects. These subjects include the dimensioning of the walls surrounding the inlet, the suitability of the inlet to be used as a marina and the necessity and length of the breakwater.

In order to determine the wave conditions inside the inlet, the numerical models SWAN and SWASH are used. Here immediately another objective comes forward, namely to investigate whether the less computational time consuming model SWAN is able to give accurate results in a situation like in this research. If so, the use of SWAN is preferred over the use of the more computational time consuming model SWASH, from which it is known that it should be able to model a situation like in this research accurate. Hydra-NL is used in this research to determine the incoming wave conditions.

Waves that enter the inlet will reflect from the walls surrounding the inlet. Therefore it is needed to know the reflection coefficient of these walls. Because the walls are a combination of a vertical wall and a berm in front of it which can be submerged sometimes due to set-up, determining the reflection coefficient is something rather complex and specific for this case. This is done by simulating a wave flume in a 1D computation in SWASH. From this computations it is concluded that the reflection coefficient varies for different set-ups and different wave heights. Therefore for every situation simulated in the remainder of the research, the right reflection coefficient is determined using this 1D computation.

To meet the objectives, 4 situations are simulated in both SWAN and SWASH. Due to demands of the waterboard it is not allowed to take the breakwater into account when concerning the ultimate limit state (ULS) conditions. Therefore for these conditions a simulation is done without a breakwater. Furthermore for the serviceability limit state (SLS) conditions a situation without a breakwater, a situation with a breakwater with the length of $100m$ and a situation with a breakwater of $130m$ is simulated. Thereafter the results of the simulations are processed using formulas obtained in a scale model test of Delft Hydraulics (Heijer, 2005) to be able to give advice concerning the wall dimensioning. To give advice about the suitability of using the inlet as marina, the movement of ships due to the waves is concerned. To say something about the necessity and length of the breakwater, the results of these simulations are compared.

From the results it can be concluded that the dimensioning of the wall as proposed by the municipality of Amsterdam meets the criteria of a maximum wave overtopping during ULS conditions of $10\ l/s/m$.

The wall height could even be decreased significantly but because of the use of the hinterland that will not be possible. The dimensioning of the berm can be based on other criteria than safety, one could think about using the area for benches etc. These type of considerations are beyond the scope of this research and therefore the dimensioning as proposed by the municipality is used. From the results of the tests to check the suitability of the inlet as a marina it can be concluded that the inlet can be used as a marina, but only for ships with a waterplane length of $15m$ or larger. Concerning the breakwater it can be concluded that it is necessary to construct a breakwater in order to make a marina possible and in order to prevent that the berm will be overtopped by waves frequently. Furthermore it can be concluded that decreasing the length from $130m$ to $100m$ does not significantly change the outcome of the simulations and therefore in the final design a breakwater of $100m$ is proposed. When comparing the outcome of the models SWAN and SWASH it can be concluded that SWAN is accurate when concerning a situation like in this research, even though it seems that diffraction would play an important role. Therefore it is advised to use SWAN in comparable situations.

It is recommended to do further research concerning the optimal length of the breakwater. In this research it is already concluded that the breakwater can be $30m$ shorter than proposed beforehand, maybe this can become even more. Furthermore it is recommended to, if it is desired to moor smaller ships in the marina, do research in wave reducing measurements in the inlet.

Content

List of symbols

a	Wave amplitude	(m)
a_i	Incoming wave amplitude	(m)
a_r	Reflected wave amplitude	(m)
c	Wave celerity	(m/s)
c_f	Dimensionless bottom friction	$(-)$
d	Depth	(m)
F	Fetch	(m)
g	Gravitational acceleration	(m/s^2)
H	Wave height	(m)
H_s	Significant wave height	(m)
h_b	Water depth on berm	(m)
h_c	Wall height relative to SWL	(m)
k	Wave number	(rad/m)
Q	Dimensionless wave overtopping	$(-)$
q	Wave overtopping	$(m^3/m/s)$
R	Dimensionless wall height	$(-)$
r	Reflection coefficient	$(-)$
s_{op}	Wave steepness	$(-)$
T_p	Peak period	(s)
$\dot{u}$	Wave acceleration	(m/s^2)
U	Wind speed	(m/s)
α_{eq}	Equivalent slope	$(-)$
γ_k	Reduction factor berm	$(-)$
η	Free surface elevation	(m)
ξ_{eq}	Breaker parameter	$(-)$
ω	Angular frequency	(rad/s)

Abbreviations

NAP	Normaal Amsterdams Peil
RAO	Response amplitude operator
SLS	Serviceability limit state
SWL	Still water level
ULS	Ultimate limit state

1 INTRODUCTION

In 1996 the municipality of Amsterdam decided to start the construction of IJburg. Phase 1 of the construction included the islands Haveneiland, Rieteiland and Steigereiland. The reason for this expansion of land was the continues growth of housing demand in the city of Amsterdam and the lack of space to realise this inside the city. This growing demand did not stop after constructing these islands so in 2010 the plan was made for further expansion which led to the idea of IJburg 2. This plan also consisted of 3 islands: Centrumeiland, Strandeiland and Buiteneiland. Due to the financial crisis in 2008, this plan was not executed immediately but the growth of the city kept going on with about 11.000 citizens a year. So in 2015 in the Nota Koers 2025 (Koers 2025, 2015) IJburg 2 was one of the locations designated to cope with the growing city. The plans of 2010 were used, supplemented with new visions, including Strandeiland that will make an area big enough for 8.000 residential houses. In the spring of 2017 the land reclamation started. In 2025 the first people should start to live on the island. Until then, a lot of work has to be done. Among others an inlet will be situated on the east side of Strandeiland. This inlet forms the basis and motivation of this research.

1.1 MOTIVATION

On the eastern side of Strandeiland, an inlet will be situated. The municipality of Amsterdam is interested in the wave agitation inside the inlet for possible future uses of the inlet. The intended use is not yet known but one of the main options is a marina. Therefore this option will be analysed during this research. Because the inlet is situated on the east side it is more exposed to incoming waves from the IJmeer and Markermeer. Therefore a breakwater is designed to protect the inlet. The municipality is interested in what the effect of the breakwater is. Furthermore the municipality wants to know what the dimensioning of the walls enclosing the inlet should be to withstand severe storms. When concerning an inlet with incoming waves partly blocked by a breakwater and walls enclosing the inlet, a lot of processes play a role in the development of the waves. It is important to know how the waves will enter the inlet, how they will reflect inside the inlet, with what height they will hit the walls, etc. Which one of these aspects is dominant depends on the dimensions and parameters of the inlet and the boundary conditions such as incoming wave height and incoming wave period. The extent to which it is important to know the different aspects of wave agitation inside an inlet depends on the intended use of the area.

To give an accurate advise in how the waves will behave, numerical modelling is an often used method, for instance in (Alabart et al., 2014). Multiple models are available for simulating a situation like the inlet at Strandeiland. In this research SWASH and SWAN are used. One of the differences in using these models is that SWAN is less time consuming than SWASH. The downside of SWAN is that is does not simulate diffraction, which is present in this situation. The question is whether using SWAN is accurate enough or not. Furthermore the municipality often uses Hydra-NL for multiple aims. This could also be used in this case to determine the wave characteristics in the area. The combination of these models and which of them is preferred is discussed in this research.

In this study the focus will be on wind waves penetrating the inlet on the east side of Strandeiland. Designing the inlet and dimensioning the vertical walls based on the findings of the modelling of the wave penetration will be the main priority of this study. Furthermore the comparison between the different models used will be made.

1.2 Problem description

The inlet on the east side of Strandeiland will be exposed to waves that can develop over the entire Markermeer. During storm conditions these wave heights can be severe and thus flood protection is needed. Due to demands of the waterboard the breakwater cannot be taken into account when concerning the ultimate limit state (ULS). Therefore the significant wave height (H_s) in the inlet which is normative during ULS conditions will be a lot higher than during SLS conditions.

Another topic that might be problematic is wave agitation inside the inlet. This is primary interesting when concerning the more daily use of the inlet. There are two reasons for which this is interesting and too large wave heights in the inlet will be problematic. First of all the horizontal berm in front of the wall. If the wave heights inside the inlet will be often large, this horizontal berm, intended for people to walk on, will overflow often and the functionality will reduce. Secondly the inlet might in the future be used as a marina. If this is the case, too much wave agitation is undesired.

For modelling the inlet, the models SWAN and SWASH are used. SWAN is a less time consuming model but it does not account for diffraction. SWASH is more time consuming and maybe too complex and accurate for what is necessary in this case, but it does take diffraction into account. Therefore it is questionable which of the two models fits better in a situation like this.

1.3 Objectives

This study aims at solving the local problems regarding the inlet at the northeast of Strandeiland and providing the key parameters necessary when designing the vertical walls surrounding the inlet. The main question to be answered is:

- What will be the normative wave conditions inside the inlet as the result of different boundary conditions and different lay-outs?

This main question can be answered by first answering the following sub-questions:

- What are the differences in using the models SWASH and SWAN, and which of both is most useable in which situation?
- What is the added value of the breakwater during wave conditions with a lower return period?
- Is the construction of a marina possible considering the wave agitation?
- If not, what should be changed in the inlet to obtain the right wave climate?

The objective of this study is to answer these questions as accurate and complete as possible.

1.4 APPROACH

In this part the approach leading to meeting the objectives of this research is discussed. First of all a literature study is done to define the theoretical background. This theoretical background includes both the more basic knowledge about wave propagation in shallow water and around obstacles as the theory behind the modelling. After this literature study the focus goes to executing the research concerning the inlet. This is done via the following steps:

1. **Sketch multiple alternatives**
   From the objectives it can be seen that multiple situations need to be analysed. Therefore it is important to have a good overview from the beginning which alternatives will be analysed. Within these alternatives the variation will be with or without breakwater, ULS or SLS, Wave height and wave direction, etc.
2. **Select the models**
   When the alternatives are known, the model or models to use can be selected. In this case SWASH and SWAN are chosen. Why specifically these models? This is based on the knowledge gathered in the literature study and on which model is able to simulate the processes in this situation. Furthermore it is important to keep in mind the availability of the models. Is it possible to use every model? SWAN and SWASH are both open source models.
3. **Obtain boundary conditions**
   The boundary conditions will mainly be obtained from Hydra-NL. This model can be inaccurate when the return period decreases. So these conditions will be checked and adjusted if not realistic.
4. **Set up the models**
   After obtaining the boundary conditions the model can be set up. This will be done using the manuals provided with the models. Both numerical input files as the choice of inputs concerning the physics are discussed
5. **Execute necessary runs**
   When everything is ready the model can run. It is important just to do the runs necessary, because of the time the runs cost.
6. **Translate output to useful data**
   From both SWASH and SWAN multiple types of output can be requested. It is important to select the useful output and to translate this to useful data. This forms the key of the research results.
7. **Analyse data and formulate conclusion concerning the inlet**
   The data obtained will be analysed to provide useful advice for the municipality of Amsterdam.
8. **Formulate conclusion concerning the use of SWASH an SWAN**
   Now that the advice towards the municipality is provided, the conclusion concerning the comparison between SWASH and SWAN can be formulated.

2 GENERAL INFORMATION

Now that the situation of Strandeiland and the objectives of the research are introduced, the more detailed description of the case and the theoretical background is stated in this chapter. During this chapter the more basic theory will only be referred to and will not be further elaborated. Just the specific theory needed for this research will be elaborated. First the case of Strandeiland will be discussed. Thereafter the theoretical background concerning waves will be treated. Here specifically different forms of reflection and the wave agitation and ship response will be discussed. Then the theory of the numerical models SWAN and SWASH and the theory of port modelling will be elaborated.

2.1 CASE

The goal of creating the island Strandeiland is to create a green area to live in, combined with a density that typifies Amsterdam. The focus in islands like Steigereiland and Rieteiland was mainly to live by, or with a view on, water. The focus of Strandeiland is more to live near the beach. Strandeiland will be one of the three islands constructed in IJburg phase 2. The other two are Centrumeiland and Buiteneiland. Centrumeiland will be located between Haveneiland and Strandeiland and will also function as residential island. Buiteneiland is situated at the north of Strandeiland and will not be residential area but a nature reserve with possibilities for recreation. The location of Strandeiland will be such that a view over the IJmeer and the Markermeer is possible. Strandeiland will be split in a north and south part by means of the so called Oergeul. On this Oergeul, which is the primeval gulch of an old river, building will be too expensive due to the soil type. Therefore no land is constructed above the Oergeul. On the western part of the island a small marina is projected. This marina is connected to the enclosed water in the middle of the island. The inlet on the eastern side of the island is separated from the included water by means of a dam. An overview of the location and the design of Strandeiland can be seen in Figure 2.1. Furthermore in Figure 2.2 an overview of the city of Amsterdam and the islands can be seen. The Oergeul is made darker blue in the figure. The focus of this research will be on the inlet circled in Figure 2.1 and the breakwater in front of this inlet. This inlet will be surrounded by walls with a berm in front of this wall. The idea is that this berm will function as a walking area for the people with less vertical distance from the water than what would be the case with just a vertical wall. Another concept is that the inlet will function as a marina in the future, and that the jetties will be attached to this berm. Whether the inlet will function as a marina is not decided jet. More information about the island can be found in (Uitgangspuntennotitie, 2018) and (Stedenbouwkundig plan Strandeiland, 2018).

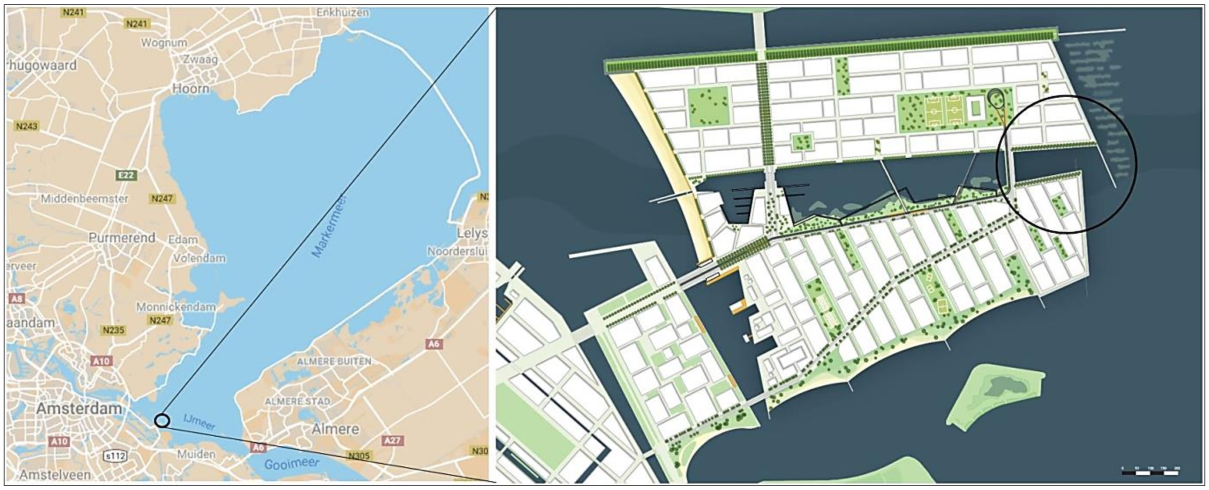

Figure 2.1: On the left the area where Strandeiland will be situated, on the right an overview of Strandeiland with the inlet circled

Figure 2.2: The city of Amsterdam and the islands at IJburg with the Oergeul made darker blue

2.1.1 Water level

Due to climate change longer periods of drought, mainly during the summer, occur in the Netherlands. Since the Ijsselmeer and the Markermeer function as important sweet water reservoirs for all kinds of purposes, Rijkswaterstaat decided to change the summer and winter water levels. Furthermore water levels are no fixed water levels, but bandwidths in between which the water levels can fluctuate. In summer (April-September) the water level is in between $-0.10m\ NAP$ and $-0.30m\ NAP$. In the months March and October the water level is in between $-0.10m\ NAP$ and $-0.40m\ NAP$. During the winter months (November-February) the water level will be kept in between $-0.20m\ NAP$ and $-0.40m\ NAP$. In summer the target water level is $-0.20m\ NAP$. In winter the minimum water level is $-0.40m\ NAP$, there is no target level in winter, just a level underneath which the water level is not allowed to drop. The average water level in winter over the last decennia is $-0.33m\ NAP$. More about the water level in the Markermeer can be found in (Peilbesluit IJsselmeergebied 2018).

For this research a water level of $-0.10m\ NAP$ is used. This water level is the highest possible water level in the area and can also be reached during stormy months like September. Also for wave agitation and the possible use of the inlet as a marina, the summer months are more interesting because these are most probably the busy months in the marina. This higher water level is chosen because it results in large wave heights.

2.1.2 The waterboard

Since the walls surrounding the inlet are primary flood defences, their responsibility lies with the waterboard. This means that the demands of the waterboard should be met. The most important demand for this case is that the breakwater should not be considered in the ultimate limit state (ULS). This means that when calculating the wave heights for the ULS, the breakwater should be deleted from the model. The waterboard has this as a demand because then the maintenance of the breakwater is not the responsibility of the waterboard. In the serviceability limit state (SLS) this breakwater can be taken into account, because then it is of no concern in terms of primary water defence but just about more daily use of the inlet. Another important demand is that the maximum wave overtopping over the vertical during ULS conditions is $10\ l/m/s$.

2.2 WAVES

This part is about the theory of waves needed for this research. This research deals with wind generated waves in shallow or intermediate depth in combination with an irregular shoreline. Processes like shoaling, refraction and diffraction play an important role in situations like these. Important for this research, and for the dimensioning of the inlet are among other the significant wave height (H_s) and the peak period (t_p). More about these methods of describing waves and about the processes that influence the waves can be found in (Holthuijsen, 2007). In this part the basic information is skipped and the focus lies on the aspects needed for this research.

2.2.1 Wave generation by wind

In (Holthuijsen, 2007) the principles of wave generation by wind are outlined. The wave generation is based on wind speed (U), fetch length (F) and in cases of shallow water also on the bottom depth (d). For this research the way the wave heights are determined in (Holthuijsen, 2007) is important in order to check or recalculate the boundary conditions concerning H_s and t_p. This subject is not discussed in more detail here.

2.2.2 Reflection

When a wave hits a vertical wall, it will almost fully reflect. This reflected wave will move away from the wall and when the incoming waves are almost regular, it will result in a standing wave. The resulting wave profile is the summation of both the incoming wave as the outgoing wave, described in the following formula:

$$\eta(x,t) = a_i \sin(\omega t - kx) + a_r \sin(\omega t + kx) \tag{1}$$

Where a_i is the amplitude of the incoming wave and a_r the amplitude of the outgoing wave. In case of full reflection, $a_i = a_r$. This results in:

$$\eta(x,t) = 2a_i \cos(kx)\sin(\omega t) \tag{2}$$

This is the basis of wave reflection, more about this can be found in (Holthuijsen, 2007).

