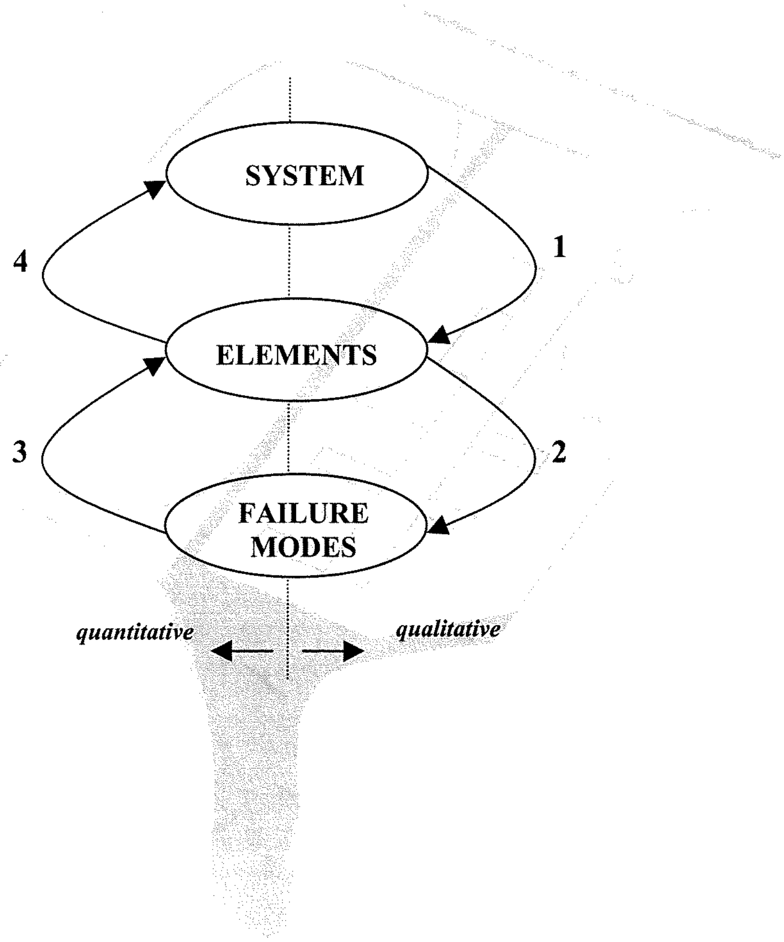
ECONOMIC OPTIMAL DESIGN OF THE

MAASVLAKTE 2

September 2001

J.P.T. Segers



TU Delft
Technische Universiteit Delft

Faculty of Civil Engineering and Geosciences
Subfaculty of Civil Engineering
Hydraulic Engineering Division

Preface

This document comprises my final thesis at the University of Technology in Delft, faculty of Civil Engineering and Geosciences, department of Hydraulic Engineering.

A theoretical model to determine an economic optimal design of the Maasvlakte 2, a proposed land reclamation in the North Sea, by cost-benefit analysis is described. In the analysis, the Maasvlakte 2 is considered as a system which consists of three elements, a breakwater, a sea defence and a terrain area.

In the theoretical model, failure modes and decision variables are selected for each element. For many combinations of values of the selected decision variables, probabilities of failure are calculated and used to determine the minimal investment costs as a function of the level of safety for each element. These results are used to determine the minimal investment costs, expected damage costs and expected benefits as a function of the level of safety of the total Maasvlakte 2. The optimal values of the net present value (NPV) of the total costs and the optimal level of safety of the Maasvlakte 2 are determined. With these values, the accompanying economic optimal design can be found.

I would like to thank ir. H.G. Voortman in particular, for his supervision during the period I worked on this thesis. I would also like to thank prof. drs. ir. J.K. Vrijling, prof. ir. A.C.W.M. Vrouwenvelder and ir. F.M. Stroeve for their contributions.

Julien Segers
Delft, September 2001

Summary

Advanced plans were made by the Dutch government to extend the port of Rotterdam by means of the construction of a land reclamation in the North Sea, the Maasvlakte 2. Because no safety standards exist for the Maasvlakte 2, it is useful to determine an optimal design from an economic point of view by cost-benefit analysis. The economic optimal design of the Maasvlakte 2 is assumed to be the design for which the net present value (NPV) of the total costs (including loss of benefits) is minimal. An accompanying optimal level of safety for the Maasvlakte 2 is also found.

In this analysis, the Maasvlakte 2 is assumed to be a system composed of three elements, a breakwater, a sea defence and a terrain area.
For each element, decision variables are selected which have influence on the resistance (strength) and the costs and benefits of the Maasvlakte 2, and also represent relations between elements. For the breakwater, the crest height and the diameter of the concrete blocks in the armour layer are considered as decision variables. For the terrain area, the height of the terrain area is the only decision variable considered. For the sea defence, the crest height, the diameter of the quarry stones in the protection layer of the outer slope and the angle of the outer slope are considered as decision variables.

A selection of failure modes is also made for each element. A distinction is made between ULS failure modes which are considered under extreme conditions and SLS failure modes which are considered under normal conditions. In case of failure by a failure mode, a (monetary) consequence is the result. The selected failure modes are:

Element	Failure mode	SLS/ULS	Consequence
Breakwater	Erosion of the armour layer	ULS	Damage by total inundation of the terrain area
	Transmission	SLS	Damage to shipping
Sea defence	Overtopping	ULS	Damage by total inundation of the terrain area
	Erosion of the outer slope	ULS	Damage by total inundation of the terrain area
Terrain area	Extremely high water level	ULS	Damage by total inundation of the terrain area

Each failure mode is written in the form of a reliability function $Z = R\text{-}S$ in which R is the resistance and S is the sollicitation. For the failure modes, hydraulic conditions (waves, water levels) represent the sollicitation. The geometry and strength of the elements, also determined by the decision variables, represent the resistance.

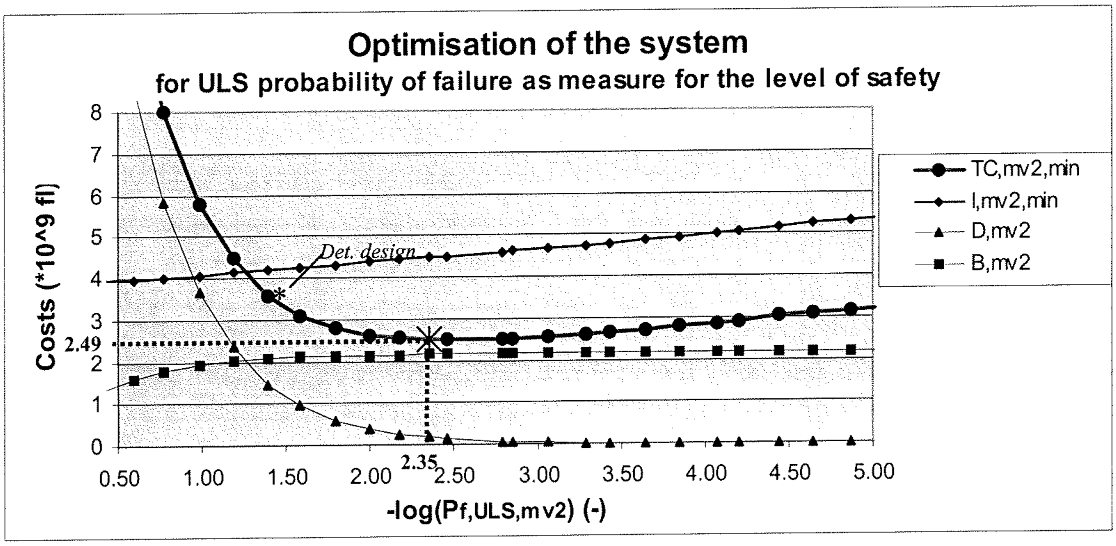
In the calculation of the economic optimal design of the Maasvlakte 2, a bottom-up approach is used:

1. at first, calculations are executed for each failure mode
2. then, these results are used in the optimisation per element
3. finally, optimal element results are used in the optimisation of the system

This means that at first, probabilities of failure are calculated for each failure mode by a probabilistic calculation method for many combinations of values of the decision variables it contains. Then for each element, the failure modes are combined and the minimal investment costs of the element are determined as a function of the level of safety of the element.

In the optimisation of the system (Maasvlakte 2), the results of the elements are combined and the levels of safety of the elements become decision variables, which leads to a reduction in the amount of decision variables.
The minimal investment costs of the Maasvlakte 2 as a function of the level of safety of the Maasvlakte 2 are determined by combination of the three minimal investment costs functions of the elements. Also the expected damage costs and benefits of the Maasvlakte 2 are calculated for many combinations of levels of safety of the elements. From these values, the minimal NPV of the total costs of the Maasvlakte 2 as a function of the level of safety of the Maasvlakte 2 is determined. In this function, the minimal value represents the optimum for the Maasvlakte 2 with optimal level of safety for the Maasvlakte 2.
From this optimal value of the level of safety of the Maasvlakte 2, the accompanying optimal values of the levels of safety of the elements are known. From these values, the optimal values of the decision variables are also known for each element. The optimal values of the decision variables of the elements represent the economic optimal design of the Maasvlakte 2.

With this optimisation method, a probabilistic design is determined and compared with a deterministic design, with chosen values for the hydraulic conditions. The optimisation is executed in two ways: for 'ULS probability of failure' as a measure for the level of safety and for 'risk' as measure for the level of safety. For risk as a measure for the level of safety, the damage costs by SLS failure modes are included in the risk. For ULS probability of failure as a measure for the level of safety, the damage costs by SLS failure modes are added to the investment costs of the element to which they belong.
Compared with the results of the deterministic design, both probabilistic designs show much better results with regard to the total costs in combination with the level of safety of the Maasvlakte 2. An increase in the level of safety without higher investment costs (!) and a strong decrease in the total costs of the Maasvlakte 2, is the result of both probabilistic designs. This means that the cost advantage is the result of 'better investments'.
The figure shows the results of the optimisation with ULS probability of failure as a measure for the level of safety. The optimal value for the ULS probability of failure is $4.43*10^{-3}$ per year.



For the optimal values of the minimal NPV of the total costs and the ULS probability of failure, a sensitivity analysis is executed. This is done by calculation of values for the absolute elasticity for many variations of variables. It follows that the three variables with the highest values for the absolute elasticity with regard to the optimal value of the minimal NPV of the total costs of the Maasvlakte 2 are:

1. the distribution of the yearly maximum water level
2. the interest in one year
3. the relative density of concrete

The three variables with the highest values for the absolute elasticity with regard to the optimal value of the ULS probability of failure for the Maasvlakte 2 are:

1. the distribution of the yearly maximum water level
2. the relative density of concrete
3. the relative density of quarry stone

Because only five failure modes are taken into account and many assumptions are made, the results of the optimisation have to be interpreted with caution. Many improvements are possible like:

- An optimisation with failure modes like 'flooding of the terrain area by waves from the basin' and 'the impossibility of loading/unloading activities at the quays', which leads to more accuracy with regard to the failure modes used in the optimisation.
- An optimisation with different failure modes like 'subsoil failure' or 'failure of the toe construction of the breakwater'.
- Extra levels of damage to create a more gradual transition between consequences.
- The influence of refraction, diffraction, reflection and local wind can be taken into account.
- The influence of dependencies between failure modes can be taken into account.
- More accurate descriptions for the amounts of monetary damage.
- More variables with a (realistic) probability distribution in the optimisation.
- Extra research on variables with high values for the absolute elasticity.
- Other design alternatives like a dune instead of a sea defence or a pitched block revetment instead of quarry stones to protect the sea defence.

ECONOMIC OPTIMAL DESIGN OF THE
MAASVLAKTE 2

TABLE OF CONTENTS:

1. Introduction

1.1 Motives for land reclamation

To maintain a mondial position of competition and a national mainport function, the Port of Rotterdam pursues continuous expansion and improvement of the quality of the environment in and around the port area. By the 'Project Mainportontwikkeling Rotterdam' (PMR) research is done on the increasing need for space in the port area of Rotterdam. This project pursues to find an integral solution, of which the land reclamation Maasvlakte 2 will be a part. By the 'Samenwerkingsverband Maasvlakte 2 Varianten' (SM2V) several alternatives have already been developed. These alternatives have been compared on different aspects as environment, logistics, recreation value and integral safety. The purpose is to achieve a reclamation with a net terrain surface of 1000 hectare. This area will be used by different industrial sectors like chemics and transport.
To improve the process of decision-making, the ministry of Transport and Public Works directed several institutions to investigate an optimal design of the Maasvlakte 2 in terms of costs and safety. This thesis contains an analysis with regard to an optimal design of the Maasvlakte 2 in terms of costs and safety.

1.2 Problem approach

The port of Rotterdam and Schiphol airport in Amsterdam are considered to be the two 'mainports' in the Netherlands. With the term 'mainports' is meant that they are very important for the development of the Dutch economy because they generate an important part of the Dutch national product and they both are large employers in the Dutch labour-market. A new land reclamation would therefore create more employment and stimulate the Dutch economy.
From an economic point of view, the development of a land reclamation would be profitable for the Dutch economy and for the port of Rotterdam specifically, if the overall benefits in its lifetime would be higher than the total costs in this lifetime. It is a goal of this study to find an optimal design in this *cost-benefit analysis*. For simplicity, benefits are considered as *negative costs*. The optimal design is then the design for which the net present value (NPV) of the total costs (including benefits) are minimal.
The term 'net present value' refers to costs and benefits realised in the future which are *discounted* in time. Investment costs are realised in the beginning and therefore 'discounted' for t=0. The total costs of the Maasvlakte 2 are composed of different cost types. Distinguished are:

- Fixed investment costs, independent on all variables
- Variable investment costs, dependent on one or more decision variables
- Damage costs, caused by an undesirable event like flooding of the terrain area or high waves in the port.
- Benefits, as a result of the production in the Maasvlakte 2 area

Each cost type is a function of the level of safety of the Maasvlakte 2.
In short: a safer construction means a lower probability of undesirable events (= lower probability of failure), so the lower the damage costs and the higher the benefits are. On the other hand holds that for a safer construction, investment costs are higher. Therefore, to find an optimal design of the Maasvlakte 2 by cost-benefit analysis, safety is an important aspect and can be considered as an instrument to perform the analysis. In this thesis, the level of

safety will be considered in two ways: as 'probability of failure (P_f)' and as 'risk'. The results of this analysis are assumed to be the main incentive to decide whether or not to construct the Maasvlakte 2.

The Maasvlakte 2 is built in the North Sea, which means that it has to resist hydraulic conditions from the sea to prevent undesirable events to happen. These conditions are combinations of incoming waves and water levels. When an undesirable event takes place, this is called 'failure'. Failure is a state that is the result of one or more *failure modes (*or: *failure mechanisms)* for which load (=sollicitation (S)) exceeds resistance (R), such that $Z = R-S < 0$. Z represents the *reliability function.* Sollicitation and resistance can be defined in several ways.
Two situations are possible with regard to failure modes: 'failure' and 'no failure'. The *probability of failure (P_f)* measures how likely an undesirable event ('failure') is to happen.

$$P_f = P(Z < 0) = P(R > S) \qquad (1.1)$$

When failure takes place, a consequence is the result. From probability of failure and consequence, risk can be determined in many ways. In this thesis, risk will be considered as:

$$Risk = P_f * consequence \qquad (1.2)$$

The consequence is mostly expressed in 'money' or 'casualties'.
This depends on the point of view with regard to risk: individual, societal or economic. The consequence of individual and societal risk is the amount of casualties. For economic risk the consequence is a monetary damage (D). The separation between individual, societal and economic risk is fictitious. There is always some risk from any of these kinds. Because of the importance of cost-benefit analysis in this thesis, economic risk ($Risk = P_f*D$) will be considered.

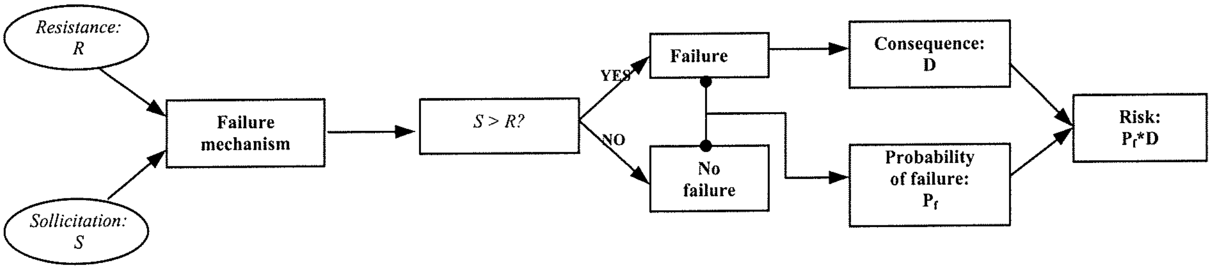


Figure 1.1: Determination of probability of failure and risk

In case of the Maasvlakte 2, hydraulic conditions represent the sollicitation and failure is the state in which the port cannot function the way it normally does. To 'resist' hydraulic sollicitations, hydraulic structures are used instead of the construction of a very high terrain area. The hydraulic structures and the terrain area represent the resistance. In this thesis, a breakwater and a sea defence are used as hydraulic structures.
Including these hydraulic structures, the Maasvlakte 2 consists of three elements: a terrain area, a breakwater and a sea defence. The three elements together are assumed to be a system, dependent on the elements and their relations. This is called a *system-element approach.* To find an optimal design of the system 'Maasvlakte 2', an analysis is performed on three levels: 'system level', 'element level' and the 'failure mode level', figure 1.2.

Four (chronological) steps are taken between the levels, from which 1 and 2 are qualitative (modelling) steps and 3 and 4 are quantitative (calculation) steps:

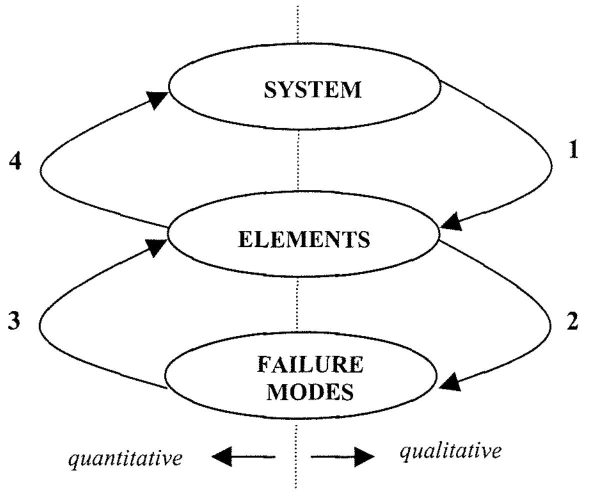


Figure 1.2: Four steps of analysis between three levels

<u>*Step 1:*</u>
The system 'Maasvlakte 2' is schematized as consisting of three elements: a breakwater, a sea defence and a terrain area. Relations between elements are determined and assumptions have to be made on boundary conditions. The decision variables of the elements are also chosen. This is done in chapter 2.

<u>*Step 2:*</u>
For each element, the failure modes which are assumed to be relevant with regard to the probability of failure of the Maasvlakte 2 are taken into account. Each failure mode is described by a reliability function Z containing decision and non-decision variables. This is done in paragraph 3.2.

<u>*Step 3:*</u>
For each failure mode, probabilities of failure are calculated by a probabilistic calculation method for different values of decision variables. Values of non-decision variables have to be assumed. Then for each element, its failure modes are combined, which leads to many combinations of values of decision variables with accompanying level of safety of the element. With these combinations, the minimal investment costs can be determined as a function of the level of safety for each element, the final result of this step. The level of safety is considered as 'probability of failure (P_f)' or as 'risk ($R=P_f*D$)'.
In this step, the amount of decision variables is reduced because the decision variables of each element are replaced by one variable, 'probability of failure' or 'risk' of the element. A more detailed theoretical description is given in paragraph 3.3. Calculations of minimal investment costs as a function of the probability of failure are executed in paragraph 5.3. Calculations of minimal investment costs as a function of the risk are executed in paragraph 5.4.

<u>*Step 4:*</u>
On system level, each element is represented by one variable, its level of safety. Summing of the minimal investment costs of the elements leads to minimal investment costs of the system as a function of the *level of safety of the system*. Damage costs and benefits are

also calculated as a function of the level of safety of the system. From these values, the *minimal NPV of the total costs* as a function of the level of safety of the system is determined. From this function, the optimal value for the *minimal NPV of the total costs* and the *optimal level of safety of the system* can be determined. Figure 1.3 shows this for P_f as a measure for the level of safety. A more detailed theoretical description is given in paragraph 3.4. The optimisation of the system with probability of failure as a measure for the level of safety is executed in paragraph 5.3. The system optimisation with risk as a measure for the level of safety is executed in paragraph 5.4.

Note:

Because no safety standards exist for the Maasvlakte 2 (yet), it is possible to determine an optimal design for various levels of safety without any constraints with regard to the probability of failure. The existing standards all concern areas which protect a lot of inhabitants (social values) apart from economic values, but this is not the case for the Maasvlakte 2.

In an earlier optimisation of the Maasvlakte 2 by Stroeve and Sies (1999), the system was at once optimised for one decision variable per element. This method is called a *top-down approach*. Disadvantage is that calculation times increase strongly when more decision variables are taken into account.
In this report, a *bottom-up approach* is used:

1. at first, calculations are executed for each failure mode
2. then, these results are used in the optimisation per element
3. finally, optimal element results are used in the optimisation of the system

Because of the reduced amount of decision variables on system level, calculation times do not increase strongly when more decision variables per element are taken into account. For each element, more than one decision variable will be used.

Figure 1.3 shows the cost types as a function of the (increasing) level of safety for the (system) Maasvlakte 2, $-log(P_{f,mv2})$.

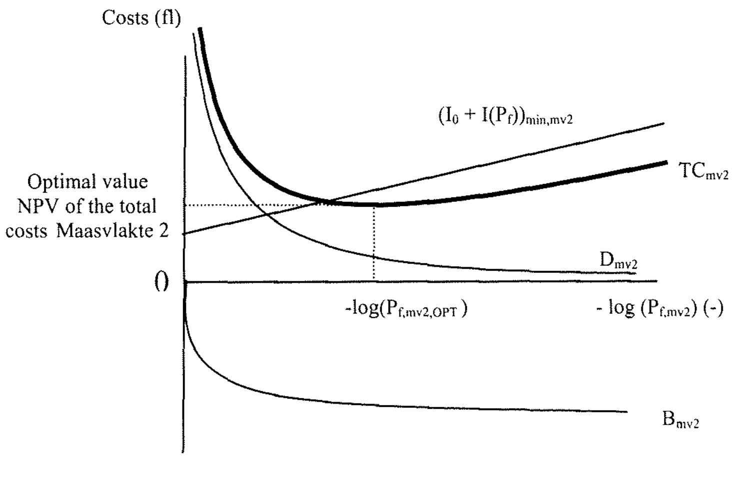


Figure 1.3: Cost types as a function of the probability of failure for the Maasvlakte 2

On system level, the NPV of the total costs is determined by:

$$TC_{mv2}\left(P_{f,mv2}\right)=\left(I_0+I_1\left(P_{f,mv2}\right)\right)_{\min}+\sum_{n=1}^{N}\frac{D_{f,mv2}\cdot P_{f,mv2}}{\left(1+r-i-g\right)^n}-\sum_{n=1}^{N}\frac{B_{nf,mv2}\cdot\left(1-P_{f,mv2}\right)}{\left(1+r-i-g\right)^n} \qquad (1.3)$$

with:

$P_{f,mv2}$ = probability of failure of the Maasvlakte 2 in one year (-)
$TC_{mv2}(\bullet)$ = NPV of the total costs of the Maasvlakte 2 (fl)
I_0 = fixed investment costs (fl)
$I_1(\bullet)$ = variable investment costs (fl)
$D_{f,mv2}$ = expected damage costs of the Maasvlakte 2 in case of failure (fl)
$B_{nf,mv2}$ = expected benefits of the Maasvlakte 2 in one year in case of 'no failure' (fl)
r = interest in one year (-)
i = inflation in one year (-)
g = economic growth in one year (-)
N = number of years (-)

1.3 Problem description

An optimal design of the Maasvlakte 2 from a cost-benefit point of view is lacking

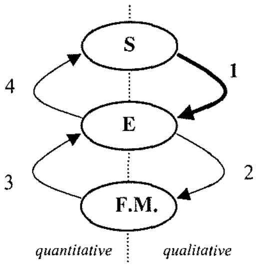
1.4 Goal description

To determine an economic optimal design of the Maasvlakte 2 expressed in terms of safety by using a bottom-up approach to perform a cost-benefit analysis.

1.5 Structure of the report

The following chapters are distinguished:
In chapter 2, the present situation, chosen lay-out and relations between the elements are described (step 1). Decision variables are also selected. In chapter 3, failure modes are described (step 2) and the optimisation on element level (step 3) and system level (step 4)are theoretically explained in more detail. Chapter 4 contains the calculation of a deterministic design and in chapter 5, the results of a basic optimisation on element level (step 3) and system level (step 4) are described. In chapter 6, a sensitivity analysis is performed with regard to the optimal design. Chapter 7 contains an evaluation of improvements of the basic optimisation. Conclusions and recommendations follow in chapter 8. The appendices contain a hydraulic model, formulae of failure modes, descriptions of investment costs functions of the elements, probabilistic calculation principles and graphic results.

2. Lay-out



2.1 Present situation

The present situation in the Port of Rotterdam is given by figure 2.1. The New Waterway is the main entrance channel of the port. The land reclamation 'Maasvlakte 1' has been constructed earlier at the west side of the port. The Maasvlakte 2 will be situated at the west side of the Maasvlakte 1.

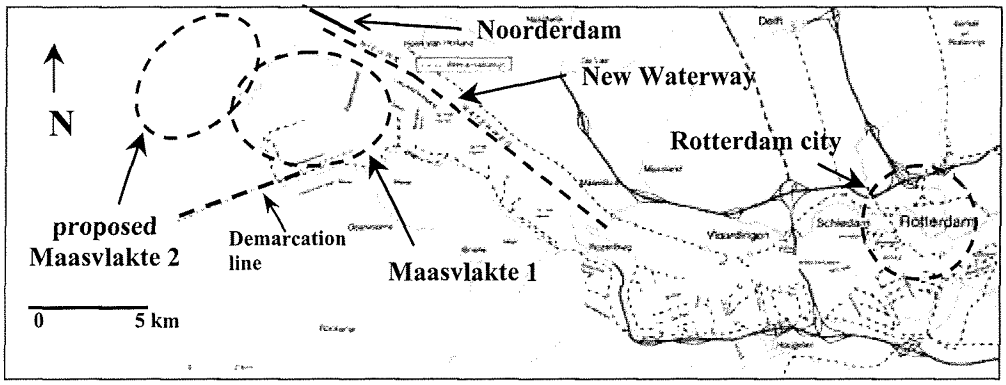


Figure 2.1: Present situation in the Port of Rotterdam

Because of agreements made by the state, the province and the involved municipalities, land reclamation will only be possible on the south side of the New Waterway. There is also a restriction on reclamation southwards because of nature conservation areas present there. The dotted line (demarcation line) in figure 2.1 represents this limitation.

The considered schematization of the Maasvlakte 2 lay-out is the same as used by Stroeve and Sies (1999). It contains sea defences on the west and the south side, the '*Westerdike*' and the '*Zuiderdike*' respectively. To form the protection on the north side of the terrain area of the Maasvlakte 2, the existing breakwater ('*Noorderdam*') will be lengthened. In figure 2.2 and 2.3, the chosen schematization of the lay-out and the cross-section of the Maasvlakte 2 is given.

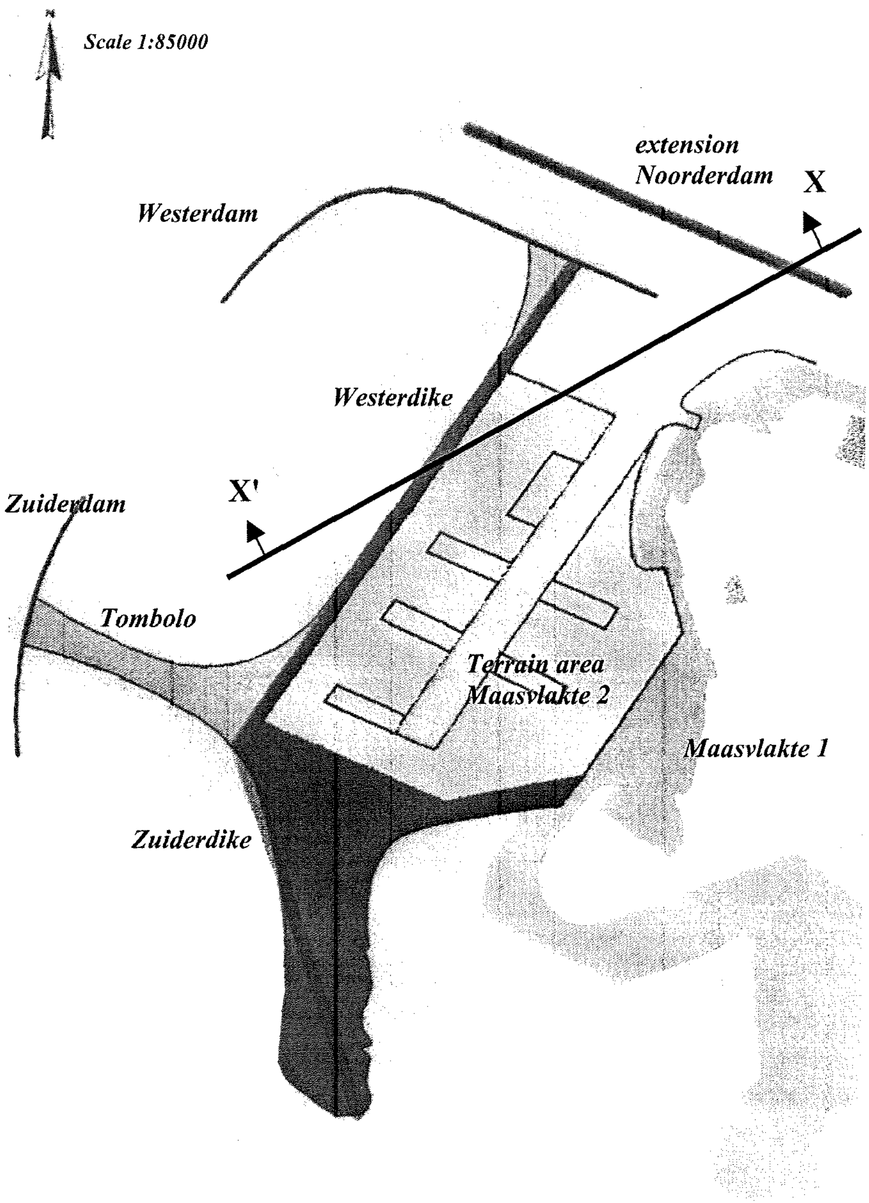


Figure 2.2: Chosen lay-out Maasvlakte 2

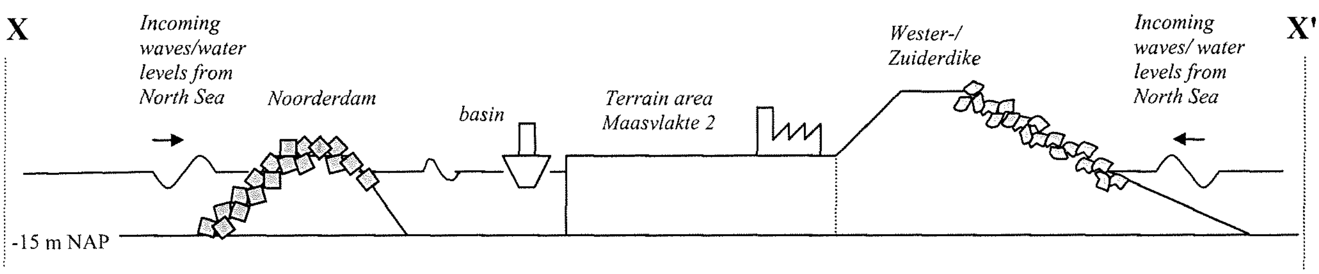


Figure 2.3: Cross-section Maasvlakte 2

2.2 Boundary conditions and assumptions

2.2.1 Hydraulic conditions

The hydraulic conditions are based on a study on caisson breakwaters for the Maasvlakte 2, (Voortman / Vrijling 1999). Because of dependencies between hydraulic parameters, joint probability distributions are needed. This holds for:

- water levels
- significant wave heights
- wave periods

With regard to the hydraulic parameters, two conditions are discerned:

- yearly (extreme) conditions
- daily (normal) conditions

2.2.1.1 Yearly conditions

Water level and significant wave height

In Appendix A.1, the hydraulic model which determines the yearly maximum water level, significant wave height and peak period at the Maasvlakte 2 is shown. The *adapted Bruinsma-function* which describes the average significant wave height at the Euro-0 platform in the North Sea for given water levels at Hook of Holland (close to the Maasvlakte 2), plays an important role in this.

For the distribution of the yearly maximum sea water level ($h_{sea,yr}$) at Hook of Holland, a Weibull distribution (equation 2.1) is chosen, (Voortman/Vrijling 1999). This distribution is supported by data of extreme water levels at Hook of Holland and is also used for the Maasvlakte 2 (figure 2.4). The report 'Basispeilen' is an earlier study on hydraulic conditions along the Dutch coast.

$$P\left(h_{sea,yr} > \eta\right) = e^{-\left(\frac{\eta-\tau}{w-\tau}\right)^k} \tag{2.1}$$

with: $w =$ 2.59 m +NAP
$\tau =$ 2.40 m +NAP
$k =$ 0.85

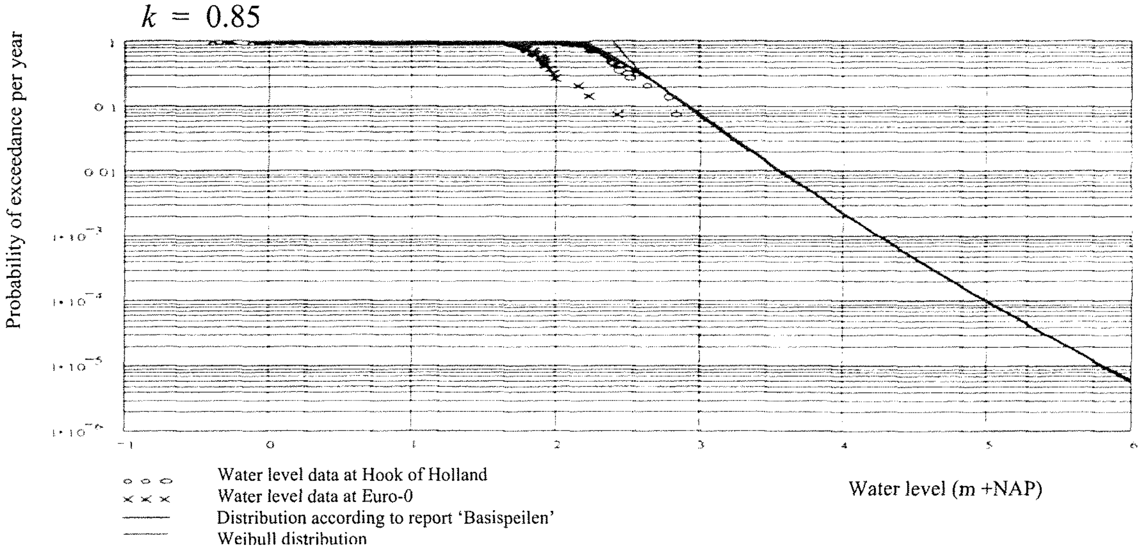


Figure 2.4: Data and distributions of water levels in extreme conditions

From the distribution of equation 2.1, values for the maximum significant wave height per year are calculated with the hydraulic model in Appendix A.1.

However, the maximum water level in one year (or one day) has no physical basis, but the *maximum water level per tidal wave* ($h_{sea,tide}$) has. In fact, all generated data are maximum values of the water level per tidal wave and when the maximum significant wave height per year ($H_{s,yr}$) is calculated, this has to be based on the probability distribution of $h_{sea,tide}$.
In Appendix A.2, an analytical description of the difference is given.

For a detailed description of the difference, more research is needed. However, large differences are not expected, so the calculation of $H_{s,yr}$ based on equation 2.1 is a good approximation which is used in this thesis.

