Master of Science Thesis

A future proof Eastern Scheldt storm surge barrier

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Master of Science Thesis

A future proof Eastern Scheldt storm surge barrier

by

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in partial fulfillment of the requirements for the degree of

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in Hydraulic Structures

Faculty of Civil Engineering and Geoscience
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Preface

This document includes the Master of Sciene Thesis (MSc. Thesis) at Delft University of Technology, faculty of Civil Engineering and Geoscience. After an orientation at different companies I have chosen to do an graduation internship at Royal HaskoningDHV at their office in Nijmegen. Royal HaskoningDHV is one of the worldwide leading engineering firms in hydraulic engineering, with a lot of expertise on many different areas of engineering. Despite of the possibilities for international MSc. Thesis subject, I have chosen a topic concerning the Eastern Scheldt storm surge barrier. One of Holland's pride in Hydraulic Engineering and well know all over the world. This graduation topic gives me the opportunity to contribute to the future of the Eastern Scheldt storm surge barrier. A storm surge barrier which is globally well known but close enough for a visit by car.

Apart from the gratitude I owe Royal HaskoningDHV I especially would like to thank Floris van der Ziel for his guidance and critical notes through the graduation process. I also want to thank the other members of the thesis committee for their important role during graduation and everyone who has, in his or her own way, contributed to the Master Thesis. Finally, I would like to thanks Willeke for her patience and confidence during graduation.

T. van der Aart
Delft, July 2015

Summary

The worst flood of the 20th century happened at the night of January 31 to February 1st, 1953. The flood was caused by a heavy storm (North-Western wind) combined with spring tide. After this disastrous flood the Dutch government came up with The Deltaworks, a large scale plan to protect the hinterland against future threats of high water. As part of the Deltaworks the Eastern Scheldt area was compartmentalized with the Eastern Scheldt storm surge barrier as imposing capstone of the compartmentalization.

The barrier has greatly (and is still) contributed to the safety of the Netherlands, but it has also had a large impact on the Eastern Scheldt area. The Eastern Scheldt area is facing future (unanticipated) challenges like the sea level rise, the sand demand of the Eastern Scheldt, the scour holes near the shores of Schouwen-Duiveland and new safety standard for flood defenses. These challenges can be influenced by the barrier. Analyses of the challenges revealed that adaptions to the storm surge barrier, which are respond to the sand demand, are the most promising to effectuate.

Sand demand

As a consequence of the compartmentalization of the Eastern Scheldt (including the construction of the Eastern Scheldt storm surge barrier), the tidal prism and the flow velocity in the tidal basin decreased with approximately 50 %. As a result, the net buildup of tidal flats stopped while the waves remained stable or even increased in height. This is shown in Figure 1. The waves crumble the edges of the tidal flats causing the tidal flats to decrease in height. On the other hand, the supply of sediment to higher tidal flats and tidal muds decreased because of the decreasing flow velocity. The tide is not 'strong' enough to transport the sediment from the main channel onto the tidal flats. This phenomenon is called the 'sand demand' of the Eastern Scheldt.

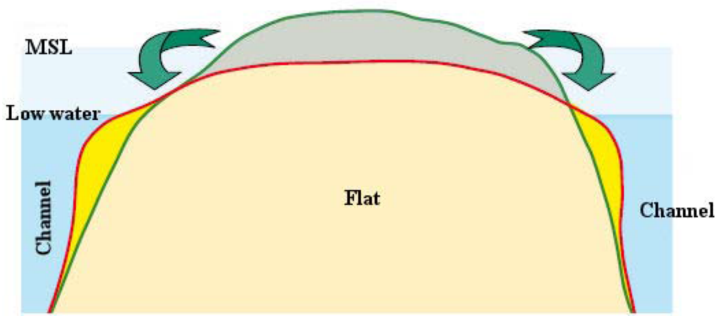

Figure 1: Definition sand demand Eastern Scheldt [van Zanten and Adriaanse, 2008])

To value the impact of the adjustments on the storm surge barrier on the sand demand, the interaction between the main channel and the tidal flat should be used. The knowlegde about the process behind the main channel - tidal flat interaction is still limited. Qualitatively is know that the flow velocity is the governing parameter for the build up of tidal flats, because an increase in flow velocity ensures more sand in suspension (the sand motor for the build up of tidal flats). The sand in suspension is transported onto the tidal flats by the vertical tide, the sediment settles and the build up begins. The built up of tidal flat depends on several variables (e.g. the flow velocity and the amount of sand in dispension). It is difficult to identify the quantitative relation between the sand in suspension and the built up of tidal flats. It is unkown whether the relation between the flow velocity and buit up of tidal flats is lineair, quadratic or if it has an asymptotic characteristic. For this Msc. thesis a lineair relation is assumed.

Lowered sill beam alternative

In the MSc. Thesis adjustments to the moveable part of the Eastern Scheldt storm surge barrier are examined. A lowered sill beam alternative is developed to influence the tide during normal tidal conditions. In this alternative the effective cross sectional area of the Eastern Scheldt storm surge barrier will increase with approximately 6,200 m^2 (from 17,900 to 24,100 m^2). The measures in this alternative include the following adjustments (See Figure 2):

1. Lowering of 50 sill beams
2. Extension of 50 gates
3. Strengthening of the sill construction

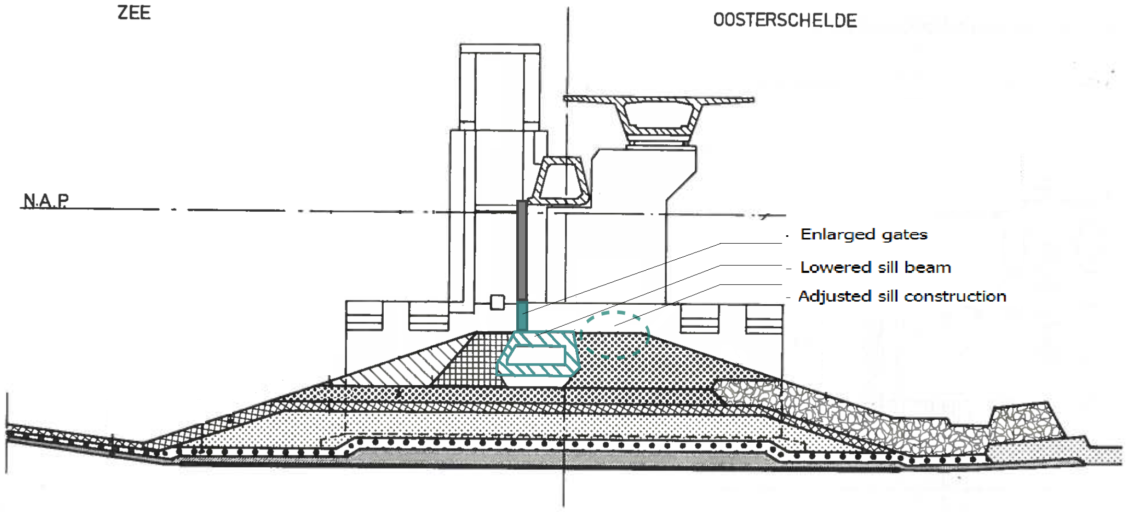

Figure 2: Cross section lowered sill beam alternative

By enlarging the cross sectional area of the barrier the tidal volume through the barrier and the tidal flow velocity in the Eastern Scheldt enlarges. The governing force in shoal build up is the tidal flow velocity, therefore the enlargement of the tidal velocity in the Eastern Scheldt should result in an improvement of the sand demand problem. Instead of removing the whole sill beam it is chosen to lower the sill beams. This to ensure a better connection between the gate and the sill and to reduce the leakage during storm. From a safety point of view it is chosen to not lower the outer sill beams to reduce the possible negative effects on the shores of Schouwen and Noord-Beveland.

Structural safety

To ensure the structural safety of the alternative, the moment and shear capacity of the halved sillbeam prestressed and the overall stability of the barrier is checked. In addition to this some remarks are placed at other necessary adjustment. This analyses showed that it is technically possible to lower the sill beams. Technical design remarks are placed at:

Stones sill construction In the alternative a part of the stones of the sill construction must be removed to create work space for construction. After the lowering of the sill beams the sill construction (consisting of place stones) is restored. Because of the lowering of the sill beam and the adjustments to the sill construction the flow pattern through the barrier changes. Calculations must show whether the stones of the top layer of the sill constructions are still stable.

Stops of the gates In the current situation, the design of the sill beam is arranged in such a way that the upper part of the beam serves as a stop for the gates. In the lowered beam alternative the stops are not present (or in limited extend). Because of this change the hydraulic load force shifts up. The overall stability of the structure is checked to see whether or not the structure is still stable. However, apart from the stability the stops might have other functions like minimizing the leakage through the gates. It has to be determined what other functions these stop have.

Enlarged gates In the lowered sill beam alternative the current gates in the Eastern Scheldt storm surge barrier will be replaced by higher gates. In the design of the alternative there is assumed that the gates will be replaced by gates with a lightweight material (e.g. fiber reinforced polymer (FRP)). In that case the weight of the gates should remain the same. If the weight of the extended gates are still larger than the weight of the existing gates, it should be investigated if the lifting capacity of the current gate is sufficient. Furthermore, in the situation with the extended gates the guidance of the gates does not cover the complete height of the gates. Dynamic flow force could exert large force on the tip of the gates. Therefore it should be examined whether this causes problems and whether the current guidance needs to be adjusted.

Execution

Execution of the alternative is completely "in the wet". In this method, the openings of the barrier also will be closed one by one. The execution method "in the wet" in general consist of:

Placing cutting frames On both sides of the sill beam a so called cutting frame will be lifted into the notches of the piers. Before the cutting can start the saw frames should lifted into their position by a floating shearleg.

Tensioning wired saw Between the two frames the wired saw should be installed. The managing of the saw installation is done from a pontoon which is positioned near the barrier. This cutting frame is the basis for the wired saw machine which is placed between the two frames.

Cutting With the help of the wire saw machine several vertical cuts will be made into the top of the sill beam. These vertical cuts are required to remove the top of the sill beam in parts and to prevent the wire saw against jamming. After applying the vertical cuts, the different top parts of the sill beam will be removed from the bottom part of the beam by a horizontal cut also made by the wired saw.

Removing After that the different part parts can be removed one by one. The removed parts will be hoisted into trucks and transported. Reuse of material will be, as much as possible, be pursued.

Conclusion

The lowered sill beam alternative must outweigh the current situation. The current situation holds the maintaining of the current barrier and planned investment in preservation of the tidal flat by sand suppletion. The estimated cost for maintaining the current barrier in its original state amount to approximately 650 million euro (within a range of +/- 20 %). The further research into the alternative has shown that the cost of adjusting the sill beam of the barrier cost more than maintaining the barrier in its current state. Because of the higher costs and the uncertainty in the actual effect of the measures, it seems more effective and safer to maintain the barrier in its current state. In this regard a critical note should be placed at designing with a lifetime of 200 years. The topic of discussion will remain whether designing with a lifetime of 200 year is reality or illusion. Large infrastructural projects have large environmental and morphological impact. In earlier stages of this project, model simulations were used to predict the impact of the measures. If during the lifetime of the structure predictions deviate from reality, there is often no room for adjustments to the structure. This also happened in the design of the Eastern Scheldt storm surge barrier. Because of the robustness of the design it is almost impossible to make, economically attractive, modifications to improve the functionality of the barrier. If designing with a lifetime of 200 year is still desirable, the structure should be designed in such a way that it is adaptive to future changes.

Samenvatting

Een combinatie van een zware Noord-Westerstorm en stormvloed veroorzaakte in de nacht van zaterdag 31januari op zondag 1 februari de ergste overstroming van de twintigste eeuw. Na deze overstroming kwam de Nederlandse regering met een grootschalig plan om dit soort overstromingen in de toekomst te voorkomen. Het grootschalige plan, ook wel de Deltawerken genoemd, omvatte onder meer het compartimenteren van de Oosterschelde. Als sluitstuk van deze compartimentering is de Oosterscheldekering gerealiseerd.

De Oosterschelde heeft mede bijgedragen (en draagt nog steeds bij) aan de veiligheid van Nederland. Toch heeft de kering grote invloed gehad op het Oosterscheldegebied. Het Oosterscheldegebied staat voor toekomstige (onvoorziene) uitdagingen, zoals de zeespiegelstijging, de zandhonger, de ontgrondingskuilen vlakbij de kust van Schouwen-Duiveland en de veranderde veiligheidsnormen voor waterkeringen. Dit zijn uitdagingen waarin de Oosterscheldekering, in meer of mindere mate, een rol kan spelen. Uit een analyse van de uitdagingen is gebleken dat de zandhonger de meest veelbelovende uitdaging is waarbij de Oosterschelde een rol kan spelen.

Zandhonger

Als gevolg van de compartimentering van Oosterschelde (inclusief de bouw van de Oosterscheldekering), is het getijdevolume en de stroomsnelheid in het Oosterschelde bekken afgenomen met ongeveer 50 %. Dit heeft geresulteerd in dat de netto opbouw van zandplaten is gestopt, terwijl de golven nog steeds sterk genoeg zijn om de randen van de zandplaten af te slaan. Door deze afname van de stroomsnelheid in combinatie met de gelijkblijvende golven nemen de zandplaten in hoogte af. Het getijde is niet meer sterk genoeg om zand vanuit de stroomgeul naar de hoog gelegen zandplaten te transporteren. De fenomeen word ook wel de 'zandhonger' van de Oosterschelde genoemd. In Figuur 3 is het fenomeen van de zandhonger weergegeven.

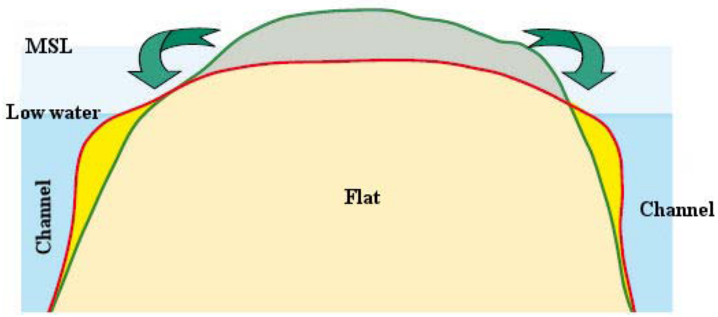

Figure 3: Definitie zandhonger Oosterschelde [van Zanten and Adriaanse, 2008])

Om de effectiviteit van aanpassingen aan de kering op de zandhonger te beoordelen zal de wisselwerking tussen de stroomgeul en de zandplaten gebruikt moeten worden. Helaas is de kennis over dit process tussen de uitwisseling van zand tussen de stroomgeul en de platen erg beperkt. Kwalitatief is bekent dat de stroomsnelheid de aandrijvende kracht voor de opbouw van zandplaten. Door een toename van de stroomsnelheid neemt de hoeveelheid zand in suspensie (de zandmotor van de plaatopbouw). Door middel van het verticale getij word het zand in suspensie getransporteerd op de platen. Daarna bezinkt het zand op de platen en de opbouw begint. Hoofdzakelijk doordat de opbouw van de zandplaten wordt bepaald door veel variabelen is het moeilijk te bepalen van de kwalitatieve relatie tussen het zand in suspensie en de opbouw van de zandplaten is. Is deze relatie lineair, kwadratisch of assymptotisch. Door deze onzekerheid is in dit afstudeerrapport een lineaire relatie tussen de opbouw van zandplaten en de stroomsnelheid aangenomen.

Verlaagde dorpelbalk alternatief

In dit afstudeerrapport is zijn aanpassingen aan de het beweegbare deel van de Oosterscheldekering onderzocht. Een 'verlaagde dorpelbalk' alternatief is ontwikkeld die tijdens normale getijdecondities het de inlaat van het getij vergroot. In het alternatief is het effectieve doorstroomoppervlakte van de kering vergroot van met ongeveer 6.200 m^2 (van 17.900 naar 24.100 m^2). De maatregelen in dit alternatief omvatten de onderstaande aanpassingen:

1. Verlagen van 50 dorpelbalken
2. Verlengen van 50 schuiven
3. Versterken van de drempelcontructie

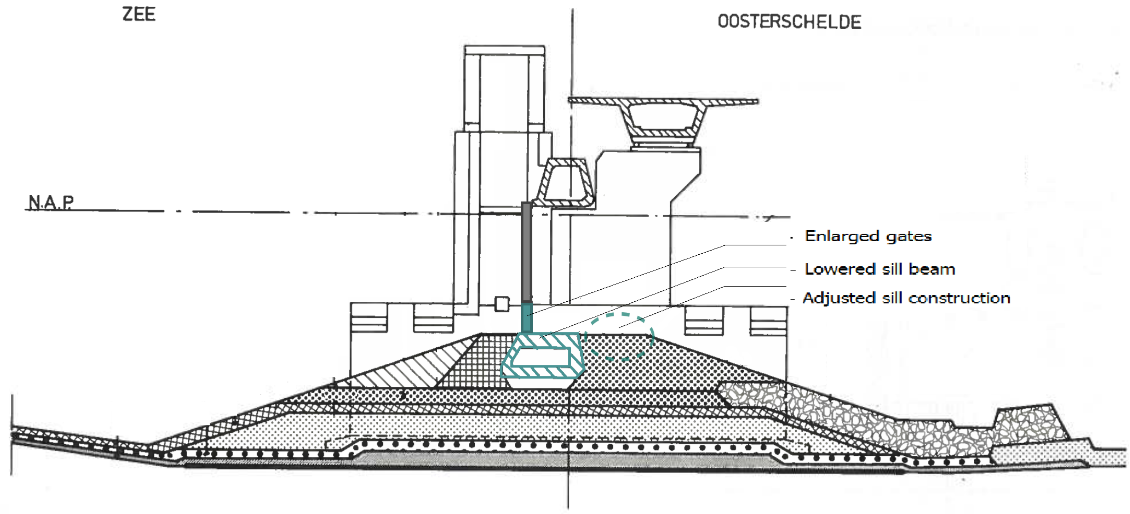

Figure 4: Doorsnede verlaagde dorpelbalk alternatief

De gedachte achter het alternatief is dat bij het vergroten van de effectieve doorstroomopening van de kering het getijdevolume door de kering en de stroomsnelheid in het Oosterschelde bekken toeneemt. Deze toename van de stroomsnelheid moet bijdragen aan een verbetering van het zandhonger probleem, omdat de stroomsnelheid de aandrijvende kracht is van de zandplaatopbouw. In plaats van het verwijderen van alle dorpelbalken is er gekozen voor het verlagen van de dorpelbalken. Hiervoor is gekozen om een betere aansluiting tussen schuif en dorpelbalk te behouden en daardoor het lekverlies door de kering te beperken. Uit veiligheidsoverwegingen is ervoor gekozen om de buitenste dorpelbalken niet te verlagen. Op deze manier moeten de negatieve effecten van groter getijdevolumen op de kusten van Schouwen en Noord-Beveland beperkt blijven.

Constructieve veiligheid

Om de constructieve veiligheid van dit alternatief te waarborgen zijn is de momenten en dwarskracht capaciteit van de verlaagde dorpelbalk gecontroleerd, de totale stabiliteit van de kering gecontroleerd en er zijn een aantal ontwerp opmerkingen over overige aanpassingen gedaan. Uit deze analyse volgt dat het alternatief technisch haalbaar is. Opmerkingen betreffende het ontwerp zijn geplaats bij:

Stenen in de drempelconstructie In het alternatief zijn een deel van de stenen verwijderd om werkruimte te creëren voor de uitvoering. Na het verlagen van de dorpelbalken zal de drempelconstructie (bestaande uit stenen) worden herstelt. Door het verlagen van de dorpelbalk en de aanpassing aan de drempelconstructie zal het stroombeeld door de kering veranderen. Nadere berekeningen zullen moeten uitwijzen of de stenen in de toplaag nog steeds stabiel zijn.

Aanslag van de schuiven In de huidige situatie is het ontwerp van de dorpelbalk zo ingericht dat deze dient als aanslag voor de schuiven. Bij hoog water aan de Noordzeezijde zal de schuif dus tegen de dorpelbalk worden gedrukt. In de nieuwe situatie ontbreekt (gedeeltelijk) deze aanslag. Door het ontbreken hiervan is het zwaartepunt van de hydraulische waterdruk naar boven verschoven. Er is daarom gecontroleerd of the totale stabiliteit van de kering nog steeds is gewaarborgd. Er is alleen nog niet bepaald of door het ontbreken van de aanslag van de schuiven er bijvoorbeeld meer lekverlies zal zijn bij hoogwater. Dit moet nader worden bekeken.

Verlengde schuiven In het verlaagde dorpelblak alternatief zullen de huidige schuiven worden vervangen door verlengde schuiven. In het ontwerp van het alternatief is er vanuit gegaan dat de schuiven worden vervangen door schuiven bestaande uit een lichter materiaal (bijvoorbeeld FRP). In het geval dat het gewicht van de schuiven toch zwaarder uitvalt dan aangenomen, moet er gekeken worden of de huidige hefcapaciteit van de schuiven voldoende is. Verder is de geleiding van de schuiven hetzelfde gebleven terwijl de schuiven langer zijn geworden. Dynamische stromingskrachten kunnen hierdoor een grotere kracht uitoefenen op de uiteinden van de schuiven. Er zal getoetst moeten worden of the huidige geleiding hier tegen bestand is. Mogelijk dienen er dus nog meer veranderingen aan de kering te worden doorgevoerd.

Uitvoering

Het verlaagde dorpelbalk alternatief zal volledig "in den natte" worden uitgoevoerd. Bij deze methode zullen de opening van kering één voor één worden aangepast. In hoofdlijnen bestaat de uitvoering uit:

Plaatsen van het zaagframe Aan beide kanten van de dorpelbalk wordt een zaagframe in de inkepingen van de pijlers geplaatst. Dit plaatsen gebeurd door middel van een drijvende bok. Deze tilt de zaagframes in positive.

Spannen draadzaag De zaagframes vormen de basis voor een draadzaagmachine. Tussen de twee zaagframes zal een draadzaag worden geïnstalleerd. Het besturen van de draadzaag gebeurd vanaf een ponton die vlakbij de kering is gepositioneerd.

zagen Met behulp van de draadzaag zullen aan de kop van de dorplebalk verschillende verticale inkepingen worden gemaakt. Deze inkepingen dienen ervoor dat de bovenkant van de dorpelbalk in delen kan worden verwijdered en dat de draadzaag niet zal vastlopen door het eigen gewicht van de dorpelbalk. Na het toepassen van verticale inkepingen zullen de verschillende delen van de dorpelbalk worden verwijderd door in horizontale richting een snede te maken.

Verwijderen dorpelbalk Hierna kunnen de verschillende, afgezaagde delen één voor één worden verwijderd. Het vrijgekomen material zal per ponton of over land worden weg gevoerd. Het vrijgekomen materiaal zal zoveel als mogelijk worden hergebruikt.

Conclusie

In dit afstudeerrapport is met de ontwikkeling van het verlaagde dorpelbalk alternatief getracht om, ten opzichte van de huidige situatie, een verbeterde functionaliteit van de kering te verkrijgen. In de huidige situatie wordt de Oosterscheldekering periodiek onderhouden en wordt er, om het probleem van de zandhonger het hoofd te bieden, zand opgespoten op de zandplaten om deze te behouden. De geschatte kosten van dit onderhouden en opspuiten van zand zijn 658 miljoen euro. Verder onderzoek naar het verlaagde dorpelbalk alternatief laat zien dat de kosten voor het alternatief meer zijn dan er in de huidige situatie wordt geïnvesteerd. Door deze hogere kosten en de onzekerheid in het effect van de dorpelbalkverlaging op het probleem van de zandhonger lijkt het effectiever en veiliger om de Oosterscheldekering in zijn huidige vorm te onderhouden. Toch moet er een kritische noot plaatsen worden bij het ontwerpen met een levensduur van 200 jaar. Onderwerp van discussie zal altijd blijven of ontwerpen met een levensduur van 200 jaar nu reëel is of niet. Bij grote infrastructurele projecten die grote ecologische en morfologische gevolgen hebben worden in een vroegtijdig stadium simulaties gedaan om de invloed te kunnen beoordelen, maar op de lange termijn blijkt het toch lastig om te voorspellen wat er daadwerkelijk gebeurd. Als tijdens de ontwerplevensduur van blijkt dat de voorspellingen afwijkingen van de werkelijkheid is er vaak geen tot weinig ruimte om het ontwerp makkelijk aan te passen. Je ziet dit nu ook bij de Oosterscheldekering: door de robuustheid van het ontwerp is het moeilijk om economisch aantrekkelijke en veilige aanpassingen aan de kering te doen. Als ontwerpen met een levensduur van 200 jaar toch gewenst is moet de constructie op zo'n manier worden ontworpen dat deze aanpasbaar is aan toekomstige veranderingen.

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Introduction

1.1. Eastern Scheldt storm surge barrier

The main topic of this Master of Science Thesis (MSc. Thesis) is the Eastern Scheldt storm surge barrier. The 9 km long barrier is part of the Dutch primary sea defense and connects the coasts of Schouwen-Duivenland and Noord-Beveland. In Figure 1.1 a bird view of the barrier is shown. The 9 km long storm surge barrier consist, amongst others, of different elements, namely: A moveable barrier, a solid barrier, a navigation lock and roads.

Figure 1.1: Top view Eastern Scheldt storm surge barrier
(*source:* `http://digitaal.zeeuwsebibliotheek.nl/beeldbank`)

Moveable barrier

With the moveable barrier the part of the Eastern Scheldt storm surge barrier is meant in which the cross sectional area, by means of moving part, can be adapted to changing water levels. This part not only includes the actual moving parts like the gates, but also the element that plays a supportive role to the moving part (e.g. piers). The moveable barrier is divided over three locations: Schaar van

Roggenplaat, the Hammen and the Roompot. The Roompot includes the largest part of the moveable barrier.

Solid barrier

The solid barrier consists out of closure dams (Roggenplaat, Neeltje Jans and Noordland) and fixed dam abutments. The dam abutments form the connection between the moveable barrier and the closure dams and the connection between the moveable barrier and the coasts of Schouwen-Duivenland and Noord-Beveland.