Reflection coefficient of vertical wall with berm

In the provisional design by the Gemeente Amsterdam the vertical walls surrounding the inlet of Strandeiland will have a berm in front of them. When the water level is above the berm, the waves will transmit over the berm, reflect at the wall and transmit back over the berm. This transmission over the berm makes this a very complex system. In for instance (Dingemans, 1994) an estimation is made based on the linear wave theory to describe this transmission. In (Heijer, 2005) scale model tests have been done to estimate the reflection coefficient and wave overtopping for this combination of a wall with berm. First of all the influence of the relative berm depth h_b/H_s is considered, where h_b is the water depth on the berm. Logically, when this relative berm depth increases, the transmission increases because the waves feel the bottom to a lesser amount, and thus the reflection coefficient of the entire structure increases. Furthermore the influence of the wave steepness is considered. It is observed that when the steepness increases, the transmission coefficient decreases. Also the relative berm width L_B/L is considered, where L_B is the width of the berm an L is the wave length in front of the berm. It is observed that when the relative berm width increases, the transmission decreases. These three observation lead to the following transmission formula:

$$\frac{H_{s,kade}}{H_{s,haven}} = 1 + \mu\left(\frac{h_b}{H_{s,haven}} - 2\right) \qquad \left(for \frac{h_b}{H_{shaven}} < 2\right) \tag{3}$$

Where μ is determined as follow:

$$\begin{gathered} \mu = 0 \quad if \quad \frac{L_B}{L} < 0.117 \\ \mu = 1.2\frac{L_B}{L} - 0.14 \quad if \quad 0.117 \le \frac{L_B}{L} \le 0.533 \\ \mu = 0.5 \;\; if \; \frac{L_B}{L} > 0.533 \end{gathered} \tag{4}$$

This formula is based on a one way travel over the berm. If there is a reflecting wall at the back of this berm the wave will travel back and the length of the berm can be doubled. In this formula it is assumed that the negative amplitude of the incoming wave is in absolute sense smaller than h_b. From the results of the scale model tests in (Heijer, 2005) it can be observed that if the height of the incoming wave is larger than h_b, the reflection coefficient increases. This can be explained with the fact that part of the wave will immediately reflect against the berm. In (Heijer, 2005) can also be found what the wave overtopping over the vertical at the back of the berm will be under certain conditions.

Detecting reflected waves from wave signal

To separate the incoming and reflected from a surface elevation record, multiple of these surface elevation records on different locations are needed. In (Goda & Suzuki, 1976) a method using two locations is proposed. Later on in (Mansard & Funke, 1980) a least square method is used with three locations of records resulting in a more accurate result. In (Zelt & Skjelbreia, 1992) this number of records is enlarged and the approach is slightly different to obtain an even more accurate result.

Wave agitation and ship movement
Wave agitation in an inlet is dependent on the reflection coefficient of the walls surrounding the inlet. This wave agitation is important to determine the possibility of constructing a marina. In (Rosen & Kit, 1984) and (Isaacson & Mercer, 1982) respectively limitation factors for the movement of ships and the response of small ships to waves are determined. By using both researches, the suitability of the inlet to use it as a marina can be checked.

2.3 SWASH AND SWAN

There are various types of wave models all with their own specifications and specialism for a certain field. In this research the focus will be on a non-hydrostatic model (SWASH) and a spectral wave model (SWAN). The main difference between the two models is that SWAN is phase averaging and SWASH is phase resolving. In part 2.3.1 and part 2.3.2 the models are introduced individually. For a more complex geometry where diffraction plays an important role, phase resolving models are preferred. In this research one of the objectives is to see if a situation as introduced in part 2.1 can be modelled accurate by SWAN.

2.3.1 SWASH

SWASH (an acronym for Simulating WAves till SHore) is a non-hydrostatic model for simulating non-hydrostatic, free surface rotational flows in one and two horizontal dimensions. The model is intended to predict transformation of surface waves (The SWASH Team, 2018). It is mainly used for predicting wave development from offshore towards and till the shore, or the development in a harbour or basin.

In this section the attention is focussed on a two-dimensional horizontal physical domain. From the incompressible Navier-Stokes equations the nonlinear shallow water equations can be derived, which describes the depth-averaged, non-hydrostatic, free-surface flow and includes the conservation off mass and momentum. The equations are stated here:

$$\frac{\partial \zeta}{\partial t} + \frac{\partial hu}{\partial x} + \frac{\partial hv}{\partial y} = 0$$

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \zeta}{\partial x} + \frac{1}{h}\int_{-d}^{\zeta}\frac{\partial q}{\partial x}dz + c_f\frac{u\sqrt{u^2+v^2}}{h} = \frac{1}{h}\left(\frac{\partial h\tau_{xx}}{\partial x} + \frac{\partial h\tau_{xy}}{\partial y}\right) \tag{5}$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + g\frac{\partial \zeta}{\partial y} + \frac{1}{h}\int_{-d}^{\zeta}\frac{\partial q}{\partial y}dz + c_f\frac{v\sqrt{u^2+v^2}}{h} = \frac{1}{h}\left(\frac{\partial h\tau_{yx}}{\partial x} + \frac{\partial h\tau_{yy}}{\partial y}\right)$$

where x and y are located at the still water level and the z-axis is positive upwards. u and v are the depth-averaged flow velocities in x- and y-directions, respectively. ζ is the surface elevation and d is the water depth at rest. Furthermore is h the sum of ζ and d. q is the non-hydrostatic pressure, c_f is the dimensionless bottom friction coefficient and the $\tau_{xx,xy,yx,yy}$ are the different horizontal turbulent stress terms (Zijlema, Stelling, & Smit, 2011). These equations differ from the standard shallow water equations by adding the non-hydrostatic pressure, which is needed for modelling frequency dispersion. The viscous stresses are neglected since they are orders of magnitude smaller than the turbulent stresses. The surface elevation $\zeta(x, y, z)$ is described as a function of the horizontal displacement and time. Because of this, overturning waves cannot be simulated. Therefore breaking waves are treated as discontinuities in the flow with mass and momentum conservation. The energy dissipation is accounted for trough the analogy of hydraulic jumps.

In SWASH the partial differential equations are discretised in a staggered grid. Discretisation in the vertical direction is done through the so called multi-layer case where the computational domain is divided into a fixed number of terrain following layers. The time integration is done through an explicit leapfrog scheme where a first order implicit time step is used for the non-hydrostatic part, a first order explicit for the viscosity term and a second order explicit time step for advection. This scheme employs staggering in time.

At the open boundaries, incoming waves are described by means of wave spectra, time series or wave height and wave period. Land boundaries are made with the bathymetry. Structures like breakwaters can be made on top of the bathymetry, where the porosity can be selected. For partial reflection, sponge layers can be used. SWASH does not account for wave growth due to wind in the domain.

2.3.2 SWAN

SWAN (an acronym for Simulating WAves Nearshore) is a third-generation numerical model to simulate random, short-crested waves in shallow water (Booij, Ris, & Holthuijsen, 1999). These waves are described with the wave action density spectrum. The development and evolution of the wave spectrum is described by the spectral action balance equation, which is (Hasselmann et al., 1973):

$$\frac{\partial}{\partial t}N + \frac{\partial}{\partial x}c_x N + \frac{\partial}{\partial y}c_y N + \frac{\partial}{\partial \sigma}c_\sigma N + \frac{\partial}{\partial \theta}c_\theta N = \frac{S}{\sigma} \tag{6}$$

with $N(\sigma, \theta)$ the action density spectrum, $c(x, y, \sigma, \theta)$ the propagation velocity in respectively x, y, σ, θ space and S the source term in terms of energy density, representing generation, dissipation and wave-wave interactions. The dissipation term is a summation of three different factors, namely whitecapping, bottom friction and depth-induced breaking. Concerning the wave-wave interaction, this consist of quadruplet interactions (wave transition from the peak of the spectrum to the higher and lower frequencies) and triad interactions (transfer of energy from lower frequencies to higher frequencies).

The integration in all dimensions of equation 6 is done with finite difference schemes. Time is discretized with a constant timestep Δt so the source term and the propagation are integrated simultaneously. Concerning Δx and Δy, SWAN has a constant resolution in respectively x and y direction. Also the discretisation of the spectrum has a constant directional resolution $\Delta\theta$ and a constant relative frequency resolution $\Delta\sigma/\sigma$.

2.3.3 Port modelling

In (van Mierlo, 2014) research is done in wave penetration in ports. Van Mierlo uses 3 numerical models (SWASH, PHAROS and TRITON) to simulate an inlet with a breakwater in front of it and thereafter compares the results with a scale model. It is concluded that PHAROS is good for a quick estimate for wave heights in ports, but does generate reliable results when reflection coefficients start to play an important role. Furthermore it is concluded that SWASH is strongly preferred over TRITON. One of the reasons is that the schematisation of structures like breakwaters is more straightforward in SWASH and that the computational times are significantly lower.

In most of the research done in wave modelling in ports, it is concluded that using phase resolving models and especially SWASH is accurate, for instance in (Alabart et al., 2014) and (Vledder & Zijlema, 2014). This is mainly due to the fact that SWASH accounts more accurate for processes like diffraction and (partial) reflection. In (Battjes, 1994) it can be found that it are those effects that are most important when modelling waves in ports.

In the newer versions of SWAN diffraction could be included more accurate by the use of a phase-decoupled refraction-diffraction approximation (Holthuijsen et al., 2003). In (Enet et al., 2006) the applicability and stability of SWAN with the new diffraction effect is tested for various grid sizes. SWAN claims (The SWAN Team, 2019) that for grid sizes of $1/10$ to $1/15$ times the wave length, diffraction is determined accurate. This is indeed confirmed in (Kim et al., 2017).

SWAN is a less time consuming model than SWASH. Therefore it is interesting to use both models and see the difference in results. According to the studies mentioned above, both models should be capable of modelling the inlet at Strandeiland.

2.3.4 Hydra-NL

Hydra-NL is a probabilistic model intended to calculate, with given return periods, the water levels, significant wave heights, spectral wave periods, peak periods, hydraulic load levels, wave overtopping and wave conditions. These parameters can be requested at different locations, also locations around Strandeiland. The model outputs are based SWAN calculations at different points located on a map provided by Hydra-NL. The consistency of among other this tool is checked in (Slomp, Diermanse, & Waal, 2015). More information about the use of Hydra-NL can be found in (Rijkswaterstaat, 2018).

3 MODELLING

Simulating the waves in the inlet of Strandeiland with the numerical models SWASH and SWAN forms the core of this research. After the introduction in chapter 1 and the general information in chapter 2, chapter 3 deals with the modelling. This chapter discusses both the model set up and all its inputs, as the results of running the different models.

It is important to know why specifically those simulations discussed in this chapter are done. First of all, the situation for the ULS has to be known in order to dimension the walls surrounding the inlet. Due to the demands of the waterboard, which are discussed in part 2.1.2, it is not allowed to take the breakwater into account when dimensioning the walls inside the inlet. Therefore simulations without the breakwater are executed when results concerning the ULS are requested. For the more daily use of the inlet, like for instance when a marina is created inside the inlet, the breakwater can be taken into account. Therefore when results concerning the SLS are requested, simulations with a breakwater are executed. In order to check the necessity of this breakwater, also simulations with the SLS conditions without the breakwater are done. And last it is checked whether changing the length of the breakwater changes something concerning the wave agitation in the inlet. This calculation is of course also done using the SLS conditions.

The choice to use both models SWASH and SWAN is based on the following reasoning. Because of the sharp angels the waves need to make behind the breakwater and to enter the inlet, diffraction will play an important role in the evolution of the waves in the inlet. Since SWAN does not include diffraction, this might affect the results such that they are not accurate enough anymore. Therefore the more complex and time consuming model SWASH is also used. SWASH does calculate diffraction in a more complex and accurate way. The reason to not only use SWASH is that the comparison between these two models can be interesting. If the SWAN model appears to be accurate enough, then in other comparable situations only the use of SWAN can be sufficient.

The structure of chapter 3 will be as follows; first, in part 3.1, the general description of both models is given. This includes the layout of the area and the wave input at the boundaries. In part 3.3 and 3.2 respectively the SWAN and SWASH simulations are treated.

3.1 GENERAL

For SWAN and SWASH, parameters like the dimensions of the inlet and the wave input at the boundary are equal, because both models should simulate the same situations. These parameters are discussed in this chapter.

3.1.1 Lay-out

For the lay-out, two alternatives are used. One with a breakwater and one without a breakwater protecting the inlet. Note that when calculating the waves for the ULS, it is due to the demands of the waterboard not allowed to take the breakwater into account. Therefore only the waves with a small return period are treated for the case with the breakwater.

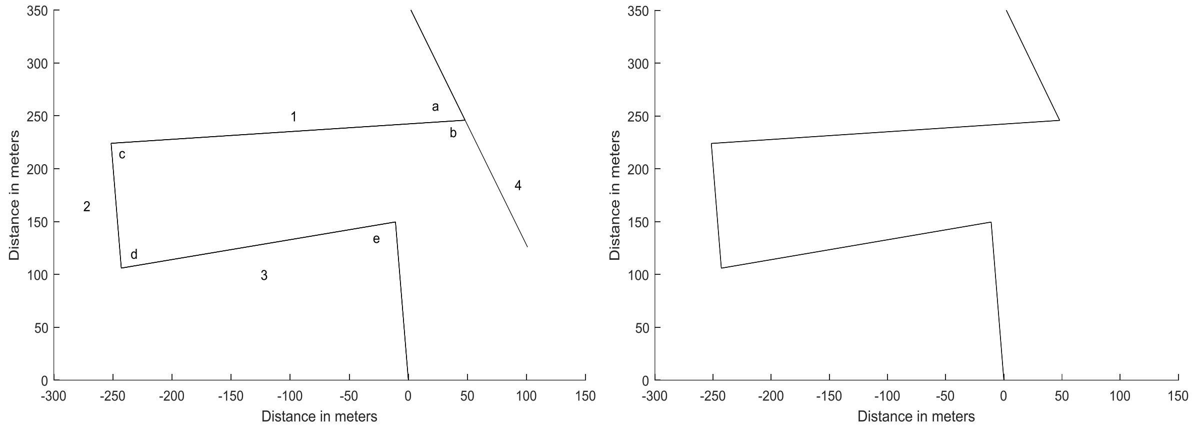

Figure 3.1 Inlet with breakwater (left) and without breakwater (right), with water on the right side of both figures and the north points upwards.

Location	Length (m)
1	300.32
2	118.32
3	236.06
4	130

Location	Angle $(°)$
a	70.9
b	109.1
c	90
d	83.5
e	83.5

Table 1: Length of segments from Figure 3.1 (left table) and angle of corners from Figure 3.1 (right table)

The walls surrounding the inlet are designed by the municipality of Amsterdam and will have a berm in front of them where people can walk on. In the provisional design this berm will have a height of $30cm$ above NAP. Furthermore it will have a width of $4m$. During storm conditions, this berm can become submerged due to set-up. Furthermore if the set-up is not high enough during storm conditions, waves still can overtop this berm. Therefore the berm will influence the reflection coefficient of the vertical wall. In part 3.1.2 this reflection coefficient is determined. It is also important to know if this berm will be accessible for pedestrians during more frequent storms, which will be determined during calculations for the SLS. The wall at the back of the berm is designed with a little angle. In the calculation concerning the reflection coefficient this wall is assumed to be perfectly vertical. This is done due to the limitations of the model calculating the reflection coefficient. Therefore the reflection coefficient will be overestimated, though it will be a very small difference. In Figure 3.2 a design of the wall with berm can be seen. This is a provisional design made by the municipality of Amsterdam.

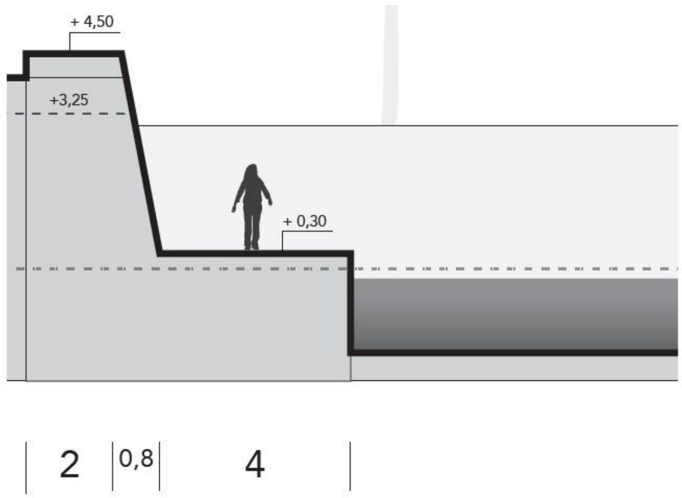

Figure 3.2: Provisional design vertical wall. The dotted line is NAP, with the values in the figure in meters related to NAP. The values underneath the figure are dimensions in meter.

3.1.2 Reflection coefficient vertical wall

As can be seen in part 3.1.1, the vertical wall is not a standard vertical wall from which the reflection coefficient is known. To be able to implement this wall in SWASH, the reflection coefficient has to be known. This is done by a 1D computation in SWASH, simulating a wave flume. This 1D mode to simulate a flume is also used in a benchmark test on the SWASH sourceforge page. In the flume test treated here, the berm in front of the wall is simulated by adjusting the depth of the flume. The berm has a height of $0{,}3m$ above NAP. So during the leading conditions, with a water level of $-0.10m\ NAP$, the berm will be $0.4m$ above the still water level (with no set-up). Furthermore the depth of the flume will be the same as in the inlet, namely $2.5m$. The vertical wall at the end of the berm is simulated using porosity. This is done by giving the wall a porosity of 0.01 and the rest of the area a porosity of 1. By using the porosity of 0.01, the wall is almost fully reflective (Reflection coefficient of 0.994 from SWASH test). The incoming waves have different periods and wave heights and will be regular. Concerning initial conditions, the water height will be $0.6m$, which is the same height as the set-up during a storm with a return period of 830 years. The input files can be found in appendix A. The reflection coefficient is obtained by first detecting the incoming and reflected waves using three different positions as described in (Zelt & Skjelbreia, 1992) and thereafter dividing the amplitude of the reflected wave with the amplitude of the incoming wave.

From a stability point of view, it is better not to have a sudden rise in bottom level like in the flume test. Therefore in the SWASH model it is preferred to simulate the reflection coefficient with porosity. This is also done using a 1-D SWASH computation. The porosity level is altered to obtain different reflection coefficients. From the results of this test it appears that when the width of the layer exceeds $6m$, the reflection coefficient is constant when widening the layer further. This test is repeated with the same wave heights and wave periods as in the previous flume tests.

The results of the test are showed in Figure 3.3. In this figure the blue and red line are respectively the upper and lower limit of the results the porosity test described above. The blue, green and red + signs are the results of the 1D SWASH computation for respectively SLS conditions (test 1), ULS conditions (test 2) and SLS conditions when no breakwater is included (test 3). The results of the tests are fitted in such a way that the maximum reflection coefficient does not exceed the upper limit. The lowest results are below the lower limit and therefore the porosity obtained from this test is conservative. The results of test 1 show a high reflection coefficient because here the water level is below the berm and therefore it can more or less be seen as a vertical wall. From the results it can be concluded that in the case of test 1 a porosity of 0.01 fits best, in case of test 2 a porosity of 0.15 fits best and in case of test 3 a porosity of 0.25 fits best.