Wave steepness and peak period
Especially in extreme conditions, the peak period (T_p) depends on the significant wave height. Normally, the wave steepness based on the peak period (s_{0p}) does not depend on the significant wave height and is therefore preferred to be used in probabilistic calculation methods. The wave steepness is given by:

$$s_{0p} = \frac{H_s}{\frac{g}{2\pi} \cdot T_p^2} \qquad (2.2)$$

with:

s_{0p} = wave steepness based on peak period (-)
H_s = significant wave height (m)
T_p = peak period (s)

Data from Euro-0 showed that the wave steepness s_{0p} can well be described by a normal distribution with a mean value of 3,8% and a standard deviation of 0,59%, (Voortman/Vrijling, 1999). This distribution is also used for the Maasvlakte 2.

2.2.1.2 Daily conditions
According to the data in figure 2.5, the significant wave height increases for higher water levels at Euro-0. Part of the variation in figure 2.5 can be explained by different wind directions on which the generated data are based. As a result of that, different values for the water level are found for the same value of the significant wave height.
To find a more accurate relation between water levels and significant wave heights, more research is needed. This will not be done in this thesis. For simplicity, significant wave height and water level are considered to be independent in case of daily conditions for both Euro-0 and the Maasvlakte 2.

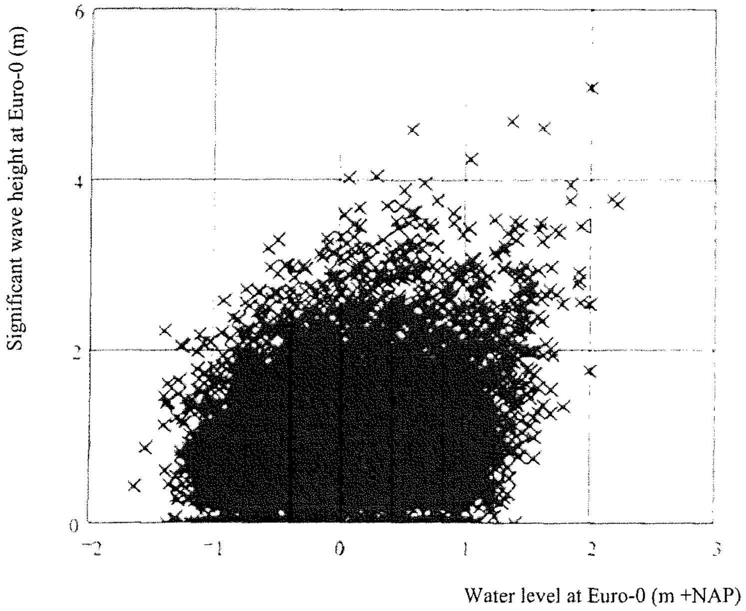


Figure 2.5: Plot of water levels and significant wave heights at Euro-0

According to figure 2.6, the generated data of water levels for daily conditions ($h_{sea,day}$) at Hook of Holland are well supported by the Weibull distribution of equation 2.3, (Voortman/Vrijling 1999). This distribution is also used for the Maasvlakte 2.

$$P\left(h_{sea,day} > \eta\right) = e^{-\left(\frac{\eta-\tau}{w-\tau}\right)^{k}} \tag{2.3}$$

with: $w = 0.23$ m +NAP
$\tau = -1.63$ m +NAP
$k = 3.06$

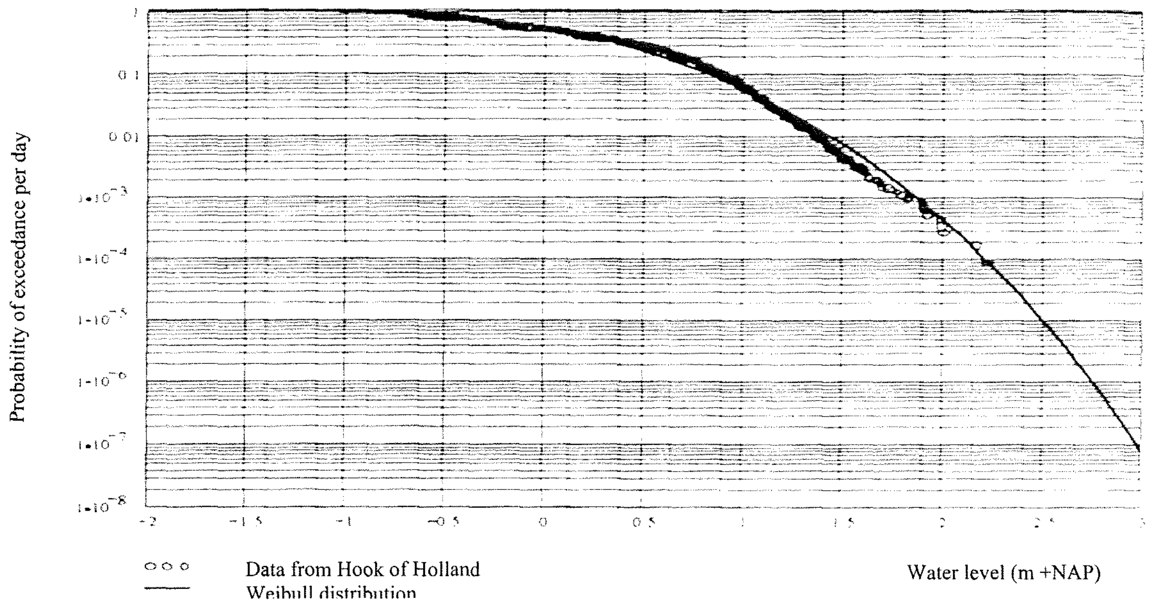


Figure 2.6: Generated data and chosen distribution of the water level for daily conditions

According to figure 2.7, the generated data of significant wave heights for daily conditions ($H_{s,day}$) at Euro-0 are well supported by the Gumbel distribution of equation 2.4, (Voortman/Vrijling 1999). The significant wave height for daily conditions at the Maasvlakte 2 is assumed to have the same distribution as the significant wave height for daily conditions at Euro-0.

$$P(H_{s,day} > \eta) = 1 - e^{-e^{-\alpha(\eta-u)}} \qquad (2.4)$$

with: $\alpha = 2.15$
$u = 0.74$ m

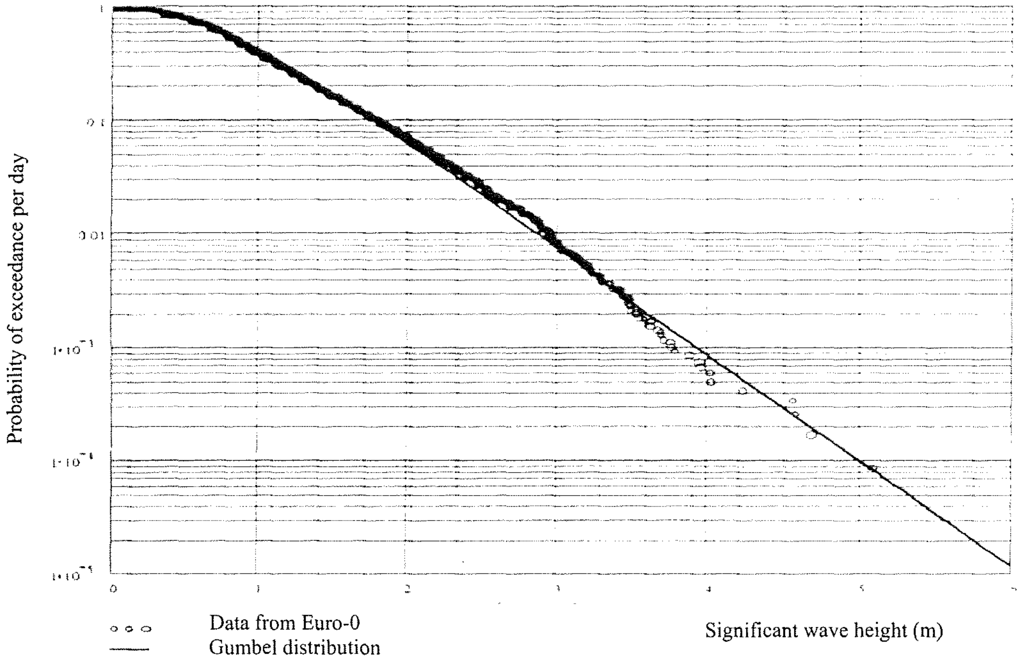


Figure 2.7: Generated data and chosen distribution of the significant wave height for daily conditions

2.2.2 Assumptions regarding the lay-out of the Maasvlakte 2

2.2.2.1 General

- The nature conservation areas on the south side of the Zuiderdike in figure 2.2 are not part of the model.
- As a conservative assumption, the tombolo, the Zuiderdam and the Westerdam are not taken into account any further in the analysis. They do not have relevant influence with regard to normative sollicitations on the sea defence. Their main function is to catch sand for beach preservation and to 'guide' incoming ships into the entrance channel.
- Waves can only enter the Maasvlakte 2 from the entrance channel and through the breakwater.
- The bottom level for the total Maasvlakte 2 is –15 m NAP.

2.2.2.2 Noorderdam (breakwater)

- The Noorderdam is considered to be a rubble mound breakwater. The structure of the cross-section and the unit prices are taken from Laenen (2000).
- The breakwater is assumed to have an armour layer that consists of concrete blocks. The production of alternatively shaped concrete elements as accropodes, dolosses and

tetrapods is more expensive than the production of concrete blocks. These alternatives are not considered in this thesis.

2.2.2.3 Westerdike, Zuiderdike (sea defence)

- The sea defence is considered as a dike and not as a dune.
- The sea defence consists of a sand core with a filter of quarry stone layers on the outer slope
- The main difference between the Westerdike and the Zuiderdike is their orientation. The Zuiderdike is oriented around the 120/300 degrees line and the Westerdike more or less perpendicular on that, around the 30/210 degrees line. It depends on the directional distribution of the incoming waves if there are large differences in their sollicitations. At first it is assumed that they both are sollicitated by the same incoming waves, so they are considered as *one element*. Only perpendicular incoming waves are taken into account

2.2.2.4 Terrain area

- The terrain area is flat and constructed of sand
- The quay wall is constructed at the same height as the terrain area

2.3 Relations between elements

In the prevention of failure of the (system) 'Maasvlakte 2', elements are related to each other. This is because all elements are built to resist the same incoming sollicitation, hydraulic conditions from the North Sea.
Several types of failure are discerned for the Maasvlakte 2; the most important are:

1. Destruction of the breakwater or the sea defence leads to total inundation of the terrain area
2. The sea water level is too high such that the terrain area is inundated
3. The wave height behind the breakwater is too high for ships to connect to pilot vessels
4. The wave height at the quays is too high for loading and unloading activities
5. Waves in the basin are overtopping the quays, such that the terrain area partially inundates
6. Too much water comes over the sea defence such that the terrain area partially inundates

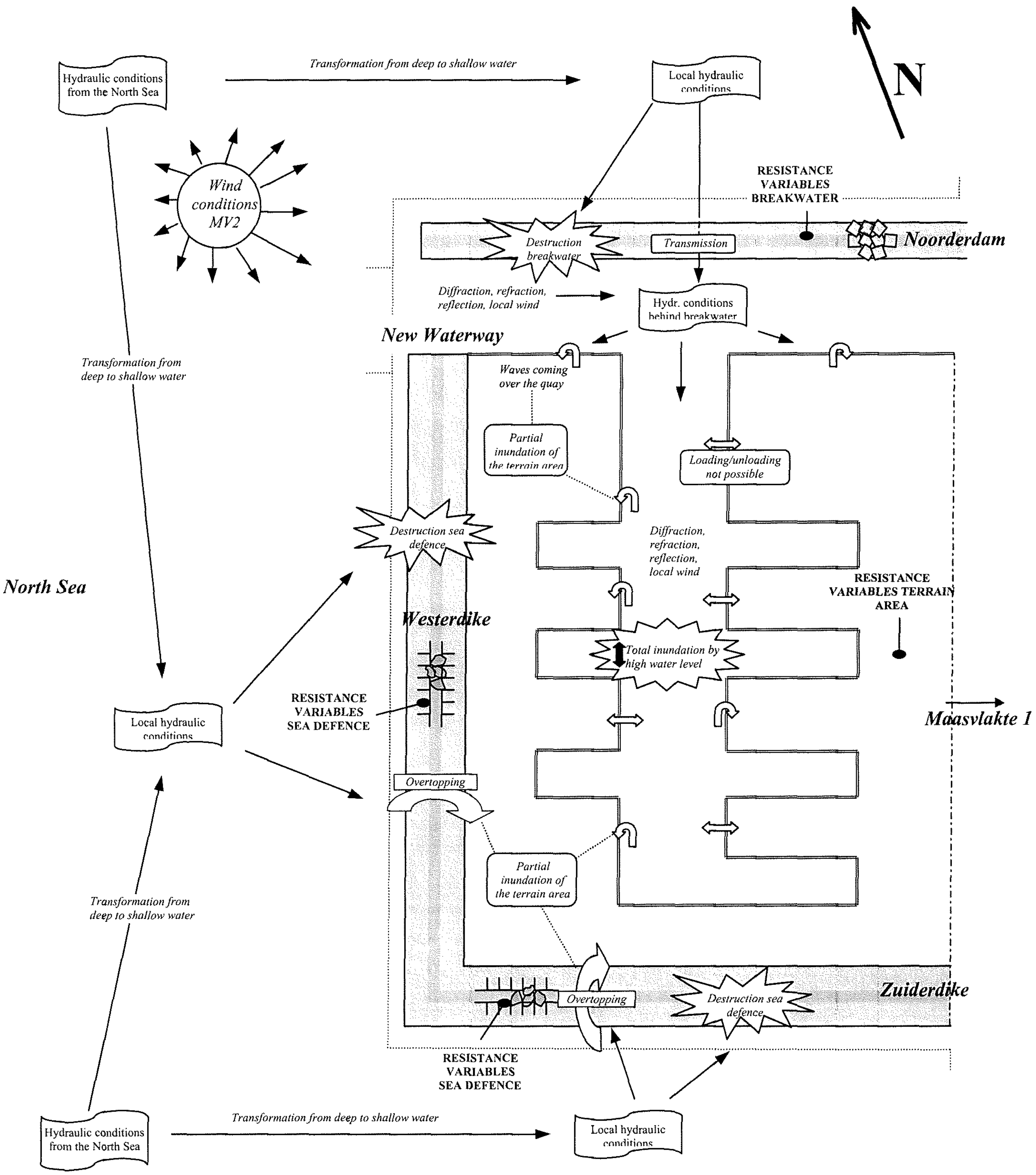


Figure 2.8: Maasvlakte 2 elements and relations

As a result of the six types of failure, three types of damage can be distinguished:

- *Damage to shipping* (no connecting to pilot vessels, no loading/unloading at the quays) by high waves in the entrance channel and/or in the basin
- *Damage by partial inundation of the terrain area*, without destruction of elements
- *Damage by total inundation of the terrain area*, possibly with destruction of elements

With regard to failure type 1, it is assumed that destruction of the breakwater or the sea defence leads to total inundation of the terrain area. This is a conservative approach.

In figure 2.9, a schematization of relations between elements and types of damage is given, including the numbers of the types of failure.

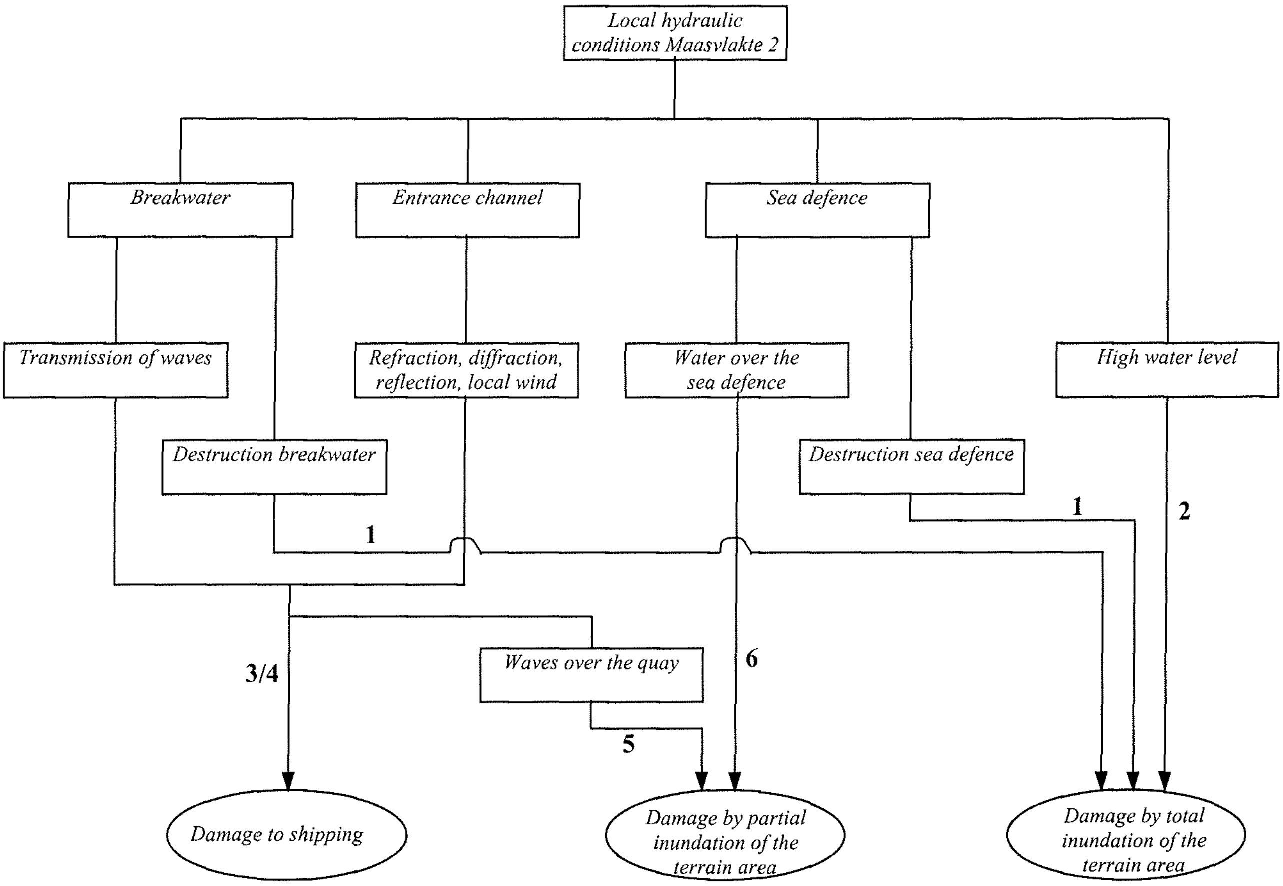


Figure 2.9: Schematization of relations between elements and types of damage

2.4 Decision variables

To determine an optimal design, decision variables are needed for each element. Because the optimisation is performed in terms of safety and by a cost-benefit analysis, it is important that these variables:

- represent the resistance (strength) of the elements
- represent relations between elements
- have important influence on costs and benefits for the Maasvlakte 2

For inundation by transmitted waves through the breakwater, Stroeve and Sies (1999) found a relation between the crest height of the breakwater ($h_{c,bw}$) and the height of the terrain area (h_{ter}). For the same levels of safety for the Maasvlakte 2, a lower crest height for the breakwater leads to a higher terrain area and the other way around. In this thesis, the crest height of the breakwater is used as a decision variable for the breakwater and the height of the terrain area as a decision variable for the terrain area. Both variables are important with regard to investment costs for the breakwater and the terrain area respectively.

The crest height of the sea defence ($h_{c,sd}$) and the height of the terrain area are not related in the same way as the crest height of the breakwater and the height of the terrain area are. The height of the terrain area cannot resist inundation of the terrain area by water coming over the sea defence. However, the crest height is still important for the amount of water coming over the sea defence which leads to partial inundation or even destruction of the sea defence which leads to total inundation of the terrain area. The crest height is therefore used as a decision variable for the sea defence.

Another important variable for water running up and overtopping the sea defence, is the angle of the outer slope (α). For small alfa's (flat slopes), the probability of failure which leads to damage by (partial or total) inundation of the terrain area decreases, but the larger surface of the cross-section leads to higher investment costs. The outer slope angle ($cot(\alpha)_{sd}$) is used as a decision variable for the sea defence.

For a breakwater holds that when in extreme conditions the outer slope (armour) layer is destructed by incoming waves, the rest of the core of the breakwater will follow. Therefore, the diameter of concrete blocks in the armour layer (D_{bw}) is used to represent the strength of the breakwater. This is also assumed for the sea defence. The diameter of quarry stones in the protection layer of the outer slope ($D_{n50,sd}$) is used as a decision variable for the sea defence. Obviously, these strength representative variables are important with regard to possible destruction of the breakwater and the sea defence (failure type 1) and as a result of that, to damage costs by total inundation of the terrain area. Apart from that, they also influence investment costs of these elements.

Table 2.1 gives an overview of the decision variables.

Element	Decision variable	Notation
Breakwater	Crest height	$h_{c,bw}$
	Diameter of the concrete blocks in the armour layer	D_{bw}
Sea defence	Crest height	$h_{c,sd}$
	Diameter of the quarry stones in the protection layer of the outer slope	$D_{n50,sd}$
	Angle of the outer slope	$cot(\alpha)_{sd}$
Terrain area	Height of the terrain area	h_{ter}

Table 2.1: Selected decision variables per element

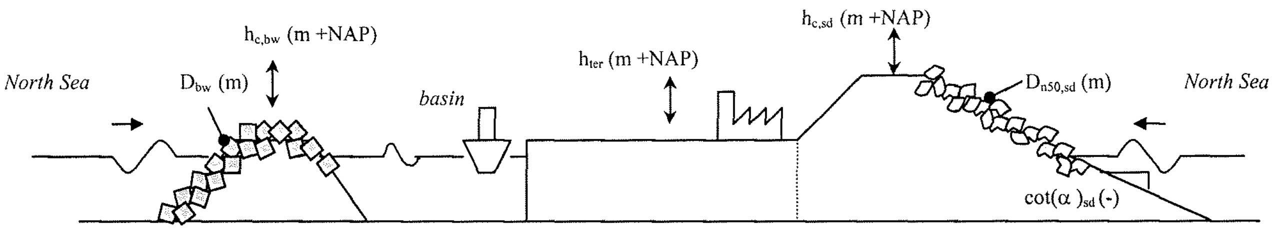


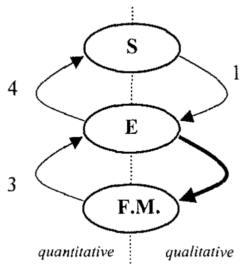
Figure 2.10: Cross-section with decision variables

3. Optimisation method

3.1 Introduction

In this chapter the methodology of the optimisation is explained in detail. Failure modes in relation with the six types of failure of paragraph 2.3 are described in paragraph 3.2. From these failure modes, a selection is made. In paragraph 3.3 and 3.4, the optimisation on element level and system level are described respectively.

3.2 Overview of failure modes



3.2.1 ULS and SLS failure modes

When failure of the Maasvlakte 2 takes place, it is not necessary that the normal functioning is interrupted by extreme hydraulic conditions. This is also possible for daily conditions. Therefore two 'states of failure' (limit states) are distinguished, the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS). In the ULS, destructive failure modes are considered under extreme conditions. In the SLS, non-destructive failure modes are considered under normal (daily) circumstances. For SLS conditions, a distinction can be made in SLS failure modes for which little repair is needed and SLS failure modes for which no repair is needed at all.
The probability of failure by SLS failure modes is expressed in a probability per day, whereas the probability of failure by ULS failure modes is expressed in a probability per year. The consequence of SLS failure modes is expressed in a damage per day. For ULS failure modes, the consequence is expressed in a damage per event. Obviously, for SLS failure modes, daily hydraulic conditions are used and for ULS failure modes, yearly (extreme) hydraulic conditions are used.

3.2.2 Failure modes by element

3.2.2.1 Breakwater

ULS failure modes:
ULS failure modes lead to total inundation of the terrain area. Destruction of the breakwater or the sea defence or an extremely high water level can cause this.
Based on Laenen (2000), the rubble mound breakwater consists of a granular or quarry stone core, one or two filter layers (at outer slope), an armour layer of large concrete units or quarry stones, and a toe construction. Several failure modes can be discerned for a rubble mound breakwater:

- outer slope: erosion of the armour layer, filter instability, slip circle
- inner slope: erosion of the armour layer, slip circle
- core settlement

- subsoil settlement
- erosion of the toe construction

Because the diameter of the concrete blocks in the armour layer is considered to be the most important strength variable for the rubble mound breakwater, erosion of the armour layer is considered as an important failure mode.

Erosion of the armour layer is measured by the damage parameter N_{od}, which represents the damage due to displaced concrete blocks in the armour layer. Failure occurs when the critical damage N_{crit} is exceeded on one place in the breakwater. As a conservative approach it is assumed that this breach leads to destruction of the breakwater and as a result of that damage by total inundation of the terrain area.
In the Van der Meer formulae for armour layer stability of concrete blocks, the fraction $H_s/\Delta D$ represents a ratio of sollicitation (S) and resistance (R). Therefore, in the reliability function $Z=R-S$, $\Delta_{con}D_{bw}$ represents R. When this resistance is exceeded by hydraulic sollicitation, destruction results: $S>R$ and $Z<0$. S is expressed by δd, which is a function of the critical damage N_{crit} (and other hydraulic parameters). A higher value for N_{crit}, which means more displaced blocks before failure takes place, leads to a lower value for δd, a decrease in sollicitation.

$$Z = \Delta_{con} D_{bw} - \delta d < 0 \tag{3.1}$$

with:

Δ_{con} = relative density of concrete (-)
D_{bw} = diameter of the concrete blocks in the armour layer (m)
δd = sollicitation according the Van der Meer formulae (m)

Appendix B contains the Van der Meer formulae for concrete blocks. D_{bw} is the decision variable that is used.

SLS failure modes:
SLS failure modes lead to a temporary disturbance of normal activities or *downtime* of the Maasvlakte 2. The consequence is damage to shipping (paragraph 2.3).
Four relevant phenomena can be discerned with regard to the interaction between incoming waves and the breakwater:

- wave reflection
- wave run-up
- overtopping
- wave transmission

Wave reflection plays a role in front of the breakwater and wave run-up at the outer slope of the breakwater. Because overtopping and transmission are important for waves behind the breakwater, they are the most relevant phenomena to take into account.
In case of overtopping, only the discharge flow *over* the breakwater is measured.
Transmission formulae are mathematical descriptions with which significant wave heights behind breakwaters are calculated. This wave height is the result of water coming *over and through* the breakwater, including the overtopping discharge. Therefore transmission is

considered to be a relevant SLS failure mode. A disadvantage is that, because of this combination, a higher variation of the wave height behind the breakwater is the result. The transmitted wave height is usually expressed by a transmission coefficient K_t.

$$K_t = \frac{H_{st}}{H_{si}} \tag{3.2}$$

with:

K_t = transmission coefficient (-)
H_{st} = transmitted significant wave height behind the breakwater (m)
H_{si} = incoming significant wave height in front of the breakwater (m)

Damage to shipping occurs when the significant wave height behind the breakwater or the significant wave height at the quays in the basin exceeds a critical value H_{cr}. This significant wave height is a combination of transmitted waves and waves which directly enter the entrance channel, influenced by refraction, diffraction and reflection. The influence of refraction, diffraction and reflection is expressed by the factor K_r. The value for K_r has to be calculated by refraction/diffraction software for different locations *(x,y)* in the entrance channel and the basin.

- Damage to shipping by no connecting to pilot vessels occurs when a critical wave height in the entrance channel is exceeded:

$$Z = H_{cr,entrance} - K_{r,xy} K_t H_{si} < 0 \tag{3.3}$$

with:

$H_{cr,entrance}$ = critical value of the significant wave height in the entrance channel (m)
$K_{r,xy}$ = multiplication factor for the influence of refraction, diffraction and reflection (-)

- Damage to shipping with regard to loading/unloading activities occurs when a critical wave height at the quays is exceeded:

$$Z = H_{cr,quays} - K_{r,xy} K_t H_{si} < 0 \tag{3.4}$$

with:

$H_{cr,quays}$ = critical value of the significant wave height at the quays (m)

Formulae to determine the transmission coefficient K_t are described in Appendix B. In these formulae, the freeboard $R_{c,bw}$ and the diameter of the concrete blocks D_{bw} are important variables. The freeboard is determined by the crest height (decision variable) and the sea water level:

$$R_{c,bw} = h_{c,bw} - h_{sea} \tag{3.5}$$

with:

$R_{c,bw}$ = freeboard (m)
$h_{c,bw}$ = crest height of the breakwater (m +NAP)
h_{sea} = sea water level (m +NAP)

3.2.2.2 Terrain area

ULS failure mode:

Total inundation of the terrain area can take place without destruction of hydraulic structures, by an extremely high water level. Despite the extra repair costs to the hydraulic structures, the impact of the consequence is considered to be the same as in case of failure by destruction of hydraulic structures. Total inundation of the terrain area by an extremely high water level is considered as an ULS failure mode for which failure occurs when:

$$Z = h_{ter} - h_{sea} < 0 \tag{3.6}$$

with:

h_{ter} = height of the terrain area (m +NAP)

Obviously, h_{ter} is the only decision variable.

SLS failure mode:

Non-destructive failure modes which lead to damage by partial inundation of the terrain area are related to a critical flooded surface, A_{cr}. Here, two alternatives are distinguished: 'partial inundation by exceedance of a flooded surface' and 'partial inundation by exceedance of the height of the terrain area'.

Partial inundation by exceedance of a flooded surface

Damage by inundation depends on the flooded surface of the terrain area. The inundation depth d is used to create a relation between the water level, wave height in the basin and the flooded surface. As a conservative approach the inundation depth is considered as:

$$d = h_{sea} + K_{r,xy} K_t H_{si} - h_{ter} \tag{3.7}$$

with:

d = inundation depth (m)

To determine the flooded surface of the terrain area A_{flood}, a relation between A_{flood} and d has to be created for which research will be needed. This will not be done in this thesis. Failure occurs when the critical flooded surface A_{cr} is exceeded:

$$Z = A_{cr} - A_{flood} < 0 \tag{3.8}$$

with:

A_{cr} = critical flooded surface of the terrain area (m^2)
A_{flood} = flooded surface of the terrain area (m^2)

Partial inundation by exceedance of the height of the terrain area

In this case, failure is assumed to take place when the height of the terrain area is exceeded by the sum of the water level and the significant wave height in the basin, see equation 3.9. In fact, the critical flooded surface is already assumed to be exceeded when the height of the terrain area is exceeded. Because no inundation depth is taken into account to determine a flooded surface, equation 3.9 can be considered as a simplified, conservative representation of equation 3.8.

Because the only difference between equation 3.6 and 3.9 is the transmitted wave height in the basin ($K_{r,xy}K_tH_{si}$), the probability of failure by equation 3.6 is totally included in the probability of failure by equation 3.9.

$$Z = h_{ter} - h_{sea} - K_{r,xy}K_tH_{si} < 0 \qquad (3.9)$$

Figure 3.1 shows alternative 1 for partial inundation by exceedance of a critical flooded surface and alternative 2 for partial inundation by exceedance of the height of the terrain area.

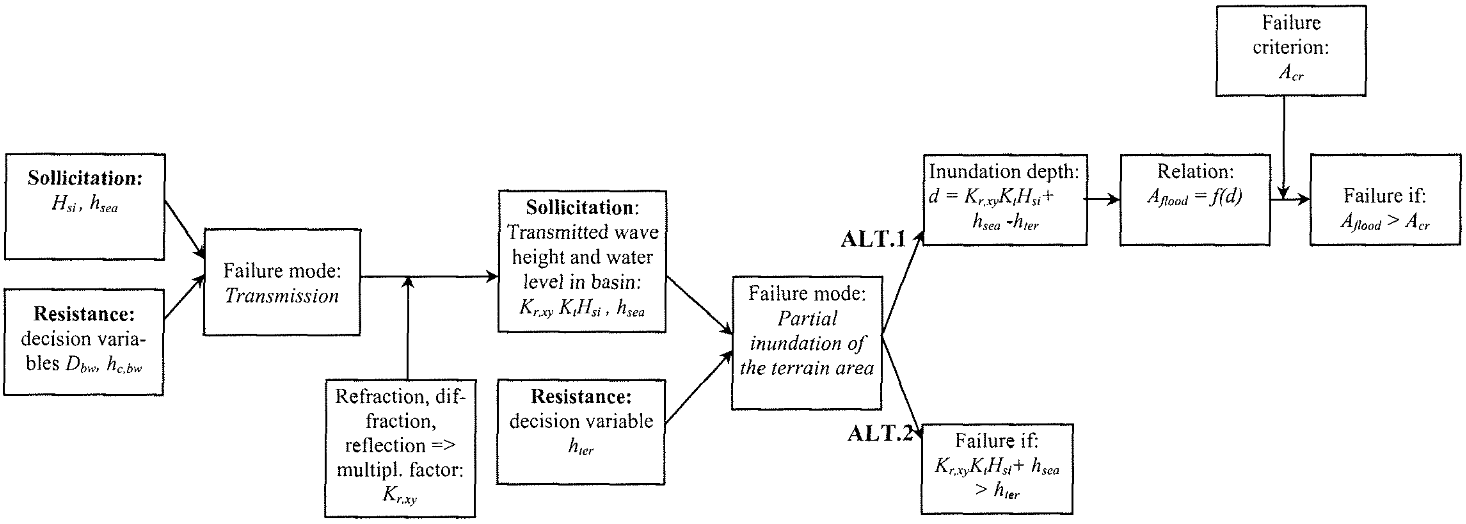


Figure 3.1: Alternatives for partial inundation of the terrain area from the basin

Figure 3.1 shows that for both alternatives failure occurs after *two* failure modes: first the failure mode 'transmission' and then the failure mode 'partial inundation of the terrain area'. This is a complication because the sollicitation of the failure mode 'partial inundation of the terrain area' depends on the values of decision variables of the breakwater. To solve this problem, assumptions have to be made with regard to the $K_{r,xy}K_tH_{si}$-term.

3.2.2.3 Sea defence

ULS failure modes:

Total inundation of the terrain area as a result of destruction of the sea defence, can be caused in several ways. The most important destructive failure modes are:

- outer slope: erosion, sliding
- inner slope: instability by overtopping
- settlement, piping

Instability of the inner slope by overtopping is considered to be an important ULS failure mode for the sea defence. In the reliability function of overtopping, the sollicitation is represented by the *required* freeboard $R_{c,req}$, which is a function of the critical overtopping discharge q_{cr}. For a higher critical overtopping discharge, which means that failure takes place when a higher overtopping discharge is exceeded, a lower required freeboard is the result. Failure occurs when the required crest height ($=R_{c,req} + h_{sea}$) exceeds the crest height of the sea defence, $h_{c,sd}$:

$$Z = h_{c,sd} - h_{sea} - R_{c,req} < 0 \qquad (3.10)$$

with:

$h_{c,sd}$ = crest height of the sea defence (m +NAP)
$R_{c,req}$ = required freeboard (m)

Because instability of the inner slope initiates destruction of the sea defence, overtopping is considered to be an ULS failure mode. This is a conservative approach, because failure is already assumed to take place when a critical overtopping discharge is exceeded.
In Appendix C, the overtopping formulae by Van der Meer and Janssen (1995) are described. Relevant variables are the freeboard ($R_{c,sd}$) and the angle of the outer slope of the sea defence ($cot(\alpha)_{sd}$).

The strength of the protection layer of the outer slope is represented by its stone diameter ($D_{n50,sd}$). Therefore, instability of the outer slope is also considered as an important ULS failure mode. Like in case of the breakwater, as a conservative approach it is assumed that failure on one place in the sea defence leads to destruction of the sea defence and as a result of that damage by total inundation of the terrain area.
Here also, the resistance is represented by $\Delta_{rock}D_{n50,sd}$. The value of the sollicitation (δd_{n50}) depends on several variables like the damage number (S), the amount of waves in a storm (N) and the porosity (P). Failure occurs when (Vrijling,1996):

$$Z = \Delta_{rock} D_{n50,sd} - \delta d_{n50} < 0 \tag{3.11}$$

with:

Δ_{rock} = relative density of quarry stone (-)
$D_{n50,sd}$ = diameter of the quarry stones in the outer slope (m)
δd_{n50} = sollicitation according to the Van der Meer formulae (m)

The considered decision variables in the Van der Meer formulae are $D_{n50,sd}$ and the angle of the outer slope, $cot(\alpha)_{sd}$.