Navigation lock

The 'Roompot' navigation lock (with accompanying outer harbors) serves as a shipping connection between the Eastern Scheldt and the North Sea.

Roads

The roads are the collective name for the motorway N57, roads for construction traffic and several cycling and walking trails.

In Figure 1.2 the different elements of the barrier are schematized in a chart. In the course of this MSc. Thesis, the chart will be expanded.

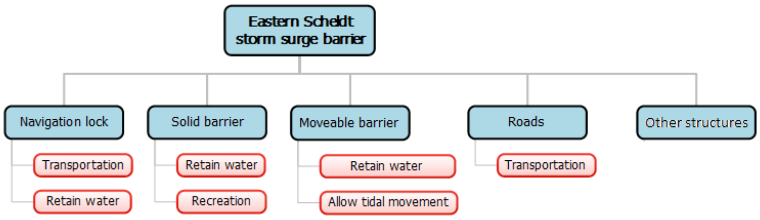


Figure 1.2: Elements Eastern Scheldt storm surge barrier

1.2. Eastern Scheldt area

The area in which the barrier is located is called the Eastern Scheldt. The Eastern Scheldt is a 350 km^2 tidal basin in the South Western part of the Netherlands (See Figure 1.3). Chapter 2 describes the Eastern Scheldt area in more detail and discusses the role of the barrier in it. The Eastern Scheldt area has or may have to deal with problems and (future) challenges that can be influenced by the Eastern Scheldt storm surge barrier. These (future) challenges are briefly described in the next paragraphs.

1.2.1. Sand demand Eastern Scheldt

Because of the compartmentalization, including the construction of the moveable barrier, the Eastern Scheldt became a tide-dominated area with low wave heights [de Bruijn, 2012]. The compartmentalization caused a decrease of the tidal prism and a decrease of the flow velocity in the estuary. Consequently the tide cannot generated enough power to transport sediment onto sandbanks and shoals in the Eastern Scheldt but is strong enough to 'eat up' the sandbanks and shoals. This phenomenon is called the 'sand demand' of the Eastern Scheldt (in Dutch: Zandhonger). In solving the 'sand demand' Rijkswaterstaat pronounced in [RWS7, 2013] their preference for a phased decision making. In phase 1 (2015-2025) the measures include the suppletion of sand on the Roggenplaat. This measure only secures the short-term goals on preserving the intertidal flats. After phase 1 the measure should be evaluated to determine the approach for phase 2 (2025 - 2060). Also future knowledge about sea level rise, further development on the erosion causes by the 'sand demand' and the developments on the population of the stelt (in Dutch: Steltloper) should be included in the decision making for phase 2. For the second phase the question arises if the Eastern Scheldt storm surge barrier could contribute in solving the sand demand.

Figure 1.3: Location Eastern Scheldt area
source: http://publicwiki.deltares.nl

1.2.2. Future sea level rise

Further developments in the sea level rise may affect the lifetime of the barrier. It furthermore accelerates the degradations of the inter tidal flats. Which effects the Eastern Scheldt area. In the last decade a few scenarios (See Chapter 3) on the sea level rise are presented in [IPCC, KNMI, Deltacommittee, 2013, 2006, 2008]. Because of the uncertainties in the different scenarios it is questionable if and how the Eastern Scheldt area (including the barrier) are affected by future sea level rise.

1.2.3. Maintenance Eastern Scheldt storm surge barrier

After commissioning the Eastern Scheldt storm surge barrier in 1986 scheduled maintenance of the barrier started. For the non-replaceable parts of the barrier a lifetime of 200 years was calculated [RWS1, 1985]. This lifetime requirement was not feasible for the gates and the operating machinery. For the part in which fatigue was governing (e.g. gates) a lifetime of at least 50 years is accounted [RWS4, 1985]. For the replaceable parts of the barrier (e.g. the hydraulic cylinders, the beam supporting operating equipment and the lifting gates) periodic maintenance is scheduled (T≈ 25-30 year).

A full description and analyses of the challenges is described in Chapter 3.

1.3. Desired situation

The barrier is designed with a technical lifetime [1] of 200 years. This means that the barrier in that time still needs to fulfill in its function although boundary conditions may change in the future. Due to progressive insights into the effect of the barrier on the Eastern Scheldt area, has been found that the effect on the environment is different than expected (See Chapter 3). The desired situation is an Eastern Scheldt storm surge barrier that functions in such a way that the safety requirements are still fulfilled although boundary conditions may have changed, the environmental requirements are better fulfilled than in the present situation and the maintenance cost are reduced.

[1] The technical lifetime of a construction refers to the period in which the construction , or parts of the construction, can be used for the intended objective. This includes the scheduled maintenance but without radical restoration is needed (source: [NEN-EN1990, 2011].

1.4. Previous research

Several studies into the Eastern Scheldt area and the impact of the Eastern Scheldt storm surge barrier on the Eastern Scheldt area has been carried out. In this paragraph the subjects of the - for this MSc. Thesis - relevant research and its conclusion are summarized. The results of the researches serve as input for this MSc. Thesis.

The future of the Eastern Scheldt with a new inlet channel.

In this research it is investigated if a new inlet channel at Neeltje Jans could form a structural solution for the 'sand demand'. The results of the study show that the large-scale effects of the Eastern Scheldt, like the ebb dominance and 'sand demand' cannot be structurally changed with a new inlet channel. The 'sand demand' and degradation of the shoals can probably be slowed down by a new inlet channel, but this introduces a new dike safety issue and high costs. The tidal prism in the Oosterschelde will probably never be completely restored to the old situation if the compartmentalization dams and storm surge barrier are not totally removed. Shoal erosion will continue, which means that suppletions will still be necessary in order to maintain the shoals [de Bruijn, 2012].

Effect of removal of the Eastern Scheldt storm surge barrier.

In [de Pater, 2012] the question is treated if removal of the Eastern Scheldt storm surge barrier will stop shoal erosion. Flow velocities at 10 observation points on the Galgeplaat are evaluated to check if shoal build up will occur when the barrier is removed. de Pater concluded that removal of the barrier causes an increase of the flow velocities of 30 to 40 %. In the MSc Thesis is concluded that based on those values shoal build up will start again when the barrier is removed. de Pater thinks there is a possibility that shoal build up will start at even smaller velocity increase but the theoretical background is missing.

Feasibility study on fibre reinforced polymer slides in the Eastern Scheldt storm surge barrier.

In [van Straten, 2013] it is investigated if it is feasible to replace the current steel gates in the Eastern Scheldt barrier by gates made of Fiber Reinforce Polymer (FRP). van Straten concludes that it is technically feasible to make the largest gates of the 62 gates of the Eastern Scheldt barrier in FRP. The most promising design is a FRP gate which is much higher than the current steel gate and which requires some adjustments on the current barrier. According to [van Straten, 2013] these FRP gates can cope with the sea level rise and can help to solve the problem with the inter tidal flats in the Eastern Scheldt on the long term. Next to that this research shows that a FRP gate is a financially attractive alternative for the current steel gates. Thereby it is feasible to replace the current steel gates in the Eastern Scheldt barrier by gates made of FRP.

1.5. Objective MSc. Thesis

In this MSc. Thesis the focus will be on how the Eastern Scheldt storm surge barrier can contribute in solving (future) challenges in the Eastern Scheldt area. The objective is to investigate which adjustments to the moveable part of the Eastern Scheldt storm surge barrier can lead to an improved functionality of the barrier and have a positive effect on the Eastern Scheldt area.

1.5.1. Main research question

The main question of the MSc. Thesis is:
What are feasible, cost-effective alternatives for an improved functionality of the Eastern Scheldt storm surge barrier?

Subresearch questions

1. *What are the main functions of the barrier?* (Chapter 4)
2. *What is the relation between these functions and the effect on the Eastern Scheldt area taken into account future challenges like e.g. sea level rise?* (Chapter 4)
3. *Which function(s) can significantly be improved by adjustments to the barrier?* (Chapter 5)

4. *With respect to those adjustments, what needs to been done technically to guarantee the structural safety of the barrier?* (Chapter 6)
5. *How can these adjustment been executed?* (Chapter 7)
6. *Are these adjustments to the current Eastern Scheldt storm surge barrier effective with respect to safety?* (Chapter 7)

1.6. Scope

The subject of this MSc. Thesis is to investigate feasible alternatives for an improved functionality of the Eastern Scheldt storm surge barrier. In this MSc. Thesis is chosen to investigate the moveable part of the Eastern Scheldt storm surge barrier. This decision was made on the basis of previous research, the students field of interest and the students Structural Engineering background.

The morphological effects on the Eastern Scheldt area as a result of intervention will be validated by assessing literature and interviewing experts. Chosen is not to do a model validation because of the magnitude of this assignment and the students field of expertise.

The option for a feasible alternative for an improved functionality will be done based on e.g. a Life Cycle Cost Analysis (LCCA). In this LCCA the construction costs of each alternative will be estimated. These cost estimates will be validated for the chosen alternative at the end of the MSc. Thesis. Because at that time there is better insight into the technical - and executive stage of the alternative. The technical feasibility will be checked by assessing the main components on strength and overall stability.

A design that can be executed will be carried out to approve the executive feasibility of the chosen alternative. In this executional design the operation sequence, the equipment and execution costs will be treated.

1.7. Reader's guide

The MSc. Thesis is structured in a way that, after the introduction in this chapter, first the Eastern Scheldt area is described in *Chapter 2*. Following by a discussion in *Chapter 3* about the challenges which the Eastern Scheldt area has to face, now and in the future.

In *Chapter 4* the functions of the barrier are described including the relation with the Eastern Scheldt area. This chapter threats the first two subresearch questions.

The question which function(s) can significantly be improvement by adjustments to the barrier is the subject of discussion in *Chapter 5*. Furthermore some alternatives for an improved Eastern Scheldt storm surge barrier are proposed. The chapter will conclude with a consideration between the proposed alternatives. Finally an alternative is chosen for further elaboration.

For the elaboration of the chosen alternative first the technical feasibility is checked in Chapter 6.

After checking the technical feasibility of the alternative, the executive feasibility will be checked in Chapter 7. The chapter concludes with a cost analysis of the chosen alternative.

The report ends with a conclusion, discussion and recommendation in Chapter 8.

2

Area description

The Eastern Scheldt storm surge barrier seperates the Eastern scheldt from the North Sea. The Eastern Scheldt is a 6 km wide and approximately 50 km long tidal basin (See Paragraph 2.2.1). The basin area is 350 km^2 [1]. The tidal basin is located in the South Western part of the Netherlands, in the province of Zeeland. The Eastern Scheldt is surrounded by the islands Schouwen-Duiveland, St.-Philipsland, Tholen, Noord-Beveland and Zuid-Beveland (See Figure 2.1).

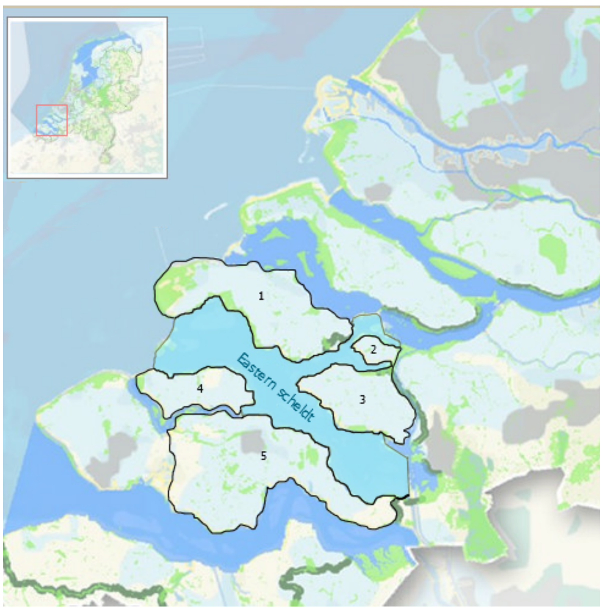


Figure 2.1: Eastern Scheldt area
1. Schouwen-Duiveland, 2. St.-Philipsland, 3. Tholen, 4. Noord-Beveland, 5. Zuid-Beveland

2.1. Historic development Eastern Scheldt area

The development of the Eastern Scheldt tidal basin as known today goes back to the late Middle Ages. As a consequence of floods and dike breaches (storms like the 'Felixvloed' in 1530) large areas around the Eastern Scheldt where flooded. This caused an increase in the tidal prism. The then present inlet of the Eastern Scheldt, the 'Hammen', was too small to process this large increase. A second inlet (the Roompot) was formed. In the sixteenth century the Eastern Scheldt has become a wide estuary with tidal channels, tidal flats and shoals. The increase in tidal prism continued, trenches where deepened and at the beginning of the twentieth century human interventions like dredging and canalization let to

[1] http://www.zeeland.nl/kust_water/zeeuwse_wateren/oosterschelde/?tid=10509

a third inlet at the mouth of the Eastern Scheldt, the 'Schaar van Roggenplaat' [Kohsiek *et al.*, 1987]. In Figure 2.2 & 2.3 the development of the Eastern Scheldt area is displayed. The figure in the right indicates the inlets.

Figure 2.2: Historic development (1550) [Kohsiek *et al.*, 1987]

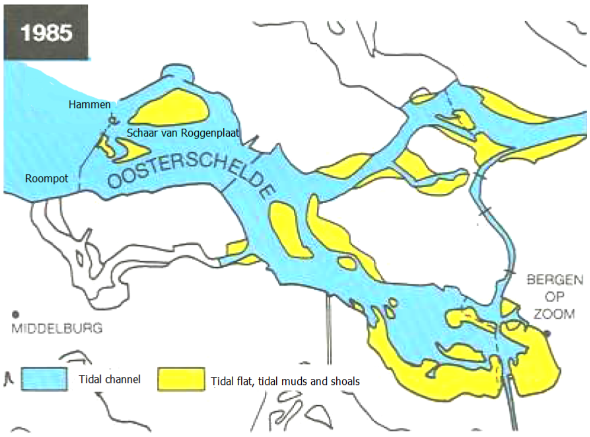


Figure 2.3: Historic development (1985) [Kohsiek *et al.*, 1987]

2.1.1. Deltaworks

The worst flood of the 20th century happened at the night of January 31 to February 1st, 1953. The flood was caused by a heavy storm (North-Western wind) combined with spring tide. After this disastrous flood the dutch government came up with a large scale plan to protect the hinterlands against future floodings. These plans are known as the Deltaworks (in Dutch: Deltawerken). 2.4 indicates the measures of the plan (red lines).

Figure 2.4: Deltaworks - Eastern Scheldt area

In the Eastern Scheldt area the Deltaworks include the construction of the Zandkreekdam (1960) and the Grevelingendam (1964). In 1967 the preparations started for the clusorue of the Easter Scheldt by a 9 km long dam. The construction was forced to stop due to negative public opinion (especially opposition from fisherman and environmental groups) and later on political pressure. In 1974 an independent Committee Oosterscheldekering (Committee Klaasesz) advised the government to construct an alternative in which the tide in the Eastern Scheldt could, more or less, be maintained. This resulted in two alternatives, a permeable dam and a closeable storm surge barrier. In 1977 the first plans for the design of the barrier where drawn. Further development of these plans let to the Eastern Scheldt storm surge barrier as known today. After the construction of the Eastern Scheldt storm surge barrier (1986), the Oesterdam (1986) and the Philipsdam (1987) the compartmentalization of the Eastern Scheldt estuary was completed.

2.2. Relation Eastern Scheldt area - Storm surge barrier

As already mentioned in Paragraph 1.2 the Eastern Scheldt storm surge barrier is part of the Eastern Scheldt area. To show its position in the area, the chart in Chapter 1.1 is extended with an additional level. Figure 2.5 indicates this in blue.

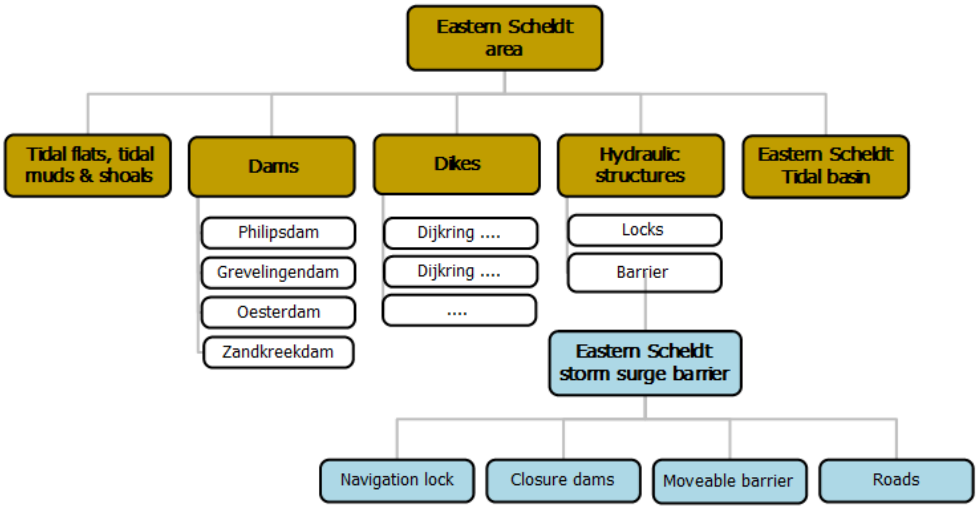


Figure 2.5: Relation Eastern Scheldt area to the storm surge barrier

The Eastern Scheldt storm surge barrier is part of the hydraulic structures group. This group, together with the tidal flats, tidal muds and shoals, the dams, the dikes and the Eastern Scheldt tidal basin, forms the Eastern Scheldt area. The next paragraphs discuss the different parts of the Eastern Scheldt.

2.2.1. Eastern Scheldt tidal basin

The Eastern Scheldt a tidal basin. A tidal basin is a body of water, almost entirely surround by land, connected to a river or sea by a (tidal) inlet and subject to tidal action. Tidal basins and their inlets are well known characteristics of lowland coasts.

The characteristic morphology of tidal basins is a meandering, braided or branched channel system, tidal flats, tidal muds and shoals (See Paragraph 2.2.2). Rivers can discharge in tidal basins. If the rivers discharge is large the tidal basin is ofter funnel shaped and the channel structure is more braided than branched (Western Scheldt). If the river discharge is large, there exists a transitional region in the basin between salt and fresh water. These basins are called estuaries. The Eastern Scheldt is an example of a tidal basin without river influence (no fresh water influence).

Also inside the tidal basin there exists a morphologic activity, primarily driven by the interaction between bottom morphology and tidal motion. This interaction is the cause of a complex three-dimensional structure of residual circulations, which are both cause and result of the morphologic structures of the basins. In a meandering channel residual circulations are the cause of a spiraling flow structure. Sedimentation, erosion and inter tidal flats are connected with these flow-structures. (source)

2.2.2. Tidal flats, tidal muds and salt marshes
A tidal mud is a outer dike, barren area. The tidal mud is flooded every high tide. When a tidal mud is surrounded by water, it is called a tidal flat. Many worms and shellfish life in the soil of the tidal flats and tidal muds. These worms and shellfish serve as food for birds. The tidal flats are also an important resting area for seals. The seals also suckle their young. When a mud is high enough silted plant growth starts. This process slowly develops until salt marshes arises. Salt marshes are [buitendijkse]areas overgrown with plants and crossed by small creeks. During flood this creeks run full with water. At low tide they are dry. Only during extreme high tide the salt marshes are flooded completely.

2.2.3. Dams
As already mentioned in paragraph 2.1.1, the Eastern Scheldt area is compartmentalized by the Philipsdam, the Grevelingendam, the Oesterdam and the Zandkreekdam.

2.2.4. Dikes
Around the Eastern Scheldt tidal basin several dikes are situated. This dikes are part of dike rings. In Figure 2.6 the dikes around the Eastern Scheld are highlighted.

2.2.5. Hydraulic structures
Besides dikes and dams, the Eastern Scheldt area holds several locks and other hydraulic structures (of which the navigation lock in the Eastern Scheldt storm surge barrier is one of them).

2.3. Functions Eastern Scheldt area
The different parts of the Eastern Scheldt area (See Paragraph 2.2) all have their role in the functioning of the whole area. The different functions of the Eastern Scheldt area are listed below.

- Retain water
- Allow tidal movement
- Provide transportation
- Enable recreation
- Ecology

The Eastern Scheldt storm surge barrier contribute to a greater or smaller extent to the first four functions. Chapter 4 discusses the functions of the Easter Scheldt storm surge barrier in more detail. In next paragraphs only the following functions are further discussed: retain water, allow tidal movement and ecology. Because these functions cope with the challenges in the Eastern Scheldt area (See Chapter 1.2 and 3).

2.3.1. Retain water
To fulfill in its water retaining function the Eastern Scheldt area is protected with a wide range of dikes, dams and hydraulics structures (See Figure 2.6).The Dutch government uses for the testing and standardizing of (primary) water defenses a so called probability of exceeding of a water level which the object should retain. For the Eastern Scheldt storm surge barrier is this a probability of exceeding of 1/4000 per year year. In 2013 the Dutch government however pronounced their wish to switch to a new and more effective standardization type for (primary) water defenses. This standardization type is based on the risk of flooding of a certain water defense system. In this approach the safety is assessed by coupling the risk of flooding to the consequences of a flood (economical damage and the number of casualties). In this way one get a clearer view where and how weak and strong certain spots of a sea defense are. This new approach makes it possible to invest more purposeful. All water defenses should fulfill in 2050 to the new requirement. To what extend this new standard will be used in the Eastern Scheldt storm surge barrier is at the time of this MSc. Thesis still unknown. Therefore the for the original design used probability of exceeding of 1/4000 per year normative water level will also be used in this MSc. Thesis.

Figure 2.6: Flood protection system Eastern Scheldt

2.3.2. Allow tidal movement

The morfology of the Eastern Scheldt area is largely determined by flow (tidal currents) and wave action. The flow in the basin causes sediment transport. This sediment transport causes erosion in the main channels of the basin or sedimentation on the tidal flats. The different types of sediment transport in water can be divided into bed load and suspended load transport.

Bed load transport

Flow velocity causes bed shear stress. When the bed shear stress is above a critical value the sediment will first start to move over the bed, which is called the bed load transport. Bed load transport is the transports of sediment close to the bed. Bed slopes have a gravitational effect on the magnitude: the slope in the initial direction of the transport (longitudinal bed slope) and the slope in the direction perpendicular to that (transverse bed slope). The longitudinal bed slope results in a change in the sediment transport rate. The primary effect of the transverse bed slope is a change in transport towards the down slope direction due to gravity.

Suspended load transport

When the flow velocity and the bed shear stress both increase the sediment will go into suspension and will be transported in the direction of the flow, this is called suspended transport. In general it is accepted that the suspended sediment transport is the sediment concentration multiplied with the flow velocity. The sediment concentration depends on several aspects like the stirring of sediment by waves and flow velocity and the settling velocity of the sediment. Thus sediment transport has a non-linear relation with flow velocity:

$$s = u^n \tag{2.1}$$

Waarin:

s	sediment transport	[m^3/s]
u	flow velocity	[m/s]

Flow velocity

The flow velocity in the tidal basin is determined by the tidal prism If the inflow is not strongly influenced by bottom friction in the inlet channel, the maximum tidal flow velocity u$_max$ is proportional to the ratio of tidal prism and cross section. In the case of the Eastern Scheldt this is can be assumed with a high probability.

2.3.3. Ecology

The tidal basin holds a variety of wildlife. I.a. several birds nesting sides are on and along the Eastern Scheldt and seals rest on the tidal flats. The tidal muds and shoals serve as an important food sites for birds and contain a lot of different plant species. Furthermore the fishery plays an important role in the Eastern Scheldt area. They fish on many species and some tidal flats are designated as oyster and mussel plots. The Eastern Scheldt also holds two main shipping channels for navigation purposes and many other shipping routes for recreational purposes.

3

Challenges Eastern Scheldt area

The challenges concerning the Eastern Scheldt area are discussed

3.1. Sand demand Eastern Scheldt

As a consequence of the compartmentalization of the Eastern Scheldt (including the construction of the Eastern Scheldt storm surge barrier), the tidal prism and the flow velocity in the tidal basin decreased with about 50 %. As a result, the net buildup of tidal flats stopped while the waves remained stable or even increased in heigh. The waves cruble the edges of the tidal flats causing the tidal flats decrease in height. On the other hand, the supply of sediment to higher tidal flats and tidal muds decreased because of the decreasing flow velocity. The tide is not 'strong' enough to transport the sediment from the main channel onto the tidal flats. This phenomenon is called the 'sand demand' of the Eastern Scheldt (in Dutch: Zandhonger). In Figure 3.1 the sand demand phenomena is graphically showed.

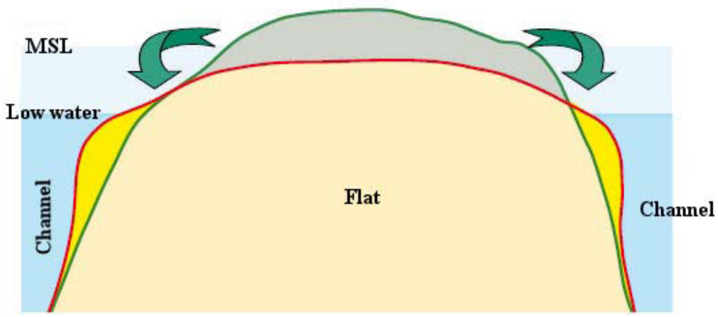


Figure 3.1: Definition sand demand Eastern Scheldt [van Zanten and Adriaanse, 2008])

3.1.1. Consequences

In 1987 Kohsiek *et al.* reported about the consequences of the decreased flow velocity on the Eastern Scheldt estuary. In [Kohsiek *et al.*, 1987] it was said that: *"If there, for any reason what so ever, is a decrease in water volume trough the main channel the flow carrying cross section of the main channel will proportionally decrease. That is exactly what, after completion of the barrier, will occur in the Eastern Scheldt: The trenches will proportional to the decrease in tidal volume fill out with sediment".* Due to progressive insights into the environmental impact of the barrier one concluded that the decrease of the inter tidal flat area is larger than expected and will continue to decrease unabated [Jacobse *et al.*, 2008]. The decrease of inter tidal flats not only affects the valuable ecosystem in the Eastern Scheldt, but also the safety of the adjoining dikes can be jeopardized [uitleggen waarom...]. Future sea level rise will only deteriorate this problem more. This decrease of tidal flats tends to continue until a new equilibrium is reached. Under the assumptions of a decrease in tidal volume of 20-30% Kohsiek *et al.* calculated the volume of sand needed to reach equilibrium is in the order of 400 à 600 million m^3 [1].