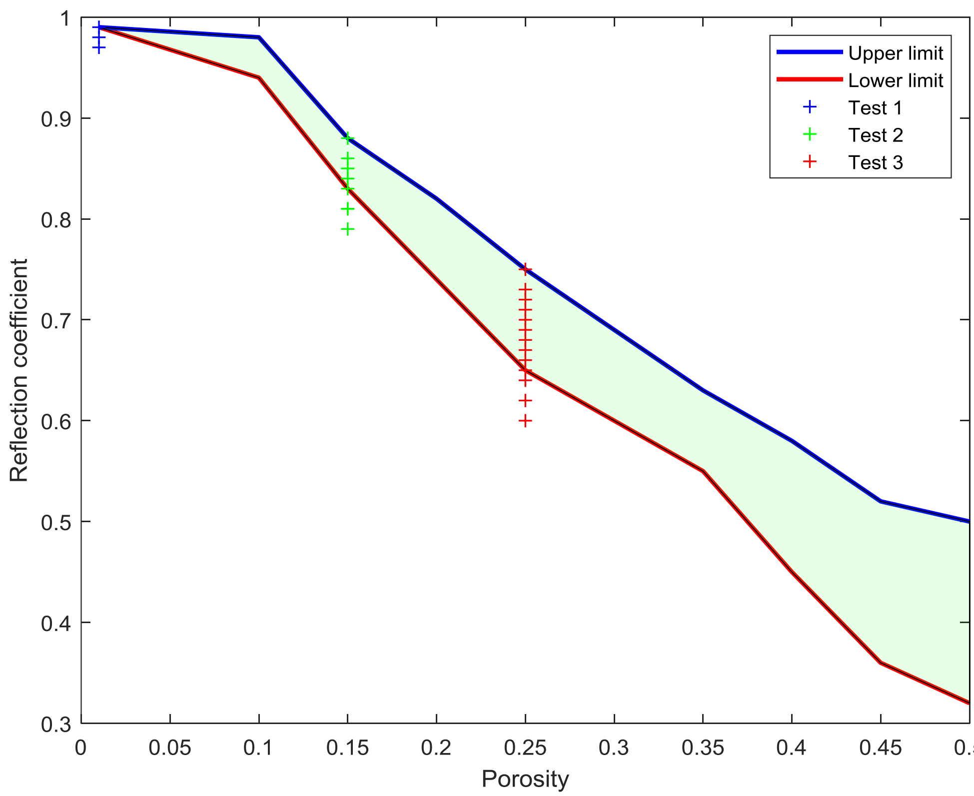

Figure 3.3: Results of the 1D SWASH computation with the different test results fitted in the reflection coefficient for a certain porosity. Parameters of the tests can be found in Table 2

	$H\ (m)$	$T\ (s)$	$Set\ up\ (m)$	$Water\ level\ (m\ NAP$
$Test\ 1$	$0.2-0.4$	$2.3-3.1$	0.2	-0.10
$Test\ 2$	$0.9-1.3$	$3.4-4.2$	0.6	-0.10
$Test\ 3$	$0.4-0.8$	$2.4-3.2$	0.6	-0.10

Table 2: Parameters for tests described in part 3.1.2.

3.1.3 Wave input

For determining the wave heights at the boundaries, the probabilistic model Hydra-NL is used (introduced in 2.3.4). Hydra-NL uses SWAN to calculate parameters like significant wave height and peak period. It extrapolates these calculations to obtain parameters for the higher return periods, but also for the lower return periods like 1 year. Here some accuracy gets lost and the effect of the water depth on the wave height and period is not taken into account with these extrapolations. Therefore all the input wave characteristics are checked whether they are realistic or not. This is done using (Holthuijsen, 2007). Here the method proposed by (Young & Verhagen, 1996) is used for depth limited growth. Furthermore it is checked if the wave steepness is realistic. In Table 3 the peak periods and significant wave heights used in the simulations are showed.

	North-east	**East**	**South-east**
H_s SLS (1 year)	0.7m	0.6m	0.3m
T_p SLS (1 year)	2.8s	2.8s	2.7s
H_s ULS (830 years)	1.2m	1.2m	0.9m
T_p ULS (830 years)	4.0s	3.8s	3.5s

Table 3: H_s and T_p for different return periods from different directions.

For the case without the breakwater (Figure 3.1 left) the significant wave heights and peak periods from the northeast, east and southeast with a small and large return period are considered (1 and 830 years respectively). For the case with breakwater (Figure 3.1 right) only the 1 year return period is considered, because the breakwater cannot be taken into account for the ultimate limit state.

In both SWAN and SWASH the significant wave heights from Table 3 are transformed to a spectrum. In SWASH this is done using a TMA spectrum. As stated in (Young & Verhagen, 1996) the TMA spectrum is suitable in a case with locally generated wind waves with a limited depth. In SWAN it is not possible to use a TMA spectrum and therefore a JOHNSWAP spectrum is used. This use of different spectra will have effect on the results of the different models. This is acceptable because both models should function at their best to be able to make a useful comparison.

3.2 SWASH

SWASH is the more time consuming model of the two. To make the SWASH model run, different input files are needed. Also translating the output files to useable files requires some steps. This will be explained in the coming parts.

3.2.1 Model set up

The SWASH model contains different input files. These input files need to fit on a grid. This grid, and also the input files, are made using Delft3D. For creating the grid, RGFGRID is used. QUICKIN is used to give values to all the grid points created in RGFGRID. In this way the other input files are created. Because of the lay out of the inlet, a curvilinear grid is more useful. This curvilinear grid is made in such a way that the vertical walls are in line with the grid lines. This ensures that the waves will reflect in the right direction. Using this grid the other input files are created, which simulates different aspects. First of all the hard structures like the vertical walls and the breakwater are modelled using a porosity input file (.por file). Using porosity instead of for instance bottom depth to simulate hard structures is preferred when concerning the stability of the model. The bottom depth is simulated using a .bot file. This bottom file is only accounted for in the wet parts, because in the dry parts the waves are blocked by the porosity. Furthermore the grainsize is simulated using a .psi file. The grainsize is equal everywhere except when concerning the breakwater, where the grainsize has such a size that large stones are simulated. An overview of the parameters of the different input files can be seen in Table 4. In appendix A screenshots of the input files made in RGFGRID and QUICKIN can be found.

Input files	Size/depth in water area	Size/depth in land area	Sice/depth in breakwater
Bottom depth (.bot)	2.5	2.5	2.5
Porosity (.por)	1	0.01-025	0.45
Grain size (.psi)	0.002	0.002	0.5

Table 4: parameters input file.

In appendix A the .sws, the swash file, can be found. In the file, first the grid and the other input files are introduced. After this the boundary conditions are created. These boundary conditions are mainly about the waves entering the domain. First the shape of the spectrum is chosen, which is the TMA spectrum in this case. A TMA spectrum fits better for wind waves in shallow water (Young & Verhagen, 1996). After that the boundary segments where the waves enter the domain and the wave characteristics like wave height, period and direction are chosen. These characteristics and segments differ for the different simulations. Then the sponge layers are introduced. These sponge layers prevent the outgoing waves from reflecting back in to the domain. So the sponge layers are situated on the outgoing boundaries. On which side these outgoing boundaries are, depends on the direction of the incoming waves and is thus different for the different situations. After inserting the boundary conditions, the different output files are requested. First the choice of on which grid points output is needed has to be made. This can be done by requesting a group of points or by requesting individual output points. Which one is needed depends on the situation. Furthermore different parameters can be requested, like for instance the significant wave height or the water level at certain time intervals. This also depends on what is needed and thus differs per situation.

During calculations for the SLS, a breakwater is included. This breakwater is modelled using a combination of adjusted bottom level and porosity. The edges of the breakwater have a porosity of 0.4, the core has a porosity of 0.01. With this settings a standard impermeable rubble mound breakwater is simulated. Furthermore the bottom level is adjusted in order to simulate diffraction around the breakwater.

The amount of layers in the vertical direction is set to 1. As can be seen in (The SWASH Team, 2018), the amount of layers is dependent on the value kd. This kd differs for the different situations but the highest value in all these situations is 1.1. When $kd < 2.9$, 1 layer is sufficient to achieve a maximum error of 3% in the normalised wave celerity ($= \frac{c}{\sqrt{gd}}$). Also, the number of layers is dependent on the ratio between depth and frequency. With a maximum frequency of $0.33HZ$ and a depth of $2.5m$, one layer is sufficient.

3.2.2 Simulations

For the different simulations some parameters vary. The key parameters that differ are the return periods of the wind/wave conditions (1 year and 830 year), the direction of the waves (NE, E and SE) and the lay-out (with or without breakwater and the length of the breakwater). Not all combinations are necessary to simulate. For instance it is not needed to simulate the case with the breakwater for the 830 year return period. This because it is not allowed to take the breakwater into account for the ULS. All the combinations simulated are showed in Table 5.

Return period: Breakwater:	830 year No	1 year No	1 year Yes, $130m$	1 year Yes, $100m$
North-east	1	4	7	10
East	2	5	8	11
South-east	3	6	9	12

Table 5: Different simulation with number.

For analyzing the results, different type of output is needed. When dimensioning the vertical wall at the end of the marina, only the incoming wave height is of importance. So the reflected wave height added by the incoming wave height, which can be seen in the output using the standard lay-out, is not useful in that case. In order to just have the incoming wave height in the results, the wall at the back of the marina is replaced with a sponge layer. By doing this, no waves are reflected back into the marina and therefore the incoming wave height near the vertical wall can be seen in the output data. This is only needed in the case without a breakwater, because that case will determine the height of the wall in the end of the inlet.

3.2.3 Results

From the results of the first runs came forward that the wave height, especially with the ULS conditions, imposed at the boundary could not be kept constant in the entire domain. This can be seen in the wave heights $200m$ in front of the inlet and in the entrance of the inlet in Figure 3.4. Here the H_s at the boundary is $1.0m$. As can be seen the wave height reduces in this stretch which indicates wave breaking. In the first case where 1 layer was used, this breaking went on until a H_s of about $35cm$ with a period of 4 seconds. This maximum wave height for a water depth of $3.1m$, including the set-up, is not realistic. When adding a second layer, the wave height in the domain increases, but not enough to make it realistic. Multiple modifications like increasing the depth at the edges of the domain and adjusting threshold parameters α and β (The SWASH Team, 2018) did not have the desired effect.

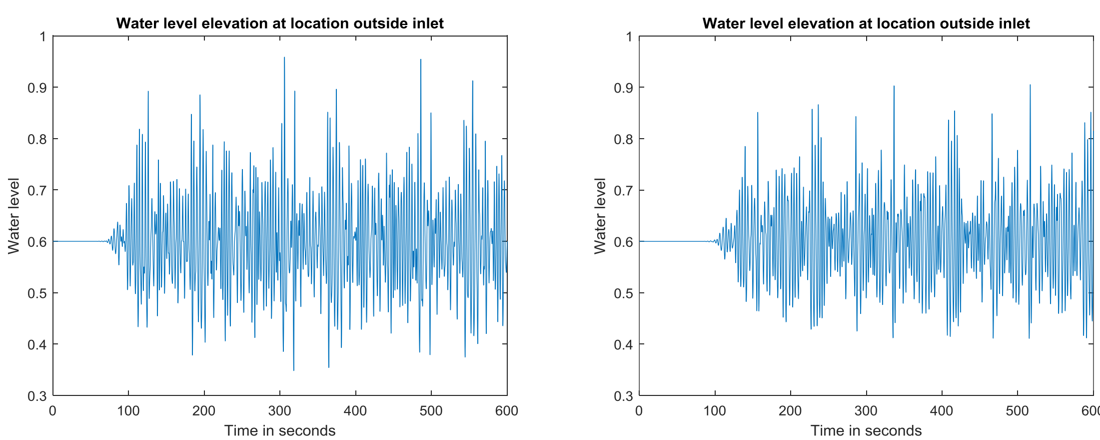

Figure 3.4: Water level elevation at location $200m$ outside the inlet (left) and just in front of the inlet (right)

From (Riedel & Byrne, 1986) it can be seen that a wave breaks when reaching a H/d ratio of 0.55, and in extreme cases with a ratio of 0.44. When checking the case with $0.6m$ set-up, a depth of $2.5m$ and a wave height of $1.2m$, a H/d ratio of 0.39 is found. This means that for this case, the waves should not jet be breaking. In part 0 the used wave heights can be found. Because it is not possible to simulate these waves in the domain, lower waves will be used and a multiplication factor is used to obtain the right wave height. This method is discussed in part 5.1.3.

Multiplication factor

The idea of the multiplication factor is simple. Multiple simulations with relative small wave heights are done. The wave height just outside of the inlet, and wave heights at different points inside the inlet are calculated. For all the points inside the inlet, a multiplication factor concerning the wave height is calculated, relative to the point outside the inlet. To make sure the multiplication factor is a useable and reliable factor, its consistency has to be checked. First a range of wave heights that can be imposed at the boundary is determined. This is done by checking that when the wave height at the boundary is changed with a certain factor, the wave height in the domain also changes with that same factor. When this does not happen anymore, the maximum wave height at the boundary is reached. This is the point where the wave breaking caused by the model start to play a role and the relation between the wave height at the boundary and the wave height in the domain is not linear anymore. This relation can be seen in Figure 3.5.

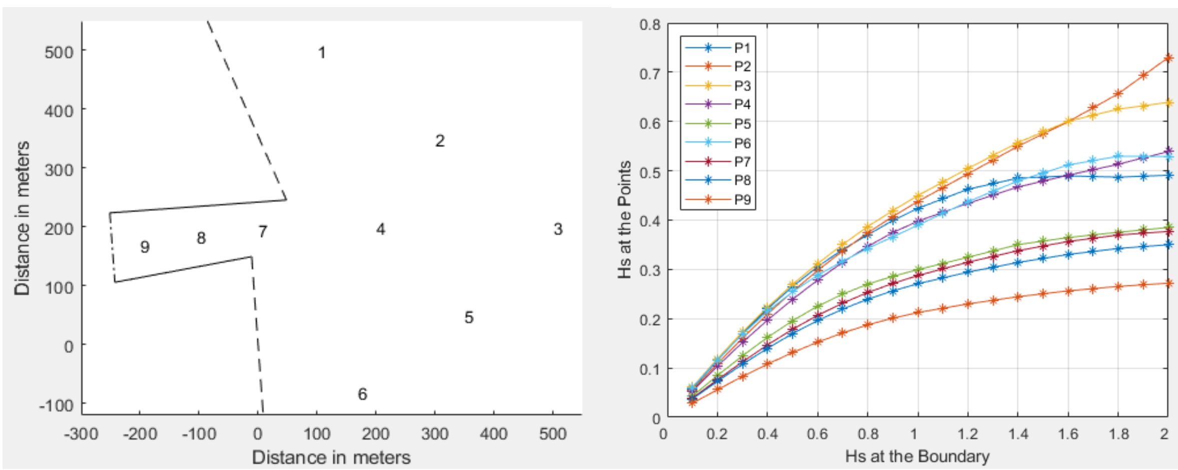

Figure 3.5: Left: locations of P1 to P9, dotted line for non-reflective wall, dot-stripe line for back wall which is in some cases non reflective. Right: relation Hs at the boundary and Hs at the locations P1 to P9.

As can be seen in Figure 3.5, the relation becomes non-linear for a wave height at the boundary of about $0.4m$. This means that when computing the multiplication factors, the wave height at the boundary should be kept below $0.4m$. The multiplication factor is a relation between the significant wave height outside the inlet and the significant wave height at some locations inside the inlet. The locations of these significant wave heights inside the inlet vary for the different situations and aims of the simulations. In the area outside the inlet, from where the significant wave height has to be known, this significant wave height varies for different locations close to each other. In order to determine the right significant wave height outside the inlet, a square area of 50 by 50 meters is analysed. From this area the average significant wave height is determined. Furthermore it is important that only the significant wave height of the incoming waves outside the inlet are included in the calculation. Therefore the outer walls and the back wall (respectively dotted line and dotted-striped line in Figure 3.5) are made non reflective. This is done by deleting the wall from the model and adding a sponge layer where the incoming waves will be absorbed. For the wave heights in the inlet it depends on the situation if an area or separate points are included. Furthermore it depends on the situation which walls should absorb the waves.

The downside of working with a multiplication factor is that it simplifies the complex non-linear way of modelling the area. It makes the solution linear for larger wave heights and this could result in unrealistic values. Mainly wave breaking is the missing element in this approach. Therefore it should always be checked if the waves in the results are possible under the given conditions. Furthermore the question if using this complex model is needed in this situation, if afterwards the results are recalculated in a linear way. This question will be discussed in part 5.1.

Model runs

In appendix B all the situations from Table 5 and their results are described. In this part it will be limited to those situations that eventually will determine the dimensions of the inlet. There are 2 situations that will be normative, namely situation 2 and situation 9 from Table 5. Furthermore situation 5 will be treated to check the effect of the breakwater and situation 12 will be treated to check the effect when adapting the length of the breakwater.

First situation 2. This is the situation where there is no breakwater. Furthermore it is the calculation for the ULS, which means that the return period of storm will be 830 years. From all the calculations it is concluded that a storm from the east is normative. The significant wave height during this storm is $1.2m$ and the peak period is $3.8s$. The goal of this simulation is to obtain the normative wave height hitting the back wall (coloured line in Figure 3.6) of the inlet because that is the wave height used for

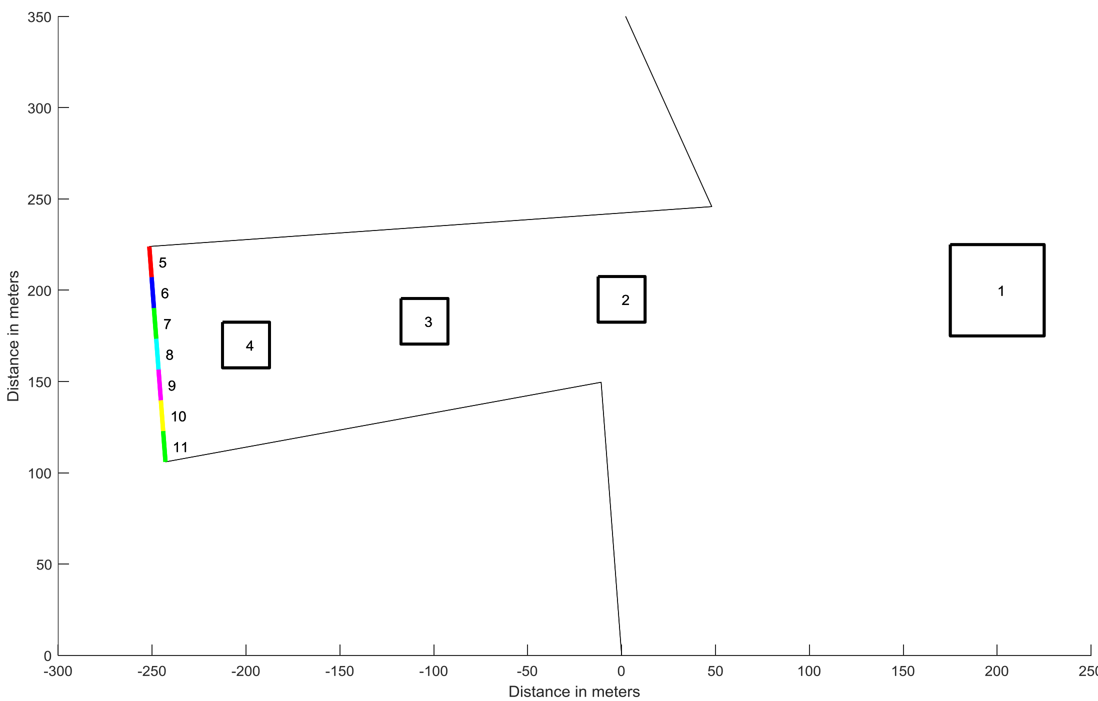

Figure 3.6: Lay-out for the ULS simulation with areas 1 to 4 to determine the H_s outside and inside the inlet, and segment 5 to 11 to determine the wave height at the wall. North is pointed upwards.

designing the height of this wall. In order to determine the significant wave height outside the inlet, the average significant wave height over an area of $50m \times 50m$ is taken. To do this without taking the reflected waves from the outer walls into account, these outer walls are deleted from the model and replaced by a sponge layer. From 3 areas in the inlet, the significant wave height is also determined. This is also done by taking the average over certain areas, in these cases areas of $25m \times 25m$. These 3 areas are analysed to see the development of the waves inside the inlet. Then the wave heights at the wall are determined. For this situation, only the incoming wave height is of importance. Therefore in the model this back wall is non-reflective. This non-reflectiveness of the back wall also makes sure that the significant wave heights in the inlet, as discussed before, are just the incoming wave heights. To determine the wave heights at this back wall, and to see the distribution of these wave heights, the wall at the back is divided in 7 parts. From all these 7 parts the maximum wave height from each separate part is determined. All that is described can be seen in Figure 3.6.