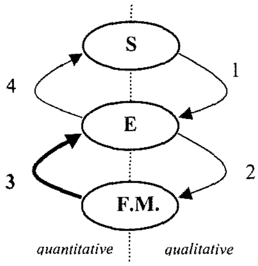
3.2.3 Selection of failure modes

Several failure modes are selected in order to be used in the optimisation. Because of the importance of destructive mechanisms (paragraph 2.3), the ULS failure modes 'extremely high water level', 'overtopping of the sea defence' and 'erosion of the armour layer' for both the breakwater and the sea defence are taken into account.
Apart from that, the SLS failure mode 'transmission' is used with regard to the exceedance of a critical wave height in the entrance channel. The SLS failure mode 'partial inundation of the terrain area' is not considered any further. The influences of diffraction, refraction, reflection and local wind are also not taken into account. With regard to paragraph 2.3, the first three types of failure are taken into account in the optimisation.
Table 3.1 shows the selected failure modes.

Element	**Failure mode**	**SLS/ULS**	**Decision variables**	**Consequence**
Breakwater	Erosion of the armour layer	ULS	D_{bw}	Damage by total inundation of the terrain area
	Transmission	SLS	D_{bw}, $h_{c,bw}$	Damage to shipping
Sea defence	Overtopping	ULS	$h_{c,sd}$, $cot(\alpha)_{sd}$	Damage by total inundation of the terrain area
	Erosion of the outer slope	ULS	$D_{n50,sd}$, $cot(\alpha)_{sd}$	Damage by total inundation of the terrain area
Terrain area	Extremely high water level	ULS	h_{ter}	Damage by total inundation of the terrain area

Table 3.1: Selected failure modes

3.3 Optimisation on element level



3.3.1 Optimisation method

With the selected failure modes and decision variables, probabilistic calculations are executed. The results are combined in the optimisation on element level. The following steps can be discerned in the optimisation on element level:

1. For each decision variable of the considered element, a set of realistic values is chosen.
2. For each failure mode, the probability of failure is calculated for combinations of values of decision variables it contains.
3. The results of the failure modes which belong to the considered element are combined. All combinations of the decision variables of the considered element are known.
4. For each combination of the decision variables of the considered element, investment costs and the level of safety of the element ('probability of failure' or 'risk') are calculated.
5. The values for the investment costs of the element are divided in classes with respect to the accompanying value of the level of safety of the element.
6. For each class, the combination of values of the decision variables for which the investment costs of the element are minimal is selected. This is the optimal combination of decision variables in that class with accompanying level of safety of the element.
7. By interpolation between the values of the minimal investment costs of the element, the *minimal investment costs as a function of the level of safety of the element* are determined. This is done for each element.

This numerical optimisation method is denoted a *direct search* method. In Appendix H, detailed information is given with regard to direct search and alternative numerical optimisation methods.

For optimisation on element level, the decision variables and the investment costs function have to be known for each element. Also, the variable that measures the level of safety has to be chosen:

- The decision variables have been determined for each element in paragraph 2.4
- In Appendix D, E and F, the investment costs functions of the breakwater, the terrain area and the sea defence are described.
- As already mentioned in chapter 1, the level of safety is in this thesis considered in two ways: by 'probability of failure' and by 'risk'. Differences are explained in the following paragraph.

3.3.1.1 Alternative measures for the level of safety

Probability of failure

The optimisation problem on element level with probability of failure as a measure for the level of safety can be written as:

$$\min_{\bar{p}}\left(I_X(\bar{p},\bar{u})+D_{SLS,X}(\bar{p},\bar{u})\right) \tag{3.12}$$

s.t.

$$P_{f,ULS,X}(\bar{p},\bar{u}) \leq P_{f,ULS,max} \tag{3.13}$$

in which:

$$D_{SLS,X}(\bar{p},\bar{u})=\sum_{n=1}^{N}\sum_{m=1}^{M}\frac{365\cdot P_{f,SLS,m}(\bar{p},\bar{u})\cdot D_{SLS,m}}{(1+r-i-g)^n} \tag{3.14}$$

$$P_{f,ULS,X}(\bar{p},\bar{u})=\sum_{k=1}^{K}P_{f,ULS,k}(\bar{p},\bar{u}) \tag{3.15}$$

with:

$I_X(\bullet)$	= investment costs of element X (fl)
$D_{SLS,X}(\bullet)$	= discounted expected damage costs by SLS failure modes for element X (fl)
$D_{SLS,m}$	= expected damage costs per day in case of failure of the mth SLS failure mode (fl)
$P_{f,SLS,m}(\bullet)$	= probability of failure per day of the mth SLS failure mode (-)
$P_{f,ULS,X}(\bullet)$	= ULS probability of failure of element X per year (-)
$P_{f,ULS,k}(\bullet)$	= probability of failure per year of the kth ULS failure mode (-)
K	= number of ULS failure modes for element X (-)
M	= number of SLS failure modes for element X (-)
N	= number of years (-)
r	= interest in one year (-)
i	= inflation in one year (-)
g	= economic growth in one year (-)
$\bar{p}$	= vector of decision variables on element level
$\bar{u}$	= vector of random and deterministic input variables

Because SLS and ULS failure modes have different units and consequences (see paragraph 3.2.1), the probability of failure of the element cannot be determined by the sum of the probabilities of failure for the ULS and SLS failure modes.
To avoid this problem, the expected damage costs by SLS failure modes are added to the investment costs of the element. This is a good assumption when the expected damaged costs of an SLS mechanism depend on decision variables of one element, like in case of damage by shipping in the entrance channel, for which the breakwater is the only element of influence. When decision variables of more elements influence the expected damage costs of an SLS mechanism, it is less obvious to which element the expected damage costs have to be added. This is the case for 'flooding of the terrain area' by transmitted waves.

When the damage costs by SLS failure modes are added to the investment costs, only the ULS failure modes remain. Their probabilities of failure are added according to 3.15, which is an *upper bound* for the ULS probability of failure.
The *sum* of ULS probabilities of failure implies *independent* failure modes. Because sollicitations of all ULS failure modes are based on the same hydraulic conditions, independence of failure modes is probably not a good assumption and leads to an overestimation of the ULS probability of failure. However, the assumption of independence can be considered as a conservative approach. The influence of dependencies between failure modes is not analysed in this thesis.
The result of the optimisation on element level are the minimal investment costs (including $D_{SLS,X}$) as a function of the *ULS probability of failure of the element per year*. The expected damage costs by ULS failure modes are calculated on *system* level, after summing of ULS probabilities of failure per year of all elements.

Risk

The optimisation problem on element level with risk as a measure for the level of safety can be written as:

$$\min_{\vec{p}} I_X(\vec{p},\vec{u}) \tag{3.16}$$

s.t.

$$R_X(\vec{p},\vec{u}) \leq R_{\max} \tag{3.17}$$

in which:

$$R_X(\vec{p},\vec{u}) = \sum_{k=1}^{K} P_{f,ULS,k}(\vec{p},\vec{u}) \cdot D_{ULS}(\vec{p},\vec{u}) + \sum_{m=1}^{M} 365 \cdot P_{f,SLS,m}(\vec{p},\vec{u}) \cdot D_{SLS,m} \tag{3.18}$$

with:

$R_X(\bullet)$ = risk of element *X* per year (fl)
$D_{ULS}(\bullet)$ = expected damage costs per event in case of ULS failure (fl)

In case of *risk* as a measure for the level of safety, the product of probability of failure and consequence has the same unit for both SLS and ULS failure modes, so *it is allowed to add risks*. Summing of risks of SLS and ULS failure modes leads to *risk of the element* and can for both SLS and ULS failure modes be expressed in units of money per year. This is an easy and practical way to combine SLS and ULS failure modes. The result of the optimisation on element level are the minimal investment costs as a function of the *risk of the element per year*.

Some remarks with regard to risk as a measure for the level of safety:

- It is not possible to take dependencies between ULS failure modes of different elements into account. The ULS probability of failure is already multiplied with its consequence in the optimisation on element level. Therefore, it is only possible to calculate an *upper bound* of the risk.

- Consequences now also influence the level of safety. It is questionable if this is a realistic assumption.
- The value of D_{ULS} depends on the investment costs of the *system*, so on the values of the decision variables of *all* elements. Assumptions with regard to the investment costs of the two not considered elements have to be made, to determine the value of R_X.
- When both SLS and ULS probability of failure are included in the level of safety, they are not optimised with regard to each other anymore. This is in contrast with the optimisation with ULS probability of failure as a measure for the level of safety, where for each value of the ULS probability of failure, an optimal value of the SLS probability of failure (in combination with the investment costs) is found. As a result of this difference, a slightly less optimal design may be found.

Note:

Theoretically, it would be a better alternative to determine the investment costs as a function of *both* the ULS and SLS probability of failure. This leads to curves in the $P_{f,SLS,X}$-$P_{f,ULS,X}$-plane, for which investment costs are equal. Like this, $P_{f,SLS,X}$ and $P_{f,ULS,X}$ are optimised with regard to each other *and* the damage costs by the SLS failure modes do not have to be added to the investment costs of the element. Disadvantages are that it is a more complex method to use in practice and the number of decision variables on system level increases as a result of $P_{f,SLS,X}$, which also becomes a decision variable on system level. This alternative is not considered in this thesis.

Figure 3.2 shows a flow diagram of the optimisation on element level for both probability of failure and risk as a measure for the level of safety. This diagram has to be followed for each element of the system.

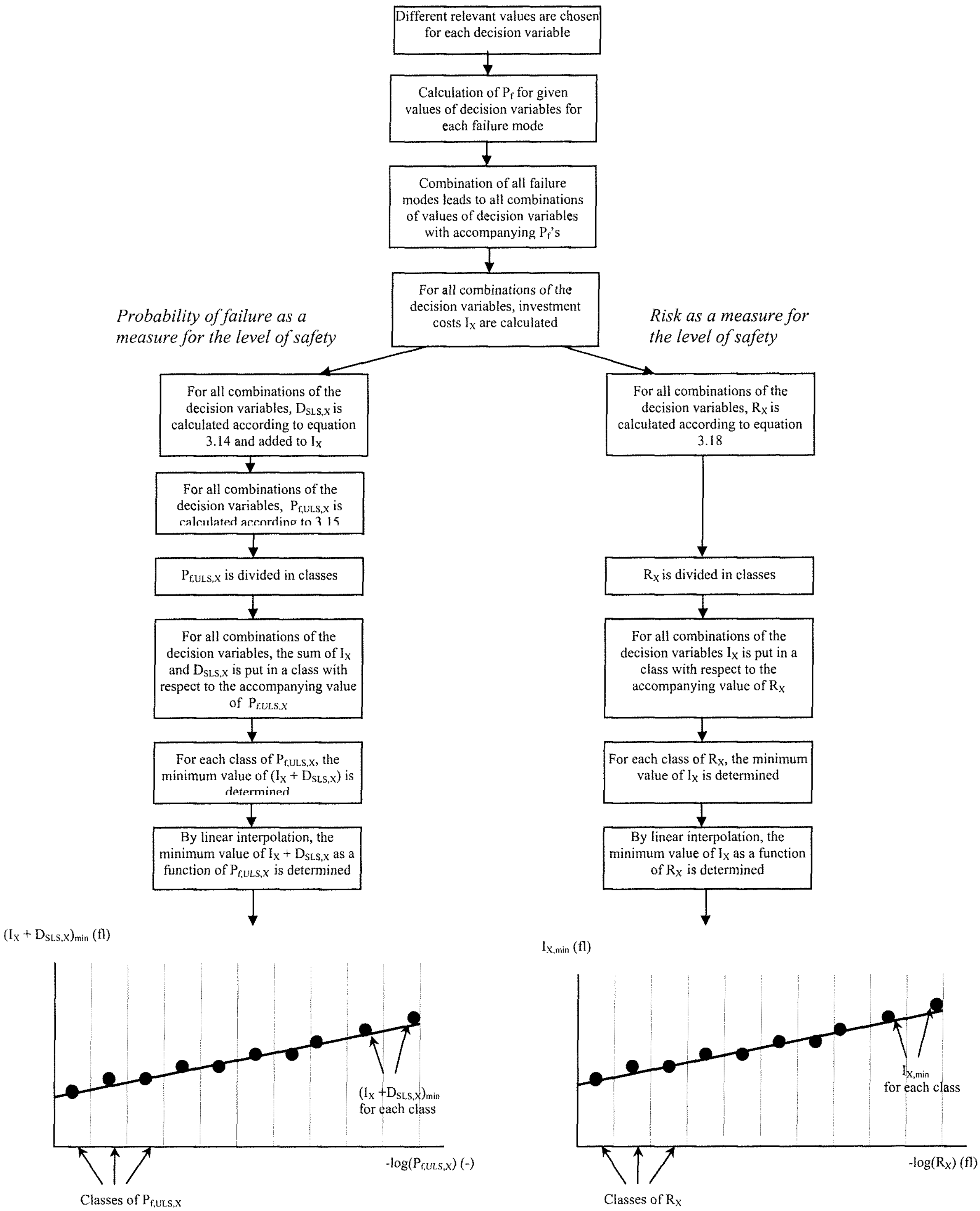


Figure 3.2: Optimisation on element level

3.3.2. Fault trees and risk trees of the elements of the Maasvlakte 2

In case of probability of failure as a measure for the level of safety, for each element a *fault tree* can be constructed. In a fault tree on element level, the ULS probability of failure of the element in one year is determined by the sum of the probabilities of failure of the ULS failure modes of the element in one year. Herefore, OR-gates are used, see figure 3.3.

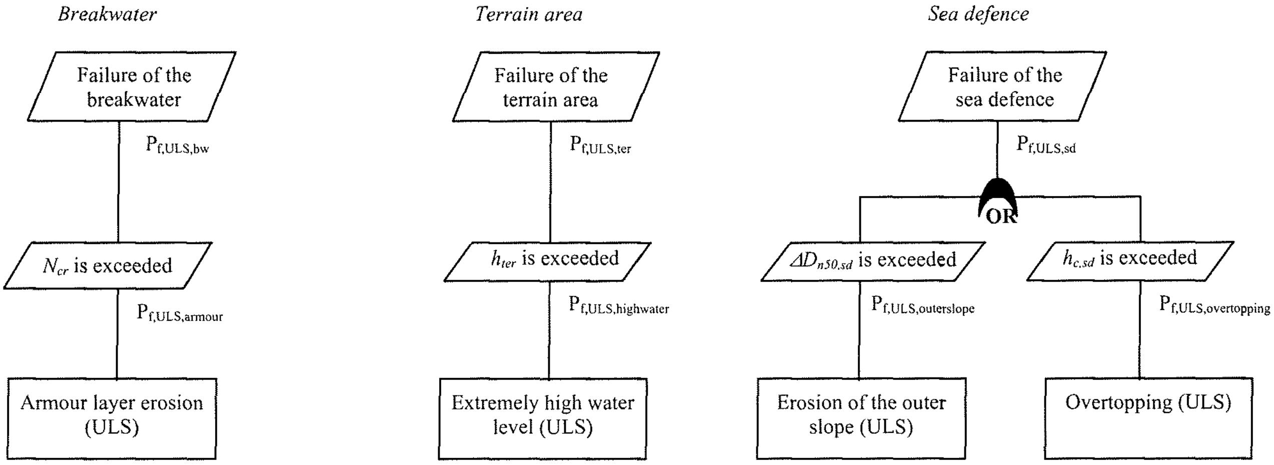


Figure 3.3: Fault trees by element

Like fault trees can be constructed in case of probability of failure as a measure for the level of safety, *risk trees* can be constructed in case of risk as measure for the level of safety. In a risk tree on element level, the risk of the element in one year is determined by the sum of the risks by ULS and SLS failure modes of the element in one year. The top events of the fault trees and risk trees are the same. Figure 3.4 shows the risk trees for the three elements.

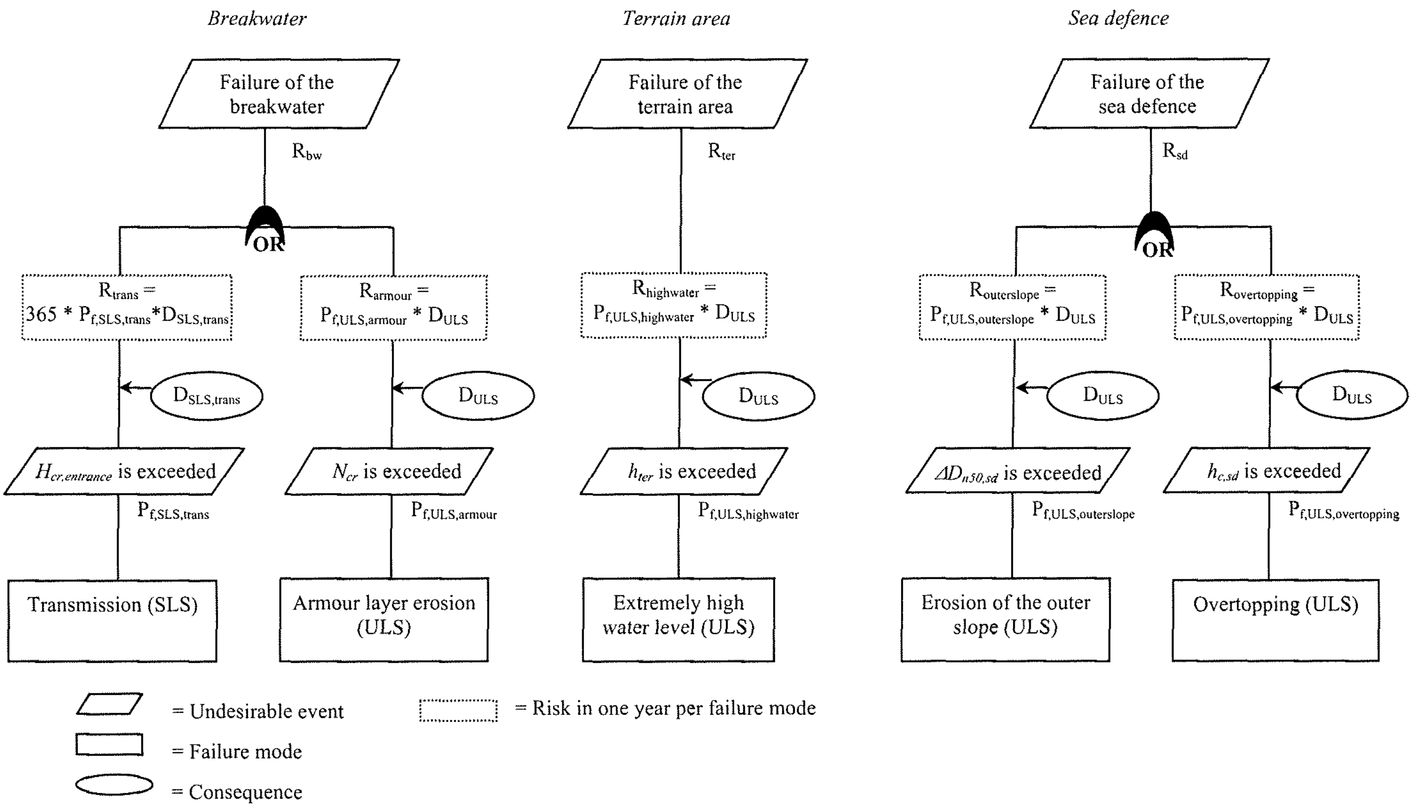


Figure 3.4: Risk trees by element

3.3.3 Optimisation of the breakwater

For the breakwater, the failure modes 'erosion of the armour layer (ULS)' and 'transmission (SLS)' are used to determine the minimal investment costs as a function of the level of safety of the breakwater. For erosion of the armour layer, the diameter of the concrete blocks in the armour layer, D_{bw}, is the only decision variable. Transmission contains both D_{bw} and the crest height of the breakwater, $h_{c,bw}$, as decision variables.

For erosion of the armour layer, D_{bw} is described as a function of $P_{f,ULS,armour}$ in figure 3.5a. For different values of D_{bw}, values of $P_{f,ULS,armour}$ are calculated.
For transmission, D_{bw} and $h_{c,bw}$ are determined as a function of $P_{f,SLS,trans}$ in figure 3.5b. For different combinations of D_{bw} and $h_{c,bw}$, values of $P_{f,SLS,trans}$ are determined. This leads to curves in the D_{bw}-$h_{c,bw}$-plane for which the $P_{f,SLS,trans}$ is constant. The closer to the origin of the coordinate system, the higher is the value of $P_{f,SLS,trans}$.

Then, failure modes are combined. A distinction is made between probability of failure and risk as a measure for the level of safety.
In figure 3.5c, for each combination of D_{bw} and $h_{c,bw}$, investment costs of the breakwater (I_{bw}) are calculated. Also $D_{SLS,bw}$ is calculated by equation 3.14 (with $M=1$) and added to the investment costs for each combination. Apart from D_{bw} and $h_{c,bw}$, random and deterministic variables like the unit price of concrete and the length of the breakwater also determine the values of I_{bw} and $D_{SLS,bw}$. These variables are represented by the vector $\vec{u}$.
Then, $P_{f,ULS,bw}$ is determined for each combination of D_{bw} and $h_{c,bw}$. Because only one ULS mechanism is taken into account, holds that $P_{f,ULS,bw} = P_{f,ULS,armour}$. This leads to lines in the D_{bw}-$h_{c,bw}$-plane for which $P_{f,ULS,bw}$ is equal. These lines are horizontal, because $h_{c,bw}$ does not influence the ULS probability of failure of the breakwater. Then, when classes are determined with regard to the value of $P_{f,ULS,bw}$, the minimal value of $I_{bw}+D_{SLS,bw}$ is determined for each class. Each minimal value has an accompanying combination of D_{bw} and $h_{c,bw}$ and an accompanying value for $P_{f,ULS,bw}$. In the D_{bw}-$h_{c,bw}$-plane, a minimal investment costs (including $D_{SLS,X}$) function is shown.

In figure 3.5d, for each combination of D_{bw} and $h_{c,bw}$, investment costs of the breakwater (I_{bw}) are calculated. The other variables are also represented by $\vec{u}$. For each combination, also R_{bw} is calculated according to equation 3.18 (with K=1 and M=1). This leads to lines in the D_{bw}-$h_{c,bw}$-plane for which R_{bw} is equal. These lines are not horizontal, because $h_{c,bw}$ does have influence on the risk of the breakwater.
Then, when classes are determined with regard to the value of R_{bw}, the minimal value of I_{bw} is determined for each class. Each minimal value has an accompanying combination of D_{bw} and $h_{c,bw}$ and an accompanying value for R_{bw}. In the D_{bw}-$h_{c,bw}$-plane, a minimal investment costs function is shown.

Now, the minimal investment costs (including $D_{SLS,X}$) as a function of $P_{f,ULS,bw}$ (figure 3.5e) and the minimal investment costs as a function of R_{bw} (figure 3.5f) can be determined.

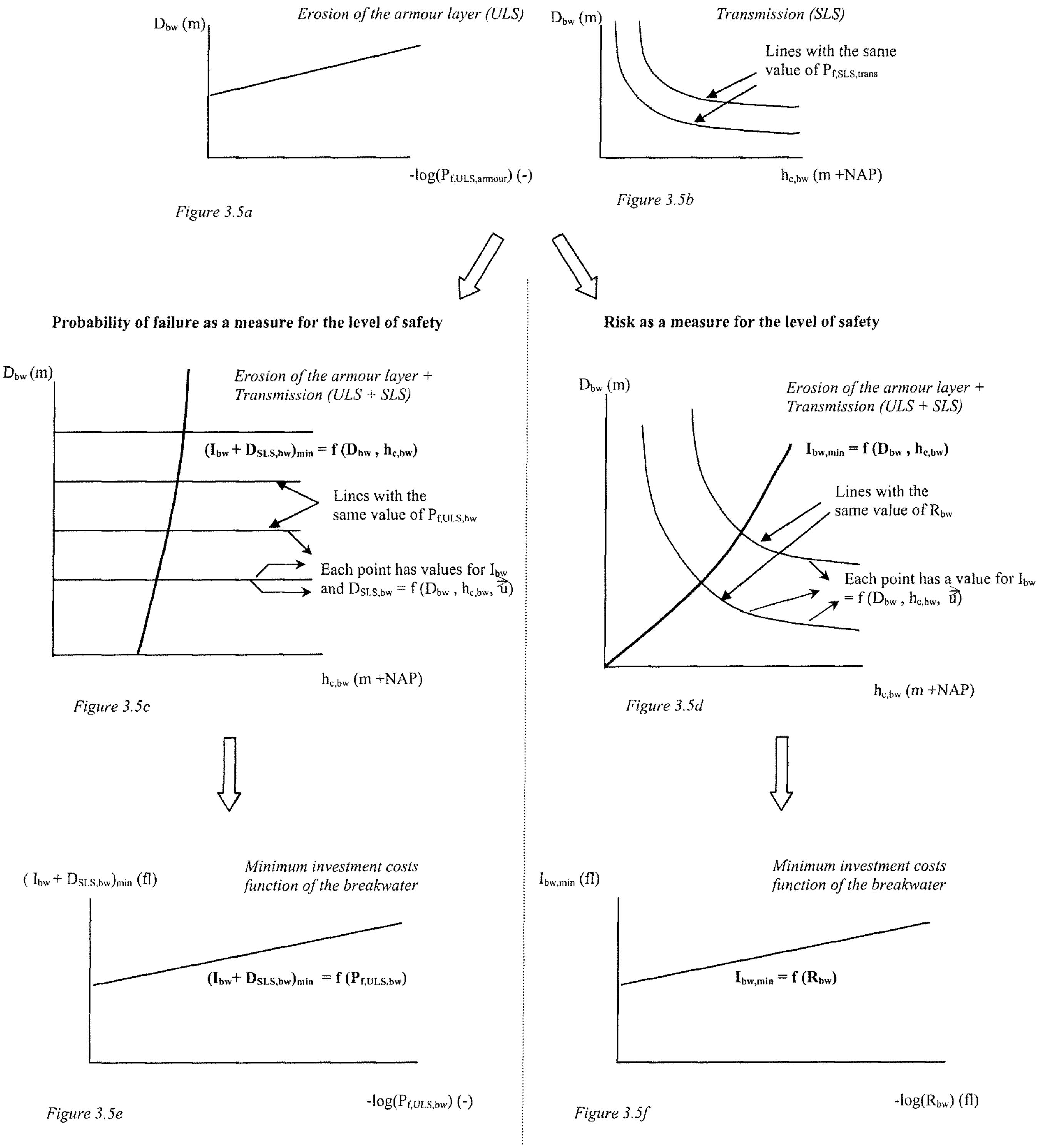


Figure 3.5a-f: Optimisation of the breakwater

3.3.4 Optimisation of the terrain area

For the terrain area, only the failure mode 'extremely high water level (ULS)' is used to determine the minimal investment costs as a function of the level of safety of the terrain area. The height of the terrain area h_{ter} is the only decision variable.

In figure 3.6a, h_{ter} is described as a function of $P_{f,ULS,highwater}$. For different values of h_{ter}, values of $P_{f,ULS,highwater}$ are calculated.

Then, a distinction is made between probability of failure and risk as a measure for the level of safety. Because only one failure mode with one decision variable is taken into account, the combination of more failure modes and decision variables is not possible. Therefore, figure 3.6b is almost the same as figure 3.6a, apart from the notation of the variable on the horizontal axis. According to 3.15 holds that $P_{f,ULS,ter} = P_{f,ULS,highwater}$.
For figure 3.6c holds the same, but now with R_{ter} on the horizontal axis. R_{ter} is calculated with equation 3.18 (with K=1 and M=0).

Apart from the height of the terrain area, the investment costs of the terrain area (I_{ter}) also depend on other random and deterministic variables, $\vec{u}$. As a result of one decision variable of the element, the calculated values of the investment costs for different values of h_{ter} are also the *minimal* investment costs of the terrain area. The 'minimal' investment costs as a function of $P_{f,ULS,ter}$ and R_{ter} are given in figure 3.6d and 3.6e respectively.

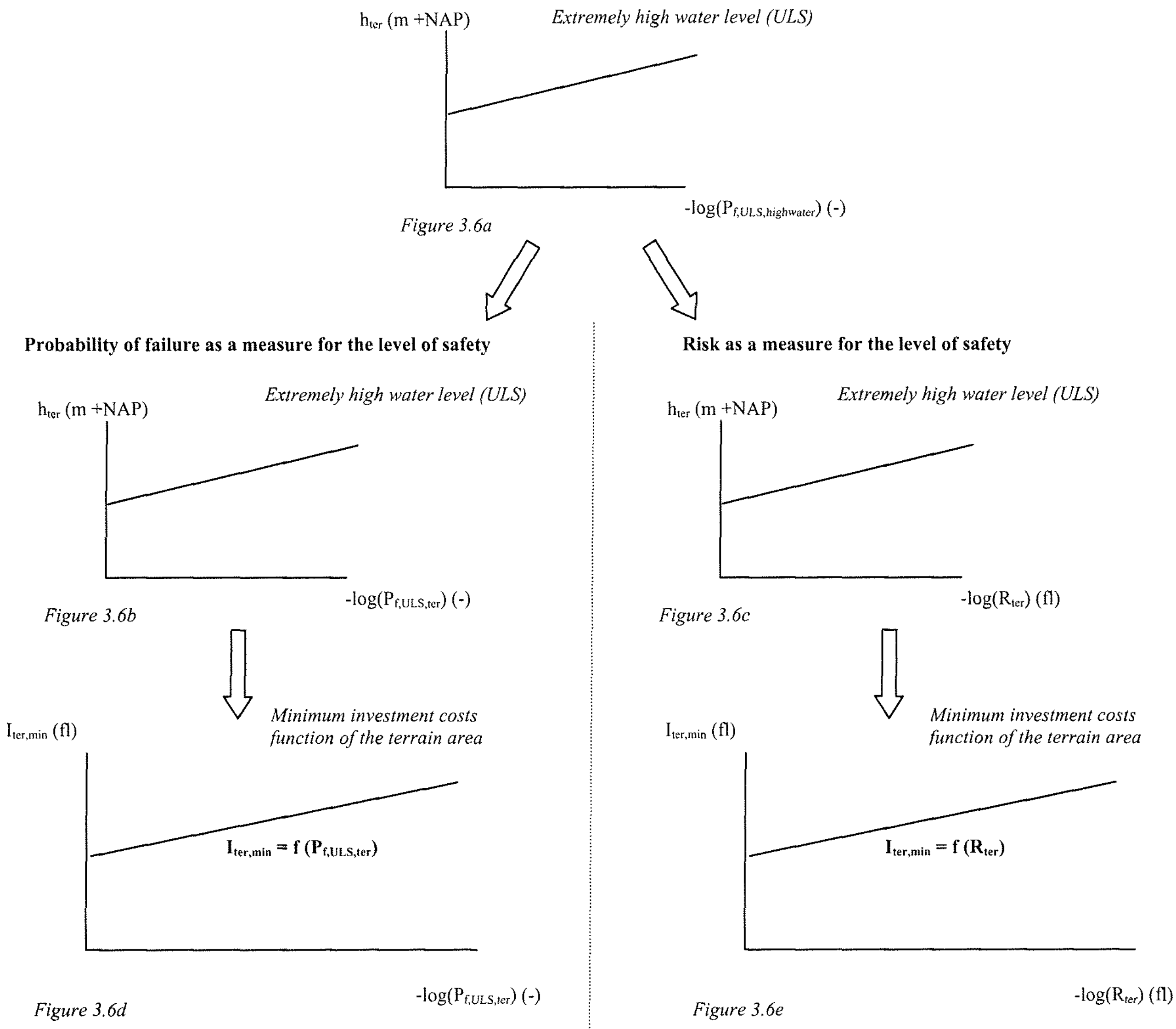


Figure 3.6a-e: Optimisation of the terrain area

3.3.5 Optimisation of the sea defence

For the sea defence, the failure modes 'erosion of the outer slope (ULS)' and 'overtopping (ULS)' are used to determine the minimal investment costs as a function of the level of safety of the sea defence. For erosion of the outer slope, the diameter of the quarry stones in the protection layer of the outer slope, $D_{n50,sd}$, and the angle of the outer slope, $cot(\alpha)_{sd}$, are the decision variables. Overtopping contains the crest height of the sea defence, $h_{c,sd}$, and $cot(\alpha)_{sd}$ as decision variables.

For erosion of the outer slope, $D_{n50,sd}$ and $cot(\alpha)_{sd}$ are described as a function of $P_{f,ULS,outerslope}$ in figure 3.7a. For different values of $D_{n50,sd}$ and $cot(\alpha)_{sd}$, values of $P_{f,ULS,outerslope}$ are calculated. This leads to curves in the $D_{n50,sd}$ - $cot(\alpha)_{sd}$ -plane for which $P_{f,ULS,outerslope}$ is constant.
For overtopping, $h_{c,sd}$ and $cot(\alpha)_{sd}$ are described as a function of $P_{f,ULS,overtopping}$ in figure 3.7b. For different values of $h_{c,sd}$ and $cot(\alpha)_{sd}$, values of $P_{f,ULS,overtopping}$ are calculated. This leads to curves in the $h_{c,sd}$ - $cot(\alpha)_{sd}$ -plane for which $P_{f,ULS,overtopping}$ is constant.
For both figure 3.7a and 3.7b holds: the closer to the origin of the coordinate system, the higher the value of $P_{f,ULS,outerslope}$ and $P_{f,ULS,overtopping}$.

Then, failure modes are combined. A distinction is made between probability of failure and risk as a measure for the level of safety.
In figure 3.7c, for each combination of $D_{n50,sd}$, $h_{c,sd}$ and $cot(\alpha)_{sd}$, investment costs of the sea defence (I_{sd}) are calculated. Apart from $D_{n50,sd}$, $h_{c,sd}$ and $cot(\alpha)_{sd}$, the investment costs also depend on random and deterministic variables like the unit price of rock stone and the length of the sea defence. These variables are represented by $\vec{u}$.
Also $P_{f,ULS,sd}$ is determined for each combination of $D_{n50,sd}$, $h_{c,sd}$ and $cot(\alpha)_{sd}$. According to equation 3.15 holds that $P_{f,ULS,sd} = P_{f,ULS,outerslope} + P_{f,ULS,overtopping}$. This leads to planes in the $D_{n50,sd}$-$h_{c,sd}$-$cot(\alpha)_{sd}$ -space for which $P_{f,ULS,sd}$ is equal. Then, when classes are determined with regard to the value of $P_{f,ULS,sd}$, the minimal investment costs are determined for each class. Each value of the minimal investment costs has an accompanying combination of $D_{n50,sd}$, $h_{c,sd}$ and $cot(\alpha)_{sd}$ and an accompanying value for $P_{f,ULS,sd}$. In the $D_{n50,sd}$-$h_{c,sd}$-$cot(\alpha)_{sd}$ -space, a minimal investment costs function is shown.

In figure 3.7d, for each combination of $D_{n50,sd}$, $h_{c,sd}$, and $cot(\alpha)_{sd}$, investment costs of the sea defence are calculated. The other variables are also represented by $\vec{u}$. For each combination, also R_{sd} is calculated according to equation 3.18 (with K=2 and M=0). This leads to planes in the $D_{n50,sd}$-$h_{c,sd}$-$cot(\alpha)_{sd}$ -space for which R_{sd} is equal.
When classes are determined with regard to the value of R_{sd}, the minimal investment costs are determined for each class. Each value of the minimal investment costs has an accompanying combination of $D_{n50,sd}$, $h_{c,sd}$ and $cot(\alpha)_{sd}$ and an accompanying value for R_{sd}. In the $D_{n50,sd}$-$h_{c,sd}$-$cot(\alpha)_{sd}$ -space, a minimal investment costs function is shown.

Now, the minimal investment costs as a function of $P_{f,ULS,sd}$ (figure 3.7e) and the minimal investment costs as a function of R_{sd} (figure 3.7f) can be determined.