This amount of sand can be obtained from sandbanks and shoals or can be imported from the North Sea. It is not desirable that this difference in amount of sand is filled by sand from sandbank and shoals. On top of that the maximum possible degradation of the tidal flats can only supply approximately 140-160 million m^3 of sand [van Zanten and Adriaanse, Kohsiek *et al.*, 2008, 1987].

3.1.2. Proposed solutions sand demand

Through the years many possible measure to solve the 'sand demand' are discussed. On the basis of [Jongeling, 2007] five possible measures to improve the sediment transport trough the Eastern Scheldt Storm Surge Barrier into the Eastern Scheldt are discussed in [Huisman and Luijendijk, 2009], namely:

1. **Filling one or more scour hole(s) at the seaward side of the Eastern Scheldt barrier and protecting them by means of (rock) filter layers.**
   The basic idea of this solution is that by filling out the scour holes at the seaward side of the barrier (e.g. geocontainers) no sediment will be trapped at the seaward side of the barrier. In [Hoogduin, 2009] was concluded that only filling the scour holes does not contribute significantly to solving the sand demand of the Eastern Scheldt. In [Eelkema, 2013] is furthermore concluded that *"the main cause for the lack of sediment import is a lack of sediment transport capacity, not the presence of scour holes"*.

2. **Adjusting the structure of the barrier to reduce the hydraulic resistance.**
   In this solution an increase of the tidal prism can be accomplished by smoothing the slope towards the barrier or adjust the streamlining of the concrete sills and piers. Huisman and Luijendijk concluded that these measures only have a small contribution in enlarging the tidal volume.

3. **Opening the dams at the landward sides of the barrier**
   Another option in increasing the flow carrying cross section is to open the dams at the landward sides of te barrier. But extra measures like building a concrete sill are necessary. An opening by using hollow block to get an permeable dam is also possible. In [de Bruijn, 2012] research is done on: how a new inlet at the Neeltje Jans could influence the hydrodynamics and sediment transport in order to solve the 'sand demand'. de Bruijn concluded that: *"large-scale effects of the Eastern Scheldt, like the ebb dominance and 'sand demand' cannot be structurally changed with a new inlet channel"*

4. **Manipulation of the opening and closing time frames of the gates of the barrier during a tidal cycle (operation of the Eastern Scheldt barrier).**
   This measure is based on the manipulation of opening and closing regime of the gates. This manipulation should influence the flow pattern which could lead to an increase in flow velocity and probably an increasing in sediment transport in the basin. Huisman and Luijendijk, however, did not consider this as a practical solution because the possible effects are small and additional maintenance of the barrier is required.

5. **Directly or indirectly nourishing sediment into the basin.**
   Under the assumption that the storm surge barrier blocks the sediment Huisman and Luijendijk suggested to directly or indirectly, by means of a pipeline from the barrier sluices to the scour hole(s) at the basin side of the barrier, nourishing sediment into the basin. After that the tide should transport the sediment through the Eastern Scheldt. Although the costs of this measure are expected to be significant smaller than the other solution the question is if this leads to preservation of sandbanks [Huisman and Luijendijk, 2009]. A different interpretation of this measure could be the sand suppletion on inter tidal flats.

[1] This amount of sand stand equals to approximately 1 to 1.5 times the amount of sand needed for Maasvlakte 2. Where 365 m^3 sand was needed (https://www.maasvlakte2.com/uploads/factsheet_zandwinning.pdf).

3.1.3. Storm surge barrier influence on the sediment transport

Due to the decrease of the cross sectional area and the compartmentalization of the Eastern Scheldt a decrease of the tidal prism [2] and the flow velocity occurred. The construction of the storm surge barrier caused the abrupt decrease of the tidal volume in the Eastern Scheldt with about 25% from 1.230 Mm^3 to 880 Mm^3 per tide (See Figure 3.2).

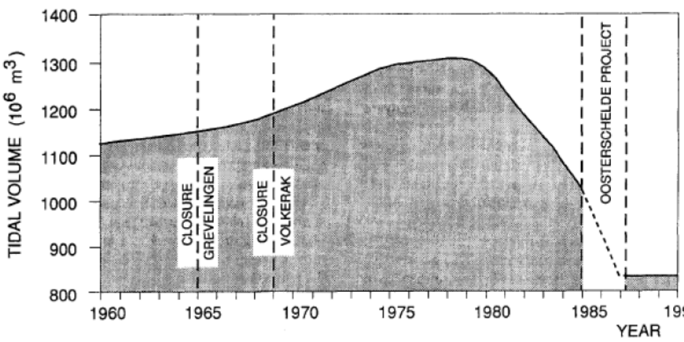


Figure 3.2: Change in tidal volume due to Delta works (Source: [Das, 2010]

In Figure 3.3 the morphodynamic equilibrium relationship between tidal volume and cross sectional area for different tidal inlets are shown. For estuary and deltas it has been shown that there is a more or less linear relation between the wet cross sectional area and the tidal prism [Deltadienst, 1967; Coastal Engineering,1994; Waterloopkundig Laboratorium; 1994]. In Figure 3.3 also indicating the effects of the Eastern Scheldt hanges in tidal volume and cross sections. The 1990' drop in the tidal volume resulted in a shortage of sediment in the main channel. Because the main channels in the Eastern Scheldt are instantly too wide for the transport volumes the flow velocities are linearly decreased from 1.5 m/s to 1.0 m/s on average tide [Oosterlaan and Zagers, 1996] [RIKZ, 1999b].

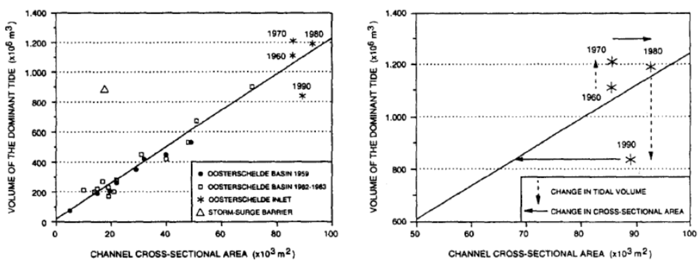


Figure 3.3: Morphodynamic equilibrium relationship between tidal volume and cross sectional area for different tidal inlets, indicating the effects of changes in tidal volume and cross sections (Source: [Das, 2010])

Conclusion barrier influence on sediment transport

From this can be concluded that an enlargement of the cross sectional area could influence the tidal prism - cross sectional area relation in a positive way. Also the flow velocity would increase.

3.1.4. Current policy sand demand

In solving the 'sand demand' Rijkswaterstaat pronounced in [RWS7, 2013] their preference for a phased decision making. In phase 1 (2015-2025) the measures include the suppletion of 1.65 million m^3 sand on the Roggenplaat. This measure only secures the short-term goals on preserving the inter tidal flats. After phase 1 the measure should be evaluated to determine the approach for phase 2 (2025 - 2060). Also future knowledge about sea level rise, further development on the erosion causes by the 'sand demand' and the developments on the population of the stelt (in Dutch: Steltloper) should be included in the decision making for phase 2.

[2]The tidal prism is the volume of water per tide which passes a certain cross section

3.2. Maintenance Eastern Scheldt storm surge barrier

After commissioning the Eastern Scheldt storm surge barrier in 1986, scheduled maintenance of the barrier started. For the non replaceable part of the barrier a lifetime of 200 years was designed [RWS1, 1985]. For the gates and the operating machinery a lifetime of at least 50 years is accounted for [RWS4, 1985]. This because for this part fatigue was governing. For these parts of the barrier periodic maintenance is scheduled.

Scour protection

Also for the scour protection of the barrier schedule maintenance was planned. Till recently one thought this maintenance policy worked out well. Until in 2012 it was found that as a consequence of deepening of the scour holes at the edges of the scour protection large liquefaction and land shift have occurred. The most pressing consequence of this was that the stability of the primary water defense on the coast of Noord-Beveland was no longer guaranteed. In addition there are damages at the edge of the scour protection observed due to liquefactions. In the current situation there are scour holes at each location of the moveable barrier. The current scour holes have a depth of 21 to 34 m. De expectations are that these depths can grow to 25 to 75 m in 2050. The maximum scour hole depth now amounts 34 m (NAP -60 m). These scour depths where already foreseen in the original design of 1976 but this is adjusted in 1982 en 1988. The scour holes develop slower then in 1988 was expected, but the continuous deepening of the scour holes means that at certain places side slopes become steeper. The scale of the instabilities of the side slope at four locations (Roompot East (2x), Roompot West, Hammen-East) reached such a level that the scour protection is damaged. This phenomenon has been underestimated in previous design and maintenance plans ([RWS6, 2013]). In [RWS6, 2013] the project group OOS pronounces, regarding the scour holes, a few control measures. These measures include:

- A stone dumping program, in addition to the emergency rock fills in 2012 and 2013, to protect the edge of the prefabricated mats and to protect too steep slopes in the scour holes.
- To elongate the already applied scour protection for the coast of Noord-Beveland to the bottom of the scour holes. This in combination with the removal of loosely packed sand on higher slopes.

Costs

After the construction of the barrier a maintenance plan was presented in which the annual maintenance cost are approximated 17 million guldens per year [RWS9, 1987]. The current maintenance cost of the Easter are estimated between 10 and 18 million euro per year [3]

Conclusion maintenance

The alternatives for an improved functionality of the Eastern Scheldt storm surge barrier should not further deteriorate the problems with the scour holes. Because of an steeper slope more liquefactions may occur in the direction of the barrier axis. This could compromise the stability of the Eastern Scheldt storm surge barrier. Furthermore safety of the primairy sea defence at the coast of Noord-Bevelands should not be compromised. With respect to the replaceable parts of the Eastern Scheldt storm surge barrier (See Paragraph 3.2) should the alternatives not lead to an increased maintenance time period.

3.3. Sea level rise

The developments in the course of the future sea level rise will affect the lifetime of the barrier. Furthermore it accelerates the degradations of the inter tidal flats, because by a increasing water level the time when inter tidal flat are dry become less. In the last decade a few scenarios on the sea level rise are presented in [IPCC, KNMI, Deltacommittee, 2013, 2006, 2008] but because of the uncertainties in the different scenarios it is questionable if and how the barrier should be adjusted with respect to the different scenarios. In the next paragraph the different sea level rise scenarios will be discussed and their relation to the barrier is explained.

[3] http://www.rijkswaterstaat.nl/water/feiten_en_cijfers/dijken_en_keringen/oosterscheldekering/index.aspx

3.3.1. Scenarios

Extrapolate the measured sea level rise 1901-2010

Globally the sea level has rose 19 cm between 1901 and 2010. This means a yearly average sea level rise of 1.7 mm. Looking to specific time intervals a yearly average sea level rise was 2.0 mm was observed between 1971 and 2010 and 3.2 mm/y between 1993 and 2010[IPCC, 2013]. A scenario could be that this trend 3.2 mm/ year continues.

Intergovernmental Panel on Climate Change (IPCC)

At the end of 2013 the IPCC presented their 5th Assessment Report on Climate change [IPCC, 2013]. In which stated that the sea level will continue to rise during the 21st century. Furthermore they concluded that it is 'very likely' the rate of sea level rise will exceed the observed sea level rise during 1971 to 2010. This is according to the IPCC due to increased ocean warming and the increased loss of mass from glaciers and ice sheets. The IPCC have considered four sea level rise scenarios (RCP2.6, RCP4.5, RCP6.0 and RCP8.5). The global mean sea level rise for 2081–2100 relative to 1986–2005 will likely be in the ranges of:

- *Scenario RCP2.6:* 0.26 to 0.55 m
- *Scenario RCP4.5:* 0.32 to 0.63 m
- *Scenario RCP6.0:* 0.33 to 0.63 m
- *Scenario RCP8.5:* 0.45 to 0.82 m

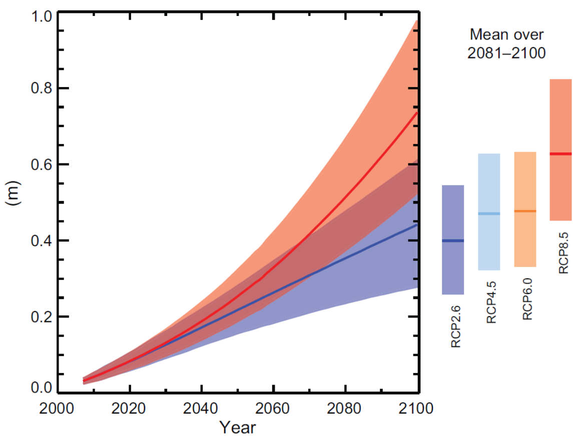


Figure 3.4: Global mean sea level rise over the 21st century relative to 1986–2005 [IPCC, 2013]

For RCP8.5, the rise by the year 2100 is 0.52 to 0.98 m, with a rate during 2081 to 2100 of 8 to 16 mm/y. In Figure 3.4 the different scenarios are graphically presented.

Deltacommittee 2008

In [Deltacommittee, 2008] the Deltacommittee advised the Dutch government *on how to protect the Dutch coast and the low-lying hinterland against the consequences of climate change*. The goal of the committee was to ensure a long term safety against flooding. The Delta Committee concludes that: "a regional sea level rise of 0.65 to 1.3 m by 2100, and of 2 to 4 m by 2200 should be taken into account". In this calculation the Deltacommittee included the effect of land subsidence and they based their predictions on the IPCC 'likely' upper limit of +6°C in 2100 with respect to 1990. Furthermore they included an extreme extrapolation of the uncertainties of the icecap dynamics.

KNMI 2006

Based on IPCC Fourth Assessment Report: Climate Change 2007 the KNMI presented in [KNMI, 2006] four scenarios for the future sea level rise. The KNMI uses in their predictions a global temperature rise of +2°C of +4° in 2100 with respect to 1990. The result are a focused on the year 2050 and 2100. Concerning the upper and lower limit the sea level rise around 2050 varies in the scenarios between 15 and 35 cm. Around 2100 the sea level rise varies between 35 and 85 cm. The sea level will continue to rise after 2100, and in 2300 it will amount to 1 m up to 2.5 m. The difference with the predictions of the Deltacommittee 2008 lies in the less extreme extrapolation of the uncertainties of the icecap dynamics.In Figure 3.5 the difference between the KNMI and Deltacommittee scenarios are graphically presented.

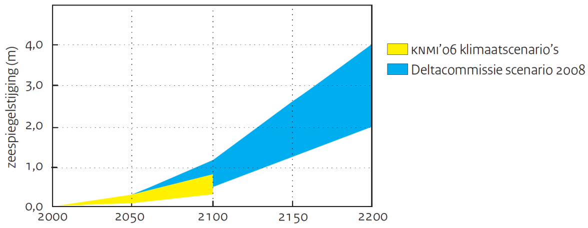


Figure 3.5: The sea level increase off the Dutch coast expected in 2050, 2100 and 2200 [KNMI, 2009]. *(Year of reference 1990, land subsidence is not included in these data)*

3.3.2. Future sea level rise and Eastern Scheldt storm surge barrier

As mentioned, the design lifetime of the barrier is 200 years (started from 1986). The retaining height of the Eastern Scheldt storm surge barrier is NAP +5.6/+5.8 m. With the design water levels of NAP +5.3/+5.5 m it seems 0.3 m sea level rise is accounted for. From Paragraph 3.3.1 can be concluded that most sea level rise scenarios surpass the 0.3 m accounted for. In responds to these sea level rise scenarios could the barrier be adapted or the lifetime should be revised downwards. In [Deltacommittee, RWS8, 2008, 2011] statements about this topic are done.

In [RWS8, 2011] a vision for the future with respect to the Southwestern Delta is explained. About the storm surge barrier the following is said: *"The future of the Eastern Scheldt storm surge barrier is of great importance to the safety of the Southwestern part of the Delta. Based on the current closing rate the dikes and hydraulic structures should grow with the rising sea level. If an increasing closing rate is acceptable this growth could somewhat damped. The lifetime of the Eastern Scheldt is 200 years but the moving parts of the barrier need to be replaced at an early stage. It would, therefore, be the moment to adjust the barrier for a possible increasing sea level. Possibilities are for example enlarging the gates".*

The Deltacommittee gives in [Deltacommittee, 2008] a somewhat extreme solution by coupling their statement about the barrier to their sea level rise predictions (a regional sea level rise of 0.65 to 1.3 m by 2100, and of 2 to 4 m by 2200, including the effect of land subsidence). They says about this topic the following:

To 2050 *"The Eastern Scheldt storm surge barrier is adequate until at least 2050. The barrier's disadvantages (restricting tidal action) should be alleviated soon by compensating the losses in the intertidal zones with nourishment, bringing in sand from outside (from the shallows seaward of the Delta Works, for example)".*

Post 2050 *"Extend the life of the Eastern Scheldt storm surge barrier. This can be done up to a sea level rise of around 1 m. Estimates of maximum sea level rise give 2075 as the earliest year when this can occur, but it could happen as late as 2125 or thereabouts. After that time, measures must be taken to guarantee safety. At such time as the Eastern Scheldt barrier no longer suffices, the Committee can see good arguments for implementing such safety solutions as will restore (nearly) all the tidal dynamics of the Eastern Scheldt. Choices will have to be made several decades before the barrier reaches that point, so that the full range of options can be employed".*

3.4. New safety standards

The old safety standard for sea defenses are based on an exceedance of water levels. In 2013, the Minister of Infrastructure and Environment in 2013, suggested a new and more efficient standardization of the safety of primary water defenses. She suggested a standardization based on the probability of flooding. The new standard are expected to be used from 2017. With this new it will be possible to see more clearly where the strengths and weaknesses in the water defenses are and makes it explicit how strong or weak it actually is. In this way investment can be done more efficient. During the completion of the MSc. Thesis new standards for storm surge barriers and dam are still under developed and therefore not been used. In Figure 3.6 the suggested standardization of water defenses in the Netherlands are displayed.

Figure 3.6: Suggested standardization water defenses

3.5. Conclusion challenges

The conclusions of the previous treated challenges are summarized in the next paragraph. From the analyses of the challenges requirements will be determined. The requirement form the basis from which feasible alternatives for an improved functionallity of the Eastern Scheldt storm surge barrier are drafted.

Sand demand

From this can be concluded that an enlargement of the cross sectional area could influence the tidal prism - cross sectional area relation in a positive way. Also the flow velocity would increase. In solving the 'sand demand' Rijkswaterstaat pronounced in [RWS7, 2013] their preference for a phased decision making. In phase 1 (2015-2025) the measures include the suppletion of 1.65 million m^3 sand on the Roggenplaat. After phase 1 the measure should be evaluated to determine the approach for phase 2 (2025 - 2060). In [RWS7, 2013] one of the scenarios for phase 2 includes a 100 % preservation of the tidal flats. This scenario describes one suppletion until 2025 and from 2025 to 2060 one suppletion every five years. In total 65 million m^3 sand is supplemented. The costs of this measure are estimated on 422 million euros. In this MSc. Thesis the 100 % preservation scenario will serve as starting point from which more cost effective alternatives should be determined.

Maintenance

The alternatives for an improved functionality of the Eastern Scheldt storm surge barrier should not further deteriorate the problems with the scour holes. Furthermore safety of the primary sea defense at the coast of Noord-Beveland should not be compromised. With respect to the replaceable parts of the Eastern Scheldt storm surge barrier (See Paragraph 3.2) the alternatives not lead to an increased maintenance time period.

Sea level rise

From the IPCC report [IPCC, 2013] different sea level rise scenarios are listed in Paragraph 3.3.1. In this MSc. Thesis a certain sea level rise should be included. For the determining of feasible alternatives for an improved functionality of the Eastern Scheldt storm surge barrier it seems reasonable to extrapolate the observed sea level rise in the past. But the IPCC concluded in their report that it is 'very likely' the rate of sea level rise will exceed the observed sea level rise during 1971 to 2010 (2 mm/year). Assuming a sea level rise of 3.2 mm/year (which is observed for the period between 1993 and 2010 [IPCC, 2013]) the sea level rise in the year 2100 would be 0.30 m [4]. This 0.30 m falls within the RCP2.6 scenario of the IPCC. That is why for this MSc. Thesis is chosen to take the RCP2.6 scenario upperbound of 0.55 m for the relative sea level rise in the year 2100. This sea level rise should be taken into account in analyzing the current Eastern Scheldt storm surge barrie, in the determination of an altnerative for an improved functionallity of the barrier and in the effect of the acceleration of the degradation of the inter tidal flats by the sea level rise.

Effect Eastern scheldt storm surge barrier

In the previous paragraph was stated that in determining alternatives for an improved functionality of the Eastern Scheldt storm surge barrier a sea level rise of 0.55 m should be accounted for. In [Leeuwdrent, 2012] is concluded that a sea level rise of 0.5 m is not signify structural failure of the Eastern Scheldt storm surge barrier and its component. Since this assumed 0.5 m not differs too much from the 0.55 m which should be accounted for, it can be assumed that with respect to sea level no improved functionality is needed. The parts of the Eastern Scheldt storm surge barrier are sufficient to fulfill in its water retaining function until 2100. For the required design lifetime of the alternatives 50 years accounted for.

[4] Relative to the sea level rise between 1971 to 2010 with the base level the year 2005. Land subsidence is not included in this sea level rise

4

Analysis of the current design Eastern Scheldt storm surge barrier

This chapter elaborates on the subreachseach questions 1 & 2 (as mentioned in Chapter 1.5.1). First the historic development of the design is treated. Following by defining the objectives of the construction of the Eastern Scheldt storm surge barrier. Then the functions of the Eastern Scheld area from Chapter 2 are being translated into functions which are insured by the barrier. The link between the main functions and the challenges in Chapter 3 is made and the relation of the functions to the Eastern Scheldt area is explained. Finally the discussion about the current functionality of the storm surge barrier serves as a stepping stone to the analyses into the desired situation of the Eastern Scheldt storm surge barrier in 5.

4.1. Historic development of the design

In 1967 the preparations for the closure of the Eastern Scheldt by a 8 km long dam started. First three islands (Roggenplaat, Neeltje Jans and Noordland) where constructed by heightening the shallow parts in the Eastern Scheldt inlet. After that, Neeltje Jans and Noordland where connected by a 4 km long dam. The three remaining openings (The Hammen, Schaar van Roggenplaat en The Roompot) with a length of 3 km where left to close [Antonisse, 1986]. Before one can proceed to dam the remaining part the construction stopped under the pressure of first the public opinion (especially opposition from fisherman and environmental pressure groups) and later political pressure. In 1974 a independent Committee Oosterscheldekering (Committee Klaasesz) advised the government to construct an alternative in which the tide in the Eastern Scheldt could, more or less, be maintained. This resulted first in an alternative with a permeable dam and later on an alternative with a closeable storm surge barrier. The alternative with a closeable storm surge barrier resulted in the Eastern Scheldt storm surge barrier as we know today (See Figure 4.1). In the current situation the total length of the barrier is 9 km in which 3 km is covered by the movable barrier. The movable barrier is divided over three locations: The Hammen, Schaar and the Roompot (which included the largest part of the movable barrier). The wet cross sectional area of the barrier is approximately 17,900 m^2 [RWS3, 1985]. Which is a decrease of approximately 80% (from 80,000 to 17,900 m^2). The tidal prism decrease with approximately 25%. In Figure 4.2 the top view lay-out of the whole barrier is shown.

4.2. Objectives Eastern Scheldt storm surge barrier

From [RWS1, 1985] the objectives of the Eastern Scheldt storm surge barrier are gained. Behind every objective the corresponding function is listed.

- To protect the areas behind the barrier against high storm surges in accordance with the standards of the Delta Committee. To secure the dikes and hydraulic structures around the Eastern Scheldt (Water retaining function).

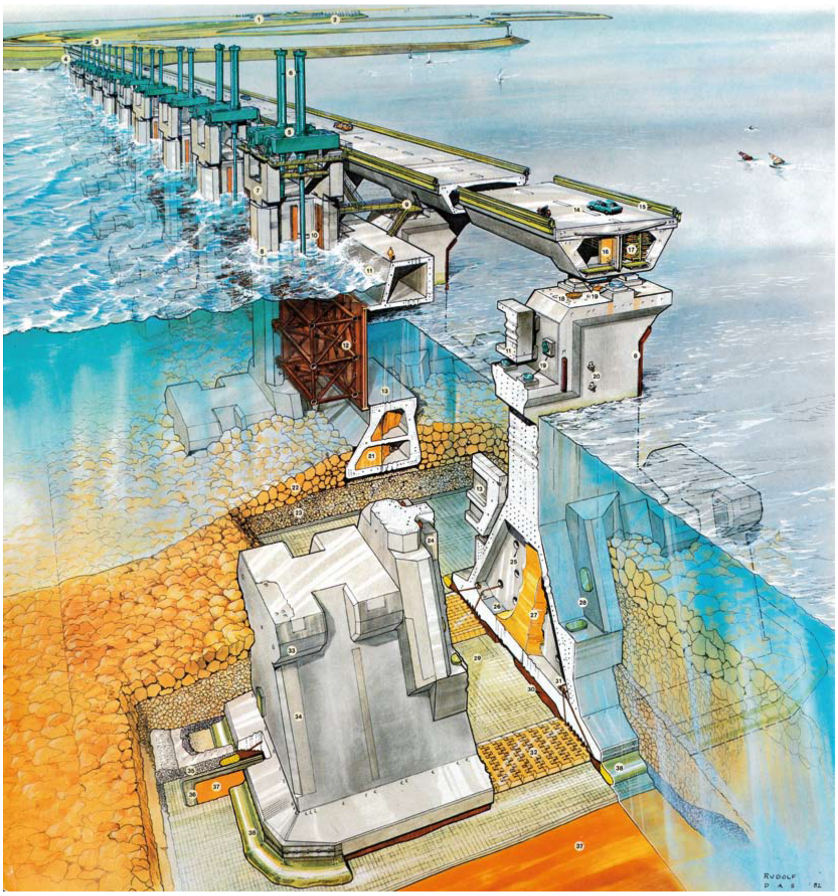


Figure 4.1: Moveable part Eastern Scheldt storm surge barrier [Biesboer, 2011]

- To maintain the existing environment of the Eastern Scheldt estuary given a certain minimum (Allow tidal movement function).
- To provide a road connection between Schouwen and Noord-Beveland (Transportation function).
- In regard to navigation: A (limited) direct connection between the Eastern Scheldt and the North Sea has to be maintained (Transportation function).
- To use the created infrastructure for other purposes such as recreation and industrial development (Recreation function).