After calculating the values for the different locations, these values are converted to a value relative to location 1 in Figure 3.6. So the value in location 1 will have the value of 1, and all the other values in the figure are a factor of that value. This is done to make further calculation more easy and to give a clear view of how a wave evolves in the inlet. An overview of the results can be found in Table 6.

Location	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
$\boldsymbol{H_s}$	0.36	0.28	0.33	0.28	0.18	0.16	0.21	0.26	0.31	0.27	0.23
$\boldsymbol{H_s}$ **relative to 1**	1	0.87	0.91	0.77	0.51	0.45	0.57	0.71	0.86	0.74	0.64

Table 6: H_s at locations in Figure 3.6.

The last row from Table 6 is the multiplication factor. This factor is used to transform the wave height outside the inlet to the required wave heights inside the inlet. The normative H_s for waves coming from the east for the ULS is 1.2, as can be seen in Table 3. With the multiplication factor the significant wave heights for the ULS can be calculated. An overview is plotted on the back wall in Figure 3.7.

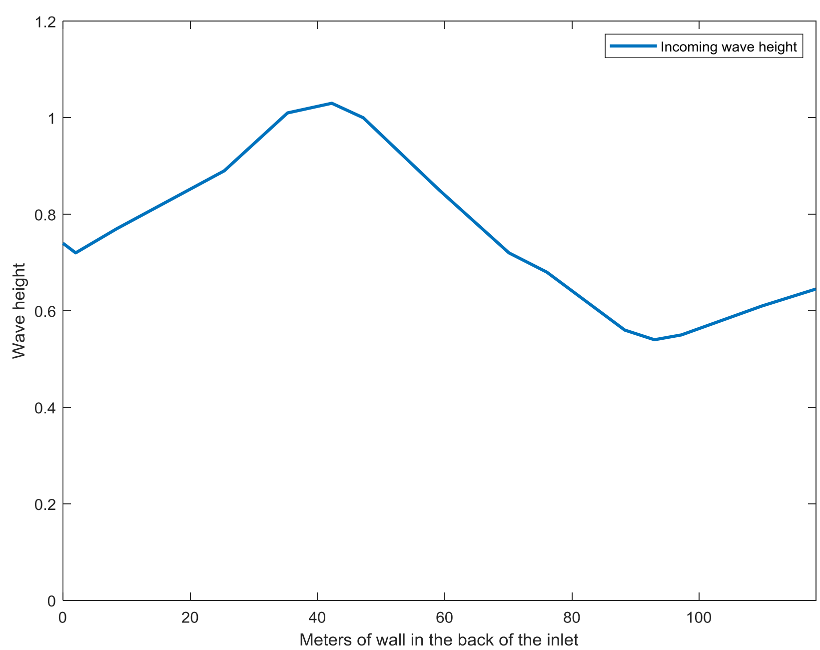

Figure 3.7: Incoming wave height projected on the wall at the back of the inlet for SLS conditions.

Situation 9 from Table 5 is the next normative situation. In this situation the SLS is simulated and a breakwater is included. From all the calculations can be concluded that the normative situation is with waves coming from the south-east. The H_s in this case is $0.3m$ with a t_p of $2.8s$. The main goal for this situation is to obtain information about wave agitation in the inlet. This is necessary to know for possible future uses of the inlet like for instance a marina. It is also interesting to know if the berm in front of the vertical wall will overflow often. The back wall is not open anymore in the model, which

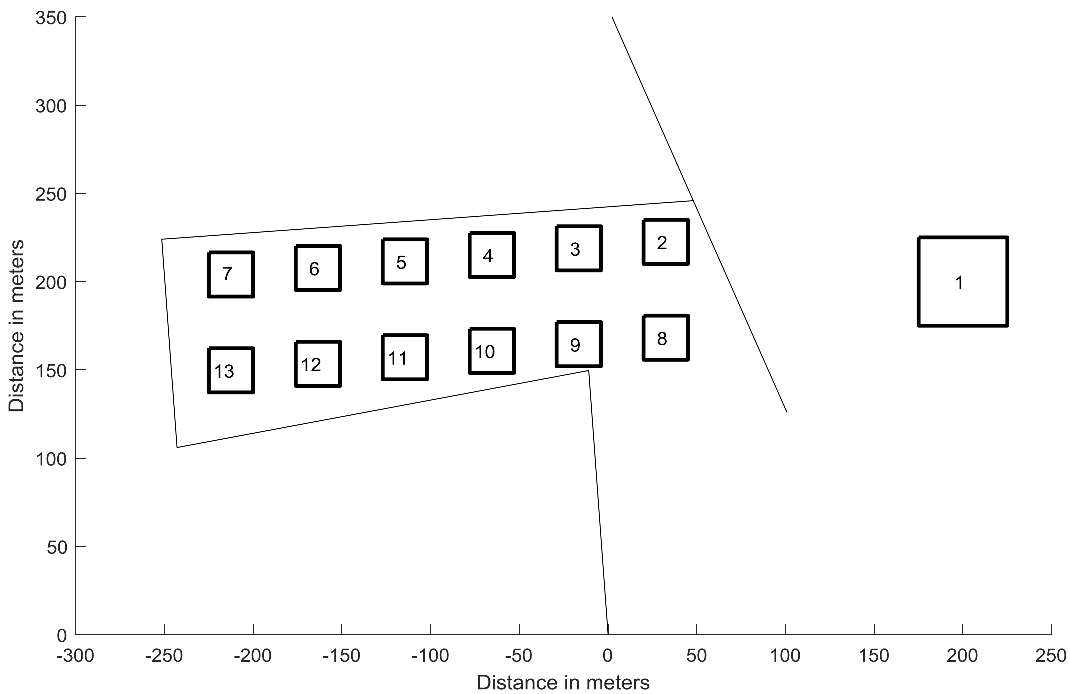

Figure 3.8: Lay-out for the SLS situation with areas 1 to 13 to determine the H_s outside and inside the inlet.

implies that this wall will reflect the incoming waves. This is done because in this situation it is not important to know the incoming wave height at the back wall. It is so that the overflowing of the berm can be caused by antinode created by a reflected wave. This will be higher than just the incoming wave and therefore it is not needed to know the height of this incoming wave. Furthermore the normative wave heights for designing the back wall are already determined in the previous calculation. The output requested in this situation is different than in the other situation. The wave heights inside the inlet will still be related to the wave height just outside the inlet. Therefore the area from which the average H_s outside the inlet was determined in the previous simulation will stay the same in this simulation. Inside the inlet there will be 12 areas of $25m \times 25m$ from which the maximum H_s is determined. Inside the inlet the maximum H_s is important because this indicates anti-nodes and will have most effect on the possible usage of the area. The area as described is made visual in Figure 3.8.

With this simulation the same is done as in the previous situation. The only difference is that in this case the wave height at the boundary is less than $0.4m$. As can be seen in Figure 3.5, this means that the wave height will be constant in the domain. Therefore it is not needed to calculate a multiplication factor. The right H_s can be imposed at the boundary. Because of a little loss in H_s directly at the boundary, a H_s of $0.34m$ as boundary condition is enough to obtain a H_s of $0.3m$ in the domain. The results of this simulations can be found in Figure 3.9.

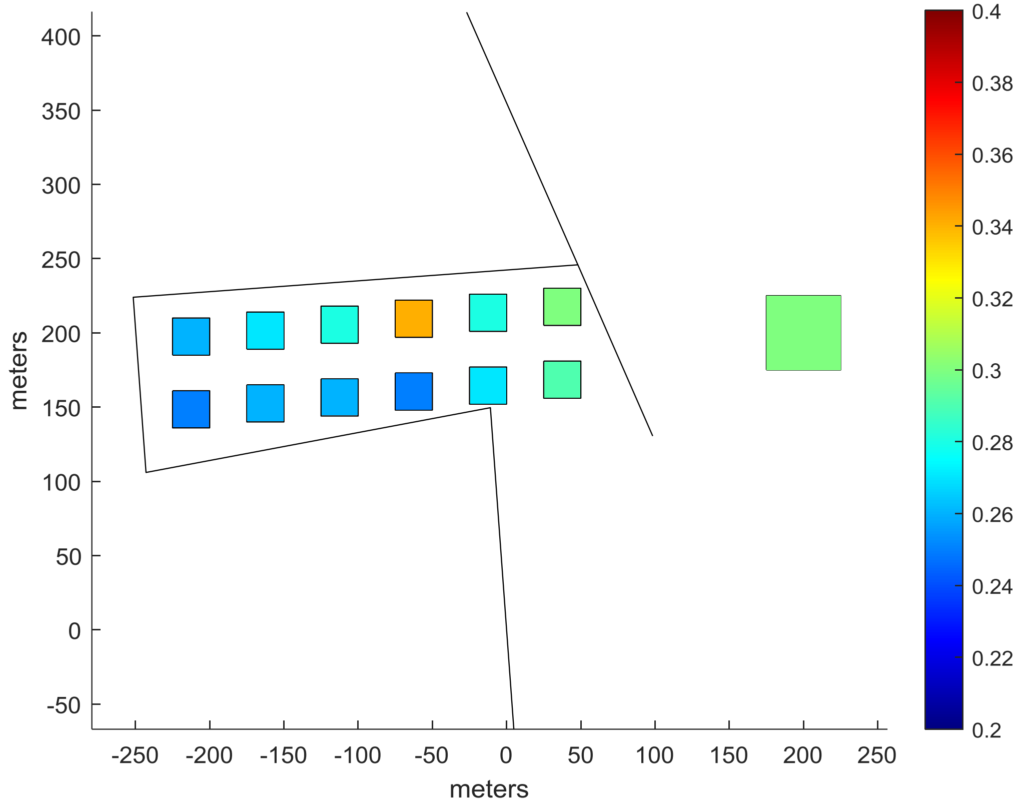

Figure 3.9: Wave height in the inlet for SLS conditions in SWASH.

In order to check if the breakwater indeed has its desired effect to limit the wave agitation for the SLS, a situation without breakwater in SLS condition is simulated. From the different calculations it is concluded that situation 5 is the normative condition concerning the wave height for the SLS with no breakwater. This means a H_s of $0.6m$ and a t_p of $2.8s$ from the east. To determine the H_s in front of the inlet, it is important to just have in incoming wave height and not the reflected wave height. Therefore first a run is done where both the back wall and the outer walls are non-reflective walls, as illustrated in the left part of Figure 3.5. After that a run is done where these walls are made reflective again. From that run the maximum significant wave heights of the different areas 2 to 13 are obtained. An overview of the situation can be seen in Figure 3.10.

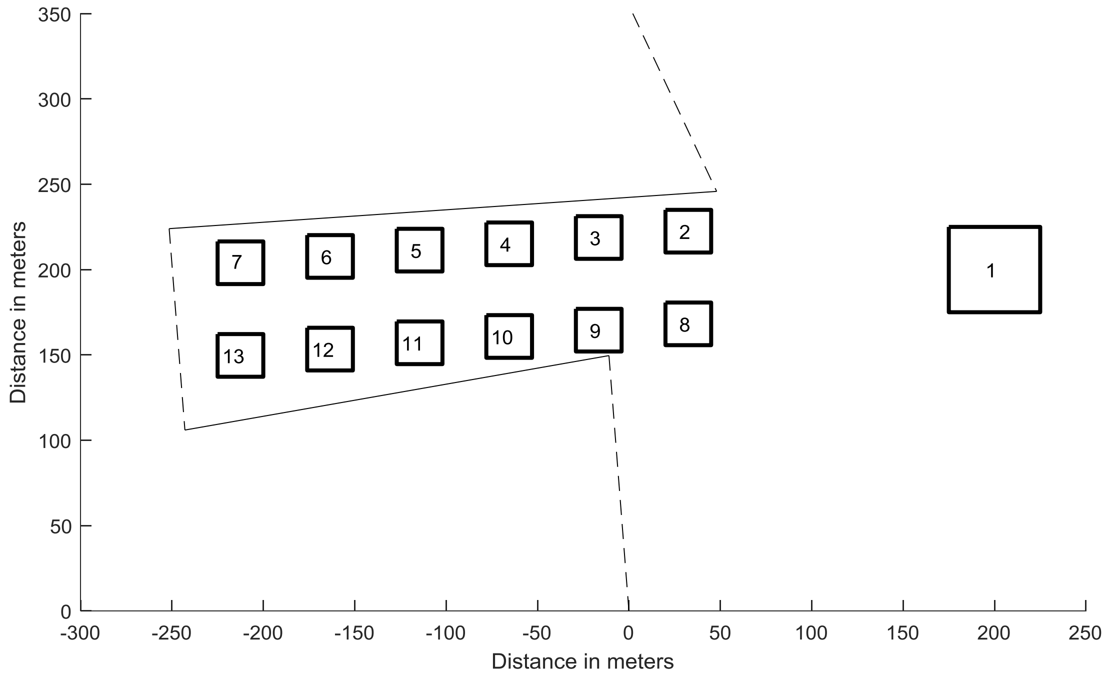

Figure 3.10: Lay-out SLS with dotted lines non reflective when H_s in area 1 is calculated, otherwise reflective.

In this situation the multiplication factor is needed again because the imposed H_s is $0.6m$, exceeding the limit of $0.4m$. The results of this simulation can be found in Table 7.

Location	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
$\boldsymbol{H_s}$	0.36	0.39	0.42	0.42	0.48	0.44	0.41	0.33	0.38	0.41	0.42	0.38	0.33
$\boldsymbol{H_s/H_s1}$	1	1.08	1.17	1.17	1.33	1.22	1.14	0.92	1.06	1.34	1.17	1.06	0.92

Table 7: H_s at the locations in Figure 3.10.

From the last row of Table 7, the multiplication factor, the significant wave heights in the inlet for the normative H_s can be calculated. The results are showed in the left part of Figure 3.12.

To check what adapting the length of the breakwater does, situation 11 is simulated. In this situation the length of the breakwater is reduced from $130m$ to $100m$. From the calculations it turns out that also with this shorter breakwater, the normative direction of the waves is south-east. Therefore just like in situation 9, the waves have a H_s of $0.3m$ and a t_p of $2.8s$. The main goal of this simulation is to check what the breakwater length does with wave agitation inside the inlet and therefore the significant wave heights at different locations inside the inlet is requested as output. This is also done by taking the maximum H_s at different locations with the same sizes as done in situation 9. Furthermore all the walls have a reflection coefficient like calculated before. Therefore the lay-out looks very similar to the lay-out of situation 9, but then with a shorter breakwater. This lay-out can be found in Figure 3.11.

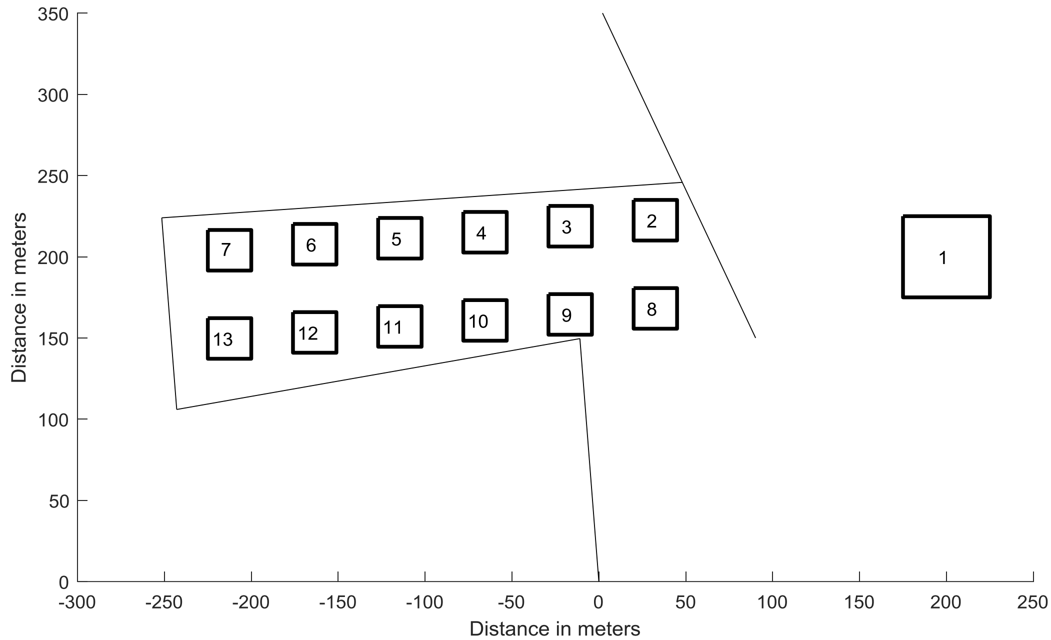

Figure 3.11: Lay-out with $100m$ breakwater and locations 1 to 13 to determine the significant wave heights inside and outside the inlet.

Because the waves imposed at the boundary are below $0.4m$, no multiplication factor is needed. Just like in the simulation of situation 9, a H_s of $0.34m$ is imposed at the boundary. The results of the simulation can be seen in the right part of Figure 3.12.

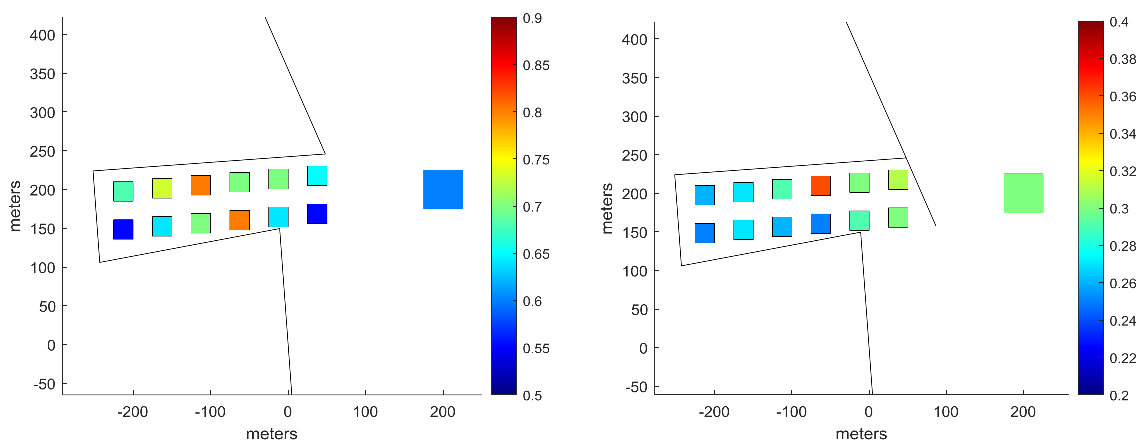

Figure 3.12 : Wave height in the inlet during SLS conditions for the situation without breakwater (left) and with shorter breakwater (right).