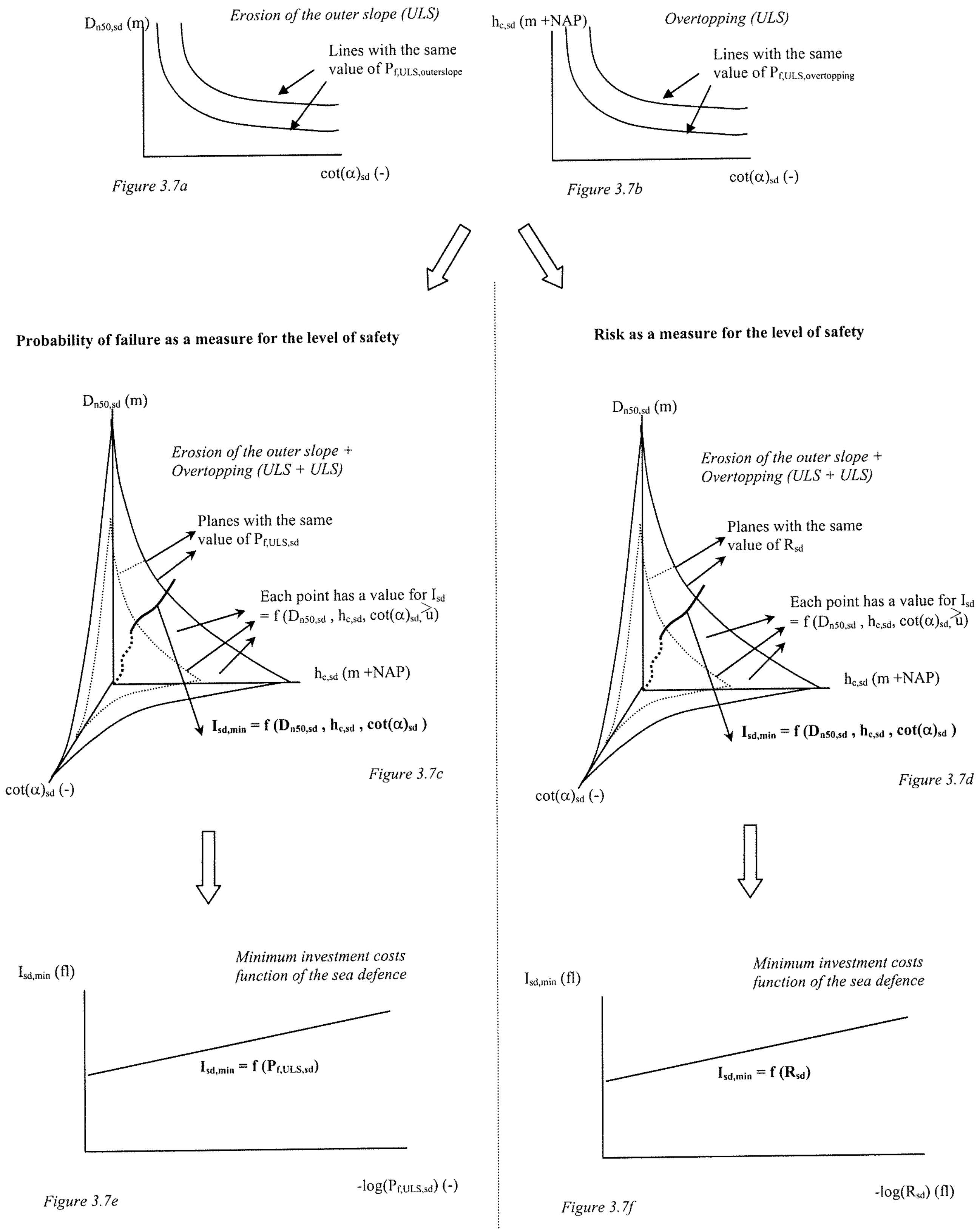
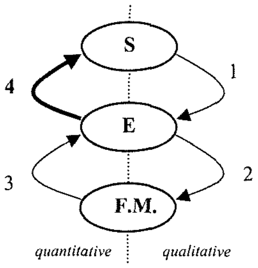


Figure 3.7a-f: Optimisation of the sea defence

3.4 Optimisation on system level



3.4.1 Optimisation method

In the optimisation on system level, the ULS probabilities of failure and risks of the elements per year become decision variables. Instead of the 6 decision variables that are used on element level, on system level only 3 decision variables are used. The results from the optimisation on element level are combined. The following steps can be discerned in the optimisation on system level:

1. As a result of the optimisation on element level, values of the minimal investment costs with accompanying values for the level of safety, the decision variables on system level, are selected for each element.
2. From the selected values of the three decision variables, all possible combinations are made. For each combination, the minimal investment costs of the *system* are calculated.
3. For each combination of the decision variables, the expected damage costs and expected benefits of the system are calculated
4. For each combination of the decision variables, the NPV of the total costs of the system and the level of safety of the system are calculated.
5. The NPV's of the total costs are divided in classes with respect to the accompanying value of the level of safety of the system.
6. For each class, the combination of values of the decision variables for which the NPV of the total costs of the system are minimal, is selected. This is the optimal combination of decision variables in that interval with accompanying level of safety of the system.
7. By interpolation between the values of the minimal NPV of the total costs of the system, the *minimal NPV of the total costs as a function of the level of safety of the system* are determined.
8. The minimal value of this function represents the optimum value of the *minimal NPV of the total costs* with accompanying *optimal level of safety of the system.* From these values, the optimal level of safety, the minimal investment costs and the optimal values of the decision variables of each element are also known. The values of these decision variables represent the *economic optimal design of the Maasvlakte 2.*

ULS probability of failure as a measure for the level of safety

The optimisation problem on system level with ULS probability of failure as a measure for the level of safety can be written as:

$$\min_{\vec{P}_{f,ULS,X}} TC_{sys}\left(\vec{P}_{f,ULS,X}\right) \tag{3.19}$$

s.t.

$$P_{f,ULS,sys}\left(\vec{P}_{f,ULS,X}\right) \leq P_{f,ULS,\max} \tag{3.20}$$

in which:

$$P_{f,ULS,sys}\left(\vec{P}_{f,ULS,X}\right) = \sum_{X=1}^{Xtot} P_{f,ULS,X} \tag{3.21}$$

$$TC_{sys}\left(\vec{P}_{f,ULS,X}\right) = I_{sys,\min}\left(\vec{P}_{f,ULS,X}\right) + D_{sys}\left(\vec{P}_{f,ULS,X}\right) - B_{sys}\left(\vec{P}_{f,ULS,X}\right) \tag{3.22}$$

$$I_{sys,\min}\left(\vec{P}_{f,ULS,X}\right) = \sum_{X=1}^{Xtot} I_{X,\min}\left(P_{f,ULS,X}\right) \tag{3.23}$$

$$D_{sys}\left(\vec{P}_{f,ULS,X}\right) = \sum_{n=1}^{N}\sum_{X=1}^{Xtot} \frac{P_{f,ULS,X} \cdot D_{ULS,X}}{\left(1+r+i-g\right)^{n}} \tag{3.24}$$

$$B_{sys}\left(\vec{P}_{f,ULS,X}\right) = \sum_{n=1}^{N} \frac{\left(1-\sum_{X=1}^{Xtot} P_{f,ULS,X}\right) \cdot B}{\left(1+r+i-g\right)^{n}} \tag{3.25}$$

with:

$\vec{P}_{f,ULS,X}$	= vector of ULS probabilities of failure of the elements per year, the decision variables on system level
$P_{f,ULS,sys}(\bullet)$	= ULS probability of failure of the system per year (-)
$TC_{sys}(\bullet)$	= NPV of the total costs of the system (fl)
$I_{sys,min}(\bullet)$	= minimal investment costs of the system (fl)
$D_{sys}(\bullet)$	= NPV of the expected damage costs of the system (fl)
$B_{sys}(\bullet)$	= NPV of the expected benefits of the system (fl)
B	= benefits of the system per year (fl)
X_{tot}	= number of elements in the system (-)

Under the assumption of independent failure modes, summing of ULS probabilities of failure of the elements per year leads to the upper bound of the ULS probability of failure of the system per year in equation 3.21. In equation 3.23, the NPV of the expected damage costs of SLS failure modes are included in the investment costs of the elements ($I_{X,min}$ $(P_{f,ULS,X})$).

Risk as a measure for the level of safety

The optimisation problem on system level with risk as a measure for the level of safety can be written as:

$$\min_{\vec{R}_X} TC_{sys}\left(\vec{R}_X\right) \tag{3.26}$$

s.t.

$$R_{sys}\left(\vec{R}_X\right) \leq R_{\max} \tag{3.27}$$

in which:

$$R_{sys}\left(\vec{R}_X\right) = \sum_{X=1}^{Xtot} R_X \tag{3.28}$$

$$TC_{sys}\left(\vec{R}_X\right) = I_{sys,\min}\left(\vec{R}_X\right) + D_{sys}\left(\vec{R}_X\right) - B_{sys}\left(\vec{R}_X\right) \tag{3.29}$$

$$I_{sys,\min}\left(\vec{R}_X\right) = \sum_{X=1}^{Xtot} I_{X,\min}\left(R_X\right) \tag{3.30}$$

$$D_{sys}\left(\vec{R}_X\right) = \sum_{n=1}^{N} \frac{\sum_{X=1}^{Xtot} R_X}{\left(1+r+i-g\right)^n} \tag{3.31}$$

$$B_{sys}\left(\vec{R}_X\right) = \sum_{n=1}^{N} \frac{\left(1-P_{f,ULS,sys}\right)\cdot B}{\left(1+r+i-g\right)^n} \tag{3.32}$$

with:

R_{sys}	= risk of the system per year (fl)
$\vec{R}_X$	= vector of risks of the elements per year, the decision variables on system level

Note:

B_{sys} depends on the value of $P_{f,ULS,sys}$. This value is according to equation 3.21 determined by the sum of the ULS probabilities of the elements. For risk as a measure for the level of safety, this equation can also be used because from the optimal values of R_X on element level, the accompanying values of $P_{f,ULS,X}$ are also known.

Figure 3.8 shows a flow diagram of the optimisation on system level for both probability of failure and risk as a measure for the level of safety.

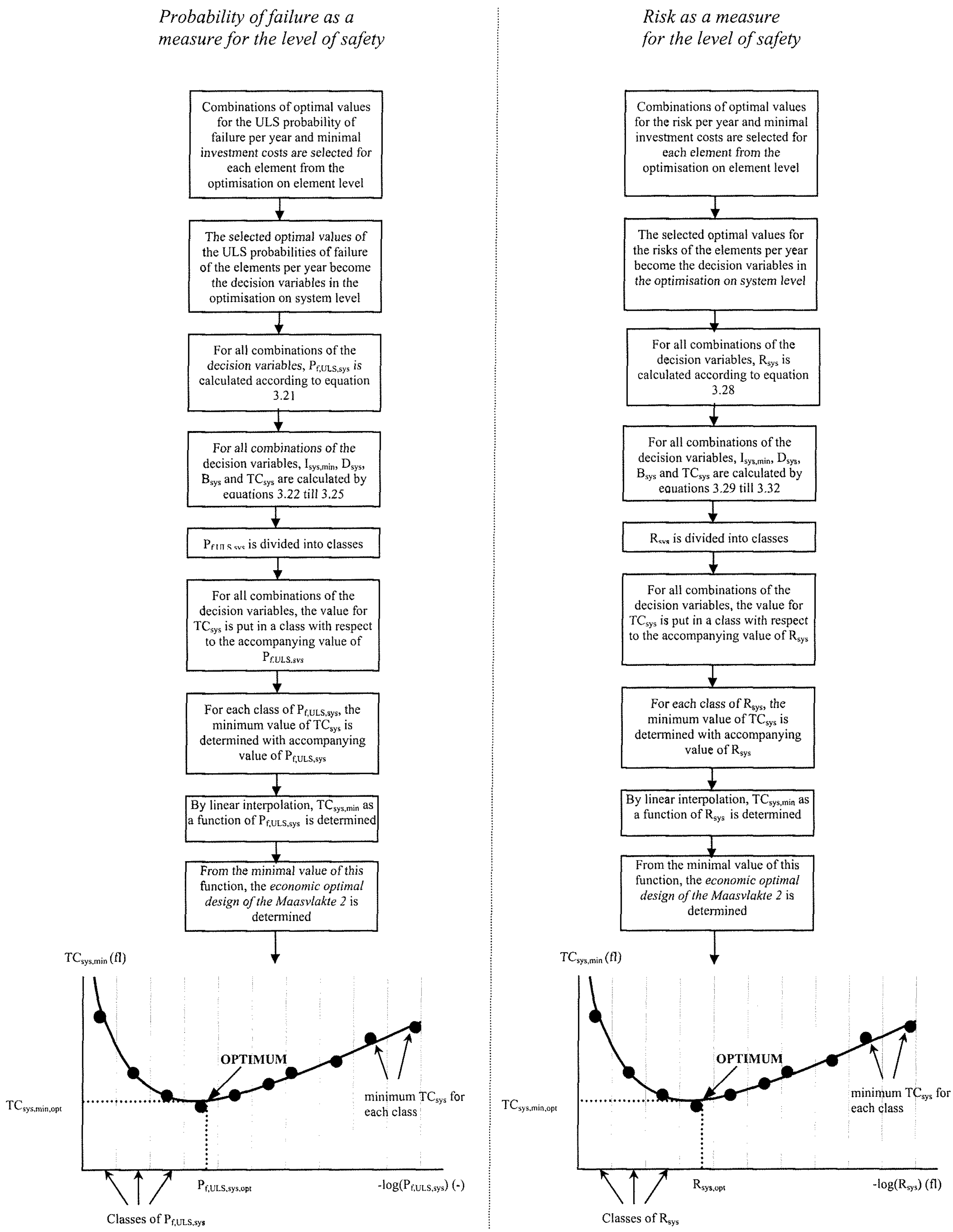


Figure 3.8: Optimisation on system level

3.4.2 Fault tree and risk tree of the Maasvlakte 2

The combination of the fault trees by element leads to the fault tree for the system, figure 3.9. The ULS probability of failure of the Maasvlakte 2 per year is determined by the sum of the ULS probabilities of failure of the elements per year, the upper bound.

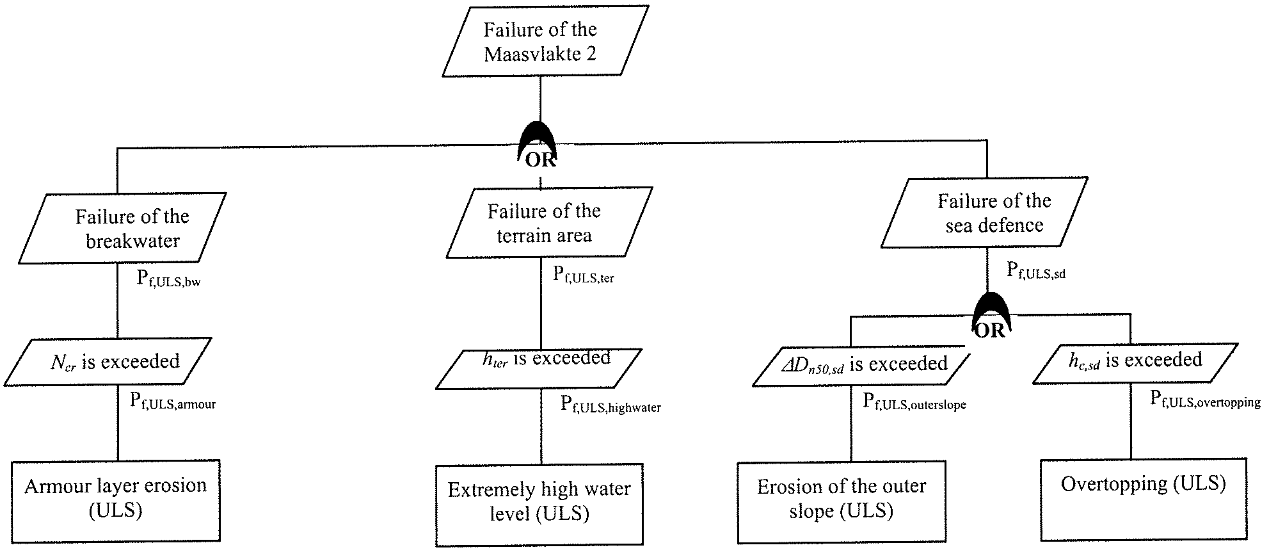


Figure 3.9: Fault tree of the Maasvlakte 2

The combination of the risk trees per element leads to the risk tree of the Maasvlakte 2, figure 3.10. The risk of the Maasvlakte 2 (R_{mv2}) is determined by the sum of the risks of the elements, the upper bound.

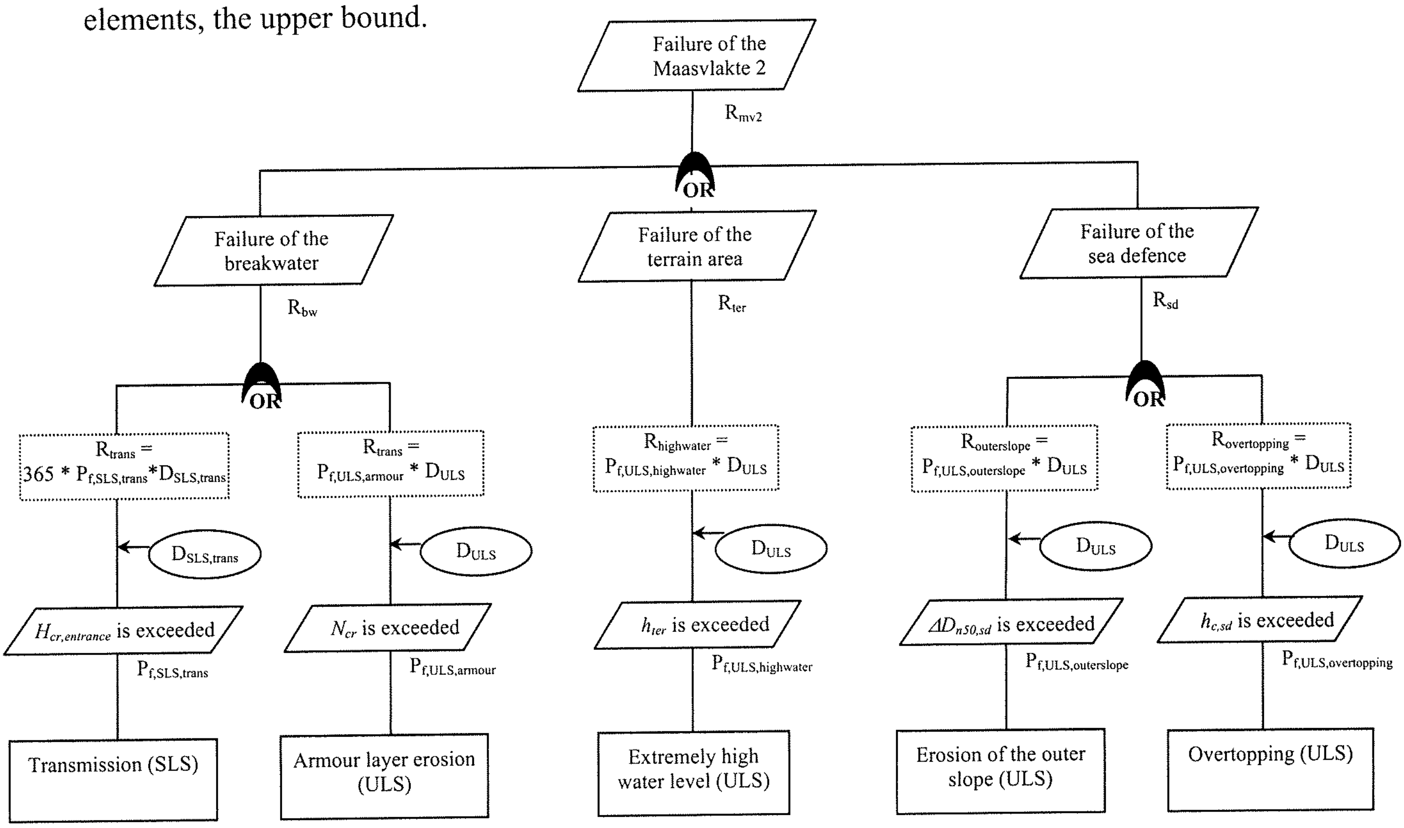


Figure 3.10: Risk tree of the Maasvlakte 2

3.4.3 Optimisation of the Maasvlakte 2

In the optimisation on system level, $P_{f,ULS,bw}$, $P_{f,ULS,ter}$ and $P_{f,ULS,sd}$ are the decision variables in case of ULS probability of failure as a measure for the level of safety and R_{bw}, R_{ter} and R_{sd} in case of risk as measure for the level of safety.

In figure 3.11a, for combinations of $P_{f,ULS,bw}$, $P_{f,ULS,ter}$ and $P_{f,ULS,sd}$, values of $P_{f,ULS,mv2}$ are calculated by equation 3.21 with $X_{tot} = 3$. This leads to planes for which $P_{f,ULS,mv2}$ has the same value.
For each combination of $P_{f,ULS,bw}$, $P_{f,ULS,ter}$, $P_{f,ULS,sd}$ and $P_{f,ULS,mv2}$, values for $I_{mv2,min}$, D_{mv2} and B_{mv2} can be calculated from equation 3.23 till 3.25 respectively. From these results, TC_{mv2} can be calculated for each combination. Combinations of $P_{f,ULS,bw}$, $P_{f,ULS,ter}$ and $P_{f,ULS,sd}$, with accompanying values for $P_{f,ULS,mv2}$ and TC_{mv2} are known now.
When classes are determined with regard to the value of $P_{f,ULS,mv2}$, the minimal value for TC_{mv2} is determined for each class. Each minimal value of TC_{mv2} has an accompanying combination of $P_{f,ULS,bw}$, $P_{f,ULS,ter}$ and $P_{f,ULS,sd}$ and an accompanying value for $P_{f,ULS,mv2}$. In the $P_{f,ULS,bw}$-$P_{f,ULS,ter}$-$P_{f,ULS,sd}$ -space, a *minimal* TC_{mv2}-function is shown.

In figure 3.11b the same procedure is followed for combinations of R_{bw}, R_{ter} and R_{sd}. At first values of R_{mv2} are calculated by equation 3.28 with $X_{tot} = 3$. This leads to planes for which R_{mv2} has the same value. Then, values for $I_{mv2,min}$, D_{mv2} and B_{mv2} are calculated from equation 3.30 till 3.32 respectively. From equation 3.29, TC_{mv2} is calculated. Combinations of R_{bw}, R_{ter} and R_{sd}, with accompanying values for R_{mv2} and TC_{mv2} are known now.
When classes are determined with regard to the value of R_{mv2}, the minimal value for TC_{mv2} is determined for each class. Each minimal value of TC_{mv2} has an accompanying combination of R_{bw}, R_{ter} and R_{sd} and an accompanying value for R_{mv2}. In the R_{bw}-R_{ter}-R_{sd} -space, a *minimal* TC_{mv2}-function is shown.

Now, $TC_{mv2,min}$ as a function of $P_{f,ULS,mv2}$ (figure 3.11c) and the $TC_{mv2,min}$ as a function of R_{mv2} (figure 3.11d) can be determined. The minimal values of these functions, represent the optima ($TC_{mv2,min,opt}$), with accompanying optimal value for the ULS probability of failure in one year, $P_{f,ULS,mv2,opt}$ (figure 3.11c) and optimal value for the risk in one year, $R_{mv2,opt}$ (figure 3.11d). From these optimal values, the optimal values of the decision variables on system level are known ($\vec{P}_{f,ULS,X}$ and $\vec{R}_X$). From these values, the optimal values of the decision variables on element level ($\vec{p}$) are known. These values represent the *economic optimal design of the Maasvlakte 2.*

Probability of failure as a measure for the level of safety

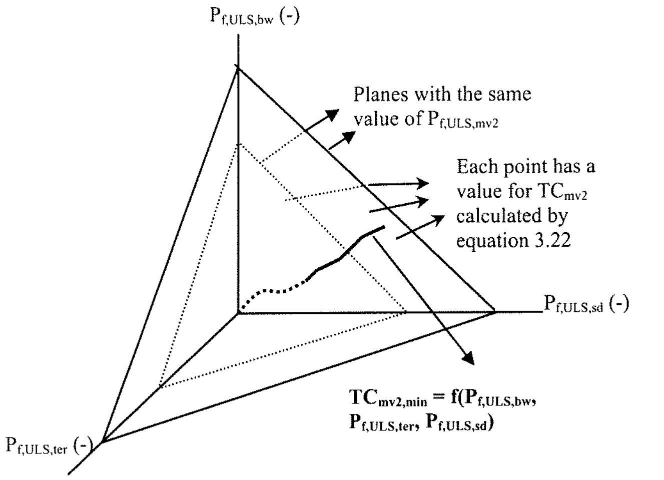


Figure 3.11a

Risk as a measure for the level of safety

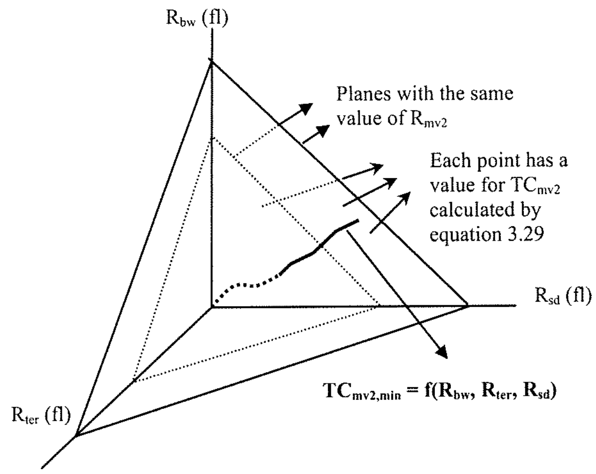


Figure 3.11b

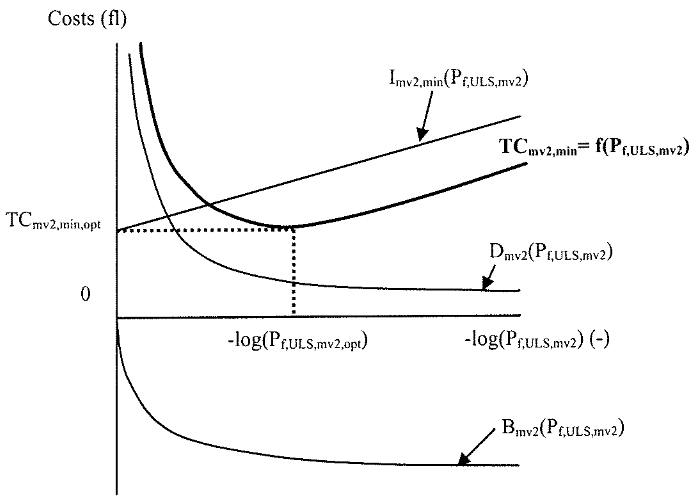


Figure 3.11c

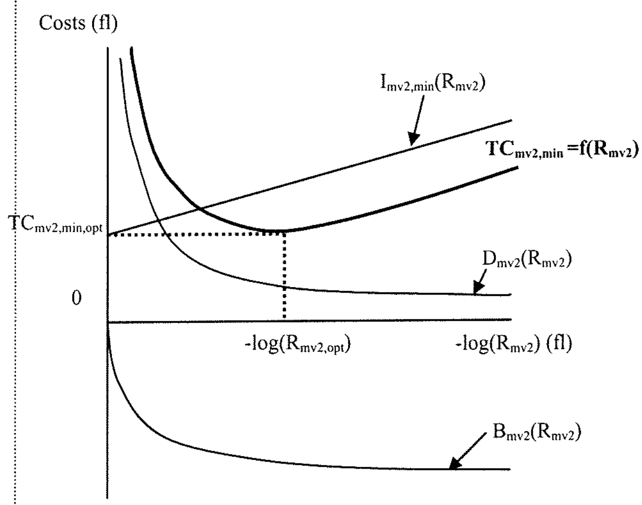


Figure 3.11d

Figure 3.11a-d: Optimisation of the Maasvlakte 2

4. Deterministic design

4.1 General

In the deterministic design, a probability of exceedance is chosen for the hydraulic conditions. For ULS failure modes (yearly conditions), the water level with a probability of exceedance of 10^{-4} (once in 10.000 years) is considered. For the SLS failure mode (daily conditions), both water level and significant wave height with a probability of exceedance of $2.7*10^{-3}$ (1 day per year) are considered.
For ULS failure modes, the significant wave height is calculated from the water level with the hydraulic model of Appendix A.1. The adapted Bruinsma-function determines the expected value of the significant wave height at Euro-0 for a given value of the water level at Hook of Holland. To determine the significant wave height at the Maasvlakte 2, two extra distributions are added:

- conditional probability distribution of the significant wave height at Euro-0 for given water levels at Hook of Holland, $f(H_{s,Euro}/h_{sea,H.o.H.})$; a normal distribution with a mean value of 0 m and a standard deviation of 0.6 m.
- conditional probability distribution of the local significant wave height for given significant wave heights at Euro-0, $f(H_{s,local}/H_{s,Euro})$; a normal distribution with a mean value of 0 m and a standard deviation of 0.21 m.

These conditional distributions are *not* taken into account in the deterministic design, so the *expected value of the significant wave height* is used.
With the formulae for the selected failure modes (Appendix B and C), the values of the decision variables are calculated for the deterministic design. For some of the non-decision variables in the formulae, a probability distribution is used. In the calculations of the deterministic design, the mean value is taken for these variables.
For the sea defence, three decision variables have to be calculated by two equations ('overtopping' and 'erosion of the outer slope'). Therefore, the outer slope angle of the sea defence ($cot(\alpha)_{sd}$) has a chosen value of 4.6.

4.2 Calculation of values of the decision variables

- *Erosion of the armour layer of the breakwater (ULS)*

Variable	Description	Value	Unit
$h_{sea,yr}$	Maximum water level with a probability of exceedance of 10^{-4} per year	4.99	m +NAP
$H_{s,yr}$	Maximum significant wave height per year, based on $h_{sea,yr}$	5.63	m
s_{0p}	Wave steepness based on peak period	0.038	-
T_p/T_m	Peak period / Average period	1.2	-
Δ_{con}	Relative density of concrete	1.4	-
N_{cr}	Critical damage for exposed core	2	-
N	Number of waves in a storm	3000	-
Calculated			
D_{bw}	Diameter of the concrete blocks in the armour layer	1.67	m

Table 4.1: Deterministic calculation for erosion of the armour layer of the breakwater

- *Transmission of the breakwater (SLS)*

Variable	Description	Value	Unit
$h_{sea,day}$	Maximum water level with a probability of exceedance of $2.75*10^{-3}$ per day	1.69	m +NAP
$H_{s,day}$	Maximum significant wave height with a probability of exceedance of $2.75*10^{-3}$ per day	3.48	m
s_{0p}	Wave steepness based on the peak period	0.038	-
D_{bw}	Diameter of the concrete blocks in the armour layer	1.67	m
B	Crest width = $3*D_{bw}$	5.01	m
H_{crit}	Critical significant wave height behind the breakwater	0.5	m
Calculated			
$R_{c,bw}$	Freeboard of the breakwater with respect to design water level	2.05	m
$h_{c,bw}$	Crest height of the breakwater	3.74	m +NAP

Table 4.2: Deterministic calculation for transmission of the breakwater

- *Extremely high water level (ULS)*

Variable	Description	Value	Unit
$h_{sea,yr}$	Maximum water level with a probability of exceedance of 10^{-4} per year	4.99	m +NAP
Calculated			
h_{ter}	Height of the terrain area	4.99	m + NAP

Table 4.3: Deterministic calculation extremely high water level

- *Overtopping of the sea defence (ULS)*

Variable	Description	Value	Unit
$h_{sea,yr}$	Maximum water level with a probability of exceedance of 10^{-4} per year	4.99	m +NAP
$H_{s,yr}$	Maximum significant wave height per year, based on $h_{sea,yr}$	5.63	m
s_{0p}	Wave steepness based on the peak period	0.038	-
$cot(\alpha)_{sd}$	1/ tan(angle of the outer slope)	4.6	-
modfac	Model factor	1	-
γ_{tot}	Reduction factor berm, wave angle and friction = $\gamma_b*\gamma_\beta*\gamma_{fr}$	1	-
q_{crit}	Critical overtopping discharge per metre width	10	l/s/m
Calculated			
$R_{c,sd}$	Freeboard of the sea defence with respect to design water level	7.72	m
$h_{c,sd}$	Crest height of the sea defence	12.71	m +NAP

Table 4.4: Deterministic calculation for overtopping of the sea defence

- *Erosion of the outer slope of the sea defence (ULS)*

Variable	Description	Value	Unit
$h_{sea,yr}$	Maximum water level with a probability of exceedance of 10^{-4} per year	4.99	m +NAP
$H_{s,yr}$	Maximum significant wave height per year, based on $h_{sea,yr}$	5.63	m
s_{0p}	Wave steepness based on the peak period	0.038	-
$cot(\alpha)_{sd}$	= 1/ tan(angle of the outer slope)	4.6	-
Δ_{rock}	Relative density of rock stone	1.65	-
T_p/T_m	Peak period / Average period	1.2	-
P	Porosity	0.3	-
N	Number of waves in a storm	3000	-
S	Damage parameter in case of an exposed core	15	-
Calculated			
$D_{n50,sd}$	Diameter of quarry stones in the protection layer of the outer slope	0.85	m

Table 4.5: Deterministic calculation for erosion of the outer slope of the sea defence

Table 4.6 and figure 4.1 represent the deterministic design.

Decision variable	Value	Unit
D_{bw}	1.67	m
$h_{c,bw}$	3.74	m +NAP
h_{ter}	4.99	m +NAP
$D_{n50,sd}$	0.85	m
$h_{c,sd}$	12.71	m +NAP
$cot(\alpha)_{sd}$	4.6	-

Table 4.6: Deterministic design

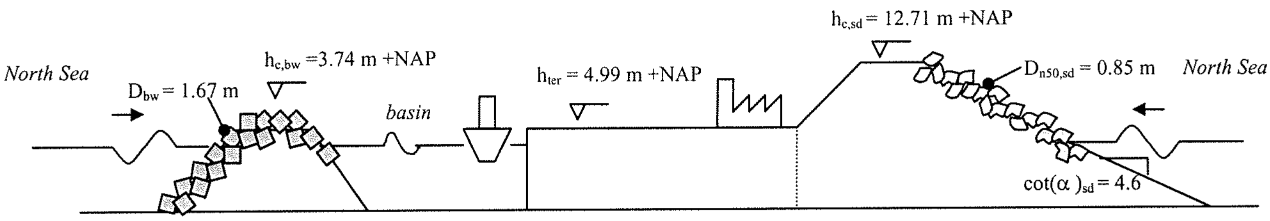


Figure 4.1: Cross-section Maasvlakte 2 for deterministic design

4.3 Calculation of costs

To determine the NPV of the total costs for the deterministic design, the investment costs of the elements and the NPV of the expected damage costs and benefits have to be known. These values depend on the values of the decision variables of table 4.6, the probabilities of failure of the failure modes and other deterministic and random variables. Table 4.7 till 4.9 contain the other variables, divided into geometry variables, unit prices and other parameters respectively.

Geometry variables		Value	Unit
$cot(\alpha)_{os,bw}$	1/tan(outer slope angle) of the breakwater	1.5	-
$cot(\alpha)_{is,bw}$	1/tan(inner slope angle) of the breakwater	1.5	-
L_{bw}	Length of the breakwater	5700	m
A_{ter}	Surface of the terrain area	1250	ha
l_{quay}	Quay length	25000	m
$cot(\alpha)_{is,sd}$	1/tan(inner slope angle) of the sea defence	3	-
$d_{os,sd}$	Depth on the outer side of the sea defence	-15	m (+NAP)
$d_{is,sd}$	Depth on the inner side of the sea defence	$h_{c,sd}$ - 4	m (+NAP)
B_{sd}	Crest width of the sea defence	6	m
L_{sd}	Length of the sea defence	12000	m

Table 4.7: Geometry variables

Unit prices	Nominal diameter (m)		Value	Unit
UP_{rock}	0.30 – 0.49	Unit price of rock stone	75	fl/m^3
	0.49 – 0.72		80	fl/m^3
	0.72 – 1.04		90	fl/m^3
	1.04 – 1.31		100	fl/m^3
	1.31 – 1.55		120	fl/m^3
	1.55 – 2.12		140	fl/m^3
	>2.12		200	fl/m^3
UP_{con}		Unit price of concrete	750	fl/m^3
UP_{sand}		Unit price of sand	5	fl/m^3

Table 4.8: Unit prices

Other parameters		Value	Unit
r	Interest in one year	0.06	-
g	Economic growth in one year	0.02	-
i	Inflation in one year	0.02	-
N	Considered number of years for the Maasvlakte 2	100	-
$\gamma_{investment}$	Multiplication factor which takes extra construction costs for the sea defence and the breakwater into account	1.3	-
$D_{SLS,trans}$	Expected damage costs per day in case of failure by the SLS failure mode 'Transmission'	1.000.000	fl
D_{ULS}	Expected damage costs per event in case of ULS failure	0.2 *I_{mv2}	fl
B	Benefits of the Maasvlakte 2 in one year	50.000.000	fl

Table 4.9: Other parameters

In order to determine the probability of failure for each failure mode, calculations were executed by a probabilistic calculation method for the values of the decision variables and all other variables of the deterministic design. For a realistic comparison with a probabilistic design (chapter 5), all probability distributions are taken into account.
Table 4.10 shows the probabilities of failure for each failure mode.

