Decision alternatives for the safety of the Eastern Scheldt

Will it be cost-effective to remove the Eastern Scheldt storm surge barrier in case of sea level rise?

Msc Thesis
W.J. Leeuwdrent
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Witteveen + Bos

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SUMMARY

Background
After the disastrous floods in 1953 in the Netherlands, measures are taken to improve the safety in the Netherlands. In the Southwest of the Netherlands a couple of tidal basins are closed by the construction of dams and storm surge barriers. The closure of tidal basins shortens the coastal length and can result in a cost reduction comparing with heightening of dikes. The Eastern Scheldt storm surge barrier is one of the most well-known storm surge barriers. During the decision-making in the '70s it was decided to construct a barrier with closable gates, in order to maintain the tide in the Eastern Scheldt. In that way the barrier can be closed during a storm surge and the tide can (mostly) be maintained during normal conditions. Tidal influence is important for ecology and fishery in the Eastern Scheldt.

In view of the expected sea level rise the Deltacommission 2008 stated that the barrier cannot operate safely at a sea level rise of more than 1,0m and preferably, therefore, has to be removed. Furthermore the barrier causes sand hunger whereby large parts of the sandbars will ultimately disappear under water. This will affect subterranean animals and birds which are dependent on the sandbars. When the barrier will be removed these consequences can (largely) be prevented.

This study
In this study the best decision alternative for the safety of the Eastern Scheldt is investigated in view of costs and flood risk. This is done by defining two alternatives: (1) maintaining the Eastern Scheldt storm surge barrier and (2) removing the Eastern Scheldt storm surge barrier. In the analysis 3 scenarios of sea level rise are regarded: the current sea level, a sea level rise of 0,5m and a sea level rise of 1,0m. The consequences of the alternatives, such as the required dike reinforcement (heightening and widening) and reinforcements on the barrier, are determined and a cost-benefit analysis is made.

The required safety is regarded based on two principles: the current safety standard and a cost-benefit analysis. According to the current safety standard the flood defences in Zeeland are designed to withstand water levels (and waves) which occur once per 4.000 year. In a cost-benefit analysis, such as applied in 'Flood protection 21st century' (Dutch: Waterveiligheid 21e eeuw (WV21)), the safety standard and the corresponding dike height is based on the values to be protected and the costs.

This research focuses specifically on safety and the costs and benefits of related measures. Additional costs for adjustments along the Eastern Scheldt, e.g. the construction of small storm surge barriers for yacht harbours by removing the barrier, are not considered in this study. Benefits, such as ecology, mobility and recreation, are considered qualitatively but are not part of the cost-benefit analysis.

The results
Based on a consideration of the design loads of the Eastern Scheldt storm surge barrier and using expert opinion it appears that the barrier can withstand a sea level rise until 1,0m without major adaptations. It is not investigated whether the barrier can withstand more sea level rise. The barrier and the current dikes provide for a lot of safety against floods around the Eastern Scheldt, especially because of the high construction dike height before

the construction of the barrier. The probability of failure of the dike due to overtopping at the current sea level is calculated at 1/3.500.000 per year. When the barrier will be removed the probability of high water levels increases and the probability of overtopping becomes 1/500 per year.

In this study two mechanisms of failure of dikes are taken into account: failure due to overtopping of waves and failure due to the flow of water beneath the dike (piping). These are the most important failure mechanisms, partly because the dike cover is currently reinforced by 'Projectbureau Zeeweringen'. Measures against overtopping and piping are respectively dike heightening and dike widening. Based on local data one representative profile is used for all dikes around the Eastern Scheldt.

Beside the heightening and widening of dikes there are costs of maintenance of the barrier (20 million euro per year) and the costs of the removal of the barrier (estimated at 1 billion euro). The total costs of the alternatives are determined using the Net Present Value (NPV). In this way the yearly costs in the future are translated to the current value. The results on dike reinforcements and costs, when satisfying to the current safety standard (for Zeeland a water level which occurs once per 4.000 year), are shown in Table 1. As can be seen in the table every investment leads to a negative present value. The investment costs more than it delivers. However, ecology, fishery and other benefits are not taken into account in this study. Therefore the minimum required benefits for a NPV of zero are visible in the last column. At a NPV of zero or higher the investment is cost-effective.

	sea level rise	dike heightening	length piping measures (dike widening)	Net Present Value	required benefits for NPV=0
maintain barrier	0,0 m	0 m	0 m	-384 M€	20 M€/year
	0,5 m	0 m	0 m	-384 M€	20 M€/year
	1,0 m	0 m	7 m	-413 M€	22 M€/year
remove barrier	0,0 m	0,7 m	15 m	-1755 M€	91 M€/year
	0,5 m	1,4 m	19 m	-2333 M€	122 M€/year
	1,0 m	2,2 m	22 m	-3126 M€	163 M€/year

Table 1: measures and costs when the current safety standard is satisfied

It turns out that, if the current safety standard is satisfied, maintaining the barrier is a lot cheaper than removing the barrier. This is partly due to the magnitude of the costs for the removal of the barrier.

When the safety is based on the values to be protected around the Eastern Scheldt, it turns out that the optimum flooding probability is 1/500 per year. This also approximates the results of WV21. Because of the relative low economic value and number of people around the Eastern Scheldt, the safety can be lower from an economically point of view. Only the optimum dike height is considered, the optimal dike width is not calculated. In principal also the optimum dike width can be calculated, but this is not done since it makes the calculation more complex and does not influence the result significantly. When the dike height is based on the values to be protected this results in dike reinforcements and costs as can be seen in Table 2.

	sea level rise	dike heightening	Net Present Value	required benefits for NPV=0
maintain barrier	0,0 m	0 m	-384 M€	20 M€/year
	0,5 m	0 m	-384 M€	20 M€/year
	1,0 m	0 m	-384 M€	20 M€/year
remove barrier	0,0 m	0 m	-1259 M€	66 M€/year
	0,5 m	0,6 m	-1796 M€	94 M€/year
	1,0 m	1,3 m	-2351 M€	123 M€/year

Table 2: measures and costs when the safety is based on the values to be protected

When the barrier will be removed the dike height can be lower when the required flooding probability is 1/500 per year. This makes the difference in costs between maintaining and removing the barrier smaller, but still maintaining the barrier is cheaper.

When the barrier will be maintained the sandbars will ultimately disappear under water, which has negative consequences for the current ecological conditions. It is expected that this can be (largely) prevented when the barrier will be removed. A measure to maintain the sandbars in case of maintaining the barrier is to supply it with sand. When these costs (indication: 45 million euro per year) are taken into account, still maintaining the barrier is cheaper.

Recommendations

As can be seen in the tables every investment results in a negative present value. An investment in safety is cost-effective if the reduction in flood risk is bigger than the costs for gaining safety. The current flood risk however is already low, this can hardly be reduced. A measure to reduce the costs is to take the gates of the barrier out of operation. The barrier only functions as an obstacle for storm surge and waves. The gates take a lot of the maintenance costs (20 million euro) into account. From this study it appears that the dikes are still high enough when the gates of the barrier are not closed during high water levels. The dikes will have to be widened to reduce the probability of piping. The problem with the sand hunger will not be solved by this measure.

SAMENVATTING

Achtergrond

Na de watersnoodramp in 1953 in Nederland, zijn maatregelen getroffen om Nederland veiliger te maken. In het zuidwesten van Nederland zijn een aantal zeearmen afgesloten door het bouwen van dammen en stormvloedkeringen. Het afsluiten van zeearmen verkort de kustlijn en kan zo een kostenbesparing ten opzichte van dijkversterkingen opleveren. De Oosterscheldekering is één van de meest bekende stormvloedkeringen. Om het getij in de Oosterschelde te behouden is tijdens de besluitvorming in de jaren 70 van de vorige eeuw gekozen voor een ontwerp met beweegbare schuiven. Hiermee kan de kering bij een stormvloed worden gesloten, maar kan de getijwerking (grotendeels) worden behouden in normale omstandigheden. Getij-invloed is belangrijk voor de ecologie en de visserij in de Oosterschelde.

Met het oog op de verwachtte zeespiegelstijging heeft de Deltacommissie in het jaar 2008 gesteld dat de kering na een zeespiegelstijging van 1,0m niet meer veilig functioneert en daarom bij voorkeur verwijderd moet worden. Bovendien veroorzaakt de kering zandhonger in de Oosterschelde waardoor een groot gedeelte van de zandplaten op den duur onder water zal verdwijnen. Dit zal invloed hebben op bodemdieren en vogels die afhankelijk zijn van de zandplaten. Als de kering verwijderd wordt kunnen deze gevolgen (grotendeels) voorkomen worden.

Dit onderzoek

In deze studie is onderzocht wat met het oog op kosten en overstromingsrisico's de beste beslissing is voor de veiligheid van de Oosterschelde. Dit is gedaan door twee alternatieven te analyseren: (1) het behouden van de Oosterscheldekering en (2) het verwijderen van de Oosterscheldkering. In de analyse zijn 3 scenario's voor zeespiegelstijging bekeken: het huidige zeeniveau, een zeespiegelstijging van 0,5m en een zeespiegelstijging van 1,0m. De consequenties van de alternatieven, zoals de benodigde dijkversterking (verhoging en verbreding) en ingrepen aan de stormvloedkering, zijn bepaald en hiervan is een indicatieve kosten-baten analyse gemaakt.

De benodigde veiligheid in het gebied is beschouwd op basis van twee methoden: de huidige norm en een kosten-baten analyse. Volgens de huidige norm zijn de waterkeringen in Zeeland ontworpen om waterstanden (en golven) veilig te weerstaan die gemiddeld eens per 4.000 jaar voorkomen. Bij een kosten-baten analyse, zoals die ook toegepast is in Waterveiligheid 21^e^ eeuw (WV21), wordt de norm en de bijbehorende dijkhoogte gebaseerd op de te beschermen waarden en de kosten.

Dit onderzoek richt zich specifiek op het aspect veiligheid en de kosten en baten van maatregelen die hiermee samenhangen. Aanvullende kosten voor aanpassingen langs de Oosterschelde, bijvoorbeeld het bouwen van keringen voor jachthavens bij het verwijderen van de kering, zijn in deze studie niet beschouwd. Andere baten, zoals ecologie, mobiliteit en recreatie, zijn kwalitatief beschouwd maar geen onderdeel van de kosten-baten analyse.

De resultaten

Op basis van een beschouwing van de ontwerpbelastingen van de Oosterscheldekering en op basis van experts blijkt dat deze een zeespiegelstijging tot 1,0m aankan zonder grote aanpassingen. Het is niet onderzocht of de kering nog functioneert bij een zeespiegelstijging van meer dan 1,0m. De kering en de huidige dijken bieden een grote veiligheid tegen overstromingen rondom de Oosterschelde, vooral omdat de dijken relatief hoog zijn aangelegd in de periode voordat de kering werd gebouwd. De kans dat bij het huidige zeeniveau de dijk bezwijkt door het overslaan van golven is berekend op 1/3.500.000 per jaar. Door het verwijderen van de kering neemt de kans op hoge waterstanden toe in de Oosterschelde en stijgt de overstromingskans tot 1/500 per jaar.

In deze studie zijn twee mechanismen voor het falen van dijken onderzocht: bezwijken door het overslaan van golven (overslag) en bezwijken doordat water onder de dijk stroomt (piping). Dit zijn de belangrijkste faalmechanismen, mede doordat de bekleding van de dijken op dit moment wordt versterkt door Projectbureau Zeeweringen. Maatregelen tegen overslag en piping zijn respectievelijk dijkverhoging en dijkverbreding. Op basis van lokale gegevens is uitgegaan van een representatief dijkprofiel voor de dijken rondom de Oosterschelde.

Naast het verhogen en verbreden van dijken zijn er de kosten van het onderhoud van de kering (20 miljoen euro per jaar) en de kosten van het verwijderen van de kering (geschat op 1 miljard euro). De totale kosten voor de alternatieven zijn bepaald door de Netto Contante Waarde (NCW) te gebruiken. Jaarlijkse kosten in de toekomst worden op die manier vertaald naar de huidige contante waarde. Als voldaan wordt aan de huidige veiligheidsnorm (voor Zeeland een waterstand die eens per 4.000 jaar voorkomt) resulteert dat in dijkversterkingsmaatregelen en kosten zoals te zien in Tabel 1. Zoals in de tabel te zien is leidt elke investering tot een negatieve contante waarde. De investering kost meer dan deze oplevert. Echter, ecologische, visserij en andere baten zijn niet meegenomen in dit onderzoek. Daarom is in de laatste kolom te zien wat de minimaal benodigde baten zijn om op een NCW van nul uit te komen. Bij een NCW van nul of hoger is de investering kosteneffectief.

	zeespiegelstijging	dijkverhoging	lengte piping-maatregelen (dijkverbreding)	Netto Contante Waarde	baten benodigd voor NCW=0
behouden kering	0,0 m	0 m	0 m	-384 M€	20 M€/jaar
	0,5 m	0 m	0 m	-384 M€	20 M€/jaar
	1,0 m	0 m	7 m	-413 M€	22 M€/jaar
verwijderen kering	0,0 m	0,7 m	15 m	-1755 M€	91 M€/jaar
	0,5 m	1,4 m	19 m	-2333 M€	122 M€/jaar
	1,0 m	2,2 m	22 m	-3126 M€	163 M€/jaar

Tabel 1: maatregelen en kosten als voldaan wordt aan de huidige veiligheidsnorm

Het blijkt dat, als de huidige norm gehandhaafd wordt, het behouden van de kering een stuk goedkoper is dan het verwijderen van de kering. Dit komt mede door de grootte van de kosten voor het verwijderen van de kering.

Als de veiligheid wordt gebaseerd op de te beschermen waarden rondom de Oosterschelde, blijkt de optimale overstromingskans ongeveer 1/500 per jaar te zijn. Dit benadert ook de resultaten van Waterveiligheid 21^{e} eeuw (WV21). Doordat de economische waarde en het aantal inwoners rondom de Oosterschelde relatief laag zijn, kan economisch gezien volstaan worden met minder veiligheid. Alleen de optimale dijkhoogte is beschouwd, de optimale dijkbreedte is niet berekend. In principe kan ook de optimale dijkbreedte worden berekend, maar omdat dit de berekening complexer maakt en het resultaat niet significant beinvloed wordt is dit niet gedaan. Als de dijkhoogte wordt gebaseerd op de te beschermen waarden resulteert dat tot dijkversterkingsmaatregelen en kosten zoals te zien in Tabel 2.

	zeespiegelstijging	dijkverhoging	Netto Contante Waarde	baten benodigd voor NCW=0
behouden kering	0,0 m	0 m	-384 M€	20 M€/jaar
	0,5 m	0 m	-384 M€	20 M€/jaar
	1,0 m	0 m	-384 M€	20 M€/jaar
verwijderen kering	0,0 m	0 m	-1259 M€	66 M€/jaar
	0,5 m	0,6 m	-1796 M€	94 M€/jaar
	1,0 m	1,3 m	-2351 M€	123 M€/jaar

Tabel 2: maatregelen en kosten als de veiligheid wordt gebaseerd op de te beschermen waarden

Als de kering wordt verwijderd hoeven de dijken minder hoog te worden als de overstromingskans 1/500 per jaar is. Dit maakt het verschil in kosten tussen het behouden en verwijderen van de kering kleiner, echter, het behouden van de kering blijft goedkoper.

Als de kering behouden blijft zullen de zandplaten op den duur onder water verdwijnen, wat negatieve gevolgen heeft voor de huidige ecologische omstandigheden. Het wordt verwacht dat dit (grotendeels) voorkomen kan worden als de kering weggehaald zal worden. Een maatregel om de zandplaten te behouden als de kering behouden blijft, is het toepassen van zandsuppletie. Als deze kosten (indicatie: 45 miljoen euro per jaar) meegenomen worden blijkt dat het behouden van de kering nog steeds goedkoper is.

Aanbevelingen

Zoals in de tabellen te zien is leidt elke investering tot een negatieve contante waarde. Een investering in veiligheid is kosteneffectief als de reductie in overstromingsrisico groter is dan de kosten voor het verkrijgen van veiligheid. Het huidige overstromingsrisico is echter al laag, dit kan vrijwel niet meer gereduceerd worden. Een maatregel om de kosten te verlagen is om de schuiven van de kering buiten werking te stellen. De kering werkt dan alleen nog als een reductor voor de stormopzet en de golven. De schuiven nemen een groot deel van de onderhoudskosten (20 miljoen euro per jaar) in beslag. Uit deze studie blijkt dat de dijken nog steeds hoog genoeg zijn als de schuiven van de kering niet sluiten tijdens hoog water. De dijken zullen dan wel verbreed moeten worden om de kans op piping te beperken. Het probleem met de zandhonger in de Oosterschelde zal hierdoor niet verdwijnen.

PREFACE

This Master thesis shows whether it is cost-effective to remove the Eastern Scheldt storm surge barrier (Dutch: Oosterscheldekering) in case of sea level rise. The thesis is done for the faculty of Civil Engineering from the TU Delft, and for the engineering consultant Witteveen+Bos.

Although a Master Thesis has to be done independently, a couple of people helped me during the work. First I would like to thank my daily supporters; Maarten Jansen and Bas Jonkman, thanks for the enthusiasm and ideas! Furthermore I want to thank the rest of my thesis commission; prof. Vrijling, Jos Timmermans and Matthijs Boon. Thanks for the reviews and critical notes. I also want to thank Wouter ter Horst, for giving information from FLORIS, and Krijn Saman, for the interesting interview about the Eastern Scheldt Storm surge barrier. This half year study is done at the office at Witteveen+Bos, I want to thank my colleagues for the pleasant stay there.

Willem Leeuwdrent
June 2012

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1. INTRODUCTION

Last decennia a lot of attention has been paid to climate change and its impact on global warming and sea level rise. Models to predict sea level rise are getting better and due to the growing world population and its impact on climate it is expected that the sea level will rise with an increasing speed. Especially low lying countries like the Netherlands are influenced by sea level rise. In 2008 a team of scientists from different research fields (economy, socio-economics, safety and climate) have investigated how the Netherlands have to deal with sea level rise [DELTACOMMISSIE, 2008].

One of the areas in the Netherlands which is influenced more than average by sea level rise is the Southwest Delta (Figure 1). The water level is influenced from the East by the rivers Rhine and Meuse and from the West by the North Sea. This study focuses on decision alternatives for the Eastern Scheldt dealing with sea level rise.

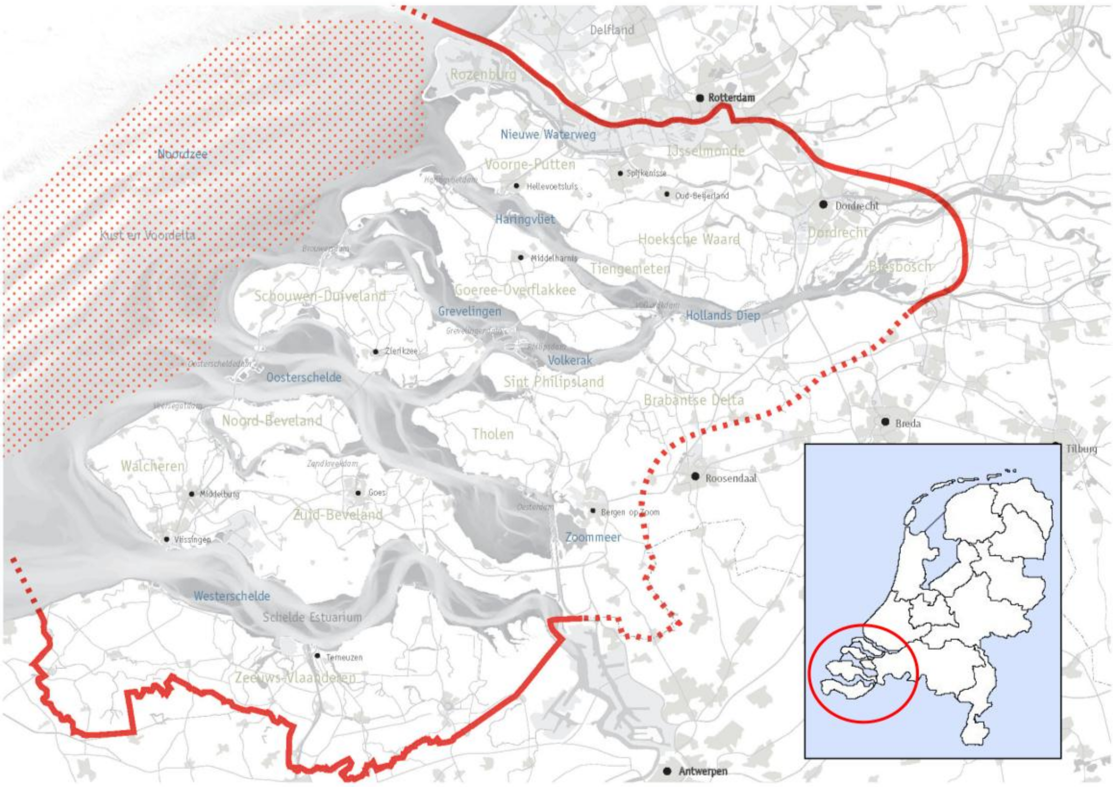

Figure 1: current Southwest Delta [STUURGROEP ZUIDWESTELIJKE DELTA, 2009]

The Eastern Scheldt derived its name from the river Scheldt which ended via the Eastern Scheldt in the sea in the past, see Figure 2. By the influence from sea level rise and floods in the Middle Ages the delta area grew in size. The river Scheldt then used both the Western and the Eastern Scheldt to reach the North Sea. From the 18th century the control of human on the delta grew. Dikes were created which in the end led to the current shape of the area. Nowadays the Eastern Scheldt is not connected to rivers any more.

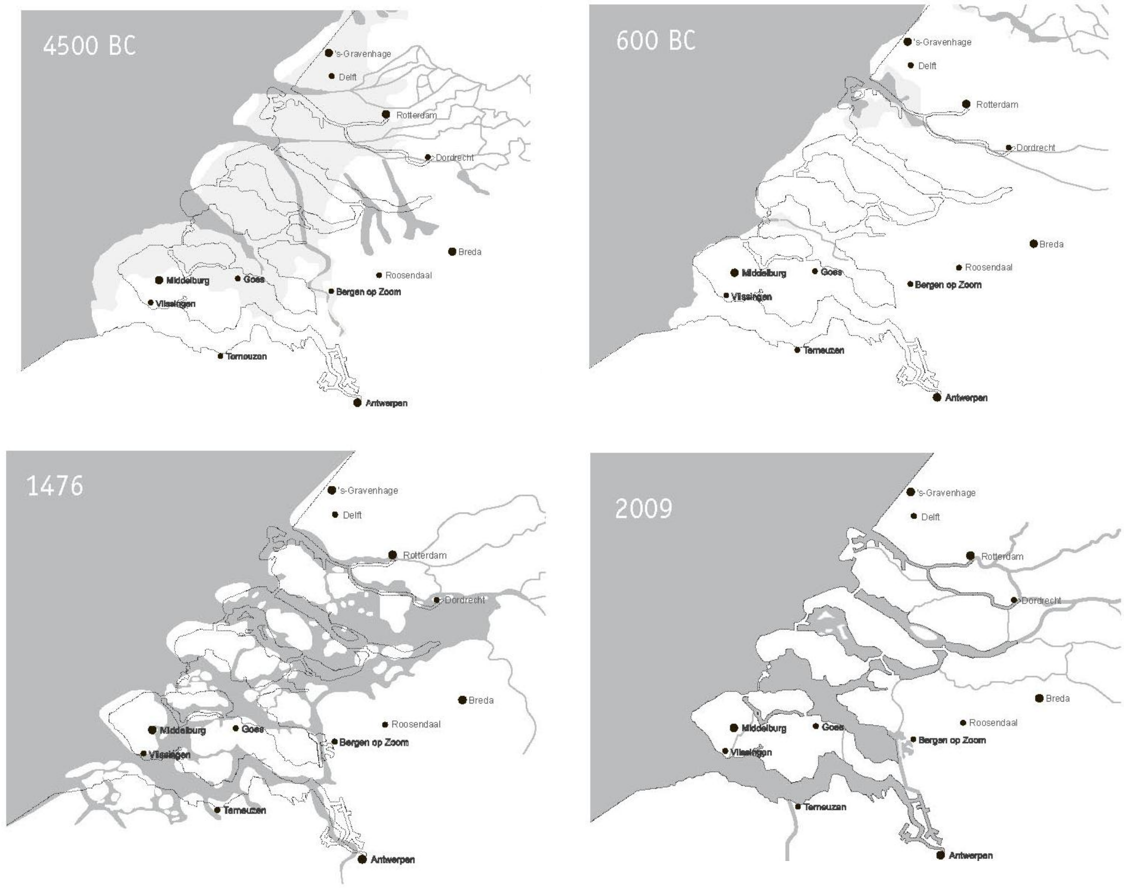

Figure 2: Eastern Scheldt from past till present [STUURGROEP ZUIDWESTELIJKE DELTA, 2009]

The last important influence on the shape of the delta was the flood in 1953. Due to extreme North-Western winds the water levels in the North Sea reached high levels which caused flooding of big parts of the delta. After the flood, plans were made to prevent the Netherlands from floods. To shorten the coastline dams were planned to close of the Haringvliet, Grevelingen and the Eastern Scheldt.

The Eastern Scheldt was planned to be the final closure. However, due to the expected decrease in ecological value and the decrease in fishery, the closure of the Eastern Scheldt was prevented. Instead of making a dam, a storm surge barrier was constructed. During normal conditions the gates of the barrier are open. When a water level of NAP +3,0m at the sea side of the barrier is expected the gates will be closed to prevent the Eastern Scheldt from extreme water levels. The barrier was finished in 1986 [VAN HEEZIK, 2011]. Since then, the probability of flooding became at least 1/4.000 year.

Since 1986 the barrier has closed 24 times because of storm surges. The last closure is almost five years ago; the 9th of November 2007[1]. The closure level of NAP +3,0m implies that the barrier has to close once per year from a statistically point of view. This corresponds quite well with the number of closures since 1986. Nowadays the Eastern Scheldt is still a tidal basin, in contrary to the Grevelingen and Haringvliet. However, the tidal volume

[1] www.hmcz.nl (assessed at 4-5-2012)

has decreased with approximately 30% since the construction of the barrier [NIENHUIS AND SMAAL, 1994]. Also the construction of a compartmentalization dam contributes to the decrease in tidal volume. The decrease in tidal volume influences the tidal flats in the Eastern Scheldt. Because of the decrease in tidal volume, the tidal velocity has also decreased. This causes erosion of the tidal flats. It is expected that in 2060 most of the tidal flats has disappeared [JACOBSE et al, 2008].

Due to the expected sea level rise the question arises to which extend the safety of the dike-rings around the Eastern Scheldt can be guaranteed by the barrier and the dikes. Possibly adaptations on the barrier have to be made, or possibly the barrier will have to be removed and replaced by another one, or the safety may have to be guaranteed by the heightening of dikes. This study focuses on decisions alternatives for the safety of the Eastern Scheldt in case of sea level rise.

1.1. Problem description

There are two motivations to study the decision possibilities for the Eastern Scheldt: the influence from sea level rise and the ecological influence. Sea level rise will affect the safety of the dike-rings around the Eastern Scheldt and due to the presence of the storm surge barrier the ecological value of the Eastern Scheldt is decreasing.

1.1.1. Sea level rise

According to [DELTACOMMISSIE, 2008] the Eastern Scheldt storm surge barrier can withstand a sea level rise of 0,5 meter and by updating the barrier a maximum sea level rise of 1,0 meter. The Deltacommission poses that a sea level rise of 1,0 meter will occur in 2075 at the earliest. In order to have the ability to use the full range of measures, choices have to be made a few decennia on forehand, say 2030-2040.

But is sea level rise a serious threat for the storm surge barrier when looking to 2100 for instance? In Figure 3 the historic monthly mean sea level for Oostende (Belgium) is shown. A linear fit through the data shows a mean sea level rise of 1,77 millimetres/year with a 95% confidence interval of +/- 0,26 mm/year based on data from 1937 to 2009. So the sea level is currently rising with 18 cm/century, with confidence bounds of 15 cm/century and 20 cm/century.

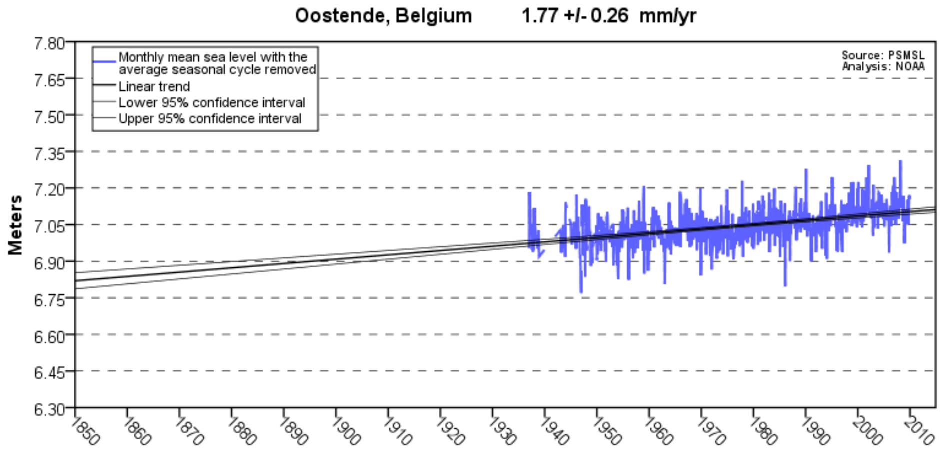

Figure 3: Mean Sea Level in 20th century for Oostende (Belgium)[1]

An organisation which evaluates the impact of climate change is IPCC (Intergovernmental Panel on Climate Change). It is a group of hundreds of independent experts from all over the world who reviews and assesses technical and socio-economic information about climate change produced worldwide. About every five years it publishes reports with their findings on the state of knowledge on climate change. The latest one is 'Climate Change 2007', the Fourth IPCC Assessment Report. The fifth one will be published in 2013. The expected sea level rise presented by IPCC (MEEHL *et al*, 2007) is shown in Figure 4. This is global sea level rise, not specified per location. Different scenarios (B1, B2, A1, A2) for sea level rise are calculated. These scenarios differ in economic growth and emission.

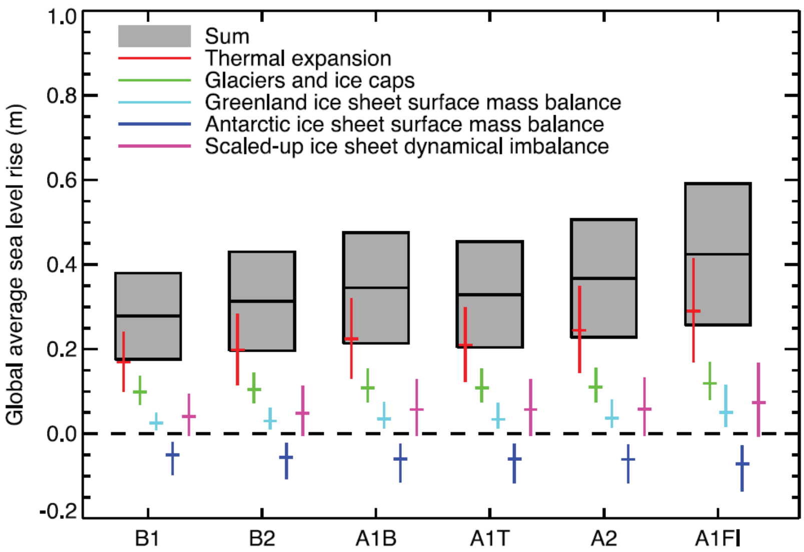

Figure 4: predicted global average sea level rise for 2100 by IPCC (MEEHL *et al*, 2007) page 821. The scenarios differ in amount of economic growth and emission.

The Koninklijk Nederlands Meteorologisch Instituut (KNMI) had done model studies for sea level rise with the focus on the Netherlands (VAN DEN HURK *et al*, 2006). The scenarios which are used by KNMI are different from the scenarios used by IPCC. However, scenar-

[1] Source: http://tidesandcurrents.noaa.gov

ios G and G+ resemble B1 and B2 and W and W+ resemble A1(FI) and A2. The KNMI scenarios predict more sea level rise than the scenarios from IPCC do. According to KNMI this difference is caused by regional effects. The results are shown in Figure 5.

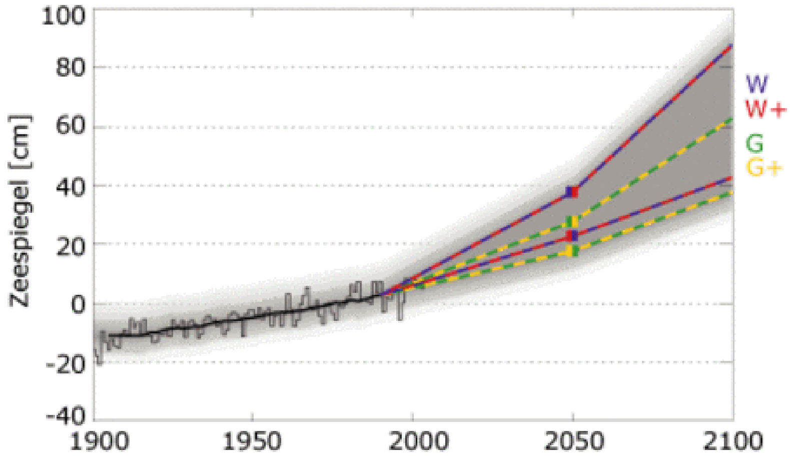

Figure 5: predicted sea level rise by KNMI (VAN DEN HURK *et al*, 2006)

By order of [DELTACOMMISSIE, 2008] a team of researchers have also investigated sea level rise for the Netherlands. They had to investigate maximum scenarios. The result of this research is shown in Figure 6. The differences with the KNMI scenarios are caused by the assumptions. Maximum values for CO_2 emission and rise of temperature are used. According to [DELTACOMMISSIE, 2008] it is not likely that the scenarios will be exceeded.

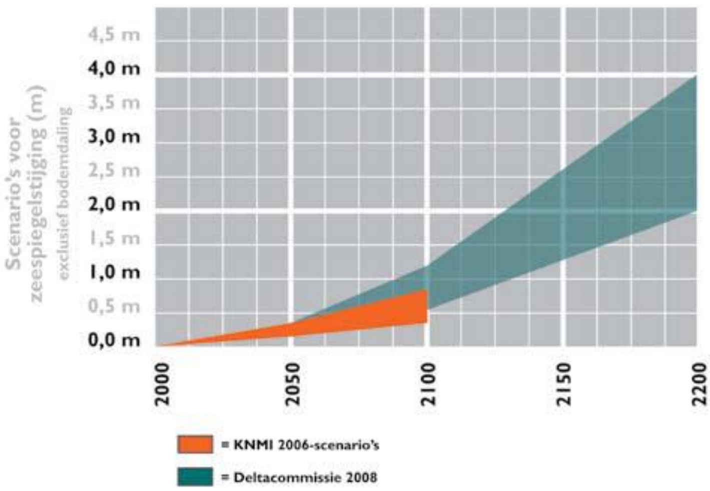

Figure 6: predicted sea level rise by the Deltacommision [DELTACOMMISSIE, 2008]

According to the KNMI scenarios the expected sea level rise in 2100 varies between 0,4 and 0,85m. The maximum sea level rise in 2100 according to [DELTACOMMISSIE, 2008] differs between 0,55 and 1,20m. According to IPCC (MEEHL *et al*, 2007) the global sea level rise in 2100 differs between 0,18 and 0,55m. A sea level rise of 1,0m in 2075 which is calculated by [DELTACOMMISSIE, 2008] is a worst case scenario.

1.1.2. Ecology

The other reason for this study is the influence from the barrier on ecology. Before the construction of the barrier the Eastern Scheldt was a sediment exporting basin [TANCZOS *et al,* 2001], the Eastern Scheldt contained too much sand. In 1987 the construction of the barrier was completed. Also the Markiezaatdam was completed. The Eastern Scheldt then became a sedimentation basin; the channels are too wide for the tidal volume. However, due to the construction of the barrier sand transport is blocked. Sand has to come from inside the basin, which causes eroding of the intertidal flats. According to [JACOBSE et al, 2008] in 2060 most of the tidal flats will be disappeared. Only in the east part of the Eastern Scheldt tidal flats will be maintained. In Table 3 some characteristics before and directly after the construction of the barrier are shown.

	Pre-barrier	Post-barrier
Total surface [km^2]	452	351
Water surface, MWL [km^2]	362	304
Tidal flats [km^2]	183	118
Salt marshes [km^2]	172	6,4
Cross section barrier in open position [m^2]	80.000	17.900
Mean tidal range, Yerseke [m]	3,70	3,25
Max. flow velocity [m/s]	1,5	1,0
Residence time [days]	5-50	10-150
Mean tidal volume [m^3 x 10^6]	1230	880
Total volume [m^3 x 10^6]	3050	2750
Mean freshwater load [m^3/s]	70	25

Table 3: changes in the Eastern Scheldt after construction of the barrier and compartmentalization dams [NIENHUIS AND SMAAL, 1994]

In Figure 7 the decrease in intertidal area between 1983 and 2001 is visible. Around NAP the decrease is about 30% [GEURTS VAN KESSEL, 2004]. It is also visible that the sand is transported to the tidal channels; the area of deeper parts is growing.