All the normative simulations for the ULS and SLS are discussed now. Also the check simulation concerning the necessity of the breakwater is discussed. In chapter 4 these results will be analyzed.

3.3 SWAN

Now the simulations in SWAN are treated. This will be done by briefly explaining the model set up and the different simulations. After that the results of these simulations are analysed.

3.3.1 Model set up

In the SWAN file first the grid is outlined. In this case it is a rectangular grid with a different size as in the SWASH computation, namely $600m \times 400m$. From (Kim et al., 2017) it can be concluded that when simulating a situation where diffraction plays a significant role a grid size of $1/10$ to $1/15$ times the wave length is best. The wave length in all the simulations varies between approximately 12 and 19 meters. Therefore a grid size of about $1.25m$ would be in the range. This results of a grid of 480×320 grid points.

The SWAN model is more easy to set up than the SWASH model. No curvilinear grid is needed. All the walls can be imposed as obstacles. These obstacles do not have to be in line with the grid lines. SWAN simply calculates the angle of incidence and reflects the waves with the same angle. At this boundary also the reflection coefficient can be imposed, so this needs no further calculations like in SWASH. Also the breakwater can be imposed as an obstacle, but then with a reflection coefficient of 0.4. This reflection coefficient is a little conservative, obtained from (Zanuttigh & Meer, 2008). At the boundary different wave heights can be imposed using a JONSWAP spectrum. In SWAN it is not possible to impose a TMA spectrum.

3.3.2 Simulations

The simulations done in SWAN are similar to that in SWASH because the results should be comparable and because these are the simulations needed to find the normative wave heights for all situations. So Table 5 also counts for the SWAN computations. What makes it more easy is that when the walls should be non-reflective, the reflection coefficient of the obstacles can simply be adjusted. This is for instance the case when the incoming wave height at the back wall is needed. The breakwater can be added and removed by adding or removing an extra obstacle.

3.3.3 Results

Unlike in the SWASH model, in the SWAN model the H_S stayed constant in the domain. It only reduced a little, close to the boundary. So for all the simulations with different H_S in the domain, calibration is done. For the 3 situations described in this part, the imposed boundary conditions for H_S leading to the right H_S in the domain, can be found in Table 8.

Situation	$\boldsymbol{H_S\ (m)}$ **in the domain**	$\boldsymbol{H_S\ (m)}$ **at the boundary**
ULS east	1.2	1.38
SLS east	0.6	0.67
SLS south-east	0.3	0.32

Table 8: H_S at the boundary after calibration

Model runs

Like in the SWASH model, the situations 2 and 9 are the normative situations. Furthermore situation 5 is the situation to check the necessity of the breakwater and situation 11 is done to check what the effect is of adapting the length of the breakwater. So from the calculations can already be concluded that the same wave directions are normative for both models.

First situation 2 is described. In this situation it is important just to take the incoming wave height into account, because this situation is used for the dimensioning of the walls surrounding the inlet. So the back wall and the outer walls are all non-reflective (the dotted lines in Figure 3.10). This is done by setting the reflection coefficient of the obstacles simulating these walls to 0. Furthermore the areas from where the wave heights are obtained are similar as in the simulation of situation 2 in SWASH. Also the segments covering parts of the wall at the back of the inlet are the same in this simulation. Therefore also Figure 3.6 is applicable for this situation. The waves are coming from the east and have a H_s of $1.2m$ and a T_p of $3.8s$ inside the domain. Since this is not a multiplication factor, this is directly the final result of the simulation and they are plotted in Figure 3.13.

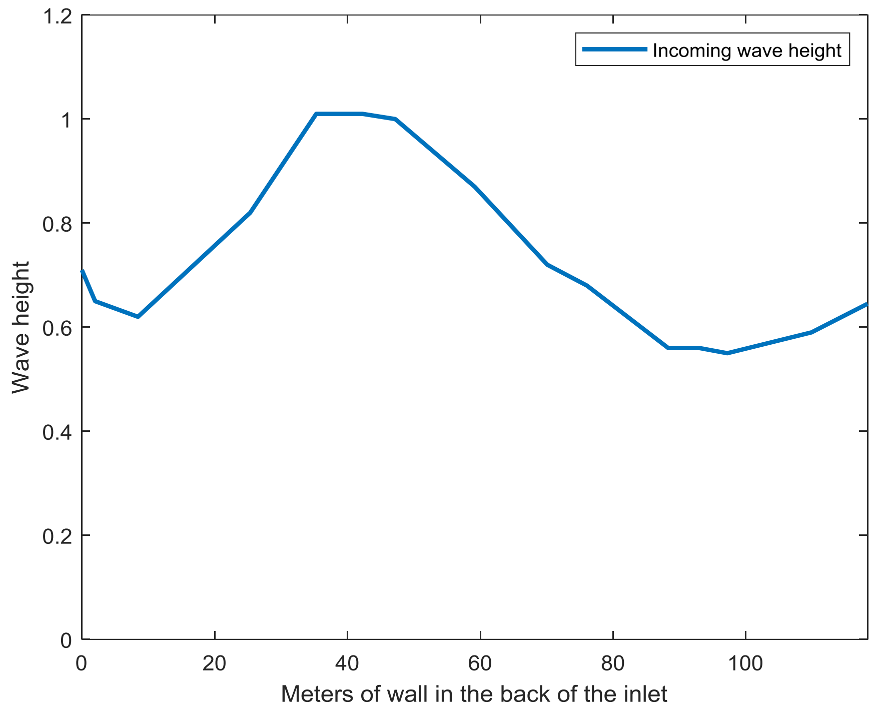

Figure 3.13: Incoming wave height projected on the wall at the back of the inlet for SLS conditions.

For the simulation of situation 9, the same lay-out is used as in SWASH, which can be seen in Figure 3.8. For the breakwater 3 obstacle lines are used to make the shape of a rectangular. This is done to make it a 2D structure, with an actual width, in this case $15m$. The reflection coefficient of these obstacles is set to 0.4. The outer walls are fully reflective and the inner walls have a reflection coefficient of 0.88. The set-up in this situation is, just like in the comparable situation in SWASH, $0.2m$. Concerning the areas from which the output is requested, they are the same as in the comparable situation in SWASH and can be seen in Figure 3.8. For this situation also no multiplication factor is needed. The imposed waves are coming from the south-east and have a H_s of $0.3m$ and a t_p of $2.8s$ inside the domain. The results of the simulation can be found in Figure 3.14.

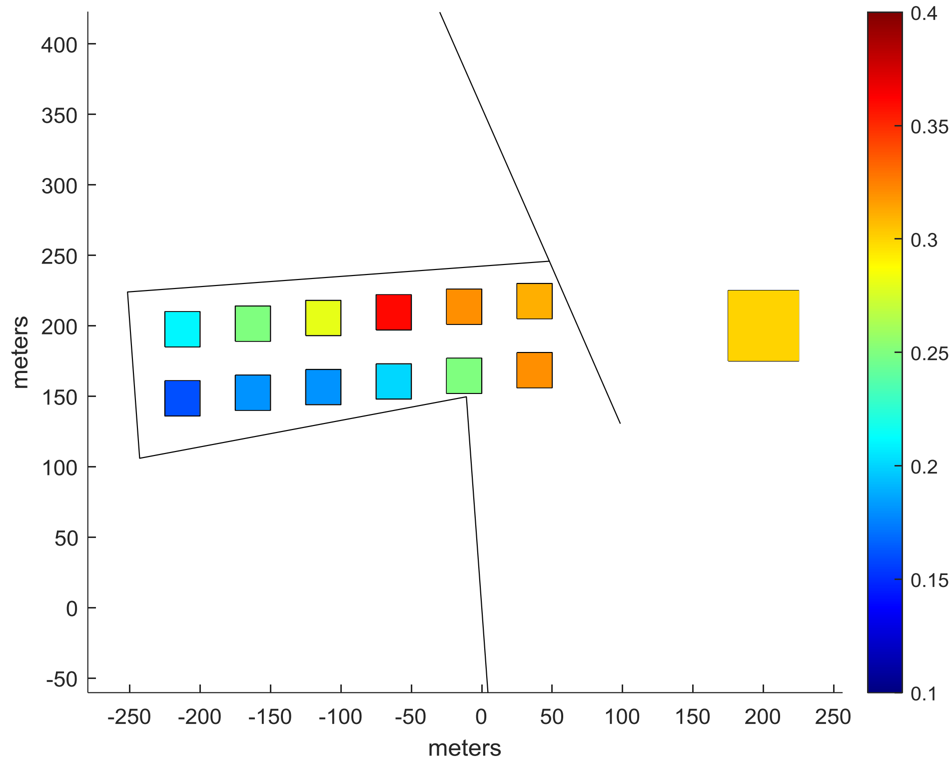

Figure 3.14: Wave height for the SLS conditions in SWAN

To check the effect of the breakwater, the normative situation for the SLS in the case without a breakwater is simulated in situation 5. For this simulation the same lay-out as in the SWASH run is used which can be seen in Figure 3.10. Also the same locations to obtain the H_s from the figure are used. When calculating the H_s outside the inlet, the dotted walls are made non-reflective. When calculating the H_s inside the inlet these walls are reflective like in the other cases. The waves are coming from the east and have a H_s of $0.6m$ and a t_p of $2.8s$. During this simulation again no multiplication factor is needed. The results of this simulation are showed on the left of Figure 3.15.

In situation 11 the effect of adapting the length of the breakwater is checked. This is done in the same way as that it is done in the SWAN simulation of situation 9, but then with a $100m$ breakwater instead of a $130m$ breakwater. The H_s in the domain should be $0.3m$. From Table 8 it can be seen that the imposed H_s at the boundary should be $3.2m$. The results of this calculation are shown on the right of Figure 3.15

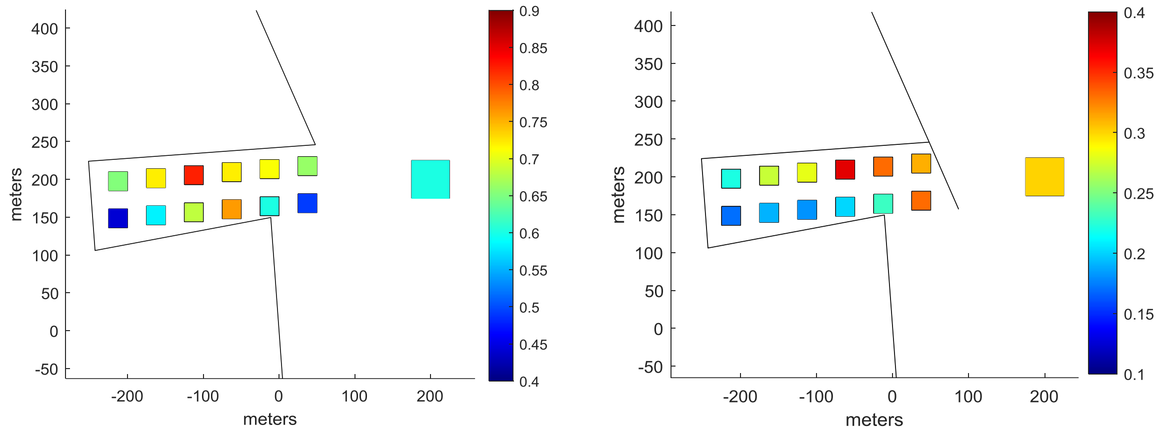

Figure 3.15: Wave height in the inlet during SLS conditions for the situation without breakwater (left) and with shorter breakwater (right).

4 ANALYSIS

Now that the modelling of the inlet for all the different situations is done, the results of the simulations are analysed. First this is done for the vertical wall with berm under ULS conditions. This is a complex situation where the waves first will transmit over the berm and then hit the vertical wall at the back of the berm. Part of the wave might overtop the wall, the rest will reflect and dissipate further on the berm until it reaches the deeper water again. This complex situation will be analysed for different dimensions of the wall and the berm. After that the wave agitation in SLS conditions will be analysed and the effect of that on a possible marina. Wave agitation will induce ship movement and to what extend that is acceptable will be discussed. Now the only thing left to analyse is the breakwater. The question is asked whether a breakwater is needed and if so, what the length of the breakwater does to the agitation in the inlet.

When all the analysis about the lay-out of the inlet are done, the difference in results between SWAN and SWASH will be analysed. This will done by analysing the difference due to diffraction. Furthermore the presence of steep waves will be discussed and the use of SWASH for 1D computation will be treated.

4.1 ANALYSIS RESULTS FOR DESIGN INLET

From the results obtained in chapter 3 the answers concerning ULS and SLS condition can be provided. First in part 4.1.1 the dimension concerning the walls for the ULS conditions are provided. These will mainly include the height of the walls and the shape of the berm. Then in part 4.1.2 the advice concerning wave agitation during SLS condition is provided. In part 4.1.3 the necessity of the breakwater and the length of the breakwater will be discussed.

4.1.1 Wall analysis ULS

Designing the wall for the ULS condition includes two aspects: dimensioning the berm in front of the wall and dimensioning the needed height of the wall behind the berm. The combination of the two should prevent exceeding the overtopping limit, which is $10\ l/m/s$ under ULS conditions. As stated in (Heijer, 2005), there are two ways to approach this combination and determine the wave transmission over the berm and the wave overtopping over the vertical wall. These two ways are as follows:

1. The transmission of the waves over the berm is seen as a disconnected process. After this the wave overtopping over the wall is calculated. When using this method, the H_s at the berm should be used to calculate the wave overtopping over the wall.
2. The process of transmission over the berm and overtopping over the wall is seen as one process. In this case the H_s in front of the berm should be used.

This approach is also used by (van der Meer, 1997) but then for a dike with berm. When the length of the berm is smaller than a quarter of the incoming wave length, then the second approach is used. In this case the wave length is about $19m$ and the width of the berm is designed as $4m$, which is less than a quarter of the wavelength. Furthermore the H_s at the berm is very unstable and therefore hard to determine accurately. The results would become unreliable when using this H_s. Therefore approach 2 is used. Among others in (van der Meer, 1997) the following definitions are used:

$$R = \frac{h_c}{H_s}$$
$$Q = \frac{q}{\sqrt{gH_s^3}} \tag{7}$$

Where:

R	:	$Dimensionless\ wall\ height\ (-)$
h_c	:	$Wall\ height\ relative\ to\ SWL\ (m)$
H_s	:	$Significant\ wave\ height\ (m)$
Q	:	$Dimensionless\ wave\ overtopping\ (-)$
q	:	$Wave\ overtopping\ (m^3/s/m)$

In (Heijer, 2005) these definitions form the basis of experiments done to calculate the wave overtopping over among others a vertical wall on a berm. The formulas resulting from these experiments are divided in formulas that count when the waves break on the berm and formulas that count when the waves do not break on the berm:

Waves that break on the berm $\left(\xi_{eq} <\approx 2\right)$:

$$Q_b = 0.06 \exp\ (-4.7\ R_b)$$

where:

$$R_b = \frac{h_c}{H_s} \frac{\sqrt{s_{op}}}{\tan \alpha_{eq}} \frac{1}{\gamma_{total}} \tag{8}$$

$$Q_b = \frac{q}{\sqrt{g\ H_s^3}} \sqrt{\frac{s_{op}}{\tan \alpha_{eq}} \frac{1}{\gamma_b}}$$

Where:

ξ_{eq}	:	$Breaker\ parameter = \tan \alpha_{eq} / \sqrt{s_{op}}\ \ (-)$
s_{op}	:	$Wave\ steepness = H/L\ (-)$
$\tan \alpha_{eq}$	:	$Equivalent\ slope\ (-)$
γ_{total}	:	$Total\ of\ the\ reduction\ factors\ (-)$

Waves that do not break on the berm ($\xi >\approx 2$):

$$Q_n = 0.2\exp(-2.3\,R_n)$$

where:

$$R_n = \frac{h_c}{H_s}\frac{1}{\gamma_{total}} \tag{9}$$

$$Q_n = \frac{q}{\sqrt{g\,H_s^3}}$$

Formulas (8) and (9) are also useable for sloping compositions and therefore the equivalent slope ($\tan\alpha_{eq}$) is added to the formula. The equivalent slope is defined by the following formulas:

$$\begin{aligned} \tan\alpha_{eq} &= \left(\frac{|h_B|}{2H_s}\right)\tan\alpha_{verticall\ wall} + \left(1-\frac{|h_b|}{2H_s}\right)\tan\alpha_{berm} && if\ (|h_B| < 2H_s) \\ \tan\alpha_{eq} &= \tan\alpha_{verticall\ wall} && if\ (|h_B| \geq 2H_s) \end{aligned} \tag{10}$$

Where:

$\tan\alpha_{verticall\ wall} = 1$

$\tan\alpha_{berm} = \dfrac{2H_s}{B_{berm} + 2H_s}$

$h_B = SWL - h_{berm}$

This equivalent slope is important because the influence of the berm decreases when the water level relative to the berm height increases. When the water level above the berm is more than 2 times the H_s, the influence of the berm can be neglected. This will never be the case in this research. The equivalent slope can be seen as a fictional quantity which makes it possible to use the same calculation method as for dikes. It includes both the width of the berm as the height of the berm relative to the water level. With this equivalent slope the breaker parameter is calculated as can be seen in equation (8). From (Heijer, 2005) also the reduction factor of the berm γ_k is obtained, namely:

If $\left(\frac{h_B}{H_s} \leq 0.5\right)$ then $\gamma_k = 0.6$ (11)

With formulas (8), (9), (10) and (11) the dimensions of the vertical wall and berm can be optimized and dimensioned for a certain amount of wave overtopping. This is done for a variety of berm width, berm height and wall height. For the H_s the maximum at the back wall from Figure 3.7 and Figure 3.13 is used. This is for the ULS conditions. Furthermore a set-up of $0.6m$ is used what leads to a water depth of $3.0m$. The results of the calculations can be seen in Table 9.