ULS failure mode	$P_{f,ULS}$ per year
Erosion of the armour layer of the breakwater	1.52*10^{-2}
Extremely high water level	1.04*10^{-4}
Overtopping of the sea defence	7.9*10^{-4}
Erosion of the outer slope of the sea defence	1.59*10^{-2}
SLS failure mode	**$P_{f,SLS}$ per day**
Transmission of the breakwater	1.51*10^{-4}

Table 4.10: Probabilities of failure per failure mode

The deterministic design is calculated for ULS probability of failure as a measure for the level of safety. This means that the expected damage costs by transmission (SLS) are added to the investment costs of the breakwater. The NPV of the expected damage costs by ULS failure modes and the NPV of the expected benefits are calculated according to equation 3.24 and 3.25 respectively.

Cost type	Value	Unit
Investment costs breakwater	1.02	$*10^9$ fl
Investment costs terrain area	2.94	$*10^9$ fl
Investment costs sea defence	0.58	$*10^9$ fl
NPV expected damage costs	1.22	$*10^9$ fl
NPV expected benefits	-2.09	$*10^9$ fl
NPV total costs	3.67	$*10^9$ fl

Table 4.11: Costs of deterministic design

5. Probabilistic design

5.1 General

According to the system-element approach described in chapter 3, the Maasvlakte 2 is optimised by using a probabilistic calculation method. A Fortran90 program is used to calculate probabilities of failure for different values of decision variables. Calculations are executed according to the *First Order Reliability Method (FORM)* of Hohenbichler/Rackwitz (1983) in which linearization of the reliability function Z in U-space is used to find the design point. In this optimisation method, the variables in Z have to be *independent* and *standard normal distributed*. For this reason, normal and non-normal distributed variables are transformed into standard normal distributed variables. In Appendix G, the probabilistic calculation method is described in more detail.

5.2 Selection of random variables

The same variables that were used in the calculation of the deterministic design are also used in the calculation of the probabilistic design. With regard to the hydraulic variables, the full probability distributions are taken into account instead of the chosen deterministic value in chapter 4. This also holds for random variables for which the mean value was used in chapter 4. Other variables remain the same deterministic value as in chapter 4. The *changed* variables are given in table 5.1.

Basic variable	Distribution type	Shift	Scale	Shape
Yearly hydraulic conditions				
$h_{sea,yr}$	Weibull	2.4	0.19	0.85
$fH_{s,Euro}$	Normal,additive	0	0.6	
$fH_{s,loc}$	Normal,additive	0	0.21	
s_{0p}	Normal	0.038	0.0059	
Daily hydraulic conditions				
$H_{s,day}$	Gumbel	0.74	0.45	
$h_{sea,day}$	Weibull	-1.63	1.83	3.06
Geometry				
Δ_{con}	Normal	1.4	0.1	
Δ_{rock}	Normal	1.65	0.1	
Other				
fh_{ter}	Normal,additive	0	0.1	
modfac	Normal,multiplicative	1	0.106	

Table 5.1: Random variables considered in basic optimisation

5.3 Basic optimisation

With the random variables of table 5.1 and the other deterministic variables, a basic optimisation is executed. In this optimisation, the ULS probability of failure ($P_{f,ULS}$) is used as a measure for the level of safety. This means that the minimal investment costs of each element are calculated as a function of the ULS probability of failure of the element per year. In the optimisation on system level, all cost types are calculated as a function of the ULS probability of failure of the system per year.

In all figures, $-log(P_{f,ULS})$ is used instead of $P_{f,ULS}$ on the horizontal axis. This means that a value of 1 represents $P_{f,ULS} = 10^{-1}$, 2 represents $P_{f,ULS} = 10^{-2}$ etcetera.

Classes are chosen with regard to the value of $-log(P_{f,ULS})$. The width of each class is chosen at 0.4 for the optimisation on element level. This means that the first class contains those combinations of values of decision variables of element X for which holds:

10^0 (=1) $\geq P_{f,ULS,X} \geq 10^{-0.4}$; for the second class holds: $10^{-0.4} \geq P_{f,ULS,X} \geq 10^{-0.8}$ etcetera. For the optimisation on system level, the width of each class is chosen at 0.2.

Table 5.2 shows which values of the decision variables are considered. For the breakwater, $40^2 = 1600$ combinations of values of decision variables can be made. For the terrain area and the sea defence, 160 and $15^3 = 3375$ combinations can be made respectively. In the following sub-paragraphs, the results of the optimisation on element level and system level are shown.

Decision variable	Lowest value	Highest value	Step size	Number of values
D_{bw}	1.25	3.2	0.05	40
$h_{c,bw}$	1	6.85	0.15	40
h_{ter}	2.525	6.5	0.025	160
$D_{n50,sd}$	0.81	1.37	0.04	15
$h_{c,sd}$	8.5	15.5	0.5	15
$cot(\alpha)_{sd}$	3.2	6	0.2	15

Table 5.2: Variations of values of the decision variables

5.3.1 Results of the optimisation of the breakwater

In figure 5.1, the minimal investment costs of the breakwater are given as a function of $-log(P_{f,ULS,bw})$. It shows that for decreasing ULS probability of failure per year, the minimal investment costs increase, because a higher resistance is needed. The NPV of the expected damage costs by transmission are added to the investment costs of the breakwater for each combination of values of the decision variables.
In figure 5.2, the optimal values of the decision variables are given as a function of $-log(P_{f,ULS,bw})$. When this figure shows a more or less fluent pattern, the optimal values of the decision variables do not change very much for a small change in the ULS probability of failure of the breakwater per year. This line is expected to be an upward sloping line, because (most) decision variables have an increasing value for smaller probabilities of failure. However, it is also possible that a decrease of the optimal value of one variable is compensated by an increase of the optimal value of another variable.

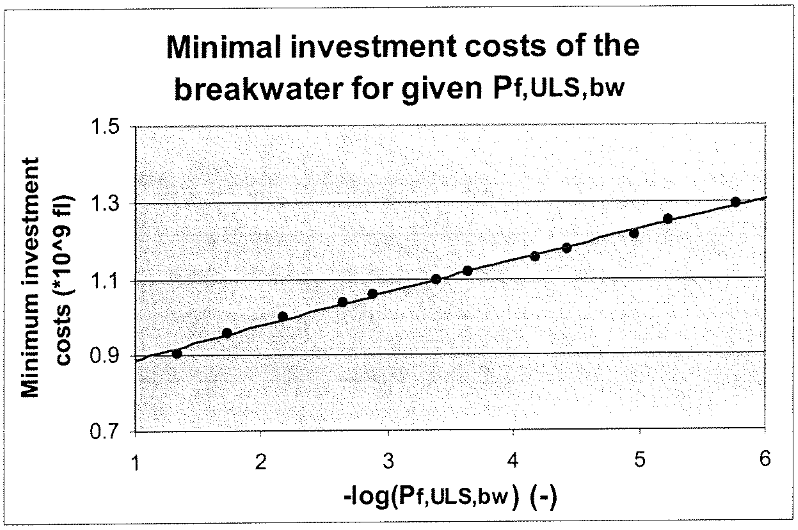


Figure 5.1: Minimal investment costs of the breakwater

The minimal investment costs function can be approximated by:

$$I_{bw,\min}\left(P_{f,ULS,bw}\right) = a - b \cdot \log\left(P_{f,ULS,bw}\right) \tag{5.1}$$

with:

a = 0.79 (*10^9 fl)
b = 0.087 (*10^9 fl)

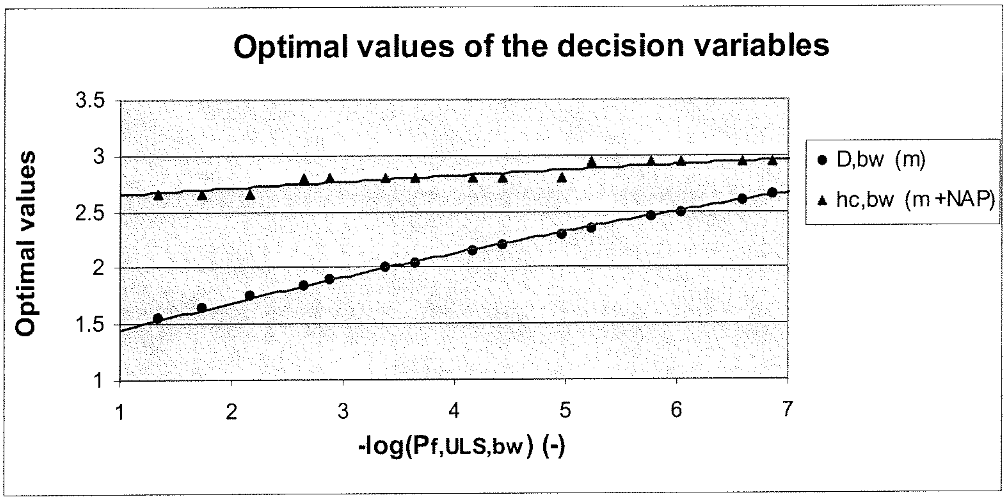


Figure 5.2: Optimal values of the decision variables of the breakwater

Like in figure 3.5a till 3.5c, for erosion of the armour layer, transmission and the combination of both, different values of the ULS probability of failure per year are determined for combinations of the decision variables D_{bw} and $h_{c,bw}$, see figure 5.3 till 5.5.

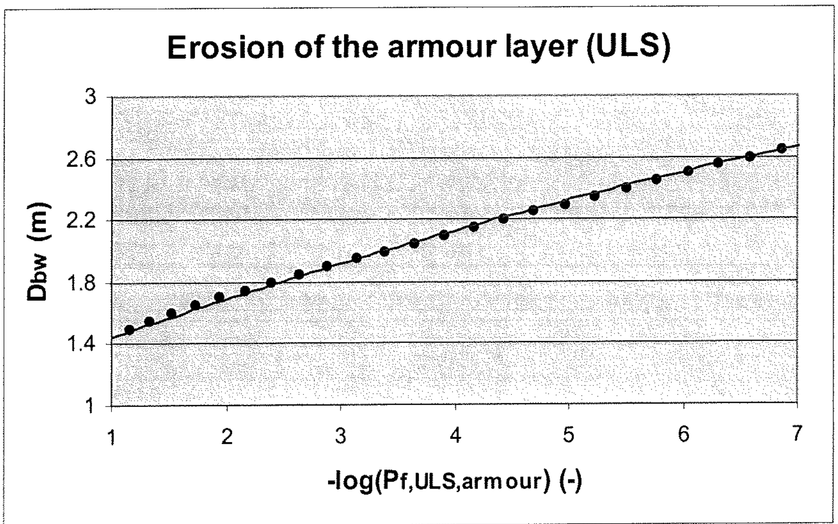


Figure 5.3: Diameter of the concrete blocks in the armour layer for different values of the ULS probability of failure per year by erosion of the armour layer

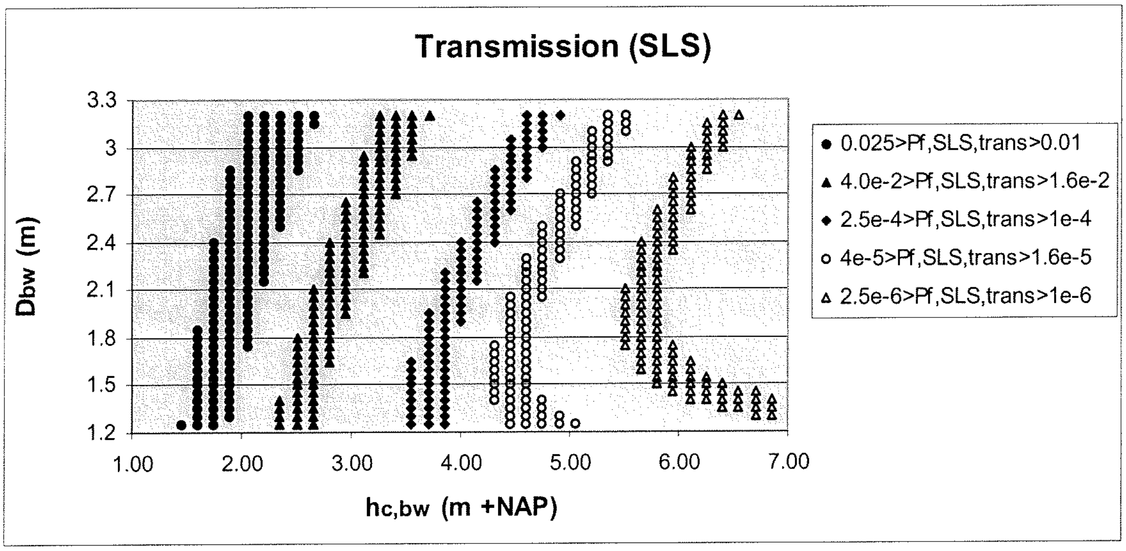


Figure 5.4: Diameter of the concrete blocks in the armour layer and crest height of the breakwater for different values of the SLS probability of failure per day by transmission

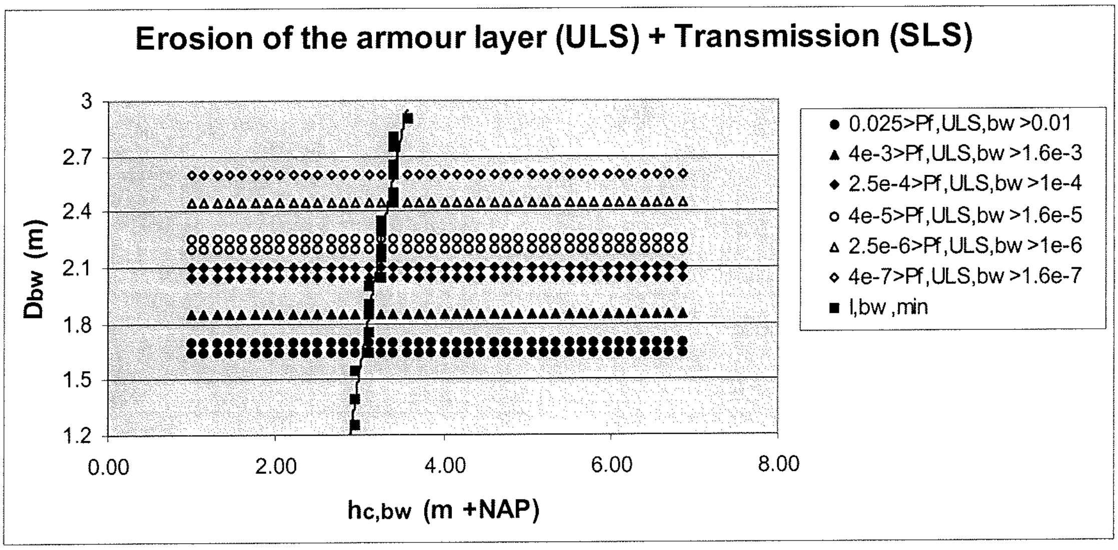


Figure 5.5: Combination of erosion of the armour layer (ULS) and transmission (SLS)

For erosion of the armour layer, the diameter of the concrete blocks increases for decreasing ULS probability of failure per year. With regard to transmission, when the diameter of the concrete blocks increases, in most cases a higher crest height is needed for the same SLS probability of failure per day by transmission ($P_{f,SLS,trans}$). This means that larger concrete blocks are stronger, but also more permeable, so more wave energy is able to pass through the breakwater. The more or less vertical patterns in figure 5.4 show that with regard to the $P_{f,SLS,trans}$, $h_{c,bw}$ has more influence than D_{bw}. When the value of $h_{c,bw}$ changes, the change in $P_{f,SLS,trans}$ is higher than when the value of D_{bw} changes.
Figure 5.5 shows the combination of the two failure modes for different ULS probabilities of failure of the breakwater per year ($P_{f,ULS,bw}$). The horizontal patterns show that $h_{c,bw}$ has no influence at all with regard to $P_{f,ULS,bw}$. This was expected, because $h_{c,bw}$ is only used in the transmission formula (SLS).

5.3.2 Results of the optimisation of the terrain area

For the terrain area, the same graphs are given in figure 5.6 and 5.7. Because only one decision variable is considered and only one failure mode is taken into account, there is no difference between 5.7 and a graph for the ULS failure mode: 'extremely high water level' except for the notation of the variable on the horizontal axis. For 'extremely high water level' this would be $P_{f,ULS,highwater}$. However, in this case holds that $P_{f,ULS,ter} = P_{f,ULS,highwater}$, see also figure 3.6a and b.

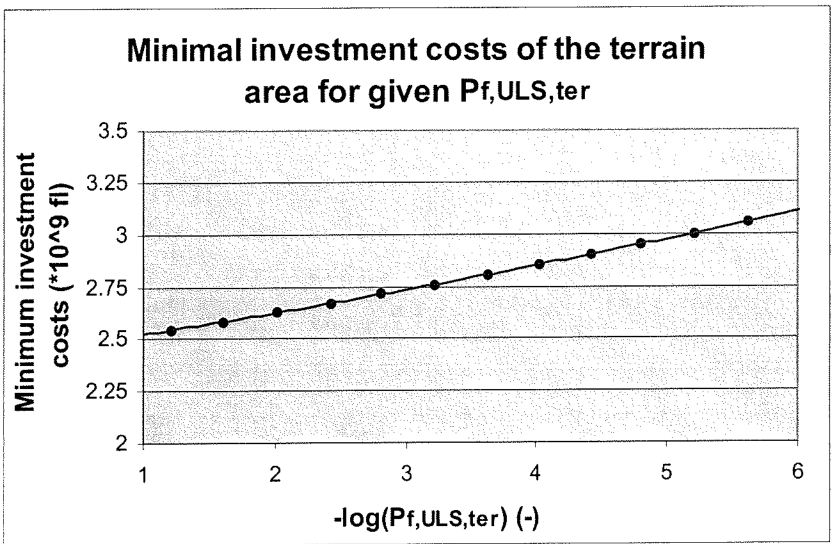


Figure 5.6: Minimal investment costs of the terrain area

The minimal investment costs function can be approximated by:

$$I_{ter,\min}\left(P_{f,ULS,ter}\right) = a - b \cdot \log\left(P_{f,ULS,ter}\right) \tag{5.2}$$

with:

a = 2.41 (*10^9 fl)
b = 0.11 (*10^9 fl)

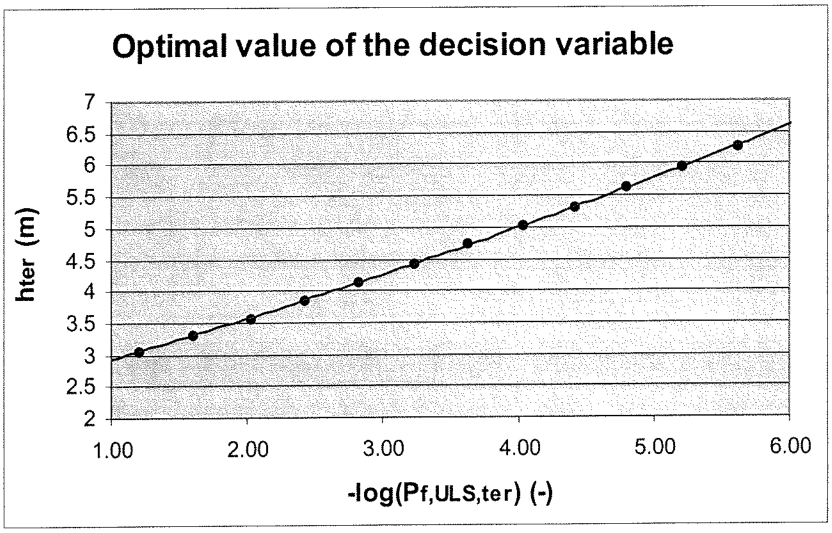


Figure 5.7: Optimal value of the decision variable of the terrain area

5.3.3 Results of the optimisation of the sea defence

The results of the optimisation of the sea defence are given in figure 5.8 and 5.9.

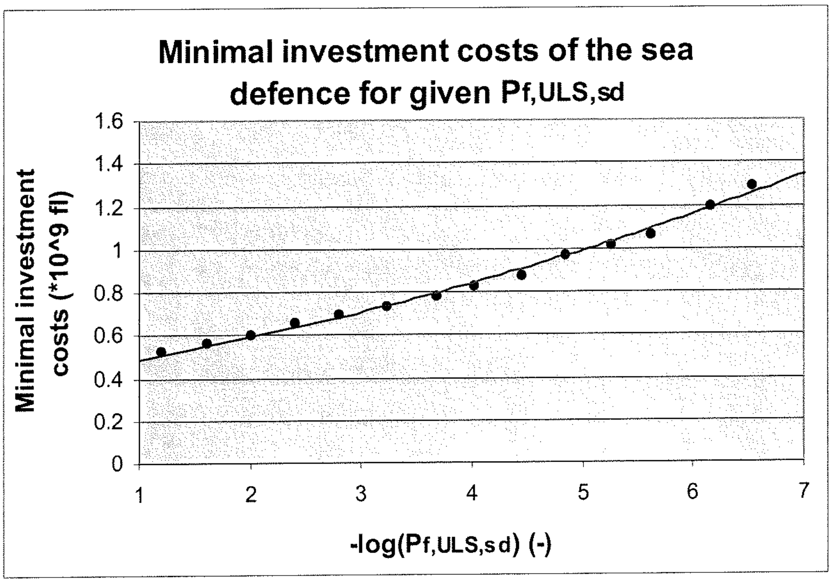


Figure 5.8: Minimal investment costs of the sea defence

The minimal investment costs function can be approximated by:

$$I_{sd,\min}\left(P_{f,ULS,sd}\right) = a - b \cdot \log\left(P_{f,ULS,sd}\right) \qquad (5.3)$$

with:

a = 0.33 (*10^9 fl)
b = 0.14 (*10^9 fl)

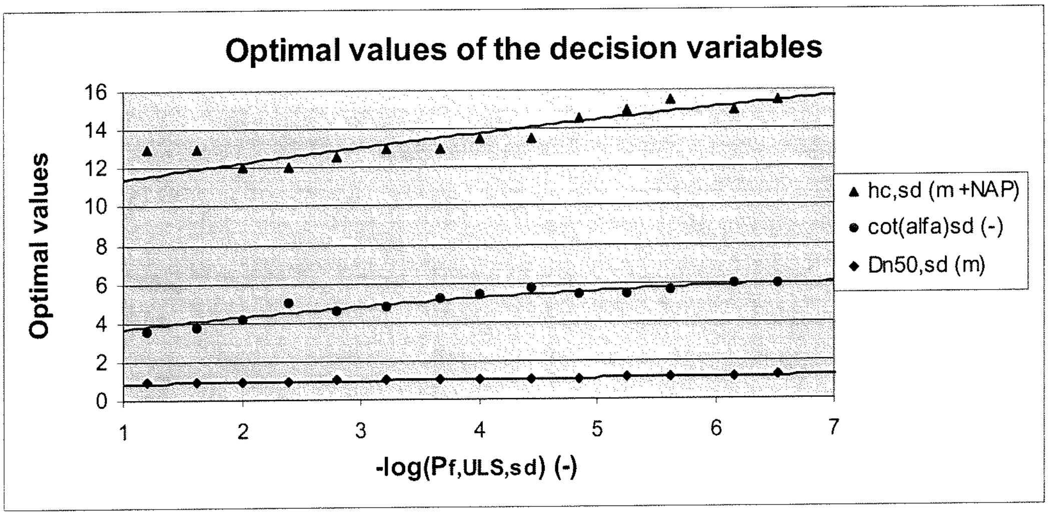


Figure 5.9: Optimal values of the decision variables of the sea defence

Like in figure 3.7a and b, for erosion of the outer slope and overtopping, different values of the ULS probability of failure per year are shown for combinations of the decision variables. The flat patterns in figure 5.10 and 5.11 show relatively large influences of $h_{c,sd}$ and $D_{n50,sd}$. With regard to the ULS probability of failure per year for both failure modes, $cot(\alpha)_{sd}$ can be considered as a less important decision variable.

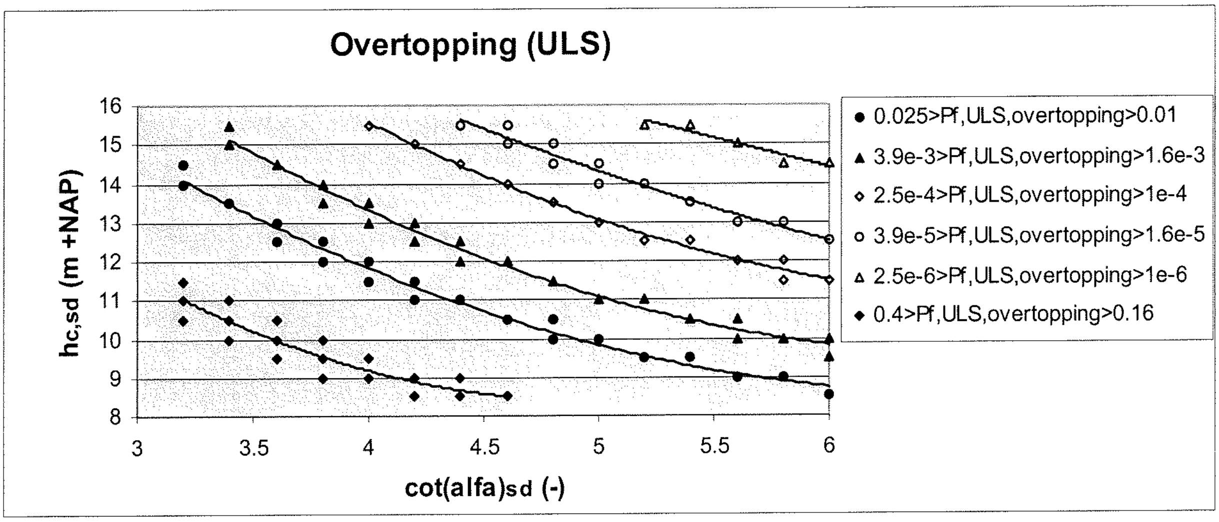


Figure 5.10: Crest height and angle of the outer slope for different values of the ULS probability of failure per year by overtopping

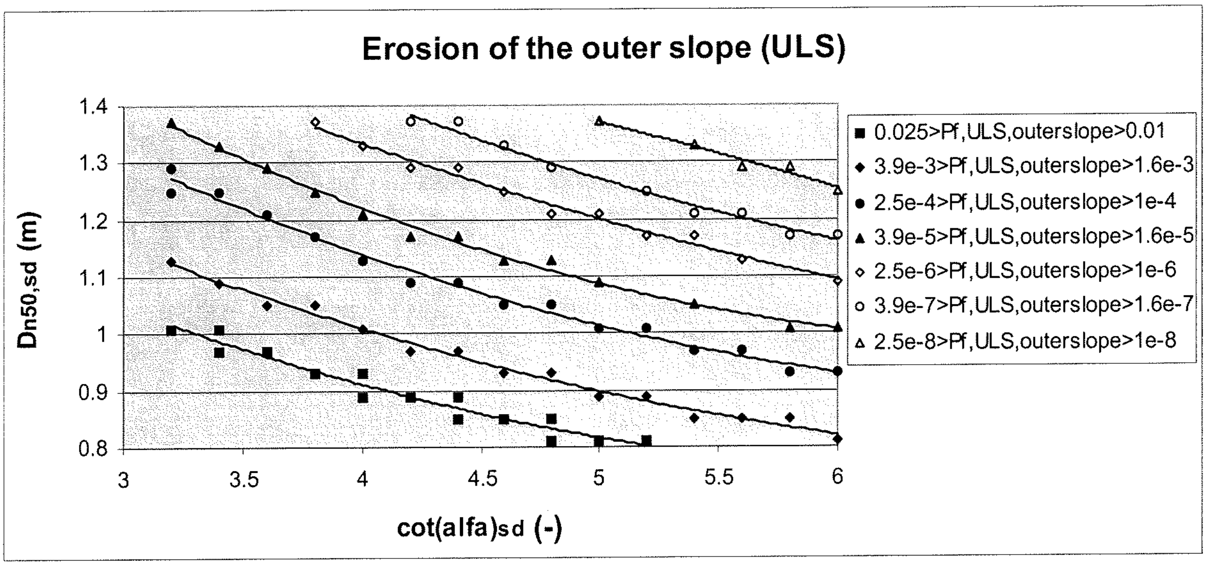


Figure 5.11: Diameter of the quarry stones and angle of the outer slope for different values of the ULS probability of failure per year by erosion of the outer slope

5.3.4 Results of the optimisation on system level

In the optimisation on system level, the results of the element optimisations are combined and the optimal combination of the decision variables $P_{f,ULS,bw}$, $P_{f,ULS,ter}$ and $P_{f,ULS,sd}$ is determined by the *optimal value of the minimum NPV of the total costs* ($TC_{mv2,min,opt}$), see figure 3.11c. Figure 5.12 shows the results of the optimisation on system level. In this figure, the NPV of the expected benefits of the Maasvlakte 2 (B_{mv2}) has a positive value. However, for each calculation of TC_{mv2} (equation 3.19), the benefits are considered as negative costs.

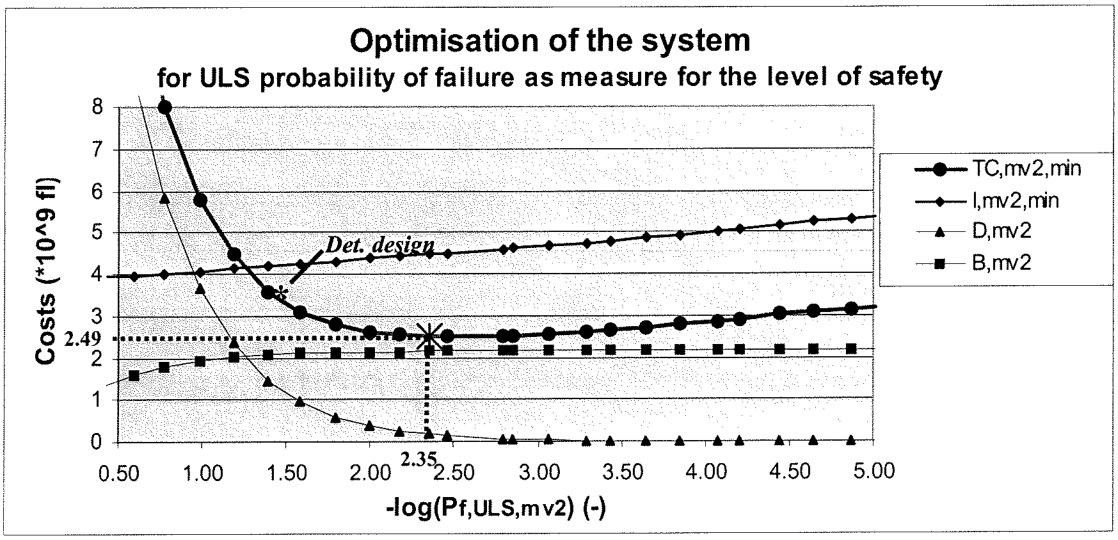


Figure 5.12: Optimisation of the system

Figure 5.12 shows an optimal value for the minimal NPV of the total costs of $2.49*10^9$ fl with an optimal ULS probability of failure of the Maasvlakte 2 per year of $10^{-2.35} \approx 4.43*10^{-3}$. This means that total inundation of the terrain area is expected once in 225 years. This means a large cost advantage with regard to the deterministic design ($TC_{mv2} = 3.67*10^9$ fl), which is almost 50% more expensive. Besides that, the probabilistic design contains a higher level of safety than the deterministic design, in which $-log(P_{f,ULS,mv2}) = 1.50$.
In figure 5.12, the deterministic design is shown somewhat above the $TC_{mv2,min}$-line, because the deterministic design was not optimised with regard to its investment costs. This is slightly inefficient, such that the deterministic design is not exactly situated on the $TC_{mv2,min}$-line. The probabilistic design is compared with the deterministic design in table 5.3.

Probabilistic design			**Deterministic design**	
Decision variable				
D_{bw}	1.90	m	1.67	m
$h_{c,bw}$	2.8	m (+NAP)	3.74	m (+NAP)
h_{ter}	4.13	m (+NAP)	4.99	m (+NAP)
$h_{c,sd}$	12.5	m (+NAP)	12.71	m (+NAP)
$cot(\alpha)_{sd}$	4.6	-	4.6	-
$D_{n50,sd}$	1.01	m	0.85	m
Investment costs per element				
$I_{bw,min}$	1.06	$*10^9$ fl	1.02	$*10^9$ fl
$I_{ter,min}$	2.71	$*10^9$ fl	2.94	$*10^9$ fl
$I_{sd,min}$	0.69	$*10^9$ fl	0.58	$*10^9$ fl
SLS/ULS probability of failure per element				
$P_{f,SLS,bw}$	$2.22*10^{-3}$	-	$1.51*10^{-4}$	-
$P_{f,ULS,bw}$	$1,31*10^{-3}$	-	$1.52*10^{-2}$	-
$P_{f,ULS,sd}$	$1.57*10^{-3}$	-	$1.60*10^{-2}$	-
$P_{f,ULS,ter}$	$1,54*10^{-3}$	-	$1.04*10^{-4}$	-
Cost type on system level				
$I_{mv2,min}$	4.46	$*10^9$ fl	4.54	$*10^9$ fl
D_{mv2}	0.17	$*10^9$ fl	1.22	$*10^9$ fl
B_{mv2}	-2.15	$*10^9$ fl	-2.09	$*10^9$ fl
$\boldsymbol{TC_{mv2,min}}$	**2.49**	$\boldsymbol{*10^9}$ **fl**	**3.67**	$\boldsymbol{*10^9}$ **fl**
$P_{f,ULS,mv2}$	$4.43*10^{-3}$	(-)	$3.12*10^{-2}$	(-)

Table 5.3: Comparison of probabilistic design and deterministic design

Comments:

Breakwater

- The probabilistic design includes a larger diameter of the concrete blocks in the armour layer (D_{bw}) than the deterministic design. This leads to a strong reduction of $P_{f,ULS,bw}$, so a strong reduction of the NPV of the expected damage costs for the Maasvlakte 2 (D_{mv2}). As a result of the higher value for D_{bw}, investment costs increase.
- The increase of $I_{bw,min}$ is small because of the lower crest height ($h_{c,bw}$) for the probabilistic design. Together with the higher value of D_{bw}, a slightly higher value of $I_{bw,min}$ is the result for the probabilistic design.

Terrain area

- A lower value for the height of the terrain area (h_{ter}) leads to a higher value for $P_{f,ULS,ter}$ and a lower value for the minimal investment costs of the terrain area ($I_{ter,min}$) in case of the probabilistic design.

Sea defence

- For the sea defence, the higher value for the diameter of the quarry stones ($D_{n50,sd}$) leads to a strong decrease of the ULS probability of failure per year by erosion of the outer slope for the probabilistic design. Erosion of the outer slope has by far the largest contribution to $P_{f,ULS,sd}$ ($2.59*10^{-2}$ of $2.60*10^{-2}$) for the deterministic design. The reduction of the ULS probability of failure per year by erosion of the outer slope leads to a lower value of $P_{f,ULS,sd}$. As a result of that, a lower value for the NPV of the expected damage costs for the Maasvlakte 2 (D_{mv2}) and a higher value for the expected benefits for the Maasvlakte 2 (B_{mv2}) are found for the probabilistic design.
- The more or less equal value for the crest height ($h_{c,sd}$) does not change the increased minimal investment costs of the sea defence ($I_{sd,min}$) as a result of the higher value for $D_{n50,sd}$ in case of the probabilistic design.

Total Maasvlakte 2

- For the probabilistic design, the extra investment costs of the sea defence and the breakwater are overcompensated by the lower investment costs of the terrain area. This leads to a lower value for the minimal investment costs of the Maasvlakte 2 ($I_{mv2,min}$).
- The ULS probability of failure per year ($P_{f,ULS,mv2}$) is lower despite the lower investment costs of the Maasvlakte 2. This leads to a lower value for D_{mv2} and a higher value for B_{mv2}. Altogether, $TC_{mv2,min}$ is much lower for the probabilistic design. So an increase in the level of safety without higher investment costs (!) and a strong decrease in the total costs of the Maasvlakte 2, is the result of the probabilistic design. This means that the cost advantage is the result of 'better investments'. Because the lower value of $P_{f,ULS,mv2}$ is mostly caused by a stronger breakwater and sea defence, the breakwater and the sea defence are considered as *cost-efficient protections* of the Maasvlakte 2.