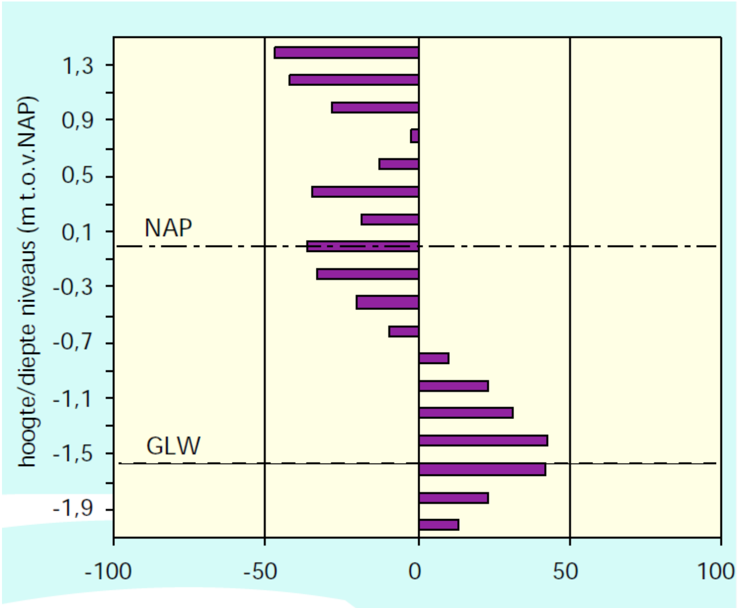

Figure 7: Percentage of decrease in intertidal area between 1983 and 2001 for different levels [GEURTS VAN KESSEL, 2004]

It is expected that the erosion of the tidal flats will have an effect on subterranean animals and birds [GEURTS VAN KESSEL, 2004]. The foraging time for birds decreases due to the decrease in duration that the flats are above the water level. Changes in the population of oysters and oystercatchers were visible; however, this was caused by the oyster-fishery and hard winters. Also changes in population of other birds were not visible. Also [VAN ZANTEN AND ADRIAANSE, 2008] investigated the influence from erosion of the flats on ecology. They also predict the decrease in birds, like the oystercatcher.

When the barrier will be removed, sand exchange with the outer delta can take place, so sand does not have to come from the tidal flats in the Eastern Scheldt. This lowers the decrease of the tidal flats.

1.2. Objective of this study

Taken into account the problems outlined in the problem description the aim of this study is to investigate the best investment option for the safety of the dike-rings around the Eastern Scheldt when dealing with sea level rise. In this study the problems with the decreasing tidal flats in the Eastern Scheldt are taken into account by defining an alternative which will (partly) prevent further erosion of the flats.

1.3. Alternatives

In this study two alternatives are investigated; to maintain the barrier and to remove the barrier.

1.3.1. Maintain the barrier

Maintaining the barrier is the reference alternative. The current situation will be maintained until it can no longer withstand sea level rise. It is investigated to which extend the barrier can withstand sea level rise. Also possible adaptations on the barrier are investigated. When the safety of flooding appeared to be too low, the amount of dike reinforcement is calculated.

In this option the disappearance of tidal flats will continue. Most of the tidal flats are expected to erode and the sand will deposit into the tidal channels. Only in the east part of the basin tidal flats will remain [JACOBSE et al, 2008].

1.3.2. Remove the barrier

The second alternative is to remove the barrier. For the intertidal flats removing the barrier is the preference option. The erosion of the flats can be (partly) prevented. This is important for instance for subterranean animals and birds. The safety in this alternative can be guaranteed by the dikes. However, dikes logically have to be higher in the case that the barrier will be removed.

For the alternatives it has to be noted that sea level rise is a process which plays a role in the (very) future but the disappearance of flats is nowadays an issue. So from an ecological point of view it is better to remove the barrier now, but the sea level may influence the barrier in the very future.

2. FRAMEWORK

2.1. System description

To determine the flood risk of the Eastern Scheldt and its surroundings a conceptual model is set up which is a simplification of the real area. The layout of the conceptual model is shown in Figure 8.

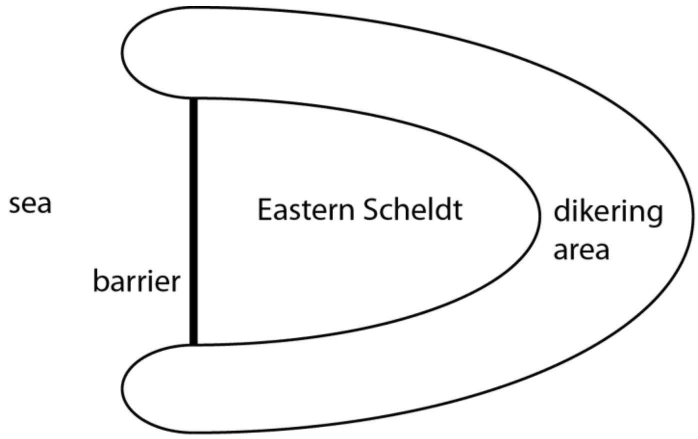

Figure 8: conceptual model Eastern Scheldt

The safety of the dike-ring is guaranteed by two components: the closable barrier, which safes the Eastern Scheldt from high water levels, and the dike, which safes the dike-ring area from floods.

The closable barrier safeguards the tidal basin behind it from extreme water levels and waves from the sea. During normal conditions the tide and the wind waves from the sea can penetrate into the tidal basin. During high water the barrier closes. Only little discharge through the barrier will take place during closure by leakage and overtopping. The basin then acts as a lake. All waves are internally generated and the waterlevel is almost constant. The barrier is shown in Figure 9.

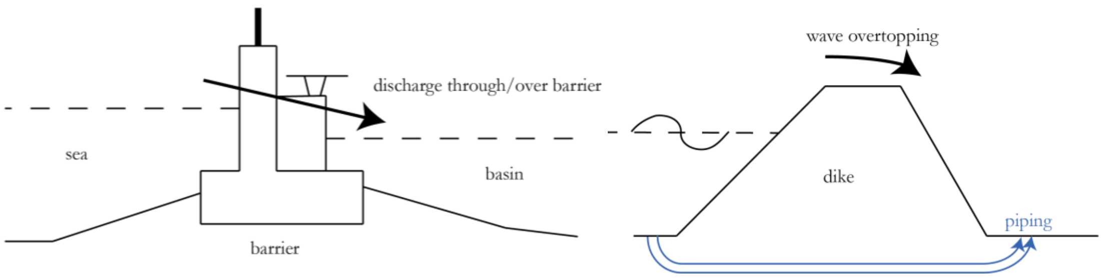

Figure 9: closable barrier and dike

The second safety component is the dike located along the tidal basin (Figure 9). The dike protects the polder behind it from flooding. The dike fails when there is a breach at a certain spot along the dike.

The values which have to be protected by the barrier and the dikes are the cities and villages, the infrastructure, the industry, the agriculture and the inhabitants inside the dike-ring area. Significant parts of the polders lay below NAP, so a breach will usually inundate a big part of the polder.

2.2. Determining safety

The safety of flooding for dike-rings around the Eastern Scheldt can be calculated by determining the flooding probability of the dikes. For that calculation the mechanisms of failure of dikes have to be known and the safety standards on which the strength of the dike has to be based. These are explained in this paragraph.

2.2.1. Failure mechanisms of dikes

Dikes can fail by many mechanisms, for instance overtopping, piping, the instability of slopes and erosion of the outer slope, see Figure 10. In Zeeland landslide was an important failure mechanism in the past [MINISTERIE VAN RIJKSWATERSTAAT, 1973]. In this study overtopping and piping are investigated. This is done because from FLORIS (dutch: Veiligheid Nederland in Kaart, VNK) [VNK, 2011] it appeared that overtopping and piping are currently the main failure mechanisms for dikes around the Eastern Scheldt. Erosion of the dike revetment is not relevant because the dike revetment is currently improved by 'Projectbureau Zeeweringen'[1].

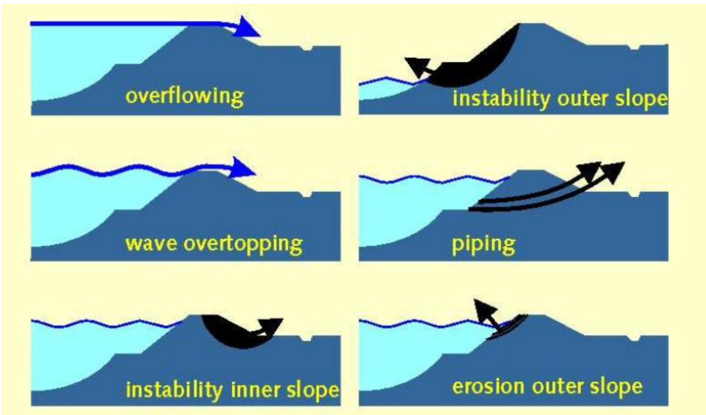

Figure 10: failure mechanisms dikes

2.2.2. Determining the reliability

A couple of formulas are available for calculating the reliability of a dike. For overtopping the formula 'PC-overtopping' [PULLEN *et al*, 2007] is mostly used in the Netherlands. For piping the formula of Sellmeijer [TAW, 1999b] is mostly used. Also in this study these formulas are used.

For calculating the reliability of the dike a couple of input parameters are given a distribution. The reliability can be calculated using several computational methods. For this study the First Order Reliability Method (FORM) [CUR 190, 1997] is used.

A typical result for a reliability calculation for overtopping is shown in Figure 11. The reliability is calculated for a range of loads (water levels). Calculating this way the output is a so called fragility curve; see Figure 11, first graph. The fragility curve is the cumulative distri-

[1] http://www.zeeweringen.nl/

bution function (CDF) of the failure probability, given a certain water level. The water level also has a distribution, see Figure 11, second graph. The failure probability per year (Figure 11, third graph) can be calculated using the convolution integral [CUR 190, 1997]:

$$P_f = P(S > R) = \int_{-\infty}^{\infty} \left(1 - F_S(R)\right) f_R(R) dR \qquad (2.1)$$

Where:

P_f	=	failure probability per year
$(1 - F_S(R))$	=	exceedance probability of the (hydraulic) load related to the water level
$f_R(R)$	=	probability density function of the resistance related to the water level

In the calculation the fragility curve is transformed to a probability density function by taking its derivative. In the fourth graph of Figure 11 the dike profile and parameters are shown.

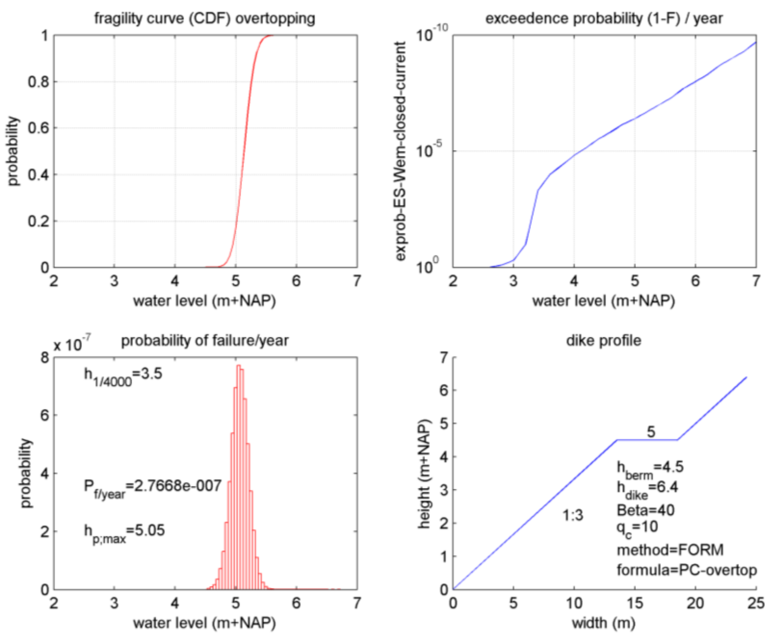

Figure 11: typical result from a FORM calculation

2.2.3. Safety standards

The safety will be assessed based on two principles, the current safety standards and a cost-benefit-analysis.

Currently the safety standard for flooding in the Netherlands is based on a fixed standard. This safety standard is a hydraulic load with a certain probability of exceedance which the flood defence has to withstand. The basis for this safety standard is a cost-benefit analysis done in the fifties by [VAN DANTZIG AND KRIENS, 1960] for Centraal-Holland. In that study the economic value in the area is determined in relation to the costs for a higher protection level. The principle is shown in Figure 12.

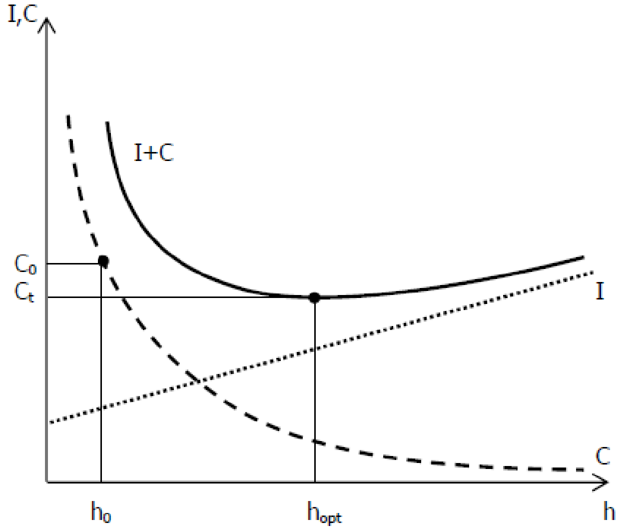

Figure 12: economic optimal dike height [CUR 190, 1997]

Figure 12 shows when the dike becomes higher, the consequence of a flood (C) will decrease, but the investment in dike heightening (I) will increase. At a certain point an optimum is reached (h_{opt}). [VAN DANTZIG AND KRIENS, 1960] found for Centraal-Holland an optimal probability of flooding of 1/125.000 year. According to [Deltacommissie, 1960] this flooding probability matched to an exceedance probability of the hydraulic load of 1/10.000 year. This difference is caused by extra safety which is always present by the design of flood defences. Based on the optimal safety standard for Centraal-Holland the standards for other flood-prone areas are determined. For Zeeland this standard is set to a water level with an exceedance probability of 1/4.000 year. Actually this method is based on a cost-benefit analysis for Centraal-Holland but in the end it became more a political decision for other parts of the Netherlands.

In the project 'Flood protection 21st century' (Dutch: Waterveiligheid 21e eeuw (WV21), [DELTARES, 2011]) research is done to determine safety levels based on a cost-benefit-analysis for each dike-ring area. Since the fifties insight in the consequences and probabilities of flooding has greatly increased, so the analysis can be done with a much higher accuracy. The concept is more detailed as used by [VAN DANTZIG AND KRIENS, 1960]. In the project WV21 a model is made which calculates the optimum time of investment as well as the amount of investment.

Results for the Southwest Delta area of the Netherlands are shown in Table 4. The optimum flooding probabilities are determined for the year 2050. In the model economic growth and sea level rise are taken into account. For sea level rise the scenario W+ from KNMI (VAN DEN HURK *et al*, 2006) is used, see Section 1.1.1. This means that a sea level rise of 35 cm between 2015 and 2050 is taken into account. The optimum probabilities are average values: if the probabilities are exceeded there is still time to invest for more safety.

The optimum flooding probability for the year 2050 for the dike-rings around the Eastern Scheldt are shown in Table 4.

	opt. flooding probability
dike-ring 26_1	1/2400
dike-ring 26_2	1/3100
dike-ring 27_1	1/1600
dike-ring 28_1	1/800
dike-ring 30_1	1/700
dike-ring 31_1	1/1100

Table 4: optimum flooding probabilities of dikerings around the Eastern Scheldt for the year 2050, found by WV21 [DELTARES, 2011]

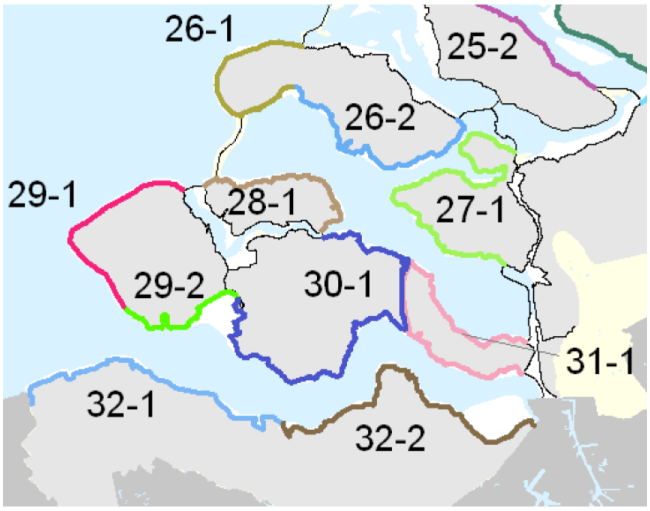

Figure 13: location of the dike-rings

Probably the safety standards in the Netherlands will change in the future and based on a cost-benefit-analysis. The question rises on which criterion the decision on the safety of the Eastern Scheldt will be made. For completeness both criteria (the current safety standard and a cost-benefit analysis) will be investigated. The economic optimum failure probabilities shown in Table 4 are not used because these probabilities are only valid for the year 2050. Instead of that the optimum probabilities are found by applying the method shown in Figure 12.

The water level which have been withstanding by the flood defences is legally regulated. This water level can be used by calculating overtopping, but for other failure mechanisms (e.g. piping or failure of revetment) no legally standards are determined. In this study the recommendations from [TAW, 1999a] are used:

$$P_f \text{ (other mechanisms than overtopping)} = 0{,}1 \cdot \text{current safety standard} \tag{2.2}$$

It proposes a standard of 0,1 times the standard for other failure mechanisms than over-topping. This means that the probability of failure for e.g. piping may not exceed 1/40.000 year for dike-rings in Zeeland.

2.3. Criteria for decision

To be able to evaluate the different alternatives, criteria have to be defined. The best decision option will be chosen with respect to costs. There can be different kind of costs: cost of heightening of dikes, maintenance costs of the barrier and costs of removing the barrier. There can also be benefits: increase of ecological value and avoided risk. The monetary value of ecology is not taken into account in this study.

The costs are determined using the Net Present Value (NPV). The NPV takes into account the time value of money and is mostly used when calculating the costs and benefits from an investment. When the investment is done now but most of the benefits will be earned much later, the value of the benefits has to be expressed in the current value. The Net Present Value can be described by the following formula [EIJGENRAAM *et al*, 2000]:

$$NPV = \sum_{t=0}^{T_j} \frac{B_t - C_t}{(1+r)^t} \qquad (2.3)$$

t	= time	[year]
T	= time horizon project	[year]
r	= discount rate	[-]
B_t	= benefits in year t	[€]
C_t	= costs in year t	[€]

For a positive Net Present Value the investment leads to an economically improvement. In this study a discount rate of 5,5% is taken (real and risk aversive). 'Real' means that the inflation is taken into account and 'risk aversive' means that uncertainties such as climate change and macro-economic risk is taken into account. This is currently prescribed by the Dutch governments for cost-benefit analysis[1]. An infinite time horizon is chosen.

It is possible that economic productivity around the Eastern Scheldt will grow and that the population will grow. In [PROVINCIE ZEELAND, 2008] expectations about the population in the province Zeeland are described and its consequences on economy. It is expected that the population will be stable till 2025 and will decrease after 2025. The consequences on economy are hard to predict. It is possible that due to innovation the productivity may grow, despite the decreasing population. In this study no economic growth is taken into account.

The prices used in this study are prices for 2012. The costs which are found in this study are recent, so the costs are not converted.

[1] Letter to the Dutch government, 8 March 2007, doc nr 29 352, nr. 3.

2.4. Calculation procedure

The impact from sea level rise on the safety of the dike-rings around the Eastern Scheldt will be investigated for three scenarios:

- No sea level rise
- A sea level rise of 0,5 meter
- A sea level rise of 1,0 meter

For these amounts of sea level rise it is investigated what the best decision option is when this sea level rise has appeared. No reference is made to years because of the uncertainty of the amount of sea level rise. This is done for the situation that the barrier will be maintained and the situation that the barrier will be removed. The procedure is shown in Figure 14.

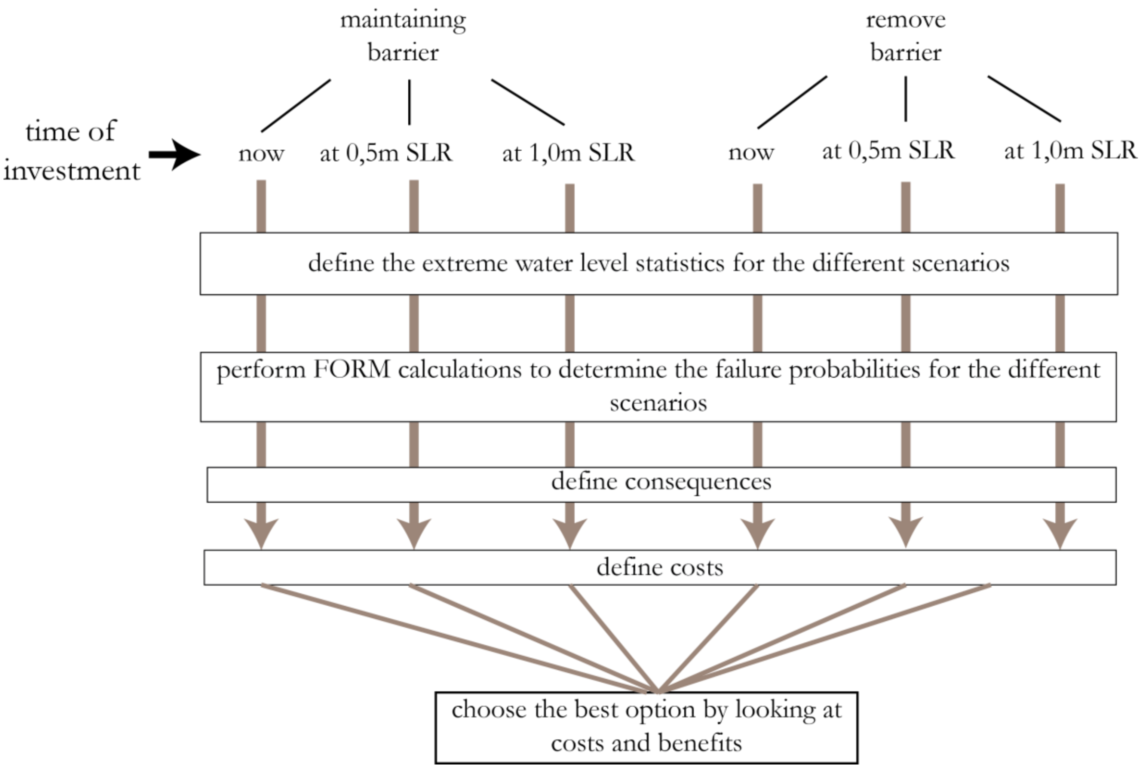

Figure 14: calculation procedure

First the probability of failure is calculated by defining the extreme water level statistics in the Eastern Scheldt and defining the different parameters of the dike. Then the consequences and the costs of the different alternatives and scenarios are calculated. When the probability of failure is known and the consequences are known the risk can be calculated by:

$$risk = probability \times consequences \tag{2.4}$$

To determine the total costs of an alternative, the risk has to be added to the investment costs.

3. LIFETIME OF THE BARRIER

To calculate the costs of the safety of dike-rings around the Eastern Scheldt, the lifetime of the barrier in relation to sea level rise has to be known. The barrier consists of replaceable and irreplaceable parts. The replaceable parts are the dynamic parts such as the gates and the cylinders. The irreplaceable parts are the foundation, the pillars, the under and upper sill and the threshold [RIJKSWATERSTAAT, 1985a], see Figure 15 for the different parts.

The irreplaceable parts are designed for a lifetime of 200 years. The replaceable parts were constructed for a lifetime of at least 50 years [RIJKSWATERSTAAT, 1985d].

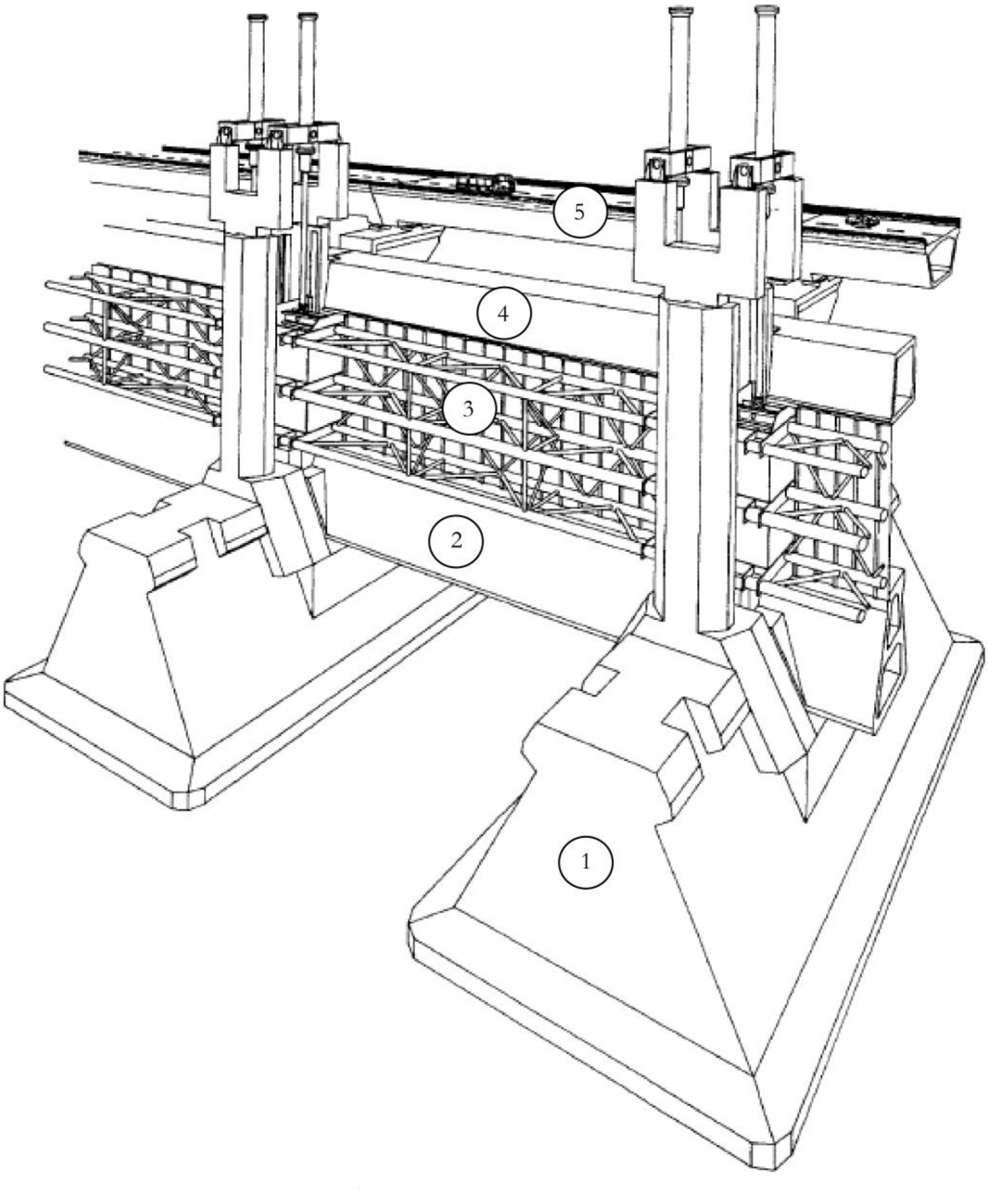

Figure 15: main parts storm surge barrier: 1. Pillar, 2. Lower sill, 3. Gate, 4. Upper sill, 5. Bridge structure [RIJKSWATERSTAAT, 1985d]

3.1. History of the design

Originally a dam was planned to be built [DELTACOMMISSIE, 1960a]. The construction of the dam was already started in 1967. The work islands Roggenplaat, Neeltje Jans and Noordland were created by heightening the area with sand. The bottom protection for Hammen and Roompot was under construction. The pillars for the cableway were installed and were about to be ready to drop stones on top of the bottom protection to build the dam. In 1978 the dam was planned to be finished.
However, in July 1974 the construction of the dam was stopped. From studies and from practice from other closures (Grevelingen and Haringvliet) it appeared that closing a basin decreased the ecological value of the closed area. In the sixties the social awareness for ecology had grown and ecologists successfully join hands with the fishery to protest against the dam [VAN HEEZIK, 2011].

Several alternatives were investigated for the dam, for instance a porous dam, caissons in combination with a storm surge barrier and heightening of the dikes. A porous dam appeared to be too unstable. Heightening of dikes was not an option because of the duration of the project (15 -20 years), the costs and the relatively high probability of failure of dikes [COMMISSIE OOSTERSCHELDE, 1974]. In November 1974 the Dutch government decided for the combination of caissons with a storm surge barrier, which had to be finished in 1985. At the same time the dikes had to be heightened to a safety level of 1/500 year. The dike heightening was needed to guarantee safety for the period of the construction of the dam. It took place from 1975-1980. The dike heightening was in contradiction with the conclusion from the Deltacommissie 1960 [DELTACOMMISSIE, 1960a], and the conclusion from Commissie Oosterschelde [COMMISSIE OOSTERSCHELDE, 1974] that dike heightening would cost too much time and would be too expensive.

After two and a half years of study the design of the barrier was finished. The combination of caissons and a barrier appeared to be impossible when looking at costs and construction time. Instead of that only a storm surge barrier was designed. A closable structure was designed to withstand storm surges, but maintained the tide during normal conditions. The design of the barrier is shown in Figure 15. In 1977 the construction of the storm surge barrier was started and on the fourth of October 1986 it was finished.

3.2. Design of the barrier

An important design criterion for the barrier was the maximum allowed water level in the Eastern Scheldt. An average water level of NAP +4,3m was not allowed to appear more than 1/4000 year [RIJKSWATERSTAAT, 1985a]. For the different parts of the barrier more specific criteria are defined, such as the maximum hydraulic head between the North Sea and the Eastern Scheldt, the wave load, the velocity of water, the velocity of wind etc.

The barrier has three parts: Hammen, Schaar and Roompot, see Figure 16. In Figure 17 the cross section of one of the three inlets is shown. In the figure the original bottom profile is shown as well as the reduced opening after construction of the barrier. The total profile decreased from 80.000 m^2 to 15.500m^2 [RIJKSWATERSTAAT, 1985a]. To maintain the same bottom profile as in the situation before the construction, the gates are designed to follow the original bathymetry. The height of the gates varies between 5,9 and 11,9m.

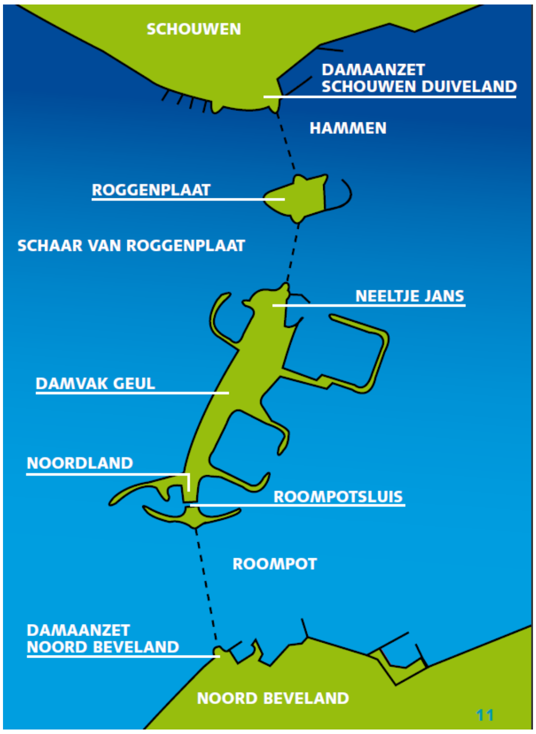

Figure 16: location of the barrier

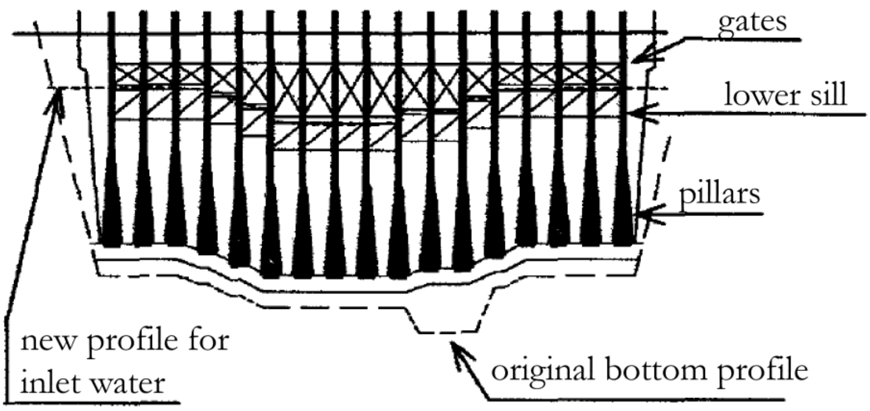

Figure 17: original and new profile [RIJKSWATERSTAAT, 1985a]

During normal conditions the gates of the barrier are open. When a water level of NAP+3,0m is expected, the gates will be closed. The water level in the Eastern Scheldt will become approximately stagnant after closure of the barrier. Only by leakage between the gates and the sill (Figure 18), leakage through the porous threshold and overtopping, the water level rises during closure. The total leakage area μA is currently calculated at 1.250m^2.

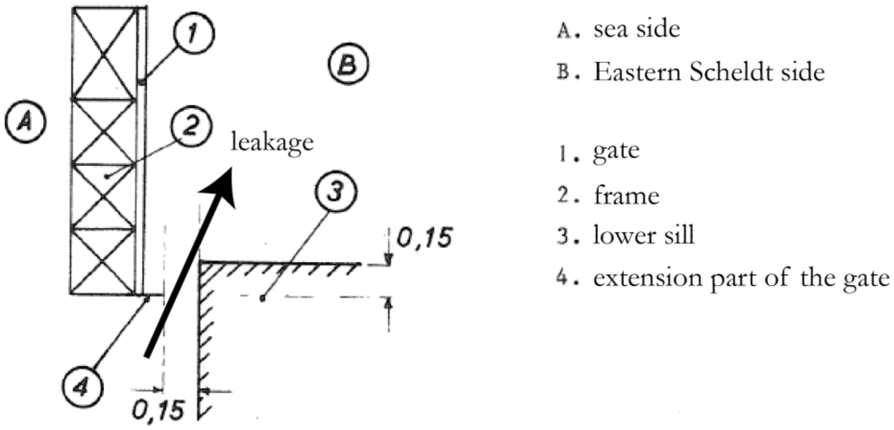

Figure 18: opening in barrier between gates and sill (source: DUSSELDORP AND OLDENZIEL, 1978)

The retaining height of the barrier is determined by the upper sill. The upper sill has a height of NAP+5,8m at Roompot and Schaar and a height of NAP+5,6m at Hammen [RIJKSWATERSTAAT, 1985c]. The design wave height [H_s] for the irreplaceable parts was determined for Roompot at 5,3m and for Schaar and Hammen at 4,1m. The corresponding wave period [T_p] was determined at 9,5 seconds at each location. So for the 1/4000 year water level significant overtopping was taken into account.

The total costs of the barrier were 8,5 billion guilders, with a price level for the year 1994 [CUR 212, 2003]. When calculating with an average inflation of 2% per year, the costs of the barrier in 2012 are about 5,5 billion euro.

3.3. Impact sea level rise on structure

The influence of sea level rise on the structure is investigated by comparing the design loads with the current loads. Information about the design of the barrier is mostly obtained from [RIJKSWATERSTAAT, 1985a,b,c,d]. Only extreme loads are investigated, it is expected that the daily loads are not normative for the safety. The safety of other parts of the structure (e.g. the islands Neeltje Jans and Roggenplaat) are not investigated. It is assumed that the costs of adaptation of these parts are relatively low comparing with the costs of adapting the barrier.

The most important parameters for the safety of the barrier when dealing with sea level rise are the maximum allowed water level at the North Sea and the Eastern Scheldt and the wave attack in the situation the gates are closed. Parts of the barrier which are influenced by the waves and the water level are:

- Pillars
- Upper sill
- Lower sill
- Threshold
- Gates
- Bridge structure
- Bed protection

All these parts are designed based on an extreme water level of NAP +5,5m at the North Sea with a corresponding water level at the Eastern Scheldt of NAP -0,7m, so a total head difference of 6,2m. These values were based on a return period of 1/4000 year. In the design a sea level rise of 0,3m was taken into account [RIJKSWATERSTAAT, 1985a]. For that reason the structure height of the upper sill was designed at NAP +5,8m. During the construction of the barrier it appeared that it was better to lower the foundation depth for Hammen. The final structure height for Hammen became NAP +5,6m.

Currently the 1/4000 year water level at the sea side of the barrier is NAP +5,2m, calculated by HR2006 [MINISTERIE VAN VERKEER EN WATERSTAAT *et al*, 2007]. This means that for Roompot and Schaar 0,6m sea level rise is acceptable according to the design of the structure and for Hammen a sea level rise of 0,4m is acceptable. It is likely that the barrier can deal with a sea level rise of 0,5m. The influence from a sea level rise of 1,0m on the structure is elaborated in the following sections.

3.3.1. Gates

The design wave heights for the gates according to [RIJKSWATERSTAAT, 1985d] and the current 1/4000 year wave loads according to HR2006 are shown in Table 5. The spectral wave period $T_{m-1,0}$ can be transformed to the peak period by $T_p/T_{m-1,0}$=1,1. As can be seen quite a lot reserve is currently present.

	design conditions		current conditions (HR2006)		
location	design wave H_s	design period T_p	H_s	$T_{m-1,0}$	β
Hammen	4,1-4,25m	9,5s	2,6m	5,8s	10°
Schaar	4,1-4,25m	9,5s	3,0m	6,1s	20°
Roompot	5,6-5,75m	9,5s	3,2m	6,0s	20°

Table 5: wave loads on the gates

At a sea level rise of 1,0m the 1/4000 year water level at the sea side becomes NAP +6,2m. In the design a water level of NAP +5,8m is accounted for. At the Eastern Scheldt side the design water level is NAP -0,7m. Currently the water level is at least NAP +1,0m at extreme conditions.

The gates are designed for a lifetime of at least 50 years. It is expected that the gates have to be replaced before sea level rise will cause problems. In the new design the influence from 1,0m sea level rise can be taken into account.