In Figure 4.3 the whole barrier is presented in a chart. The different part of the moveable barrier are highlighted. In de next paragraph the link between the parts of the moveable barrier and its functions is laid.

4.2.1. Functions

In Chapter 2.3 the functions of the Eastern Scheldt area are discussed. The functions in which the barrier fullfills are:

Retain water

This function is fulfilled by the moveable barrier, the solid barrier and the navigation lock. It means that during storm conditions the barrier safely retains storm surges up too a water level and a wave height with a probability of exceedance of 1/4000 per year. The accompanying water levels with this probability of failure are NAP +5.5 m at the Noord-Beveland side and NAP +5.3 m at the Schouwen side. A certain discharge volume penetrating the barrier (through leakage or wave overflow) is acceptable because of the Eastern Scheldt basin storage capacity. Furthermore the barrier should be able to retain a negative water head. This means that the water level in the basin is heigher than the sea level.

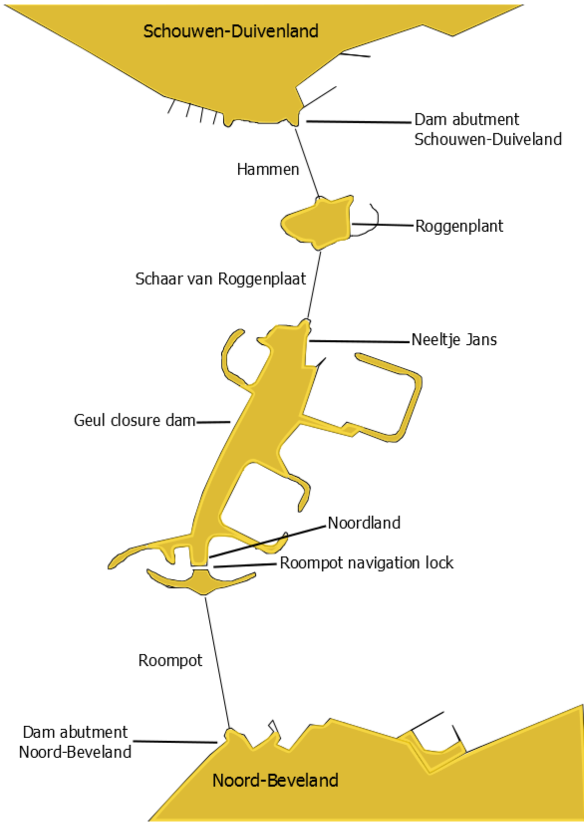


Figure 4.2: Lay-out Eastern Scheldt storm surge barrier

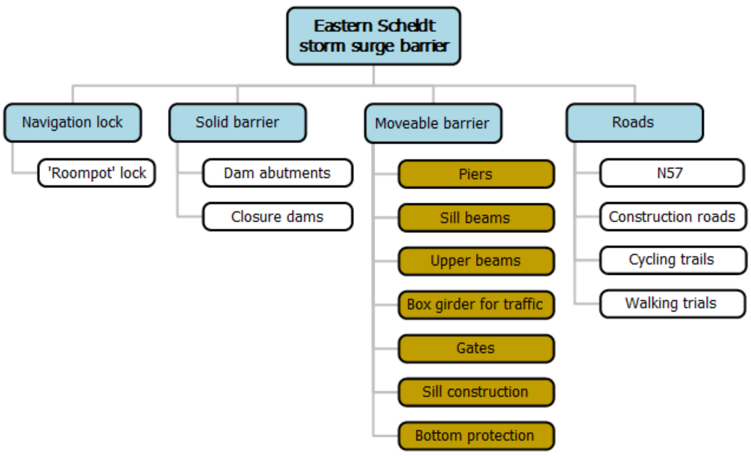


Figure 4.3: Parts Eastern Scheldt storm surge barrier

Allow tidal movement

During normal conditions the Eastern Scheldt storm surge barrier allows tidal movement in the Eastern Scheldt. This is needed to minimize the the morphological disturbance of the environment. This function is fulfilled by the moveable part of the barrier. With respect to the original tide before the construction of the barrier a limited reduction of the original tidal difference is accepting. A minimum tidal difference of 2.7 m at Yerseke is taken as a reference level. This means in practice that the barrier have a sufficient effective cross sectional area μA.

Provide transportation

The barrier must provide a road connection between Schouwen and Noord-Beveland. With respect to navigation, the barrier should provide a connection between the Eastern Scheldt and the North Sea. The motorway N57, roads for construction traffic and several cycling and walking trails provide a road connection between Schouwen and Noord-Beveland. The navigation lock 'Roompot' foresees in a connection between the North Sea and the Eastern Scheldt.

Provide recreation

It should be possible to use the barrier for recreational purposes. These function is mainly fulfilled by parts of the solid barrier (e.g. Neeltje-Jans).

4.3. Analyses moveable part Eastern Scheldt storm surge barrier

In Chapter 1 the different parts of the barrier are already briefly discussed. The Eastern Scheldt storm surge barrier consists of a navigation lock, a solid barrier, a moveable barrier and roads. This MSc. Thesis focuses on the moveable part of the barrier. The movable barrier is divided over three locations: The Hammen, Schaar and the Roompot (See Figure 4.2). The Roompot included the largest part of the movable barrier. The main parts of the moveable barrier are: the piers, the sill beams, the upper beams, the steel gates, the sill construction, the foundation bed, the scour protection and the box girders for traffic. Characteristic of the components of the movable barrier is that they form one system (inter-dependency) that has to carry out the assigned functions.

The non replaceable parts of the barrier (e.g. the piers, the sill beams, the upper beams, the box girders and sill construction) are designed for a lifetime of 200 year [RWS1, 1985]. For the gates and the operating machinery a design lifetime of at least 50 years was accounted for [RWS4, 1985]. During this lifetime boundary conditions may change, because of sea level rise. In figure 4.4 the main parts of the barrier are shown. In the next paragraphs the main parts of the moveable barrier are treated.

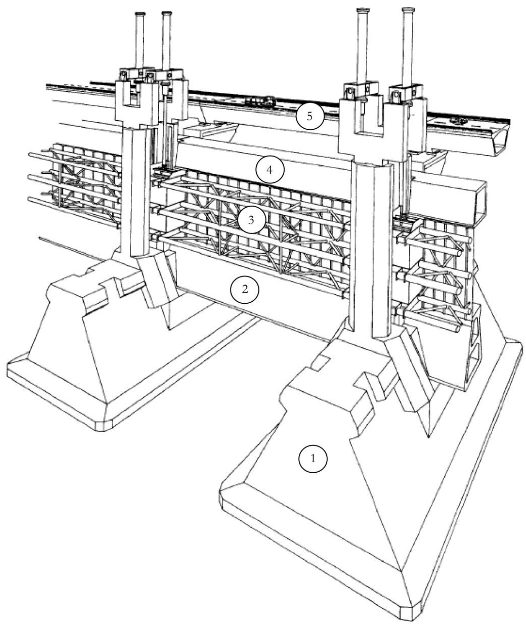


Figure 4.4: Main parts barrier
1. Pier, 2. Sill beam, 3. Gate, 4. Upper beam, 5. Box girders for traffic [Leeuwdrent, 2012]

4.3.1. Piers

The 65 concrete piers have a height between 30-40 m depending on their position. The piers not only support the upper beams and the box girders for traffic but serve as a support for the hydraulic cylinders, the supporting operating equipment and steel lifting gates.

Strenth
The water level with a probability of 1/4000 per year is NAP +5.5 at the Noord-Beveland seaside (Southern part of the barrier) and +5.3 m at the Schouwen seaside (Nothern part of the barrier). The water level at the Eastern Scheldt side is NAP -0.7 m. Taking into account settlements and relative sea level rise [1] the design water level becomes NAP +5.8 m (Roompot and Schaar van Roggenplaat) and NAP +5.6 m (Hammen).

4.3.2. Sill beams
As a part of the sill construction 62 concrete sill beams of 39 m are placed between the piers.

Strength
For the sill beams the same water level are taken as the piers. Namely, NAP +5.5 m for the extreme sea level and NAP -0.7 m for the water level in Eastern Scheldt.

4.3.3. Gates
The Eastern Scheldt storm surge barrier has 62 steel gates. The gates are connected to its accompanying lifting equipement. In its existence the barrier is closed 25 times of which the last time in 2013[2]. So with a global frequency of once per year. In the past years maintenance on the hydraulic cylinders, the beam supporting operating equipment and lifting gates is done. The work mainly consists of preserving the steel parts. Corroded parts are covered and repainted.

Strength
For the gates three governing load cases are defined in [RWS4, 1985]: 1) End of closing, 2) Closed, positive water level difference (Governing load case horizontal direction) 3) Closed, negative water level difference (Governing load case vertical direction).

1. End of closing:

- Sea level NAP +4.4 m
- Water level Eastern Scheldt NAP +0.2 m
- Significant wave height Hs = 3.4 m
- Waveperiod Tp = 5.6 s

2. Closed, positive water level difference

- Sea level NAP +5.5 m
- Water level Eastern Scheldt NAP -0.7 m
- Incoming wave height Hi = 5.75 m
- Waveperiod Tp = 9.5 s

3. Closed, negative water level difference: Governing for the sheeting and the stops of the gate.

- Sea level NAP -1.0 m
- Water level Eastern Scheldt NAP +2.4 m
- Incoming wave height Hi; = 0.38 m
- Waveperiod Tp = 5.0 s

The governing load case for the vertical load (wave impact) is reach when the water level at the sea side of the barrier NAP +3,5 m. Maximum vertical load:

[1] Relative sea level rise is the ...
[2] http://www.rijkswaterstaat.nl/water/feiten_en_cijfers/dijken_en_keringen/oosterscheldekering/index.aspx

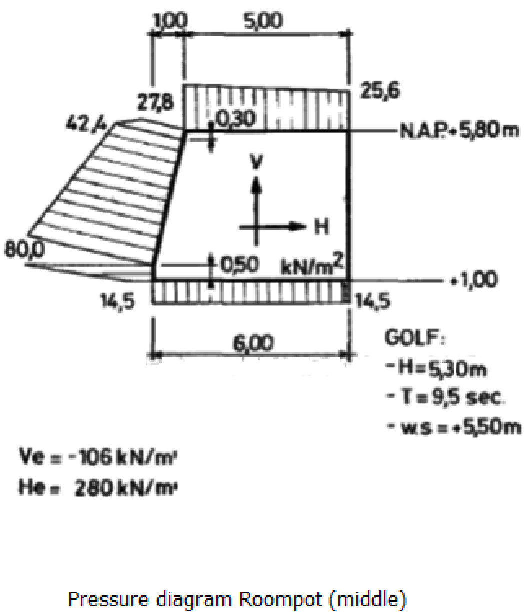
- Sea level NAP +3.5 m
- Water level Eastern Scheldt NAP -0.7 m
- Significante golfhoogte Hi = 6.2 m
- Golfperiode Tp = 6.1 s

4.3.4. Upper beams

At the top level of the barrier 62 upper beams are placed between the piers. The top level of the upper beams is NAP +5.80 m. Which is also the water retaining height.

Strength

Governing load case is the wave impact:



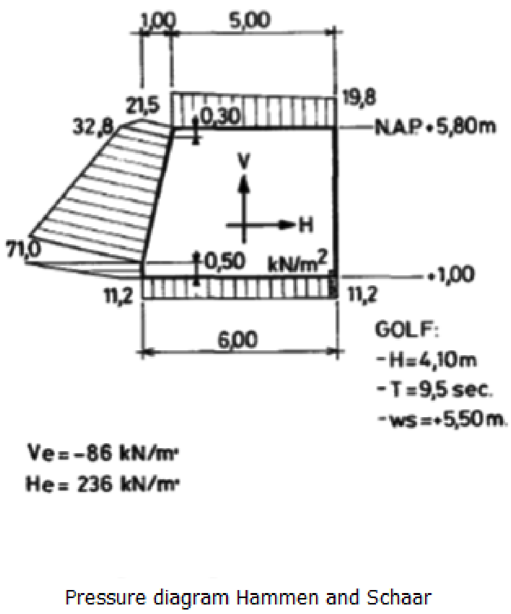


Figure 4.5: Governing load case upper beam [RWS1, 1985]

4.3.5. Box girders for traffic

The concrete box girders which are placed on the piers form the traffic connection between Schouwen-Duiveland and Noord-Beveland.

4.3.6. Foundation bed

Functional requirement for the foundation bed and foundation subsoil underneath the piers are that the temporary or permanent deformations in the foundation which cause tilting of the piers and/or differences in positions between the piers (caused by stresses on the piers or by other processes), are bound to certain acceptability limits during the entire life-span of the barrier'. Considering the required life-span of 200 years for the barrier, meeting this requirement is of decisive importance for the effectiveness of the barrier, especially since it is almost impossible to repair or compensate significant deformations.

4.3.7. Scour protection and sill construction

The scour protection and the sill construction ensure the stability of the barrier is not being compromised by scours holes in the axis of the storm surge barrier. Although the barrier should fulfill in its water retaining function it can happen during or after storm surge conditions gates are fail to close or open. To still fulfill in the requirement to ensure stability of the barrier three load cases are specified.

- The barrier is opened under storm surge conditions
- One or more gates are open under storm surge conditions
- One or more gates will not open, especially when there is a negative head.

The scour protection and sill construction are design in such a way that failure is excluded.

Strength

In Table 4.1 is per load case indicated what water head difference the scour protection and sill construction should withstand

Table 4.1: Design load sill and transitional structure

Water head difference	Failure	Begin of motion of scour protection
Load case	[m]	[m]
Closed barrier	5.3	3.4
Closed barrier + rejecting gate	4.15	2.4
Open barrier	1.7	1.5
Opening barrier	-	1.5
Closing barrier	4.2	-
Open barrier + reversed w.l.	1.7	1.5

5

Desired situation Eastern Scheldt storm surge barrier

In this chapter sub research question 3 is elaborated. First possibilities to improve the functionality of the Eastern Scheldt storm surge barrier are discussed. Then alternatives are drafted from these improvement possibilities. The consequences for the Eastern Scheldt area are discussed and finally the alternatives are assessed by comparing them with the current situation.

5.1. Improved functionality

In Chapter 3 & 4 the current barrier and its functionality is already discussed. Progressive insights into the environmental impact of the barrier shows that the decrease of the inter tidal flat area is larger than expected and will continue to decrease unabated, [Jacobse *et al.*, 2008]. Jacobse *et al.* claim that the tidal muds and salt marches will decrease in height by 50 to 100 cm over a period of decades. The decrease of inter tidal flats not only affects the valuable ecosystem in the Eastern Scheldt, but also affects the safety of the adjoining dikes. The higher risk on the flood safety can be the result of an increase of wave height or the foreland decreases largely which could lead to instability of the outer slope. Future sea level rise will deteriorate this problem. This means that especially in the 'allow tidal movement' function (See 4.2.1) additional functionality can be achieved.

5.1.1. Improve tidal movement function

The Eastern Scheldt storm surge barrier allows tidal movement in the Eastern Scheldt during normal conditions. Because morphological changes mainly occur during successive spring tides in normal conditions there should be searched for alternatives which are effective during normal conditions. [Das, 2010] concluded that the tidal flow velocity is the governing forcing for shoal build up. The idea is that by enlarging the cross sectional area of the barrier the tidal volume through the barrier and the tidal flow velocity in the Eastern Scheldt enlarges. This enlargement of the tidal velocity in the Eastern Scheldt should contribute in an improvement of the sand demand problem. To see what expansion of the cross sectional of the Eastern Scheldt storm surge barrier are feasible the contribution of the different parts of the barrier to the reduction of the cross sectional area are calculated in Appendix C and summed up in Table 5.1. The total reduction of the cross section area after the construction of the Eastern Scheldt storm surge barrier is A_{red} = 80,000-17,900 = 62,100 m^2. In Table 5.1 can be seen that the solid barrier, the sill beams and the piers take up the largest part of the original cross sectional area of the Eastern Scheldt inlet. The MSc. Thesis focuses on alternatives concerning the moveable part of the barrier (See Chapter 1.6). Further analyses to the piers, the sill beams and the foundation bed should therefore reveal where expansion of the cross sectional area is feasible.

Piers

Removing some of the piers have a great impact on the expansion of the cross sectional area. But removing the piers comes with high executional challenges. Furthermore, by removing the piers the supportive function of the box girders for road traffic expires. Also by removing the piers the connection

Table 5.1: Contribution of the different barrier parts to the reduction of cross sectional area

Part	Percentage of A_{red} [%]
Piers	30
Sill beam	29
Solid barrier	25
Foundation bed	14
Sill construction	2

of the gates get lost. This means the flood safety function of the storm surge barrier is compromised and should be restored. For that reason alternatives which includes the removal of (parts of) the piers are therefore not included in this MSc. Thesis.

Sill beams

Expanding the cross sectional area of the Eastern Scheldt storm surge barrier by adjusting the sill beams seems reasonable.

Foundation bed

Although the foundation bed contributed largely to the reduction of the cross sectional area it will not be included in the search for feasible alternatives. Because adjustment or removal of the foundation bed could lead to serious stability problems of the piers.

5.1.2. Conclusion improvement tidal movement function

Adjustment to the sill beams are taken as the main subject in the search for feasible alternatives. This means the range in which alternatives are feasible are bounded by:

1. The cross sectional area of the barrier in the current situation: effective cross sectional area of 17,900 m^2 (A_{min}).
2. The cross sectional area of the barrier whitout the sill beam (A_{max}).

To get an idea of the effectiveness of certain measures Figure 5.1 is taken as reference. Figure 5.1 shows the relation between the effective cross sectional area and the reduction of the tidal difference as a percentage of the original tide at Yerseke. The orginal tidal difference at Yerseke was NAP +3.50 m. The search range is marked with blue. The left boundary (A) is the cross sectional area of the barrier in the current situation (A_{min}). The right boundary (C) is the cross sectional area of the barrier without the sill beam (A_{max}). The zero percent reduction (D), with an effective cross sectional area of 80,000 m^2, refers to the situation before the construction of the barrier.

In the next paragraph the two reduction alternative are specified in more detail.

5.2. Alternatives

In this section three alternatives for the Easter Scheldt storm surge barrier are presented. The possibilities are viewed to go back to 20 % (B) and respectively 10 % (C) reduction of original tide (before the construction of the barrier) at Yerseke, in stead of the 30 % reduction (A) which is the case in the current situation. Less reduction of the original tide could contribute in less disturbance of the barrier in the ecosystem of the Eastern Scheldt. In the next paragraphs a possible alternative is outlined for case B and case C (See Figure 5.1).

5.2.1. Baseline alternative

In the baseline alternative the current barrier will be retained and maintained according current policy. To fight the negative effects of the sand demand Rijkswaterstaat pronounced in [RWS7, 2013] their preference for a phased decision making (See Paragraph 3.1.4). In phase 1 (2015-2025) the measures include the suppletion of sand on the Roggenplaat. After phase 1 the measure should be evaluated to determine the approach for phase 2 (2025 - 2060). In this baseline alternative the scenario of 100 %

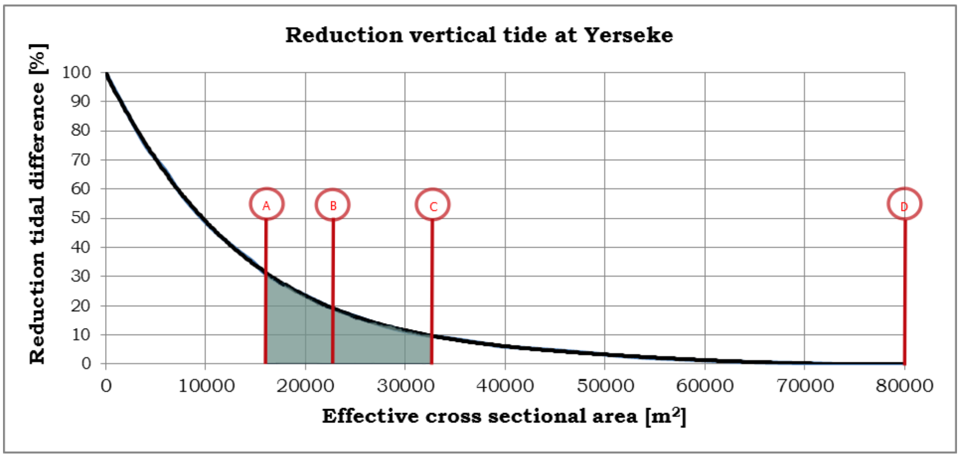


Figure 5.1: Reduction of the vertical tide at Yerseke with respect to the effective cross sectional area (based on [RWS1, 1985]) *(A) Current situation, (B) 20 % reduction of the vertical tide, (C) 10 % reduction of the vertical tide, (D) Situation before construction of the barrier*

preservation of tidal flats, tidal muds and salt marches is taken as a reference. In Figure 5.2 the cross section of the barrier is shown to compare it with the other alternatives.

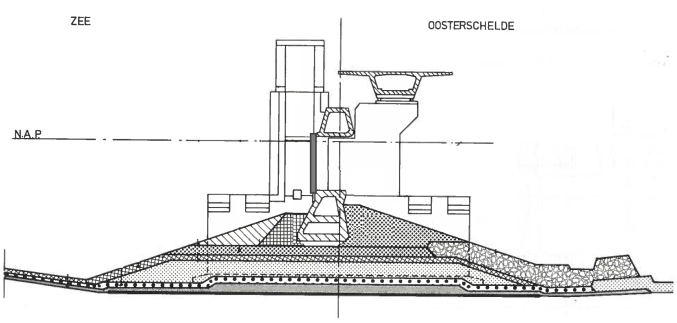


Figure 5.2: Cross section current barrier

5.2.2. Lowered sill beam alternative

In this alternative the effective cross sectional area will increase with approximately 6,200 m^2 (from 17,900 to 24,100 m^2). Based on Figure 5.1 this means a the vertical tide reduction at Yerseke is 20 % with respect to the vertical tide before the storm surge barrier. With the "20 %" solution a (relative) improvement is expected with respect to the sand demand. This is investigated further in Chapter 5.4. The measures in this alternative include the following measures:

1. Lowering of 50 sill beams
2. Extension of 50 gates
3. Strengthening of the sill construction

Lowering sill beams

In stead of removing a whole sill beam was chosen to lower the sill beams. This is to ensure a better connection between the gate and the sill and to reduce the leakage during storm. From a safety point of view is chosen to not lower the outer sill beams to reduce the possible negative effects on the shores of Schouwen and Noord-Beveland.

Extension gates

The gate extension can be realized by means of a new gate or an adjustment to the current gates. With the extended gates the lifting capacity should not be exceeded.

Strengthening sill construction

Lowering of the sill beam could lead to a different stream pattern. Possible strengthening of the stones in the sill construction should guarantee the stability.

A typical cross section of the alternative is shown in Figure 5.3.