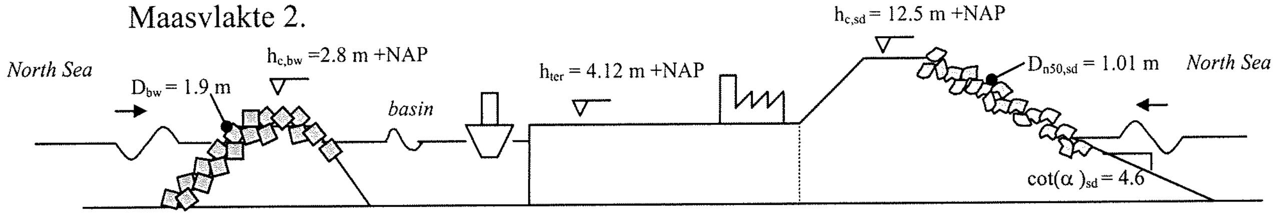


Figure 5.13: Cross-section Maasvlakte 2 after the basic optimisation

5.4 Basic optimisation for risk as a measure for the level of safety

In this paragraph, *risk* is used as a measure for the level of safety. According to paragraph 3.3.1.1, differences exist between ULS probability of failure and risk as a measure for the level of safety. Applied to the Maasvlakte 2, this means that:

For risk as a measure for the level of safety:

A. The expected damage costs per year by the (only) SLS failure mode 'transmission' are included in the risk of the breakwater per year (R_{bw}) instead of adding the NPV of the expected damage costs to the investment costs of the breakwater (I_{bw}). Now, the measure of the level of safety of the breakwater is not only determined by the value of the diameter of the concrete blocks, D_{bw}. Combinations of crest height ($h_{c,bw}$) and D_{bw} with a high value for $h_{c,bw}$ (and also for I_{bw}) 'move' to a relatively higher level of safety as a result of the high value for risk by transmission. The opposite holds for combinations in which $h_{c,bw}$ has a low value. As a result of this, the optimal value of $h_{c,bw}$ (and I_{bw}) will increase faster for decreasing risk per year than for decreasing ULS probability of failure per year ($P_{f,ULS,bw}$). In case of $P_{f,ULS,bw}$ as a measure for the level of safety, the increase of the optimal value of $h_{c,bw}$ is only determined by the optimal ratio of investment costs and SLS damage costs for different values of $P_{f,ULS,bw}$ (or: D_{bw}).

B. Values of $P_{f,SLS,bw}$ and $P_{f,ULS,bw}$ are not optimised with regard to each other anymore. This may lead to a slightly less optimal design.

C. The result of the assumption: $D_{ULS} = 0.2*I_{mv2}$ (table 4.9), is that for combinations of decision variables with the same ULS probability of failure per year in the basic optimisation, now the more expensive designs (with higher value for I_{mv2}) lead to relatively higher risks per year and the cheaper designs lead to relatively lower risks per year. This also leads to higher optimal values of $h_{c,bw}$ for decreasing R_{bw}.

D. With regard to the assumption: $D_{ULS} = 0.2*I_{mv2}$, it is not possible to calculate the amount of risk per year on element level (equation 3.18) already, because the value of the investment costs of the *system* cannot be determined. As an alternative to determine the amount of risk per year on element level, for each combination of the decision variables of the considered element, the investment costs of the system are assumed to be the sum of:
   1. The investment costs of the considered element, for the considered combination of the values of decision variables of the element
   2. The values of the investment costs of the other two elements, taken from the deterministic design, table 4.11.

Significant changes with respect to the optimisation with ULS probability of failure as a measure for the level of safety only take place in the optimisation of the breakwater. These results are given in figure 5.14 till 5.17. On the horizontal axis, the risk of the breakwater in one year (R_{bw}) is measured by the unit 10^9 fl, so for $-log(R_{bw}) = 1$ holds that $R_{bw}=10^{-1}*10^9 = 10^8$ fl; for $-log(R_{bw}) = 2$, $R_{bw} = 10^7$ fl etcetera.
The results of the optimisations of the terrain area and the sea defence are given in Appendix I.

Compared to figure 5.2 of the basic optimisation, figure 5.15 shows that the optimal value of $h_{c,bw}$ increases stronger for decreasing R_{bw} (point A and C). According to point B this is a 'less optimal' situation, so the real optimal investment costs function of the breakwater will lie somewhere between figure 5.1 and 5.14.

Figure 5.16 and 5.17 show the minimal investment costs of the breakwater as a function of the optimal value of D_{bw} and $h_{c,bw}$ respectively. This is less abstract than the optimal values of the decision variables as a function of the risk of the breakwater per year in figure 5.15.

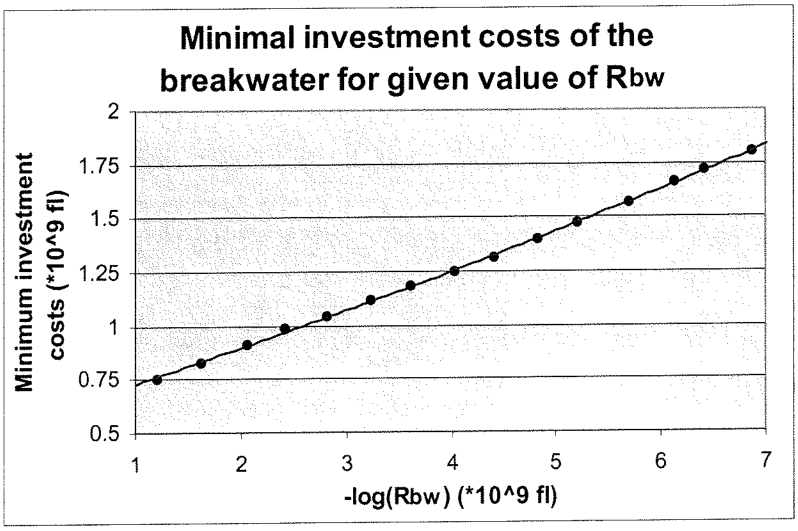


Figure 5.14: Minimal investment costs of the breakwater

The minimal investment costs function can be approximated by:

$$I_{bw,\min}\left(R_{bw}\right) = a - b \cdot \log\left(R_{bw}\right) \tag{5.3}$$

with:

a = 0.53 (*10^9 fl)
b = 0.19 (-)

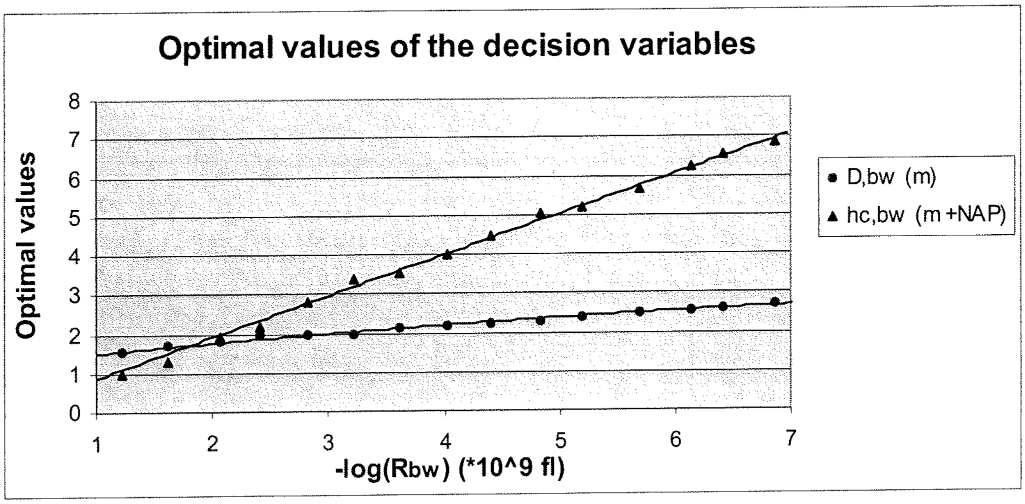


Figure 5.15: Optimal values of the decision variables of the breakwater

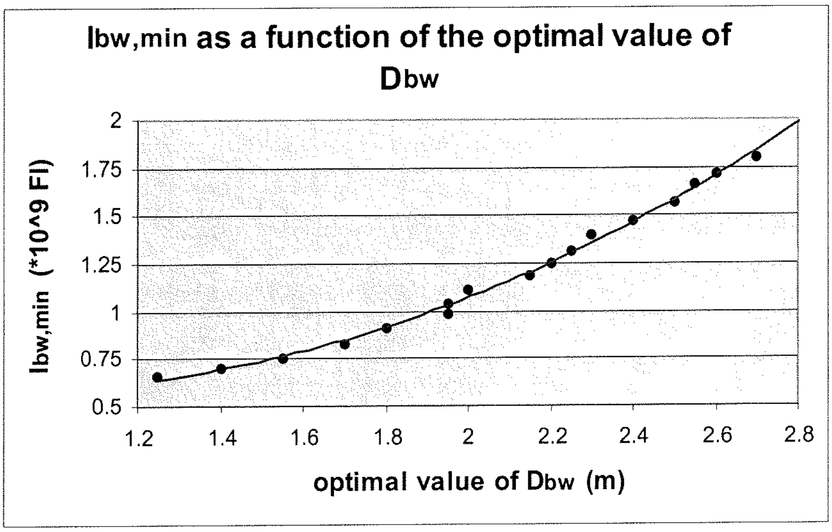


Figure 5.16: Minimal investment costs as a function of the optimal value of the diameter of the concrete blocks for the breakwater

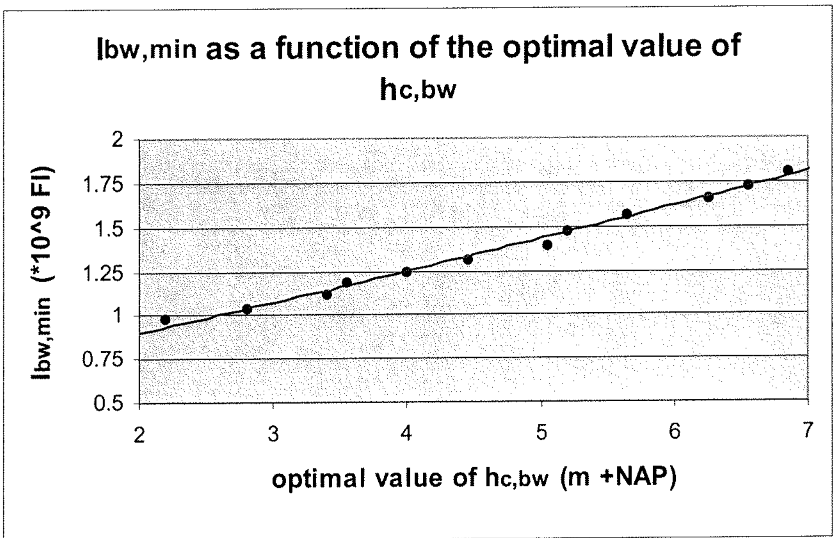


Figure 5.17: Minimal investment costs as a function of the optimal value of the crest height of the breakwater

Figure 5.18 shows the results of the optimisation on system level.
Table 5.4 compares the results of the basic optimisation with risk as a measure for the level of safety with the results of the basic optimisation of paragraph 5.3 (with ULS probability of failure as measure for the level of safety). For the optimal design with risk as a measure for the level of safety, the optimal values of all (SLS and ULS) probabilities of failure per day/year are also known and used in the table to create a realistic comparison between the two methods.

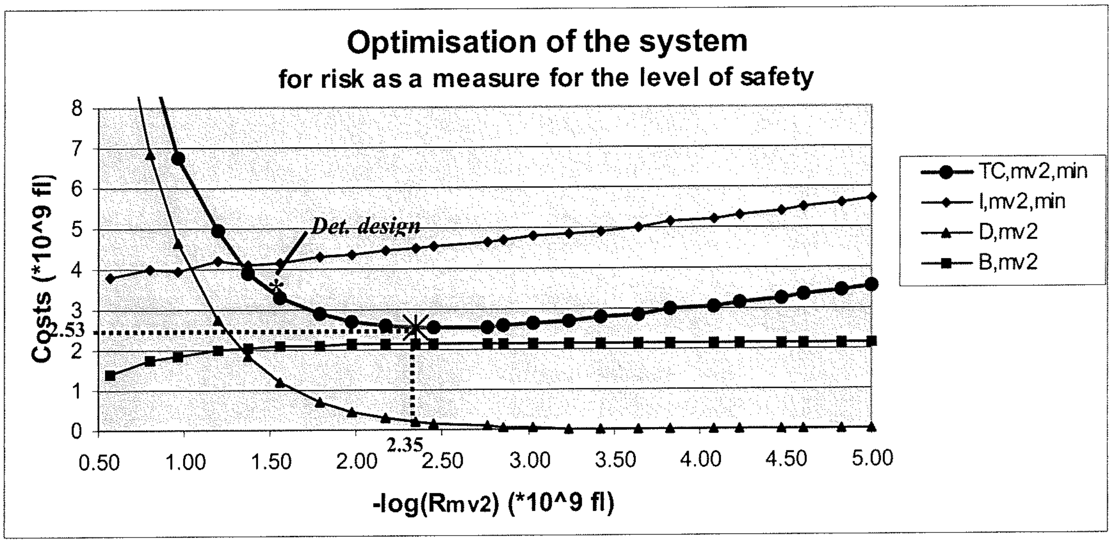


Figure 5.18: Optimisation of the system

Probabilistic design				
ULS probability of failure			***Risk***	
Decision variable				
D_{bw}	1.90	m	1.95	m
$h_{c,bw}$	2.8	m (+NAP)	2.8	m (+NAP)
h_{ter}	4.13	m (+NAP)	4.38	m (+NAP)
$h_{c,sd}$	12.5	m (+NAP)	13.5	m (+NAP)
$cot(\alpha)_{sd}$	4.6	-	4.6	-
$D_{n50,sd}$	1.01	m	0.97	m
Minimal investment costs per element				
$I_{bw,min}$	1.06	$*10^9$ fl	1.04	$*10^9$ fl
$I_{ter,min}$	2.71	$*10^9$ fl	2.75	$*10^9$ fl
$I_{sd,min}$	0.69	$*10^9$ fl	0.69	$*10^9$ fl
SLS/ULS probability of failure per element				
$P_{f,SLS,bw}$	$2.22*10^{-3}$	-	$2.36*10^{-3}$	-
$P_{f,ULS,bw}$	$1,31*10^{-3}$	-	$7.4*10^{-4}$	-
$P_{f,ULS,sd}$	$1.57*10^{-3}$	-	$1.55*10^{-3}$	-
$P_{f,ULS,ter}$	$1,54*10^{-3}$	-	$6.96*10^{-4}$	-
Cost type on system level				
$I_{mv2,min}$	4.46	$*10^9$ fl	4.48	$*10^9$ fl
D_{mv2}	0.17	$*10^9$ fl	0.19	$*10^9$ fl
B_{mv2}	-2.15	$*10^9$ fl	-2.15	$*10^9$ fl
$TC_{mv2,min}$	2.49	$*10^9$ fl	2.53	$*10^9$ fl
$P_{f,ULS,mv2}$	$4.43*10^{-3}$	(-)	$2.98*10^{-3}$	(-)

Table 5.4: Overview of optimal values of the basic optimisation and the basic optimisation with risk as a measure for the level of safety.

Comments:

<u>Breakwater</u>

- For the risk-column, the higher value of D_{bw} leads to a lower value for the ULS probability of failure per year (by armour layer erosion) and to a slightly higher value for the SLS probability of failure per day (by transmission) because permeability increases and the optimal values of $h_{c,bw}$ are equal.

- Despite the higher value for D_{bw}, the minimal investment costs of the breakwater ($I_{bw,min}$) are lower for the risk-column. This is the result of the NPV of the expected SLS damage costs included in the value of $I_{bw,min}$ in case of ULS probability of failure as a measure for the level of safety. This value is $0.035*10^9$ fl, so when these costs are subtracted for a realistic comparison, $I_{bw,min}$ becomes $1.025*10^9$ fl for the ULS probability of failure-column and $I_{bw,min}$ is higher for the risk-column.

Terrain area

- The higher value for h_{ter} leads to a higher value for $I_{ter,min}$ and a lower value for the $P_{f,ULS,ter}$ for the risk-column.

Sea defence

- The lower value for $D_{n50,sd}$ leads to an increase (from $5.15*10^{-4}$ to $1.28*10^{-3}$) of the ULS probability of failure per year by erosion of the outer slope for the risk-column.
- On the other hand, the ULS probability of failure per year by overtopping decreases (from $1.06*10^{-3}$ to $2.65*10^{-4}$) by the higher crest height $h_{c,sd}$ for the risk-column. This overcompensates the first effect, so the value of $P_{f,ULS,sd}$ is lower for the risk-column, but the minimal investment costs are not higher. This is not expected and may be a result of the inaccuracy of the direct search optimisation method.

Total Maasvlakte 2

- On system level, $I_{mv2,min}$ is higher for the risk-column, especially when the NPV of the expected SLS damage costs are subtracted and $I_{bw,min} = 1.025*10^9$ fl for the ULS probability of failure-column. The higher value for $I_{mv2,min}$ is combined with a lower value for the ULS probability of failure of the Maasvlakte 2 per year ($P_{f,ULS,mv2}$).
- For a realistic comparison, the (subtracted) NPV of the expected SLS damage costs have to be added to the NPV of the expected damage costs of the Maasvlakte 2 (D_{mv2}). This leads to a higher value of D_{mv2} ($0.17 + 0.035 = 0.205*10^9$ fl) for the ULS probability of failure-column. This was expected because of the higher value of $P_{f,ULS,mv2}$ and the more or less equal values for $P_{f,SLS,bw}$.
- The difference in the value of $P_{f,ULS,mv2}$ is too small to make significant differences with regard to the value of the expected benefits of the Maasvlakte 2 (B_{mv2}).
- A lower value for $P_{f,ULS,mv2}$ is combined with a higher value for the minimal NPV of the total costs of the Maasvlakte 2 ($TC_{mv2,min}$) for the risk-column. It is difficult to determine if this optimal design is 'too safe' as a result of not optimising $P_{f,ULS,bw}$ and $P_{f,SLS,bw}$ with regard to each other. More differences between the two optimisations were already mentioned and apart from that, the inaccuracy of the optimisation method also plays a role. It is concluded that by the considered basic optimisations, only small differences were found in the results, so both optimisations with risk and ULS probability of failure as a measure for the level of safety can be considered as useful methods to determine an economic optimal design of the Maasvlakte 2.

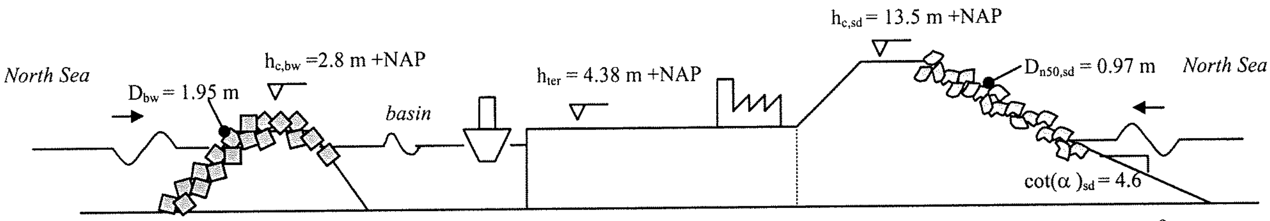


Figure 5.19: Cross-section Maasvlakte 2 after basic optimisation with risk as a measure for the level of safety

6. Sensitivity analysis

6.1 Method

In this chapter, a sensitivity analysis is executed with regard to the results of the basic optimisation of paragraph 5.3. This includes that the influence of changes in the values of variables is analysed with regard to:

- the optimal value of the minimal NPV of the total costs, $TC_{mv2,min,opt}$
- the optimal value of the ULS probability of failure of the Maasvlakte 2 per year, $P_{f,ULS,mv2,opt}$

From this analysis, the variables are ranked with regard to their value of the sensitivity. The higher the value of the sensitivity, the more the values of $TC_{mv2,min,opt}$ and $P_{f,ULS,mv2,opt}$ are influenced by possible changes in the value of the variable. Because of this large influence, it is important that the values of the variables with high sensitivities are well estimated.
Of course, for each optimal design, a new sensitivity analysis can be executed and after each analysis different values will be found. It is not the intention of this analysis to pay much attention to the exact values of the sensitivities, but to get an indication of the variables which are important with regard to the optimal value for the total costs and the level of safety of the Maasvlakte 2.

In this thesis, the sensitivity of a variable is measured by its 'absolute elasticity', which is calculated by the relative change of $TC_{mv2,min,opt}$ (or: $P_{f,ULS,mv2,opt}$) divided by the relative change of the value of the considered variable. From this value, the absolute value is taken. For several variables, more than one variation is used. The variation which leads to the largest absolute elasticity is chosen.

Absolute elasticity in formula:

$$e = \left| \frac{\frac{dp}{p}}{\frac{dx}{x}} \right| = \left| \frac{dp}{dx} \cdot \frac{x}{p} \right| \qquad (6.1)$$

In which:

e	= absolute elasticity
dx/x	= relative variation of variable x
dp/p	= relative change of variable p by variation of variable x

An overview of which variations are used, can be found in Appendix J.

6.2 Results

The variables with the highest values for the absolute elasticity are selected in table 6.1 and 6.2. The 'total value' refers to the sum of all calculated values of the absolute elasticity. Because the value of the absolute elasticity depends on the values of $TC_{mv2,min,opt}$ and $P_{f,ULS,mv2,opt}$, which are very different, a comparison of values between the two tables is not allowed. Values are only compared within one table. For the value of $TC_{mv2,min,opt}$, it is shown that the largest part of the total value is caused by the distribution of the maximum sea water level in one year ($h_{sea,yr}$), the interest in one year (r) and the relative density of concrete (Δ_{con}). For $P_{f,ULS,mv2,opt}$, which is not influenced by r, Δ_{con} has the second highest value after $h_{sea,yr}$.

In figure 6.1 and 6.2, the values for the absolute elasticity are given as a percentage of the total value with regard to $TC_{mv2,min,opt}$ and $P_{f,ULS,mv2,opt}$ respectively.

Variable	Description	Old value	New value	Absolute elasticity
$h_{sea,yr}$	Maximum sea water level in one year, parameter *w*	2.59	2.65	7.92
r	Interest in one year	0.06	0.05	2.16
Δ_{con}	Relative density of concrete	1.4	N(1.3,0.1)	1.31
i	Inflation in one year	0.02	0.04	1.03
B	Benefits of the Maasvlakte 2 in one year	0.05	0.03	0.86
g	Economic growth in one year	0.02	0.03	0.72
l_{quay}	Quay length	25000	22500	0.61
$\gamma_{investment}$	Multiplication factor which takes extra construction costs for the sea defence and the breakwater into account	1.3	1.43	0.60
UP_{sand}	Unit price of sand	5	6	0.54
Δ_{rock}	Relative density of quarry stone	N(1.65,0.1)	N(1.55,0.1)	0.51
N	Considered number of years for the Maasvlakte 2	100	50	0.42
Total value				22.5

Table 6.1: Highest values for the absolute elasticity with regard to '$TC_{mv2,min,opt}$'

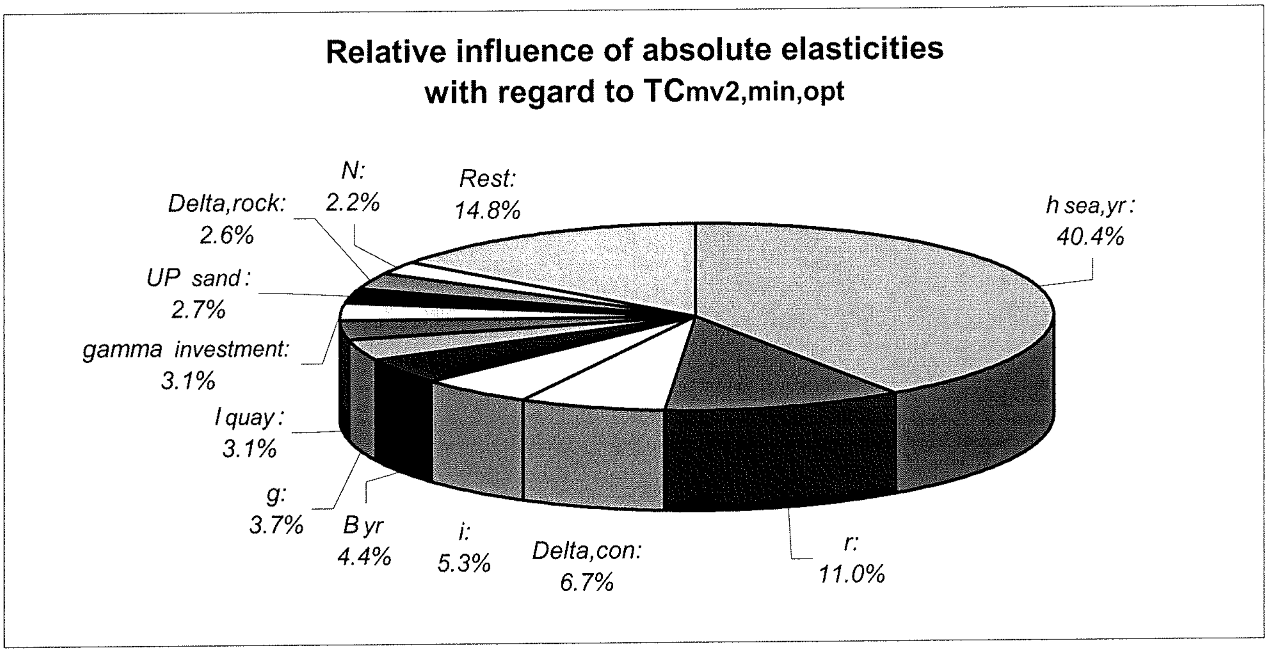


Figure 6.1: Relative influence of the absolute elasticities with regard to '$TC_{mv2,min,opt}$'

Variable	Description	Old value	New value	Absolute elasticity
$h_{sea,yr}$	Maximum sea water level in one year, parameter w	w=2.59	w = 2.65	104.00
Δ_{con}	Relative density of concrete	1.4	N(1.3,0.1)	17.20
Δ_{rock}	Relative density of quarry stones	N(1.65,0.1)	N(1.55,0.1)	6.64
$fH_{s,Euro}$	Conditional distribution of the significant wave height at Euro-0 for given water levels at Hook of Holland, parameter σ	N(0,0.6)	N(0,0.8)	5.41
N_{cr}	Critical damage for exposed core of the breakwater	2	1.5	2.11
s_{0p}	Wave steepness based on the peak period	N(0.038,0.0059)	N(0,038,0.009)	1.86
T_p/T_m	Peak period/average period	1.2	1.3	1.44
P	Porosity of the protection layer of the outer slope of the sea defence	0.3	0.2	1.38
N	Number of waves in a storm	3000	4000	1.38
Total value				145.14

Table 6.2: Highest values for the absolute elasticity with regard to '$P_{f,ULS,mv2,opt}$'

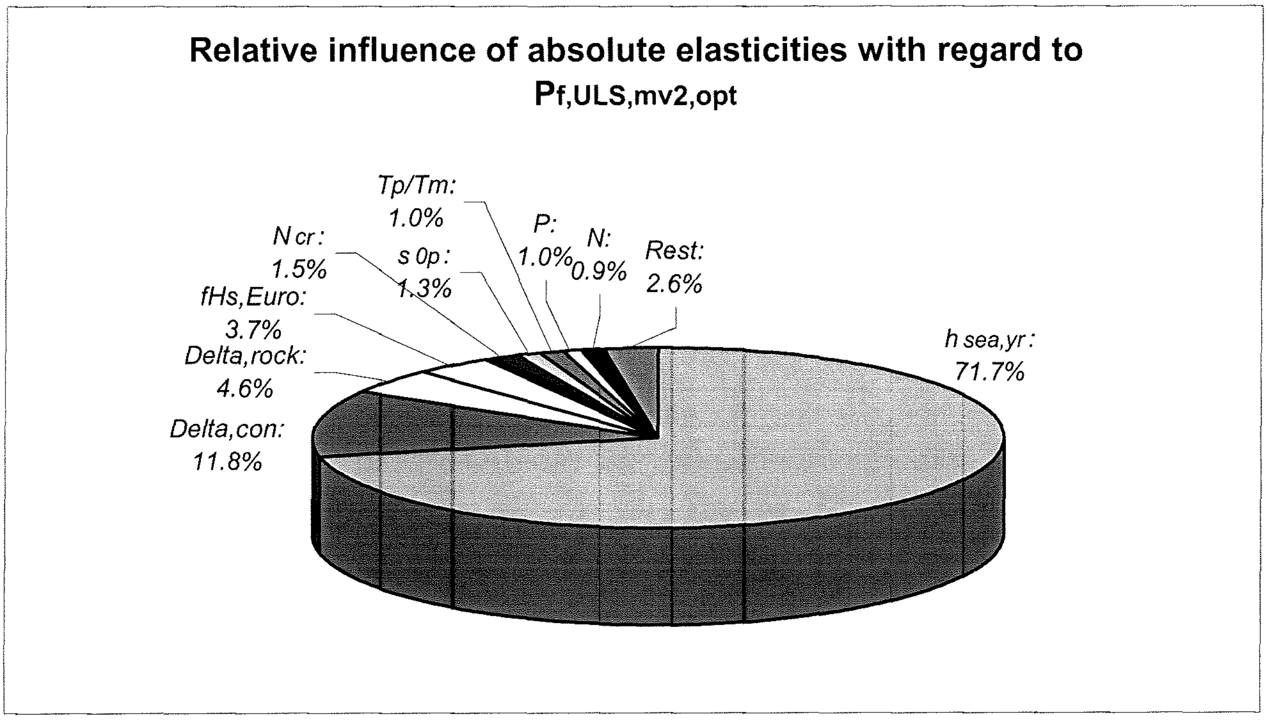


Figure 6.2: Relative influence of the absolute elasticities with regard to '$P_{f,ULS,mv2,opt}$'

7. Evaluation of improvements of the optimisation

With regard to the basic optimisations in chapter 5, many improvements are possible. This will not be done in this thesis. Some improvements are described:

- The extra SLS failure mode: 'Partial inundation of the terrain area', which is described in paragraph 3.2.2.2. To use this failure mode, the wave conditions in the basin ($K_{r,xy}K_tH_{si}$) have to be known. This is not at once possible in the optimisation of the terrain area, because the wave conditions depend on the values of the decision variables of the breakwater, which are not known yet.
  To handle this problem, the system (Maasvlakte 2) is first optimised *without* 'partial inundation of the terrain area', like in chapter 5. Optimal values of the decision variables of the breakwater are the result. These values are used to determine the wave conditions behind the breakwater.
  Then, the optimisation of the terrain area (on element level) *including* 'partial inundation of the terrain area', and the optimisation on system level is executed again. (the results of the optimisations of the breakwater and the sea defence do not change, so they do not have to be optimised again). New optimal values of the decision variables of the breakwater (and the other elements) are the result. The new optimal values of the decision variables of the breakwater are used to determine the new wave conditions behind the breakwater.
  Again the optimisation of the terrain area (on element level) including 'partial inundation of the terrain area' and the optimisation on system level are executed. This leads to a new optimal design and so on. This process ends when the optimal design of the breakwater (and the other elements) is stable.

- Correlation coefficients with regard to ULS probabilities of failure for different failure modes can be used. The summing of ULS probabilities of failure implies independent ULS failure modes. In fact, especially in case of extreme hydraulic conditions, the sollicitations (water level, significant wave height) are highly correlated for the ULS failure modes. It is not likely that these conditions will for instance only affect the breakwater and not the sea defence. The same holds for daily hydraulic conditions with low probabilities of failure.
  When correlation coefficients are taken into account, it is not possible to optimise the system for risk as a measure for the level of safety.

- With regard to the wave height in the entrance channel and the basin, refraction and diffraction can be taken into account. Research has been done by the Alkyon studies (1995), which have determined values for K_r, the multiplication factor for the significant wave height as a result of refraction and diffraction, for different locations in the entrance channel and the basin of the Maasvlakte 2. This leads to higher significant wave heights behind the breakwater and in the basin and as a result of that, higher probabilities of failure with regard to damage to shipping.
  In the basin and the entrance channel, wave reflection also plays a role. To measure the influence of reflection, tests with a scaled model of the Maasvlakte 2 could be used. Apart from that, local wind can be taken into account, which leads to higher significant wave heights in the entrance channel and the basin.
  When the influences of these phenomena are not taken into account, an underestimation of the significant wave height in the entrance channel and the basin is the result.

In this thesis, the underestimation is assumed to be compensated by the low value that is chosen for the critical significant wave height behind the breakwater (0.5 m).

- Hydraulic conditions based on the probability distribution of the maximum water level and the significant wave height per tidal wave can be used, see paragraph 2.2.1.1.

- Use extra damage levels for a more accurate determination of the expected damage costs by SLS failure modes. The separation between 'failure' and 'no failure' is fictitious. It is not true that a damage of 1.000.000 fl per day is caused by a transmitted significant wave height of 0.50m and for a transmitted significant wave height of 0.49m, there is no damage at all. Therefore, when more damage levels are taken into account, a more accurate estimation of the damage costs is possible.
Apart from more damage levels, the determination of the amount of the monetary damage per day can also be specified, instead of using a fixed value.

- A more detailed determination of the expected benefits of the system. The benefits of the system in one year now proportionally depend on the ULS probability of failure and the (assumed) benefits of the system in one year. For low probabilities of failure, the expected benefits of the system do not influence the optimal value for $TC_{mv2,min}$ anymore, see figure 5.12.

- For elements which contain SLS and ULS failure modes, in the optimisation on element level, the minimal investment costs as a function of both the SLS and ULS probability of failure can be determined. Like this, the ULS and SLS probability of failure are optimised with regard to each other *and* the expected damage costs by SLS failure modes do not have to be added to the investment costs of the element. A disadvantage is that this is a more complex method and the number of decision variables on system level increases, because now the SLS probability of failure also becomes a decision variable on system level.

- The physical separation between the terrain area and the sea defence is assumed. This influences the investment costs functions of the sea defence and the terrain area. A different separation changes the minimal investment costs functions of both elements and could also influence the optimal level of safety of the elements after optimisation on system level.

- The assumption that ULS failure by overtopping occurs when q_{crit} is exceeded can be changed. In fact, the process of destruction of the sea defence is only initiated.

- Although other failure modes like erosion of the toe construction of the breakwater and settlements are not taken into account, they are important with regard to possible destruction of hydraulic structures. When these failure modes are also used in the optimisation, extra decision variables will be needed.

- For more variables, instead of a deterministic value, a probability distribution can be used. Research can lead to more realistic estimations.

- Other design alternatives like a dune instead of a sea defence or a protection of a pitched block revetment instead of quarry stone layers can be used.

8. Conclusions and recommendations

By using a bottom-up approach in which the Maasvlakte 2 is assumed to be a system composed of three elements (a breakwater, a terrain area and a sea defence) and for which decision variables and failure modes are selected for each element, an economic optimal design is determined.

This bottom-up approach implies that first, for each failure mode, probabilities of failure are calculated for given hydraulic conditions and different values of decision variables. Then, the results of the failure modes are combined per element. This leads to optimal results for the elements, in which the minimal investment costs are described as a function of the level of safety for each element. Finally, the results per element are combined in the optimisation of the system, where optimal values of the minimal NPV of the total costs and the level of safety of the Maasvlakte 2 are determined. From these values, the economic optimal design has been known. With regard to the determination of the economic optimal design it can be concluded that,

- It is possible to determine an economic optimal design of the Maasvlakte 2, in which the minimal investment costs per element are expressed in terms of the level of safety of the element and not in terms of values of the decision variables of the element. This leads to a reduction in the number of decision variables in the optimisation of the system.
  The results of the optimisation of the Maasvlakte 2 are not based on an exact calculation, but on an estimation due to:
    - The finite number of combinations of values of decision variables that is used to determine the minimal investment costs as a function of the level of safety per element.
    - The selection of classes with regard to the value of the level of safety per element, for which one combination of decision variables represents the optimal combination for that class. Interpolation is needed to determine continuous functions. In the optimisation of the system, the same holds for the classes with regard to the level of safety of the system

- When instead of ULS probability of failure, risk is used as a measure for the level of safety, the damage costs of SLS mechanisms are included in the risk instead of added to the investment costs of the element. Like this, the minimal investment costs of the element are determined as a function of the risk, in which both the SLS and ULS probability of failure takes part.
  Some remarks of using risk as measure for the level of safety are:
    - When the SLS and ULS probability of failure both are included in the level of safety (risk), they are not optimised with regard to each other anymore. This may lead to slightly less optimal results.
    - It is not possible to take dependencies between failure modes of different elements into account. This implies that it is only possible to calculate an upper bound for the value of risk.
    - Consequences now also influence the level of safety. It is questionable if this is a realistic approach, apart from the fact that it may lead to complications and more assumptions in the optimisation.