$H_s = 1.03m \quad t_p = 3.8s \quad \gamma_k = 0.6$											
h_b	B_{berm}	h_c	h	$\tan\alpha_{eq}$	ξ_{eq}	R_b	Q_b	q_b	R_n	Q_n	q_n
0.2	4.0	0.5	3.0	0.44	1.81	0.44	0.0073	0.041	0.81	0.0311	0.102
0.2	4.0	1.0	3.0	0.44	1.81	0.90	0.0009	0.005	1.62	0.0048	0.016
0.2	4.0	1.5	3.0	0.44	1.81	1.34	0.0001	0.0006	2.43	0.0008	0.0025
0.2	4.0	2.0	3.0	0.44	1.81	1.79	0.0000	0.0001	3.24	0.0001	0.0004
0.2	3.0	1.0	3.0	0.46	2.08	0.78	0.0015	0.009	1.62	0.0048	0.016
0.2	5.0	1.0	3.0	0.36	1.61	1.00	0.0005	0.003	1.62	0.0048	0.016
0.1	4.0	1.0	3.0	0.37	1.66	0.97	0.0006	0.003	1.62	0.0048	0.016
0.3	4.0	1.0	3.0	0.44	1.95	0.83	0.001	0.007	1.62	0.0048	0.016
0.1	4.0	1.1	2.9	0.37	1.65	1.08	0.0004	0.002	1.78	0.0033	0.011

Table 9: Results wall dimension calculations for ULS

The experiments from (Heijer, 2005) can be used within certain validity areas. These validity areas are formulated as follows:

- Relative wall height : $1 < h_c/H_s < 3$
- The berm must be submerged : $h_b > 0$
- Relative berm length : $0.25 < B_{berm}/L < 0.5$
- Wave steepness : $0.01 < s_{op} < 0.04$
- Equivalent slope : $0.25 < \tan\alpha_{eq} < 0.65$
- Breaker parameter : $1.5 < \xi_{eq} < 5$

When checking the values most of them are in the validity area, except for the relative berm length (0.2 when berm is $4m$) and the wave steepness (0.049). Concerning the wave steepness, this means that the wave are steeper in this research than in the experiments of (Heijer, 2005). This would imply that when travelling over the berm, the waves will start breaking sooner. To compensate for that the limit for the breaker parameter for which calculations are done with non-breaking waves, which is 2 now, should be increased. In this research that would mean that for all the combinations in Table 9 the waves will be breaking. Furthermore the exceedance of the maximum wave steepness is limited and therefore it is safe to use this method for calculating the overtopping. Nevertheless it will reduce the accuracy and this should be included in the conclusions concerning the final design. Concerning the relative berm length, this can be compensated by calculating the overtopping over a berm with $B_{berm}/L = 0.25$ and linearly interpolate this result with a calculation as if there is no berm, which can be done using (Franco, Gerloni, & Meer, 1994). When doing this it can be seen that when $B_{berm}/L = 0.2$, the difference between using this interpolation method or using the calculations as done in Table 9 is of an order of magnitude of 1%. Therefore the values from Table 9 are used.

Al the results from Table 9 are easy to explain. First of all when increasing h_c, the wall height measured from the SWL, the amount of overtopping decreases. This of course makes sense. Increasing the berm width decreases the amount of overtopping because the waves can break for a longer stretch and therefore more energy gets dissipated and the wave height is less when reaching the wall. When decreasing the berm width the same happens the other way around. When decreasing the h_b, the water level measured from the berm, the amount of overtopping decreases, because the energy dissipation due to breaking increases. When the water level (h) decreases, what happens in the last row of Table 9, the overtopping also decreases. This is because of multiple reasons but the most important is that when decreasing the water level this logically also leads to a decrease of h_b.

As can be seen in Figure 3.2 the wall height in the provisional design is $4.5m$ above NAP. With a set-up of $0.60m$ and a SWL of $-0.10m\ NAP$, this would mean a h_c of $4.0m$. This would mean, no matter the dimensioning of the berm, that there will be almost no wave overtopping using this design. Keeping in mind that the maximum allowed wave overtopping is $10l/m/s$, the minimum wall height would be about $1m$ (above the berm). Therefore the dimensioning of the berm can be based on other criteria and with that design the minimal wall height can be calculated. In part 5.2 the different considerations concerning designing the optimal berm are discussed.

4.1.2 Wave agitation SLS

For analysing the wave agitation in the inlet during SLS conditions the results from Figure 3.9 and Figure 3.14 are used. The main reason to analyse the wave agitation in the inlet is because in the future the inlet might be used as a marina. In this section it will be checked if the wave conditions are suitable for a marina. It has to be mentioned that the approach used in this section is just to give a general idea of the ship response to the waves in the inlet. Ship movement is also dependent on the shape of the ship, not just the length as assumed in this section. Taking that into account would be to detailed and beyond the scope of this research. Nevertheless the results of this analysis are valuable for a good estimation of the suitability of the inlet as a marina.

In (Rosen & Kit, 1984) safety criteria concerning the movement of small ships in a marina are determined. It is assumed that the main factor in human safety and behaviour on a ship is the acceleration of the human body, and not its velocity. Therefore the limitation factors are in most cases accelerations and established as follows:

- Maximum linear acceleration : $0.4\ m/s^2$
- Maximum angular acceleration : $2.0\ deg/s^2$
- Maximum peak to peak roll : $6.0\ deg$

The last criteria is needed to prevent entanglement of masts while the vessels are moored. With a maximum H_s of 0.34 in the inlet, the wave steepness will be low, namely 0.14. Therefore both the maximum angular wave acceleration as the peak to peak roll will not be low enough. Furthermore, the roll acceleration and the peak to peak roll is also influenced by the wind in the masts and therefore giving an accurate advise on these factors would be beyond the scope of this research. In (Isaacson & Mercer, 1982) the relation between wave amplitude and ship motion amplitude is researched. This relation is as follows:

$$RAO = \frac{motion\ amplitude}{wave\ amplitude} \tag{12}$$

Where RAO stands for response amplitude operator. This RAO value is a value dependent on the wave length, waterplane length of the ship and the steepness of the waves. In this section the wave steepness is relatively low and therefore only the low steepness measurements are accounted for. First the motion amplitude of the ship is calculated. After that the maximum acceleration of the ship is calculated using the same formula as for wave motion because it can be assumed that the ship makes the same motion as the wave, but then dampened (Isaacson & Mercer, 1982). This formula is as follows:

$$\dot{u} = \omega^2 a \frac{\sinh k(z+h)}{\sinh kd} \sin(\omega t - kx) \tag{13}$$

Where:

$\dot{u}$	:	Wave acceleration
ω	:	$2\pi/T$
k	:	$2\pi/L$
a	:	Wave amplitude
z	:	Vertical co-ordinate, positive upward, origin at still water level

In (Isaacson & Mercer, 1982) a difference is made between head seas (waves hitting the ship from the front) and beam seas (wave hitting the ship sideways) which lead to different RAO values and which should both be analysed. The RAO values in this research are obtained from graphs in (Isaacson & Mercer, 1982). In this research the waterplane length of the ship and the wave height vary. This is done because then different ships on different locations in the inlet are included, both for head seas as for beam seas. In Table 10 and Table 11 the results of the calculations for the different situations are showed.

$\boldsymbol{Head\ seas\ \ h = 2.6m\ \ T = 2.8s\ \ c = 5.05m/s\ \ L = 14.14m}$				
$\boldsymbol{L_w}$	RAO	a_{wave}	a_{ship}	$\dot{u}_{ship}$
6	0.6	0.18	0.11	0.54
8	0.5	0.18	0.09	0.45
10	0.5	0.18	0.09	0.45
12	0.4	0.18	0.07	0.36
15	0.4	0.18	0.07	0.36
20	0.2	0.18	0.04	0.18
6	0.6	0.13	0.08	0.39
8	0.5	0.13	0.06	0.33
10	0.5	0.13	0.06	0.33
12	0.4	0.13	0.05	0.26
15	0.4	0.13	0.05	0.26
20	0.2	0.13	0.03	0.13

Table 10: Ship acceleration due to head seas for different ship sizes and wave heights

$\boldsymbol{Beam\ seas\ \ h = 2.6m\ \ T = 2.8s\ \ c = 5.05m/s\ \ L = 14.14m}$					
$\boldsymbol{L_w}$	RAO	a_{wave}	a_{ship}		$\dot{u}_{ship}$
6	1.0	0.18	0.18		0.91
8	1.1	0.18	0.20		1.00
10	0.9	0.18	0.16		0.82
12	0.8	0.18	0.14		0.73
15	0.6	0.18	0.11		0.54
20	0.4	0.18	0.07		0.36
6	1.0	0.13	0.13		0.65
8	1.1	0.13	0.14		0.72
10	0.9	0.13	0.12		0.59
12	0.8	0.13	0.10		0.52
15	0.6	0.13	0.08		0.39
20	0.4	0.13	0.05		0.26

Table 11: Ship acceleration due to beam seas for different ship sizes and wave heights

It can be seen in Table 10 that for the ships with a waterplane of $10m$ and smaller, the wave agitation is too severe in the area with higher wave height. For the lower wave heights in Table 10 also the smaller ships up until a waterplane length of $6m$ meet the limitation factor from (Rosen & Kit, 1984). In Table 11 it can be seen that for the larger wave height, only the ship with a waterplane length of $20m$ meets the limitation factor. For the smaller wave height, ships with a waterplane length of $15m$ and larger meet the limitation factor. Therefore it can be concluded that due to beam seas only the larger ships can moor in the inlet. From the results obtained in this part, an advise concerning using the inlet as a marina is given in part 5.2.2.

4.1.3 Breakwater

In this section the effect of the breakwater protecting the inlet from incoming waves is analysed. Because the breakwater may not be accounted for when concerning the primary water defence, this section only includes the SLS conditions. This section consists of two parts; first the effect of the breakwater is tested by analysing the results of the simulation without the breakwater. After that the simulation concerning the length of the breakwater is analysed.

No breakwater

For the situation with no breakwater the values from Figure 3.12 (left) and Figure 3.15 (left) are analysed. Since the wave height in the inlet is a lot higher for this case, only larger ships are concerned. For this analysis the same calculation methods as in part 4.1.2 are used. The results can be seen in Table 12 and Table 13

$\boldsymbol{Head\ seas\ \ h = 3.0m\ \ T = 2.8s\ \ \ c = 5.43m/s\ \ L = 15.19m}$				
$\boldsymbol{L_w}$	RAO	a_{wave}	a_{ship}	$\dot{u}_{ship}$
15	0.4	0.41	0.16	0.83
20	0.2	0.41	0.08	0.41
25	0.2	0.41	0.08	0.41
15	0.4	0.30	0.12	0.60
20	0.2	0.30	0.06	0.30
25	0.2	0.30	0.06	0.30

Table 12: Ship acceleration due to head seas for different ship sizes and wave heights in the case of no breakwater

$\boldsymbol{Beam\ seas\ \ h = 3.0m\ \ T = 2.8s\ \ \ c = 5.43m/s\ \ L = 15.19m}$				
$\boldsymbol{L_w}$	RAO	a_{wave}	a_{ship}	$\dot{u}_{ship}$
15	0.6	0.41	0.25	1.24
20	0.4	0.41	0.16	0.83
25	0.3	0.41	0.12	0.62
15	0.6	0.30	0.18	0.91
20	0.4	0.30	0.12	0.60
25	0.3	0.30	0.09	0.45

Table 13: Ship acceleration due to beam seas for different ship sizes and wave heights in the case of no breakwater

From Table 12 it can be concluded that only in the more sheltered areas of the inlet ship with a waterplane length of $20m$ and more meet the limitation factor. When concerning beam seas in Table 13 it can be concluded that even ships of $25m$ in the more sheltered areas do not meet the limitation factor. Therefore a marina without a breakwater would not be possible.

Short breakwater

For the situation with a short breakwater Figure 3.12 (right) and Figure 3.15 (right) are used. For this analysis the same waterplane lengths are used as in the analysis for the longer breakwater. Furthermore the calculations of part 4.1.2 are used. The results can be seen in Table 14 and Table 15.

$\boldsymbol{Head\ seas\ \ h = 2.6m\ \ T = 2.8s\ \ \ c = 5.05m/s\ \ L = 14.14m}$				
$\boldsymbol{L_w}$	RAO	a_{wave}	a_{ship}	$\dot{u}_{ship}$
6	0.6	0.19	0.11	0.56
8	0.5	0.19	0.09	0.47
10	0.5	0.19	0.09	0.47
12	0.4	0.19	0.07	0.37
15	0.4	0.19	0.07	0.37
20	0.2	0.19	0.04	0.19
6	0.6	0.14	0.08	0.41
8	0.5	0.14	0.07	0.34
10	0.5	0.14	0.07	0.34
12	0.4	0.14	0.05	0.27
15	0.4	0.14	0.05	0.27
20	0.2	0.14	0.03	0.14

Table 14: Ship acceleration due to head seas for different ship sizes and wave heights in the case of short breakwater

$\boldsymbol{Beam\ seas\ \ h = 2.6m\ \ T = 2.8s\ \ \ c = 5.05m/s\ \ L = 14.14m}$				
$\boldsymbol{L_w}$	RAO	a_{wave}	a_{ship}	$\dot{u}_{ship}$
6	1.0	0.19	0.19	0.93
8	1.1	0.19	0.20	1.02
10	0.9	0.19	0.17	0.84
12	0.8	0.19	0.15	0.74
15	0.6	0.19	0.11	0.56
20	0.4	0.19	0.07	0.37
6	1.0	0.14	0.14	0.68
8	1.1	0.14	0.15	0.75
10	0.9	0.14	0.12	0.61
12	0.8	0.14	0.11	0.54
15	0.6	0.14	0.08	0.41
20	0.4	0.14	0.05	0.27

Table 15: Ship acceleration due to beam seas for different ship sizes and wave heights in the case of short breakwater

When comparing the results in Table 14 and Table 15 with the results in Table 10 Table 11 it can be concluded that the short breakwater does not lead to much more wave agitation. Also in this situation with beam seas in sheltered area only the large ships meet the limitation factor. These results are more or less in line with (AS 3962, 2001) where is stated that waves in a marina should not exceed $0.3m$ on a yearly basis.

4.2 COMPARISON SWASH AND SWAN

In this research both the model SWASH and SWAN are used for the simulation of the area. For the calculation of the reflection coefficient of the vertical wall with berm, only SWASH is used. In this part the comparison between the two models in this situation is made.

4.2.1 Diffraction

The main reason in the beginning of this research to use both SWASH and SWAN was to check if SWAN is accurate enough concerning diffraction and reflection. From the results of the modelling shown in chapter 3 it can be seen that indeed there is some deviation in results in the area where diffraction influences the results. This is most present when comparing the results from Figure 3.9 and Figure 3.14. Still this difference is not significant enough to conclude anything about diffraction in SWAN, also because these results are probably even more influenced by reflection inside the inlet. Therefore other results should be analysed to formulate a proper comparison.

For this comparison simulation number 7 from Table 5 is considered. This means waves from the north-east during SLS condition which means that the H_s is $0.7m$ and the t_p is $2.8s$. For this situation the set-up is $0.6m$ and with a still water level of $-0.10m\ NAP$ this means that h_b will be $0.2m$. For this case it can be assumed that the wave height in the inlet is reduced that much that the negative amplitude of the incoming wave will be less than h_b. Therefore the reflection coefficient will be small and represented with a porosity of 0.40 (see part 3.1.2). Furthermore the same locations for measuring the H_s will be used as in Figure 3.8. The results are shown in Figure 4.1.

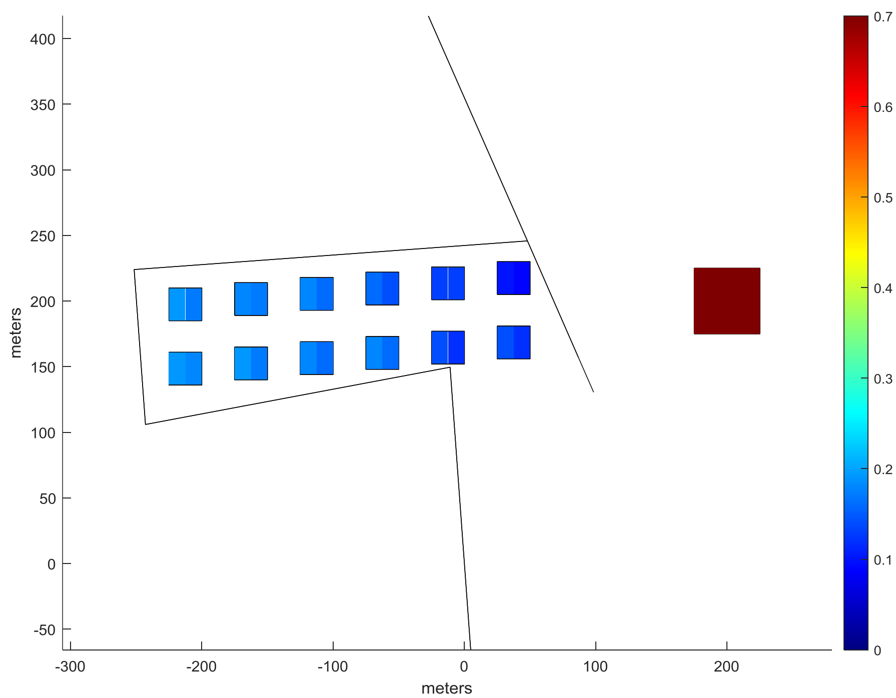

Figure 4.1: Comparison SWASH and SWAN calculation, with on the left of every square the SWASH results and on the right of every square the SWAN results.

4.2.2 Steep waves

The results in SWASH show that the model as it is set up in this research cannot deal with the steep wave imposed at the boundaries. This can be seen from test runs where either the depth is increased (leading to a large wave celerity and thus larger wave length and thus less steep waves) or the peak period is extended (also leading to a large wave length and thus less steep waves) resulting in less decrease of H_s in the domain. Also lowering the H_s, which implies lowering the steepness, leads to a smaller decrease of H_s inside the domain. Therefore it can be concluded that it is the steepness of the waves that is lowering the H_s in the domain. That is strange because when checking the parameters for this situation in (Weggel, 1972) the waves should at least have a H_s of $1.9m$ to start breaking. From (Riedel & Byrne, 1986) it can be concluded that the H_s should be in the most extreme case $1.32m$ to start wave breaking. So no matter what way the breaker height is calculated, the waves implemented in this case should not be breaking.

4.2.3 1D computation in SWASH

An important argument that the use of SWASH is necessary in this situation, is that the 1D computation to calculate the reflection coefficient of the vertical wall with berm in part 15 would not have been possible in SWAN. This 1D computation is crucial for this research because in literature not this exact case has been treated. Only something similar (Heijer, 2005) has been tested, which can be used as a validation for the 1D tests in this research.

5 DISCUSSION, CONCLUSION & RECOMMENDATIONS

In chapter 5 the results and analysis are discussed an conclusions are made. At first some of the aspects of the research where there is room for discussion concerning accuracy and the method used are discussed. This is important to know where certain flaws could have been made. Thereafter the conclusion concerning the dimensions of the wall with berm, the breakwater, the marina and the usage of SWAN and SWASH are made. At last recommendations for further research are made.

5.1 DISCUSSION

In the discussion part some of the assumption and calculations done in the research are being discussed.

5.1.1 Wave input

During this research the wave input at the boundary was either from Hydra-NL or calculated using the method proposed in (Holthuijsen, 2007). For the longer return periods Hydra-NL gives an accurate estimation of the H_S in front of the island. So for these return periods the Hydra-NL results are used. When the H_S of the lower return periods are used, mainly of the 1 year return period, a calculation from (Holthuijsen, 2007) is used. From these calculations it appeared that the wave heights predicted by Hydra-NL were correct but the peak periods were deviating from the peak periods in Hydra-NL. Therefore for the peak periods with lower return periods the calculations from (Holthuijsen, 2007) were used. The question is if these calculations are accurate enough. The accuracy of the wave input influences the accuracy of the entire research. Nevertheless, the calculations from (Holthuijsen, 2007) with the larger return periods are accurate enough, and therefore it is assumed that the calculations for smaller return periods are also accurate enough. If a more accurate estimation of the wave conditions is desired, for instance a wave buoy could be used to predict the wave conditions for the smaller return periods.