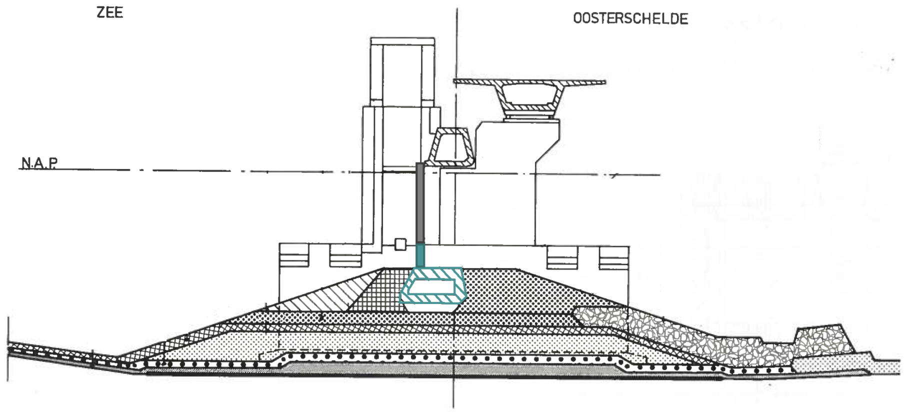


Figure 5.3: Cross section lowered sill beam alternative
The adjustments are marked with blue

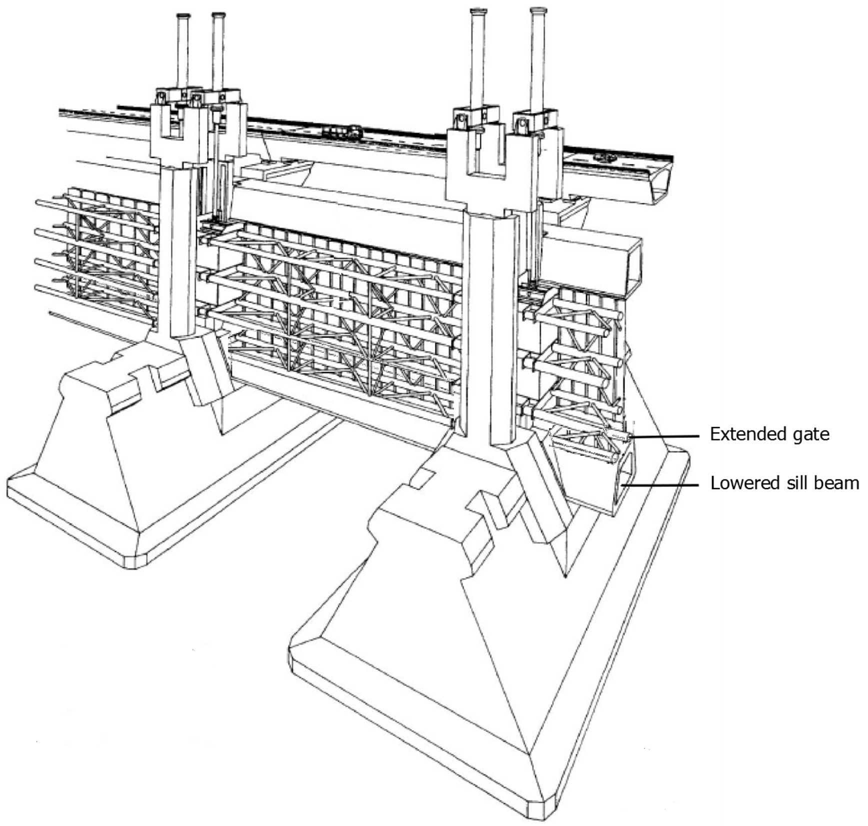


Figure 5.4: Frontview lowered sill beam alternative

5.2.3. Lowered sill alternative

The measures generate an increase of the effective cross sectional area with approximately 14,100 m^2 (from 17,900 to 32.000 m^2). Based on Figure 5.1 lowering of the sill beams have significant impact on the vertical tide at Yerseke. With this alternative the vertical tide reduction at Yerseke is 10 % with respect to the vertical tide before the storm surge barrier. With the "10 %" solution a large improvement is expected with respect to the sand demand. The measures in this alternative include the following measures:

1. Removal of all the gates
2. Removal of all the sill beams
3. Lowering of the sill construction
4. Strengthening of the sill construction
5. Strengthening dikes around the Eastern Scheldt

Removal gates

In this alternative is chosen to remove the gates permanently, because of the absence of guidance of gates by the piers and the (difficult) connection with the bottom protection.

Removal sill beams

In this alternative all the sill beams will be removed.

Lowering sill construction

Removal of the sill beams means also a lowering of the stones in th sill construction, because the stone are positioned against the sill beams.

Strengthening sill construction

The removal of the sill beams would lead to a different stream pattern. A (to be determined) new top layer will be laid upon the remaining bottom protection to guarantee the stability.

Strengthening dikes around the Eastern Scheldt

Although the discharge function in this alternative improves, one have to accept the barrier loses its water retaining function and will only lower the water level due to friction. This means dikes around the Eastern Scheldt has to be strengthened.

A typical cross section is shown in Figure 5.5.

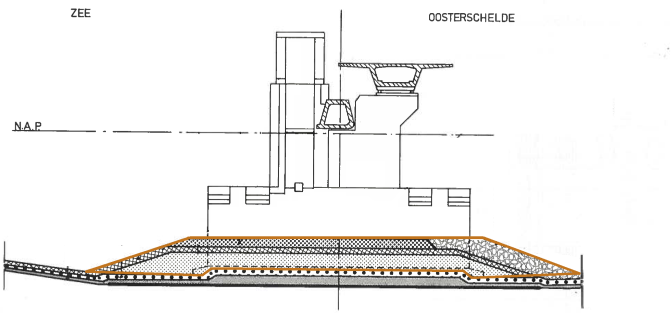


Figure 5.5: Cross section lowered sill alternative
The area, in which adjustment to the old construction are needed, is outlined with orange

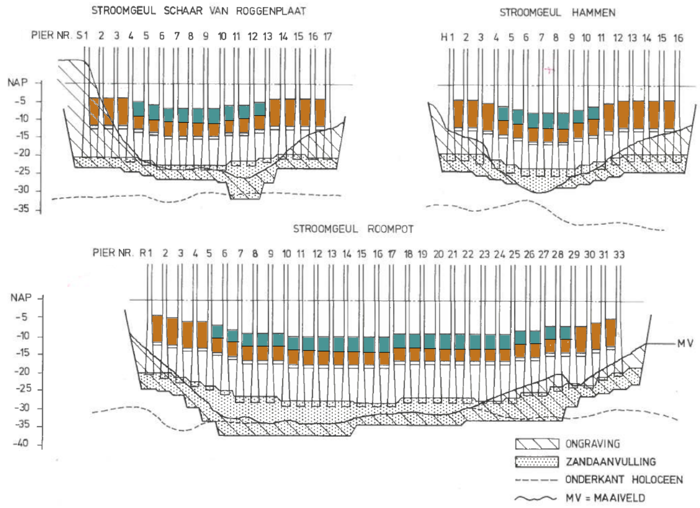


Figure 5.6: Frontview lowered sill alternative
All openings in the barrier needed to be adjusted. The two colors indicate the difference between the lowered sill beam and the lowered sill alternative

5.3. Impact alternative on the sand demand

To value the impact of the alternative on the sand demand the interaction between the main channel-tidal flat should be used. The process knowledge about the main channel-tidal flat interaction is still very limited. Qualitatively is know that the flow velocity is the governing force for the built up of tidal flats [Das, 2010], because an increase in flow velocity ensure more sand in suspension (the sand motor for the built up of tidal flats). The sand in suspension is transported unto the tidal flats by the vertical tide, the sediment settles and the built up begins. Mainly because the built of tidal flats depends on several variables it is difficult to identify the quantitative relation between the sand in suspension and the built up of tidal flats. Is the relation linear, quadratic or has it asymptotic characteristics? Due to this uncertainty, is in this MSc. Thesis a linear relation between the built up of tidal flats and the flow velocity is assumed. This relation is needed to value the different Eastern Scheldt storm surge barrier alternatives in Chapter 5.2. This is done by expressing a flow velocity increase in a percentage decrease of the sand suppletions (See paragraph 5.3.3. In Figure 5.7 is the formation of this relation is graphically shown.

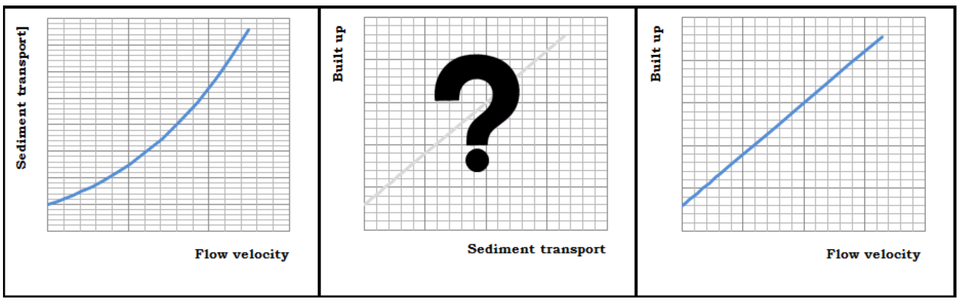


Figure 5.7: formation flow velocity - built up tidal flats

In the left figure the relation between the sediment transport s and the flow velocity is given. According to Formula 2.1 this relation is quadratic. This means that an increase in flow velocity leads to an exponential increase in the sediment transport. The figure in the middle shows the unknown relation between the sediment transport and the built up of the tidal flats. This is marked with a question mark. The figure on the right shows the assumed linear relation between the built up of tidal flats and the flow velocity.

5.3.1. Impact on the flow velocity

A linear relation between the velocity and the tidal volume is assumed to value the impact of a cross sectional increase (See Figure 5.8). Das concluded that the built up of tidal flats will start again when the barrier is removed. A 30 to 40% increase of flow velocity ensures the tidal flats will build up again. It must be noted that only one simulation with respect to increased flow velocities is performed by [Das, 2010], therefore it is possible that a smaller increase of the flow velocity also causes shoal build up or a higher velocity is needed for the built up of tidal flats. The flow velocity in the tidal basin is decreased from 1.5 m/s before the construction of the barrier to 1.0 m/s after the construction ([Huisman and Luijendijk, 2009]). The flow velocity in the tidal basin is expressed as an average per tide. The 1.0 m/s corresponds with a cross section of 17,900 m^2 (present situation) and the 1.5 m/s corresponds with a cross section of 80,000 m^2 (before situation). According to [Das, 2010] shoals build up will occur again when the barrier is removed. A 30 to 40% increase in the flow velocity means an increase to 1.3-1.4 m/s. In this MSc. Thesis the average of 1.35 m/s is taken as the velocity when built up of tidal flats will start again.

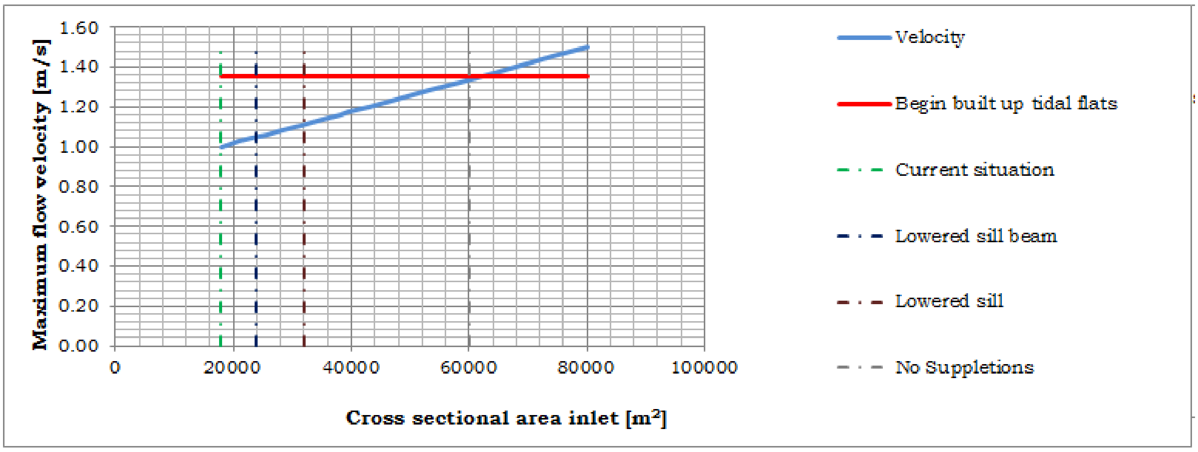


Figure 5.8: Relation maximum flow velocity - cross sectional area storm surge barrier

5.3.2. Impact on the vertical tide

To get an idea of the impact of certain measures on vertical tide Figure 5.1 is taken as reference. Figure 5.1 shows the relation between the effective cross sectional area and the reduction of the tidal difference as a percentage of the original tide at Yerseke (NAP + 3.50 m). The marked area in Figure 5.1 indicates the range in which alternatives are feasible (See paragraph 5.1.2). The left boundary (A) is the cross sectional area of the barrier in the current situation, effective cross sectional area of 17,900 m^2. The right boundary is considered reasonable in which measures still has significant improvement as result. The zero percent reduction (D), with an effective cross sectional area of 80,000 m^2, refers to the situation before the construction of the barrier.

5.3.3. Impact on the suppletion rate

To value the impact of a increase of velocity on the suppletion rate a linear relation is assumed. The current average flow velocity of 1.0 m/s is taken as the 100 % suppletion rate. 1.35 m/s is taken as the 0 % suppletion rate. This 1.35 m/s is the average between the 1.30 to 1.40 m/s where according to Das the built up of tidal flats will start. In the next paragraph the two alternative are assessed with respect to the baseline alternative. In Figure 5.9 the relation between the flow velocity and the amount of required suppletion is graphically shown. This means in the lowered sill beam alternative 85 % and in the lowered sill construction alternative 65 % of the required suppletions are still needed.

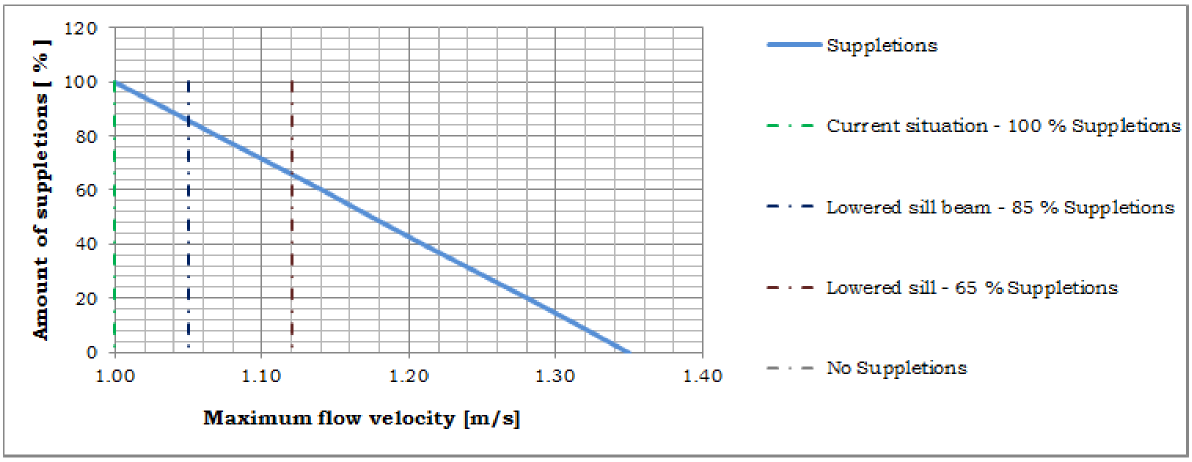


Figure 5.9: Relation flow velocity - percentage suppletions

5.3.4. Impact scour protection and scour holes

Beschrijf je wat er nu veranderd? niet volume, maar enkel harder stromen? water harder naar binnen: problemen voor vooroevers?

5.4. Assessment of the alternatives

The alternative will be evaluated on the basis of three criteria, namely: The impact on the sand demand, the societal impact and the costs. The results of the impact on the sand demand (Paragraph 5.3) are expressed in costs. The total cost is determined by performing a Life Cycle Cost Analyses (LCCA). The final assesment will be performed with a multi criteria analyses of the alternatives.

5.4.1. Impact sand demand

In Pagraph 5.3 the impact of a cross sectional increase on the sand demand is discussed. From Figure 5.9 can be concluded that the lowere sill alternative has the largest impact on the sand deman. For the 'lowered sill beam' alternative 15 % reduction of suppletions and for the 'lowered sill' alternative 35 % reduction of the suppletion can be reachedof the suppletions is still needed.

5.4.2. Societal impact

In the lowered sill beam alternative it is still possible to close the storm surge barrier. This is in the lowered sill alternative not the case. Permanently opening a barrier that is designed to be close could undermine the credibility of the Dutch water sector. The lowered sill beam alternative is in this perspective more favorable.

5.4.3. Costs

An important assesmentcriterium are the costs of the adjustment. These costs should outweight the costs in the baseline alternative (100 % suppletion). For all alternatives a life cycle cost analyses (LCCA) is calculated for a lifetime of 50 years. This LCCA gives a first insight into the cost differences between the alternative. The costs are determined using the Net Present Value (NPV). The Net Present value can be described by the following formula:

$$NPV = \sum_{i=1}^{T} \frac{C_y}{(1+r)^t} \tag{5.1}$$

With:

t = time $[year]$
T= time horizon project $[year]$
r = discount rate $[-]$
C_Y = costs per year [€]

In this MSc. Thesis a interest rate of 2.5 % is used. In the next paragraph the input values for the LCCA are discussed. The paragraph ends with a summary of the calculation results. The full LCCA calculation is included in Appendix A.

Construction costs Eastern Scheldt storm surge barrier

In [Dircke *et al.*, 2010] the construction costs of the barrier where approximated on 3.85 billion euros (price level: 2010). With an average interest rate of 2 %, the construction costs of the Eastern Scheldt storm surge barrier are approximately 4.1 billion euros in 2014. Based on i.a. quantities and the construction method, an estimate for the cost contribution per part of the barrier is made. In Table 5.2 this cost distribution is shown.

Table 5.2: Cost contribution of the different barrier parts to the total cost of the storm surge barrier

Part	Contribution
Piers	30 %
Gates	20 %
Bottom protection	20 %
Sill beam	10 %
Sill construction	5 %
Upper beam	5 %
Traffic girders	5 %
Other parts	5 %
Total	**100 %**

Maintenance cost Eastern Scheldt storm surge barrier

From [Leeuwdrent, 2012] three values for the maintenance cost are extracted. Leeuwdrent dug up value which differ from 25 M€/year [RWS10, 2008], 10-18 M€/year [1] to 17 M€/year according to the website of Neeltje Jans. The major part of these cost are for: extensive maintenance of the cilinders, maintenance of the piers, maintenance traffic girders and conservation of the gates. In this MSc. Thesis maintenance costs of 18 M€/year are used. For the LCCA is assumed that after replacing the gates by FRP gates the maintenance cost will decrease by 50 %. In the lowered sill alternative the maintenance cost are assumed to be 1/3 of the original cost (due to the absence of te gates).

Adjustment Eastern Scheldt storm surge barrier

For both alternatives adjustments on the barrier have to be done. To get a first insight into the feasibility per alternative the demolishing cost are approximated on 30 % of the construction costs [2].

In the baseline alternative, the original 62 gates will be replacement after approximately 50 years from the opening of the barrier in 1986. This means the gates will be replaced in 2036. It is presumed that the gates will be replaced by less maintenance intensive FRP gates. In [van Straten, 2013], where a feasibility study on FRP-slides in the Eastern Scheldt storm surge barrier is performed, a value of 225 M€ for the gate replacement is found. In the lowered sill beam alternative a large part of the gates (40 of the 62) will be replaced earlier because of the need for enlarged gates. Assuming a average gate height of 10 m in the current situation and a average gate height of 14 m with the enlarged gates, the costs for the 40 enlarged gates are estimated on $\frac{225}{62} * \frac{14}{10} * 40 = 203$ M€.

Sand suppletions Eastern Scheldt

In [RWS7, 2013] one of the scenarios includes a 100 % preservation of the tidal flats. This scenario describes one suppletion until 2025 and from 2025 to 2060 one suppletion every five years. Total costs of the suppletion of 65 million m³ sand are estimated on 422 million euros. This 100 % preservation scenario is part of the baseline alternative. The amount of suppletion needed in the other alternatives

[1] http://www.rijkswaterstaat.nl/images/Factsheet%20sluiting%20Oosterscheldekering%20bij%20stormvloed_tcm174-356433.pdf

[2] Interview: A. van der Toorn

(lowered sill beam and lowered sill) are expressed in a percentage of the 100 % preservation scenario. In Figure 5.9 the relation between the maximum flow velocity and the amount of suppletions is graphically shown. This means for the 'lowered sill beam' alternative 85 % of the suppletions is still needed and for the 'lowered sill' alternative 65 % of the suppletions is still needed.

Dike heightening

The lowered sill alternative includes a heightening of the dikes around the Eastern Scheldt. In [Leeuwdrent, 2012] is concluded that when the barrier will be removed, a dike heightening of 0.7 m is needed to satisfy the current safety standard. Furthermore Leeuwdrent concluded that when a sea level rise of 0.5 m will appear the dikes have to be heightened with 1.4 m. A sea level rise of 1.0 m will require a dike heightening of 2.2 m. Based on Chapter 3.3 a sea level rise of 0.5 m is account for (consistant with a heightening of the dikes with 1.4 m). Because in the lowered sill alternative not the whole barrier will be removed, it seems reasonable to downscale the rewuired heightening calculated by Leeuwdrent. Based on [de Boom, 2013] a rough estimate of 0.5 m water level decrease, causes by the friction of the Eastern Scheldt storm surge barrier, is included. This means in lowered sill alternative a total heightening of the dikes with 0.9 m is accounted for.

In [Leeuwdrent, 2012] is also the average costs per kilometer around the Easter Scheldt calculated. The average costs for 1.0 meter dike heightening of the dike around the Eastern Scheldt are calculated at 4,10 M€/km. This value is used in the LCCA. De investment cost of the dike heightening will be divided over 20 years.

Calculation results LCCA

In Table 5.3, 5.4 and 5.5 the calculation results of the LCCA are summarized. In Appendix A the full calculation is included.

Table 5.3: LCCA Baseline alternative

Baseline alternative	**NPV [M€]**
Maintenance	385
Suppletions	300
Cost replacement gates	152
Total	**836**

Table 5.4: LCCA Lowered sill beam alternative

Lowered sill beam alternative	**NPV [M€]**
Maintenance	264
Suppletions	255
Cost replacement gates	257
Lowering sill beam	92
Other adjustments	50
Totaal	**918**

5.5. Conclusion assessment alternatives

In this chapter two alternatives for an improved functionality of the Eastern Scheldt storm surge barrier are assesed for costs, societal impact and impact on the sand demand. On the basis of these criteria the different alternatives are weighted in a multi criteria analyses (see Table 5.6)

Although the baseline alternative is cheaper then the other alternatives and the impact assessment in [RWS7, 2013] shows the suppletion measures manage to fight the negative effects of the sand demand on the environment (100 % preservation of the tidal flats), the tidal flats are, after suppletion,

Table 5.5: LCCA Lowered sill alternative

Lowered sill alternative	NPV [M€]
Maintenance	176
Suppletions	225
Remove sill beam	123
Removal part of sill construction	62
Adjustments sill construction	80
Dike heightening	459
Totaal	**1124**

Table 5.6: Multi criteria analyses (MCA) alternatives

	Baseline	Lowered sill beam	Lowered sill
Impact sand demand	0	+	+
Societal impact	0	-	- -
Costs	0	-	- -
Total	**0**	**-**	**- - -**

still temporary unsuitable for wildlife. The lowered sill beam alternative seems a feasible alternative to fight the negative effects of the sand demand and seems better for the environment.

The following chapter are show how the, for this alternative, needed adjustment of the Eastern Scheldt are technically performed and how the execution is done. A more detailed cost analyses should show whether the assumed costs for the adjustments fit the real costs

6

Design of the lowered sill beam alternative

In this chapter the design of the lowered sill beam is elaborated (subresearch question 4). The full design holds many aspect which are itemized below

- **Strength of the lowered sill beam**
- **Stability of the lowered sill beam**
- **Overall stability barrier**
- Enlarged gate design
- Design stones lowered sill construction

In this chapter the bold items are treated in detail. Strength and stability calculation of the lowered sill beam are performed and the total stability of the barrier is checked. Other adjustment to the barrier, like the gates, bottom protection and the sill construction are elaborated in outline in Paragraph 6.6.

6.1. Lowered sill beam alternative

In Chapter 5 the different measures concerning the lowered sill beam alternative are threated. The measures in this alternative include the following measures:

1. Lowering of 50 sill beams
2. Extension of 50 gates
3. Strengthening of the sill construction

Lowering sill beams

In the "lowered sill beam" alternative 50 sill beams will be lowered by 4 m over an average length of 32 m. This is to ensure a better connection between the gate and the sill and to reduce the leakage during storm. From a safety point of view is chosen to not lower the outer sill beams to reduce the possible negative effects on the shores of Schouwen and Noord-Beveland.

Extension of gates

The gate extension can be realized by means of a new gate or an adjustment to the current gates. With the extended gates the lifting capacity should not be exceeded. Furthermore the gates at the location of the lowered sill beams are being enlarged to ensure a closed barrier during high water.

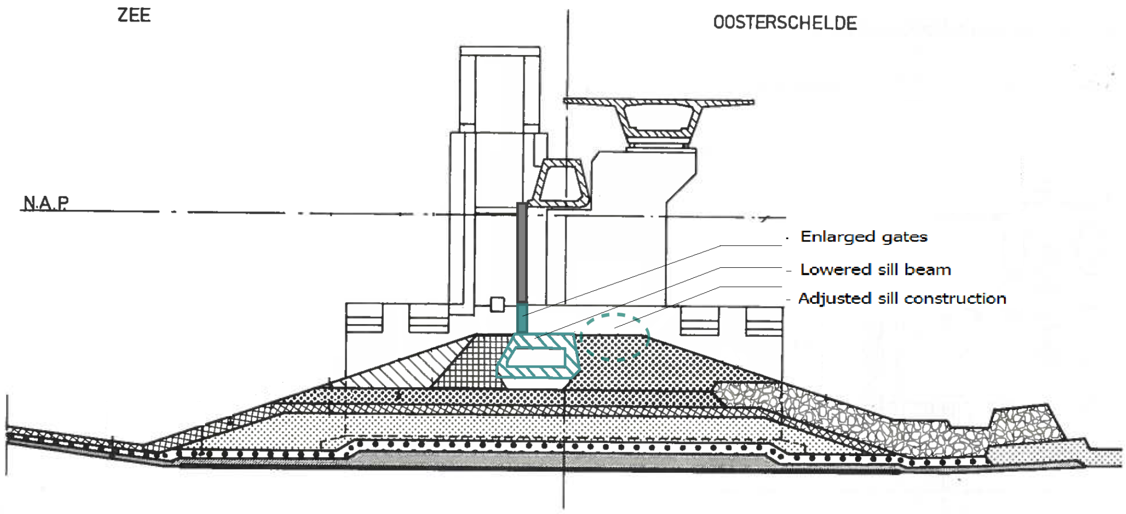


Figure 6.1: Cross section lowered sill beam alternative

Strengthening sill construction

Lowering of the sill beam could lead to a different stream pattern. Possible strengthening of the stones in the sill construction should guarantee the stability. The bottom protection around the lowered sill beams should be adjusted: a part of the 1-3 tons stones should be removed. In Figure 6.1 the alternative including the different interventions are shown.

6.2. Information, requirements and boundary conditions

6.2.1. Geometry

First the location of prestressed tendons are identified by analyzing the documents made available by Rijkswaterstaat. In Figure 6.2 - 6.6, the positions of the prestressed tendons are included.