- Costs as a function of risk is more abstract than costs as a function of the ULS probability of failure

- Compared to a deterministic design, in which fixed values are chosen for hydraulic conditions, the probabilistic designs for ULS probability of failure and risk as a measure for the level of safety both show much better results with regard to the combination of total costs and level of safety for the Maasvlakte 2.
  The NPV of the total costs (including benefits) is almost 50% more expensive in case of the deterministic design. Besides that, the value of the ULS probability of failure per year is much lower for the probabilistic designs without an increase of the investment costs of the Maasvlakte 2. This means that the cost advantage is the result of 'better investments'.
  Because the reduction of the ULS probability of failure (which leads to much lower damage costs for the Maasvlakte 2) is mainly caused by lower probabilities of failure with regard to erosion of the armour layer of the breakwater and erosion of the outer slope of the sea defence, the breakwater and the sea defence are considered as *cost-efficient protections* of the Maasvlakte 2.

Probabilistic design			**Deterministic design**	
	ULS prob. of failure	**Risk**		
Cost type				
Investment costs	4.46	4.48	4.54	$*10^9$ fl
Damage costs	0.17	0.19	1.22	$*10^9$ fl
Benefits	-2.15	-2.15	-2.09	$*10^9$ fl
Total costs (incl benefits)	**2.49**	**2.53**	**3.67**	$\mathbf{*10^9}$ **fl**
ULS probability of failure	$4.43*10^{-3}$	$2.98*10^{-3}$	$3.12*10^{-2}$	(-)

Table 8.1: Cost types for probabilistic designs and determinstic design

- For the two considered basic optimisations with ULS probability of failure and risk as a measure for the level of safety, only small differences were found in the results, so both optimisations can be considered as useful methods to determine an economic optimal design of the Maasvlakte 2 under the assumption of independent failure modes.

- From the sensitivity analysis follows that:
  The three variables with the highest values for the absolute elasticity with regard to the optimal value of the minimal NPV of the total costs ($TC_{mv2,min,opt}$) are:
  1. the distribution of the yearly maximum water level ($h_{sea,yr}$)
  2. the interest in one year (r)
  3. the relative density of concrete (Δ_{con})

  The three variables with the highest values for the absolute elasticity with regard to the optimal value of the ULS probability of failure of the Maasvlakte 2 per year ($P_{f,ULS,mv2,opt}$) are:
  1. the distribution of the yearly maximum water level ($h_{sea,yr}$)
  2. the relative density of concrete (Δ_{con})
  3. the relative density of quarry stone (Δ_{rock})

  It is important that the variables with high values for the absolute elasticity, are estimated well. Extra research on the distribution of the yearly maximum water level is recommended to increase the reliability of the optimal values. For this purpose, the probability distribution of the yearly maximum water level has to be based on the probability distribution of the maximum water level per tidal wave, the physical basis.

- Only a limited number of failure modes is taken into account in the optimisation. Therefore, the results have to be interpreted with caution. The use of extra failure modes is recommended to get a better representation of the actual situation. This will probably lead to more decision variables per element. As a result of this, more combinations of decision variables can be made, so more calculations have to be executed to attain the same level of accuracy. However, in this thesis, the considered failure modes give a good first indication of the situation. Within the theoretical framework, improvements of the optimisation are possible.

Literature

1. d'Angremond, K; Van Roode, F.C. (1999)
   Bed, Bank and Shore Protection 2
   Delft University of Technology, Faculty of Civil Engineering and Geosciences, Hydraulic Engineering Division

2. Centre for Civil Engineering and Research and Codes (CUR) (1997)
   Kansen in de Civiele Techniek
   Ministry of Transport, Public Works and Water Management

3. Dantzig, D. van (1960)
   The Economic Decision Problem Concerning the Security of the Netherlands against Storm Surge
   Report Delta Committee

4. Gill, P.E; Murray, W; Wright, M.H. (1981)
   Practical Optimisation
   Academic Press, London

5. Hasofer, A.M; Lind, N.C. (1974)
   Exact and Invariant Second-Moment Code Format
   Journal of Engineering Mechanics Division ASCE, volume 100, pg 111-121

6. Hohenbichler, M; Rackwitz, R. (1983)
   First-Order Concepts in System Reliability
   Structural Safety, 1983, volume 1, pg 177-188

7. Laenen, K.C.J. (2000)
   Een Probabilistisch Model voor de Vergelijking van twee Golfbrekertypen op basis van Economische Optimalisatie
   Final thesis
   Delft University of Technology, Faculty of Civil Engineering and Geosciences, Hydraulic Engineering Division

8. Meer, J.W. van der (1988)
   Deterministic and Probabilistic Design of Breakwater Armour Layers
   Journal of Waterway, Port, Coastal, and Ocean Engineering, vol.114, No 1,January

9. Meer, J.W van der; Janssen J.P.F.M. (1995)
   Wave Run-up at Dikes and Wave-overtopping at Dikes and Revetments
   Delft Hydraulics

10. Stroeve, R; Sies, R. (1999)
    Integrale Ontwerpaanpak Maasvlakte 2, Economische Optimalisatie
    Rijkswaterstaat, Bouwdienst Utrecht

11. Stroeve, R; Sies, R. (2000)
    Integrale Ontwerpaanpak Maasvlakte 2, Economische Optimalisatie Vervolgstudie
    Rijkswaterstaat, Bouwdienst Utrecht

12. Stroeve, R; Sies, R. (2001)
    Integral Optimisation of Land Reclamation in the North Sea
    Proceedings of the International Conference in Malta: Safety, Risk and Reliability – Trends in Engineering, pg 543-548

13. Voortman, H.G. (2000)
    Een Risico-gebaseerde Optimalisatiemethode voor Dijkringen
    Delft University of Technology, Faculty of Civil Engineering and Geosciences, Hydraulic Engineering Division

14. Voortman, H.G; Kuijper, H.K.T; Vrijling, J.K. (1998)
    Economic Optimal Design of Vertical Breakwaters
    Proceedings of the International Conference on Coastal Engineering (ICCE) Copenhagen: American Society of Civil Engineers (ASCE)

15. Voortman H.G; Vrijling, J.K. (1999)
    Optimalisatie van een Caissongolfbreker voor Maasvlakte 2
    In order of Samenwerkingsverband Maasvlakte 2 Varianten (SM2V)
    Delft University of Technology, Faculty of Civil Engineering and Geosciences, Hydraulic Engineering Division

16. Voortman, H.G; Vrijling, J.K. (2001)
    A Risk-based Optimisation Strategy for Large-scale Flood Defence Systems
    Proceedings of the International Conference in Malta: Safety, Risk and Reliability – Trends in Engineering, pg 543-548

17. Voortman, H.G; Vrijling, J.K; Boer S; Kortlever, W. (2000)
    Optimal Breakwater Design for the Rotterdam Harbour Extension
    Risk Analysis: Facing the New Millenium, Rotterdam 2000, pg 527-530

18. Vrijling, J.K. (1996)
    Probabilistisch Ontwerpen in de Waterbouwkunde
    Delft University of Technology, Faculty of Civil Engineering and Geosciences, Hydraulic Engineering Division

19. Vrijling,J.K; Gopalan, S; Laboyrie,J.H; Plate,S.E. (1998)
    Probabilistic Optimisation of the Ennore Coal Port
    Proceedings of the Coastlines and Breakwaters conference of the Institution of Civil Engineers (ICE)

20. Vrijling, J.K; Hengel, W. van; Houben, R.J. (1995)
    A Framework for Risk Evaluation
    Journal of Hazardous Materials, volume 43, pg 245-261

Appendix A: Hydraulic modeling for extreme conditions

A.1 Hydraulic model

For a quantitative, reliability based optimisation procedure, joint probability distributions of hydraulic parameters are needed in case of extreme conditions. Probability distributions of yearly maximum values of sea water level, significant wave height and peak period at the Maasvlakte 2 are determined by (Voortman/Vrijling, 1999):

1. probability distribution of the water level at Hook of Holland
2. conditional probability distribution of the significant wave height at Euro-0 for given water levels at Hook of Holland, which is situated close to the Maasvlakte 2, $f(H_{s,Euro}/h_{sea,H.o.H.})$.
3. the *adapted Bruinsma-function* which describes the average significant wave height at Euro-0 for given water levels at Hook of Holland.
4. probability distribution of the wave steepness at Euro-0
5. a function which describes the peak period at Euro-0 as a function of the significant wave height and the wave steepness at Euro-0
6. conditional probability distribution of the local significant wave height for given significant wave heights at Euro-0, $f(H_{s,local}/H_{s,Euro})$.
7. a function which describes the local significant wave height as a function of the significant wave height at Euro-0

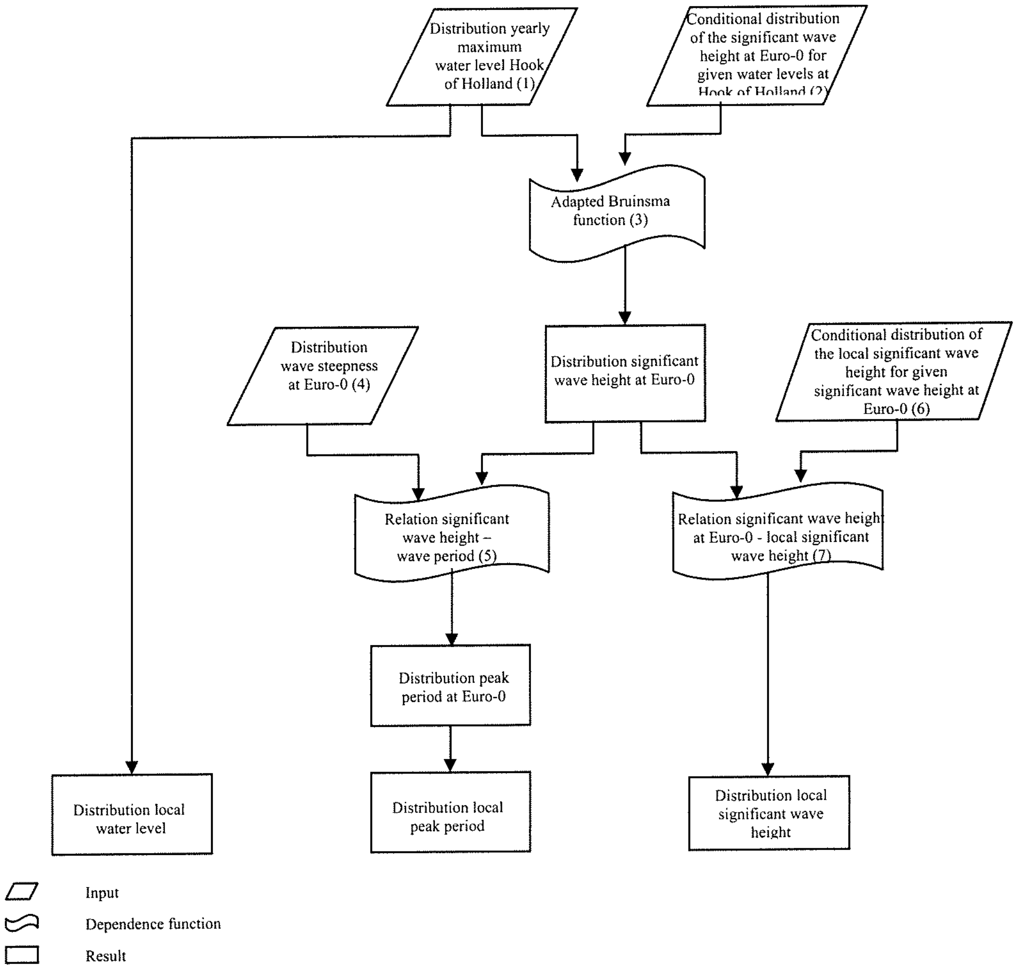


Figure A.1: Hydraulic model used to determine the yearly maximum water level, significant wave height and peak period at the Maasvlakte 2

<u>Local significant wave height</u>

With respect to data from Euro-0, the conditional probability distribution of the significant wave height at Euro-0 for given water levels at Hook of Holland (2), is described by a normal distribution with a mean value of 0 and a standard deviation of 0.6 m. This probability distribution is added to the adapted Bruinsma-function (3). Figure A.2 shows the adapted Bruinsma-function, the average significant wave height at Euro-0 for given water levels at Hook of Holland.

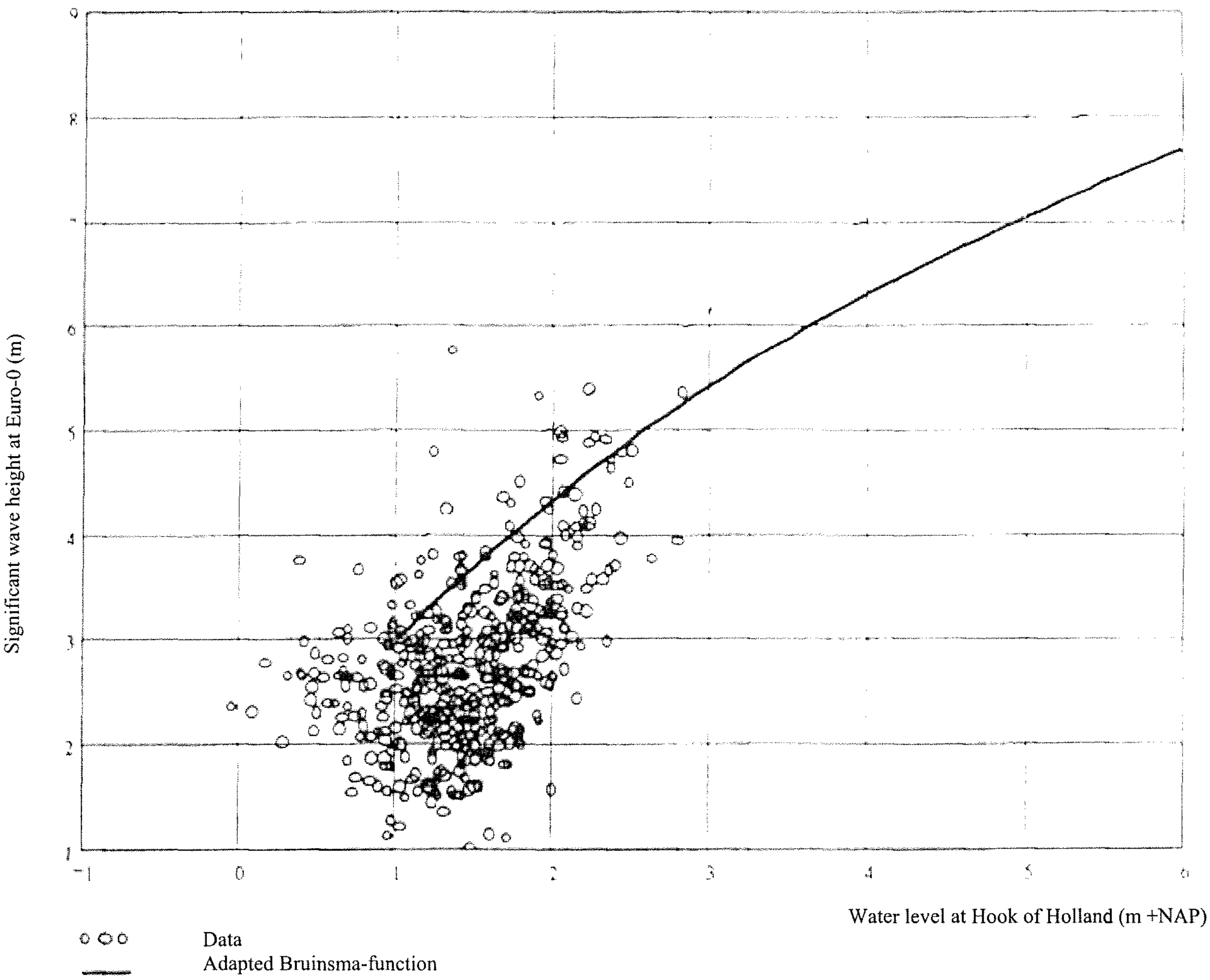


Figure A.2: Relation between water level and significant wave height for extreme conditions

Figure A.3 shows the relation between the significant wave height at Euro-0 and the local significant wave height (7). With respect to generated data at Euro-0, the conditional probability distribution of the local significant wave height for given significant wave height at Euro-0 is described by a normal distribution with a mean value of 0 and a standard deviation of 0.21 m and added to the significant wave height at Euro-0 (6). The two extra lines represent the 2,5%-97,5% confidence interval.
The wave propagation model SWAN was used to transform data of wave conditions at Euro-0 to local wave conditions at NAP-15m. For this purpose, data from extreme situations at Euro-0 were used.

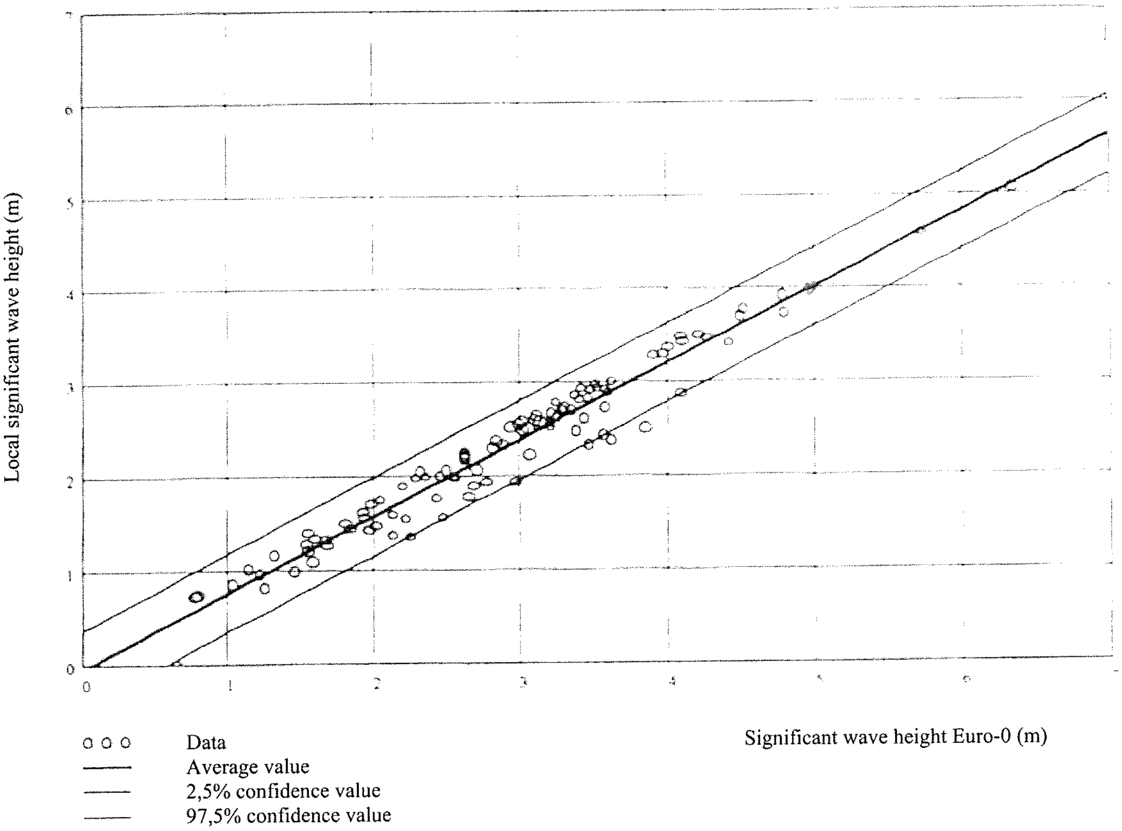


Figure A.3: Relation between significant wave height at Euro-0 and local significant wave height

<u>Local peak period</u>
To determine the distribution of the peak period at Euro-0 (5), independence between the wave steepness (4) and the wave height at Euro-0 is used. The resulting distribution is supported by data from Euro-0 (Voortman/Vrijling 1999). For the distribution of the local peak period, the same distribution is taken.

<u>Local water level</u>
For the distribution of the local water level, the distribution of the water level at Hook of Holland (1) is taken.

A.2 Extreme conditions based on maximum water levels per tidal wave

When the maximum significant wave height and water level in one year represent the sollicitation *S* in the reliability function $Z = R - S$, the probability of failure per year is in this thesis calculated by equation A.1.

$$P_{f,yr} = \int_0^{\infty} P\left(Z < 0 / h_{sea,yr}\right) \cdot f\left(h_{sea,yr}\right) dh_{sea,yr} \qquad \text{(A.1)}$$

in which:

$$Z = R - \alpha H_{s,yr} - (1-\alpha) h_{sea,yr} \qquad \text{(A.2)}$$

with:

$h_{sea,yr}$ = maximum water level in one year (m +NAP)
$H_{s,yr}$ = maximum significant wave height in one year (m)
$P_{f,yr}$ = probability of failure in one year (-)
$P(Z<0/h_{sea,yr})$ = conditional probability of failure in one year for a given value of the yearly maximum water level (-)
$f(h_{sea,yr})$ = probability density function of the maximum water level in one year, the derivative with respect to $h_{sea,yr}$ of equation 2.1 (-)
α = relative influence of the maximum significant wave height in one year on the sollicitation S (-)

When independence between values of $h_{sea,tide}$ is assumed, the probability of failure in one year based on the probability distribution of $h_{sea,tide}$, is calculated by:

$$P_{f,yr} = m \cdot \int_0^\infty P(Z < 0 / h_{sea,tide}) \cdot f(h_{sea,tide}) dh_{sea,tide} \tag{A.3}$$

In which:

$$Z = R - \alpha H_{s,tide} - (1-\alpha) h_{sea,tide} \tag{A.4}$$

with:

$h_{sea,tide}$ = maximum water level per tidal wave (m +NAP)
$P(Z<0/h_{sea,tide})$ = conditional probability of failure per tidal wave for a given value of the maximum water level per tidal wave (-)
$f(h_{sea,tide})$ = probability density function of the maximum water level per tidal wave (-)
$H_{s,tide}$ = maximum significant wave height per tidal wave, calculated by a value for $h_{sea,tide}$ (m)
m = number of tidal waves in one year (-)

From the distribution of $h_{sea,tide}$, the distribution of $h_{sea,yr}$ can be determined by:

$$P(h_{sea,yr} < \eta) = P(h_{sea,tide} < \eta)^m \tag{A.5}$$

Equation A.3 till A.5 can also be used for daily conditions, with m as the amount of tidal waves in one day.

The difference between the two approaches increases when H_s and h_{sea} have more or less the same influence on the sollicitation S. When S is almost at all determined by the significant wave height or the water level, the difference is small.

Appendix B: Selected failure modes of the breakwater

B.1 Transmission

Transmission is usually expressed by a transmission coefficient K_t. The selected transmission formulae are given by, Daemen (1991):

$$K_t = a\frac{R_c}{D} + b \tag{B.1}$$

with:

$$a = 0.031\frac{H_{si}}{D} - 0.24 \tag{B.2}$$

$$b = -5.42 s_{op} + 0.0323\frac{H_{si}}{D} - 0.0017\left(\frac{B}{D}\right)^{1.84} + 0.51 \tag{B.3}$$

in which:

K_t	$= H_{st}/H_{si}$ = transmission coefficient (-)
H_{si}	= significant wave height of incoming waves (m)
H_{st}	= significant wave height of transmitted waves (m)
R_c	= freeboard (m)
D	= nominal diameter of the concrete cubes (= edge of the cube) (m)
B	= crest width (m)
s_{op}	= wave steepness on deep water based on the peak period T_p (-)

B.2 Erosion of the armour layer

For erosion of the armour layer, the formulae of Van der Meer (1988) are used. These formulae are relevant for the design of a breakwater in an irregular wave climate. Disadvantage is that the formulae do not have a strong theoretical basis compared to alternative formulae like the Hudson or the Irribarren formula. The Van der Meer formulae are generally based on fitted data. For an armour layer of concrete blocks, the formulae are:

$$\frac{H_{is}}{\Delta_{con} D} = \left[6.7\frac{N_{od}^{0.4}}{N^{0.3}} + 1\right] s_{om}^{-0.1} \tag{B.4}$$

with:

$$\Delta_{con} = \frac{\rho_{con} - \rho_w}{\rho_w} \tag{B.5}$$

in which:

H_{is}	= significant wave height of incoming waves (m)
D	= diameter of the concrete cubes (=edge of the cube) (m)
N_{od}	= damage parameter (-)
Δ_{con}	= relative density of concrete (-)
ρ_{con}	= density of concrete (kg/m^3)
ρ_w	= density of water (kg/m^3)
N	= number of incoming waves per storm (-)
s_{0m}	= wave steepness on deep water based on the mean wave period T_m (-)

Appendix C: Selected failure modes of the sea defence

C.1 Overtopping

Van der Meer and Janssen (1995) approximated a relation between the dimensionless overtopping discharge (Q) and the dimensionless freeboard (R) for breaking and non-breaking waves:

For breaking waves, with breaker parameter: $\xi_{op} = \tan\alpha / \sqrt{s_{op}} < 2$:

$$Q_b = 0.06e^{(-4.7R_b)} \tag{C.1}$$

$$Q_b = \frac{q}{\sqrt{gH_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}} \tag{C.2}$$

$$R_b = \frac{R_c}{H_s}\frac{\sqrt{s_{op}}}{\tan\alpha}\frac{1}{\gamma_b\gamma_h\gamma_f\gamma_\beta} \tag{C.3}$$

and for non-breaking waves, with breaker parameter: $\xi_{op} = \tan\alpha / \sqrt{s_{op}} > 2$:

$$Q_{nb} = 0.2e^{(-2.3R_{nb})} \tag{C.4}$$

$$Q_{nb} = \frac{q}{\sqrt{gH_s^3}} \tag{C.5}$$

$$R_{nb} = \frac{R_c}{H_s}\frac{1}{\gamma_b\gamma_h\gamma_f\gamma_\beta} \tag{C.6}$$

In which:

Q_b	= dimensionless overtopping discharge for breaking waves (-)
Q_{nb}	= dimensionless overtopping discharge for non-breaking waves (-)
R_b	= dimensionless freeboard for breaking waves (-)
R_{nb}	= dimensionless freeboard for non-breaking waves (-)
q	= average overtopping discharge per metre width (m^2/s)
R_c	= freeboard (m)
α	= angle of the outer slope (degrees)
γ_b	= reduction factor for the influence of a berm (-)
γ_h	= reduction factor for the influence of a shallow foreshore (-)
γ_f	= reduction factor for the influence of roughness (-)
γ_β	= reduction factor for the influence of the angle of wave attack (-)

C.2 Erosion of the outer slope

The outer slope of the sea defence will be protected by layers of quarry stones. The influence of irregular waves, the duration of a storm, a damage parameter and the porosity are taken into account in the Van der Meer (1988) formulae, which are used for the outer slope protection layer:

For breaking waves, with breaker parameter $\xi_m = \tan\alpha / \sqrt{s_{0m}} < 2$:

$$\frac{H_s}{\Delta_{rock} D_{n50}} = 6.2 P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \frac{1}{\sqrt{\xi_m}} \tag{C.7}$$

For non-breaking waves, with breaker parameter $\xi_m = \tan\alpha / \sqrt{s_{0m}} > 2$:

$$\frac{H_s}{\Delta_{rock} D_{n50}} = 1.0 P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot\alpha}\, \xi_m^P \tag{C.8}$$

With:

$$S = \frac{A}{D_{n50}^2} \tag{C.9}$$

$$D_{n50} = \left(\frac{W_{50}}{g \rho_{rock}} \right)^{\frac{1}{3}} = \left(\frac{M_{50}}{\rho_{rock}} \right)^{\frac{1}{3}} \tag{C.10}$$

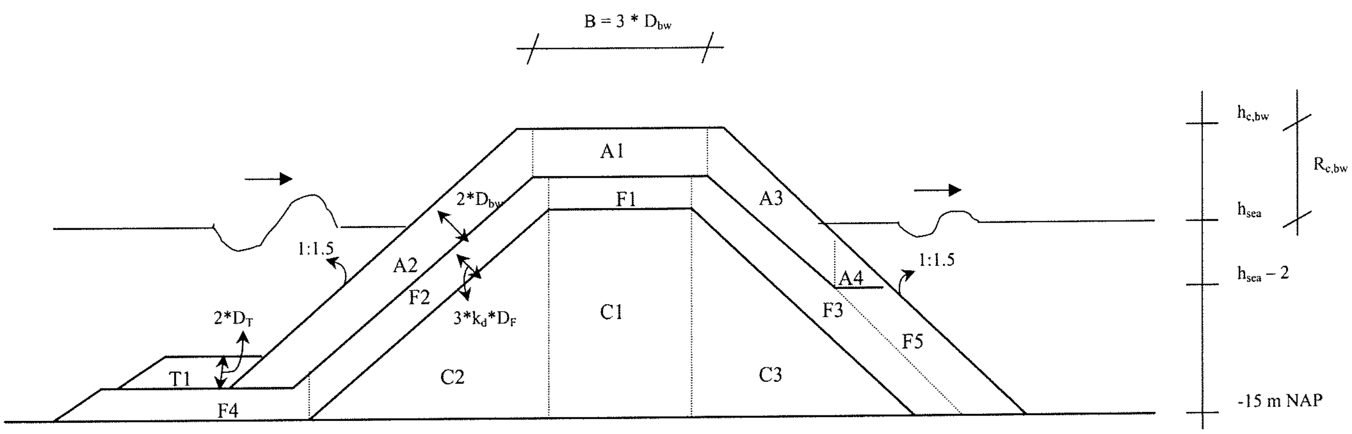
The transition between breaking and non-breaking waves is:

$$\xi_{mcrit} = \left(6.2 P^{0.31} \sqrt{\tan\alpha} \right)^{\frac{1}{P+0.5}} \tag{C.11}$$

In which:

P	= porosity (-)
S	= damage number (-)
N	= number of incoming waves per storm (-)
A	= eroded area in a cross section (m^2)
W_{50}	= mean value for the weight of the armour stones in the protection layer (N)
M_{50}	= mean value for the mass of the armour stones in the protection layer (kg)
Δ_{rock}	= relative density of quarry stone (-)
ρ_{rock}	= density of quarry stone (kg/m^3)

Appendix D: Investment costs function of the breakwater



A = *Armour layer*
F = *Filter layer*
C = *Core*
T = *Toe*

The nominal diameters of the quarry stone in the filter layer and the core are determined by filter rules. With the required ratios for the weights, the ratios of the diameters are calculated. The following ratios are used:

$$\frac{W_{50,A}}{W_{50,F}} = 10 \tag{E.1}$$

$$\frac{W_{50,F}}{W_{50,C}} = 20 \tag{E.2}$$

$$\frac{W_{50,A}}{W_{50,T}} = 5 \tag{E.3}$$

It is assumed that:

$$W_{50} = \rho \cdot D_{n50}{}^{3} \tag{E.4}$$

With ρ_{con} = 2400 kg/m^3 and ρ_{rock} = 2650 kg/m^3, it follows that:

$$D_{n50,F} = 0.45 \cdot D_{bw}$$

$$D_{n50,C} = 0.17 \cdot D_{bw}$$

$$D_{n50,T} = 0.57 \cdot D_{bw}$$

When the decision variables $h_{c,bw}$ and D_{bw} and the unit prices are known, the surface of each part of the cross-section is calculated and multiplied with its unit price. Like this, the investment costs per metre length are calculated. A multiplication factor ($\gamma_{investment}$) is used to take extra construction costs into account. Under the assumption that the profile of the cross-section is constant over the full length of the breakwater, the investment costs of the breakwater are calculated by multiplication with the total length of the breakwater.