5.1.2 Oblique incident waves

For the 1D simulation for calculating the reflection coefficient of the vertical wall with berm in front of it, only incoming waves perpendicular to the wall are used. The reflection coefficient obtained from the 1D simulation is used in the entire inlet, where also oblique waves hit the wall and therefore are reflected with the same reflection coefficient as waves hitting the wall perpendicular. In reality this reflection coefficient is expected to be less. In literature no research is found for the particular situation with oblique waves hitting a vertical wall with berm. In (Das & Bora, 2014) the reflection coefficient of oblique waves hitting a porous structure is researched. There it is concluded that the reflection coefficient decreases with oblique incident waves. There is not much more literature that is about the reflection coefficient of oblique waves let alone of the specific situation needed in this research. Setting up a formula to be able to calculate the reflection coefficient for oblique waves would be very complex and the added value for this research would not be worth it. Therefore the reflection coefficient of perpendicular waves is also used for the reflection coefficient for oblique waves. It is assumed that like in (Das & Bora, 2014) the reflection coefficient will also in this situation decrease. Therefore taking the reflection coefficient of perpendicular waves will lead to an overestimation of the reflected waves and will thus lead to a conservative approach, which is from a safety point of view better than underestimating the reflection coefficient.

5.1.3 Linear multiplications of the results of the nonlinear model SWASH

Due to the fact that the H_s could not kept on an acceptable level in the domain, a multiplication factor is introduces. By using a multiplication factor all results from simulations with smaller incoming waves are than multiplied linearly by a certain factor. By doing this, all the non-linear effects are not taken into account anymore for the increase of the H_s. This mainly implies that in Figure 5.1 the value on the y-axis will decrease while the value on the x-axis will stay the same. When the H_s in the domain is decreased to the steady level, only second order Stokes effects play a role in the non-linearity while if the wave height would stay at the level as imposed on the boundary, also third order Stokes waves are present. Note that this only counts for the ULS conditions. For the SLS conditions only the second order Stokes waves are present and on top of that no multiplication factor is used. The result of denying this third order Stokes wave by using the multiplication factor is that the asymmetry of the wave is not correct anymore and therefore the height of the peak of the wave will be underestimated. Although the height of the peak will be underestimated, also the depth of the through will be overestimated and therefore the underestimation of the H_s will not be that large. Since the dimensioning of the wall height in part 4.1.1 is based on the H_s, the miscalculation due to this multiplication factor will not be that large. Nevertheless it is a factor that should be noted since it alters the outcome.

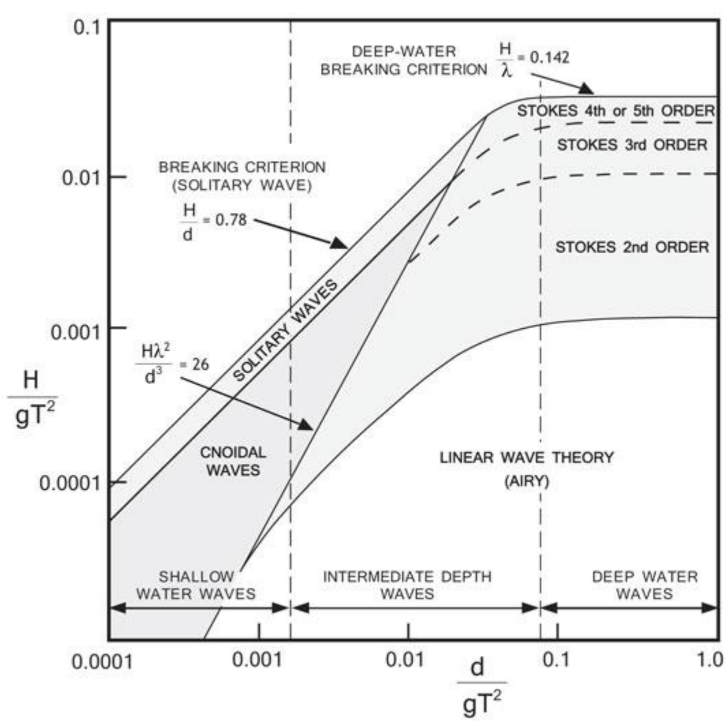

Figure 5.1: the ranges of applicability of the various wave theories

5.1.4 Dimensioning the berm

The dimensioning of the berm will be based on the submergence of the berm due to set-up and the overflowing of the berm due to waves. This all will be discussed with the SLS conditions, so with a return period of 1 year. When considering these conditions and the berm as in the provisional design with a height of $+0.30m\ NAP$, it can be seen that a set-up of $0.5m$ will be reached every year. If this comes in combination with the highest water level possible in the Markermeer, this would lead to a water level of $+0.40m\ NAP$, and therefore the berm will be submerged with $10cm$ of water. If the water level is at the target level, the water level plus set-up will be $+0.30m\ NAP$ leading to a water level just as high as the berm. When below the target level, the water level will be below the berm. It can be concluded that with the provisional design the berm will be submerged due to the set-up, but only about once a year. Concerning the waves overflowing the berm, this cannot be seen apart from the set-up. The highest waves in the inlet occur whit a set-up of $20cm$. The maximum H_s in the inlet for that set-up is $36cm$. If the water level in the Markermeer is at its maximum, the water level plus the set-up will be $+10cm\ NAP$ so the berm will be $20cm$ above the still water level. So the highest waves in the inlet will overtop the berm in this situation, though it will just be a few waves only once a year and only with the highest possible water level. If the set-up is $50cm$ as described in the previous situation, the waves will of course overtop the berm when the highest possible water level occurs at the same time. This also counts for a combination with the target level. When lower water levels occur, there will also be some waves overtopping the berm, though it will be just few little waves because the H_s in this situation will be low, as can be seen in the results of the simulations. It can be said that the berm height as dimensioned in the provisional design is high enough if it is accepted that it can overflow or be submerged about once a year. Increasing the height of the berm by only $10cm$ would lead to a decrease of overflowing and the berm being submerged to the amount of less than once a year. Decreasing the height would increase that number significantly and is therefore not advised. It depends on what is desired, what height should be chosen.

The width of the berm has in this case, when assuming the wall height will be large enough, not any influence. Therefore dimensioning the width can be based on other criteria which are beyond the scope of this research. When concerning the inlet being used as a marina, other influences of the berm width and height can be considered. This will be done in part 5.2.2.

5.2 CONCLUSION

In this part the conclusions that can be drawn from the research are described. These conclusions should answer the questions asked in the objectives in part 1.3.

5.2.1 Final Design

Wall with berm

From the analyses of the results concerning the design of the walls surrounding the inlet it can be concluded that the provisional design as proposed by the Gemeente Amsterdam meets the design criteria. The height of the walls ($+4.50m\ NAP$) of the provisional design is more than enough, since from the analysis of the simulations it came forward that only a height of $+2.00m\ NAP$ would be enough to meet the requirements of a maximum overtopping of $10\ l/s/m$. This wall height would not be realistic when considering the use and the height of the hinterland. Therefore in the remainder of this part the proposed wall height of $+4.50m\ NAP$ is assumed. With this wall height all the combinations of berm height and width are possible. Therefore the dimensioning of the berm does not depend on the demands concerning safety during ULS conditions, so the berm dimensioning can be based on other considerations. This is discussed in 5.1.4.

Breakwater

The analysis of the breakwater contains two parts. First of all the necessity of the breakwater is tested. Thereafter it is tested what the influence of the length of the breakwater is. The analysis of the breakwater only includes the SLS condition. In this research a non-permeable breakwater with no overtopping is used from which only the length is altered. From the analysis it can be concluded that the breakwater is necessary to create a more sheltered inlet. A marina inside the inlet without a breakwater would not be possible due to too extensive wave agitation. Furthermore the berm would overflow more often, or its height should be increased. Decreasing the length of the breakwater is possible without having much effect on the agitation in the inlet. In this research the length is decreased from $130m$ to $100m$. This had just a little effect on the wave agitation in the inlet. This cannot be done unlimited because it is concluded that a breakwater is necessary. An optimal length and optimal dimensioning is something for further research.

Final design

It can be concluded that the walls as proposed by the Gemeente Amsterdam do meet the criteria and can therefore be used in the inlet. Furthermore the breakwater can be shorter than proposed, namely $100m$ instead of $130m$. Keep in mind that the dimensioning of the wall with berm can be altered due to other criteria. Furthermore research is recommended to determine the optimal length of the berm.

5.2.2 Marina

The possibility of the construction of a marina is among others dependent on the wave agitation in the inlet. In part 4.1.2 the wave agitation and its effect on the movement of the ships in the inlet is analysed. From that it can be concluded that the waves hitting the ships sideways are the limiting factor. Furthermore it can be concluded that in the inlet, due to the reflection the waves go in all directions. This can be seen in Figure 5.2. Because of that it will not be possible to align the marina in such a way that in some places the ships will only have head seas. Therefore it is advised that when creating a marina in the inlet it should be a marina just for ships larger than $15m$ in the more sheltered area and for ships larger than $20m$ in the less sheltered areas. So when designing the marina the results from the simulation of the agitation during SLS conditions should be taken into account to decide where to make berthing space for the larger and smaller ships. If a marina which is also accessible for smaller ships is desired, other measurements can be considered to reduce the wave agitation inside the inlet. Giving a good advice concerning these measurements is beyond the scope of this research but suggestions can be found in part 5.3.

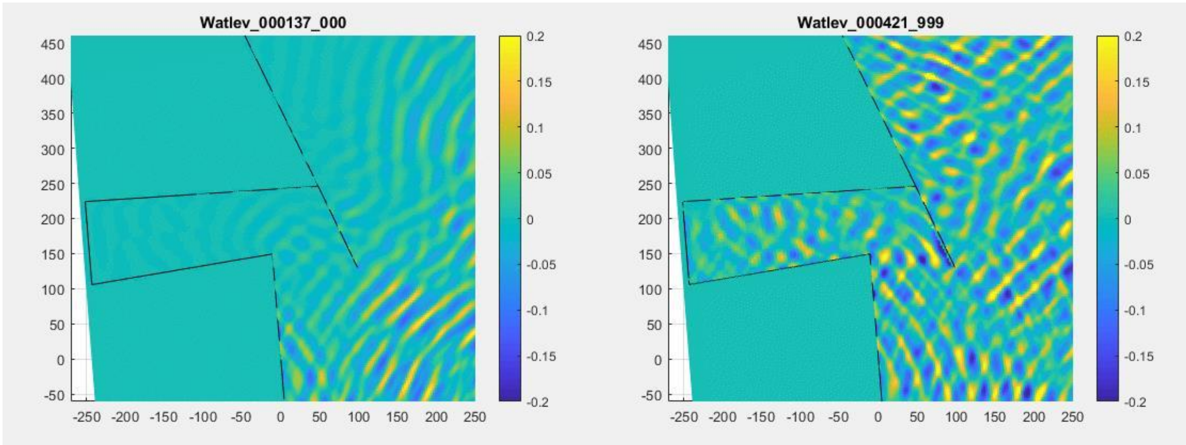

Figure 5.2: Incoming waves mainly from one direction in the beginning of the simulation on the left, wave agitation in the inlet with waves going in all directions on the right.

5.2.3 Usage of SWAN and SWASH

Another important objective of this research is to compare SWAN and SWASH when used for a not too complex situation like the inlet in Strandeiland. First of all it can be concluded that SWASH is not easy to use when a situation with steep waves in shallow water occurs. The wave height inside the domain decreases to a level which SWASH sees as the maximum possible H_s in that area. This maximum possible H_s is far below the realistic maximum H_s for waves in shallow water. Therefore the results from SWASH were multiplied in a linear way which reduced the accuracy. Nevertheless the results were still close to the results obtained in SWAN. In SWAN this problem did not occur. Only a slight decrease in H_s occurred near the boundary but this could easily be undone using some calibration. Furthermore in the inlet SWAN appeared to be accurate enough with a maximum difference of 14%. In absolute sense this is a difference in H_s of just $2cm$ which is acceptable. This in combination with the fact that when using SWAN no multiplication factor was needed leads to the conclusion that in a situation with relative shallow water and steep waves at the boundary with a relative simple lay-out the use of SWAN is preferred. Furthermore it can be concluded that SWAN also accounts accurate enough for diffraction. These conclusions are made worth more by the situation during SLS conditions with waves from the south-east. In this situation no multiplication factor is needed when using SWASH and then also the results from SWAN are very close to the results from SWASH.

In this research SWASH is unmissable when calculating the reflection coefficient of the vertical wall with a berm in front of it. This is not possible in SWASH. For this research the reflection coefficient of these walls is important to determine the wave agitation in the inlet. When checking these results using (Heijer, 2005) it can be concluded that the results are in line with the results of the scale model tests done in that research.

5.3 RECOMMENDATIONS

In this part some recommendations are done for further research. The recommendation will include all the aspects of this research.

5.3.1 Wave input

In this research it is debatable if the wave input at the boundary is accurate enough, especially concerning the SLS conditions. For a final design is it advised to use a wave buoy to gather more data concerning the SLS conditions in order to make a more accurate simulation of the situation.

5.3.2 Wall with berm

For the berm dimensioning other criteria than the safety criteria concerning wave overtopping are dominant, since these safety criteria will be met with the proposed height of the wall. These criteria that will dimension the berm should be mapped. This can be done in a follow-up study where the final design of the area is made.

5.3.3 Breakwater

From the simulations where the length of the breakwater was altered it could be seen that reducing the length of the breakwater from $130m$ to $100m$ did not have much effect on the wave height in the inlet. Therefore it is advised to do a study about the length of the breakwater in which the optimal breakwater length will be calculated. This could reduce the costs of the construction significantly.

5.3.4 Marina

The lay-out as it is proposed right now with the vertical walls with berm and the breakwater in front of the inlet, lead to agitation in which ships with a minimum length of $15m$ can berth. In some places the minimum length will be $20m$. If this is what the marina is intended for, than the agitation due to this lay-out is acceptable. If smaller ships should be able to dock in the marina, measurement to reduce the agitation should be imposed. There are several options that need further research to see its effect. Some suggestion are: wave absorbing vertical walls like for instance the Jarlan type breakwater (Jarlan, 1961) could be situated at tactical locations. Furthermore the back wall could be replaced by a sloping beach, if enough space is available. More options are possible, a study to those would be advised if small ships should berth in the marina.

5.3.5 SWASH and SWAN

The main obstacle during this research was that the wave height modelled by SWASH did reduce significantly inside the domain. Therefore in some situation the results in SWASH reduced in accuracy. It is recommended to do a study to the specific situation in SWASH where the combination of relatively shallow water (about $3m$ in this case) and steep waves are present.

List of figures

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A SWAN AND SWASH INPUT FILES

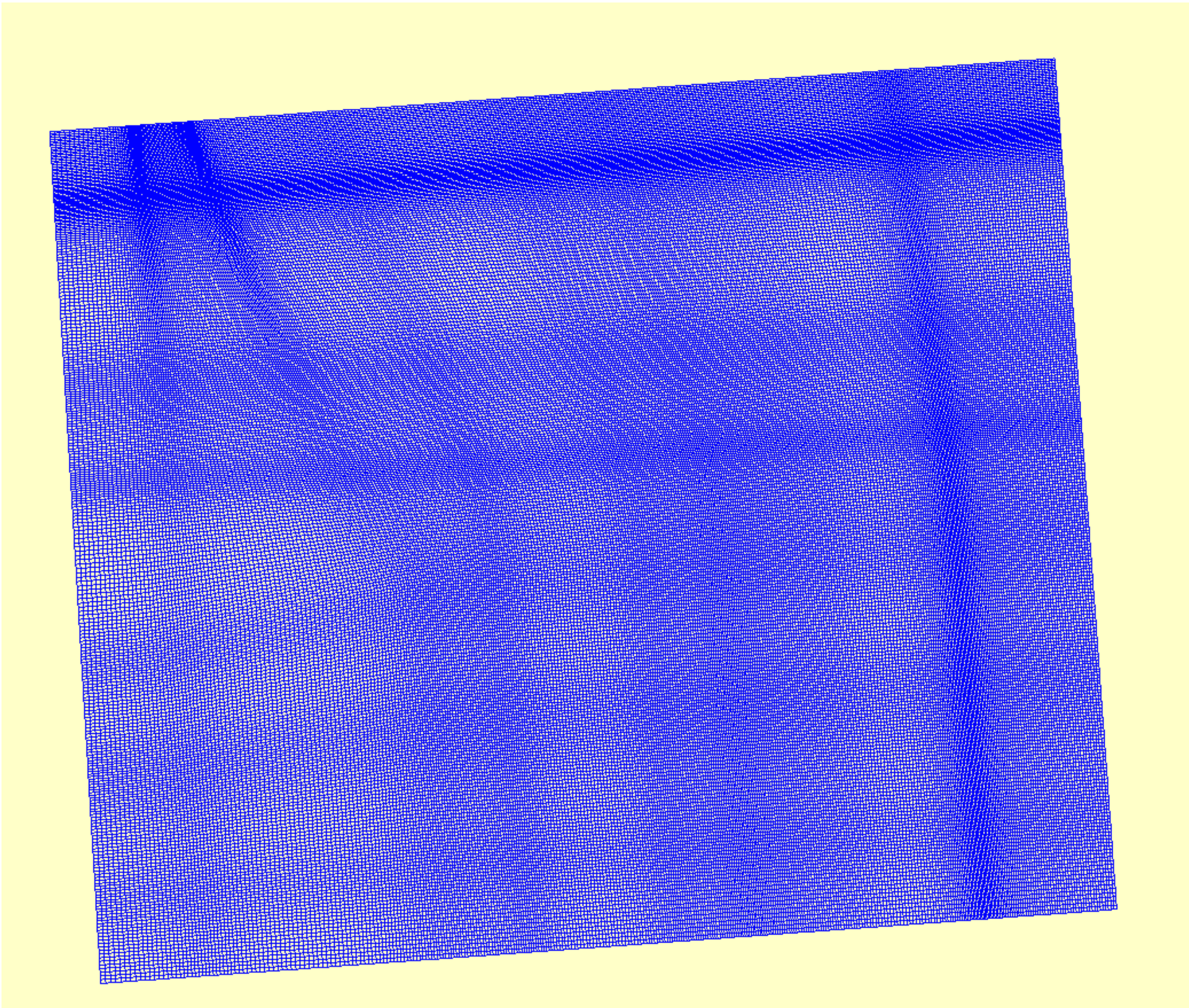
In this appendix some examples of the input files of SWAN and SWASH are shown