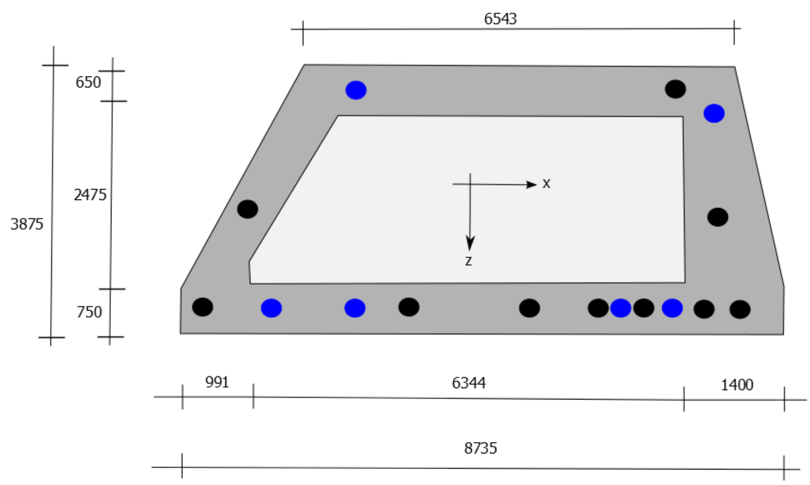


Figure 6.2: Cross-section sill beam (middle) – Positions prestressed tendons

The chosen numbering of the prestressing strands corresponds with the numbers used in the current design. One sill beam contains 21 tendons in total. 5 tendons are used for lifting the original sill beam. The other 16 cables are used for strength purposes. The 16 cables can be divided into:

- 10 cables, 19 * 15.7 mm, FeP1770 (in black)
- 6 cables, 12 * 15.7 mm, FeP1770 (in blue)

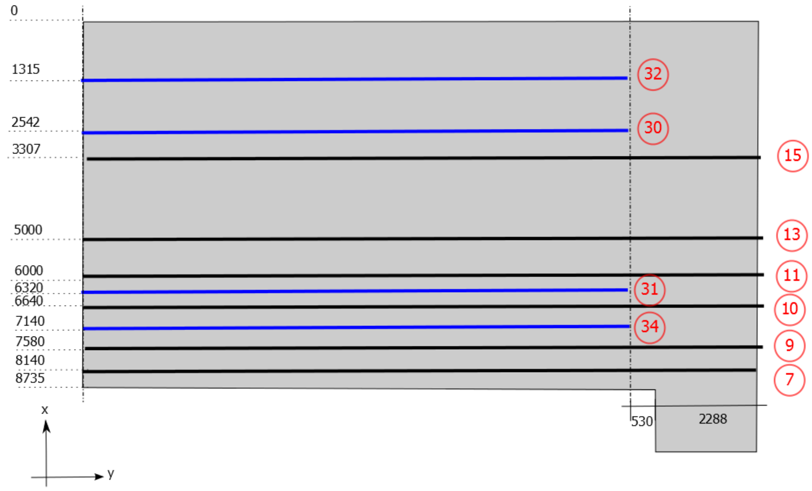


Figure 6.3: Cross-section – bottom position prestressing

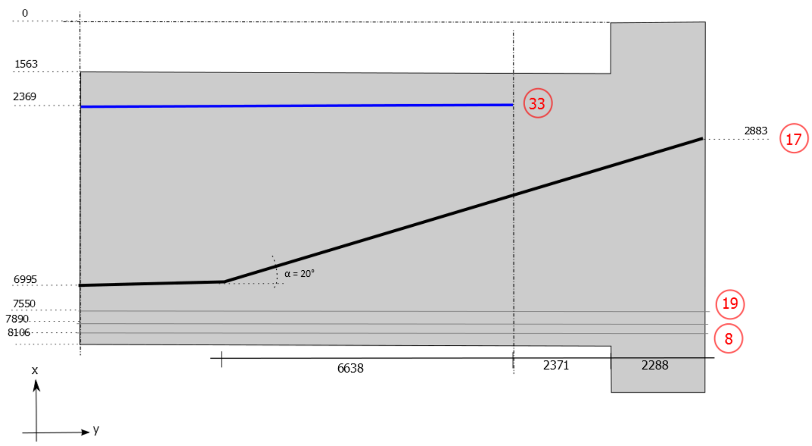


Figure 6.4: Cross-section – intermediate floor

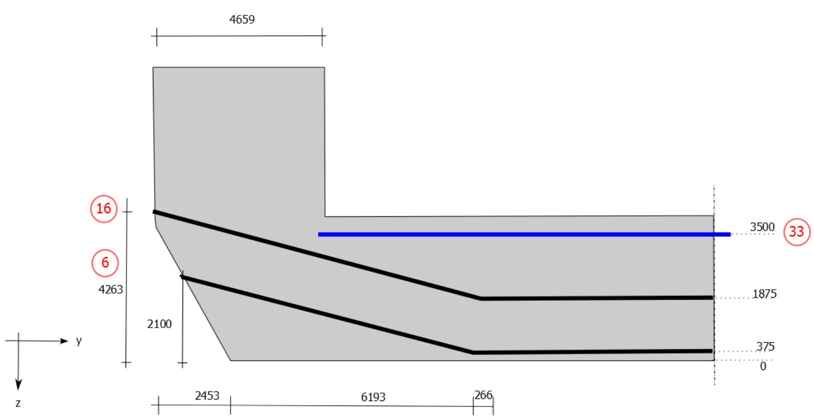


Figure 6.5: Cross-section – wall North Sea side

The 6 prestressing cables coloure blue serve in the orginal design to withstand shrinkage stresses in de bottom- and topslab of the sill beam. In the verification of the original design of the lowered sill beams these capacity of these cables are included for 50 %.

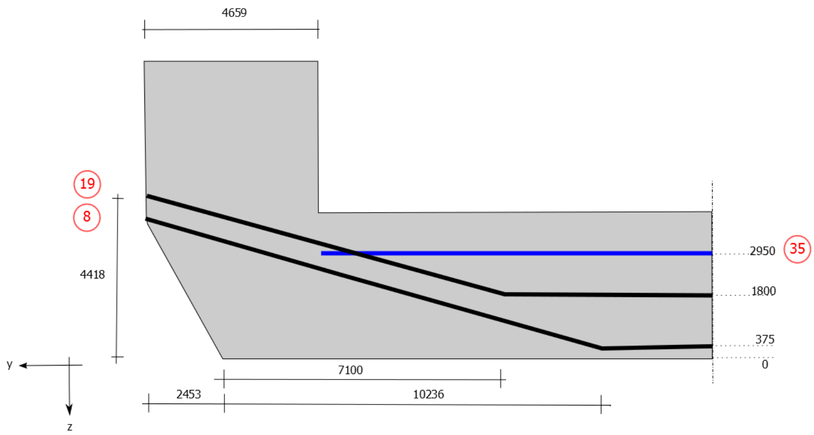


Figure 6.6: Cross-section – wall Eastern Scheldt side

6.2.2. Material properties

Prestressing steel

$FeP1770, E_p = 19500N/mm^2 \sigma_{pm,0} = 1328N/mm^2$

Concrete

$C30/37, f_{ck;cube} = 37N/mm^2, f_{ctm} = 2.9N/mm^2, E_{cm;(0)} = 32837N/mm^2$.

Partial factors material properties

Concrete: γ_c Reinforcement steel: $\gamma_s = 1.15$ Prestressing steel: $\gamma_p = 1.1$

6.2.3. Software

Excel 2010
Technosoft - MN-Kappa V5

6.3. Starting points

6.3.1. Consequence class, design period & load factors

The Eastern Scheldt storm surge barrier is a primary sea defense and thus classified as a consequence class 3 (CC3) construction (according to NEN-EN 1990). The design lifetime of the adjustments are 50 years.

Loadfactors

In the current design of the sill beams a semi-probabilistic method is used in which the following safety factors are used:

$Q_u = 1.4 \times Q_e$
Of $Q_e/1.2 = Q_k$
Of $Q_k \times 1.7 = Q_k$

With:
Q_k = characteristic load
Q_e = extreme load
Q_u = failure load.

The factors are based on an extreme loadcase with a frequency of exceedance of $2.5\text{x}10^{-4}$ per year.

In this MSc. Thesis the loadfactors according to the current standards are used. For the structural safety check (STR) the following load factors are applied:

$\gamma_G = 1.3$ Loadfactor for dead weight
$\gamma_Q = 1.65$ Loadfactor for variable load
$\gamma_p = 1.1$ Loadfactor for prestressing forces
For the uplifting check (UPL) the following partial loadfactor (γ_F) according NEN-EN 1990 are used:
$\gamma_{G;dst} = 1.0$ Loadfactor dead weight and permanent load (unfavourable)
$\gamma_{G;dst} = 0.9$ Loadfactor dead weight and permanent load (favourable)
$\gamma_{Q;dst} = 1.5$ Loadfactor variable load

In the calculation of the uplifting waterpressure is however a waterlevel 1/4000 per year been used. This already contains a certain safety level. Because of that the partial safety factor for the variable load is been reduced. The used values are displayed below:

$\gamma_{G;dst} = 0.9$
$\gamma_{Q;dst} = 1.1$

6.4. Load cases and Load combinations

6.4.1. Load cases

In this paragraph the different loads on the sill beam are calculated. All the loads are divided in load cases (LC).

Self weight concrete (LC01)

For the self-weight of concrete a density of ρ_c = 25 kN/m^3 is used. In Appendix B follows cross-sectional area of A_c = 16.2 m^2. This results in a characteristic value of the self weight of $q_{k;self}$ = 405/239 kN/m (above/below water).

Ballast sill beam (LC02)

The ballast of the sill beam consist of sand with a density of ρ_s = 18.5 kN/m^3. In [RWS4, 1985] a degree of filling of 82% for the bottom compartment is used. With this degree of filling de load caused by ballast is $q_{ballast}$ = 212 kN/m.

Vertical load stones sill construction (sill loads) (LC03)

The sill material against the sill beams provides a vertical load on the beam. For the density ρ_{rock} = 26.5 kN/m^3 is held. For the load q_{sill} = 100 kN/m according [RWS4, 1985] is used. This is a conservative approach, because in the lowered sill beam alternative a part of the sill construction is removed.

Hydraulic load (LC04)

The hydraulic load on the sill beam exist of a static and a dynamic part. The static part of the hydraulic load exist of a waterpressure difference over de barrier. The dynamic part is determined by the waveload. Furthermore three different hydraulic loadcombinations are distinguished :

(a) Closed barrier: In this loadcase the gates are completely closed. The beams are loaded with a maximum hydraulic head difference in combination with a waveload.

(b) Rejecting gate: In this loadcase a gate is partly or not closed and water flow through the opening. The beams are loaded with a reduced hydraulic head difference n combination with a waveload.

(c) Reversed slope: In this situation the gates of the storm surge barrier are closed and the water level in the Eastern Scheldt is higher than the water level at the North Sea side.

In the original design the governing hydraulic loads are determined based on a governing combination of hydraulic- and wave load 1/4000 per year. In Appendix B the static pressures and the waveloads according Goda-Takahashi are determined. In Table 6.1 and Table 6.2 the result are summarized. With the calculated values from Table 6.1 and Table 6.2 the resultant of the hydraulic load is calculated. The vertical balance check (See Chapter 6.5.2) the hydrualic load will be summed up. For the strength verification a reduced hydraulic load will be included. This reduction of the hydraulic represents the reduction of the hydraulic pressure through the sill construction. In Figure 6.7 the shape of the total load and the reduction of the hydrualic pressure is schematized.

Table 6.1: Hydraulic loads (static)

	Level	$\mathbf{R}_H$	$\mathbf{R}_v$
	[m t.o.v. NAP]	[kN/m^1]	[kN/m^1]
Closed gates	5.5/-0.7	192	-479
Rejecting gates	3.5/0.6		
Closing gates at storm surge	3.5/0.6		

Table 6.2: Hydraulic waveload

	Level	$\mathbf{R}_H$	$\mathbf{R}_v$
	[m w.r.t. NAP]	[kN/m^1]	[kN/m^1]
Closed gate	5.5/-0.7	86	-70
Rejecting gate	3.5/0.6		
Closing gate storm surge	3.5/0.6		

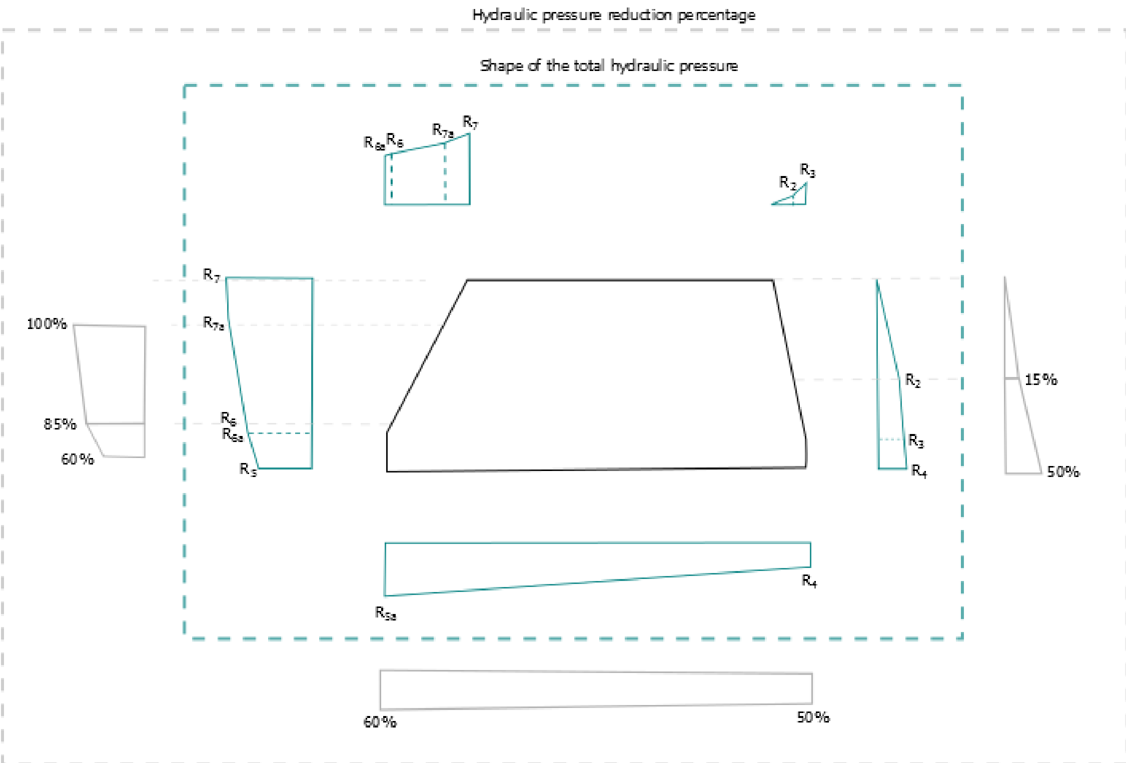


Figure 6.7: Totale hydraulic load sill beam

Prestressing loads (LC06)

5 prestressing cables have a kinked shape (See Figure ...). Four cables in z-direction and 1 cable in x-direction. The position of the tendons are determined from old design drawing. In Table ...the load as a consequence of the kinked shape are shown.

Conclusion load cases
In Figure ...the loads on the sill beam are schematized.

Summary load cases
Permanent load
$q_{k;self} = 239kN/m^1$
$F_{k;self} = 200kN/m^1$
$q_{k;ball} = 96kN/m^1$
$q_{k;dr;min} = 10kN/m^1$
$q_{k;dr;max} = 100kN/m^1$
Variable load
$q_{k;hydr;z;max} = -268kN/m^1$
$q_{k;hydr;z;min} = -252kN/m^1$
$q_{k;hydr;z;max} = 211kN/m^1$
$F_{k;hydr;z;min} = 192kN/m^1$

6.4.2. Load combinations
For the verification of the lowered sill beam design the following loadcase are checked:

LCC01: Closed barrier
This load combination contains the situation where the Eastern Scheldt storm surge barrier is completely closed, the water level at the North Sea side (NSS) reaches its 1/4000 per year storm surge level and the water level at the Eastern Scheldt side has reach a certain minimum. The load cases in this load combination are shown below:

$$LC01 + LC02 + LC03 + LC04a \tag{6.1}$$

LCC02: Rejecting gate
This load combination contains the situation where the Eastern Scheldt storm surge barrier is closed but one gates is rejecting to close or is partly closed. The load cases in this load combination are shown below:

$$LC01 + LC02 + LC03 + LC04b \tag{6.2}$$

LCC03: Reversed slope
This load combination contains the situation where the gates of the storm surge barrier are closed and the water level in the Eastern Scheldt is higher than the water level at the North Sea side. The load cases in this load combination are shown below:

$$LC01 + LC02 + LC03 + LC04a \tag{6.3}$$

6.4.3. Internal forces
In the first design loop a simplified method with a beam on two supports with uniform load will be used. The moment are determined by $1/8 * q * l^2$.

6.5. Adjustments sill beam
For the lowered sill beam alternative a part of the prestressed sill beams have to be removed. In Figure 6.8 this part is marked with red. In total 50 of the 62 sill beams will be adjusted. The 50 sill beam are not all at the same depth. This can cause a difference between the loads on the sill beams. For the technical feasibility check every load combination will be checked with the governing sill beam. In Paragraph 6.4.1 the loadcases are calculated.

6.5.1. Check main dimensions sill beam
The prestressed sill beam must be checked for the SLS requirements concerning maximum initial concrete compressive stress (EN 1992-1-1 cl. 5.10.2.2) and concrete tensile stress or crack width (EN

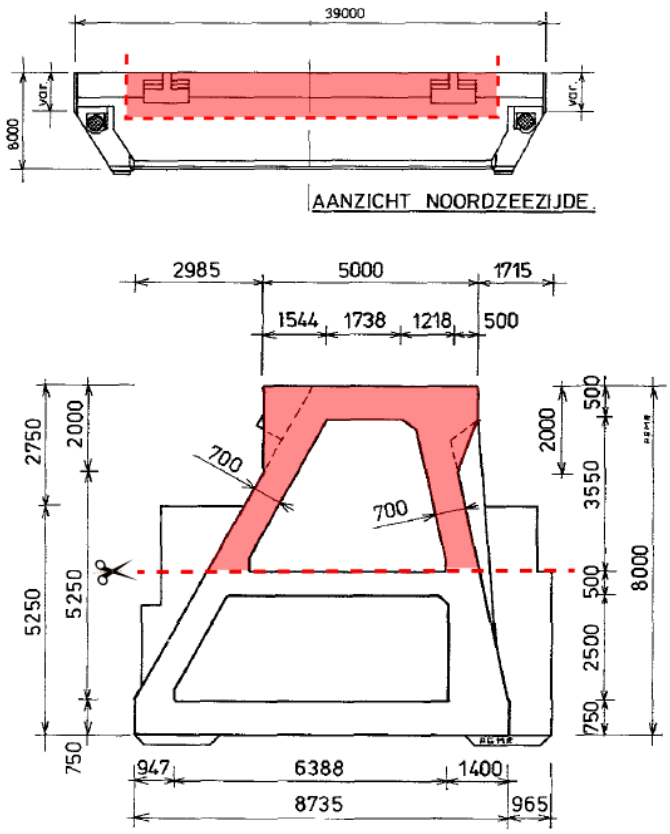


Figure 6.8: Cross section lowered sill beam alternative

1992-1-1 cl. 7.3) are met and that structural resistance meets the ULS requirements. In this analyses "fully prestressed concrete" is assumed. This means that no tensile stress should occur just after prestressing (t=0) and after some time (t=∞). The cross section at midspan is showed in Figure 6.8.

Design check SLS

In this paragraph the stresses in cross section B-B (midspan) are checked. In the analyses two situation are checked, namely: just after prestressing at t=0 and after some time at t=∞. The following requirements for the stresses should be met:

1. Allowable compressive strength:

$$\sigma_{cc} \leq 0.6 f_{ck} \tag{6.4}$$

2. No tensile stresses:

$$\sigma_{ct} \leq 0 \tag{6.5}$$

The governing situation for the extreme compressive and tensile stress occurs are summarized below for the t=0 and t=∞ case.

t=0

$$\sigma_{cc} = \frac{P_{m;0}}{A_c} + \sigma_G + \sigma_Q + \sigma_p \tag{6.6}$$

$$\sigma_{ct} = \frac{P_{m;0}}{A_c} + \sigma_G + \sigma_p \tag{6.7}$$

t=∞

$$\sigma_{cc} = \frac{P_{m;\infty}}{A_c} + \sigma_G + \sigma_Q + \sigma_p \tag{6.8}$$

$$\sigma_{ct} = \frac{P_{m;infty}}{A_c} + \sigma_G + \sigma_Q + \sigma_p \tag{6.9}$$

where:

σ_{ct} = Maximum compressive stress outer fiber
σ_{ct} = Maximum tensile stress outer fiber
A_c = Cross sectional area sill beam
P_m = Prestressing force
σ_G = Stress in the outer fiber due to the permanent load
σ_Q = Stress in the outer fiber due to the variable load
σ_p = Stress in the outer fiber due to the prestressing load

Check of stresses at t=0:

The stresses of the different loadgroups (permanent loads σ_G, variable loads σ_Q and bending stresses from prestressing σ_p) are determined by considering the double bending per loadgroup. De stresses are determined at the edges of the sill beam cross section. The theory behind this calculation is elaborated in Appendix B. The results for the stresses at t=0 are summarized in Table 6.3.

Table 6.3: Stresses in cross section B-B at t=0

Position	σ_G	σ_Q	σ_p	$\frac{P_m}{A_c}$
A	-1.13	4.13	-0.75	-2.32
B	-0.65	-2.68	1.43	-2.32
C	1.43	-5.62	1.09	-2.32
D	0.80	3.50	-1.82	-2.32

Point C is governing for the check of requirement 6.4. By filling the the values in Table 6.3 Equation 6.6 becomes:

$\sigma_{cc} = -2.32 + 1.43 - 5.62 + 1.09 = -5.42 N/mm^2 < 22.5 N/mm^2$

This means requirement 6.4 is met. Point C is governing for the check of requirement 6.5. By filling the values in Table 6.3 Equation 6.7 becomes:

$\sigma_{ct} = -2.32 + 1.43 + 1.09 = 0.20 N/mm^2 > 0$

This means requirement 6.5 is not met but the maximum value of the tensile tress is still smaller than the average axial tensile stress of the concrete $f_{ctm} = 2.90 N/mm^2$.

Check of stresses at t=∞:

For the calculations of the stresses at t=∞ the working prestressing force $P_{m;\infty}$ should know. Therefore first the prestressing losses are calculated below.
$h_0 = \frac{2*A_c}{2*h_{tot}+2*b_{deck}} = \frac{2*16.2*10^6}{2*3875+2*6543} = 1555mm$
k_h = 0.7 (according NEN-EN1992-1-1, Table 3.3)

Shrinkage

$\varepsilon_{cd,0}$ is determined by interpolation according NEN-EN1992-1-1, Table 3.2)
$\varepsilon_{cd0} = \frac{0.30-0.06*(50-f_{ck;cube})}{(50-25))} * 10^{-3} = 0.00027\ (0.27‰)$
$\varepsilon_{cd\infty} = k_h * \varepsilon_{cd0} = 1.89‰$

$\varepsilon_{ca\infty} = 2.5 * (f_{ck} - 10) * 10^{-6} = 2.5 * (30 - 10) * 10^{-6} = 0.56‰$
$\varepsilon_{cs} = \varepsilon_{cd\infty} + \varepsilon_{ca\infty} = 0.189 + 0.56 = 0.245‰$

Creep

$\varphi_{28} = 1.5$
$\varphi_{100} = 1.2$
$E_{c28} = \frac{E_{cm}*1.05}{1+\varphi_{28}} = \frac{34660*1.05}{1+1.5} = 14557N/mm^2$
$E_{c100} = \frac{E_cm*1.05}{1+\varphi_{100}} = \frac{34660*1.05}{1+1.2} = 16542N/mm^2$

$E_{c100Trost} = \frac{(E_{cm}*1.05)}{1+\varphi_{100}*0.8} = \frac{344660*1.05)}{1+1.2*0.8} = 18568N/mm^2$

$\varepsilon_c = \frac{\sigma_{0,self}}{E_{c28}} + \frac{\sigma_{0,cG}}{E_{c100}} + \frac{\sigma_{0,cQ}}{E_{c100Trost}}$

In this case only the load increment at t=0 is accounted for:
$\varepsilon_c = \frac{\sigma_{0,self}}{E_{c28}} = \frac{3.35}{14557} = 0.23‰$

Relaxations

$\mu_{relax} = \frac{\sigma_{p;0}}{f_{pk}} = \frac{\sigma_{1296}}{1770} = 0.73$
$\rho_{1000} = 2.5\%$
$t_{relax} = 500000h$
$\Delta\sigma_{pr} = 0.66 * \rho_{1000} * \exp(9.1 * \mu_{relax}) * (1/1000) * t_{relax}^{(0.75*(1-\mu_{relax})} * 10^{-5} * \sigma_{pi} = 58.3N/mm^2$

Total loss
$\sigma_{lossmax} = (\varepsilon_c + \varepsilon_{cs}) * E_p + \sigma_{pr} = ((0.245 + 0.23) * 10^{-3}) * 195000 + 58.3 = 212.4N/mm^2$
$p_{lossmax} = \sigma_{lossmax} * A_p = 6169kN$
$p_{m;\infty} = p_{m;0} - p_{lossmax} = 37650 - 6169 = 31481kN$

The loss percentage is 6169/37650 = 16 %
The results for the stresses at t=∞are summarized in Table 6.4.

Table 6.4: Stresses in cross section B-B at t=∞

Position	σ_G	σ_Q	σ_p	$\frac{P_m}{A_c}$
A	-1.13	4.13	-0.63	-1.95
B	-0.65	-2.68	1.20	-1.95
C	1.43	-5.62	0.92	-1.95
D	0.80	3.50	-1.53	-1.95

Point C is governing for the check of requirement 6.4. By filling the the values in Table **??** Equation 6.8 becomes:

$\sigma_{cc} = -1.95 + 1.43 - 5.62 + 0.92 = -5.22N/mm^2 < 22.5N/mm^2$

This means requirement 6.4 is met. Point D is governing for the check of requirement 6.5. By filling the values in Table 6.4 Equation 6.9 becomes:

$\sigma_{ct} = -1.95 + 0.8 + 3.5 - 1.53 = 0.81N/mm^2 > 0$

This means requirement 6.5 is not met but the maximum value of the tensile tress is still smaller than the average axial tensile srength of the concrete $f_{ctm} = 2.90N/mm^2$.