3.3.2. Upper and lower sill

The design wave loads for the upper sill [RIJKSWATERSTAAT, 1985c] and the current 1/4000 year loads according to HR2006 are shown in Table 6. The HR2006 wave loads are given for a water level of NAP +5,2m and the design wave loads are based on a water level of NAP +5,5m.

	design conditions		current conditions (HR2006)		
location	design wave H_s	design period T_p	H_s	$T_{m-1,0}$	β
Hammen	4,1m	9,5s	2,6m	5,8s	10°
Schaar	4,1m	9,5s	3,0m	6,1s	20°
Roompot	5,3m	9,5s	3,2m	6,0s	20°

Table 6: wave loads on the upper sill

As can be seen quite a lot reserve on wave load is present. However, at a sea level rise of 1,0m the 1/4000 year water level becomes approximately NAP +6,2m. The waves will increase due to the increased water depth. A rule of thumb is that the waves will increase with half the increase of waterdepth. The wave height for Hammen, Schaar and Roompot becomes approximately 3,1m, 3,5m and 3,7m in the case of 1,0m sea level rise. The wave impact is still acceptable in that case.

For the lower sill waves are not relevant because the upper side of the lower sill is placed at varying levels of NAP -10,5 to NAP -4,5m [RIJKSWATERSTAAT, 1985d].

The 1/4000 year water level at the sea side of the barrier for a sea level rise of 1,0m becomes NAP +6,2m. This is somewhat higher than is accounted for in the design (NAP +5,8m). At the Eastern Scheldt side quite a lot reserve is present. Currently the water level is at least NAP +1,0m at extreme conditions. The design water level at the Eastern Scheldt side is NAP -0,7m. Because of that it is expected that the upper and lower sill can deal with a sea level rise of 1,0m.

3.3.3. Bridge structure

The height of the bridge structure is designed at NAP +8,0m in order to avoid the impact from waves on the structure [RIJKSWATERSTAAT, 1985c], see Figure 19. When the sea level is rising and waves will increase this can be a problem. It is not investigated in this study what the impact from increasing wave load and water level is.

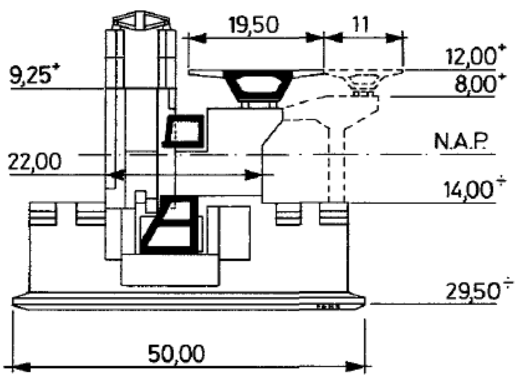

Figure 19: position of the bridge structure

3.3.4. Pillars

For the pillars it is expected that 1,0m of sea level rise is not a problem for the structure. The pillars are also based on an Eastern Scheldt water level of NAP -0,7m. The current closing regime is expected to provide enough safety to withstand a sea level rise of 1,0m. From [RIJKSWATERSTAAT, 1985c] it is not clear which wave loads are used in the design, however, it is expected that the same wave loads were used for the pillars as for the upper sill. For that reason the wave load is also expected to be not a problem in case of 1,0m sea level rise.

3.3.5. Threshold and bed protection

The threshold consists of several layers of rock, which provide for a fluent flow of water through the barrier, see Figure 20. It also blocks water to flow below the lower sill. The threshold will fail when the upper layer of rock will fail. The biggest blocks are situated at the Eastern Scheldt side of the barrier, the maximum weight of the blocks is 6-10 tons. The threshold will fail when extreme velocities will occur, for instance when a gate does not to close during extreme water levels. According to the design a maximum water level difference of 6,2m with corresponding flow velocities is acceptable [RIJKSWATERSTAAT, 1985b]. Sea level rise can influence the difference in water level between North Sea and Eastern Scheldt, however, it is expected that the closing regime is adapted at a sea level rise of 1,0m (see Section 3.4). The water level in the Eastern Scheldt is held at a higher level which is positive for the flow velocity when a gate fails.

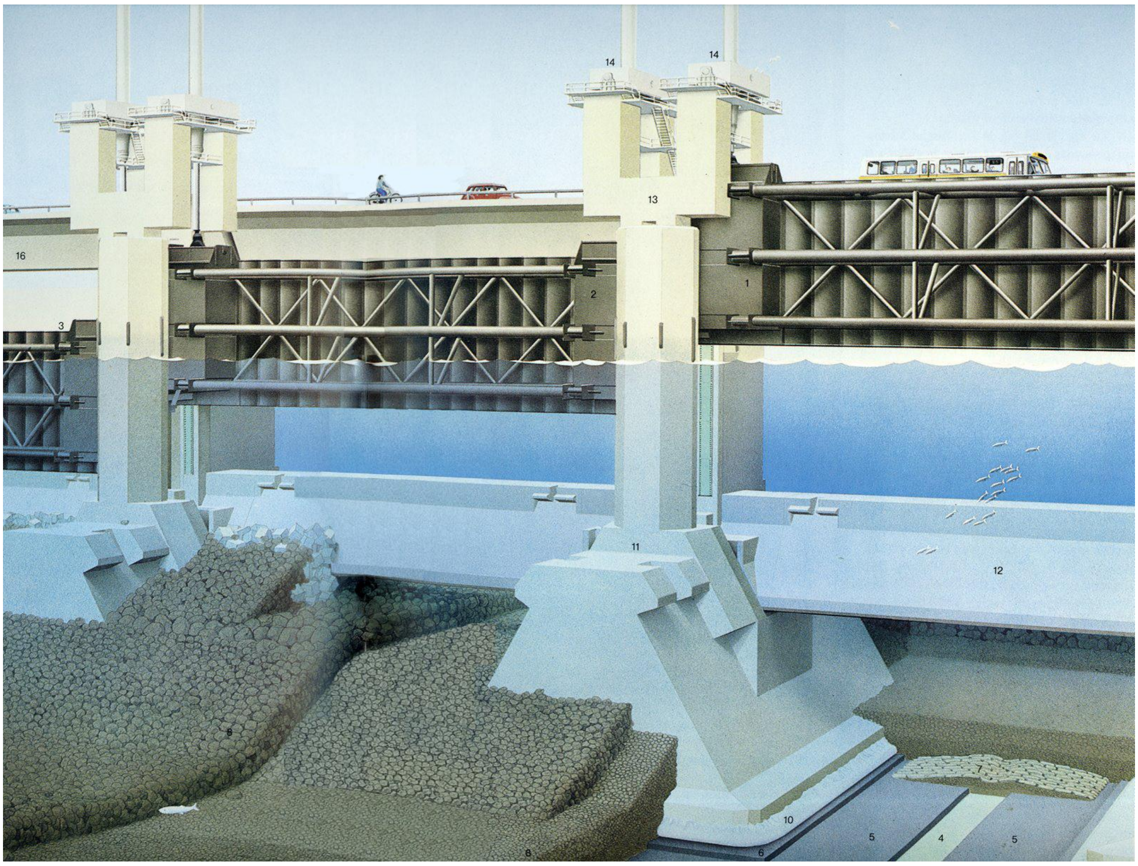

Figure 20: threshold of the barrier

Currently the bed protection is sometimes damaged at extreme conditions. This is not a problem as long as damage does not affect stability of the pillars, which is not the case. Repairing the bed protection can be done quite easily after damage. For the bed protection the same applies as for the threshold. It is expected that the closing regime is adapted at sea level rise. This reduces the hydraulic head and the flow velocities, so the threshold and the bed protection are not expected to be badly influenced by a sea level rise of 1,0m.

3.3.6. Conclusion

Comparing the current hydraulic loads with the design loads quite a lot of reserve is present. Based on this investigation a sea level rise of 0,5m is assumed to be not a problem for the structure.

At a sea level rise of 1,0m the 1/4000 year water level at the sea side of the barrier exceeds the design water level a bit. However, at the Eastern Scheldt side quite a lot of reserve is present. Comparing the design wave loads with the actual loads, a sea level rise of 1,0m seems to be acceptable. However, this way of investigating the structure is very straightforward, so possibly important processes are neglected. The bridge structure was designed to be not influenced by wave action. When the sea level rises with 1,0m it is possible that the bridge structure will be influenced at the 1/4000 year condition.

The durability of the structure is not investigated. It is assumed that with good maintenance corrosion will not affect the strength of the structure.

Because of the uncertainty of the strength of the irreplaceable parts of the structure it is determined what the costs are, when the barrier has to be adapted, see Section 4.7.5.

3.4. Impact sea level rise on closing regime

Currently the gates of the barrier will be closed when a sea level of NAP +3,0m is expected. The gates will be closed during low tide to obtain lower water levels in the Eastern Scheldt. The 1-2-1 closing strategy is currently used which means the water level in the Eastern Scheldt at the first high tide is hold at NAP+1,0m, at the second high tide at NAP +2,0m, at the third high tide at NAP +1,0m, etc.

A water level of NAP +3,0m at the sea-side of the barrier will appear once per year statistically, see Figure 21. The statistics in the plot are obtained from Waternormalen[1]. The statistics from the location 'Roompot Buiten' are used, which is located close to the North Sea side of the barrier. Since the completion of the barrier the gates are closed 24 times, so this is approximately once each year. The statistics for sea level rise are raised with the amount of sea level rise. When 0,5m sea level rise will occur the gates have to be closed approximately 5 times each year when holding the current closing strategy. At a sea level rise of 1,0m the gates have to be closed approximately 30 times each year.

Closing the barrier 30 times each year implies that the barrier has to close for long periods. A closed barrier will cause more or less stagnant water levels in the Eastern Scheldt. However, the barrier was designed to remain the tide in the Eastern Scheldt. Stagnant water levels will cause erosion of the flats because the wave attack is taken place at one stagnant water level. In this study closing the barrier 5 times each year is considered as a maximum

[1] www.rijkswaterstaat.nl/water/scheepvaartberichten_waterdata/statistieken_kengetallen/waternormalen/

number of closings. So a different closing strategy has to be chosen when a sea level rise of 1,0 meter has appeared. In this study a closure level of NAP +3,5m is used in that case. In Section 4.1 the determination of the statistics is elaborated.

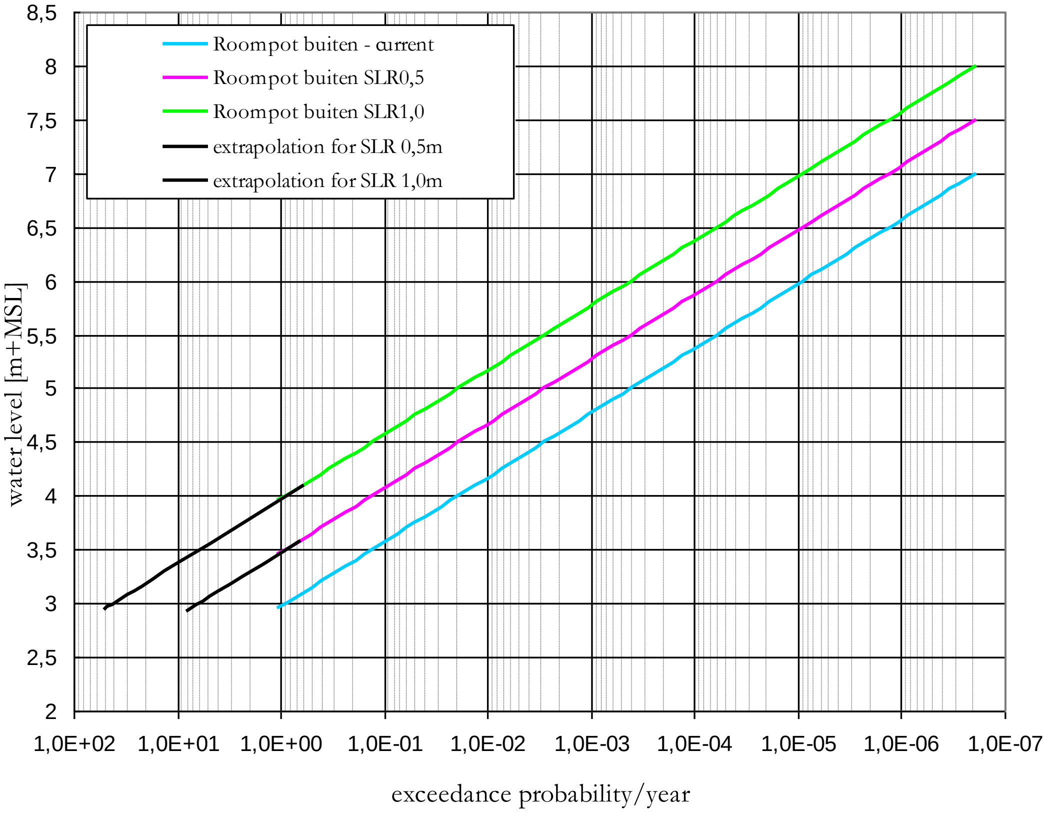

Figure 21: exceedance probability for different amounts of sea level rise (source: Waternormalen)

4. INPUT PARAMETERS

To calculate the failure probability of the dikes a couple of input parameters have to be determined. In this section the various parameters are described. These are the extreme water level statistics, the various parameters of the dike (e.g. the crest height, the berm height, the characteristics of the soil below the dike), the hydraulic loads, the consequences and the costs.

4.1. Water level statistics

The water level statistics which are used in this study are shown in Figure 23. The current water level statistics in the Eastern Scheldt are obtained from [RIJKSWATERSTAAT, 2008]. The current statistics when the barrier will be removed are based on Waternormalen[1]. These statistics are validated for the year 2006. In the Eastern Scheldt the statistics differ due to wind set-up. The water level by storm conditions in the West part is usually lower than the East part. The turning point lays somewhere on the line Wemeldinge - Stavenisse, see Figure 22 [RIJKSWATERSTAAT ZEELAND, 1985]. For this study the statistics for Wemeldinge are used. In this way the statistics represent the average water level in the Eastern Scheldt.

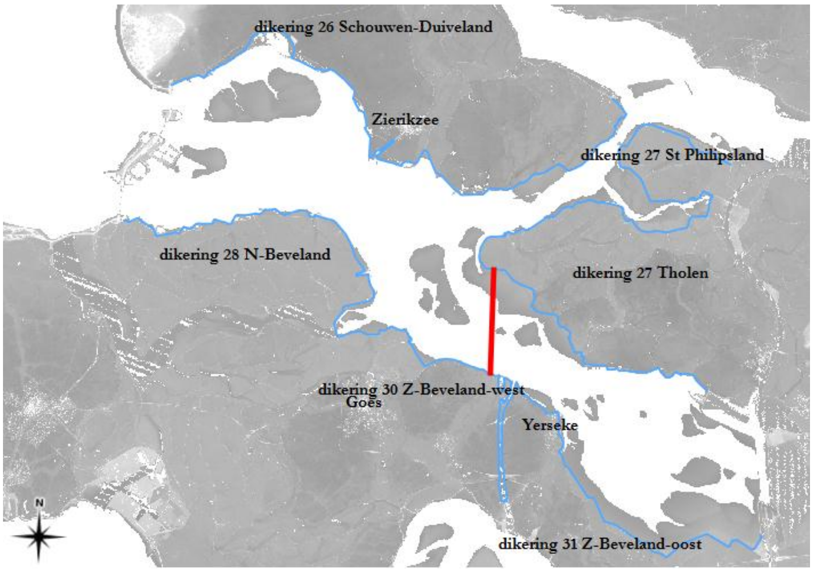

Figure 22: location of the line Wemeldinge - Stavenisse

For the situation that the barrier will be maintained the statistics are known, but when the barrier will be removed the statistics have to be determined. This is done by using the statistics from 'Roompot Buiten' (just outside the barrier) and raise the statistics with 0,5m to take into account the water level difference between the mouth and Wemeldinge. In Appendix I further details about this assumption are described.

The water level statistics when sea level rise appears will be different from the current statistics. In the case of no barrier the statistics are simply raised with the amount of sea level rise, see Figure 23. It is possible that the wind set-up differs a bit at deeper water depth, however, it is assumed that this effect is not significant.

[1] www.rijkswaterstaat.nl/water/scheepvaartberichten_waterdata/statistieken_kengetallen/waternormalen/

In the case that the barrier will be maintained the water level statistics in the Eastern Scheldt are more difficult to predict, due to the influence of the barrier. Simply raising the water levels is not correct because the barrier blocks extreme water levels from the North Sea. For a sea level rise of 0,5m it is assumed that the closing strategy does not have to be adapted, see Section 3.4. It is assumed that leakage through the barrier does not influence the statistics if the barrier will not be overflowed. This assumption is based on information from Rijkswaterstaat Zeeland[1]. When the water level becomes higher than NAP +5,8m the barrier will be overflowed. A water level of NAP +5,8m will appear with a higher exceedance probability in the case of 0,5m sea level rise. For that reason the statistics are shifted, see Figure 23 and Appendix I for further information.

For a sea level rise of 1,0m and closing at NAP +3,0m the barrier has to close 30 times each year, see section 3.4. This is assumed to be not acceptable. In this study a closure level of NAP +3,5m is used in that case. This influences the water level statistics, also in the case that the barrier is not overflowed. For that reason the water levels are raised with 0,5m for water levels till NAP +5,8m. For water levels higher than NAP +5,8m the statistics are shifted the way it is done by a sea level rise of 0,5m. See Appendix I for more detailed information.

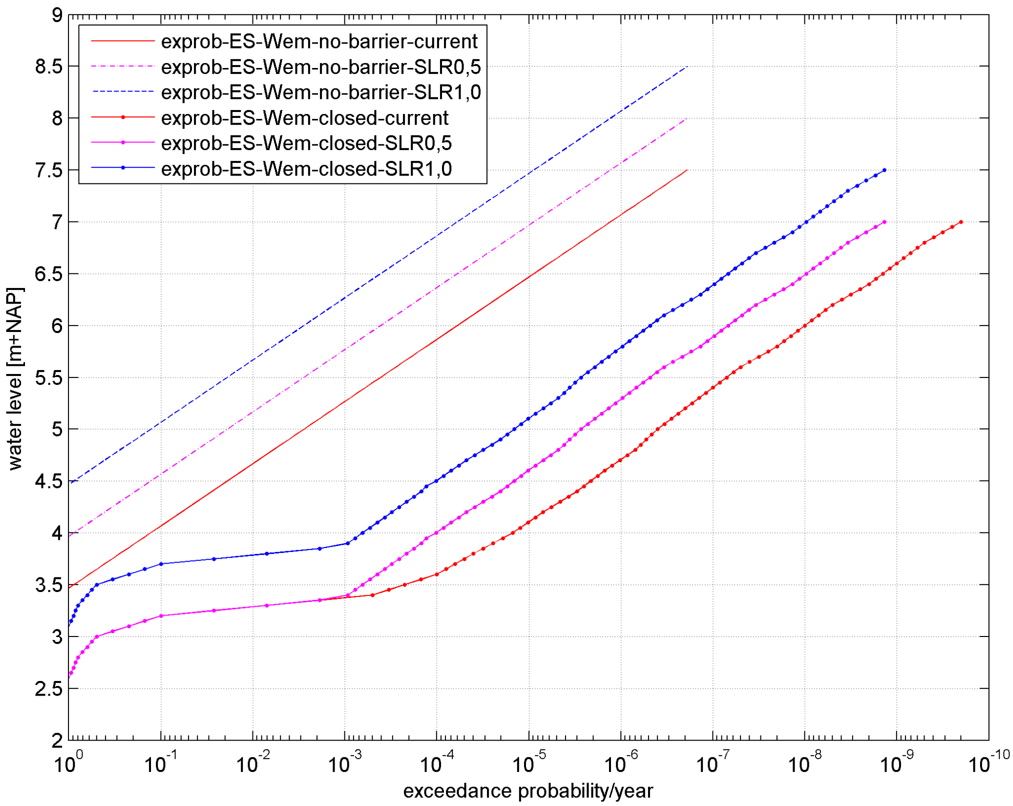

Figure 23: different water level statistics Eastern Scheldt, see Appendix I for elaboration

4.2. Dike height

The dike height differs a lot along a dike-ring, see Figure 24. The dike height is determined by drawing a line over the top of the dike on data from the AHN1 5x5m raster and taking

[1] K. Saman, interview at 25 April 2012

the highest value every 50m. The validity of this method is checked and can be seen in Appendix II.3. The AHN1 data is measured in the period 1996-2003. It is possible that the height of the dike has increased a bit since the measurement by 'Projectbureau Zeeweringen'. This project focuses on updating the dike revetment but at a couple of places the dike is heightened as well. This dike heightening is not accounted for in this study.

The question rises which dike height should be taken from all these values.
In Figure 25 the combination of the 1/4000 year water level and run-up height [z2%] is plotted for dike-ring 27 (see Appendix II.2 for other dike-rings). This is an approximation of the required dike height according to the current safety standards. The run-up height is calculated with a slope angle of 1:3, a berm width of 5 meter and a berm height of NAP +4,5m. These representative values are determined in Appendix II. The hydraulic loads calculated by HR2006 are used. As can be seen in Figure 25 there is a relation between the dike height and the hydraulic load, however, this relation is not always consistent. At most of the places the dike height is a lot higher than the required dike height according to the safety standards. Only some weaker and stronger spots are visible.

Taking into account the relation between the hydraulic load and the dike height, in this study the varying dike height is simplified by taking the average value: NAP +6,4m. Implicitly it is assumed that the failure probabilities are the same along the dikes. Taking into account the scale of this study the places for which this relation does not hold are neglected. Another argument for this assumption is that locations for which the dike has to be heightened more than average will neutralize in total costs with locations for which the dike has to be heightened less than average. In Appendix II.2 this choice is elaborated.

It is investigated whether it is possible to take smaller parts, e.g. dike-rings or a part of the dike-ring with more or less the same dike height and hydraulic condition. However, only taking very small parts (taking dike-sections, like is done by FLORIS) will lead to more accurate results. Taking such small parts is too detailed for this study, so for that reason one average dike height is chosen.

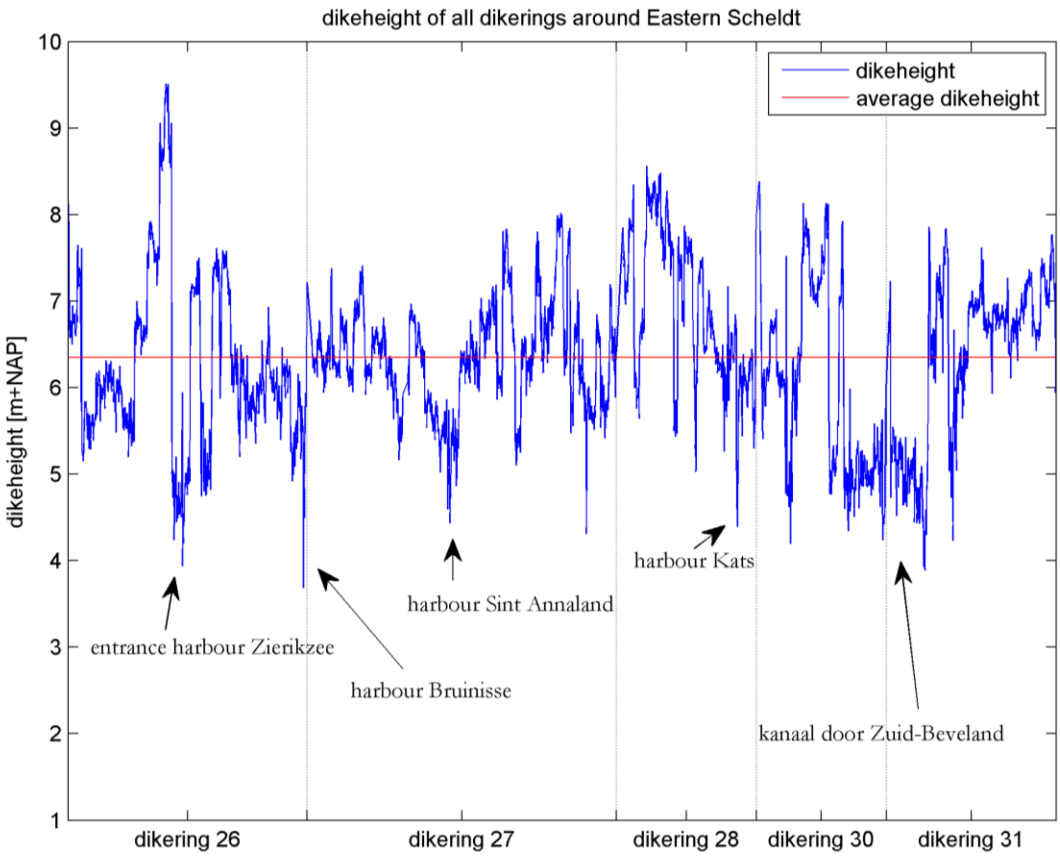

Figure 24: dike height of dikes around Eastern Scheldt (based on AHN1 data)

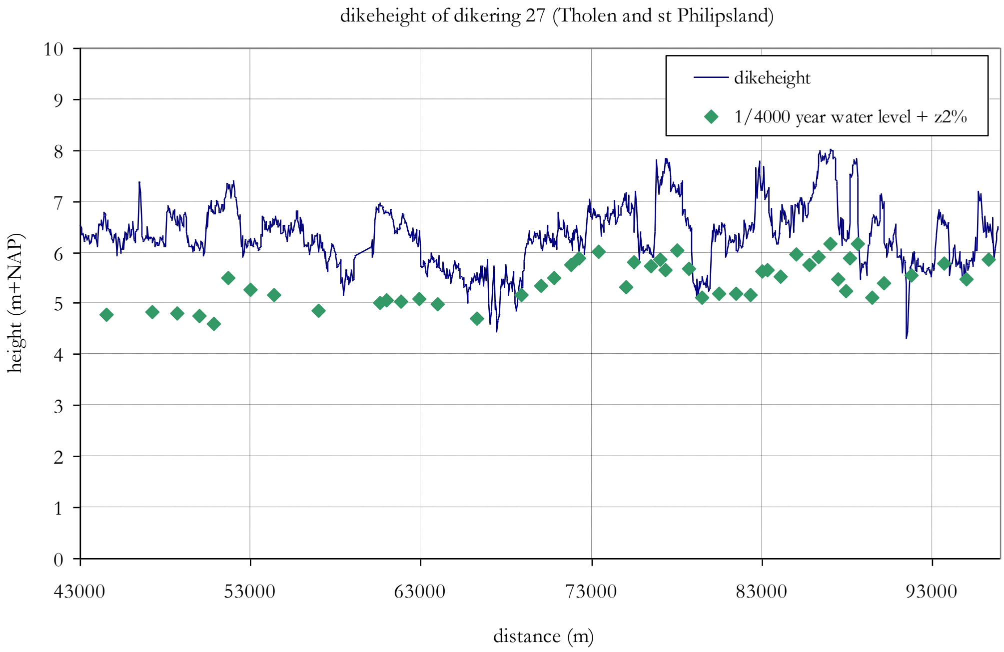

Figure 25: dike height of dike-ring 27 and the minimum required dike height based on HR2006

Along the Eastern Scheldt there are a couple of locations which are not taken into account by determining the dike height. These are the Oesterdam, the Philipsdam and the Grevelingendam. The height of these parts cannot be obtained by AHN1-data. It is assumed that these dams have to be heightened with the same amount as the dikes.

4.3. Wave loads

A representative wave load is determined similar to the determination of the dike height. From HR2006 the average wave load, wave period and angle of coincidence are determined, see Appendix III. The corresponding waterdepth is the 1/4000 year water depth at Wemeldinge, NAP +3,5m. The representative values are shown in Table 7.

significant wave height [H_s]	1,2m
spectral wave period [$T_{m-1,0}$]	3,6s
angle of coincidence waves [β]	40°

Table 7: average wave parameters in the Eastern Scheldt, based on HR2006

In the overtopping-calculation the failure probability has to be calculated for different water levels. Each water level contributes to the total failure probability. The wave height and wave period depends on the water depth because the waves close to the dike are often depth-limited. This implies that the waves are higher in deeper water. Especially when the barrier will be removed this phenomenon is important because extreme water levels will fairly increase. To take the relation between waves and water depth into account the waves are raised with 20% of a rise of water level, see Figure 26. The factor of 20% is quite a conservative value for the water level of NAP +3,5m and a wave height of 1,2m.

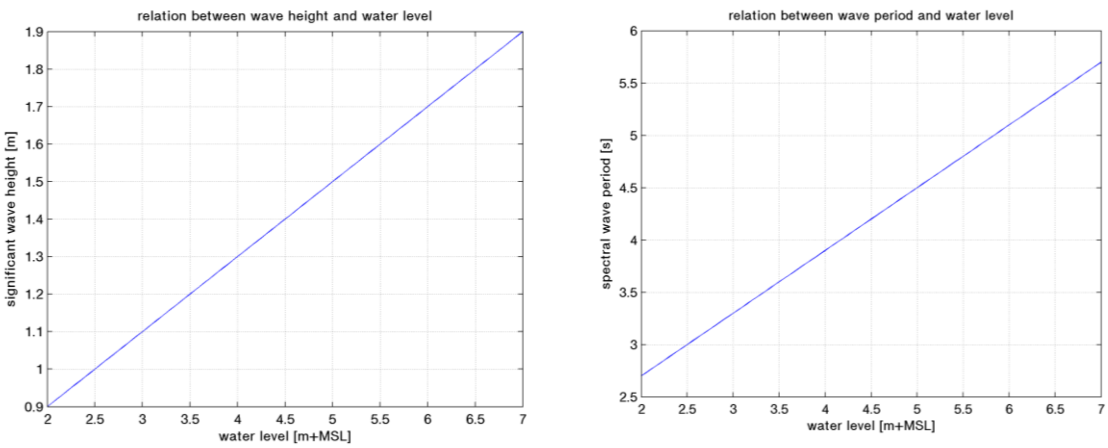

Figure 26: relation between waves and water level

4.4. Schematization for overtopping

The input parameters for overtopping can be seen in Table 8. Hereby an average dikeprofile for the Eastern Scheldt is chosen. The standard deviations from hydraulic parameters and dike parameters are not based on the varying dike heights and hydraulic loads in the plots. When taking the standard deviation from the dike height and the wave loads the relation between the dike height and the hydraulic load is neglected. Instead of that a quite small deviation is chosen in order to take into account the inaccuracy by determining the average values. The representative dikeprofile is shown in Figure 27. Parameters such as the outer slope angle, the berm width and the angle of coincidence of waves, are elaborated in Appendix II.

	average value	st. dev.
dike height [m+NAP]	6,4	0,1
berm height [m+NAP]	4,5	0,1
berm width [m]	5	0,1
angle outer slope [-]	1:3	0,05
significant wave height [m]	see Figure 26	0,1
spectral wave period [s]	see Figure 26	0,1
angle coincidence waves [deg]	40	1,0
critical overtopping discharge [l/s/m]	10	-
factor influence roughness [-]	1	-
factor influence vertical wall [-]	1	-

Table 8: different input parameters for overtopping

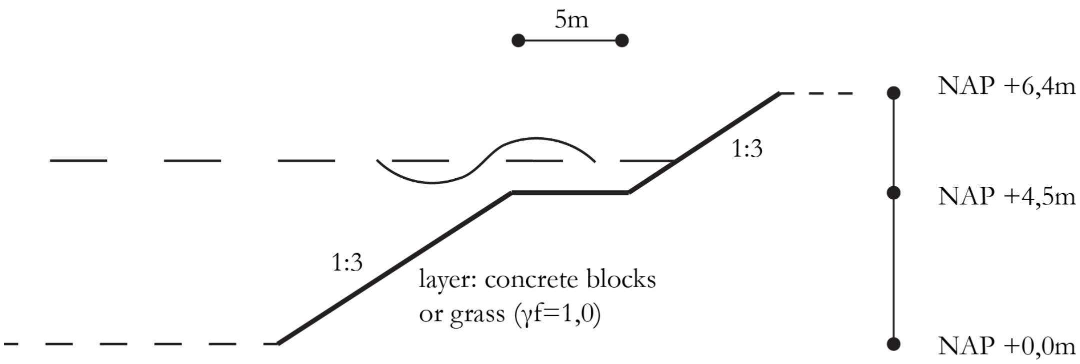

Figure 27: dikeprofile for overtopping

For the critical overtopping discharge a value of 10 l/s/m is used. Usually a critical overtopping discharge of 1,0 l/s/m is used in the design of a new dike in the Netherlands. Recently a lot of tests are done on the mechanism overtopping. Several results for determining the critical overtopping discharge are presented in [VAN DER MEER *et al*, 2009]. It appeared that the 1,0 l/s/m is quite a conservative value. Even bare clay with grass roots can withstand an overtopping discharge of 30 l/s/m. This study focuses mainly on the future. It is expected that the current standard on overtopping discharge will be adapted in the future. For that reason an overtopping discharge of 10 l/s/m is used. Comparing with the current practice in dike design more discharge is allowed in this study, which implies that the dike height can be lower. In Appendix II.1 results from overtopping tests are described.

When the barrier will be removed waves from the sea are able to enter the Eastern Scheldt during extreme storms. This will influence parts of dike-ring 26 and 28 which are located at the mouth of the Eastern Scheldt. Locally the dikes have to be heightened more than the average heightening. The influence is not investigated quantitatively in this study.

4.5. Schematization for piping

In comparison with the failure mechanism overtopping failure by piping is more difficult to predict and calculate. Important parameters for piping are the soil parameters which are hard to determine. In this study the input parameters are mostly obtained from the calculation results of dike-ring 26 done by FLORIS [VNK, 2011]. In Appendix II.2 for a couple of important parameters the results from VNK are shown. Based on these information average parameters are determined which are seen as values also valid for other dike-rings. In comparison with overtopping it is expected that the calculation results from piping are less accurate. The different input parameters can be seen in Table 9.

	average value	st. dev.
thickness sandlayer [m]	60	2
thickness covering clay layer [m]	1	0,1
seepage length [m]	42,7	2
permeability sand layer [m/s]	1,5e-5	3,0e-6
particle diameter d70 [m]	1,15e-4	1,15e-5
water level ditch [m+NAP]	0,0	0,2
internal friction angle sand [˚]	43	2
factor of White [-]	0,3	-
unit weight of sand [kN/m^3]	27	-
unit weight of water [kN/m^3]	10	-
kinematic viscosity [m^2/s]	1,33e-6	-

Table 9: different input parameters for piping

Based on these parameters the failure probability for piping can be calculated, see Chapter 5. In the reliability calculations it is assumed that cracking has already taken place, because the covering clay layer is relatively thin, no resistance to cracking is present. The required seepage length is calculated with Sellmeijer [TAW, 1999b].

The chosen standard deviations from the plot are based on the expected accuracy of the parameters and the variety from the parameters, see Appendix II.2.

A measure to improve the resistance of the dike against piping is to construct a piping berm or a seepage wall. According to [VRIJLING *et al*, 2010] a piping berm is the cheapest solution for rural areas and a sheet pile is the cheapest solution for urban areas, which is also applied in this study.

Where a seepage wall is needed, the required vertical length is calculated based on the formula of Lane, see Formula 4.1 [TAW, 1999b]. According to the formula of Lane the vertical seepage length has a weight of 3 times the horizontal seepage length. Above of that the vertical seepage length is two times the length of the seepage wall, see Figure 28. This cumulative effect results in a required vertical seepage wall length of 1/6 times the required horizontal piping berm length. The required horizontal piping berm length is calculated with the formula of Sellmeijer.

$$\Delta H \leq \Delta H_c = \frac{\left(\frac{1}{3} L_H + L_V\right)}{C_{w,creep}} \tag{4.1}$$

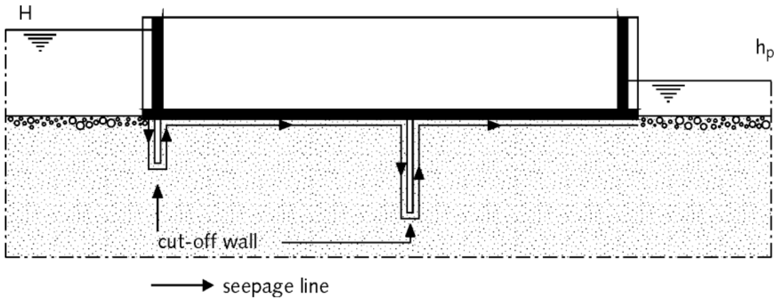

Figure 28: vertical and horizontal seepage length (source: [TAW, 1999b])

The length of the piping prevention measure depends on the amount of dike heightening. When a dike is heightened the seepage length increases, so the piping berm or the seepage wall can be shorter, see Figure 29.

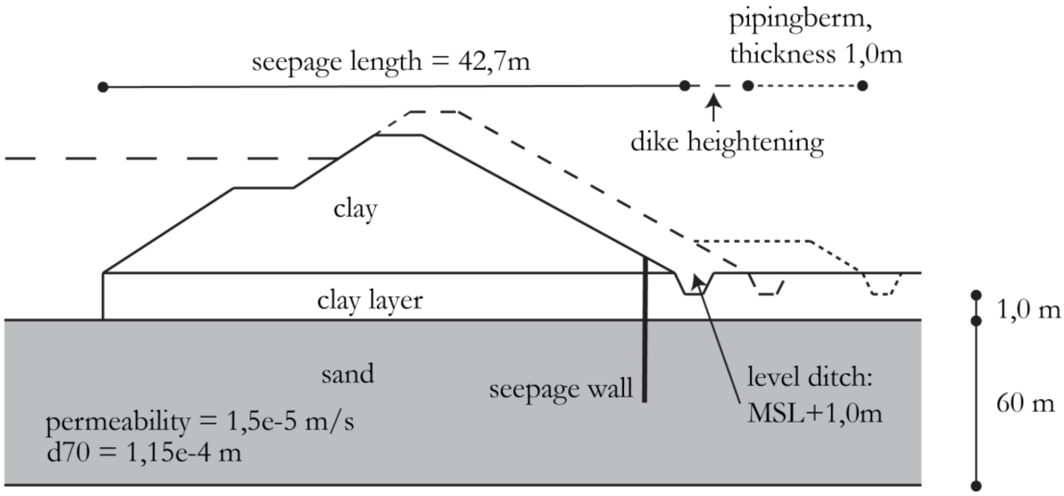

Figure 29: Schematization for piping

4.6. Consequences of a flood

The consequences from a flood can be obtained using information from 'Flood protection 21st century' (dutch: Waterveiligheid 21e eeuw (WV21), [DELTARES, 2011]). In [DE BRUIJN AND VAN DER DOEF, 2011] the consequences for every dike-ring in the Netherlands are calculated using flooding simulations.