The surfaces in the cross-section are given by, with $k_d = 0.75$:

$$A_1 = 6 \cdot D_{bw}{}^2$$

$$A_2 = \left(15 + h_{c,bw} - 3.01D_{bw}\right) \cdot 2D_{bw} \cdot \frac{\sqrt{1.5^2 + 1^2}}{1.5}$$

$$A_3 = (h_{c,bw} - h_{sea} + 2) \cdot 2D_{bw} \cdot \frac{\sqrt{1.5^2 + 1^2}}{1.5}$$

$$F_1 = 6.06D_{bw}$$

$$F_2 = F_3 = 1.5 \cdot (15 + h_{c,bw} - 3.01D_{bw}) \cdot \left(1.01D_{bw} \cdot \frac{\sqrt{1.5^2 + 1^2}}{1.5}\right)$$

$$F_4 = 4.39D_{bw}$$

$$F_5 = \left(15 + h_{sea} - 2\right) \cdot 2D_{bw} \cdot \frac{\sqrt{1.5^2 + 1^2}}{1^2}$$

$$C_1 = \left(15 + h_{c,bw} - 3.01D_{bw}\right) \cdot 6.01D_{bw}$$

$$C_2 = C_3 = 0.75 \cdot \left(15 + h_{c,bw} - 3.01D_{bw}\right)^2$$

$$T_1 = 3.25D_{bw}$$

The investment costs of the breakwater can then be calculated by:

$$I_{bw} = \left[\sum_{i=1}^{3} A_i \cdot UP_{con} + \sum_{j=1}^{5} F_i \cdot UP_{rock,F} + \sum_{k=1}^{3} C_i \cdot UP_{rock,C} + T_1 \cdot UP_{rock,T}\right] \cdot L_{bw} \cdot \gamma_{investment}$$

Appendix E: Investment costs function of the terrain area

The investment costs of the terrain area are calculated by:

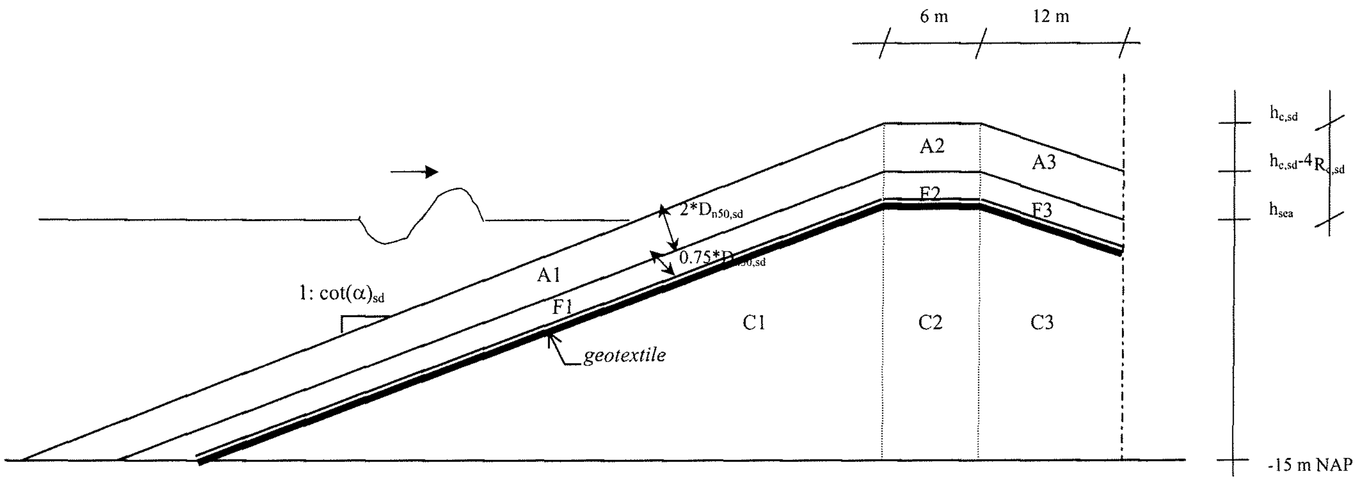
$$I_{ter} = (15 + h_{ter}) \cdot UP_{sand} \cdot A_{mv2,brute} + l_{quay} \cdot (a + b \cdot h_{ter})$$

with:

I_{ter}	= investment costs of the terrain area (fl)
h_{ter}	= height of the terrain area (m +NAP)
UP_{sand}	= unit price of sand (fl/m^3)
$A_{mv2,brute}$	= brute surface of the Maasvlakte 2 (m^2)
l_{quay}	= quay length (m)
a	= 50850 (fl/m)
b	= 3390 (fl/m^2)

The determination of the construction costs of the quay is taken from Stroeve/Sies (2000)

Appendix F: Investment costs function of the sea defence



A = *Armour layer*
F = *Filter layer*
C = *Core*

Like in case of the breakwater holds that:

$$\frac{W_{50,A}}{W_{50,F}} = 10 \tag{F.1}$$

For $\rho_{rock} = 2400$ kg/m^3 for both armour layer and filter holds:

$$D_{n50,F} = 0.5 \cdot D_{n50,sd}$$

When the decision variables $h_{c,sd}$, $cot(\alpha)_{sd}$, $D_{n50,sd}$ and the unit prices of quarry stone and sand are known, for each part of the cross-section, the surface is calculated and multiplied with its unit price. Like this, the investment costs per metre length are calculated. Under the assumption that the profile of the cross-section is constant, the investment costs of the sea defence are calculated after multiplication with the total length of the sea defence and a multiplication factor ($\gamma_{investment} = 1.3$) which takes account of extra construction costs like the costs of the geotextile.

$$A_1 = \left(\left(15 + h_{c,sd}\right) \cdot \cot(\alpha)_{sd}\right) \cdot 2D_{n50,sd} \cdot \frac{\sqrt{\cot(\alpha)_{sd}^2 + 1^2}}{\cot(\alpha)_{sd}}$$

$$A_2 = 6 \cdot 2D_{n50,sd}$$

$$A_3 = 12 \cdot \frac{\sqrt{3^2 + 1^2}}{3} \cdot 2D_{n50,sd}$$

$$F_1 = 0.75 D_{n50,sd} \cdot \frac{\sqrt{\cot(\alpha)_{sd}^2 + 1^2}}{\cot(\alpha)_{sd}} \cdot \left(15 + h_{c,sd} - 2 D_{n50,sd}\right)$$

$$F_2 = 6 \cdot 0.75 D_{n50,sd}$$

$$F_3 = 12 \cdot \frac{\sqrt{3^2 + 1^2}}{3} 0.75 D_{n50,sd}$$

$$C_1 = 0.5 \cdot \left(15 + h_{c,sd} - 2.75 D_{n50,sd}\right) \cdot \left(\left(15 + h_{c,sd} - 2.75 D_{n50,sd}\right) \cdot \cot(\alpha)_{sd}\right)$$

$$C_2 = 6 \cdot \left(15 + h_{c,sd} - 2.75 D_{n50,sd}\right)$$

$$C_3 = 24 + 12 \cdot \left(15 + h_{c,sd} - 4 - 2.75 D_{n50,sd}\right)$$

The investment costs of the sea defence can be calculated by:

$$I_{sd} = \left[\sum_{i=1}^{3} A_i \cdot UP_{rock,A} + \sum_{j=1}^{3} F_i \cdot UP_{rock,F} + \sum_{k=3}^{3} C_i \cdot UP_{sand,C} \right] \cdot L_{bw} \cdot \gamma_{investment}$$

Appendix G: Probabilistic calculation method

G.1 Deterministic and random variables

In probabilistic calculation methods, deterministic and random variables can be distinguished. Deterministic variables always have the same value. However, random variables do not have one determined value, so uncertainty is included. This uncertainty is characterized by a probability distribution and a probability density function. The probability distribution function determines the probability that a variable X exceeds some value x. Probability is always defined on the interval [0,1].
This can be written in formula:

$$F_X(x) = P(X \leq x) \tag{G.1}$$

with:

$F_X(\bullet)$ = probability distribution function of random variable X

The probability density function is described as the derivative with respect to x of the distribution function. In formula:

$$f_X(x) = \frac{dF_X(x)}{dx} = P(x < X \leq x + dx) \tag{G.2}$$

with:

$f_X(\bullet)$ = probability density function of random variable X

Distribution functions and probability density functions are described analytically by parameters. The most important parameters are the *expected value* ($E(X)$) and the *variance* (σ_X^2).
The expected value is defined as the mean value μ_X of the variable X. For this value holds: $F_X(x) = 0,5$.
The variance is the value that indicates to what extent X differs from its mean value in an absolute way. In formula:

$$\sigma_X^2 = \text{var}(X) = E((X - \mu_X)^2) \tag{G.3}$$

The standard deviation (σ_X) is often used as a measure for deviation and is defined as the squareroot of the variance. The *relative* measure of differentiation around the mean value is the variability coefficient V_X, which is defined as the standard deviation divided by the mean value, σ_X/μ_X.

G.2 Probabilistic calculation methods

G.2.1 General

In order to calculate the probability of failure by a failure mode, several methods are available. For each failure mode, a reliability function will be used according to: $Z = R - S$. Z is the reliability function, R represents 'resistance', S represents 'sollicitation' or load.

Z <0 is considered as the situation in which load exceeds resistance and failure occurs. Accordingly *Z = 0* can be defined as the separation between 'failure' (*Z<0*) and 'non-failure'(*Z>0*). This limitation is important to determine the reliability and depends on the margin between resistance and load.
To design structures that guarantee a level of safety, it is important to know the reliability for different values of the decision variables. There are several ways to determine reliability, roughly divided into three main classes, level I, level II and level III methods (CUR, 1997):

- In level I calculations, reliability is assumed to be guaranteed by taking a margin between (estimated) load and (needed) resistance using partial safety factors, γ_S and γ_R. The probability of failure is not calculated.
- In level II methods, the reliability is determined by linearization of the reliability function in a carefully chosen point. All distribution functions are assumed to be normal or standard normal distributed.
- In level III methods, the exact probability of failure is calculated by taking distribution types of all (resistance and load) variables into account. The reliability is directly related to the probability of failure.

G.2.2. Level II methods

In level II calculations, the reliability function is defined by $Z = R - S$, the difference between resistance and load, dependent on random variables ($X_1 .... X_N$). The reliability of a structure is determined by its probability of failure, the probability that load exceeds resistance, *P(R>S)*, or *P(Z<0)*. This probability of failure has a maximum in case of *Z = 0*, the limit between 'failure' and 'no failure'. The function *Z = 0* is therefore used to determine the probability of failure. The point on the function *Z = 0* with a maximum value for the probability density, is called the *design point*. Level II calculations are based on an iteration process to find the design point by linearization (1st order Taylor expansion) of the limit state function.
Level II calculations are either executed in normal space (X-space) or in standard normal space (U-space). Therefore, all variables have to be normal or standard normal distributed.
In this thesis, U-space calculations are used. For this purpose, normal distributed variables *X* are transformed into standard normal distributed variables *U* by:

$$U_i = \frac{X_i - \mu_{Xi}}{\sigma_{Xi}} \tag{G.4}$$

with:

U_i = standard normal distributed variable

The first-order Taylor expansion of the limit state function can be described by:

$$Z \approx Z(U_1^*, U_2^* .... U_n^*) + \sum_{i=1}^{n} \frac{\delta Z}{\delta U_i}(U_i^*) \cdot (U_i - U_i^*) \tag{G.5}$$

with:

U_i^* = the value of U_i in the design point

To calculate the expected value and standard deviation for *Z*, the expected value of the linearization is taken. In U-space holds: $\mu_{Ui} = 0$ and $\sigma_{Ui} = 1$

$$\mu_Z = Z(U_1^*, U_2^* \ldots U_n^*) + \sum_{i=1}^{n} \frac{\delta Z}{\delta U_i}(U_i^*) \cdot (-U_i^*) \tag{G.6}$$

$$\sigma_Z = \left[\sum_{i=1}^{n} \left(\frac{\delta Z}{\delta U}(U_i^*) \right)^2 \right]^{\frac{1}{2}} \tag{G.7}$$

The reliability index *β*, which represents the length of the perpendicular between the origin and the linearized reliability function in U-space, is given by:

$$\beta = \frac{\mu_Z}{\sigma_Z} = \frac{Z(U_1^*, U_2^* \ldots U_n^*) + \sum_{i=1}^{n} \frac{\delta Z}{\delta U_i}(U_i^*)(-U_i^*)}{\left| \sum_{i=1}^{n} \alpha_i \cdot \frac{\delta Z}{\delta U_i}(U_i^*) \right|} \tag{G.8}$$

with:

β	= reliability index
α_i	= influence coefficient of the i^{th} variable

α_i is a measure for the relative contribution of the uncertainty of each variable to the total uncertainty. In formula:

$$\alpha_i = -\frac{\frac{\delta Z}{\delta U_i}(U_i^*)}{\left[\sum_{i=1}^{n} \left(\frac{\delta Z}{\delta U}(U_i^*) \right)^2 \right]^{\frac{1}{2}}} \tag{G.9}$$

In the beginning of the iteration process, a starting point has to be chosen, for which the expected value of each variable is used. After each iteration step, a new design point is found according to:

$$U_{i,j+1}^* = \alpha_i \cdot \beta \tag{G.10}$$

The iteration process stops when the design point is approximated enough accurately, so $Z(U_1^*, U_2^* \ldots U_n^*) \approx 0$ and the realiability index β is stable for each new iteration. The design point values of the variables are:

$$X_i^* = \mu_{Xi} + U_i^* \sigma_{Xi} \tag{G.11}$$

The probability of failure can be calculated by:

$$P_f = \Phi(-\beta) \tag{G.12}$$

with:

$\Phi(\bullet)$ = standard normal probability distribution function

At the beginning of the optimisation normal distributed variables are transformed from normal into standard normal distributed variables. However, not all variables are normal distributed and easy to transform to a value for *U*. For non-normal distributed variables a transformation is executed in which the value of the probability distribution of the non-normal distributed variable is assumed to be equal to the value of the probability distribution of a standard normal distributed variable. In formula:

$$F_X(X^*) = \Phi(U^*) \tag{G.13}$$

The mean value and standard deviation of the transformed distribution are given by:

$$\sigma_x^{'} = \frac{\varphi(U_i^*)}{f_X(F_X^{-1}(\Phi(U_i^*))} \tag{G.14}$$

$$\mu_X^{'} = F_X^{-1}\left(\Phi\left(U_i^*\right)\right) - \sigma_X^{'} \cdot U_i^* \tag{G.15}$$

with:

$\varphi(\bullet)$ = standard normal probability density function

$F_X^{-1}(\bullet)$ = inverse probability distribution function of random variable X

Because the transformed values depend on design point values, a new transformation has to be executed after each iteration step.

On the next page, a flow diagram of the total procedure is given.
The relatively easy and transparant procedure makes a Level II optimisation useful, but there also are some disadvantages:

- discontinuous limit state functions and discontinuous probability density functions cannot be used
- dependencies are difficult to determine, significant inaccuracies arise when neither 'full dependence' nor 'no dependence' is a relevant assumption to combine probabilities of failure from different mechanisms
- it is difficult to take more than one limit state into account in one optimisation

For one of these situations, level III methods can be used.

In a flow diagram the total procedure is given:

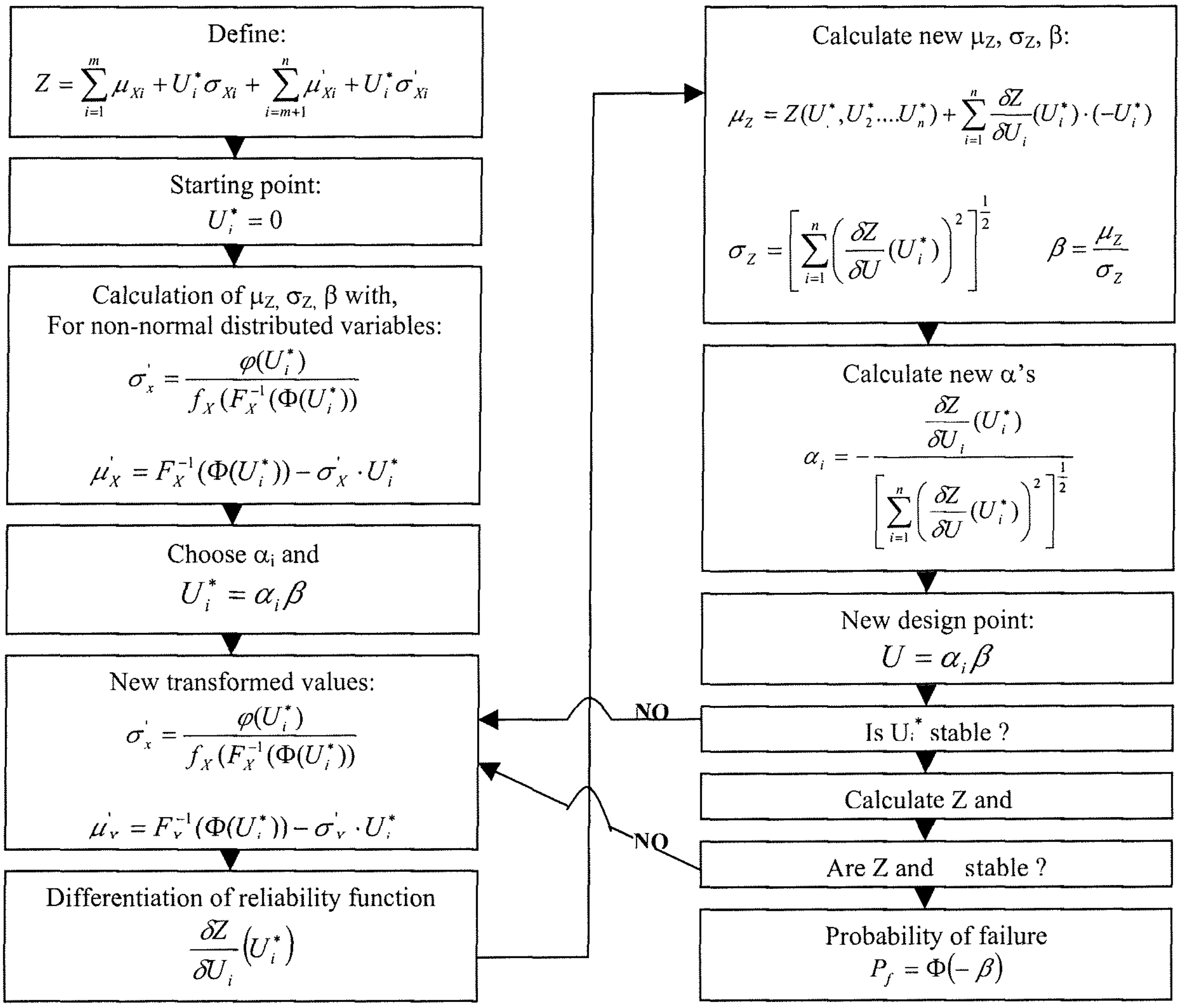


Figure G.2: Procedure Level II calculation of probability of failure

G.2.3 Level III Methods

In level III methods the distribution types of all variables are taken into account. A distinction can be made in 'Full Integration' and 'Monte Carlo simulation'.

With *full integration*, the exact probability of failure is calculated by full integration over the failure domain *Z<0*. The probability distribution functions of all resistance and load variables are taken into account. This integration is often not possible in an analytical way, so numerical integration is used. Discontinuous functions and dependencies are taken into account.

Main disadvantages are:

- strongly increasing calculation time when more random variables are used, proportional with: #integration steps $^{\#\text{random variables}}$
- not much insight in the contribution of uncertainty of variables to the total uncertainty, the α_i's. It is then not deductable for which variables it would be useful to reduce uncertainty and for which not.

Another Level III method is *Monte Carlo simulation*. Here, the probability of failure is determined by the percentage of random simulations for which *Z<0*. To obtain reliable results, a lot of simulations have to be executed. The number of simulations should at least be around $1/P_f$. For very small probabilities of failure, this is time consuming. Therefore methods have been developed to improve accuracy and reduce the variance of outcomes and calculation times. Importance sampling, direction sampling and Latin hypercubes are examples of this. Compared to the full integration method, the calculation times do not increase very fast for an increasing number of random variables, but it is (especially for small P_f's) more difficult to attain high accuracy.

Appendix H: Alternatives of numerical optimisation

In an optimisation process, several numerical optimisation methods can be used. Two basic types are the most important. Many alternative methods can be created by combinations of these types:

- Direct-search methods
- Descent methods

In *direct-search methods*, many values of a goal function (for instance: an investment costs function) are determined. The value for which the goal function is minimal, is the optimum value. This method is simple and robust, but not very accurate. It has the advantage that not only a global optimum, but also local optima are found.

Descent methods are based on finding an optimum by searching in the direction for which the derivative is mostly negative. The minimum is the value where the derivative is positive for each direction. This method is more accurate, but the minimum that is found depends on which starting point is chosen. It is possible that a local minimum is found instead of a global minimum (Gill/Murray, 1981).

In this thesis, the direct-search method is used on element and system level. On element level, for each class of the level of safety ('probability of failure' or 'risk'), the combination of decision variables with a minimum value of the investment costs is determined and used as the optimum value for that class. Then, interpolation between optimal values leads to the minimum investment costs as a function of the level of safety.
The more classes are created, the more minimal investment costs values are determined, so the interpolation distance becomes smaller. On the other hand, when very narrow classes are chosen, the amount of investment costs values per class decreases, so the minimum investment costs value for each class is determined from a smaller set of values. This can create a minimal investment costs function with a 'grass pattern', which is more inaccurate with regard to the value of the minimum investment costs. This grass pattern is created by the lack of generated data. The values of the probability of failure (or risk) and the investment costs are calculated for a finite amount of combinations of values of decision variables, because these values are varied with a step size.
For example, when two decision variables are each varied for 40 values, 40*40 = 1600 combinations can be made and also 1600 values for the probability of failure and the investment costs are calculated. When for instance 1000 classes are created, a grass pattern will be the result, because on average the minimum investment costs value per class is based on 1600/1000 = 1.6 values for the investment costs. Obviously, this amount is too small to make a good estimation of the value of the minimum investment costs.

It has to be considered what is preferred: *more accuracy* with regard to the value of the minimal investment costs, but a smaller amount of values (figure 3.4a), or *more values* of minimal investment costs, but a smaller accuracy with regard to the value of the minimal investment costs (figure H.1 and H.2).

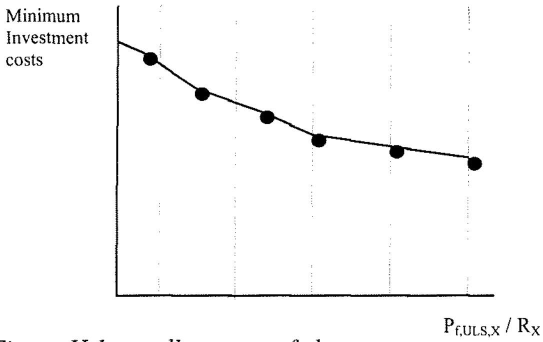


Figure H.1: small amount of classes

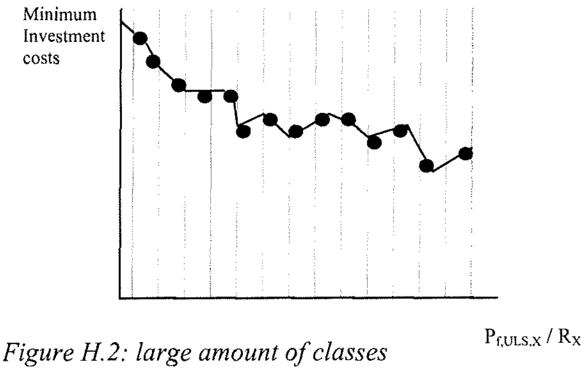


Figure H.2: large amount of classes

Appendix I: Optimisation with risk as a measure for the level of safety for the terrain area and the sea defence

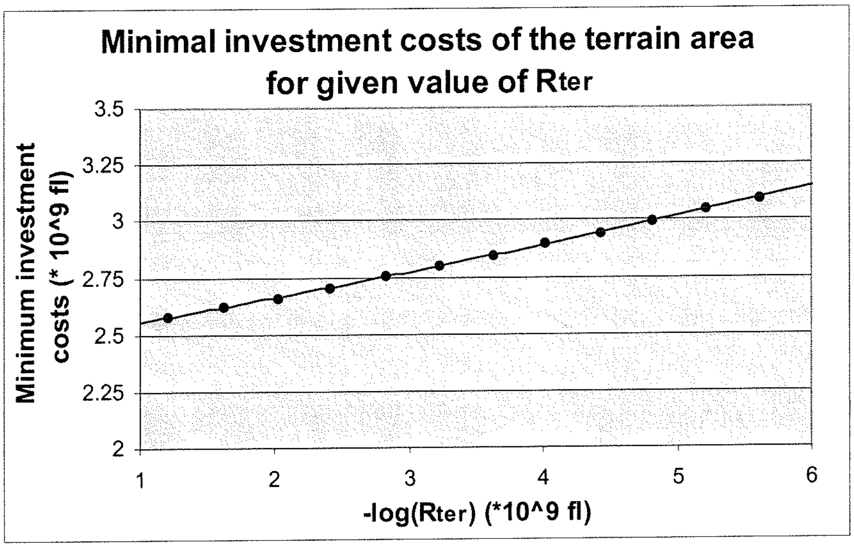


Figure I.1: Minimal investment costs of the terrain area

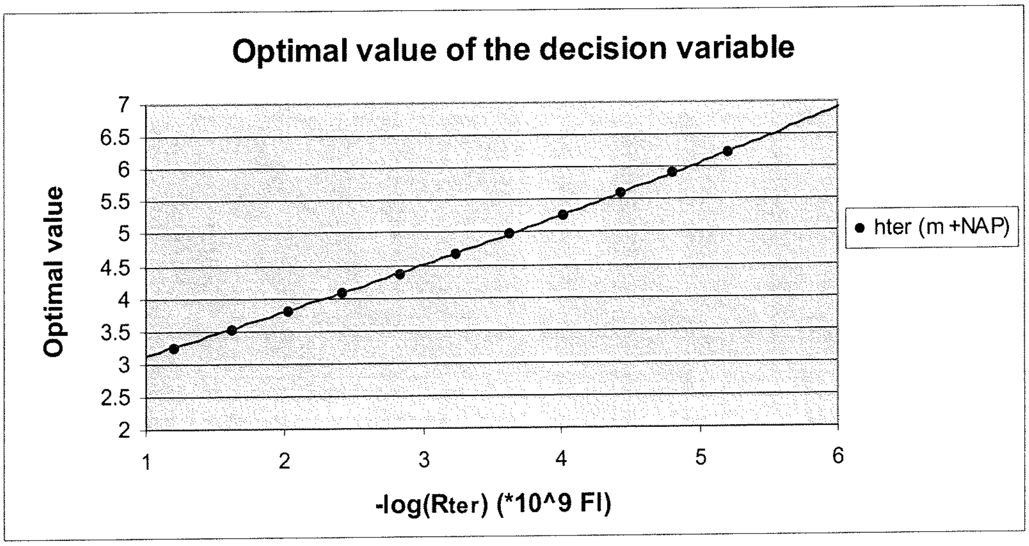


Figure I.2: Optimal value of the decision variable of the terrain area

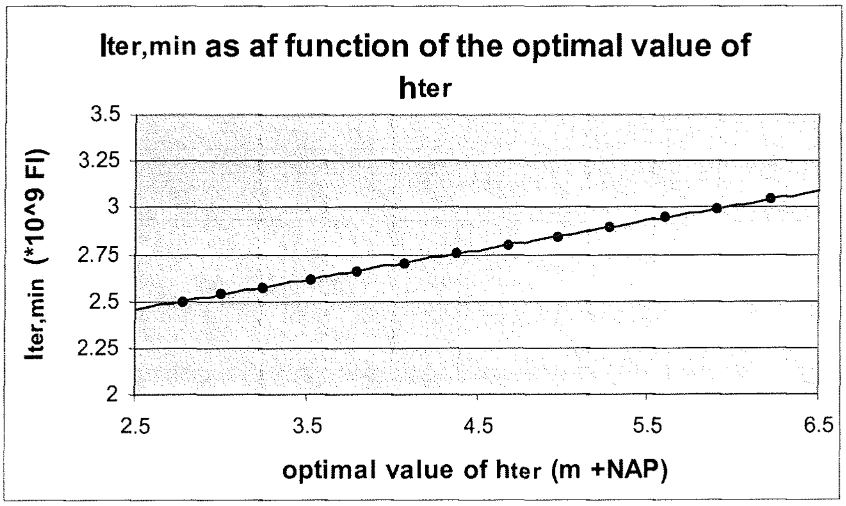


Figure I.3: Minimal investment costs as a function of the optimal value of the height of the terrain area

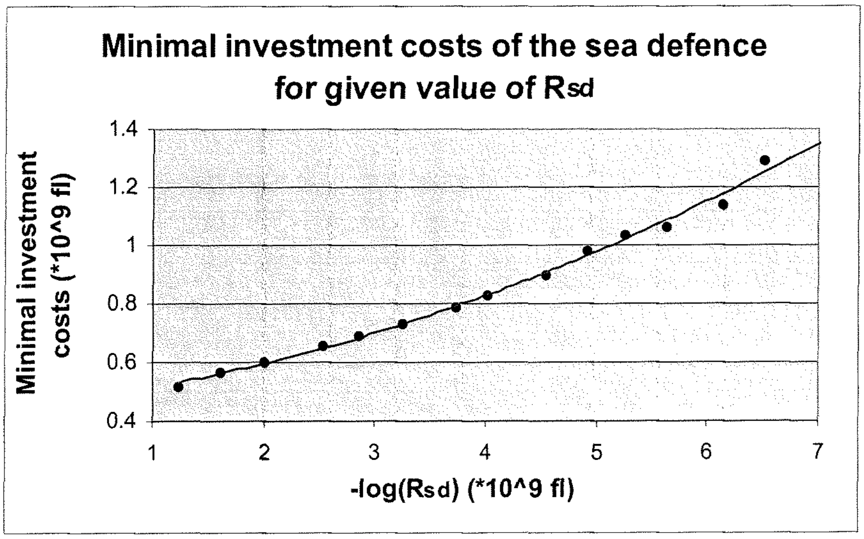


Figure I.4: Minimal investment costs of the sea defence

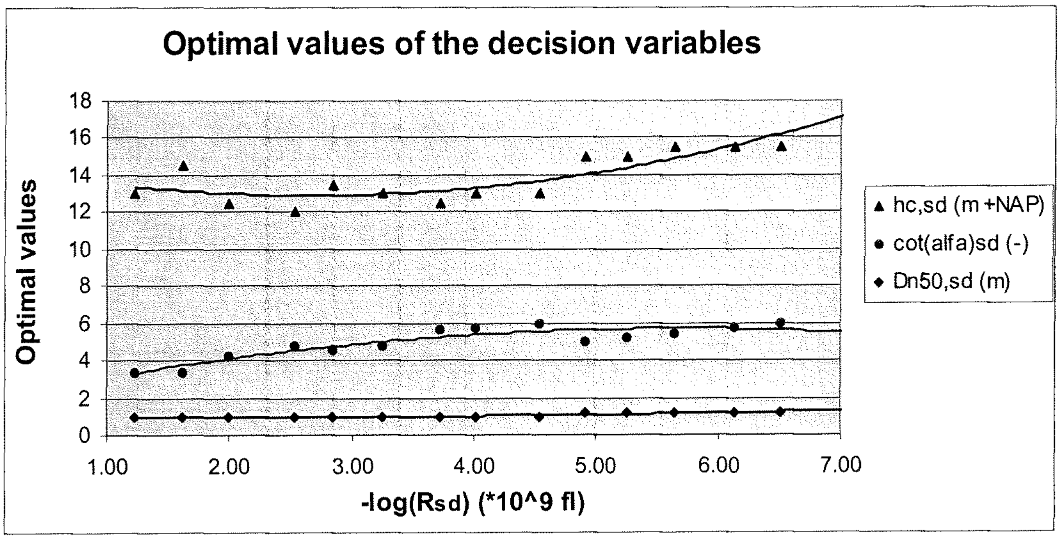


Figure I.5: Optimal values of the decision variables of the sea defence

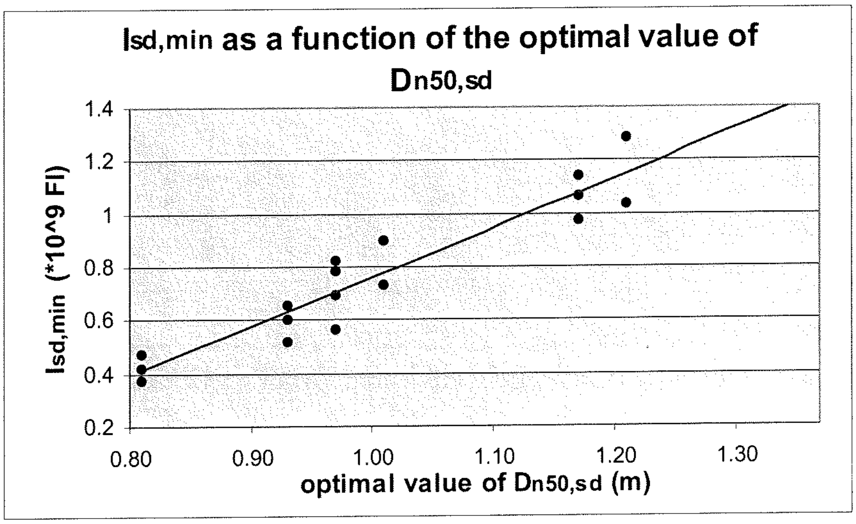


Figure I.6: Minimal investment costs as a function of the optimal value of the diameter of the quarry stones in the protection layer of the sea defence

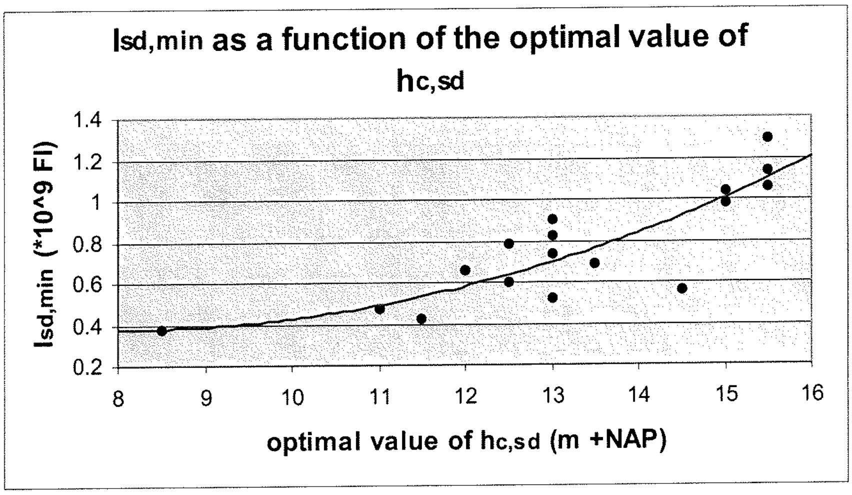


Figure I.7: Minimal investment costs as a function of the optimal value of the crest height of the sea defence

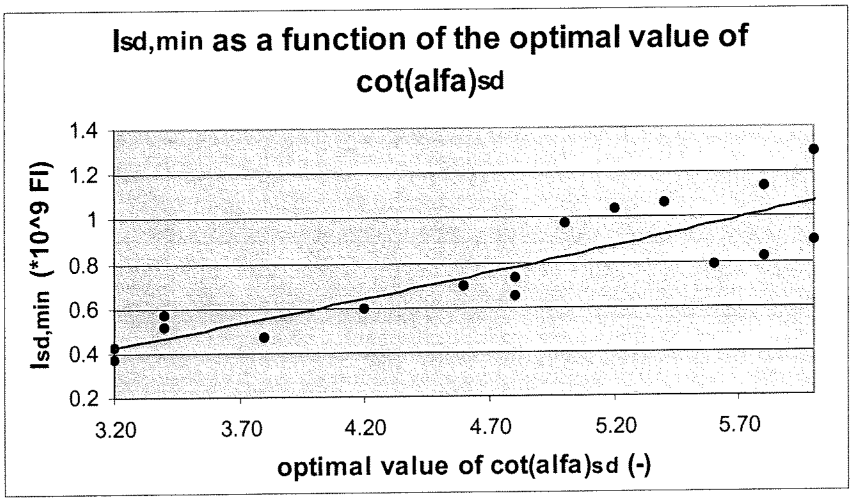


Figure I.8: Minimal investment costs as a function of the optimal value of the outer slope angle of the sea defence

Appendix J: Variations of variables used in sensitivity analysis

The following variations were used in the sensitivity analysis:

Variable	Description	Unit	Value	Variations			
Hydraulic conditions							
$h_{sea,yr}$	Maximum water level per year	m +NAP					
	parameter w		2.59	2.55	2.65		
	parameter τ		2.4	2.35	2.45		
	parameter k		0.85	0.75	0.95		
$H_{s,day}$	Maximum significant wave height per day	m					
	parameter α		2.15	2	2.3		
	parameter u		0.74	0.7	0.8		
$fH_{s,Euro}$	Conditional distribution of the significant wave height at Euro-0 for given water levels at Hook of Holland	m	Additive $N(0,0.6)$	Additive $N(0,0.4)$	Additive $N(0,0.8)$	Multipl $N(1,0.05)$	Multipl $N(1,0.15)$
$fH_{s,local}$	Conditional distribution of the local significant wave height for given significant wave heights at Euro-0	m	Additive $N(0,0.21)$	Additive $N(0,0.16)$	Additive $N(0,0.26)$	Multipl $N(1,0.025)$	Multipl $N(1,0.075)$
s_{0p}	Wave steepness based on peak period	-	$N(0.038, 0.0059)$	$N(0.038, 0.003)$	$N(0.038, 0.009)$		
T_p/T_m	Peak period / Average period	-	1.2	1.1	1.3		
N	Number of waves in a storm	-	3000	2500	4000		
Critical values							
H_{crit}	Critical significant wave height behind the breakwater	m	0.5	0.4	0.6		
q_{cr}	Critical overtopping discharge per meter width	l/s/m	5	7	10		
N_{cr}	Critical damage for exposed core of the breakwater	-	2	1.5	2.5		
S	Damage parameter in case of an exposed core	-	15	12	18		
Geometry							
B	Crest width of the breakwater	m	$3*D_{bw}$	$2*D_{bw}$	$4*D_{bw}$		
Δ_{con}	Relative density of concrete	-	$N(1.4,0.1)$	$N(1.4,0)$	$N(1.4,0.2)$	$N(1.3,0.1)$	$N(1.5,0.1)$
Δ_{rock}	Relative density of quarry stones	-	$N(1.65,0.1)$	$N(1.65,0)$	$N(1.65,0.2)$	$N(1.55,0.1)$	$N(1.75,0.1)$
P	Porosity	-	0.3	0.2	0.4	0.5	
L_{bw}	Length of the breakwater	m	5700	5130	6270		
L_{sd}	Length of the sea defence	m	12000	10800	13200		
l_{quay}	Length of the quay	m	25000	22500	27500		
modfac	Model factor	-	Multipl. 1	Additive $N(0,0.106)$	Multipl. $N(1,0.15)$		
Costs							
UP_{rock}	Unit price of quarry stones	fl/m^3					
	0.3 – 0.49 m		75	67.5	82.5		
	0.49 – 0.72 m		80	72	88		
	0.72 – 1.04 m		90	81	99		
	1.04 – 1.31 m		100	90	110		

Variable	Description	Unit	Value	Variations			
UP_{con}	Unit price of concrete	fl/m^3	750	675	825		
UP_{sand}	Unit price of sand	fl/m^3	5	4	6		
$D_{SLS,trans}$	Expected damage costs per day in case of failure by the SLS failure mode 'Transmission'	fl	1000000	800000	1200000		
D_{ULS}	Expected damage costs per event in case of ULS failure	fl	$0.2*I_{mv2}$	$0.15*I_{mv2}$	$0.25*I_{mv2}$		
B	Benefits of the Maasvlakte 2 in one year	$*10^6$ fl	50	30	80		
$\gamma_{investment}$	Multiplication factor which takes extra construction costs for the sea defence and the breakwater into account	-	1.3	1.17	1.43		
Discount factor							
r	Interest in one year	-	0.06	0.05	0.07		
i	Inflation in one year	-	0.02	0.01	0.03	0.04	
g	Economic growth in one year	-	0.02	0.01	0.03		
N	Considered number of years	-	100	50	75	125	