SWASH

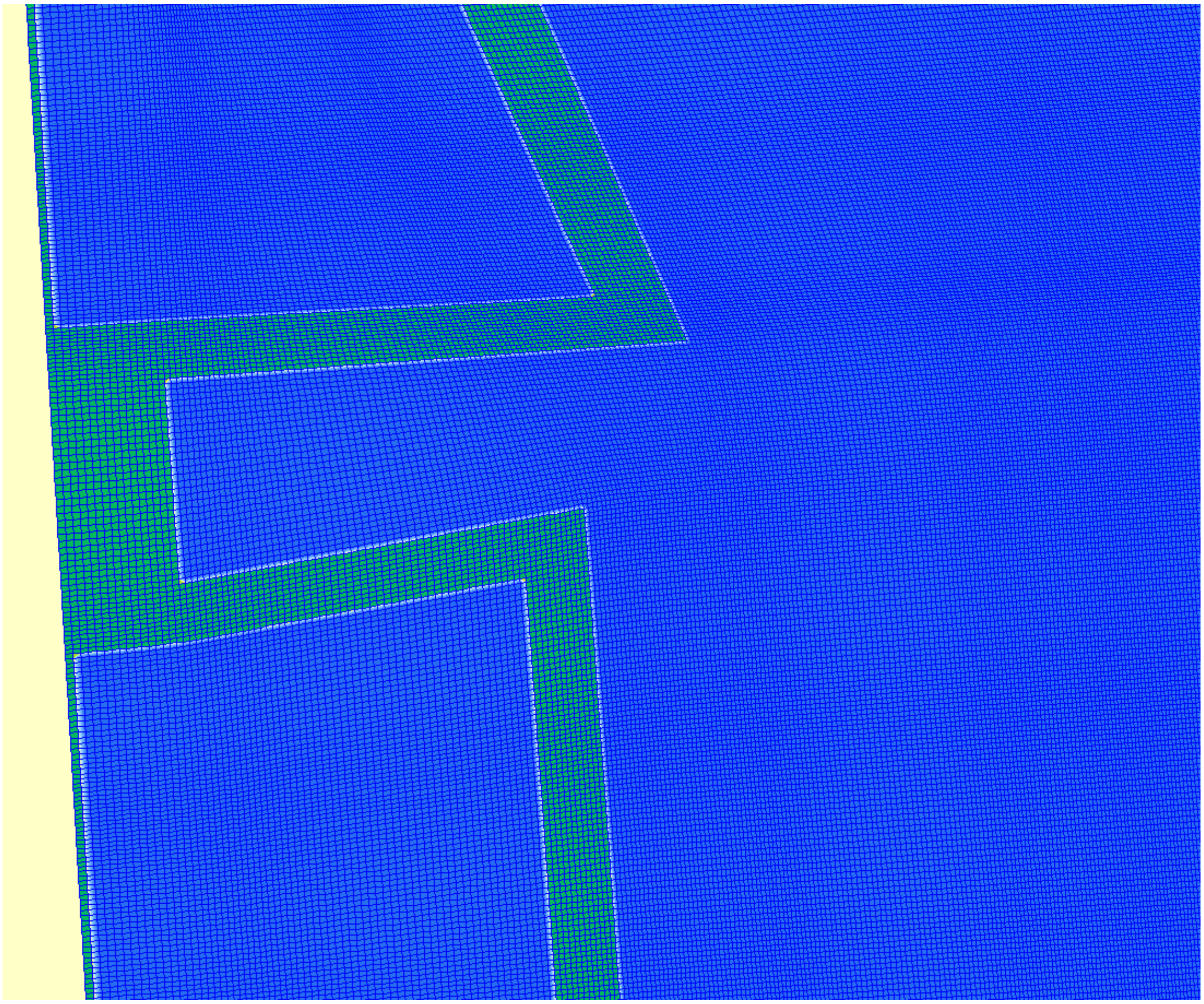
Example .SWS files east SLS conditions.

```
$****************Case Marina Curvegrid*********************
$
PROJ 'MARINALARGE4' 'A7'
$
SET SALINITY=10
$
COORDINATES CART
$
$
CGRID CURVILINEAR 352 304 EXCE 0.0 0.0
READ  COORD 1. 'curvelarge4.grd' IDLA=1 NHEDF=7 FORMAT '(11x,5E26.17)'
$
INPGRID BOTTOM CURV
READ BOT 2. 'botlarge4.bot'  IDLA=1 FRE
$
INPGRID PORO CURV
READ PORO 1. 'porlarge4.por'  IDLA=1 FRE
$
INPGRID PSIZ CURV
READ PSIZ 1. 'grainsize4.psi' IDLA=1 FRE
$
$**********************************************************
$
BOU SHAP TMA
BOU SEGM IJ 349 305 200 305 SMOO 10 SEC CON SPECT 0.7 3.8 225 10 30 MIN
$
BOU SHAP TMA
BOU SEGM IJ 353 302 353 150 BTYPE WEAK SMOO 10 SEC CON SPECT 0.7 3.8 135 10 30 MIN
$
SPON WEST 60
SPON SOUTH 60
$
$
$**********************************************************
$
GROUP 'MC' SUBG 15 340 20 290
$
POINTS 'test' file 'ijburg5.pnt'
$
$
QUANTITY HSIG dur 7 min
TABLE 'test' NOHEAD 'test.mat' hs
$
QUANTITY HSIG dur 7 min
BLO 'MC' NOHEAD 'marinalarge.mat' XP YP HS
$
BLOCK 'MC'   NOHEAD 'Run1.mat' LAY 3 TSEC TIME XP YP &
  BOTL WATL OUTPUT 000000.000 1 SEC
$
COMP 000000.000 0.01 SEC 000500.000
STOP
```

Grid created in RGFGRID



Example of (a part of) a porosity input file made in QUICKIN without breakwater



SWAN

Example .SWN file

```
$****************Marina1*********************
$
PROJ 'MARINA1' 'A1'
$
$
SET RHO 1000
$
CGRID REG 0.0 0.0 0 600 400 480 320 CIR 72 0.1 10
$
INP BOT REG 0.0 0.0 4.17 150 100 4 4
READ BOT 1. 'Depth2_5.bot' 1 0 FRE
$
$***********************************************************
$
BOUN SEGM IJ 63 100 150 100 150 0 CON PAR 0.34 2.8 225.
$
OBST TRANS 0.0 REFL 1. RSPEC LIN 249.9 450 250 249.9
OBST TRANS 0.0 REFL 1. RSPEC LIN 250 249.9 15.1 277
OBST TRANS 0.0 REFL 1. RSPEC LIN 15.1 277 15.1 185
OBST TRANS 0.0 REFL 1. RSPEC LIN 15.1 185 314.9 185
OBST TRANS 0.0 REFL 1. RSPEC LIN 347.1 298 261.9 0
$
$****************Output*************************************
$
FRA 'MARINA11' 0. 0. 0 600 400 150 100
BLO 'MARINA11' NOHEAD '.mat' HS
$
COMP STAT
STOP
```

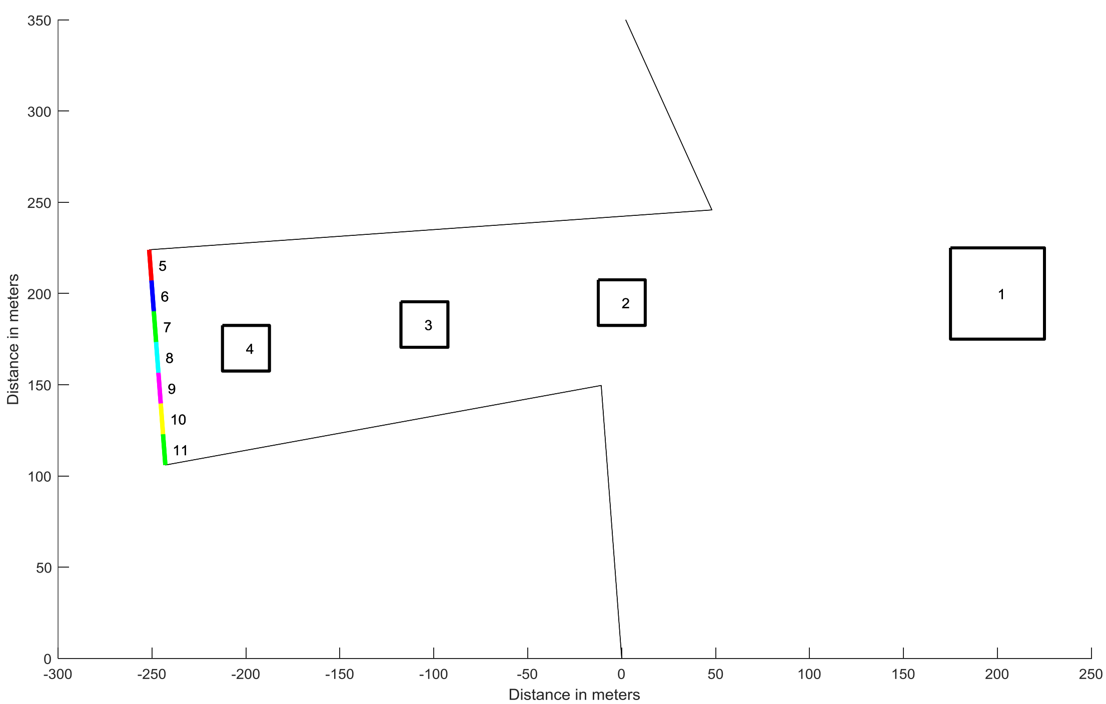
B RESULTS SWAN AND SWASH RUNS

Situations

Return period: Breakwater:	830 year No	1 year No	1 year Yes, $130m$	1 year Yes, $100m$
North-east	1	4	7	10
East	2	5	8	11
South-east	3	6	9	12

ULS conditions SWASH

Overview

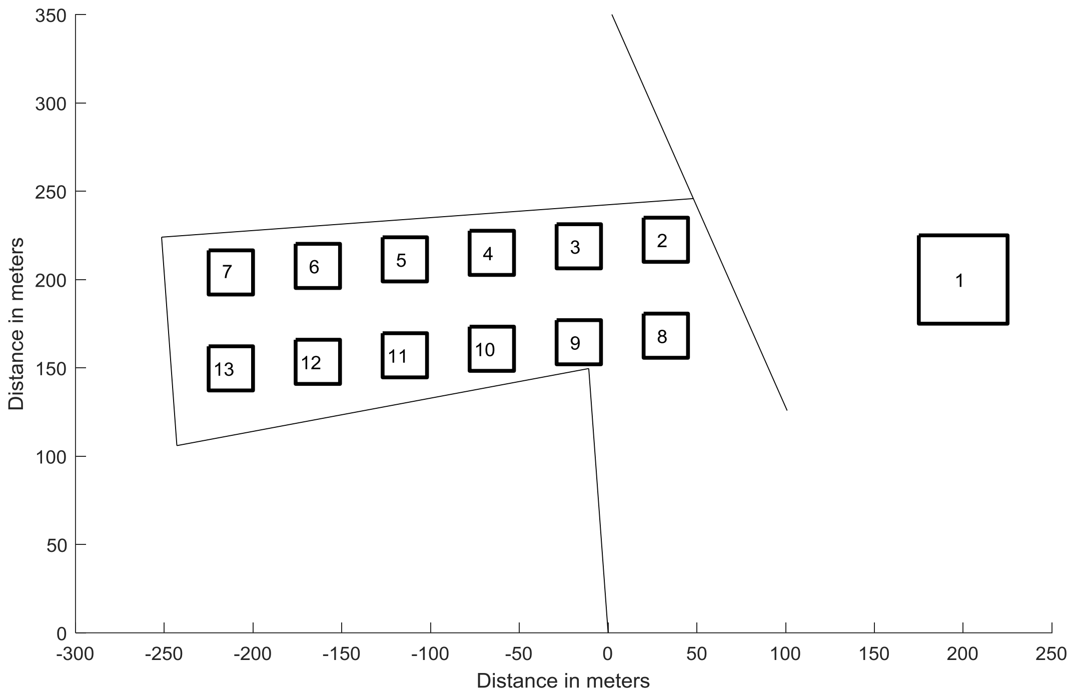


Results relevant situations 1 to 3, H_s in m

	1	2	3	4	5	6	7	8	9	10	11
1	1.2	0.84	0.81	0.66	0.38	0.38	0.41	0.43	0.46	0.49	0.54
2	1.2	1.04	1.09	0.92	0.61	0.54	0.68	0.85	1.03	0.89	0.77
3	0.9	0.88	0.68	0.59	0.49	0.47	0.47	0.45	0.41	0.39	0.36

SLS conditions SWASH

Locations for SLS conditions results



Results relevant situation 4 to 12, H_s in m

	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
4	0.70	0.55	0.48	0.48	0.47	0.49	0.48	0.64	0.60	0.57	0.53	0.51	0.51
5	0.60	0.65	0.70	0.70	0.80	0.73	0.68	0.55	0.64	0.80	0.70	0.64	0.55
6	0.30	0.30	0.29	0.33	0.27	0.25	0.23	0.30	0.27	0.24	0.24	0.25	0.24
7	0.70	0.10	0.13	0.16	0.18	0.18	0.19	0.14	0.15	0.18	0.18	0.19	0.19
8	0.60	0.09	0.10	0.13	0.16	0.16	0.15	0.12	0.14	0.15	0.14	0.15	0.15
9	0.30	0.30	0.28	0.34	0.28	0.27	0.26	0.29	0.27	0.25	0.26	0.26	0.25
10	0.70	0.11	0.14	0.16	0.19	0.19	0.19	0.14	0.16	0.18	0.19	0.19	0.19
11	0.60	0.10	0.10	0.14	0.17	0.16	0.14	0.14	0.15	0.15	0.14	0.16	0.15
12	0.30	0.31	0.30	0.36	0.29	0.27	0.26	0.30	0.29	0.25	0.26	0.27	0.25

ULS conditions SWAN

Results relevant situation 1 to 3

	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	1.2	0.83	0.81	0.65	0.36	0.36	0.39	0.42	0.45	0.47	0.52
2	1.2	1.02	1.10	0.94	0.59	0.56	0.68	0.87	1.01	0.82	0.62
3	0.9	0.86	0.67	0.57	0.48	0.47	0.46	0.45	0.42	0.38	0.36

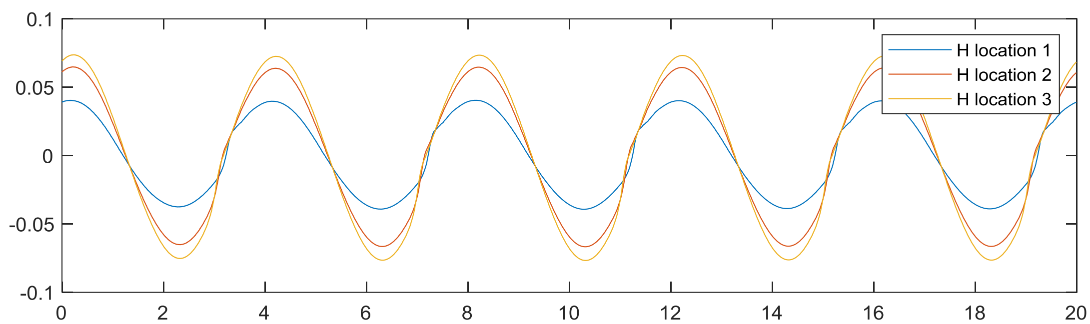
SLS conditions SWAN

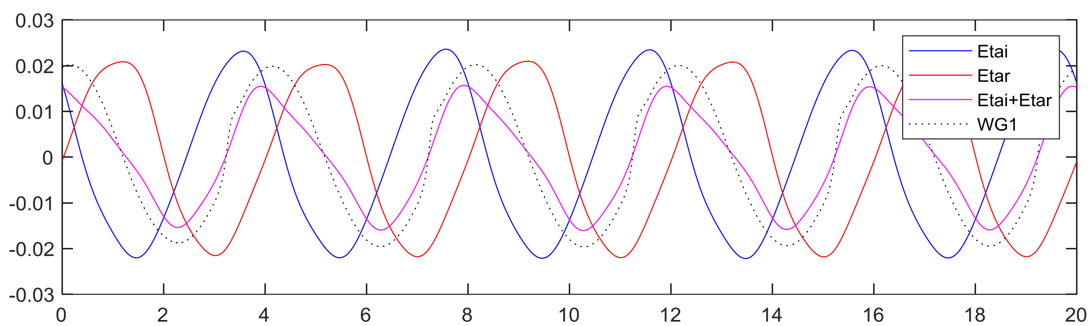
Results relevant situation 4 to 12

	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
4	0.70	0.54	0.46	0.46	0.47	0.48	0.48	0.65	0.61	0.57	0.52	0.51	0.50
5	0.60	0.66	0.71	0.72	0.82	0.72	0.65	0.49	0.60	0.76	0.68	0.58	0.44
6	0.30	0.30	0.29	0.32	0.27	0.26	0.24	0.30	0.26	0.22	0.22	0.23	0.22
7	0.70	0.09	0.13	0.14	0.16	0.17	0.17	0.12	0.13	0.16	0.16	0.17	0.18
8	0.60	0.08	0.11	0.12	0.16	0.15	0.15	0.12	0.13	0.15	0.14	0.14	0.14
9	0.30	0.31	0.32	0.36	0.28	0.25	0.21	0.32	0.25	0.20	0.18	0.18	0.16
10	0.70	0.10	0.13	0.15	0.19	0.18	0.18	0.13	0.14	0.17	0.17	0.18	0.19
11	0.60	0.09	0.10	0.14	0.16	0.15	0.14	0.13	0.15	0.14	0.14	0.15	0.15
12	0.30	0.31	0.33	0.37	0.28	0.27	0.22	0.33	0.23	0.20	0.18	0.19	0.17

C 1D COMPUTATION SWASH

Example Zelt and Skjelbreia





Results 1D tests

$H = 1.2m$ $t = 3.8s$ $Water\ level = -0.10m\ NAP$ $Set\ up = 0.6m$					
	$3.4s$	$3.6s$	$3.8s$	$4.0s$	$4.2s$
$1.3m$	0.84	0.84	0.84	0.85	0.88
$1.2m$	0.83	0.84	0.85	0.86	0.88
$1.1m$	0.81	0.83	0.83	0.84	0.84
$1.0m$	0.81	0.82	0.82	0.83	0.82
$0.9m$	0.79	0.79	0.80	0.81	0.81

$H = 0.3m \quad t = 2.7s \quad Water\ level = -0.10m\ NAP \quad Set\ up = 0.2m$					
	$2.3s$	$2.5s$	$2.7s$	$2.9s$	$3.1s$
$0.4m$	0.97	0.98	0.97	0.97	0.97
$0.35m$	0.98	0.97	0.98	0.99	0.98
$0.3m$	0.97	0.97	0.97	0.98	0.99
$0.25m$	0.98	0.98	0.98	0.99	0.98
$0.2m$	0.98	0.98	0.99	0.98	0.99

$H = 0.6m \quad t = 2.8s \quad Water\ level = -0.10m\ NAP \quad Set\ up = 0.6m$					
	$2.4s$	$2.6s$	$2.8s$	$3.0s$	$3.2s$
$0.8m$	0.71	0.73	0.73	0.75	0.76
$0.7m$	0.70	0.68	0.70	0.71	0.73
$0.6m$	0.66	0.67	0.68	0.71	0.73
$0.5m$	0.64	0.64	0.64	0.69	0.72
$0.4m$	0.60	0.62	0.65	0.69	0.70

Porosity	Reflection coefficient
0.01	0.99
0.10	0.94-0.98
0.15	0.83-0.88
0.20	0.74-0.82
0.25	0.65-0.75
0.30	0.60-0.69
0.35	0.55-0.63
0.40	0.45-0.58
0.45	0.36-0.52
0.50	0.32-0.50