The consequences by [De Bruijn and Van der Doef, 2011] are calculated by selecting parts of the dike-ring at which the consequences of flooding are the same. By giving a weight to every contribution the total consequences per dike-ring are calculated. The weight is based on the length of the dike-ring in relation to the total length. Also the effect of more breaches per part of the dike-ring is taken into account. The consequences are shown in Table 10. The locations of the dike-rings can be seen in Figure 30.

dike-ring	name	damage(M€)	affected people	casualties
26_1	Schouwen Duiveland-West	320	4.100	10
26_2	Schouwen Duiveland-Oost	830	11.000	50
27_1	Tholen and St. Philipsland	580	8.700	65
28_1	Noord-Beveland	150	2.300	5
30_1	Zuid-Beveland-West	930	14.000	180
31_1	Zuid Beveland-Oost	720	5.500	130

Table 10: Consequences from flooding [Deltares, 2011]

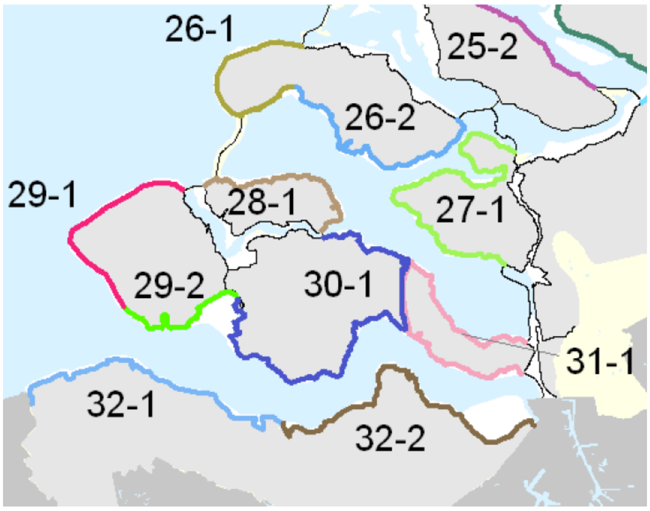

Figure 30: location of the dike-rings

The results from 27_1, 26_2 en 28 are based on flooding from only the Eastern Scheldt, but the results from 26_1, 30_1 and 31_1 are based on flooding from the North Sea and the Western Scheldt as well. The consequences for flooding for these dike-rings may be lower when only flooding from the Eastern Scheldt would be taken into consideration, however, this is not considered in this study.

The total damage for all the dike-rings around the Eastern Scheldt is 3530 M€. The total number of casualties is 440. The total number of affected people is 45.600. The monetary value of casualties, in literature the 'value of a statistical life' (VOSL), for flooding used by WV21 is 6,7 M€. This value is also used in this study. The monetary value of affected people, which consist of loss of souvenirs, loss of income etcetera, is estimated by WV21 at €12.000 per affected people. Also in this study this value is used. The consequences for the loss of life and the affected people, is 3495 M€. When a flooding occurs, the total consequences for damage and casualties are 3530+3495 = 7025 M€.

Comparing the number of casualties with the flood in 1953, the number found by WV21 is lower. In 1953 the flood causes 848 casualties for the dike-rings around the Eastern Scheldt [VAN DER KLIS *et al*, 2005]. Schouwen-Duiveland and Tholen suffered the most, only for Schouwen-Duiveland the number of casualties was 534. WV21 estimated the number of casualties for Schouwen-Duiveland at 60 people at a flooding. Since 1953 the population of the province Zeeland has grown. The number of inhabitants in 1950 was 71% from the inhabitants in 2005 [VAN DER KLIS *et al*, 2005]. However, the quality of houses, the warning system and the mobility has also grown, which is important for the number of casualties. For the damage after the flood in 1953 no specific data is found for dike-rings around the Eastern Scheldt.

For the consequences the numbers found by WV21 are used. These numbers are based on the present situation.

4.7. Costs

4.7.1. Dike heightening

For the costs of dike heightening the results from WV21 can be used. In [DE GRAVE AND BAARSE, 2011] for every dike-ring in the Netherlands a cost function is made for calculating investments for a certain heightening of the dike. The parameters for the cost function are determined by fitting a line through points where the costs for different dike heightenings are calculated. The cost function is described as follows:

$$I(u,W) = (C + b \cdot u) \cdot e^{\lambda(u+W)} \tag{4.2}$$

Where:

I	=investment costs	[M€ gross costs]
u	= dike heightening	[cm]
W	=sum of previous dike heightenings	[cm]
C	= fixed costs	[M€]
b	=variable costs	[M€/cm]
λ	=scale parameter	[1/cm]

For the different dike-rings around the Eastern Scheldt the parameters shown in Table 11 are found by [DE GRAVE AND BAARSE, 2011]. The location of the dikes can be seen in Figure 31.

dike-ring	name	length (km)	λ	C	b
26_1_1	Schouwen Duiveland	8,17	0,00095	10,07	0,21
26_1_2	Schouwen Duiveland	0,85	0,00095	2,20	0,06
26_2_1	Schouwen Duiveland	34,15	0,00336	24,88	0,65
27_1_1	St. Philipsland	16,07	0,00336	6,40	0,31
27_1_2	Tholen	36,64	0,00221	20,72	0,96
28_1_1	Noord-Beveland	23,78	0,00213	11,04	0,62
30_1_3	Zuid Beveland Oost	14,29	0,00095	16,79	0,49
30_1_4	Zuid Beveland Oost	8,00	0,00095	6,48	0,22
31_1_2	Zuid Beveland West	21,20	0,00095	18,58	0,66
31_1_3	Zuid Beveland West	7,29	0,00095	17,57	0,23

Table 11: parameters for the cost function [DE GRAVE AND BAARSE, 2011]

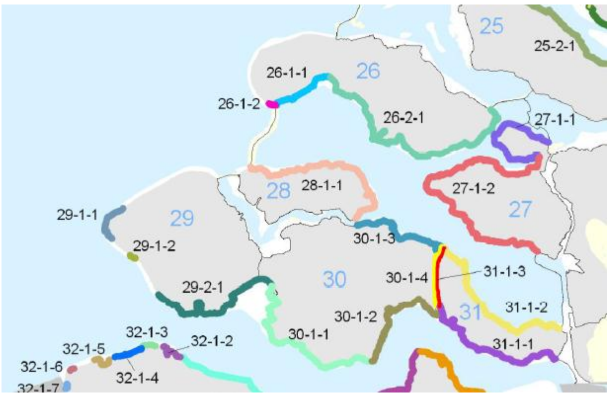

Figure 31: location of dikes

By calculating the investment costs for all parts of the dike-rings and adding them, the total costs for all the dike-rings are calculated. The average costs per kilometre width are shown in Figure 32.

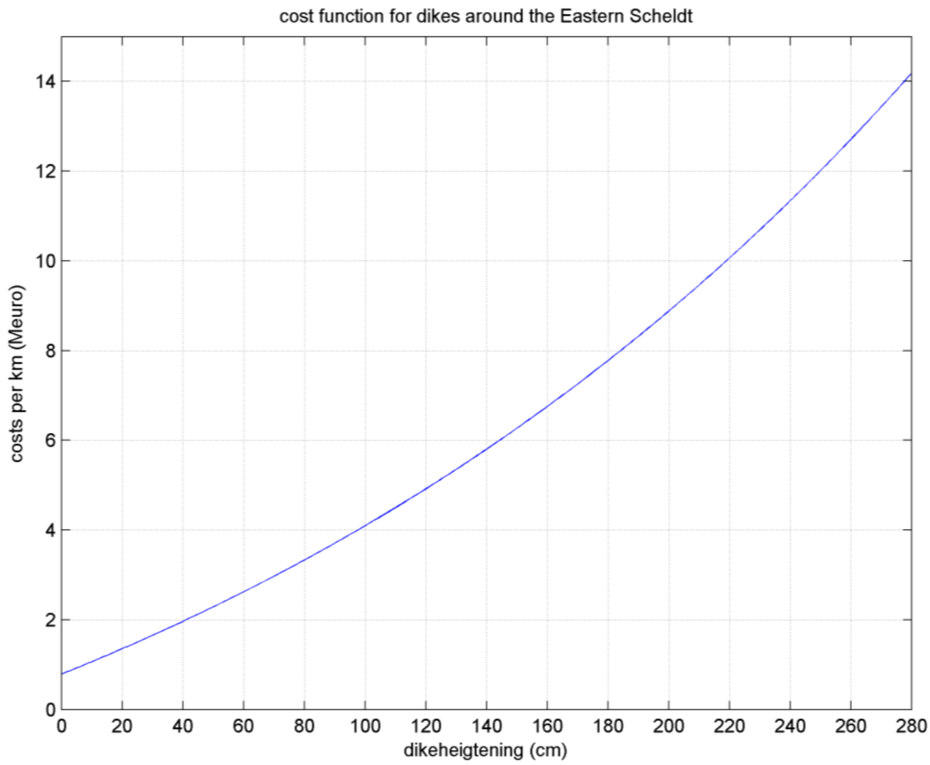

Figure 32: costs of dike heightening for dikes around Eastern Scheldt

According to this cost function the costs for 1,0 meter dike heightening of the dike around the Eastern Scheldt are 4,10 M€/km.

The costs for heightening the dike are estimated several times. A number of outcomes are shown in Table 12.

The Netherlands	
Dike (Millions € per km)	**Dike heightening (per m)** • 9 – 10.8 (rural) (Kok et al., 2008) • 18 – 21.6 (urban) (Kok et al., 2008) • 4 – 11 (rural) (Eijgenraam, 2006) • 6.9 (rural) (Fugro and Arcadis, 2006) • 13.8 (urban) (Arcadis and Fugro, 2006)

Table 12: different estimations for costs of dike heightening [HILLEN *et al*, 2010]

The biggest part of the dike-ring is situated in a rural area. An average value for the several estimates is 9 million € per meter heightening per kilometre length. This is a substantial difference with the costs found by WV21. In this calculation the results from WV21 are used because these costs are more case specific and determined more detailed.

4.7.2. Piping preventing measures

The costs of a pipingberm depends amongst others on the required length perpendicular to the dike and the height of the berm. The required height of a pipingberm can be calculated quite accurate by determining the uplifting pressure of water underneath the piping berm. This is too detailed for this study. Instead of that an average berm height of 1,0m is chosen. The costs of a pipingberm are obtained from [VRIJLING *et al*, 2010]. The costs in that study are calculated based on a berm length of 9 and 18 meters. For this study the costs are calculated per length of the berm and length of the sheet pile.

description	unit	price (€)
earth moving	m^3	22
finishing area	m^2	0,25
replacing fences	m	20
moving ditch	m	50
subtotal		**92**
one-time costs	15%	14
subtotal		**106**
preparation, management, supervision	20%	21
subtotal		**127**
Tax	19%	24
subtotal		**151**
unforeseen costs	10%	15
subtotal		**167**
land banking	m^2	10
total costs per m² berm		**177**

description	unit	price (€)
purchase sheetpile	m	421
transport + drive piles	-	25
subtotal		**167**
one-time cost	15%	25
subtotal		**192**
preparation, management, supervision	20%	38
subtotal		**230**
tax	19%	44
subtotal		**274**
unforeseen costs	10%	27
total costs per m² sheet pile		**302**

Table 13: Costs of a piping berm and a sheet pile per width and length, based on [VRIJLING *et al*, 2010]

By calculating the costs the vertical length of the sheet pile is 1/6 x horizontal length of the piping berm, see Section 4.5. From the 171 km dikes (without compartmentalization dams) it is determined that for 149,5 km a piping berm is the best option and for 21,5 km of dikes a solution with seepage walls is required due to the presence of buildings, see Figure 33. The red parts in the figure are the locations where a seepage wall is the best option, and the green parts are locations for which a piping berm is the best option. The ratio of 149,5 km piping berm and 21,5 km seepage wall, is used by calculating the costs of piping preventing measures per kilometre dike. The costs are shown in Figure 34.

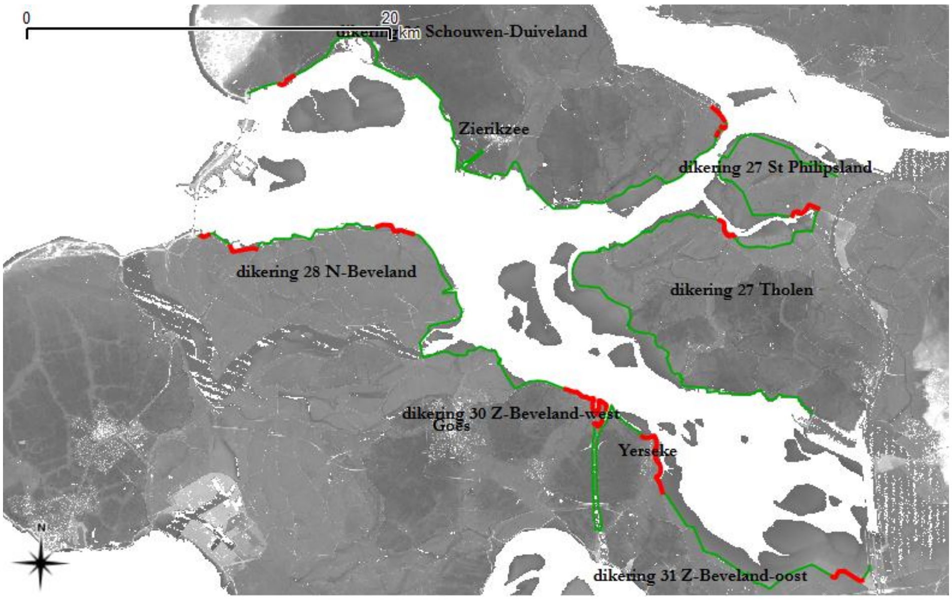

Figure 33: locations where seepage wall is the best option (red) and piping berm is the best option (green)

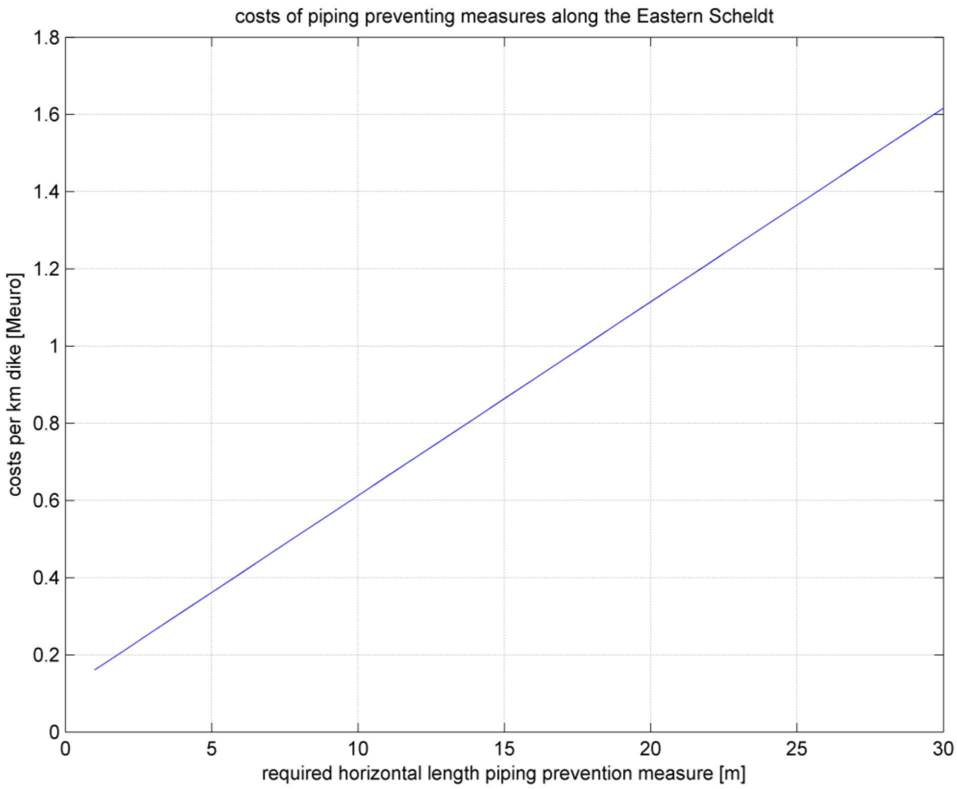

Figure 34: costs of piping preventing measures

4.7.3. Removing the barrier

By [DELTACOMMISSIE, 2008] a cost estimation is made for the different recommendations for the future of the Netherlands in relation with climate change. One of the recommendations was to remove the barrier, for which the costs are estimated. It is not so easy to remove the barrier, because most of the work must be done from open sea, which is difficult because of the tide. Most parts have to be removed when the gates are open. Due to the strong currents only during slack water operations can take place. First the movable parts have to be removed as well as the bridge and the lower and upper sill. Then the threshold with bottom protection has to be removed under water. The 66 pillars (including a reserve pillar) have to be moved and transported to a dock. To recover the tide sufficiently the islands Neeltje Jans and Roggenplaat have to be dredged.

According to the deltacommision: "An estimation of this project does not exist and cannot be made now. Purely based on intuition: 1-2 billion €." [RIJKSWATERSTAAT, 2008a]. In this study a value of 1 billion € is used.

4.7.4. Maintenance of the barrier

The Deltacommision poses in their estimation of the costs of the Deltaprogramma [RIJKSWATERSTAAT, 2008a] that the costs for maintenance of the barrier are 25 M€/year. According to the website of Neeltje Jans[1] the maintenance costs are 17 M€/year. Also values of 10-18 M€/year are found[2]. Important costs are maintenance of the cylinders, the traffic structure, the rabbets for the gates, the steal parts above the water line and preventing corrosion of the structure. In this study maintenance costs of 20 M€/year are used.

4.7.5. Adapting the barrier

According to [DELTACOMMISSIE, 2008] the Eastern Scheldt storm surge barrier can withstand a sea level rise of 0,5m. At a sea level rise of more than 0,5m the barrier has to be adapted. A solution according to the Deltacommission is decreasing the leakage area during closure to prevent the raising of water levels in the Eastern Scheldt. The costs are estimated at 1 billion euro [RIJKSWATERSTAAT, 2008a].

According to Rijkswaterstaat Zeeland[3] decreasing the leakage through the barrier is not the right solution for sea level rise. Instead of that the upper sill and the bridge structure may have to be adapted. This has to be done from sea. The number of sills is 62. Making a detailed cost-estimation is beyond the scope of the study. The total costs of the barrier with a price level for 2012 are approximately €5,5 billion. Assuming that the costs of adapting the upper sill and the traffic structure will be 1/5 of the total costs of the barrier, the adaption of the barrier will cost approximately €1 billion.

4.7.6. Maintenance of the dikes

The maintenance costs of the dike are not taken into account. The dike will be maintained in all the alternatives, so maintenance costs such as repairing the dike revetment will be approximately the same for all the alternatives.

4.8. Benefits

The benefits for the different alternatives can be the decrease in risk and the increase in ecologic value. The decrease in risk follows from the risk calculations, see Section 5. The value of ecology can be defined for instance by using the method from The Economics of

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1 http://www.neeltjejans.nl/index.php/nl/deltawerken/deltawerken/faq#16

2 http://communities.zeelandnet.nl/data/svk20jaar/index.php?page=13&showpage=72206

3 K. Saman, interview at 25 April 2012

Ecosystems and Biodiversity [TEEB]. This study focuses on safety, so the benefits from ecology are not determined.

However, to keep the value of ecology out of this study will result in an incomplete report. An interesting indicator for the benefits of ecology are the costs of suppletion of the tidal flats. According to [DELTACOMMISSIE, 2008] a sand suppletion of 3 million m^3/year has to be done to maintain the tidal flats. The costs of this suppletion are estimated at 45 M€/year [RIJKSWATERSTAAT, 2008a]. These costs can be seen as benefits in the case that the barrier will be removed, assuming that the tidal flats will not erode after the removal.

5. RESULTS FOR THE ALTERNATIVES

In Chapter 4 the calculation method and all the parameters to calculate the failure probability for the different alternatives are described. In this chapter the failure probability of the dike, the risk and the costs are calculated. First the failure probability for the current sea level is calculated. As described in Section 2.2.1 this is done for overtopping and piping. For both overtopping and piping one representative profile is determined which represent the dikes around the Eastern Scheldt. In Figure 35 the result for overtopping for the current sea level is shown. The method of calculating the failure probability is described in Section 2.2.2. The failure probability per year is shown in graph 1 of Figure 35. In Figure 36 the failure probability for piping is shown for the current sea level.

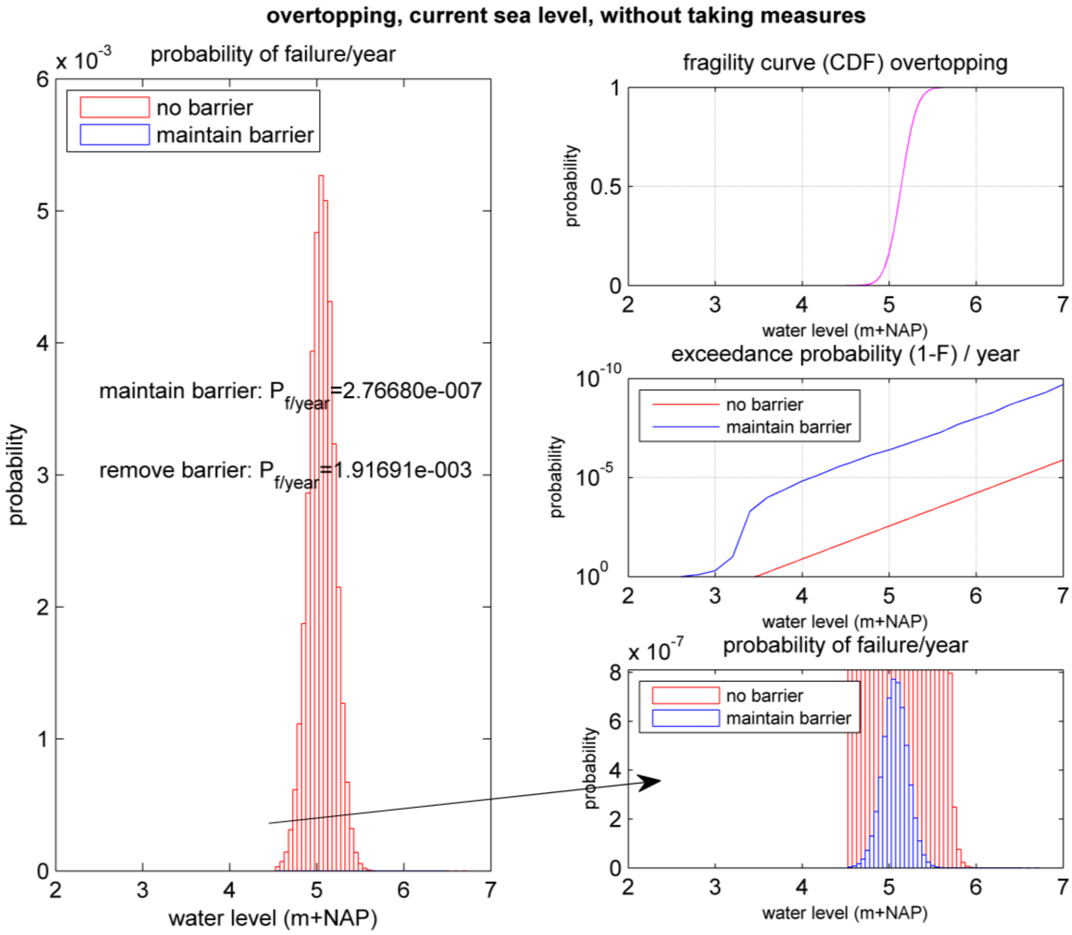


Figure 35: failure probability of the dike for overtopping for current sea level

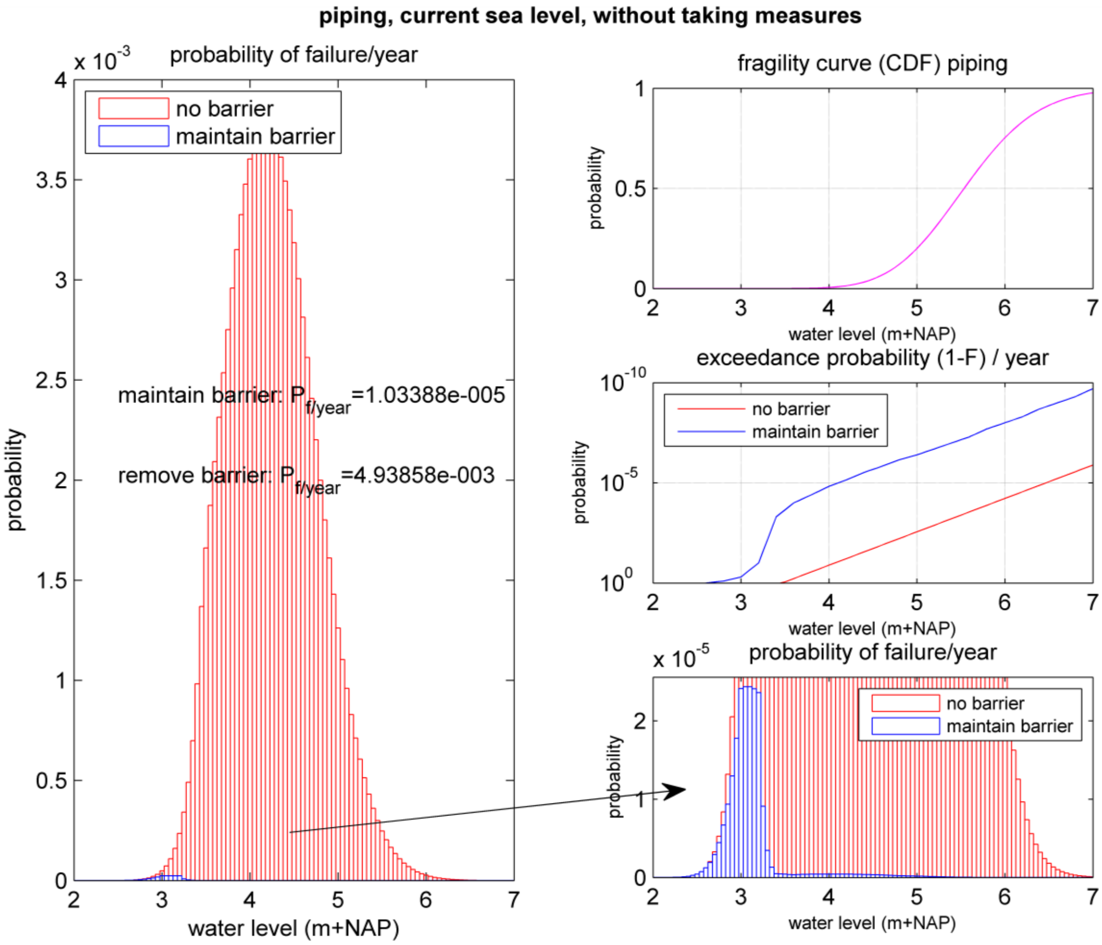


Figure 36: failure probability of the dike for piping for the current sea level

As can be seen in the figures the failure probability for overtopping by maintaining the barrier is approximately 2,77e-7/year and the failure probability by removing the barrier is approximately 1,92e-3/year. According to this calculation the failure probability per year for overtopping increases with about 7.000 times when the barrier will be taken away.

The current failure probability for piping, by maintaining the barrier, is 1,03e-5/year. When the barrier will be removed the failure probability becomes 4,94e-3/year. The calculation shows an increase in failure probability for piping of about 500 times when the barrier will be removed. Table 14 shows the failure probabilities for overtopping and piping for the current sea level.

	current failure probability/year for overtopping	current failure probability/year for piping
Maintaining barrier	2,77e-7	1,03e-5
Remove barrier	1,92e-3	4,94e-3

Table 14: current failure probability

The failure probability for overtopping after removing the barrier will become 1/520 year. It is interesting to see that this safety level is close to the safety level to which the dikes are heightened in the period before the construction of the dam, which was 1/500 year (RIJKSWATERSTAAT ZEELAND, 1985). However, in this study a critical overtopping discharge of 10 l/s/m is used. More overtopping discharge is allowed now in comparison with the critical overtopping discharge on which the dikes are designed. When using a critical overtopping discharge of 1 l/s/m the failure probability becomes approximately 1/100 year.

The failure probabilities for overtopping for dike-ring 26 calculated by FLORIS differ between 1/4000 and <1/1.000.000 year. About 1/3 of the inspected dike sections have a failure probability which is smaller than 1/1.000.000 year. The exact failure probabilities are not given in that case. The rest of the failure probabilities differ around 1/100.000 year. The calculations by FLORIS are done with a critical overtopping discharge based on the quality of the grass layer, which usually gives lower values than the 10 l/s/m which is used in this study. When calculating with an overtopping discharge of 1,0 m/s the failure probability becomes approximately 1/1.000.000 year.

The failure probabilities for piping calculated by FLORIS for dike-ring 26 are varying more than the calculated failure probabilities for overtopping. The failure probabilities are generally lower than 1/10.000 year, except for a few sections. Some weak spots have a failure probability in the order of 1/1.000 year. Average values in the order of 1/100.000 year are found. This corresponds with the failure probability for piping in Table 14, which is 1/100.000 year.

In section 2.2 it is described that the safety standard for overtopping is a water level which occurs 1/4.000 year. For piping the failure probability is not allowed to exceed 1/40.000 year. The current failure probabilities by maintaining the barrier for piping and overtopping are lower than is needed according to the current safety standards. However, when the barrier is taken away the failure probabilities are exceeding the safety standards.

What happens if sea level rise will appear? To calculate this, the statistics for sea level rise (see Figure 23) are used. In Appendix IV the calculation results are shown. The results are given in Table 15.

		failure probability / year for overtopping	failure probability / year for piping
Maintaining barrier	SLR 0,5m	1,88e-6	1,36e-5
	SLR 1,0m	1,01e-5	3,94e-4
Remove barrier	SLR 0,5m	1,31e-2	2,80e-2
	SLR 1,0m	8,90e-2	1,16e-1

Table 15: failure probability by sea level rise

According to the current safety standards the failure probability for overtopping by maintaining the barrier will not exceed the standard of 1/4000 year, even when a sea level rise of

1,0m will appear. But when the barrier will be taken away this safety standard will be by far exceeded. For piping the standard of 1/40.000 year is exceeded at a sea level rise of 1,0m by maintaining the barrier. When the barrier will be removed the safety standards for piping are exceeded as well.

5.1. Dike heightening by satisfy to the current safety standards

The question rises how much the dikes have to be heightened and what the required length of the piping berm is, to satisfy to the current safety standards. This is calculated for both alternatives and scenarios of sea level rise. In the calculations the berm height is placed at the 1/4000 year water level, which is according to the current practice in dike design.

The required horizontal length of the piping prevention measure depends amongst others on the amount of dike heightening, see Section 4.5. When a dike heightening takes place the seepage length increases, so the length of the prevention measure can be shorter. The required dike heightening is calculated by increasing the height of the dike till the safety standard of 1/4.000 year is satisfied. The required horizontal length of the piping prevention is calculated by increasing the length of the berm till a minimum failure probability of 1/40.000 year is reached. The amount of dike heightening and the required horizontal length of the piping prevention for the different amounts of sea level rise and alternatives are shown in Table 16. In Appendix IV the calculation results are shown.

		dike heightening	hor. length piping prevention
Maintaining barrier	current	0 m	0 m
	SLR 0,5m	0 m	0 m
	SLR 1,0m	0 m	7 m
Remove barrier	current	0,7 m	15 m
	SLR 0,5m	1,4 m	19 m
	SLR 1,0m	2,2 m	22 m

Table 16: dike heightening for the different alternatives and scenarios

As can be seen in Table 16 a dike heightening of 0,7m is currently needed to satisfy the safety standard when the barrier will be removed. When a sea level rise of 0,5m will appear the dikes have to be heightened with an extra amount of 0,7m to 1,4m. A sea level rise of 1,0m will require a dike heightening of 2,2m when the barrier will be removed. The differences between the scenarios are not only the difference in sea level rise. This is caused by the increase in wave load when the water level increases, see Section 4.3. The increase in the required horizontal length of the piping berm seems to be low when the sea level rises, however, dike heightening also influences the seepage length, so smaller piping prevention measures can be taken.

The risk can be calculated by multiplying the failure probability by the consequences (Formula 2.4). The consequences are estimated in Section 4.6 at 7025 M€. The failure probabilities are shown in Table 14 and Table 15. The results are shown in Table 17. The current risk is shown, as well as the risk after the measure (maintaining the barrier, removing the barrier, dike heightening or increasing length of piping prevention).

		dike heightening	hor. length piping prevention	risk overtopping before - after [M€/year]	risk piping before - after [M€/year]
Maintaining barrier	current	0 m	0 m	0,00 - 0,00	0,07 - 0,07
	SLR 0,5m	0 m	0 m	0,01 - 0,01	0,10 - 0,10
	SLR 1,0m	0 m	7 m	0,07 - 0,07	2,77 - 0,18
Remove barrier	current	0,7 m	15 m	0,00 - 1,76	0,07 - 0,18
	SLR 0,5m	1,4 m	19 m	0,01 - 1,76	0,10 - 0,18
	SLR 1,0m	2,2 m	22 m	0,07 - 1,76	2,77 - 0,18

Table 17: risk for the different alternatives and scenarios

As can be seen in Table 17 the current risk of overtopping for dike-rings around the Eastern Scheldt is extremely low, about €2.000/year. The current risk of piping is approximately €75.000/year. When the barrier will be removed and the dikes are heightened to the 1/4.000 year safety level, the risk of overtopping becomes approximately €1.800.000/year. When the barrier will be removed and the length of the piping prevention measure satisfies the safety standard of 1/40.000 year, the risk of piping will be €180.000/year. As can be seen in Table 17 the risk will increase in the case that the barrier will be removed. At the current sea level an increase from 0,00 - 1,76 M€/year is visible. The risk before the removal of the barrier is lower than the risk after removing the barrier and heightening of dikes. This is because the dikes are heightened according to the safety standard, which is lower than the actual safety level.

The costs for the different alternatives and scenarios are shown in Table 18. The total length of the dikes is 171 km and the total length of the compartmentalization dams is 21,3 km. The compartmentalization dams are assumed to be only influenced by overtopping. Piping is assumed to be not a problem, the width of the dams is at least 100 meter. Parameters for the costs can be seen in Section 4.7.

		dike heightening	Piping prevention	maintenance barrier	removing barrier
Maintaining barrier	current	-	-	20 M€/year	-
	SLR 0,5	-	-	20 M€/year	-
	SLR 1,0	-	79 M€	20 M€/year	-
Remove barrier	current	571 M€	148 M€	-	1000 M€
	SLR 0,5	1116 M€	182 M€	-	1000 M€
	SLR 1,0	1935 M€	208 M€	-	1000 M€

Table 18: costs for the different alternatives and scenarios

All costs are transformed to the Net Present Value by a discount rate of 0,055 and an infinite time horizon, see Formula 2.3. The Net Present Value for the different alternatives and scenarios are shown in Table 19. In this calculation the risk of piping is added to the risk of overtopping. It is also visible what the minimum amounts of benefits are to reach a Net Present Value of zero, the minimum value to make it cost-effective. Benefits in this case can be maintaining of the tidal flats.

		NPV	benefits needed for NPV=0
Maintaining barrier	current	-384 M€	20 M€/year
	SLR 0,5	-384 M€[1]	20 M€/year
	SLR 1,0	-413 M€[2]	22 M€/year
Remove barrier	current	-1755 M€	91 M€/year
	SLR 0,5	-2333 M€[1]	122 M€/year
	SLR 1,0	-3126 M€[2]	163 M€/year

Table 19: Net Present Value for different alternatives based on the current safety standards

As can be seen in Table 19 the best investment option for the current sea level, for a sea level rise of 0,5m and for a sea level rise of 1,0m, is to maintain the barrier.

All the investment options will lead to a negative NPV, the investments will lead to loss of money. The reason for the negative NPV is the current high safety level. It is almost impossible to reduce the risk further. Only in the case the maintenance costs for the barrier will disappear the investment can be cost-effective, however, removing the barrier is very expensive.