Design check ULS
With regard to ULS, one of the resistances to check is the bending moment resistance of the sill beams.

Furthermore the rotational capacity should be sufficient to prevent brittle failure. The height of the compression zone, x_u, should be limited. The following condition should be met:

$$\frac{x_u}{d} \leq \frac{500}{500+f} \tag{6.10}$$

where:

x_u = Height compression zone
d = Effective depth cross section
f = $\left(\frac{f_{pk}}{\gamma_s} - \sigma_{pm,\infty}\right)$

In Appendix B the full calculation of the moment capacity is added. A summary of the results is shown below. $M_R d = 141375.7$ kNm > M_{Ed} = 68907.5 kNm

From that calculation can be concluded that the requirement with respect to the moment capacity are met.

6.5.2. Stability lowered sill beam

In Chapter 6.4.1 it has been found that the self-weight and the weight of the ballast is decreased, the horizontal hydraulic load is decreased and the vertical hydraulic load remained the same. In order to check the vertical balance the sills at the deepest part of the Eastern Scheldt are considered. In Appendix B dots the vertical balance of the sill beam are checked according to Equation ref eq: vertical equilibrium. It follows that the vertical stability of the construction is ensured.

$$\gamma_{Q;dst} \times V_{up} \leq \gamma_{G;dst} \times V_{down} \tag{6.11}$$

With:
V_{up} Upward hydraulic load [kN]
V_{down} downward hydraulic load [kN]

6.5.3. Overall stability barrier

In the current situation, the design of the sill beam is arranged in such a way that the upper part of the beam serve as a stop for the gates. In the lowered beam alternative the stops are not present (or in limited extend available). Due to the change of the stops, the position of the hydraulic load force shifts up. In that extend, in this paragraph the overall stability of the barrier is checked. To value the impact of the lowered sillbeam first the current loads on the piers are being extracted from [RWS3, 1985]. The changed loads are then being compared.

Loads

In Table 6.5 the current loads on the piers are shown.

Table 6.5: Loads piers

		V [kN]		H [kN]	M [kNm]	
V_1	Self weight	107000	43%			
V_2	Ballast	52200	21%			
V_3	Sill beam	14800	6%			
V_4	Gates	3935	2%			
V_5	Top beam	14384	6%	10596	5696	0.3%
V_6	Box girder for traffic	23180	9%		14360	1%
V_7	Loads sill construction	32230	13%			
H_8	Hydraulic load			46445	1681292	99%
	Totaal	**247729**		**57041**	**1701348**	

In Figure 6.9 forces concerning the overall stability of the barrier are displayed.

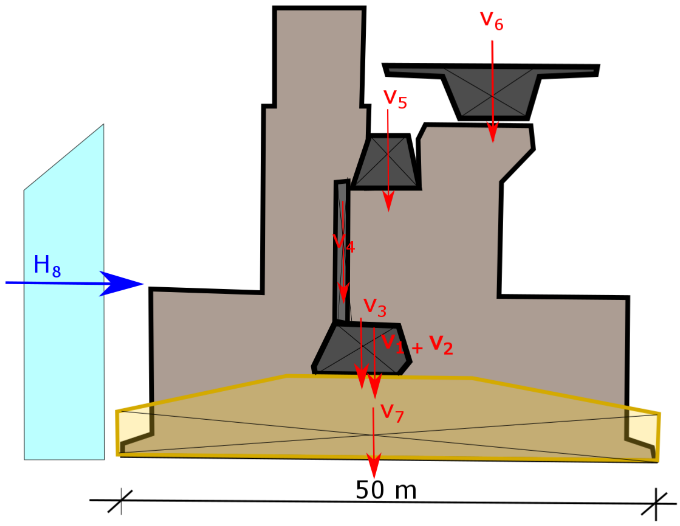


Figure 6.9: schematization forces stability

Horizontal stability

In order to guarantee horizontal stability the friction force of the subsoil should withstand the resulting total horizontal force. This friction force is determined by the total of the forces acting on the structure in vertical direction, multiplied by a dimensionless friction coefficient f.

The equation for the horizontal capacity check is:

$$\frac{\Sigma H}{\Sigma V} \leq f = tan(2/3 * \varphi) \tag{6.12}$$

The buoyant forces are included in ΣV. For the angle of friction a (conservative) value of φ = 30 is assumed in checking the horizontal stability in the sliding plane between the foundation and the subsoil.

The values in Table 6.5 are used for the calculation. Hereby is the sum of the vertical force reduced by the weight of the sill beam and multiplied by 0.9. The requirements for horizontal stability are fulfilled because:

$\frac{\Sigma H}{\Sigma V} = \frac{57041}{0.9*(247729-14800)} = 0.27 < tan(2/3 * 30) = 0.45$

Rotational stability

The rotational stability is checked. Hereby the foundation length L_f is checked. The bases of the calculation holds that soil not have to exert to tensile stresses. Especially the adhesive and cohesive properties of sand are very poor so tensile stress cannot be provided by the subsoil. This is the case if the resulting action force intersects with the core of the structures (defined as L/6). The equation for the rotational capacity check is:

$$e_R = \frac{\Sigma M}{\Sigma V} \leq L/6 \tag{6.13}$$

The values in Table 6.5 are used for the calculation. Hereby is the sum of the vertical force reduced by the weight of the sill beam and multiplied by 0.9. Equation 6.13 becomes:

$e_R = \frac{1701348}{0.9*(247729-14800)} = 8.1 < L_f/6 = 50/6 = 8.3$

So even without taken the weight of the sill beam into account requirement 6.13 concerning the rotational stability is still met.

Vertical stability

The vertical stability check is required because the soil should resist the stress due to the active loads ($\sigma_{k.max}$).

The equation for the vertical capacity check is:

$$\sigma_{k.max} = \frac{F}{A} + \frac{M}{W} = \frac{\Sigma V}{b \cdot l} + \frac{\Sigma M}{1/6 \cdot l \cdot b^2} < p'_{max} \tag{6.14}$$

Since the sum of the vertical forces is decreased, no exceedance of $\sigma_{k.max}$ is expected.

6.6. Design remarks

In this chapter some design issues are been underexposed. This paragraph places per underexposed design issue some remarks.

6.6.1. Bottom protection removal

In the Lowered sill beam alternative a part of the stones of the sill construction must be removed to create work space. After the lowering of the sill beams the sill construction (consisting of place stones) is restore. Because of the lowering of the sill beam and the adjustments to the sill construction the flow pattern through the barrier changes. Calculations must show whether the stones top layer of the sill constructions are still stable. Where necessary, larger stones should be used or the existents stones must be penetrated with colloidal under water concrete. Also the effect on the edges of the scour protection should be examined. In Paragraph 3.2 the problems around these edges are described. The lowering of the sill construction should not further deteriorate the problems with the scour holes. Furthermore safety of the primary sea defense at the coast of Noord-Beveland should not be compromised.

6.6.2. Stop of the gates

In the current situation, the design of the sill beam is arranged in such a way that the upper part of the beam serve as a stop for the gates. In the lowered beam alternative the stops are not present (or in limited extend available). Due to the change of the stops, the position of the hydraulic load force shifts up. In that extend the overall stability of the structure is checked in paragraph 6.5.3, but there has to be determined whether these stops are still necessary for other reasons. For example because the leakage through the gates become too much.

6.6.3. Adjustment to the gates

In the lowered sill beam alternative the current gates in the Eastern Scheldt storm surge barrier will be replaced by higher gates. In the design of the alternative there is assumed that the gates will be replaced by gates with a lightweight material (e.g. FRP). In that case the weight of the gates should remain the same. If the weight of the extended slide are still higher than the weight of the existing gates, one should examine if the lifting capacity if the current gate equipment is sufficient. Furthermore, in the situation with the extended gates the guidance of the gates will not include the complete height of the gates (See Figure 6.10). The gates will be enlarged with about 1/3-2/3 of the original height. In the guidance of the gates three different materials are used; aluminum bronze for the sliding plates, steel for the anchoring plates and cast-iron for the 'chairs' on the side guidance (See Figure 6.11). Associated with the working sequence of the original execution, the guidance is designed with a large safety margin [RWS4, 1985]. In the original calculation of the guidance of the gates, the final water pressure an wave forces exerted on the guidance where only known after applying the anchorage structures in the piers. The maximum contact pressures occurs in closed situation at the 6m gates at the Roompot location. The location with the highest forces is at the lower slide stops. The maximum contact pressure between the stop and the aluminum bronze is 15 N/mm^2.

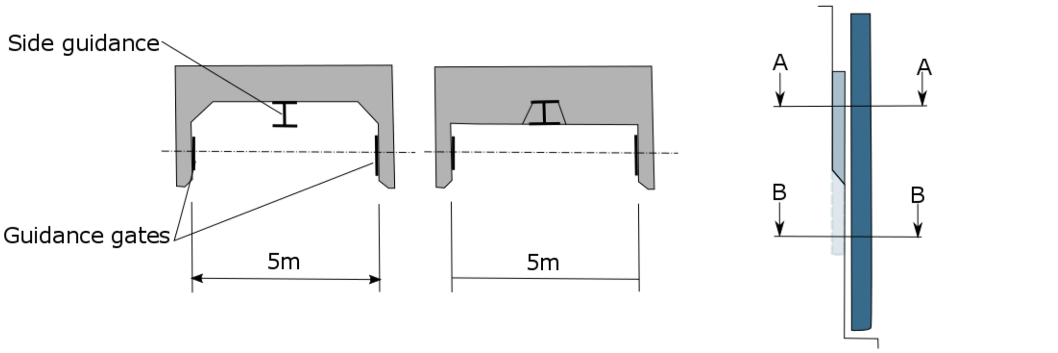


Figure 6.10: Cross-section gates guidance

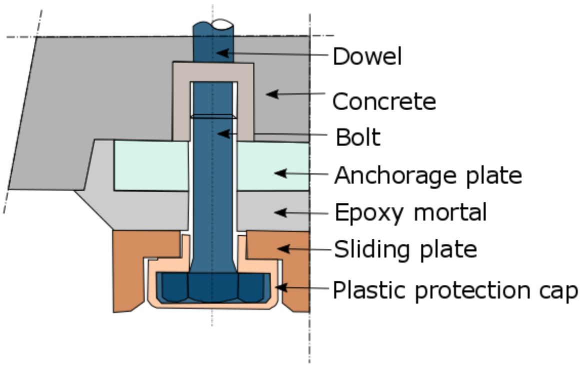


Figure 6.11: Materials guidance gate

Then by epoxy mortal the load is spreading with an angle of 45 degrees from the sliding plates to the anchorage plates. The maximum pressure in the epoxy mortar is 11 N/mm^2 . The measured compressive strength, at the initial stage, is in the range from 65 to 100 N/mm^2 [RWS4, 1985]. Due to the increase of the height of the gates and the unchanged guidance height, the enlarged surface pressures on the bearings and the guidance of the 6m gates is checked in the next paragraph.

Surface pressure check bearings and guidance gates

The height enlargement of the 6m gates is about 4m. To get first insight in load increasement the original maximum contact pressure is increased with 4/6. To account for dynamic flow, which could exert large force on the tip of the gates, an additional dynamic load factor of 1.5 is used. This means the maximum contact pressure of the epoxy mortar in the new situation becomes of 2 times the original contact pressure (22 N/mm^2). The compressive strength of the epoxy mortal, at the initial stage, is in the range of from 65 to 100 N/mm^2. Assuming a strength reduction of 50 %, the compressive strength is still higher than the acting load.

7
Construction and planning

This chapter threats sub research question 5 & 6. The chapter covers the executional method, including the required equipment for executing the lowered sill beam alternative. First a proper construction approach is determined. Next, the construction approach is elaborated. The last topic of this chapter contains a planning of the work.

7.1. Construction

The main parts of the execution of the lowered sill beam are elaborated in this chapter. The items are listed below:

1. Removal (part of) the bottom protection
2. Lowering sill beam
3. Replacement bottom protection
4. Replacement of the gates

When executing civil work of this size is often, in early stage, already determined which execution direction is chosen. A distinction is made between an execution ”in the wet” (wet conditions) or ”in the dry” (dry conditions). Both directions have their advantages and disadvantages. Although for some activities it is obvious to execute ”in the wet” , for some activities it’s worth to make a good analysis to perform “in the wet” or “in the dry”.

7.1.1. Bottom protection removal

In the Lowered sill beam alternative a part of the stones of the sill construction must be removed. This activity is carried out “in the wet”. The work mainly consist of the removal of the 1-3 tons stones that are placed against the sill beam on the Eastern Scheldt side of the barrier. The current 1-3 tons stone are placed by means of drilled in hoisting eye. These hoisting eyes could be used to remove the necessary part of the stones. If from lifting attempts it appears that the hoisting eyes are not sufficiently strong they should be moved to the lower part of the sill construction. The removal can be done by a backhoe which is a hydraulic crane positioned on a pontoon. The pontoon can be secured by spud poles. The work can also be done by converting a grab dredger for lifting works. The maximum depth from which stones have to be remove is NAP -14.5 m. With a Mean High Water of NAP +1.33m the maximum lifting depth is about 16 m. In Figure 7.1 the used equipment for this activity is shown.

7.1.2. Lowering sill beam

The most important part of chosen alternative is the lowering of the sill beam. The actual lowering can been done by e.g. a buzz saw, a wire saw or a crane with mechanic sludge hammer. The lowering method largely depend on the executional method. In the next paragraphs two execution methods are elaborated. Two executional methods are elaborated, namely: “in the wet” and “in the dry”.

Figure 7.1: Backhoe dredger
(*source:* http://www.boskalis.com/)

Sequence of work - "In the dry"

In this method, the openings of the barrier will be closed one by one. The execution method "in the dry" will in general consist of the activities outlined in next paragraphs. Each part will be briefly explained.

Placement cofferdam The openings will be closed by the placement of prefabricated steel molds. These molds can be reused for all the openings. The steel molds span about 40-45 m. The steel mold span about the same width as the original gates but have to resist a maximum water pressure difference of about 14 m (mean sea level - half way the original sill beam). The structure of the steel mold is not design in this MSc. Thesis, but the mass is estimated on 2 times the weight of the original gates (about 2*500 tons = 1000 tons). This means the mold have to be placed from the water by a floating sheerleg (See Figure 7.4). The mold are not placed between the piers, but are placed against the piers. After the placement of the prefabricated steel molds, steel struts are being placed between the mold to strengthening the building pit. The struts should be placed in such a way that enough working space is guaranteed. [1]

Removing water building pit Than the building pits is pumped dry. At the outer edges of the steel mold, rubbers profiles are being attached. This is to ensure the water tightness of the building pit. The pumps will be placed at the top of the steel molds. Outside the working space of the equipment. Leakage trough the bottom protection should also being prevented, but it seems rather difficult to achieve this at such a depth and against such a water pressure.

Placement of equipment To not disturb the traffic the required equipment will be hoisted from the water into the building pit. The equipment consist of an excavator with a hydraulic pulverizer (crusher) and/or hydraulic hammers.

Lowering sill beam Than the sill beam will be lowered by of an excavator with a hydraulic pulverizer and hydraulic hammer. The concrete is pulverized and the reinforcement steel pinched of by the hydraulic pulverizer. The width of the excavator is about 3-4 m. This is approximately the same width as the upper part of the sill beam. This give the excavator enough room to maneuver. The height in the building pit is quite limited which makes it hard for the excavator to move his hydraulic arm up and down.

[1] During the analysis of the current situation, it has been found that it is not possible to hoist in a steel mold at the Eastern Scheldt side of the barrier without applying major alterations to the traffic deck. Given the disturbance that this entails it is concluded that this method of execution is not feasible. For the further elaboration of the execution method "in the dry", however, it is still assumed that placement is possible. An execution method to sail in the steel molds from the Eastern Scheldt side of the barrier are in this MSc. Thesis not considered.

After-treatment concrete After the lowering of the sill beam a part of the reinforcement steel has become exposed. This needs to be restored to prevent further damage to the reinforcement. The restoration is done by grouting the exposed reinforcement. In Figure 7.2 the working sequence execution ”in the dry” is elaborated graphically.

Sequence of work - ”In the wet”

In this method, the openings of the barrier also will be closed one by one. The execution method “in the wet” will in general consist of the activities outlined in the next paragraphs. Each part will be briefly explained.

Placing cutting frames On both sides of the sill beam a so called cutting frame will be lifted into the notches of the piers by a floating sheerleg. The cutting frames are fixing against the piers by hydraulic jacks. The cutting frames consist of steel truss girders. A robust design of the cutting frames is expected, because during the cutting process high forces are exerted on the frame. The mass is estimated in the same order as the original gates (about 500 tons).

Tensioning wired saw Between the two frames the wired saw should be installed. The managing of the saw installation is done from a pontoon which is positioned near the barrier. This cutting frames are the basis for a wired saw machine which is placed between the two frames. The material of the wired saw consist of diamond. Despite of that it is still expected that the wired saw have to be replaced after lowering each sill beam.

Cutting With the help of the wire saw machine several vertical cuts will be made into the top of the sill beam. These vertical cuts are needed to remove the top of the sill beam in parts and to prevent the wire saw against jamming. After applying the vertical cuts, the different top parts of the sill beam will be removed from the bottom part of the beam by a horizontal cut, also made by the wired saw.

Removing After that the different parts can one by one be removed. The discharge of released material will be hoisted into trucks and will be transported. Reuse of material will be, as much as possible, be pursued.

After-treatment concrete After the lowering of the sill beam a part of the reinforcement steel has become exposed. This needs to be restored to prevent further damage to the reinforcement. The restoration is done by grouting the exposed reinforcement. This activity is difficult, because grouting the exposed reinforcement have to be done with large flow velocity and poor visibility. Therefore the after-treatment of concrete is easier in execution method “in the dry”. In Figure 7.3 the working sequence execution ”in the wet” is elaborated graphically.

Execution method consideration

In this paragraph the two executions directions are assessed for cost, disturbance and risk. On the basis of these criteria the two execution methods (“in the wet” and “in the dry”)different alternatives are weighted in a multi criteria analyses (see Table 7.1)

Table 7.1: Multi criteria analyses (MCA) execution method lowering sill beam

	In the wet	**In the dry**
Cost	-	0
Disturbance	0	-
Risk	0	-
Total	**0**	**-**

The cost of execution “in the wet” are higher, because of the use of heavy machinery. While execution “in the dry can be done by the use of relatively ‘small’ hydraulic cranes. Despite of that the risks of execution “in the dry” are higher. The dewatering of the building pit introduces high force on the rest of the structure and it is doubtful if the building put can be dewatered completely. This is not

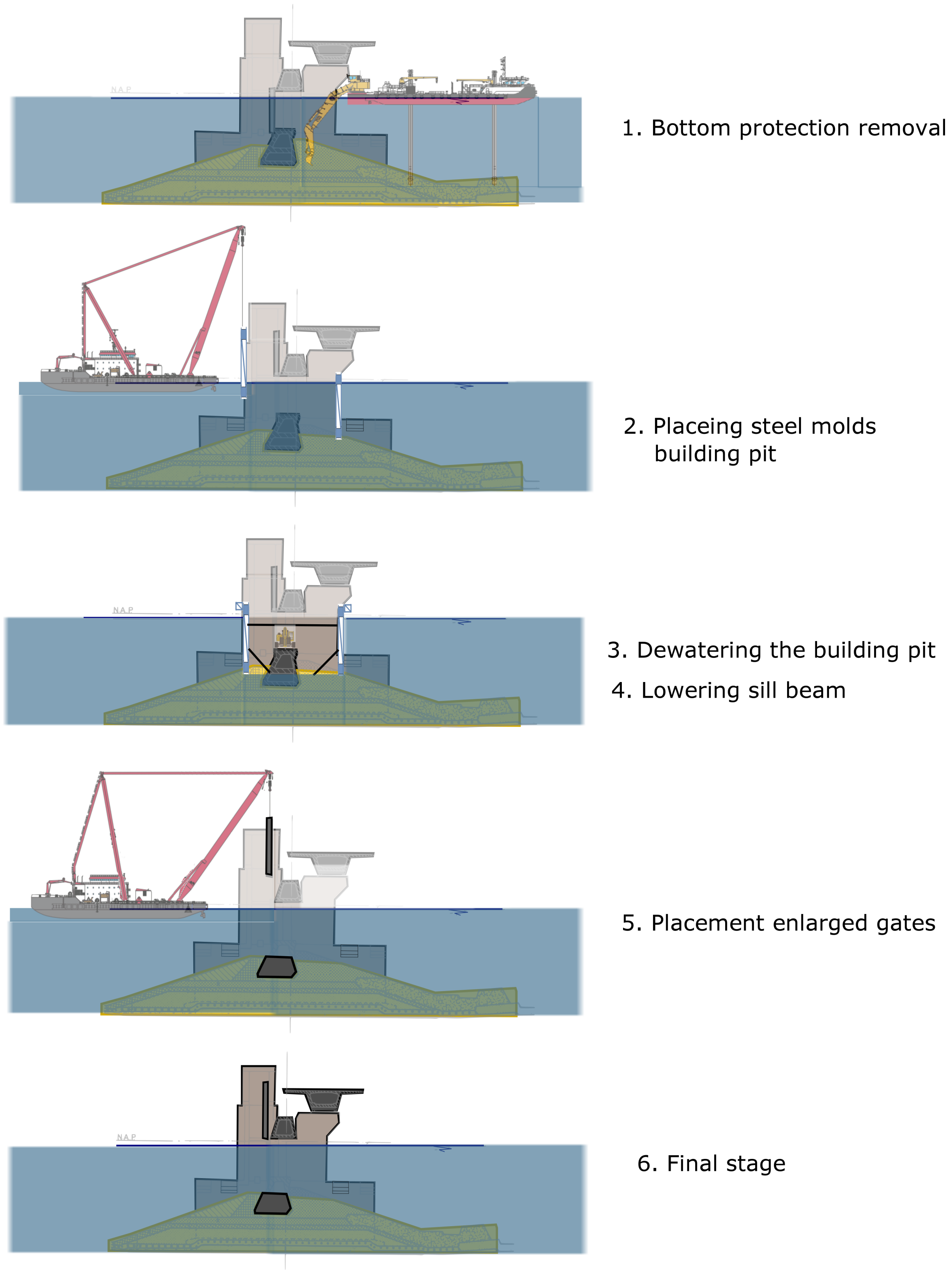


Figure 7.2: Working sequence execution ”in the dry”

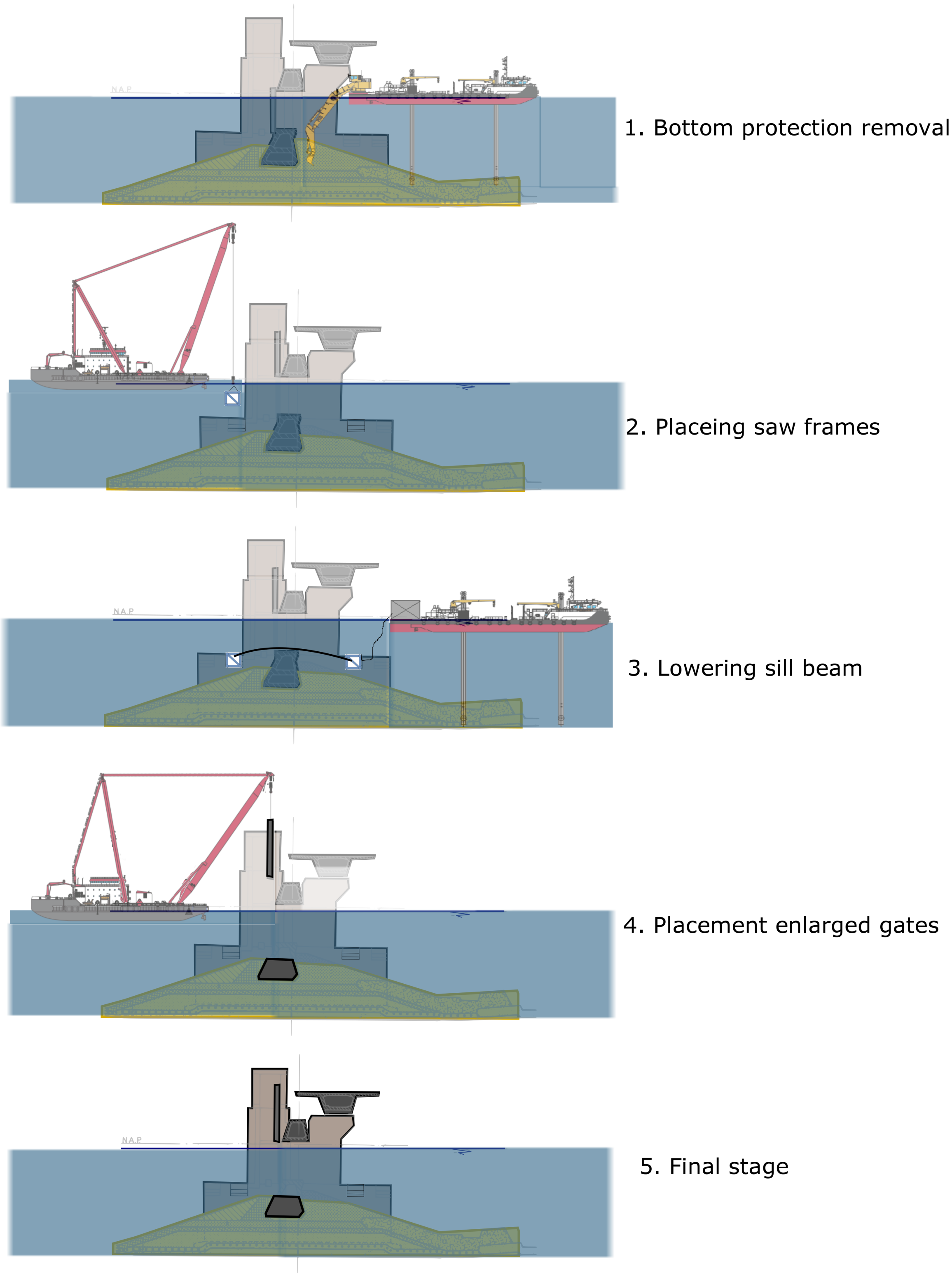


Figure 7.3: Working sequence execution "in the wet"

the case when execution "in the wet". Also disturbance when execution "in the wet" is less, because everything can be done from the water.