5.1.1. What if the dike height is based on a cost-benefit analysis?

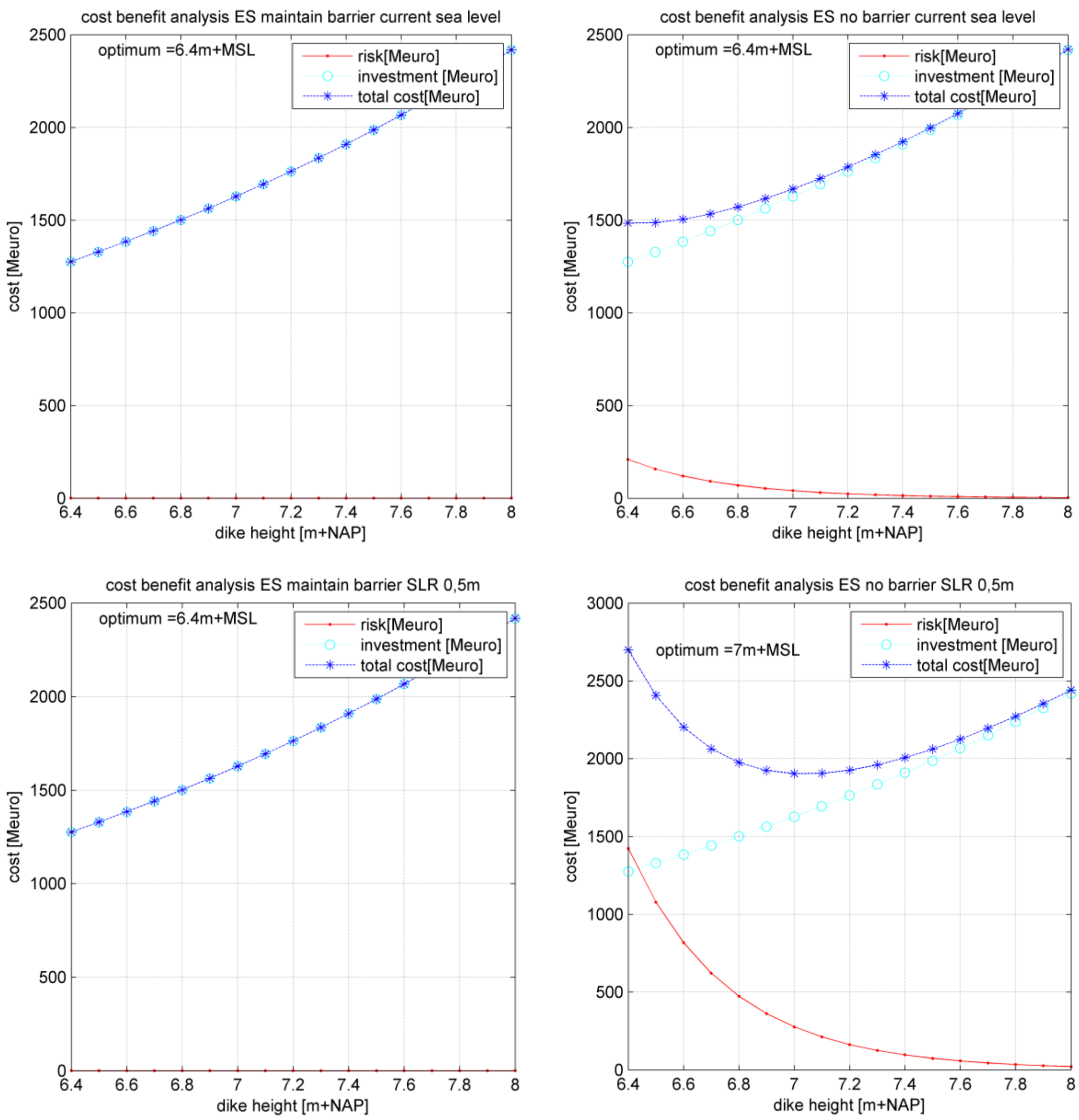
Instead of determining the required safety by satisfy to a safety standard, the safety can also be based on a cost-benefit analysis. The principle is the same as used by [VAN DANTZIG AND KRIENS, 1960] with the difference that in this case also costs of removing and maintaining the barrier are included. In principle also piping can be included in the cost-benefit analysis. The length of the piping berm depends on the amount of dike heightening, because the width of the dike also increases with heightening of the dike. An optimum can be found when the results are plotted in a 3-dimensional graph. In this study only a cost-benefit analysis is done on dike heightening.

The berm in the cost-benefit calculation is placed at 1,5m below the crest. This is approximately the optimum berm height, given the wave conditions in the Eastern Scheldt.

[1] This is the Net Present Value at the time 0,5m sea level rise has appeared

[2] This is the Net Present Value at the time 1,0m sea level rise has appeared

All costs are transformed to the Net Present Value by a discount rate of 0,055 and an infinite time horizon, see Formula 2.3. The results for the current sea level, a sea level rise of 0,5m and a sea level rise of 1,0m are shown in Figure 37.



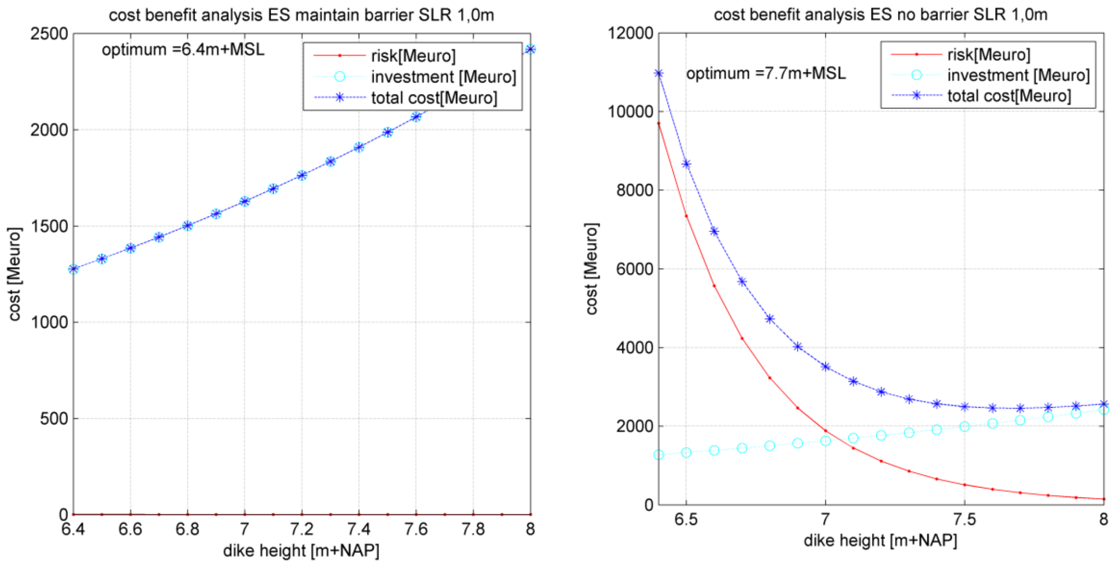


Figure 37: Cost benefit analysis for an investment in removing the barrier and heighten the dikes

The optimum dike height, failure probability and risk according to a cost-benefit analysis are shown in Table 20.

		dike heightening	failure probability / year for over-topping	risk overtopping before - after [M€/year]
Maintaining barrier	current	0 m	2,77e-7	0,00 - 0,00
	SLR 0,5m	0 m	1,88e-6	0,01 - 0,01
	SLR 1,0m	0 m	1,01e-5	0,07 - 0,07
Remove barrier	current	0 m	1,92e-3	0,00 - 13,49
	SLR 0,5m	0,6 m	2,16e-3	0,01 - 15,17
	SLR 1,0m	1,3 m	2,40e-3	0,07 - 16,86

Table 20: required dike heightening, failure probability and risk based on CBA

In Table 20 it can be seen that the optimum failure probability, in case of no barrier, at a sea level rise of 0,5m is approximately 1/500 year. In Section 2.2.3 the optimum failure probabilities for the dikerings around the Eastern Scheldt found by WV21 are described. The average optimum failure probability for the year 2050 (0,35m sea level rise) found by WV21 is approximately 1/1.500 year. This is somewhat lower than found by this study. By WV21 average probabilities are used; when the average failure probability is reached still a reserve is present for the time between the plan-making process and the dike heightening. Another difference with the WV21 study is the economic growth; in this study no economic growth is taken into account. By WV21 1,9% economic growth is taken into account.

The costs for the different alternatives are shown in Table 21.

		dike heightening [m]	dike heightening	maintenance barrier	removing barrier
Maintaining barrier	current	0	-	20 M€/year	-
	SLR 0,5	0	-	20 M€/year	-
	SLR 1,0	0	-	20 M€/year	-
Remove barrier	current	0	-	-	1000 M€
	SLR 0,5	0,6	505 M€	-	1000 M€
	SLR 1,0	1,3	1029 M€	-	1000 M€

Table 21: costs for the different alternatives and scenarios based on CBA

The Net Present Value of the alternatives is shown in Table 22.

		NPV	benefits needed for NPV=0
Maintaining barrier	current	-384 M€	20 M€/year
	SLR 0,5	-384 M€[1]	20 M€/year
	SLR 1,0	-384 M€[2]	20 M€/year
Remove barrier	current	-1259 M€	66 M€/year
	SLR 0,5	-1796 M€[1]	94 M€/year
	SLR 1,0	-2351 M€[2]	123 M€/year

Table 22: Net Present Value for different alternatives based on CBA

Calculating the required dike heightening by removing the barrier based on a cost-benefit analysis, the dike height decreases with 0,7m for the current sea level and 0,9m in the case of 1,0m sea level rise, see also Table 19. This is quite a lot of reduction. That can be explained by the relatively low economic value of the area and the relatively low population density.

The reduction in dike heightening decreases the costs of an investment in removing the barrier, however still maintaining the barrier is the best alternative when looking at costs and benefits. It has to be noted that the costs of a piping berm are not included.

5.1.2. What if the barrier cannot deal with 1,0m sea level rise?

It is quite sure that the barrier can deal with a sea level rise of 0,5m. But what happens if the barrier fails at a sea level rise of 1,0 meter? When the barrier fails the gates and the upper sill will fail. Also the bridge structure may fail. In Section 4.7.5 it is determined that the costs of adapting the upper sill and the bridge structure are 1,0 billion euro. The rest of the

[1] This is the Net Present Value at the time 0,5m sea level rise has appeared

[2] This is the Net Present Value at the time 1,0m sea level rise has appeared

costs can be found in Table 18. Adapting the barrier at a sea level rise of 1,0m will lead to a Net Present Value of € -1413, see Table 23.

		NPV	Benefits needed for NPV=0
Maintaining barrier	current	-384 M€	20 M€/year
	SLR 0,5	-384 M€[1]	20 M€/year
	SLR 1,0	-1413 M€[2]	74 M€/year
Remove barrier	current	-1755 M€	91 M€/year
	SLR 0,5	-2333 M€[1]	122 M€/year
	SLR 1,0	-3126 M€[2]	163 M€/year

Table 23: Net Present Value for the alternatives when the barrier cannot deal with 1,0m SLR

Still this investment is lower than investing in removing the barrier and heighten the dikes (NPV € -3126 for heightening according to the current safety standards and NPV € -2351 for dike heightening based on cost-benefits analysis), however, the costs are more close to each other.

5.2. Recommendations for investments in the safety of the Eastern Scheldt

In this paragraph a summary is given of the calculation results. A couple of recommendations are given for an investment in the safety of the Eastern Scheldt.

The current risk of overtopping and piping is low, in the order of 2.000 €/year for overtopping and 75.000 €/year for piping. It has to be noted that these values are average values for all the dikes around the Eastern Scheldt. From FLORIS it appeared that locally the failure probability (and the risk) of overtopping and piping is higher.

Because of the low average risk an integral investment in reducing the failure probability of overtopping and piping is not cost-effective. Only small, local investments may be cost-effective.

An investment in removing the barrier and heighten the dikes is more expensive than maintaining the barrier, even when a sea level rise of 1,0m appears. For the current sea level, by maintaining the barrier, benefits of 20 M€/year are needed to make it cost-effective. Removing the barrier and heighten the dike by satisfy to the current safety standard, will need benefits in the order of 90 M€/year. At a sea level rise of 1,0m the benefits for maintaining the barrier will be 22 M€/year (in case of no adaptations on the barrier) and for removing the barrier the benefits need to be 163 M€/year.

When the dike is heightened based on a cost-benefit analysis the costs of removing the barrier are lower. However, maintaining the barrier will be cheaper than removing the barrier.

[1] This is the Net Present Value at the time 0,5m sea level rise has appeared

[2] This is the Net Present Value at the time 1,0m sea level rise has appeared

Also when adaptations on the barrier have to be made, maintaining the barrier is the best option.

The monetary value of ecology is not taken into account in this study. However, in Paragraph 4.8 it is described that solving the problem of the erosion of the tidal flats by sand suppletion, will cost 45 M€/year. Assuming that the erosion process of the tidal flats will stop after removing the barrier, 45 M€/year is needed to maintain the same amount of tidal flats in case of maintaining the barrier. When adding these costs by the costs of maintaining the barrier, still the best investment option is to maintain the barrier. Only when the dike is heightened based on a cost-benefit analysis, and the barrier has to be adapted at a sea level rise of 1,0m (costs 1.000 M€), the costs of maintaining the barrier approximately equals the costs of removing the barrier. However, several side effects (e.g. adapting locks and construct small storm surge barriers, see Chapter 6) in case of removing the barrier are not taken into account. When these effects are taken into account, maintaining the in combination with sand suppletion is still cheaper than removing the barrier.

All investments lead to a negative Net Present Value, so neither maintaining the barrier nor removing the barrier is a good investment when looking at costs and benefits. An option which may lead to a Net Present Value of zero, the minimum amount to make it cost-effective, is to maintain the barrier without doing maintenance on the structure. The maintenance costs of 20 M€/year can be reduced in that case. This option is elaborated in the next paragraph.

5.2.1. Maintaining the barrier without doing maintenance

Maintaining the barrier without doing maintenance on the structure implies that the replaceable parts of the barrier (the gates, the cylinders) are not needed anymore. Only maintenance on the concrete parts may have to be done. The gates and the cylinders can be removed or left out of operation. This will have consequences on the extreme water level in the Eastern Scheldt.

The extreme water level statistics in case that the gates will be open, are obtained from FLORIS. The statistics for 'Roompot Binnen' are obtained, which is close to the barrier. To determine the statistics for Wemeldinge, the statistics are raised with 0,5m, like is done by determining the statistics in case of no barrier, see Appendix I.4.

The parameters of the dike are the same as used by other calculations in this study (see Table 8 and Table 9. Only the extreme water level statistics inside the Eastern Scheldt are different. The failure probabilities of overtopping of the dike, when the gates of the barrier are open and closed, are shown in
Figure 38. The failure probability of piping is shown in Figure 39. As can be seen the failure probability of overtopping increases with a factor 2,5 to 6,68e-7/year. The failure probability of piping increases with a factor of about 50 to 4,76e-4/year. The average failure probability of piping is exceeding the safety standard of 1/40.000. The average failure probability of overtopping is still far below the minimum safety required to satisfy the current safety standard. The risk of overtopping and piping becomes respectively 0,00 M€/year and 3,34 M€/year.

The failure probability of piping increases more when the gates are open, than the failure probability of overtopping. Piping is also relevant for lower water levels, the fragility curve is less steep (compare graph two of Figure 38 and Figure 39).

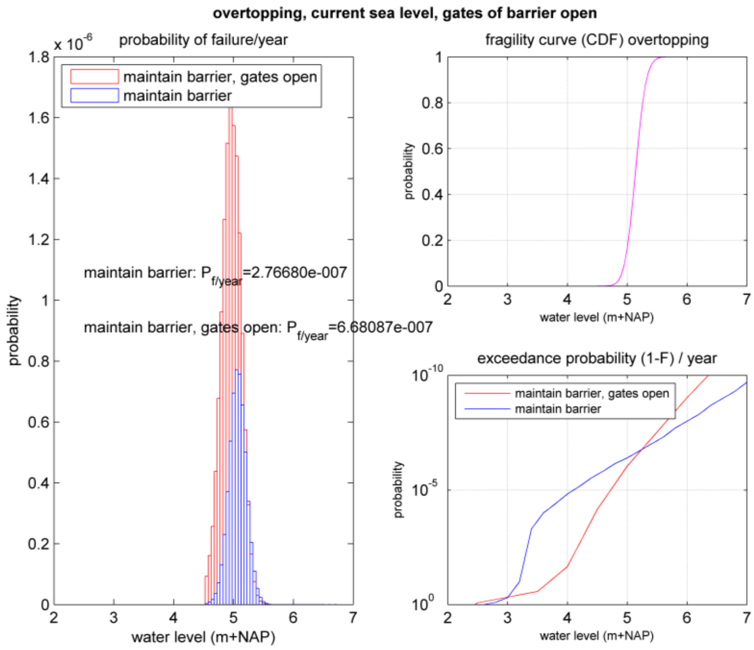


Figure 38: failure probability of overtopping by maintaining the barrier when the gates are closed and open at storm surges

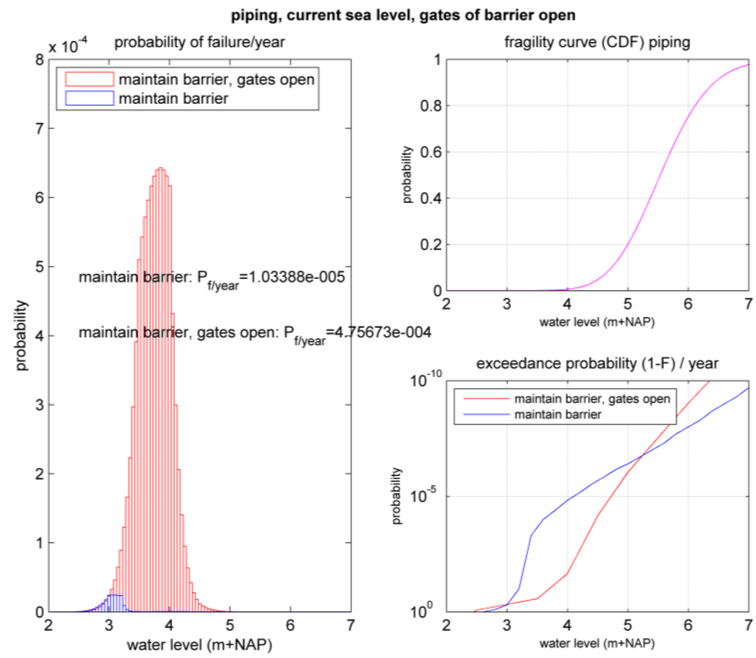


Figure 39: failure probability of piping by maintaining the barrier when the gates are closed and open at storm surges

As can be seen at the extreme water level statistics (third graph of Figure 38 and Figure 39) the exceedance probability at a water level between NAP +3,0m and NAP +5,0m, in case of open gates, is higher than in case of closed gates. When the water level exceeds NAP

+5,0m however, the water level statistics when the gates are open become more favourable. The reason for this is not known exactly. A possible explanation can be the failure of the gates. When the gates are closed, and one of the gates will fail, the velocity through the opening will be very high, due to the high water level difference between the North Sea and the Eastern Scheldt. This can cause failure of the structure. This process is not relevant anymore when the gates of the barrier are always open.

According to this calculation the failure probability of overtopping still satisfies the current safety standard of 1/4.000 year, but the failure probability of piping exceeds the standard of 1/40.000 year. The risk of overtopping and piping however, is still quite low, in the order of 3,5 M€/year. Maintaining the replaceable parts is more expensive (20 M€/year) than the risk of flooding when the gates are not used any more. When looking at costs and benefits a good option may be to take the gates out of operation. Piping preventing measures may have to be taken to reduce the failure probability of piping. It has to be noted that the calculated probabilities are average values. From FLORIS it appeared that at some locations the failure probabilities are higher.

5.3. Comparing the Eastern Scheldt to other closed tidal basins

The shortening of the coastal defence by building a dam or barrier for flood safety reasons is typical a Dutch concept. The first time it is applied to the Afsluitdijk in 1932. Beside an increase in safety another important argument for the closure was the reclamation of land. After the flood in 1953 the Grevelingen is closed by a dam, and the Haringvliet by discharge sluices to discharge the fresh river water to the North Sea. Also a couple of compartmentalization dams are build. In the north of the Netherland the Lauwersmeer is closed after the flood. The last time a shortening of coastal defence is applied by the construction of the Maeslantkering in 1992.

Closing a tidal basin for flood safety reasons is also applied outside the Netherlands. For instance Germany (Eider Barrage, 1973), England (Thames barrier, 1984), Russia (Saint Petersburg Dam, 2008) and Italy (the MOSE project, Venetia, under construction, 2014).

For a couple of closed tidal basins it is roughly investigated if removing the barrier/dam can be a good solution in case of sea level rise. The investigated areas are shown in Table 24. These basins are chosen because the removal of the barrier/dam can possibly lead to reduction in costs or an increase in ecology.

	type of closure	length	closed basin	shortening of coastal defence [km]	year
Eastern Scheldt	storm surge barrier	8,5 km	tidal basin	190 km	1986
Grevelingen	dam, discharge sluice	5,8 km	salt water lake	60 km	1971
Haringvliet	dam, freshwater sluices	3,5 km	fresh water lake	250 km	1969
Lauwersmeer	dam, freshwater sluices	13 km	fresh water lake	19 km	1969

Table 24: comparing other closed tidal basins with the Eastern Scheldt

5.3.1. Grevelingen

After the completion of the Brouwersdam the tidal basin Grevelingen became slowly a fresh water lake [HOEKSEMA, 2002]. Because of the poor water quality a discharge sluice is built to connect the lake Grevelingen with the North Sea. Nowadays the Grevelingen is a stagnant salt water lake. The water quality is quite high, the concentrations nitrogen and phosphate are below the test levels. Nowadays the lake Grevelingen is important for recreation. Surfing, diving and sailing are popular sports in the lake Grevelingen.

In comparison with the Eastern Scheldt the removal of the dam will be cheaper. However, the dikes are not heightened after the flood in 1953. The dikes will have to be heightened more in comparison with the Eastern Scheldt dikes. The flood risk of the Grevelingen dikes is currently very low, because the influences from tide and river are not present. So for safety, removing the barrier is not a good option. The maintenance costs of the dam are low in comparison with the Eastern Scheldt storm surge barrier. The population density is approximately equal to the dike-rings around the Eastern Scheldt.

5.3.2. Haringvliet

The ecological influence from removing the Haringvliet sluices was already part of an investigation [WWF, 2010]. From that study it appeared that an open Haringvliet will lead to ecological benefits of at least 500 M€/year [BOEHNKE-HENRICHS AND DE GROOT, 2010]. The costs (e.g. heightening of dikes and fresh water supply) are not investigated in that study.

Removing the Haringvlietdam will be a bit cheaper than removing the Eastern Scheldt storm surge barrier, regarding the length of the structure. The length of the dikes which have to be heightened will be longer. Currently the tidal influence (from the port of Rotterdam) in the river Waal is present till Zaltbommel. It is expected that the dikes will have to be heightened till at least Zaltbommel, for both the river Waal and Meuse, if the Haringvliet sluices will be removed. This covers a length of approximately 250 km of dikes. Suppose that the dikes have to be heightened with an average value of 1,0m and the costs are 9 M€/km per meter heightening of dike (see Table 12 for average costs), only the heightening of the dikes will cost 2250 M€.

5.3.3. Lauwersmeer

After the flood in 1953 it is decided to build a dam with discharge sluices to close off the Lauwerszee from tidal influences. The water slowly transformed from salt to fresh water. Currently the Lauwersmeer is an important national reserve; in 2003 it became a national park. Also recreation is important; the Lauwersmeer is popular for sailing and windsurfing[1].

When the dam and the discharge sluices (total length of 13 km) will be removed, 32 km of dikes will have to be heightened. This length of the dikes is low in comparison with the dikes around the Eastern Scheldt, so the costs of this project are expected to be lower. However, the new tidal area which will be obtained is also quite small. The ecological benefits are expected to be small, because the Lauwersmeer is currently an important ecological area.

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[1] http://nl.wikipedia.org/wiki/Lauwersmeer (assessed at 06-06-2012)

5.3.4. Conclusion

In comparison with the Eastern Scheldt the costs of removing the closure work of the Grevelingen, Haringvliet and Lauwersmeer will be cheaper. The length of the dikes which have to be heightened is also shorter for the Grevelingen and Lauwersmeer. For the Haringvliet it is estimated that 250 km of dike has to be heightened.

The flood risk of the Haringvliet, Grevelingen and Lauwerszee is lower than the flood risk at the Eastern Scheldt. In comparison with the Eastern Scheldt the tide is not important any more in the Haringvliet, Grevelingen and Lauwerszee. For flood safety reasons, re-open the Haringvliet, Grevelingen and Lauwerszee is not preferable.

The ecological and recreational value of the Grevelingen and Lauwerszee is high. It is not expected that re-open the area will greatly increase the ecological value. By [BOEHNKE-HENRICHS AND DE GROOT, 2010] it is expected that re-open the Haringvliet will lead to benefits of at least 500 M€/year. In that study the current economic value of the area is compared to the expected economic value when the Haringvliet will be re-opened. Assuming that the benefits of 500 M€/year are a good estimation, re-open the Haringvliet may be promising to further investigate.

6. SIDE EFFECTS OF THE ALTERNATIVES

Removing the barrier and heighten the dike will not only influence the flooding safety. In this section the implication on other subjects are described.

6.1. Recreation

Removing the storm surge barrier will have a small influence on recreation during normal conditions. The flow velocities and the waves will increase a bit. However, at extreme floods the water level in the Eastern Scheldt becomes much higher without storm surge barrier. This affects for instance holiday houses at the Roompot Marina harbour. A lot of holiday houses are situated at the Eastern Scheldt side of the dike-ring. These houses have to be moved inside the dike-ring area when the barrier will be taken away, or rebuild after extreme conditions.

In the Eastern Scheldt 7 yacht harbours are situated outside the dike-ring, see Figure 40. During normal conditions these yacht harbours will not be influenced when the barrier will be removed. However, at extreme water levels the harbours will be flooded. Retaining structures have to be built to prevent flooding.

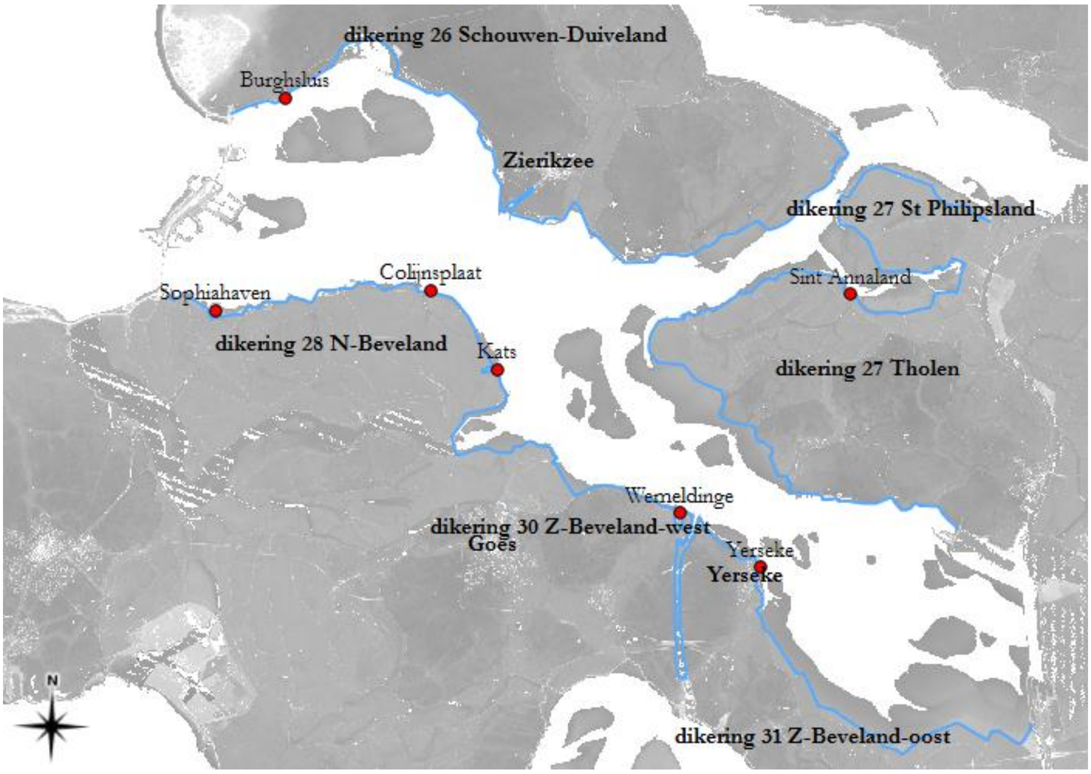

Figure 40: yacht harbours around the Eastern Scheldt

6.2. Ecology

As described in Section 1.1.2 the tidal flats in the Eastern Scheldt are eroding. Before the construction of the barrier the Eastern Scheldt was an erosion basin [TANCZOS *et al,* 2001], the Eastern Scheldt contained too much sand. That was caused by big floods (e.g. the Felixflood in 1530), dredging and canalizing from 1870-1960 and more recently the construction of the Grevelingendam (1965) and the Volkerakdam (1969). During that period the tidal volume increased. Because of the dikes the contour line of the Eastern Scheldt was fixed, so widening of the basin could not take place. Instead of that the tidal channels were deepened and the sand was transported to the sea. Between 1872 and 1983 about 340 million m^3 sand was transported to the sea [KOHSIEK *et al*, 1987]. In 1987 the construction of the barrier was completed. Also the Markiezaatdam was completed. The tidal volume decreased with 28% [NIENHUIS AND SMAAL, 1994]. The Eastern Scheldt then became a sedimentation basin, the channels are too wide for the tidal volume. However, due to the construction of the barrier sand transport is blocked. To fill the tidal channels sand has to come from inside the basin, which causes eroding of the intertidal flats. According to [JACOBSE *et al*, 2008] in 2060 most of the tidal flats will be disappeared.

Due to the erosion process the surface of the tidal flats decreases, as well as the period that the tidal flats are above the water level. The tidal flats are important for the flora and fauna in the Eastern Scheldt. Due to the decrease in area of tidal flats and the decrease in time when the flats are above the water level, the abundance of food for birds decreases. This is investigated for instance by [GEURTS VAN KESSEL, 2004]. The number of cockles decreased with 30% between 1985 and 2001. However, the number of Japanese oysters increased enormously. The number of oystercatchers decreased between 1980 and 2001 (Figure 41), however, this was caused by the oyster-fishery and some hard winters. An effect on other types of birds is not clearly visible. It is expected that due to the decrease in area of intertidal flats and the time when flats are above the water level, the number of birds will decrease [GEURTS VAN KESSEL, 2004].

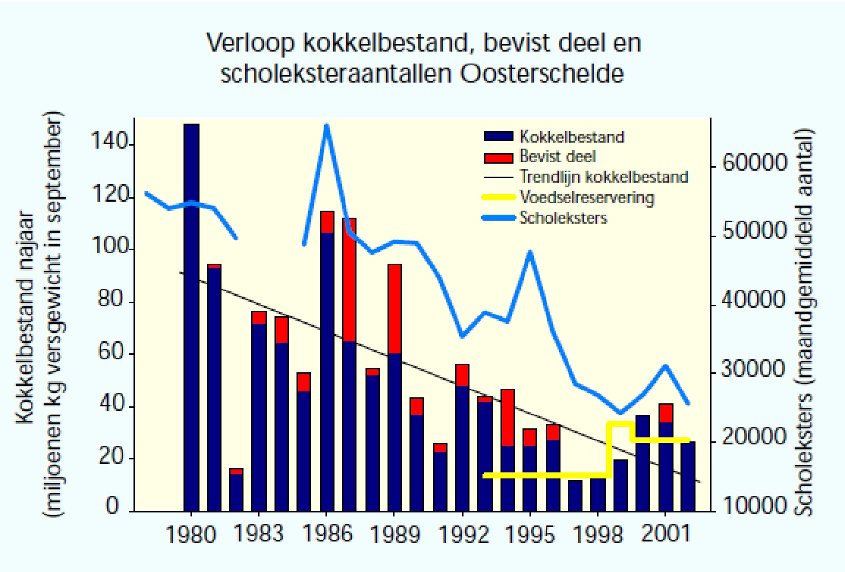

Figure 41: number of oystercatchers (bleu line) between 1985 and 2001 in the Eastern Scheldt

By the removal of the barrier sediment exchange between the Eastern Scheldt and the outer delta can take place. Also the tidal volume increases, which can stop the erosion process of the tidal flats. It is questionable whether removing the barrier will totally solve the problem of erosion of the flats. According to [LIEVENSE AND DEKKER, 2002] 60% from the reduction of the tidal volume is caused by the barrier and 40% by the compartmentalization dams.

There are also positive influences on ecology after the construction of the barrier. Due to the decrease of turbulence the water became clearer after construction of the barrier and the productivity of flora and fauna has not decreased. According to [Tanczos *et al*, 2001] the amount of fishes in the Eastern Scheldt is high, especially when comparing to the (more dynamic) Western Scheldt.

6.3. Hydraulic structures

6.3.1. Retaining structures for entrance harbour

In case that the barrier will be removed the dikes have to be heightened. This can have consequences for transport over the dike to reach the harbour form land side. Probably coupures have to be built, e.g. at harbours close to a village (Yerseke, Wemeldinge, Colijnsplaat).

6.3.2. Locks

The Eastern Scheldt is connected with other waters by locks. At the following places locks are situated:

- Oesterdam
- Zandkreekdam
- Kanaal door Zuid-Beveland
- Philipsdam
- Grevelingendam

Three harbours are situated inside the dike-ring: Zierikzee, Stavenisse and Het Sas. These harbours are protected from extreme water levels by locks.

In order to withstand higher water levels some of these locks may have to be adapted. Finding out where problems occur is beside this study.

6.3.3. Pumping stations

The Eastern Scheldt storm surge barrier is closed twice to provide a sufficient low water level in the Eastern Scheldt for the pumping stations[1]. At high water the pumping stations are not able to discharge the water from the polders into the Eastern Scheldt. When the barrier will be removed it is expected that these pumping stations have to be adapted to be able to discharge water at extreme storms. There are about 5 pumping stations located along the Eastern Scheldt[2].

6.4. Infrastructure

If the barrier will be totally removed the bridge connection over the barrier between Schouwen-Duiveland and Noord-Beveland will be removed as well. It is possible to replace another bridge, but that will be expensive. Another option is to use the Zeelandbrug which is located 15km to the East. The accessibility of parts of Zeeland will decrease in that case.

[1] http://www.hmcz.nl (assessed at 05-06-2012)

[2] http://www.quai.nl/gemalen/kaart.php (assessed at 05-06-2012)

6.5. Summary of side effects

In Table 25 a summary is visible of the different expected effects when the barrier will be removed and maintained. The costs of the different effects are not investigated.

	Maintaining barrier	Removing barrier
infrastructure	not affected	connection at barrier removed
intertidal flats	only in east part flats will be maintained [JACOBSE et al, 2008]	not known exactly, approximately the current area
adapted locks	not affected	8 locks may have to be adapted
pumping stations	not affected	5 pumping stations may have to be adapted
retaining structures	not affected	4 coupures may have to be build
recreation	not affected	1 recreation park has to be moved, 7 yacht harbours have to be prevented from extreme storms

Table 25: summary of side effects of maintaining and removing the barrier

7. DISCUSSION

This study focuses on decision options for the safety of the Eastern Scheldt dealing with sea level rise. Two alternatives are investigated: maintain the Eastern Scheldt storm surge barrier and remove the Eastern Scheldt storm surge barrier. These two alternatives were examined because of the influence from sea level rise and the influence from ecology. The question is to which extend the Eastern Scheldt storm surge barrier can withstand sea level rise. Possibly the barrier has to be adapted, or a new barrier has to be constructed, or the barrier has to be removed. For ecology the best option is to remove the barrier to acquire a highly dynamic open Eastern Scheldt.

The flood protection of dike-ring areas in both the alternatives can be obtained from the dikes around the Eastern Scheldt. When the barrier will be removed the dikes have to be higher than when the barrier will be maintained. From this study it appeared that the dikes don't have to be heightened if the barrier will be maintained, even at a sea level rise of 1,0m. Only small piping prevention measures may have to be taken. When the barrier will be removed, the amount of dike heightening for the current sea level, a sea level rise of 0,5m and a sea level rise of 1,0m is respectively 0,7m, 1,4m and 2,2m.

The actual safety is calculated by determining the average dike height in the Eastern Scheldt and determining the average hydraulic loads. With these parameters the probability of overtopping and piping of dikes is calculated at respectively values in the order of $3x10^{-7}$/year and $1x10^{-5}$/year.

For the failure probability of overtopping the average value for the dike height and the hydraulic load seems a very rough estimation. In the project FLORIS for instance it appeared that the probability of overtopping for dike-ring 26 varies in the order of 1/4000 and <1/1.000.000 year. This assumption however is justified by the relation between the hydraulic load and the dike height. When the hydraulic load is low, the height of the dike is also low, except for a few locations. Still the varying failure probabilities of FLORIS do not totally support this observation. It is assumed that the costs at locations for which the dike has to be heightened more than average will neutralize the costs for locations where the dike has to be heightened less than average. It is investigated if taking smaller parts with more less the same characteristics will lead to more accurate results but it appeared that only taking very small parts (taking dike sections, like is done by FLORIS) will lead to more accurate results. Taking such small parts is too detailed for this study, so using the average dike height and average wave load is the best option.

The failure probability of piping is calculated less accurate than the failure probability of overtopping. The probability of piping depends on the soil parameters below the dike, which are hard to determine. In this study the soil parameters are based on results from FLORIS for dike-ring 26. Average values are used for which it is assumed that they represent the other dike-ring areas around the Eastern Scheldt. For piping the same simplification is made as for overtopping. However, the relation between the resistance against piping and the soil parameters below the dike is less strong than it is for overtopping. By FLORIS for piping values in the order of 1/1.000 and <1/1.000.000 year are found in the current situation. The same assumption is done for piping as it is done for overtopping: locations for which the required length of the piping prevention measures is bigger than the average length are neutralized in costs for locations where smaller measures are needed.

The extreme water level statistics in the Eastern Scheldt are obtained from literature. The statistics are based on a risk analysis on the storm surge barrier and are determined quite accurately. The water level statistics when sea level rise appears are determined by doing assumptions. The main assumption is that the barrier fails when the outer water level exceeds NAP+5,8m. In the case of sea level rise this level is reached with a higher exceedance probability. To acquire more detailed statistics a full risk analysis on the barrier has to be done with the hydraulic boundary conditions for the amount of sea level rise. It is expected that the assumed statistics for sea level rise will differ from statistics which are determined by a full probabilistic approach, because a lot of other processes play a role in the statistics.

In this study it is assumed that waves in the Eastern Scheldt are still internally generated in the case that the barrier will be removed. However, high waves from the North Sea are not influenced by the barrier anymore and are able to enter the Eastern Scheldt during extreme storms. This will need a higher crest of dikes at the mouth of the Eastern Scheldt than is calculated for in this study. This leads to an increase in costs of dike heightening if the barrier will be taken away. It is expected that this effect will not influence conclusions done in this study.

Some of the costs are calculated quite accurate (dike heightening, piping) and some costs are based on intuition (removing the barrier and adapting the barrier). The costs of removing the barrier are never estimated accurately. A cost estimation for adapting the barrier depends on the exact measures which have to be taken and beside that it is hard to determine the costs of such a big adaptation. Because of the amount of expected costs these measures can be sensitive for the final result of this study. To check the sensitivity from adaptations on the barrier on the conclusion, the costs of adaptations are included. It appeared that the choice for an open or closed Eastern Scheldt does not change for the current sea level, a sea level rise of 0,5m and a sea level rise of 1,0m. For the costs of adapting the barrier a value of 1000 M€ is used. It is expected that the upper sill and probably the traffic structure will have to be adapted. The costs of adapting the barrier are determined on intuition. Regarding the total costs of the barrier (NPV: 5,5 billion €), this is seen as a conservative estimation.

All the investment alternatives are resulting in a negative Net Present Value. That is caused by the current low failure probability of flooding. Before the construction of the barrier the dikes are heightened to a safety level of 1/500 year. Due to the construction of the barrier extreme water levels in the Eastern Scheldt are much lower. Also results from FLORIS show the current low failure probability. Currently the risk of flooding for all the dike-rings is in the order of €80.000/year. An investment in the safety of the Eastern Scheldt can only be cost-effective if the costs of the investments are low.
The consequences of a flooding are obtained from WV21. This study uses 'state of the art' models to determine the consequences of a flood. It is hard to determine the accuracy of the estimations from the model, because the last big flood was the flood in 1953. When comparing the number of casualties calculated by the model with the casualties in 1953, the model gives a much lower number of casualties. For instance for Schouwen-Duiveland the model estimates a number of 60 casualties, while in 1953 543 people died by the flood. This may be explained by the increased quality of houses, the increase of mobility and warning systems. However, the population of Zeeland since 1953 has grown. The total consequences of a flood for damage and people for dike-rings around the Eastern Scheldt

are determined at 7025 M€. It is not expected that the relatively low risk calculated in this study comes from an underestimation of the consequences.

In this study the benefits from ecology are not taken into account, because this study focuses on safety. However, an interesting parameter to determine the value of ecology is the cost of supplying the tidal flats with sand, in case of maintaining the barrier. The costs of supplying the area with sand are estimated by [DELTACOMMISSIE, 2008] at 45 M€/year. Suppose that the monetary value of ecology after removing the barrier equals the monetary value of ecology by maintaining the barrier and suppletion of sand, the ecological benefit from an open Eastern Scheldt is 45 M€/year. However, it is questionable if the erosion process will stop after removing the barrier. Due to the construction of the compartmentalization dams the channels are too wide for the tidal volume in case of removing the barrier.

A benefit which is hard to determine by an amount of money, but does not have to be underrated, is the image from the barrier on the expertise of the Dutch knowledge on flood defenses.

8. CONCLUSIONS

In this chapter the main results will be presented. Also some recommendations are done for further research.

- The current flooding safety of dike-rings around the Eastern Scheldt is very high. The average exceedance probability for overtopping of the dikes around the Eastern Scheldt is in the order of $3x10^{-7}$/year and for piping in the order of $1x10^{-5}$/year.

- Because of the low failure probability and the relatively low economic value and inhabitants of the dike-rings around the Eastern Scheldt, the risk of flooding is low; in the order of €80.000/year. Only local investments in the safety of dike-rings around the Eastern Scheldt can be cost-effective.

- For a sea level rise of 1,0m the barrier has to close approximately 30 times per year when the current closing strategy of closing at NAP +3,0m will be maintained. Closing at a higher level implies that the water level statistics for the Eastern Scheldt will be less favourable. However, from this study it appeared that the crest height of the dikes satisfies the current safety standards, even in the case of 1,0m sea level rise. Only small piping prevention measures may have to be taken when the sea level rises with 1,0m.

- When the barrier will be removed, the dikes have to be heightened for the current sea level, a sea level rise of 0,5m and a sea level rise of 1,0m, with an average amount of respectively 0,7m, 1,4m and 2,2m, based on the current safety standards.

- For the current sea level and a sea level rise of 0,5m, the best investment option is to maintain the Eastern Scheldt storm surge barrier. Benefits of removing the barrier need to be in the order of 90 M€/year for the current sea level and 125 M€/year for a sea level rise of 0,5m to make that decision cost-effective. The required benefits by maintaining the barrier are 20 M€/year.

- Whether the structure of the barrier can deal with a sea level rise of 1,0m is not sure. However, when the barrier has to be adapted, still the best investment option is to maintain the barrier. For the costs of adapting the barrier a conservative price of 1000 M€ is used. Adaptations may have to be made on the upper sill and the traffic structure. Benefits in the order of 165 M€/year are needed when the barrier will be removed. When the barrier will be maintained, benefits in the order of 22-75 M€/year are needed, depending on adaptations on the barrier.

- Dike heightening based on a cost-benefit analysis instead of the current safety standards, will lead to a reduction in the amount of heightening if the barrier will be removed. At the current sea level an integral dike heightening is not needed. Only at some locations the dike may have to be heightened. At a sea level rise of 1,0m the reduction can be 0,9m. Optimum failure probabilities in the order of 1/500 year are found. Maintaining the barrier is still the cheapest alternative.

- The benefits from ecology are not taken into account in this study. However, when the barrier will be maintained and the tidal flats are supplied with sand to prevent eroding, still the best investment alternative is to maintain the barrier.

- In this study it is investigated what the impact is when the gates are not closed at extreme storms. It appeared that the average probability of overtopping of dikes is $7x10^{-7}$/year in that case. The average failure probability of piping will become $5x10^{-4}$/year. According to the safety standards dike heightening is not needed; only piping preventing measures may have to be taken. If the gates are not needed anymore a lot of reduction can be obtained for the current maintenance costs of approximately 20 M€/year.

- The removal of the barrier will not only require higher dikes, but also locks have to be adapted and small storm surge barriers have to be built for the yacht harbours.

Recommendations

- It is not investigated thoroughly if the barrier can deal with a sea level rise of 1,0m. This has to be investigated to acquire accurate costs of adapting the barrier.

- From this study it appeared that the average probability of overtopping of the dikes, in the case that the gates are not closed at extreme storms, is $7x10^{-7}$/year and for piping $5x10^{-4}$/year. It is recommended to further investigate the consequences of an open storm surge barrier.

- Beside the Eastern Scheldt a couple of other tidal basins are closed by a dam or a barrier, as well in the Netherlands as abroad. A possible interesting basin to investigate is the Haringvliet in the Netherlands. According to [WWF, 2010] the benefits from an open Haringvliet are at least 500 M€/year.

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APPENDICES

I. EXTREME WATER LEVEL STATISTICS

I.1. Current statistics

The statistics for extreme water levels in the Eastern Scheldt can be determined based on a risk analysis on the storm surge barrier. The storm surge barrier closes when a water level of NAP +3,0m is expected. In principal the waterlevel in the Eastern Scheldt will become stagnant after closure, however, due to failure of gates, failure of management of the gates, leakage through the barrier and overtopping the water level can rise during closure and will cause water levels higher than NAP +3,0m. Calculations for extreme water level statistics are done several times and can be found for instance in [RIJKSWATERSTAAT, 2008b]. The results from this report for location Wemeldinge are shown in Figure 42.

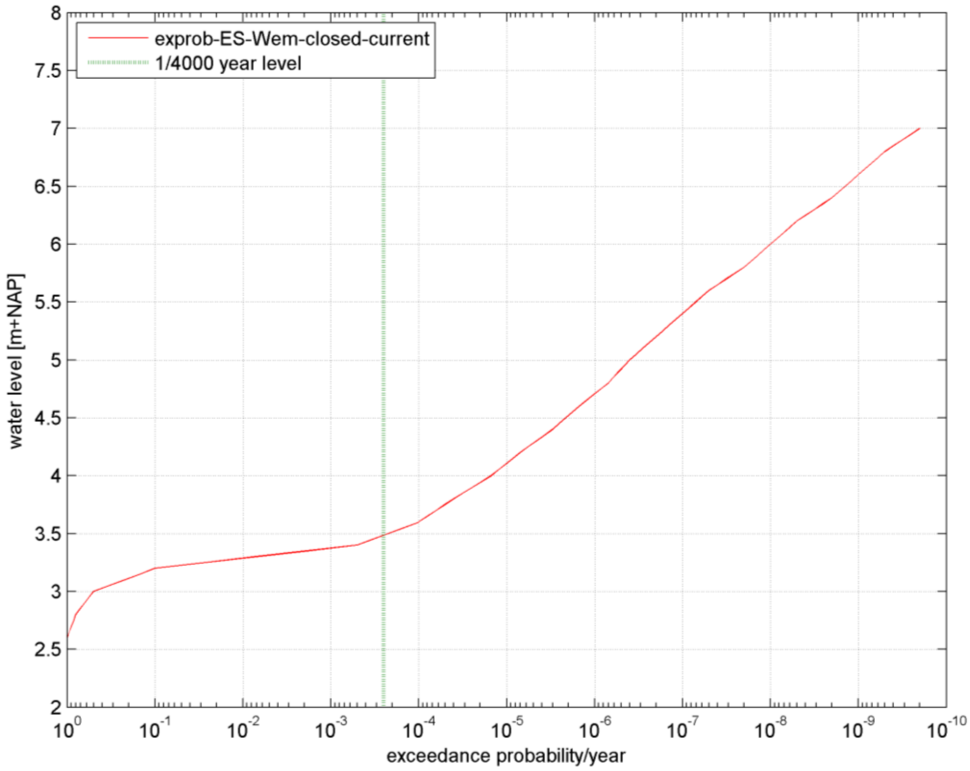

Figure 42: current extreme water level statistics for Wemeldinge

I.2. Statistics without barrier

When calculating the failure probability of dikes when the barrier will be removed the statistics for an open Eastern Scheldt have to be known. After the flood in 1953 the Deltacommision studied extreme water levels in the Eastern Scheldt. In [DELTACOMMISSIE, 1960b] statistics for the Eastern Scheldt are calculated. In Figure 43 the results are shown.

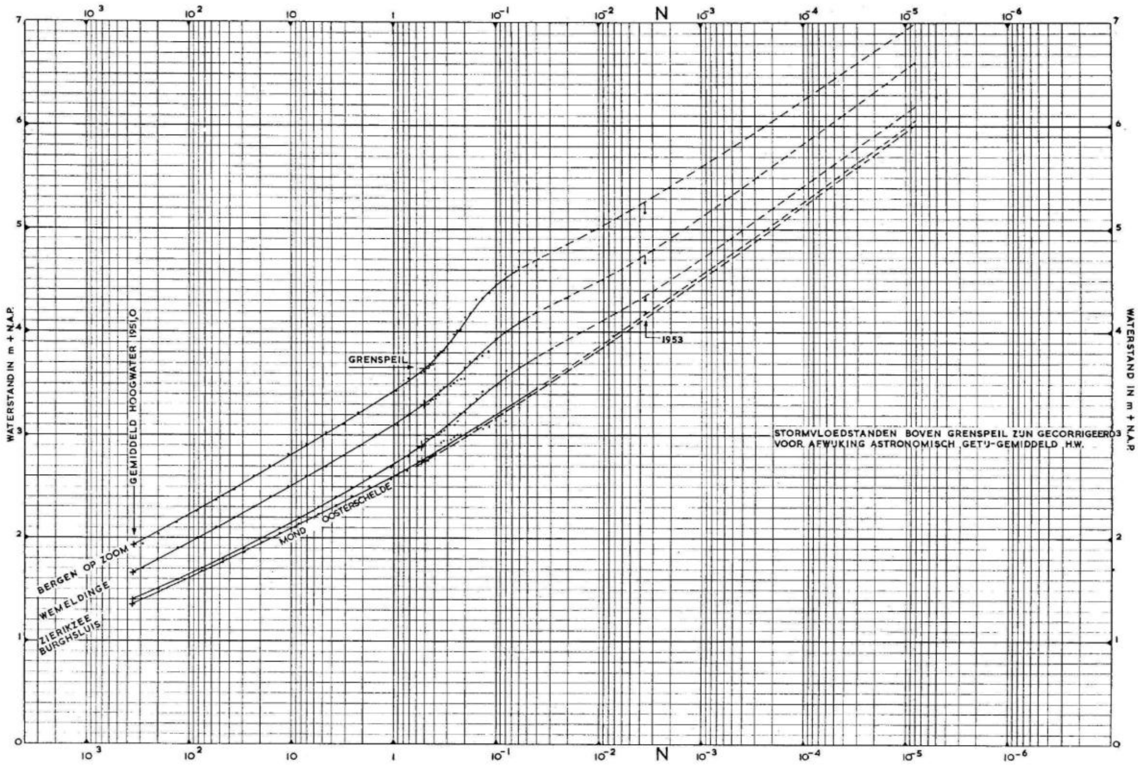

Figure 43: exceedance probability of water level in Eastern Scheldt according to [DELTACOMMISSIE, 1960b]

In this study the statistics calculated by [DELTACOMMISSIE, 1960b] are not used, because it appeared that these statistics are not useful anymore. When comparing the current statistics outside the barrier (Figure 45) with the statistics from Figure 43 (Mond Oosterschelde) quite a lot difference is visible. Instead of using the statistics from [DELTACOMMISSIE, 1960b] the statistics from Waternormalen for Roompot Buiten (just outside the barrier) are used. However, these statistics are not valid for Wemeldinge. In Figure 44 the difference in statistics between Roompot Binnen (close to the barrier) and Wemeldinge is shown. As can be seen the statistics differ with approximately 0,5m. Also in the calculations done by [DELTACOMMISSIE, 1960b] this difference is visible, see Figure 43.

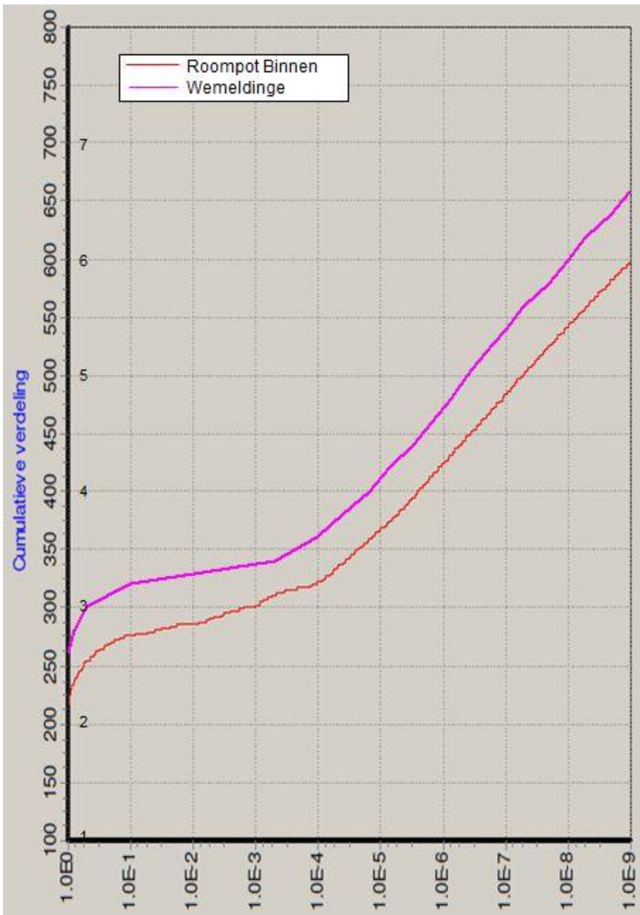

Figure 44: current statistics at Roomport Binnen (close to the barrier) and Wemeldinge (middle of basin)

Based on these results the statistics for Roompot Buiten are raised with 0,5m to determine the statistics for Wemeldinge. The result is shown in Figure 45.

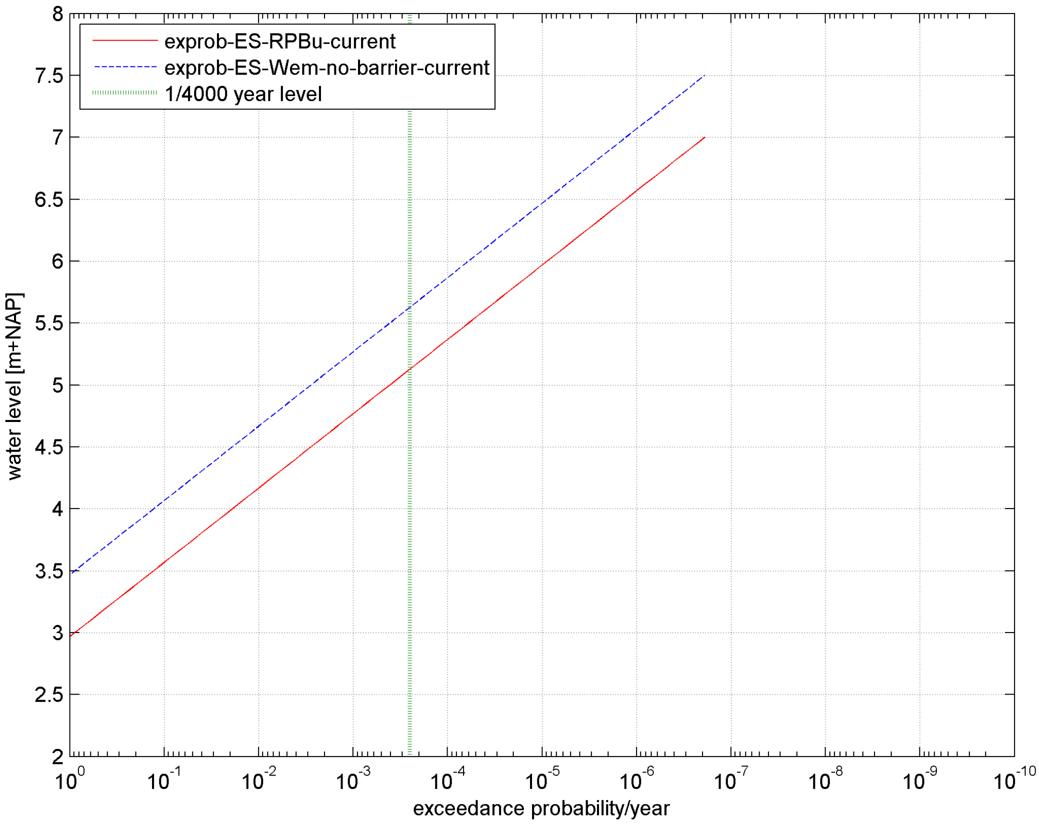

Figure 45: extreme water level statistics for an open Eastern Scheldt at location 'Wemeldinge'

I.3. Statistics for sea level rise

To calculate the flooding probability in the case of sea level rise, extreme water level statistics need to be available. To calculate the statistics accurate, probabilistic models are needed. In this study these models could not be used. Instead of that assumptions need to be made.

For the situation without barrier the statistics are simply raised with the amount of sea level rise. It is possible that this assumption does not hold because in the Eastern Scheldt the wind setup depends a bit on the water depth. Possibly less wind-setup occurs in the case of deeper water depths. However, for this study the influence from water depth is neglected.

For the situations that the barrier will be maintained the statistics are more difficult to determine. Simply raising the statistics with the amount of sea level rise is not correct, because the inner water levels are influenced by the barrier. The statistics are determined based on the following reasoning:

For a sea level rise of 0,5m it is assumed that the barrier maintains its current characteristics. This means that the closing regime remains the same, so closure at NAP +3,0 and the 1-2-1 strategy, see Section 3.4. It is assumed that leakage does not influence the statistics significantly. This assumption is based on experts from Rijkswaterstaat Zeeland[1]. This as-

[1] K. Saman, interview at 25-4-2012

sumption implies that till an outer water level of NAP +5,8m is reached, the statistics in the Eastern Scheldt remains the same. At higher water levels the barrier is overflowing and is going to fail, because it is not designed to withstand higher water levels. In the case of 0,5m sea level rise a water level of NAP +5,8m occurs more often from a statistic point of view. So the safety of the structure decreases a little bit. This effect is accounted for in the statistics, see Figure 46. It has to be noted that a water level at the mouth of the Eastern Scheldt of NAP +5,8m corresponds with a water level of NAP +6,3m at Wemeldinge.

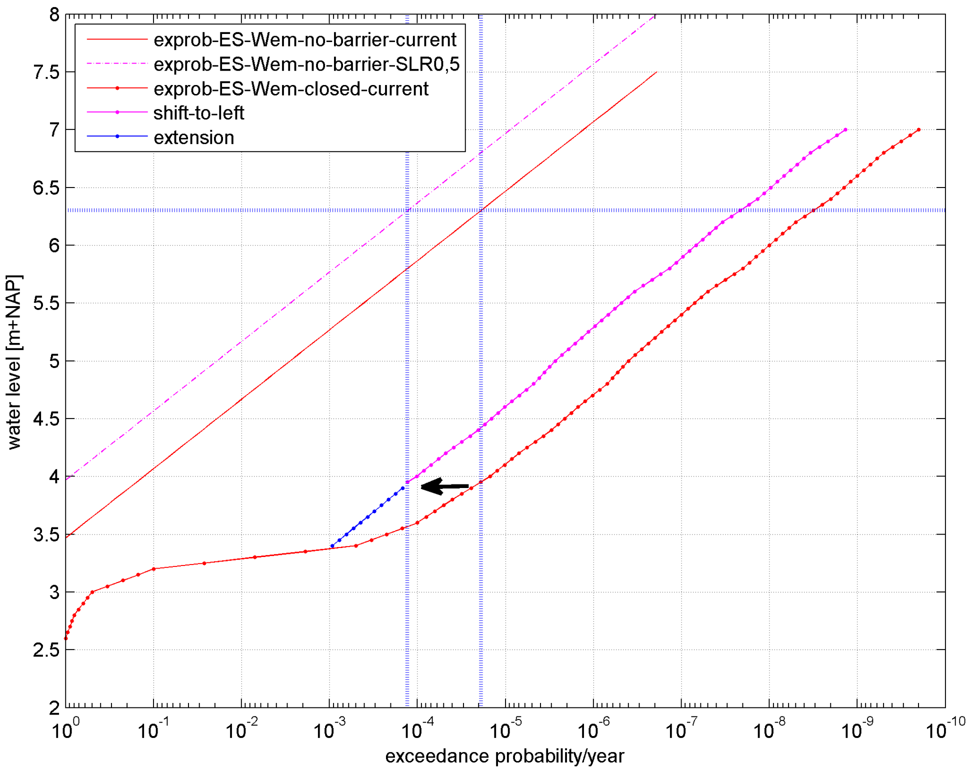

Figure 46: Statistics at 'Wemeldinge' for 0,5m sea level rise

At a sea level rise of 1,0m the barrier has to close approximately 30 times per year, see Section 3.4. Closing at an outer water level of NAP +3,0m cannot be maintained in that case. It is assumed that closing takes place at a level of NAP +3,5m. In that case the barrier has to close approximately 5 times per year. The 1-2-1 strategy in the Eastern Scheldt also has to be adapted, because often the outer tidal level will be above NAP +1,0m. Closing at NAP +3,5m influences the statistics. When the water level does not exceed NAP +5,8m it is assumed that the difference in closing regime leads to a rise of water levels for a certain exceedance probability. When the water level exceeds NAP +5,8m the same approach is applied as used by a sea level rise of 0,5m, so a shift to the left, see Figure 47.

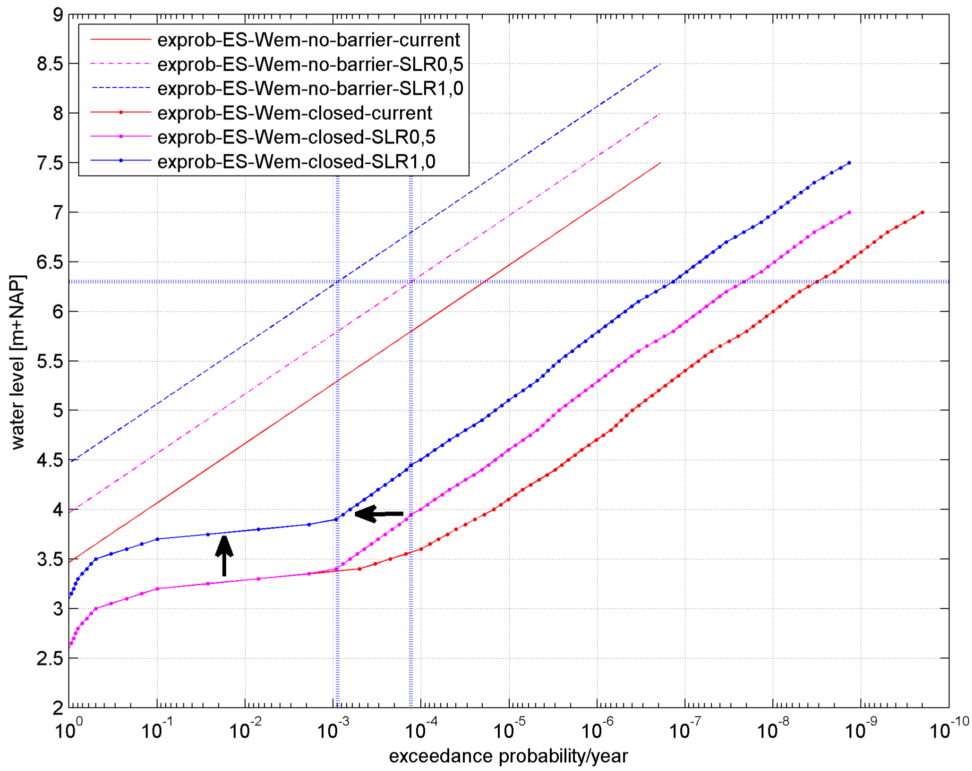

Figure 47: statistics for 1,0m sea level rise

The statistics which are used in this study are an approximation. For the purpose of this study these statistics can be used. More detailed studies require a full probabilistic calculation.

I.4. Statistics when gates of the barrier are open

In Section 5.2.1 it is calculated what the failure probability is when the gates of the barrier are open in case of extreme storms. The extreme water level statistics which are used for this calculation are obtained from FLORIS. The extreme water level statistics from 'Roompot Binnen' are obtained, which is close to the barrier. To determine the statistics for Wemeldinge the statistics are raised with 0,5m like is done in case of no barrier, see Appendix I.2. The statistics are shown in Figure 48. Also the statistics when the gates of the barrier are closed are shown.

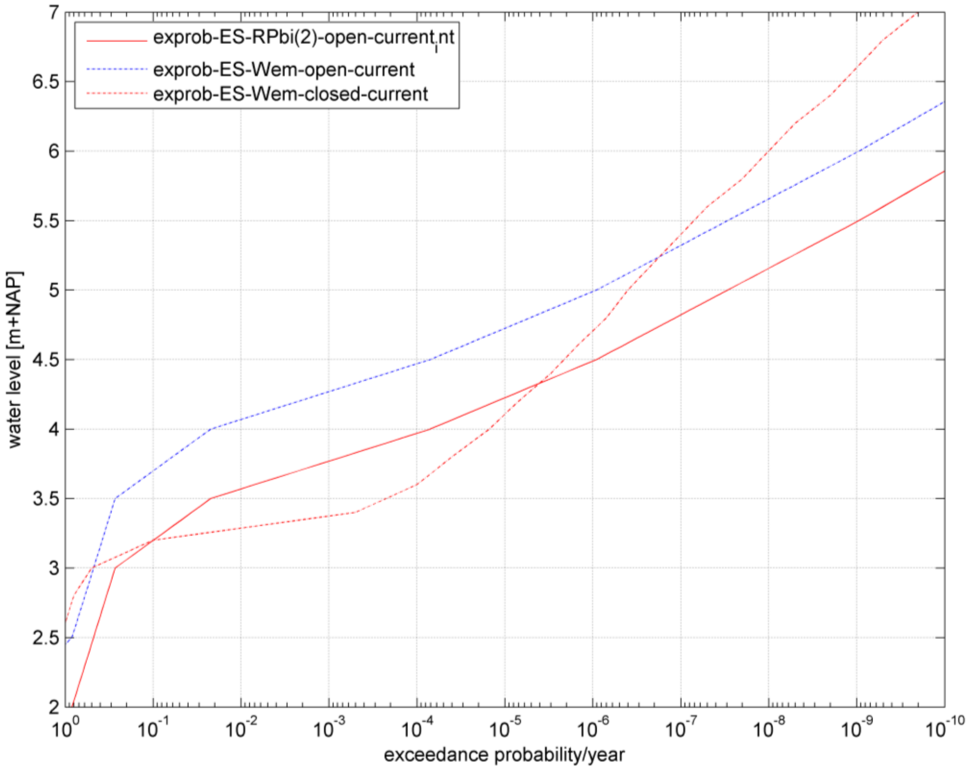

Figure 48: statistics when the gates of the barrier are open for location 'Wemeldinge' and 'Roompot Binnen'

II. DIKE PARAMETERS

II.1. Critical overtopping discharge

In the Netherlands the crest height of a dike is determined based on a critical overtopping discharge of 0,1-1 l/s/m. Recently a lot of tests are done with a wave overtopping simulator. It is a system designed by J.W. Van der Meer which can simulate overtopping waves quite precisely. With this simulator overtopping discharges from 0,1 l/s/m to 50 l/s/m can be simulated [AKKERMAN *et al*, 2007]. By updating the simulator nowadays also higher discharges can be simulated.

The Netherlands 2007-2009
In the Netherlands tests are performed in Delfzijl, Friesland, Zeeland and the Afsluitdijk. Tests were done on different dikecovers, such as normal grass, clay, near a stair case, etc. During this tests no dike failed for a mean overtopping discharge of 30 l/s/m or less. Only one section failed at 50 l/s per m; some at 75 l/s per m, but part of the sections did not fail, even not for 75 l/s per m [VAN DER MEER *et al*, 2009].

Vietnam, 2009
Destructive overtopping tests have been performed on two different sea dikes in Vietnam. The mean overtopping discharges were carried out increasing from small to large: 10; 20; 40; 70; 100 and 120 l/s per m. The resistance to overtopping discharge was different for the locations, varying from 20 to 70 l/s/m. One location showed no damage for overtopping discharges of 120 l/s/m [TRUNG *et al*, 2011].

Belgium, december 2010
Tests were done on riverdikes along the river Scheldt, with overtopping discharge from 1 to 50 l/s/m. On primary dikes no damage occurred for overtopping discharges up to 50 l/s/m. On ringdikes the slopes started to fail at overtopping discharges of 10 l/s/m [STEENDAM *et al*, 2011].

US 2010/2011
Tests were done on slope covers clay and different types of grass. On bare clay severe erosion occurred by an overtopping discharge of 19 l/s/m. However, Bermuda grass for instance did not show erosion even at discharges of 370 l/s/m [THOMTON *et al*, 2011].

Conclusion
An overtopping discharge of 0,1-1 l/s/m is too conservative. Even bare clay with grass roots could withstand overtopping discharges of 30 l/s/m [VAN DER MEER *et al*, 2009]. It is hard to determine which overtopping discharge is normative. Objects such as stair cases and local holes in the dike are weak spots which are less resistant. To be on the safe side a mean overtopping discharge of 10 l/s/m is chosen as input for the risk calculations.

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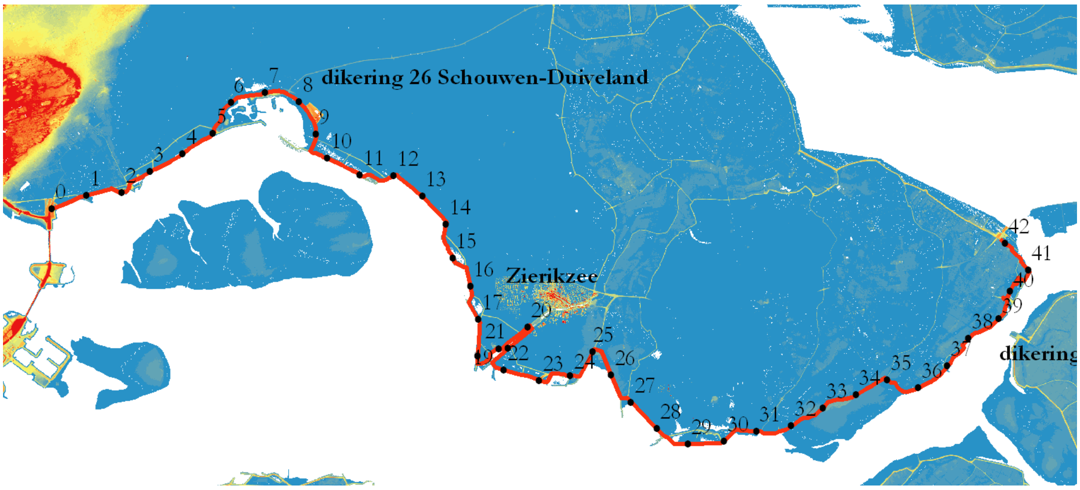
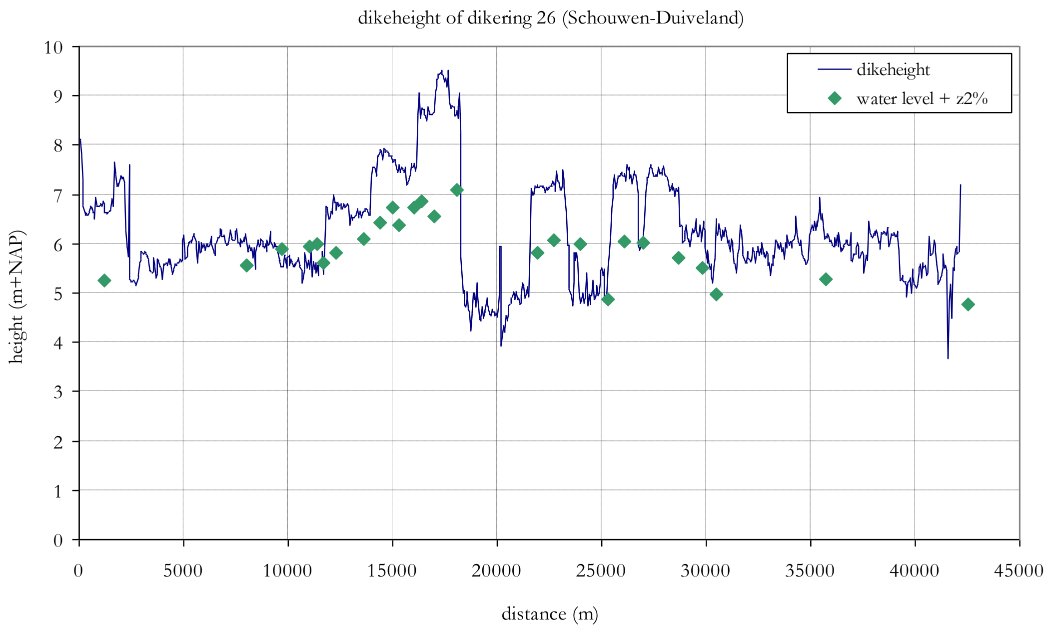
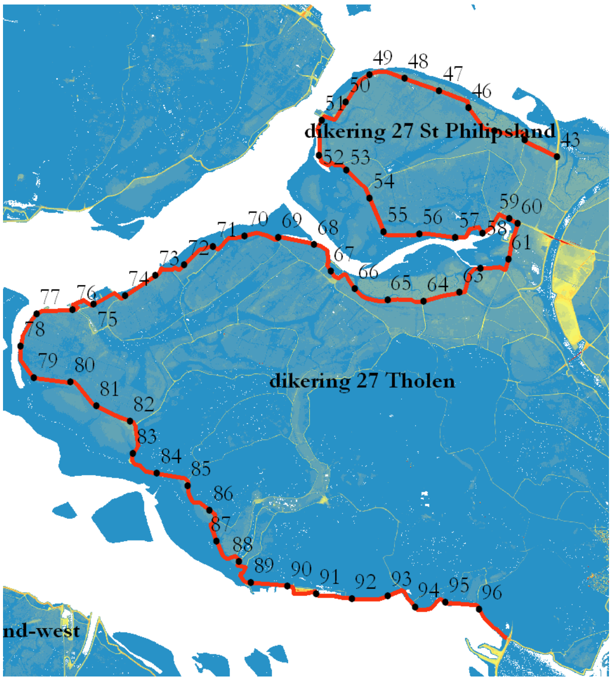
II.2. Dike height

The dike height differs a lot along a dike-ring. The dike height is determined by drawing a line over the top of the dike on data from the AHN1 5x5 raster. Every rasterpoint arcGIS takes the value of the raster. By selecting the highest value every 50 meter representative dike heights are determined, see the figures below. The validity of this method is checked and can be seen in Appendix II.3. In the figures also the combination of the 1/4000 year water level and run-up height [z2%] is plotted. This is actually the required dike height. The run-up height is calculated with a slope angle of 1:3, a berm-width of 5 meter and a berm height of NAP +4,5m. The hydraulic loads from HR2006 are used. As can be seen in the figures there is a relation between the dike height and the hydraulic load, however, this relation is not always consistent.



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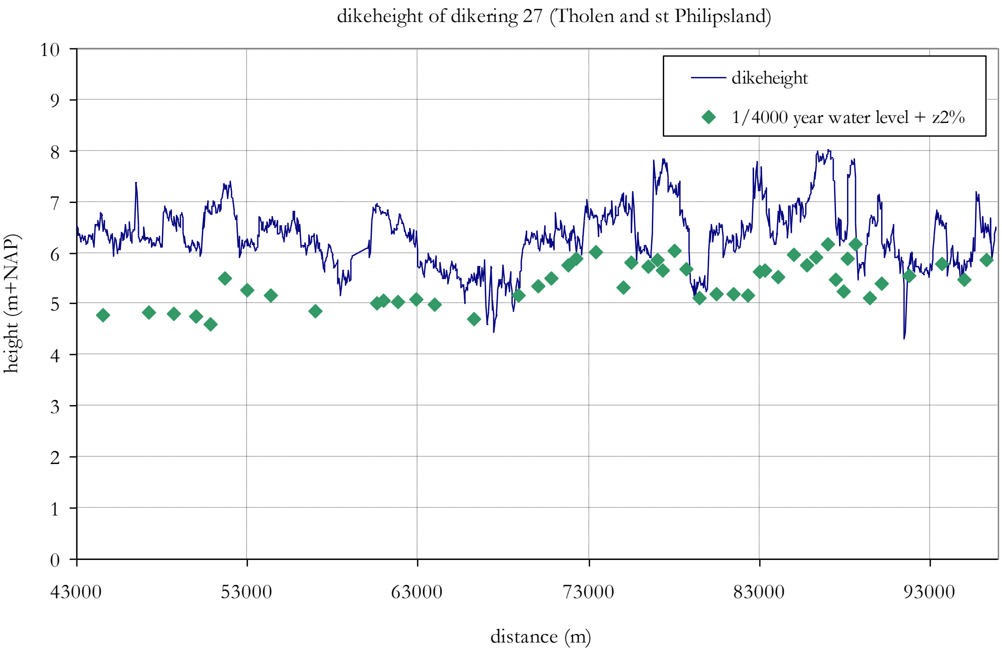
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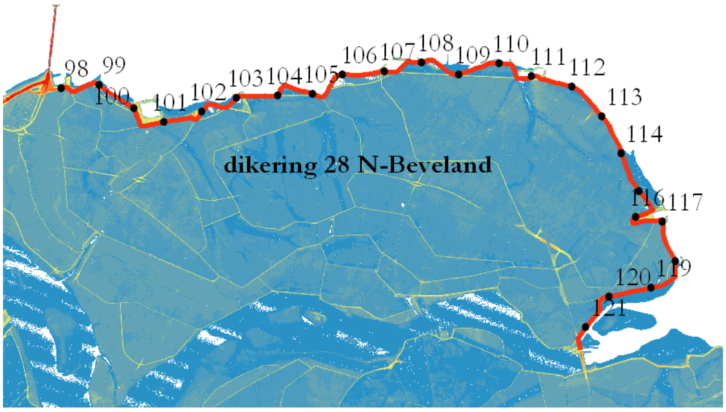
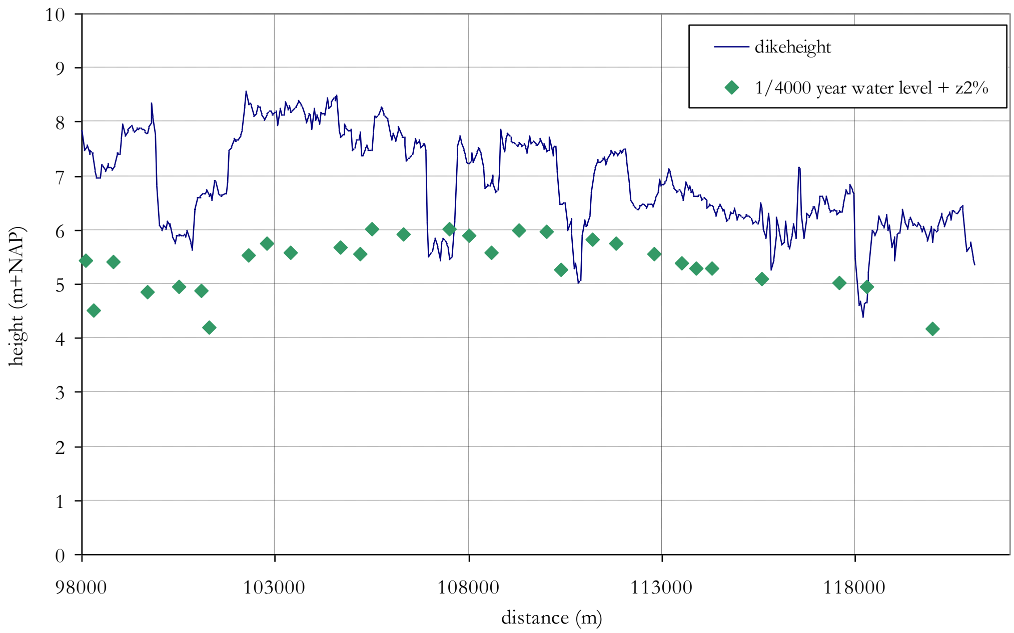
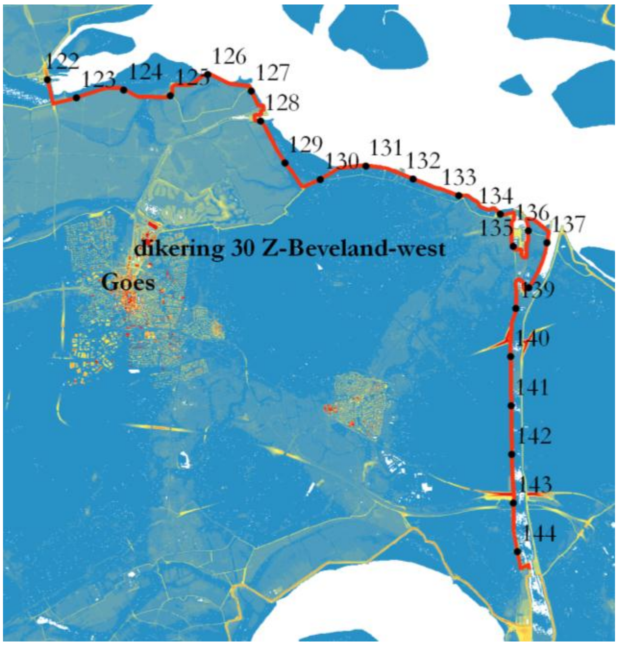
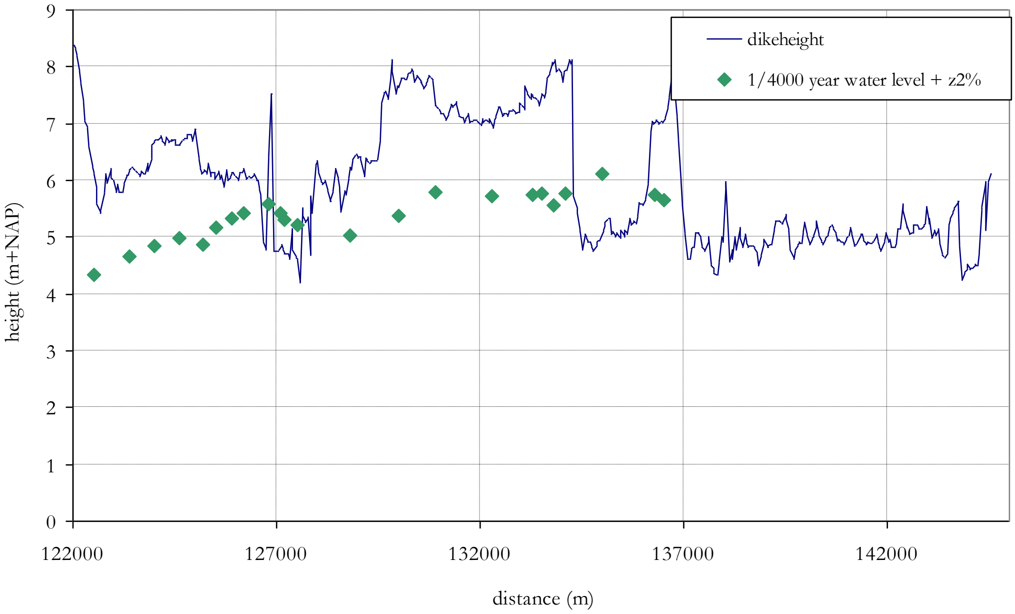


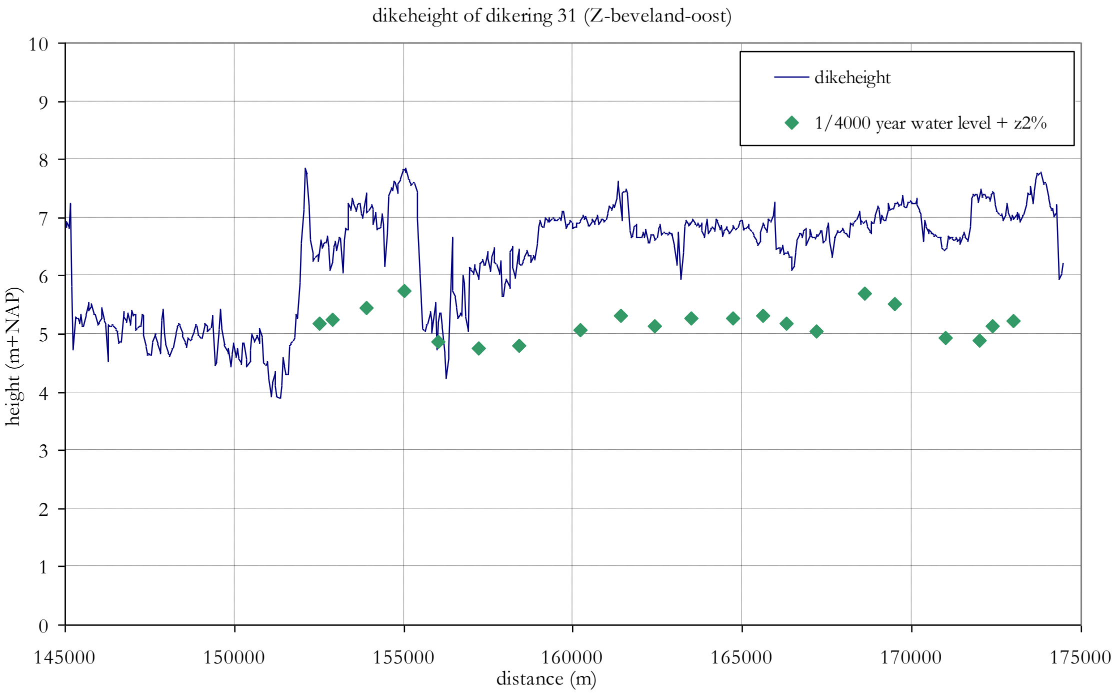
dikeheight of dikering 28 (N-beveland)





dikeheight of dikering 30 (Z-beveland-west)





At most of the places the dike height is a lot higher than the required dike height according to the current safety standards. At a couple of places the combination of the 1/4000 year water level and the run-up height z2% is exceeding the dike height. This is not necessarily a weak spot; at most of these locations a harbour is present. The hydraulic loads are given outside the harbour. Inside the harbour the wave attack is much lower.

The question rises which dike height has to be taken to calculate the failure probability of the dikes. It will cost too much time and it will be too detailed for this study to calculate the failure probability for parts with equal hydraulic conditions and dikeparameters. So a simplification has to be made. It is investigated whether it is possible to divide the dikes in smaller parts with more or less the same conditions. However, the dike height and hydraulic conditions differ too much to get more accurate results. Instead of that one representative dike height is chosen (NAP +6,4m) which represent all the dike-ring areas. This choice is justified by the relation between the dike height and the hydraulic condition. At locations with a higher hydraulic load the dike height is also higher which leads to approximately the same failure probability. For sure this relation does not hold for every location. These locations are neglected.

II.3. Difference between AHN1 and AHN2

In Figure 50 plots are shown from dikeprofiles at exact the same locations for AHN1 data and AHN2 data. The locations are shown in Figure 49. As can be seen the AHN1 data is much less detailed, sharp profile changes are not visible. For defining a dikeprofile the AHN1 data is not detailed enough.

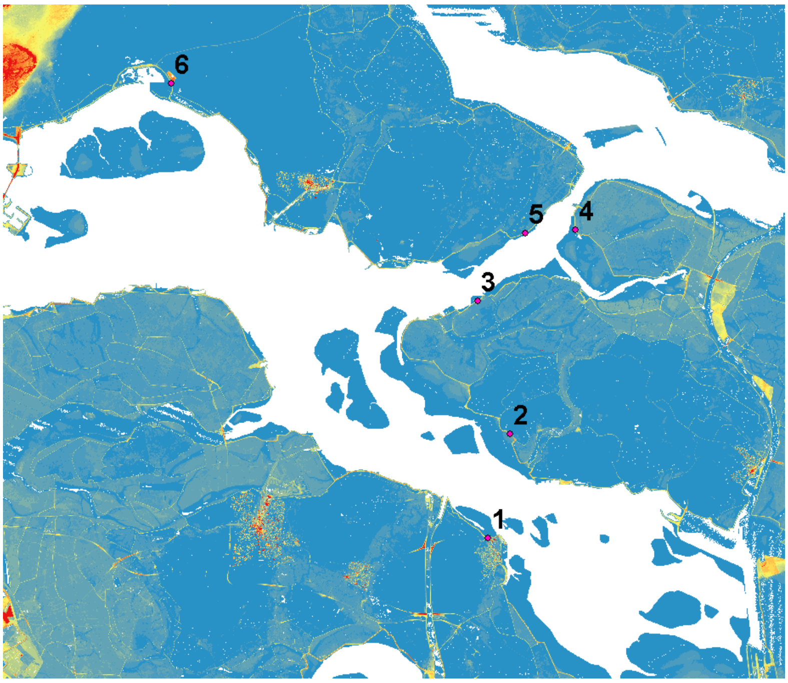
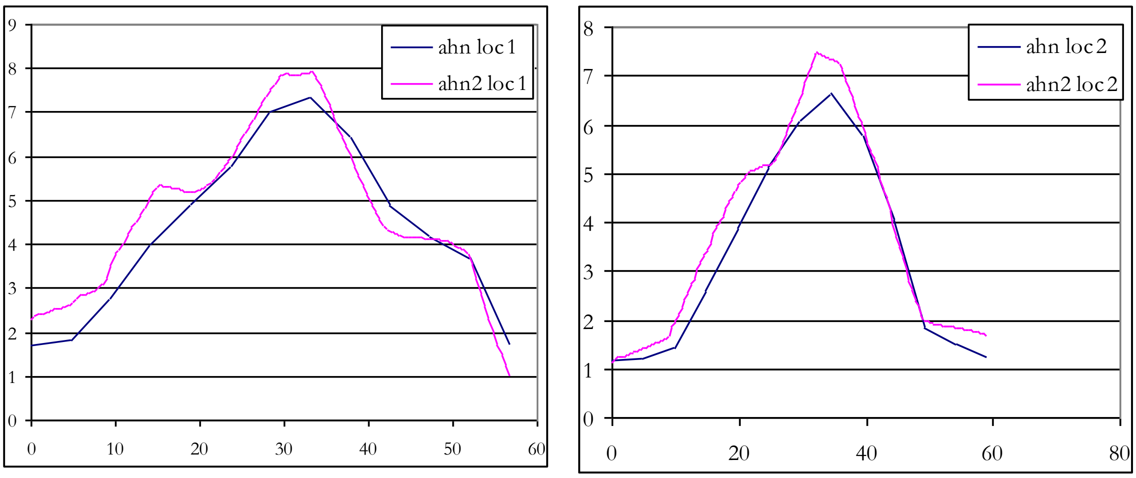


Figure 49: locations of the inspected dikeprofiles



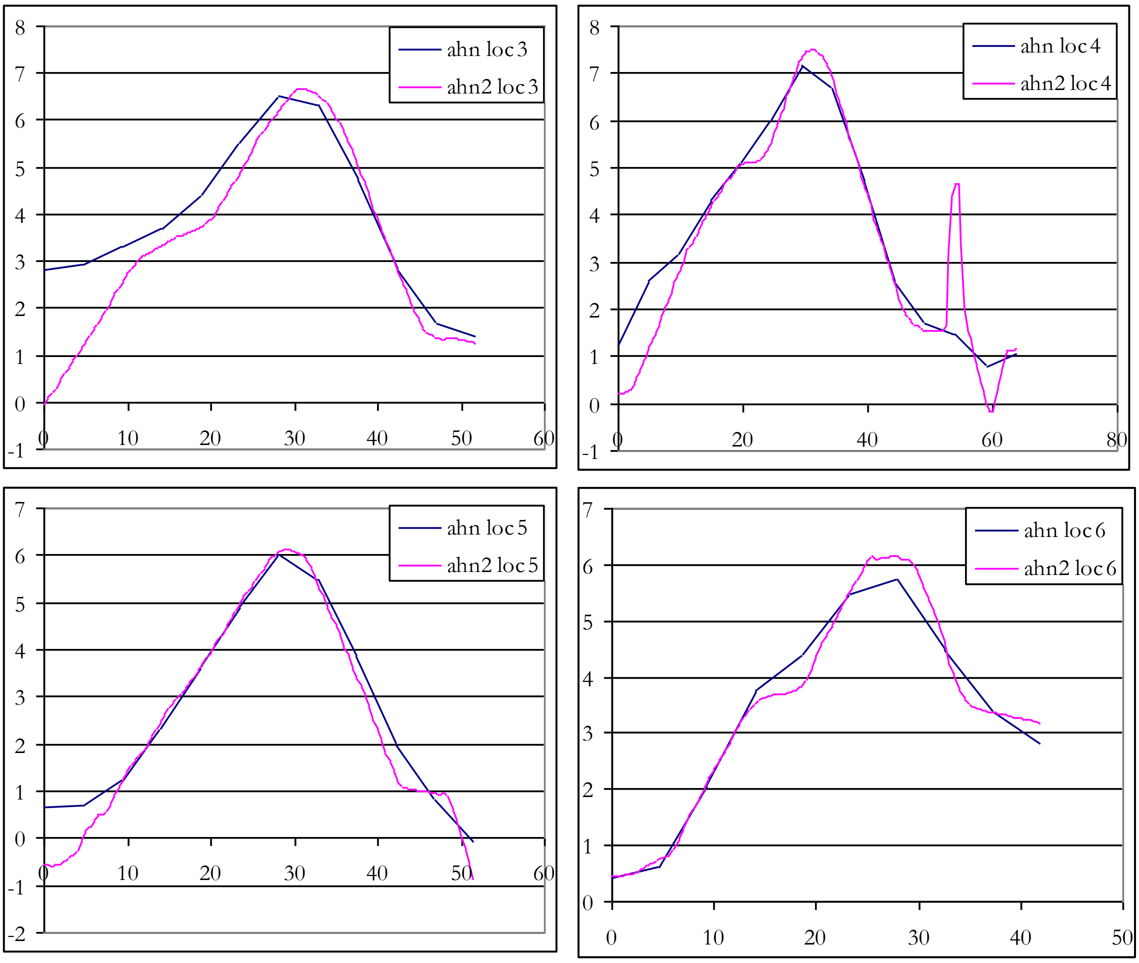
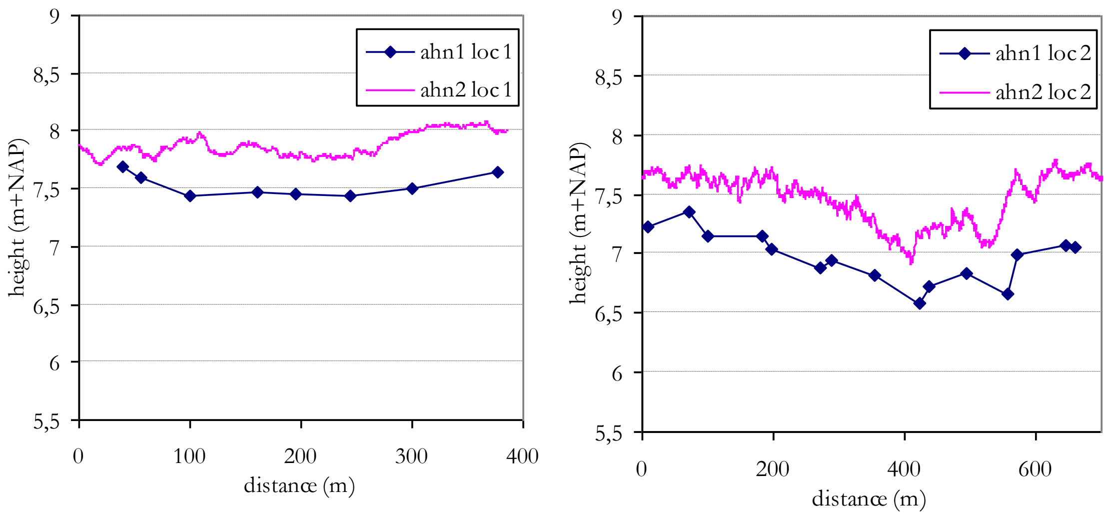


Figure 50: difference in dikeprofile for AHN1 and AHN2 data

The question rises whether AHN1 data is usable for defining the dike height. For the locations shown in Figure 49 the dike height is determined. The results are shown in Figure 51.



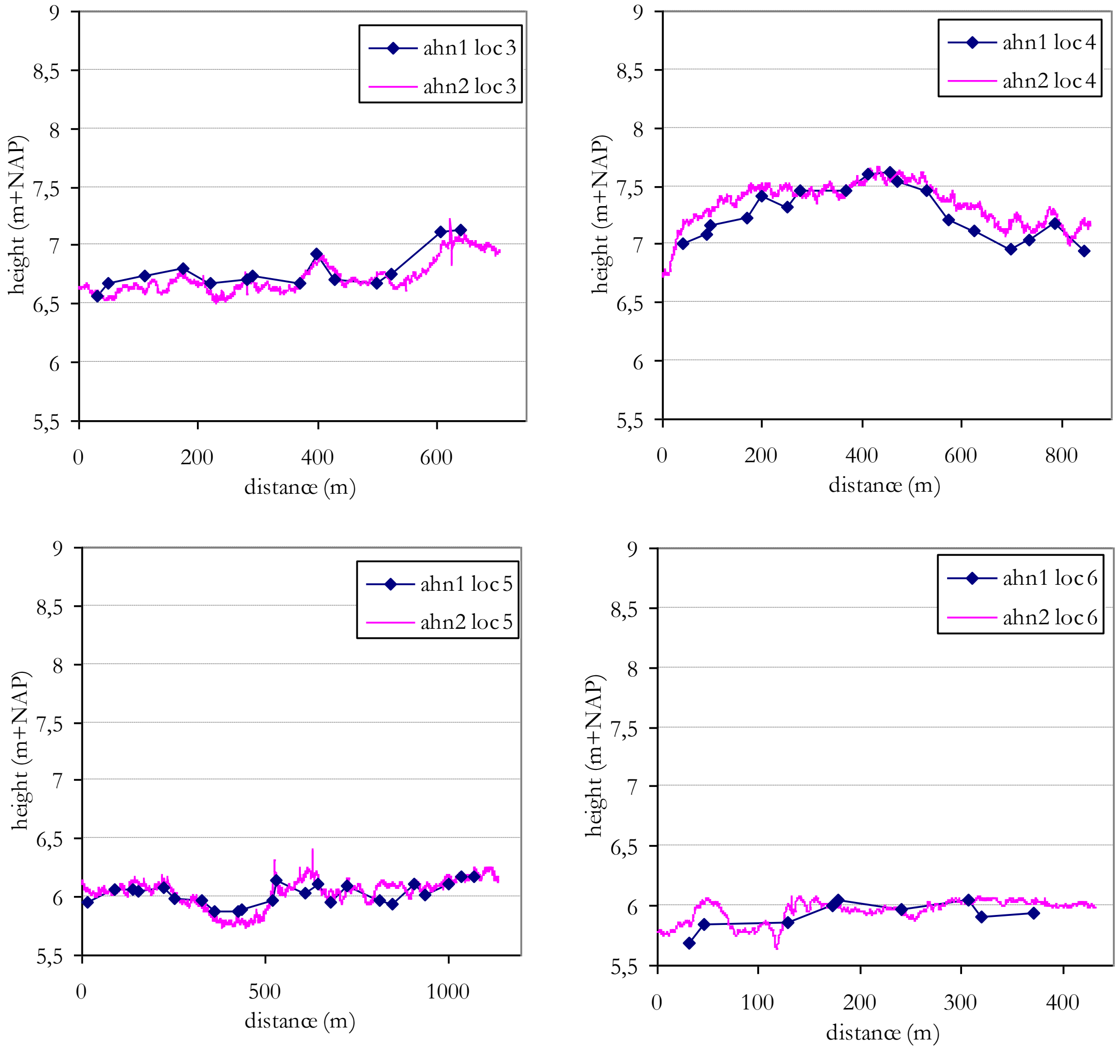


Figure 51: difference in dike height for AHN1 and AHN2 data

As can be seen the height at the locations 3-6 matches quite well. At location 1 and 2 a remarkable difference is visible. This difference is not caused by the method of defining the dike height, but is simply a difference in data. The AHN1 data is measured in the period 1996-2003 and the AHN2 data in 2009. It is very likely that the dike is heightened by Projectbureau Zeeweringen. This project focuses on the dike revetment, but at a lot of places the dike is heightened as well.

II.4. Outer slope angle

The outer slope angle cannot be measured by the AHN1 5x5m raster. For this purpose the 0,5x0,5m AHN2 raster is needed. For a couple of locations (Figure 49) the slope angle is inspected with AHN2 data (Figure 52).

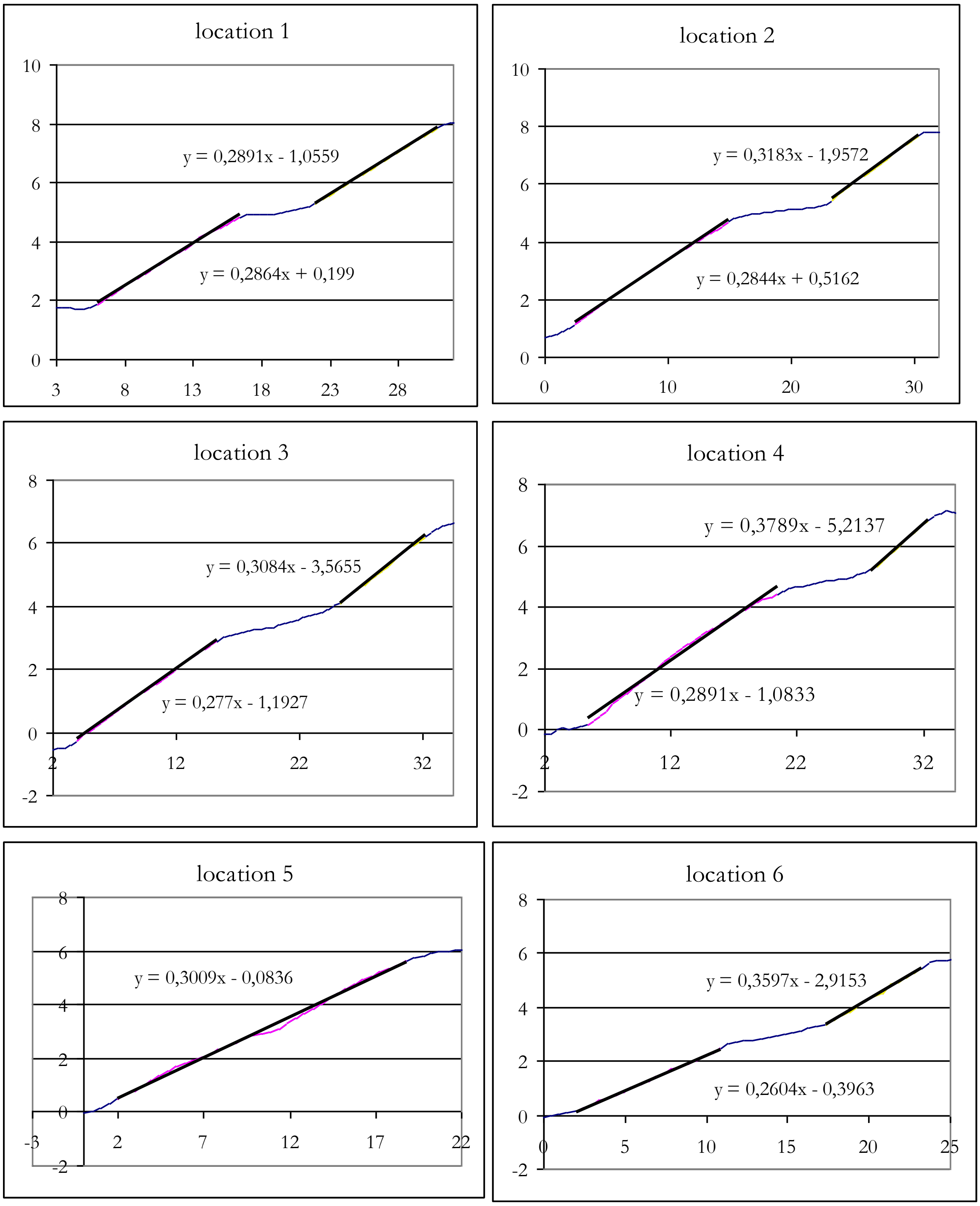


Figure 52: check for slope angle

The slope angles differ from place to place, as well as in short distance as in longer distances. There is also difference in the slope angle below the berm and above the berm. The angle lays somewhere between 1:2 and 1:4. It will cost too much time to inspect all the slope angles. In this study an average slope angle of 1:3 is used. For the bermwidth an average value of 5 meter is used. The average values for the slope angle and the bermwidth are in accordance with average values found by [RIJKSWATERSTAAT ZEELAND, 1985].

II.5. Seepage length

The seepage length is calculated based on the average crest height, average outer slope angle, average inner slope angle and the berm width. The average inner slope is determined at 1:2,5 based on [RIJKSWATERSTAAT ZEELAND, 1985]. The rest of the parameters are determined in this appendix. The seepage length is shown in Figure 53. This is in according with average seepage lengths found by FLORIS, see Figure 54.

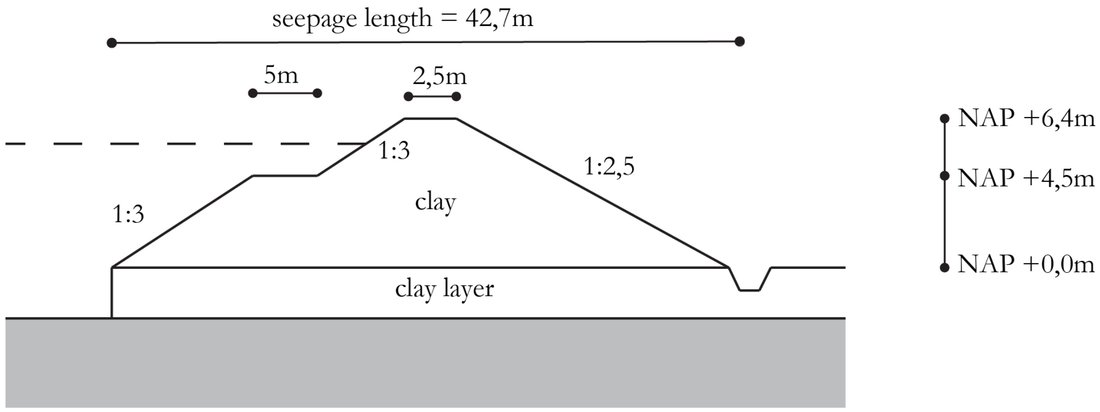


Figure 53: determining the seepage length

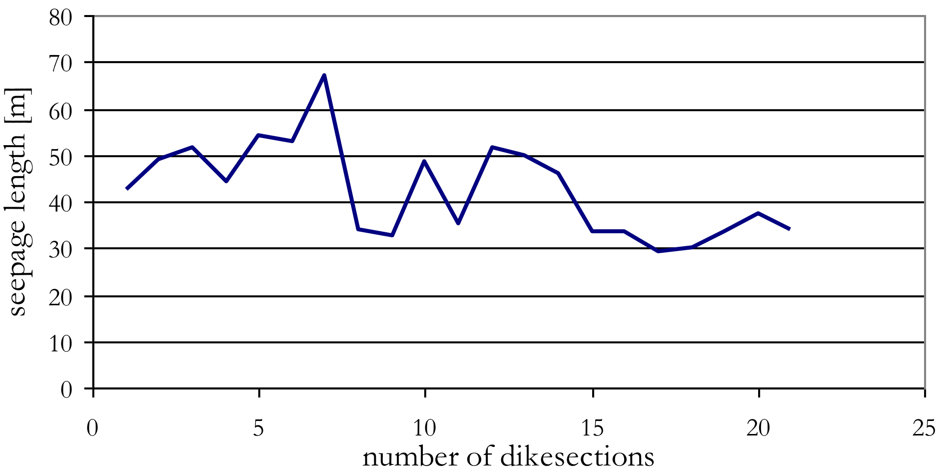


Figure 54: seepage lengths of dike-ring 26 found by FLORIS

II.6. Thickness sand layer

The thickness of the sand layer beneath the dike for dike-ring 26 calculated by FLORIS is shown in Figure 55. The thicker the sand layer is, the higher the probability of piping is. The thickness of a sand layer varies very strong, every 100m it can be different. By FLORIS for every dikesection the most unfavourable thickness is chosen. Based on the results from FLORIS a thickness of 60m for all the dike-ring around the Eastern Scheldt is chosen for this study.

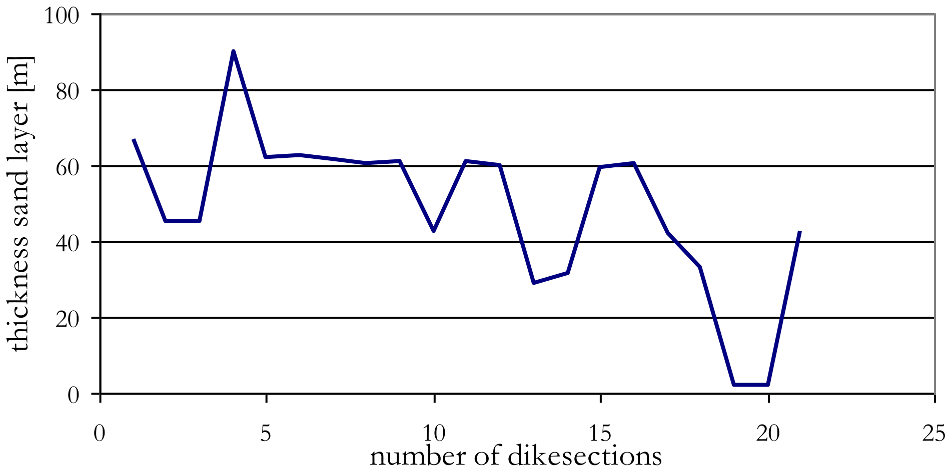


Figure 55: thickness sand layer of dike-ring 26 found by FLORIS

II.7. Permeability sand layer

The permeability of the sand layer is determined at $1{,}5x10^{-5}$ m/s. As can be seen in Figure 56 the permeability is not varying very much. The permeability used by FLORIS is based on local investigation and databases. A high permeability is unfavourable for piping.

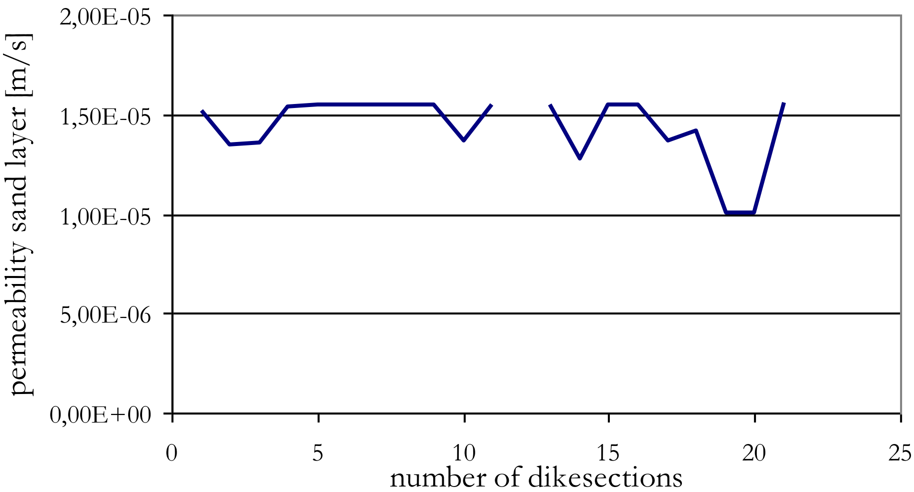


Figure 56: permeability sand layer of dike-ring 26 found by FLORIS

II.8. Particle diameter sand layer

The smaller the particle diameter of sand is, the higher the probability of piping is. Smaller particles will be moved more easily by water flow than bigger particles. In Figure 57 the particle diameter found by FLORIS for dike-ring 26 is shown. From this graph a representative particle diameter of $1{,}15x10^{-4}$ is chosen for all the dikes around the Eastern Scheldt.

Figure 57: particle diameter sand layer for dike-ring 26 found by FLORIS

II.9. Water level ditch

The water level of the ditch depends on the ground water level in the polder. Not always a ditch is present at the inner side of the dike. The level of the ditch found by FLORIS for dike-ring 26 is shown in Figure 58. When no ditch is present FLORIS has used the ground level as the level for the ditch. The higher the level of the ditch is, the higher the resistance to piping is. From this graph a representative ditch level of NAP +0,0m is chosen.

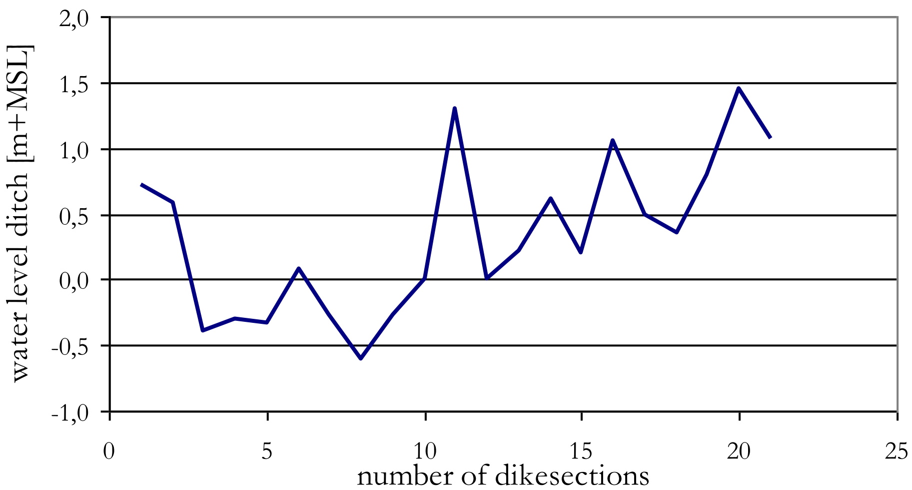


Figure 58: water level ditch for dike-ring 26 found by FLORIS

II.10. Thickness covering clay layer

A covering clay layer is positive to prevent piping. The thicker the clay layer is, the higher the resistance to piping. In Figure 59 the thickness of the clay layer for dike-ring 26 found by FLORIS is shown. From this graph a thickness of 1,0m for all the dike-rings around the Eastern Scheldt is chosen.

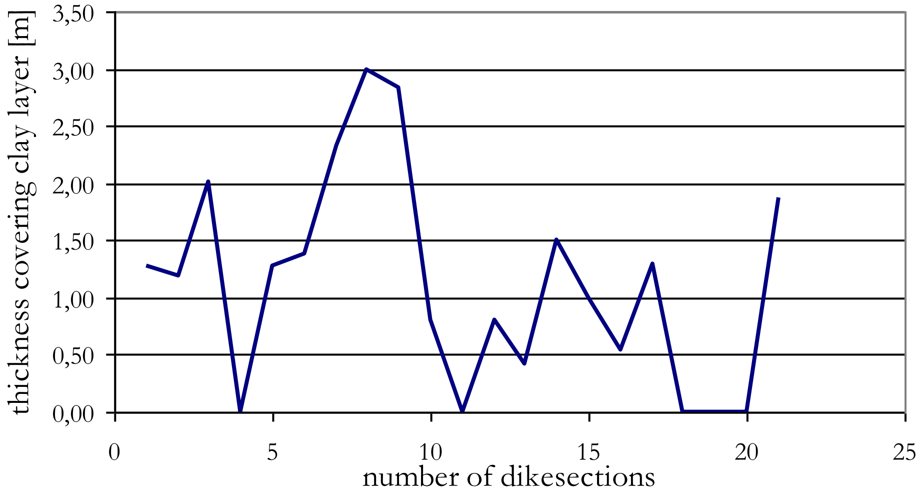


Figure 59: thickness covering clay layer for dike-ring 26 found by FLORIS

III. HYDRAULIC PARAMETERS

III.1. Wave loads

In Figure 60 and Figure 61 the test levels from HR2006 for waves around the Eastern Scheldt are plotted. From this graph a representative waveheight of 1,2m is chosen with a corresponding spectral wave period of 3,6 seconds. The water level for this representative wave is assumed to be the average test level in the Eastern Scheldt of NAP+3,5m.

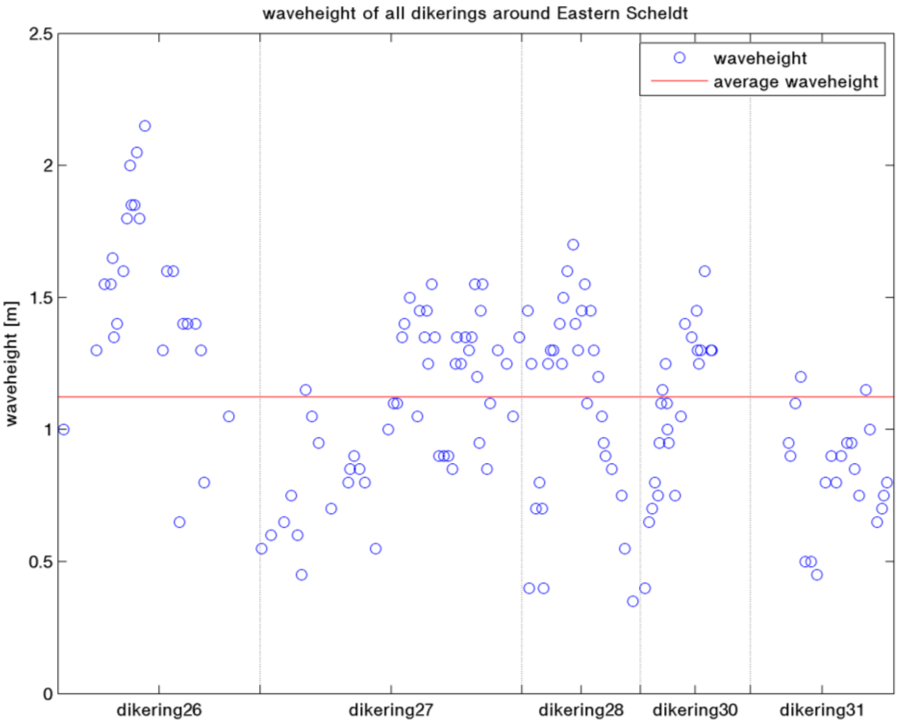


Figure 60: waveheight around Eastern Scheldt (from HR2006)

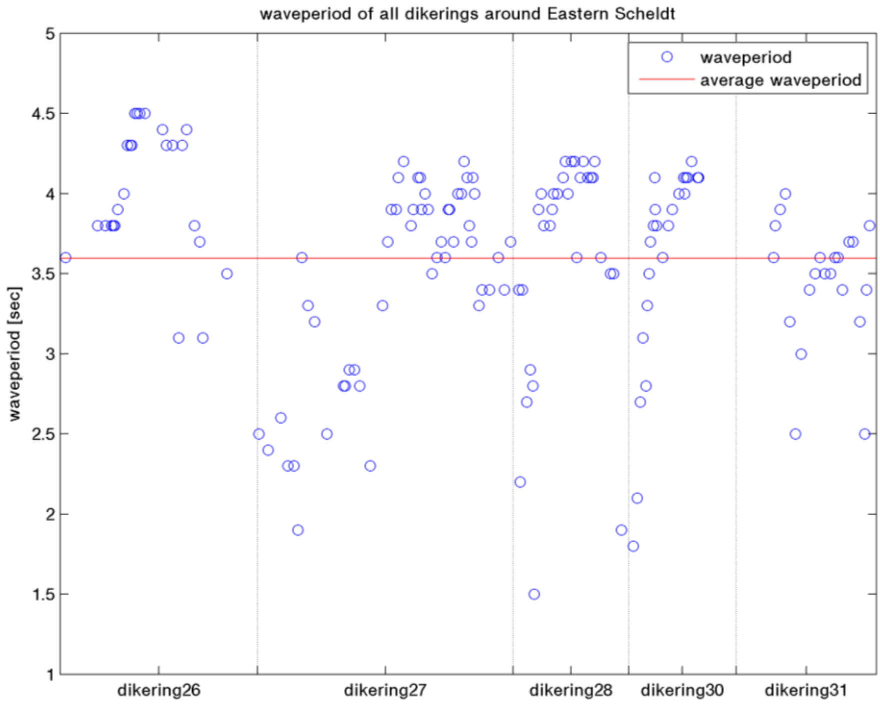


Figure 61: waveperiod around Eastern Scheldt (from HR2006)

It is too unrealistic to calculate with only one representative wave condition. The wave height and wave period varies with the water depth in the case of depth-limited waves.

To take into account the relation between the waves and the water level the waves are raised with 20% of the raise of water level. The value of 20% is based on expert opinion, it is quite a conservative value, especially for higher water levels. The corresponding wave period is defined by taking the same ratio between the representative wave height and wave period so the wave period is a factor 3,6/1,2 = 3 times the wave height. With these parameters the relation between the waterlevel and the waveheight is determined, see Figure 62.

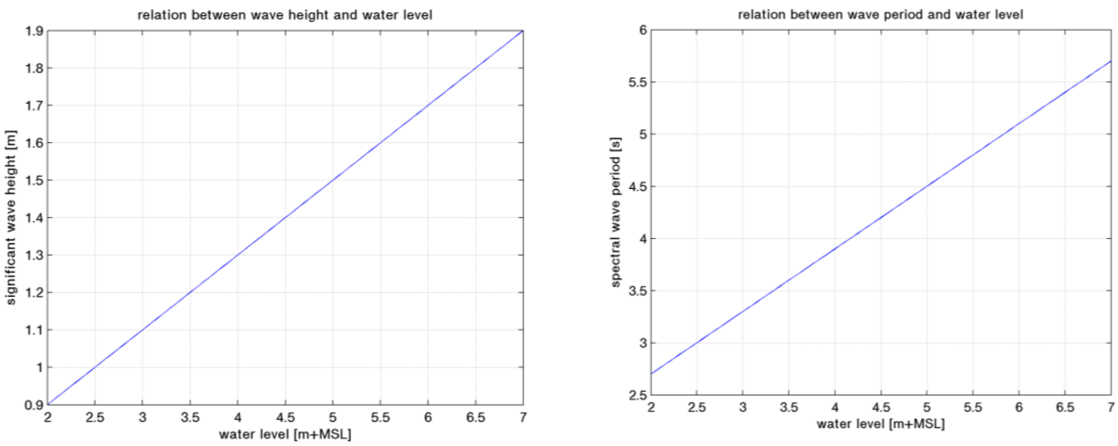


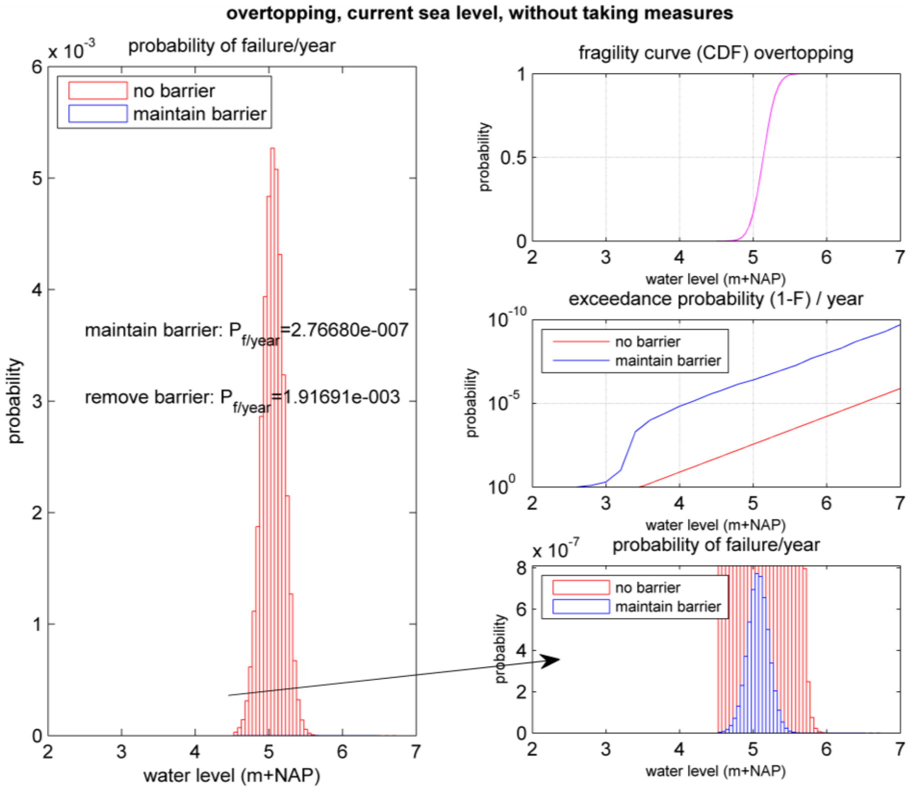
Figure 62: assumed relation between waves and water level

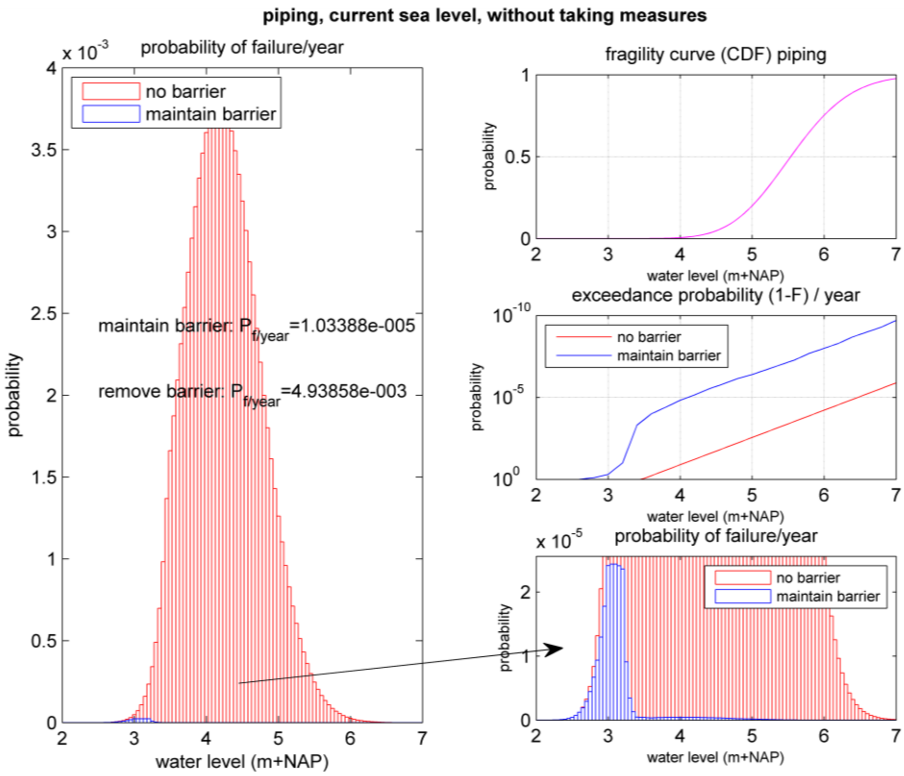
The wave directions for waves at the dike-rings around the Eastern Scheldt are shown in Figure 63. From this graph a representative wavedirection of 40 degree is taken.

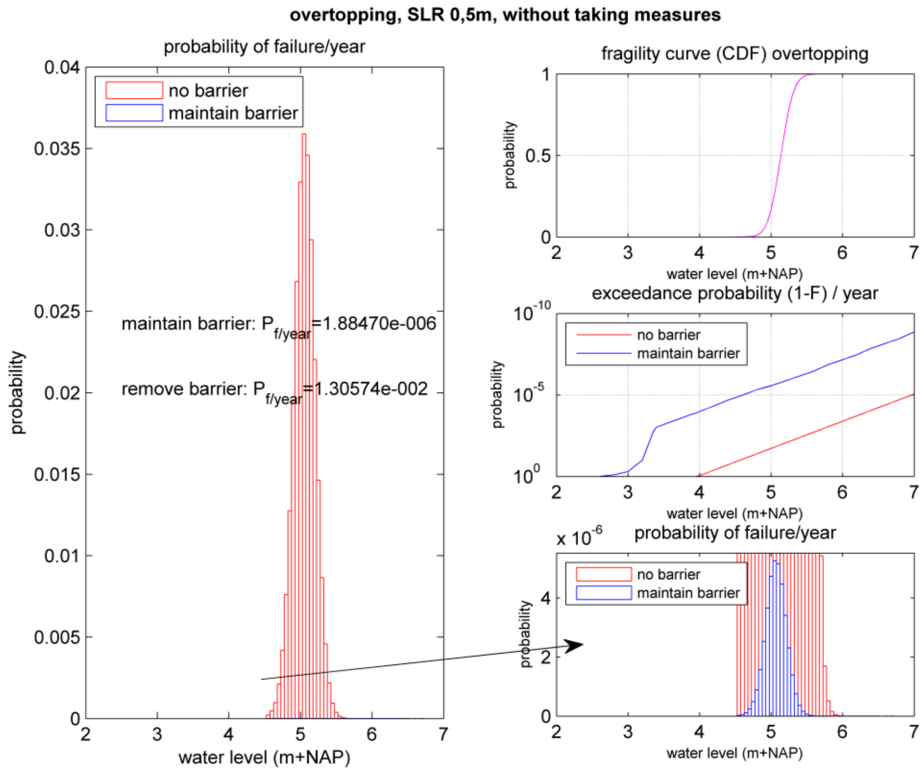
Figure 63: wave direction for dike-rings around Eastern Scheldt according to HR2006

IV. RESULTS FROM CALCULATIONS

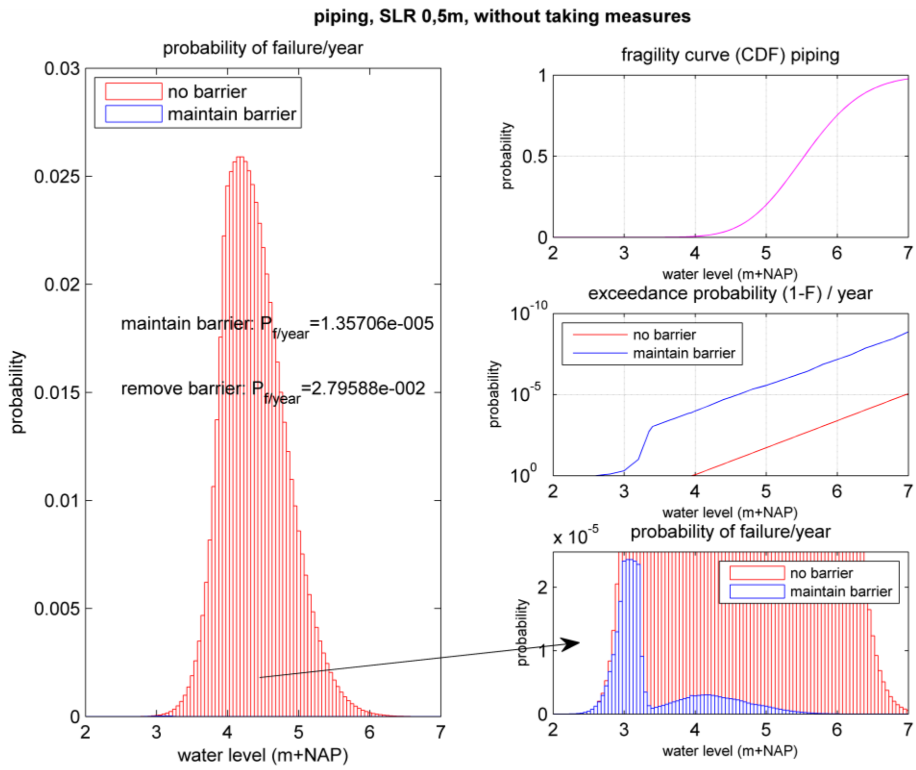
IV.1. Failure probability without taking measures



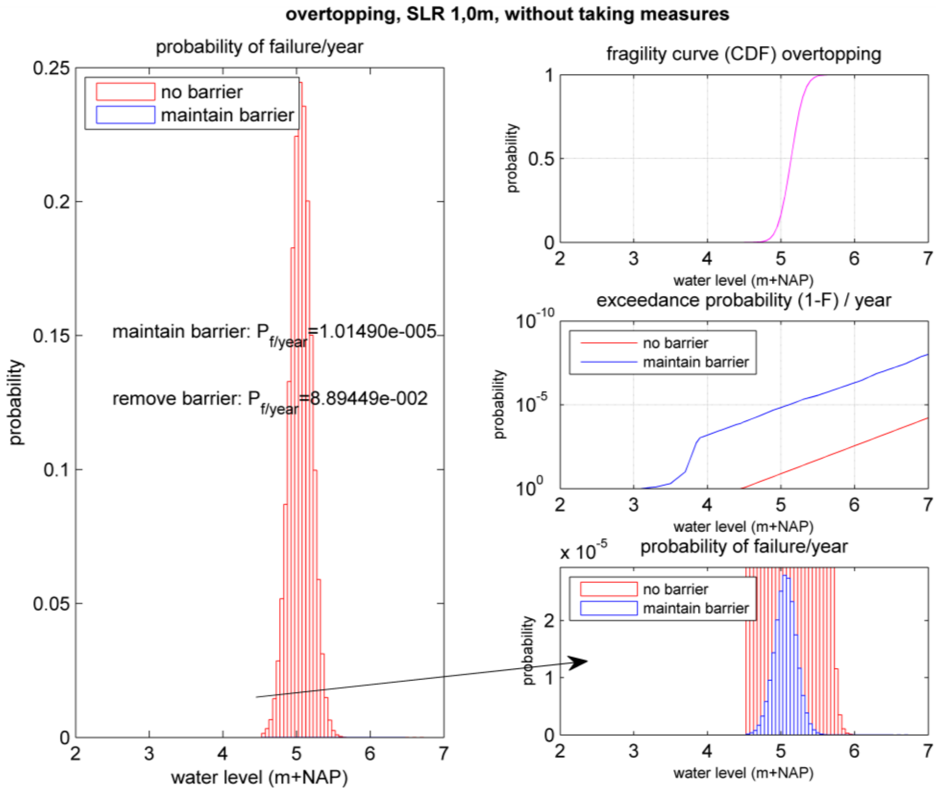


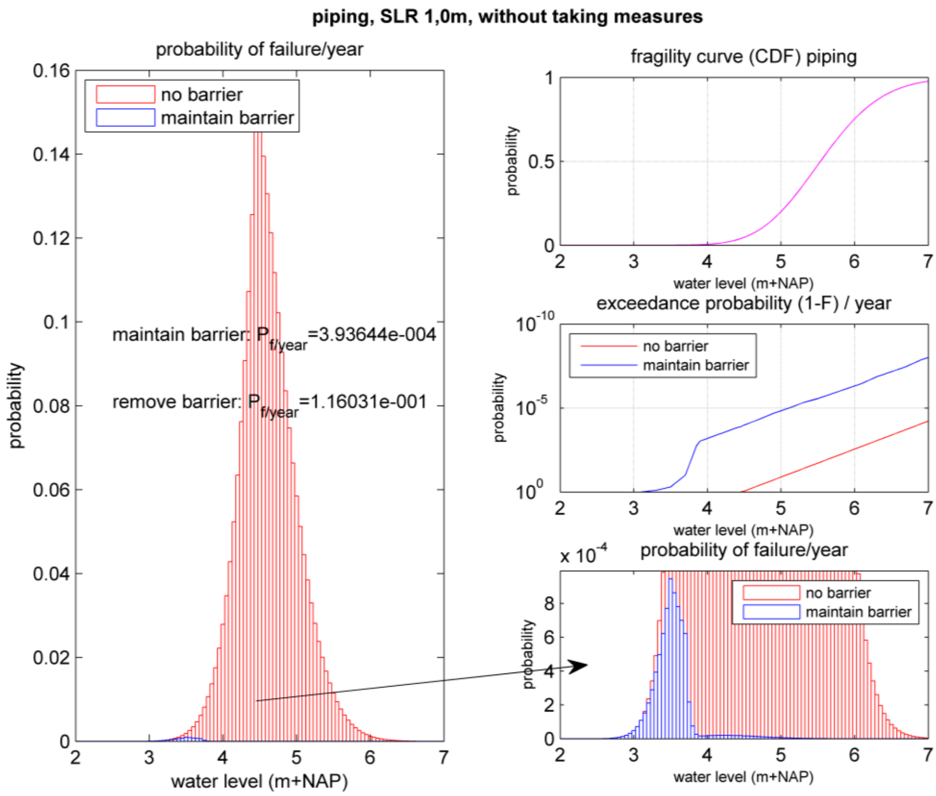


overtopping, SLR 0,5m, without taking measures
probability of failure/year
no barrier
maintain barrier
maintain barrier: $P_{f/year}$=1.88470e-006
remove barrier: $P_{f/year}$=1.30574e-002
probability
water level (m+NAP)
fragility curve (CDF) overtopping
exceedance probability (1-F) / year
x 10^{-6}

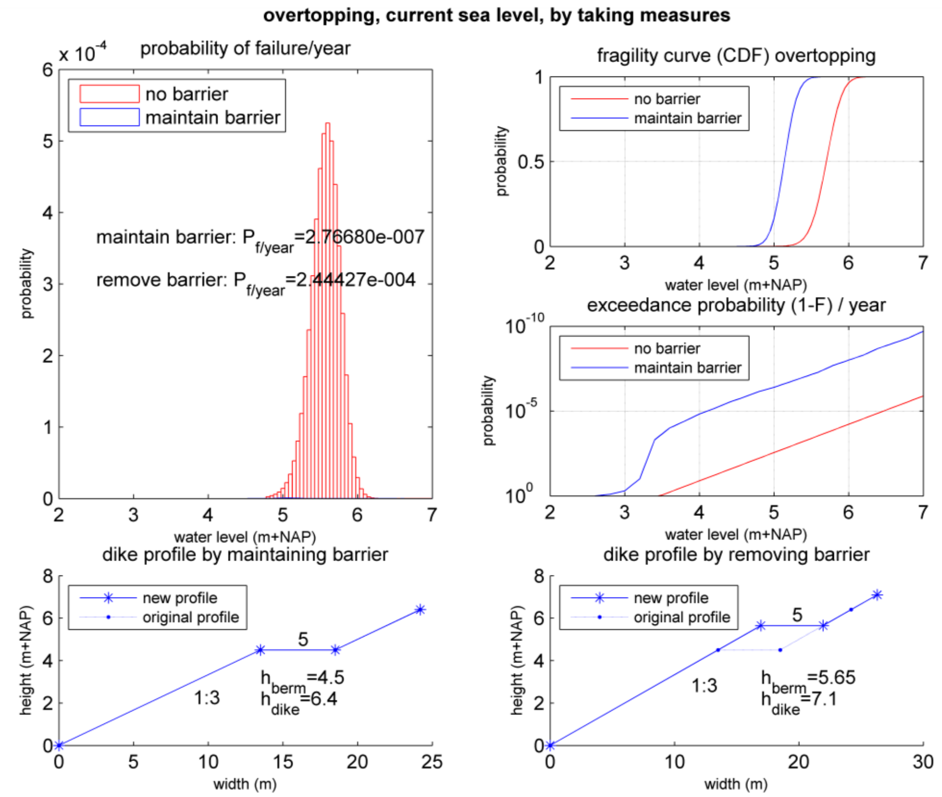


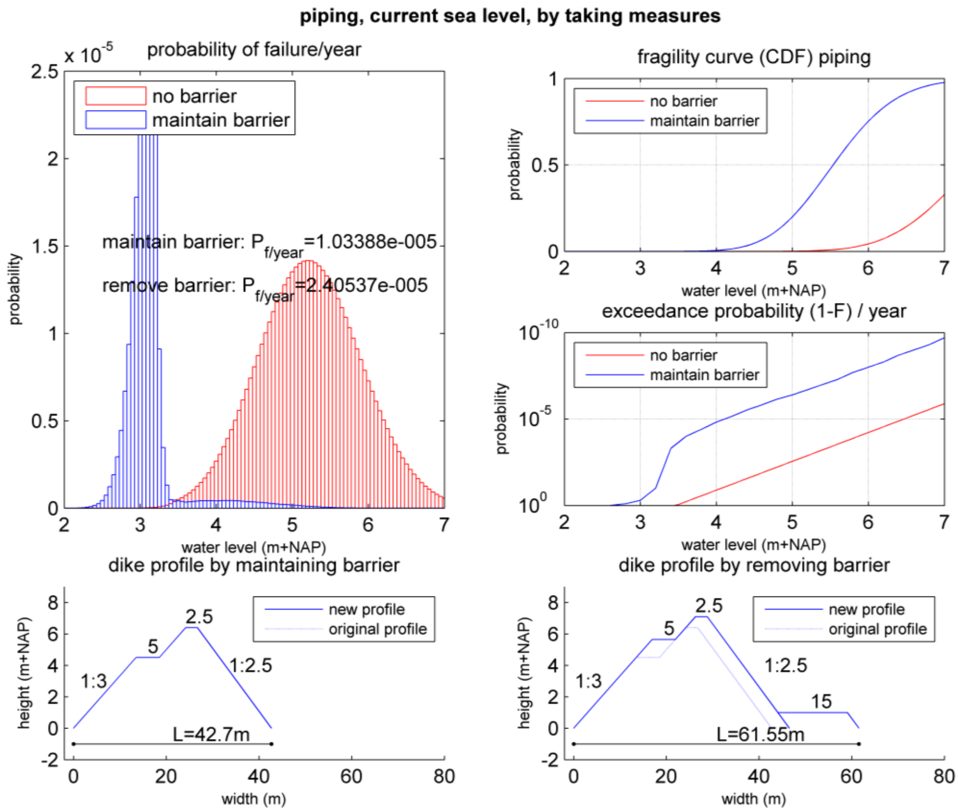
piping, SLR 0,5m, without taking measures
probability of failure/year
no barrier
maintain barrier
maintain barrier: $P_{f/year}$=1.35706e-005
remove barrier: $P_{f/year}$=2.79588e-002
probability
water level (m+NAP)
fragility curve (CDF) piping
exceedance probability (1-F) / year
x 10^{-5}

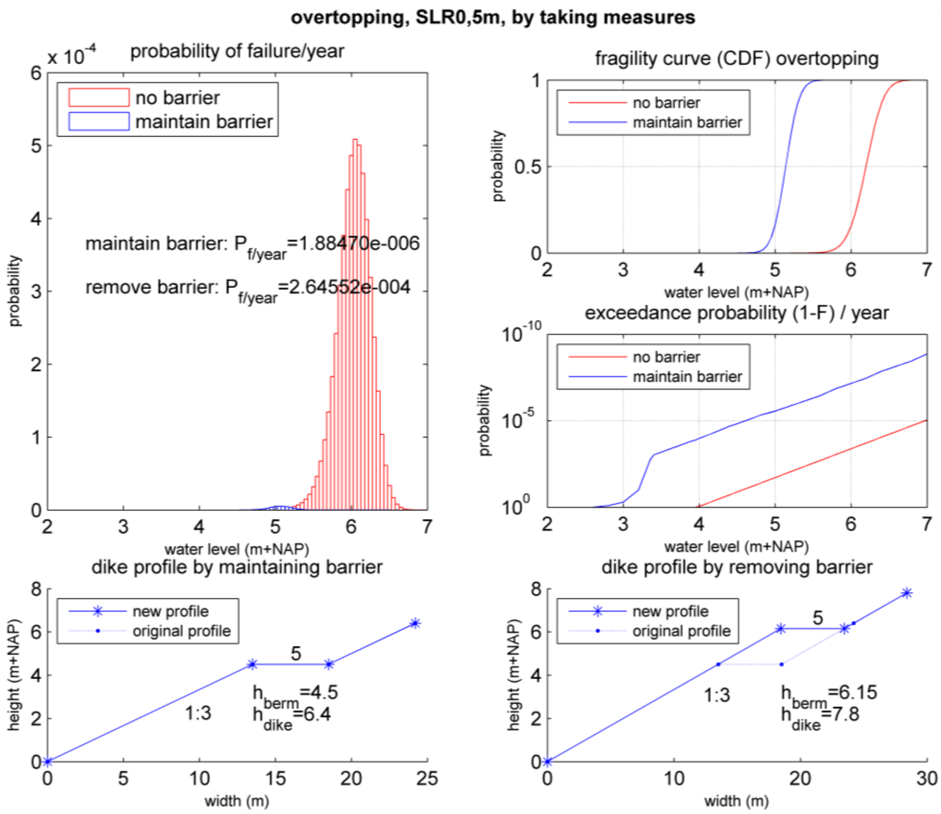

overtopping, SLR 1,0m, without taking measures
probability of failure/year
no barrier
maintain barrier
maintain barrier: $P_{f/year}$=1.01490e-005
remove barrier: $P_{f/year}$=8.89449e-002
probability
water level (m+NAP)
fragility curve (CDF) overtopping
exceedance probability (1-F) / year
x 10^{-5}

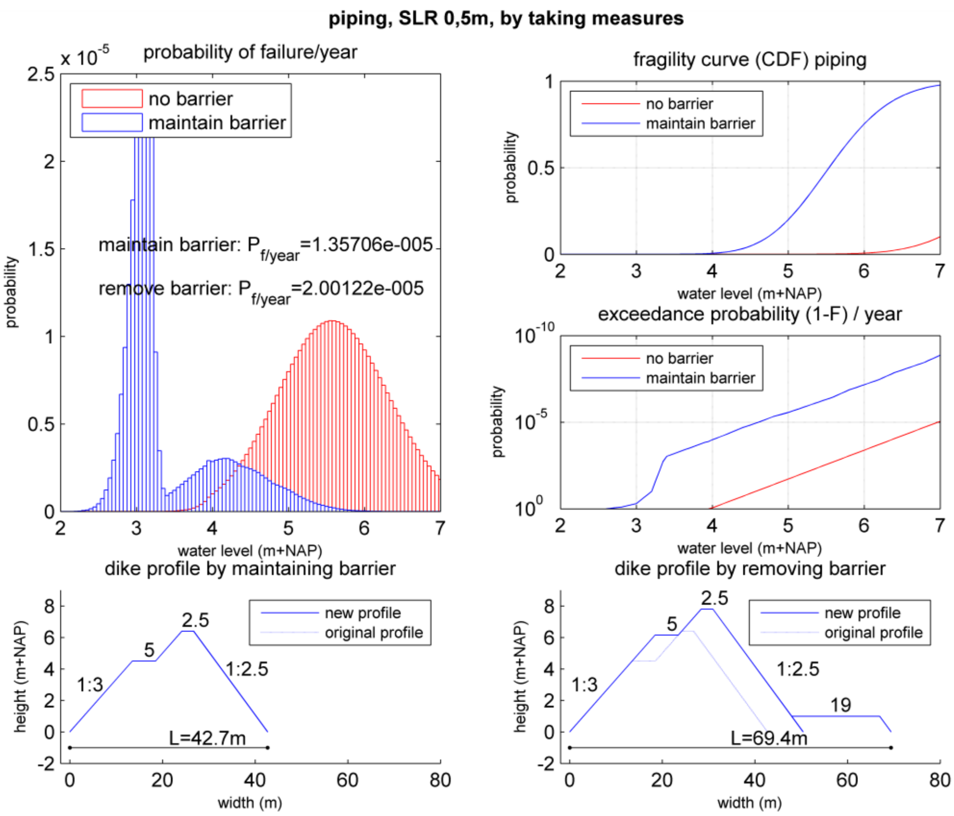

piping, SLR 1,0m, without taking measures
probability of failure/year
no barrier
maintain barrier
maintain barrier: $P_{f/year}$=3.93644e-004
remove barrier: $P_{f/year}$=1.16031e-001
probability
water level (m+NAP)
fragility curve (CDF) piping
exceedance probability (1-F) / year
x 10^{-4}

IV.2. Failure probability by satisfy to the safety standards

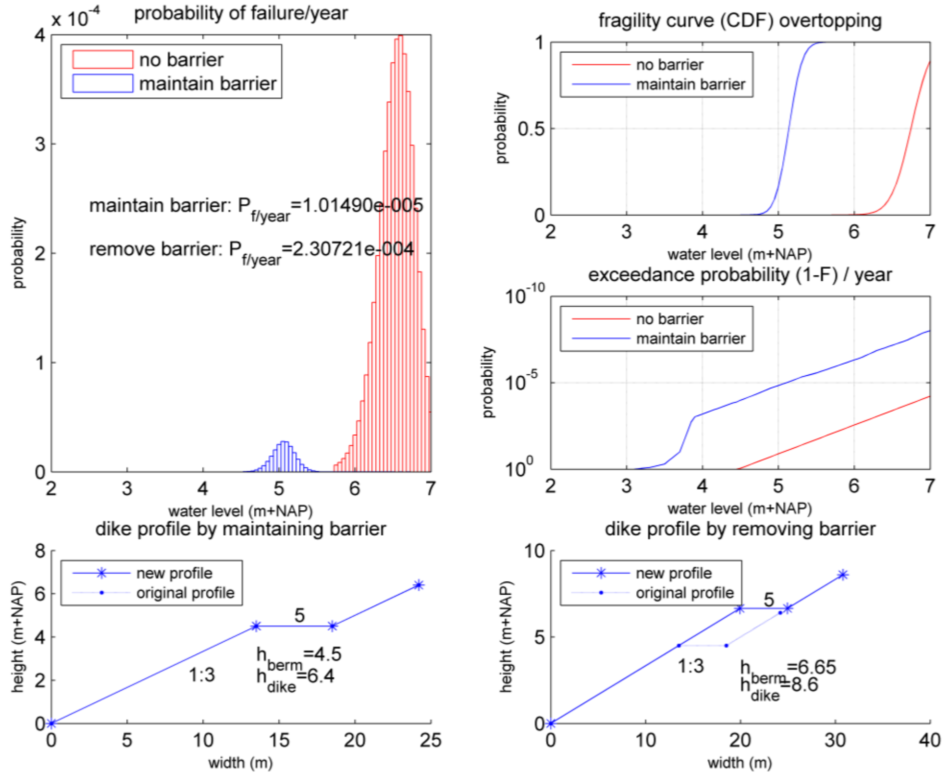





overtopping, SLR0,5m, by taking measures
probability of failure/year
x 10^-4
no barrier
maintain barrier
maintain barrier: P f/year =1.88470e-006
remove barrier: P f/year =2.64552e-004
probability
water level (m+NAP)
fragility curve (CDF) overtopping
exceedance probability (1-F) / year
dike profile by maintaining barrier
dike profile by removing barrier
new profile
original profile
h berm =4.5
h dike =6.4
h berm =6.15
h dike =7.8
1:3
5
height (m+NAP)
width (m)


piping, SLR 0,5m, by taking measures
probability of failure/year
x 10^-5
no barrier
maintain barrier
maintain barrier: P f/year =1.35706e-005
remove barrier: P f/year =2.00122e-005
probability
water level (m+NAP)
fragility curve (CDF) piping
exceedance probability (1-F) / year
dike profile by maintaining barrier
dike profile by removing barrier
new profile
original profile
1:3
5
2.5
1:2.5
19
L=42.7m
L=69.4m
height (m+NAP)
width (m)

overtopping, SLR1,0m, by taking measures



piping, SLR 1,0m, by taking measures