7.1.3. Replacement of the gates

The current gates in the Eastern Scheldt storm surge barrier will be replaced by higher gates. By using a lightweight material the weight of the new gates should be in same order as the current gates. For replacing the current gates similar equipment can be used as in construction phase of the barrier. In [RWS4, 1985] is noted that for placing of the gates the floating sheerleg Taklift 4 (See Figure 7.4) is used. This ship is still in use and therefore can be used. The gates will be transported to the current barrier on a pontoon. With the help of a customized lifting frame the Taklift 4 can place the enlarged gates into the current.

Figure 7.4: Floating sheerleg, Taklift 4
(*source:* `http://www.shipspotting.com/`)

7.2. Planning

Is this paragraph the planning of the execution is treated. The objective of this planning is to organize the activities is such a way the availability of the barrier should not be compromised and the disturbance time near the barrier should be as low as possible. The different construction activities and their durations are indicated in Table 7.2.

In the planning 15% of the total construction time is accounted for to include delay and non-working days. To achieve the availability and disturbance goals of the planning the production of the enlarged should start simultaneously with or before the production of the saw frame. So the lowering of the sill beam can be followed by placing the enlarged gates. In that way the non-availability of the barrier is minimalized and the disturbance near the barrier is less. Other measures to minimalize the non-availability is to execute the activity in a sequence per a predefined number of openings. A global planning of the construction is presented on the next page. Because of safety reasons no activities near the barrier are planned during the storm season (from October till April). As can be seen the work is carried out in blocks of three times 5 months. In the next paragraph the duration for each activity is elaborated in more detail.

7.2.1. Elaboration duration activities

As can be seen the work is carried out in blocks of three times 5 months. So during a period of 5 months the alternative has to be executed at about 50/3 $\approx$ 17 openings. This means $\approx$ 9 days per

Table 7.2: Activities execution lowered sill beam alternative

Activity	Duration [months]
Preliminary works (e.g. design, application of permits)	7
Construction enlarged gates	6
Production saw frame	3
Preparations for work on site	2
Adjustments stones sill construction	3x5
Lowering sill beams	3x5
Finishing concrete	3x5
Adjustments gate lifting equipment	3x5
Placing enlarged gates	3x5
Buffer	12

opening. In Table 7.3 the breakdown of the duration per activities, per opening is elaborated.

Table 7.3: Breakdown of duration activities per opening

Activity	Duration [days]
Adjustments stones sill construction	2
Lowering sill beams	9
Finishing concrete	5
Adjustments gate lifting equipment	2
Placing enlarged gates	3
Total	**21**

As can be seen from Table 7.3 21 days are needed per opening. This is in contradiction to the previously mentioned 9 days per opening, but the work on different openings can be executed simultaneously. The time schedule of 9 days per opening (5 months in total) is therefore based on the availability of the saw frame. On the next page the overall planning is shown.

	Year	2016												2017												2018												2019											
Planning lowered sill beam alternative	*Months*	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
Preliminary works (e.g. design, application of permits)	7																																																
Construction enlarged gates	6																																																
Production saw frame	3																																																
Preparations for work on site	2																																																
Adjustments stones sill construction	3x5																																																
Lowering sill beams	3x5																																																
Finishing concrete	3x5																																																
Adjustments gate lifting equipment	3x5																																																
Placing enlarged gates	3x5																																																

7.2.2. Planning remarks

- In the preparation of the planning it is assumed that the necessary permit for the project will be granted. This will in practice probably be a tricky thing, because the work is performed on a primary sea defense
- The suggested schedule assumes that the work at the three location of the barrier can be done in three shifts of 5 months. If during the execution it appear that more time is needed than 5 months per shift, it can be decided that for the second shift an extra saw frame is produced.

7.3. Conclusion construction and planning

From the explanation in this shaped can be stated that execution of the lowered sill beam alternative seems feasible. Although a saw frame should be produced, there is no need to produce expensive equipment. The work can been done with existing machinery. But further research is needed into the feasibility to adjust the barrier while the water is flowing with high speed.

8

Conclusions, discussion and recommendations

The MSc. Thesis ends with a conclusion in which the main results of the research are presented. Furthermore a few recommendations are done and this chapter will end with a brief discussion.

8.1. Conclusion

The objective of this MSc. Thesis was to investigate how the Eastern Scheldt storm surge barrier can contribute in solving (future) challenges in the Eastern Scheldt area. Main features where to investigate adjustments to the moveable part of the Eastern Scheldt storm surge barrier which can lead to an improved functionality of the barrier and have a positive effect on the Eastern Scheldt area. Special attention is paid to improving the "allow tidal movement" functions of the Eastern Scheldt Storm storm surge barrier. To accomplish this is in Chapter 1 a main research question was formulated:

What are feasible, cost-effective alternatives for an improved functionality of the Eastern Scheldt storm surge barrier?

In order to achieve the answering of the main research question several subresearch questions are elaborated through the course of this MSc. Thesis (See Chapter 1.5.1). In this paragraph the main results are presented pointwise.

- The Eastern Scheldt area is facing future (unanticipated) challenges like the sea level rise, the sand demand of the Eastern Scheldt, the scour holes near the shores of Schouwen-Duiveland and new safety standard for water defenses.
- The construction of the Eastern Scheldt storm surge barrier have had a great influence on the creation of the "sand demand". After the completion of the barrier in 1986, the flow velocity and tidal prism decreased with about 50 %. Causing that already 1.100 ha of tidal flats and tidal muds in the Eastern Scheldt are drowned. On an average the height of the tidal flats and tidal muds decreased with approximately 25 cm.
- The main functions of the Eastern Scheldt storm surge barrier are to: Retain water, allow tidal movement, provide transportation and provide recreation.
- The most promising function which can be improved is the "allow tidal movement function". The Dutch government is planning to supply 1.65 million m^3 sand on the Roggenplaat in the Eastern Scheldt untill 2025 to fight the symptons of the sand demand. After this first phase there are plans to supply more sand in the Eastern Scheldt. Amount up to 12 to 65 million m^3 meaning an estimation of costs of 77-422 million euros. This measure only secures the short-term goals on preserving the inter tidal flats. If one succeed to adjust the tidal movement function of the Eastern Scheldt storm surge barrier in such a way that the natural process of sand suppletion on the tidal flats is restored a more long-term solution can be achieved.

- Because morphological changes mainly occur during successive spring tides in normal conditions there should is been searched for alternatives which are effective during normal conditions. By enlarging the cross sectional area of the barrier the tidal volume through the barrier and the tidal flow velocity in the Eastern Scheldt enlarges. This enlargement of the tidal velocity in the Eastern Scheldt should contribute in an improvement of the sand demand problem.
- An alternative in which 50 of the current sill beams are lowered and the gates are enlarged is the most promising alternative for improve the functionality of the storm surge barrier. With the alternative the cross sectional area of the Eastern Scheld storm surge barrier is increasing with approximately 15 %, meaning the tidal movement is recovered to approximately 80 % of the original tide. Further increase of the cross sectional "flow" area would mean that more sill beams have to be lowered or removed or large parts of the barrier, like the piers, have to be removed. Although the effect of these measure seems larger, they could have negative effects on the shores of Schouwen and Noord-Beveland or even have negative effect on the safety of the hinterland.
- It's technically possible to lower the sill beams (strength and stability). Although it is necessary to remove a part concrete and a part of the prestressed tendons, the moment capacity of the sill beam is still sufficient. This can be explained by the fact that the pre-tensioning cables that have the most influence on the moment capacity remain intact. Also the stability of the sill beam and the overall stability of the Eastern Scheldt storm surge barrier are still guaranteed.
- The execution of the lowered sill beam alternative seems feasible. As an executional method "in the wet" should be chosen. Although a saw frame should be produced, there is no need to produce expensive equipment. The work can been done with existing machinery. But further research is needed into the feasibility to adjust the barrier while the water is flowing with high speed.
- The estimated costs of the alternative amount to 750 million euros within a range of +/- 20%.
- The estimated cost for maintaining the barrier in its original state amount to 650 million euros within a range of +/- 20%.
- This research has shown that the cost of adjusting the sill beam of the barrier cost more than maintaining the barrier in its current state. Although the structural safety of the alternative during the operational phase is guaranteed and the effect on the sand demand is more promising in the long-term, it is still doubtful if the cross sectional increase of the Eastern Scheldt storm surge barrier has the desired effect on sand demand.

8.2. Recommendations

- In Chapter 5 of this research a few assumption in the relation between the built up of inter tidal flats and the sand in suspension has been used to value the impact of adjustments to the barrier. Further investigation into this relation should verify the validity of the assumptions.
- In this MSc. Thesis alternatives in which the sill beams are fully replaced are not been treated. It should be investigated if this alternatives is a more safe and cheaper alternative.
- The cost comparison in this MSc. Thesis are based on rather rough estimated. Fine-tuning of the costs should lead to the actual cost of the alternatives
- In the alternative a part of the stones of the sill construction must be removed to create work space. After the lowering of the sill beams the sill construction (consisting of place stones) is restore. Because of the lowering of the sill beam and the adjustments to the sill construction the flow pattern through the barrier changes. Calculations must show whether the stones top layer of the sill constructions are still stable.
- In the current situation, the design of the sill beam is arranged in such a way that the upper part of the beam serve as a stop for the gates. In the lowered beam alternative the stops are not present (or in limited extend available). Due to the change of the stops, the position of the hydraulic load force shifts up. In that extend the overall stability of the structure is checked,

but there has to be determined whether these stops are still necessary for other reasons. For example because the leakage through the gates become too much.

- In the lowered sill beam alternative the current gates in the Eastern Scheldt storm surge barrier will be replaced by higher gates. In the design of the alternative there is assumed that the gates will be replaced by gates with a lightweight material (e.g. FRP). In that case the weight of the gates should remain the same. If the weight of the extended slide are still higher than the weight of the existing gates, one should examine if the lifting capacity if the current gate equipment is sufficient. Furthermore, in the situation with the extended gates the guidance of the gates will not include the complete height of the gates. Dynamic flow force could exert large force on the tip of the gates. Therefore there should be examined whether this causes problems and whether the current guidance have to be adjusted.

8.3. Discussion

In this MSc. Thesis the Eastern Scheldt storm surge barrier with a design lifetime of 200 years is treated. Topic of discussion will remain whether designing with a lifetime of 200 year is reality or illusion. Large infrastructural project have large environmental and morphological impact. In the pre stage of these project simulations model predicts the impact. If during the life of the structure predictions deviate from reality, there is often no room to make adjustments to the structure. You see this happened in the design of the Eastern Scheldt storm surge barrier. Because of the robustness of the design it is almost impossible to make, economically attractive, modifications to improve functionality of the barrier. If designing with a lifetime of 200 year is still desirable, the structure should be designed in such a way that it is adaptive to future changes.

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A

Lifecycle cost analyses

Life cycle cost analyses (LCCA)

Start:	2020	
Lifetime	50	year
Interest rate	0.025	-
Replacement current gates	2036	

Cost Eastern scheldt storm surge barrier

Construction cost			€ 4,100,000,000
In which:			
Construction sill beam		10%	€ 410,000,000
Construction sill construction		5%	€ 205,000,000

Maintenance

Current cost	€ 18,000,000	per year
Cost after replacement gates	€ 9,000,000	per year
Cost (without gates)	€ 6,000,000	per year

Adjustments

Demolishing cost sill beam	30%	of constr. cost	€ 123,000,000	
Demolishing cost sill construction	30%	of constr. cost	€ 61,500,000	
Cost new gates			€ 225,000,000	
			€ 3,629,032	per gate
Extra cost gate elongation			€ 1,451,613	per gate

Sand suppletions

Suppletion 100%		€ 422,000,000	in	45 years
Baseline alternative:	100%	€ 46,888,889	every	5 years
Lowered sill beam alternative:	85%	€ 39,855,556	every	5 years
Lowered sill alternative:	65%	€ 30,477,778	every	5 years

Dike heightening

Cost per meter per km dike heightening		€ 4,100,000	/km
km dike		150	km
		0.9	m
Total		€ 553,500,000	
	per year	€ 27,675,000	in 20 years

Baseline alternative (30%)

		NPV (*10^{6})	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043	2044	2045
Maintenance			*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*18.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*
	45%	€ 295	18.00	17.56	17.13	16.71	16.31	15.91	15.52	15.14	14.77	14.41	14.06	13.72	13.38	13.06	12.74	12.43	6.06	5.91	5.77	5.63	5.49	5.36	5.23	5.10	4.98	4.85
Suppletions			*46.89*					*46.89*					*46.89*					*46.89*					*46.89*					*46.89*
	32%	€ 211	46.89	0.00	0.00	0.00	0.00	41.44	0.00	0.00	0.00	0.00	36.63	0.00	0.00	0.00	0.00	32.38	0.00	0.00	0.00	0.00	28.61	0.00	0.00	0.00	0.00	25.29
Cost replacement gates			*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*225.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*
	23%	*€ 152*	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	151.57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total		***€ 658***																										

2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063	2064	2065	2066	2067	2068	2069	2070
9.00	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*
4.74	4.62	4.51	4.40	4.29	4.19	4.08	3.98	3.89	3.79	3.70	3.61	3.52	3.44	3.35	3.27	3.19	3.11	3.04	2.96	2.89	2.82	2.75	2.68	2.62
				46.89					*46.89*					*46.89*					*46.89*					*46.89*
0.00	0.00	0.00	0.00	22.35	0.00	0.00	0.00	0.00	19.76	0.00	0.00	0.00	0.00	17.46	0.00	0.00	0.00	0.00	15.43	0.00	0.00	0.00	0.00	13.64

Lowered sill beam alternative (20%)

		NPV (*10^{6})	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043	2044	2045
Maintenance			*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*
	23%	€ 175	9.00	8.78	8.57	8.36	8.15	7.95	7.76	7.57	7.39	7.21	7.03	6.86	6.69	6.53	6.37	6.21	6.06	5.91	5.77	5.63	5.49	5.36	5.23	5.10	4.98	4.85
Suppletions			*39.86*					*39.86*					*39.86*					*39.86*					*39.86*					*39.86*
	24%	€ 180	39.86	0.00	0.00	0.00	0.00	35.23	0.00	0.00	0.00	0.00	31.14	0.00	0.00	0.00	0.00	27.52	0.00	0.00	0.00	0.00	24.32	0.00	0.00	0.00	0.00	21.50
Cost replacement gates			*203.23*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*79.84*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*
	34%	*€ 257*	203.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	53.78	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Lowering s.	*12%*	*€ 92*	92.25																									
Other adju:	*7%*	*€ 50*	50.00																									
Totaal		***€ 754***																										

2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063	2064	2065	2066	2067	2068	2069	2070
9.00	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*	*9.00*
4.74	4.62	4.51	4.40	4.29	4.19	4.08	3.98	3.89	3.79	3.70	3.61	3.52	3.44	3.35	3.27	3.19	3.11	3.04	2.96	2.89	2.82	2.75	2.68	2.62
				39.86					*39.86*					*39.86*					*39.86*					*39.86*
0.00	0.00	0.00	0.00	19.00	0.00	0.00	0.00	0.00	16.79	0.00	0.00	0.00	0.00	14.84	0.00	0.00	0.00	0.00	13.12	0.00	0.00	0.00	0.00	11.60

Lowered sill alternative (10%)

		NPV (*10^6)	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043	2044	2045
Maintenance			*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*
	12%	€ 117	6.00	5.85	5.71	5.57	5.44	5.30	5.17	5.05	4.92	4.80	4.69	4.57	4.46	4.35	4.25	4.14	4.04	3.94	3.85	3.75	3.66	3.57	3.49	3.40	3.32	3.24
Suppletions			*30.48*	*30.48*				*30.48*					*30.48*					*30.48*					*30.48*					*30.48*
	17%	€ 167	30.48	29.73	0.00	0.00	0.00	26.94	0.00	0.00	0.00	0.00	23.81	0.00	0.00	0.00	0.00	21.04	0.00	0.00	0.00	0.00	18.60	0.00	0.00	0.00	0.00	16.44
Remove sil	*12%*	*123*	*123.00*																									
Removal pa	*6%*	*€ 62*	61.50																									
Adjustmen	*8%*	*€ 80*	80.00																									
			27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68	27.68					
Dike heigh	*46%*	***459.105***	*27.68*	*27.00*	*26.34*	*25.70*	*25.07*	*24.46*	*23.86*	*23.28*	*22.71*	*22.16*	*21.62*	*21.09*	*20.58*	*20.08*	*19.59*	*19.11*	*18.64*	*18.19*	*17.74*	*17.31*	*16.89*					
Totaal		***€ 1,007***																										

2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063	2064	2065	2066	2067	2068	2069	2070
6.00	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*	*6.00*
3.16	3.08	3.01	2.93	2.86	2.79	2.72	2.66	2.59	2.53	2.47	2.41	2.35	2.29	2.23	2.18	2.13	2.08	2.02	1.98	1.93	1.88	1.83	1.79	1.75
				30.48					*30.48*					*30.48*					*30.48*					*30.48*
0.00	0.00	0.00	0.00	14.53	0.00	0.00	0.00	0.00	12.84	0.00	0.00	0.00	0.00	11.35	0.00	0.00	0.00	0.00	10.03	0.00	0.00	0.00	0.00	8.87

B

Design report lowered sill beam

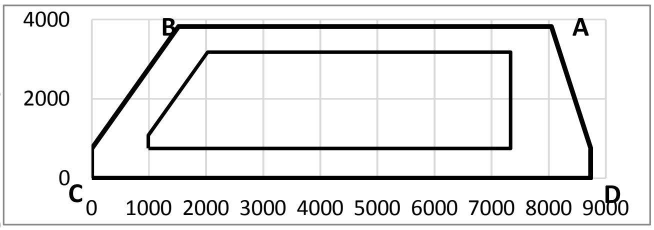
In this appendix the main dimensions of the lowered sill beam will be checked.

Belastingen situatie: Gesloten kering

#	Gegevens	
	Diepte (L)	5,5
	Diepte (R)	-0,7
	Hoogt totaal	3,825
		0,75
		1

1520 6528 687

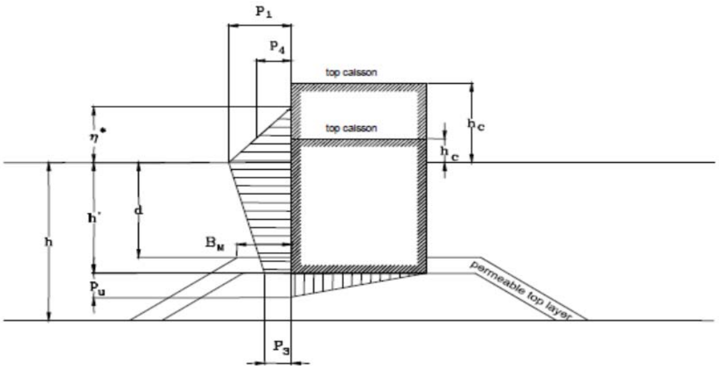
3075 3075



750 750

8735

Golfbelasting conform Goda-Takahashi



 # Invoer

NAP [m]	-12,5	(O.k dorpelbalk in m t.o.v. NAP)
H_s [m]	2,88	(Significante golfhoogte)
T [s]	9,5	(Golfperiode)
H_d [m]	6,33	(Ontwerphoogte golf voor de constructie, 2.2*Hs)
L_0 [m]	132,0	(Golflengte)
ρ_w	1025	(Soortelijk gewicht water)
b	0,0	(Hoek inkomende golf)
d [m]	10,00	(Waterdiepte boven de bovenkant van de drempelconstructie)
h [m]	36,00	(Waterdiepte voor de drempelconstructie)
h_b	36	(Waterdiepte op 5*H_D vanaf de constructie)
h` [m]	13,00	(Waterdiepte boven de fundering van de constructie)
h_{sill} [m]	26,0	(Drempelhoogte, =h-h')
B_M [m]	20	(Breedte drempel)
h* [-]	9,49	(Toename gemiddelde waterdiepte)
h_c	0,30	(Hoogte constructie boven de gemiddelde waterdiepte)
h_c*	0,30	(min(h*,h_c))

 # Berekening coëfficienten

λ_1 [-]	1	(Factor afhankelijk van de vorm van de constructie en de golfcondities)
λ_2 [-]	1	(Factor afhankelijk van de vorm van de constructie en de golfcondities)
λ_3 [-]	1	(Factor afhankelijk van de vorm van de constructie en de golfcondities)
α_1 [-]	0,625	
α_2 [-]	0,096	
α_3 [-]	0,765	
α_4 [-]	0,968	

 # Uitvoer

p_1 [kN/m^2]	45,87	
p_3 [kN/m^2]	35,09	
p_4 [kN/m^2]	44,42	
p_u [kN/m^2]	30,40	
F_{max} [kN]	**539,74**	(maximale kracht tegen de constructie)
z_v	**6,95**	

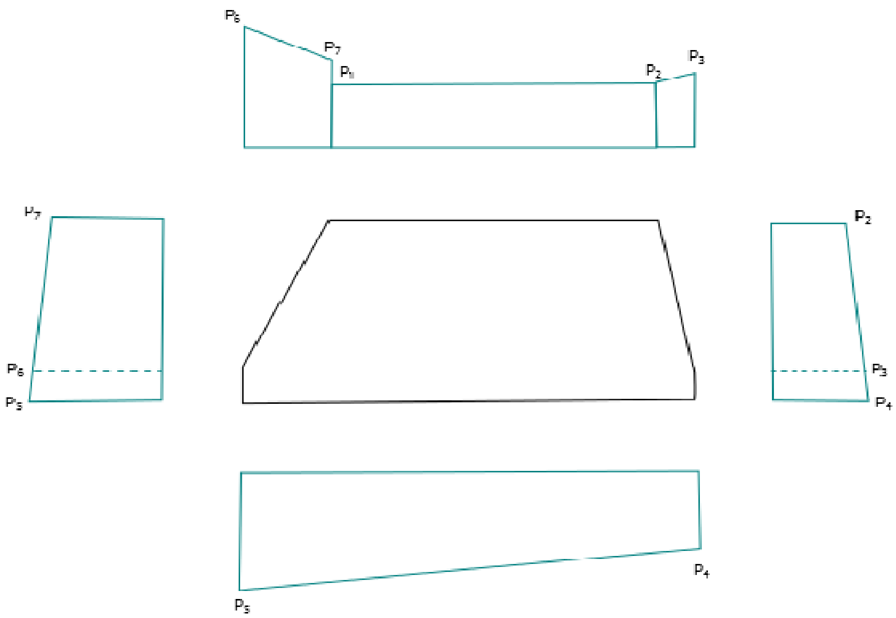
 # Samenvatting golfdrukken per dorpelbalk diepte

	O.k dorpelbalk in m t.o.v. NAP						
Druk	-12,5	-13,5	-14,5	-14,5	-16,5	-17,5	-18,5
p_1 [kN/m^2]	45,9	44,6	43,7	42,9	42,4	41,9	41,6
p_3 [kN/m^2]	35,1	33,3	31,8	30,5	29,4	28,3	27,3
p_4 [kN/m^2]	44,4	43,2	42,3	41,6	41,0	40,6	40,3
p_u [kN/m^2]	30,4	29,7	29,0	28,2	27,5	26,8	26,1
F_{max} [kN]	539,7	558,7	579,1	600,5	622,3	644,5	666,6
z_v [m]	6,95	7,50	8,05	8,61	9,18	9,75	10,32

Hydraulische belasting - Gesloten kering

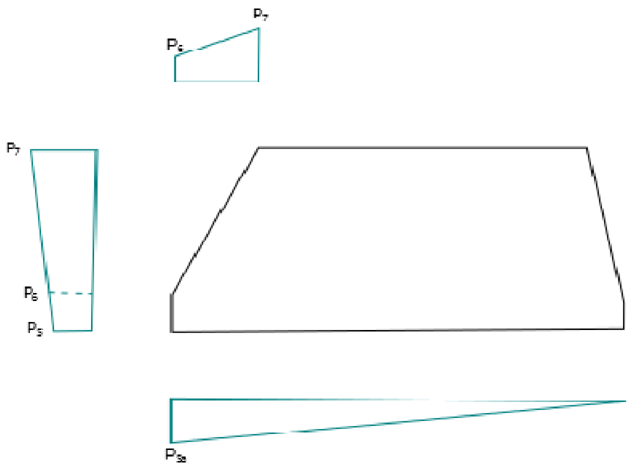
Hydraulische belasting (statisch)

	p1	p2	p3	p4	p5	p6	p7	Rh	Rv
-8,675	80	80	111	119	181	173	143	192	-479
-9,675	90	90	121	129	191	184	153	192	-479
-10,675	100	100	131	139	201	194	163	192	-479
-11,675	110	110	141	149	211	204	173	192	-479
-12,675	120	120	151	159	221	214	183	192	-479
-13,675	130	130	161	169	231	224	193	192	-479
-14,675	141	141	171	179	241	234	203	192	-479



Hydraulische belasting (golf) - Conform Goda-Takahashi

	p1	p2	p3	p4	p5	p5a	p6	p7	Rh	Rv
-8,675				0	35	30	36	38	104	-77
-9,675				0	33	30	34	36	101	-77
-10,675				0	32	29	32	34	97	-76
-11,675				0	31	28	31	33	94	-75
-12,675				0	29	28	30	32	91	-73
-13,675				0	28	27	29	31	89	-72
-14,675				0	27	26	28	30	86	-70



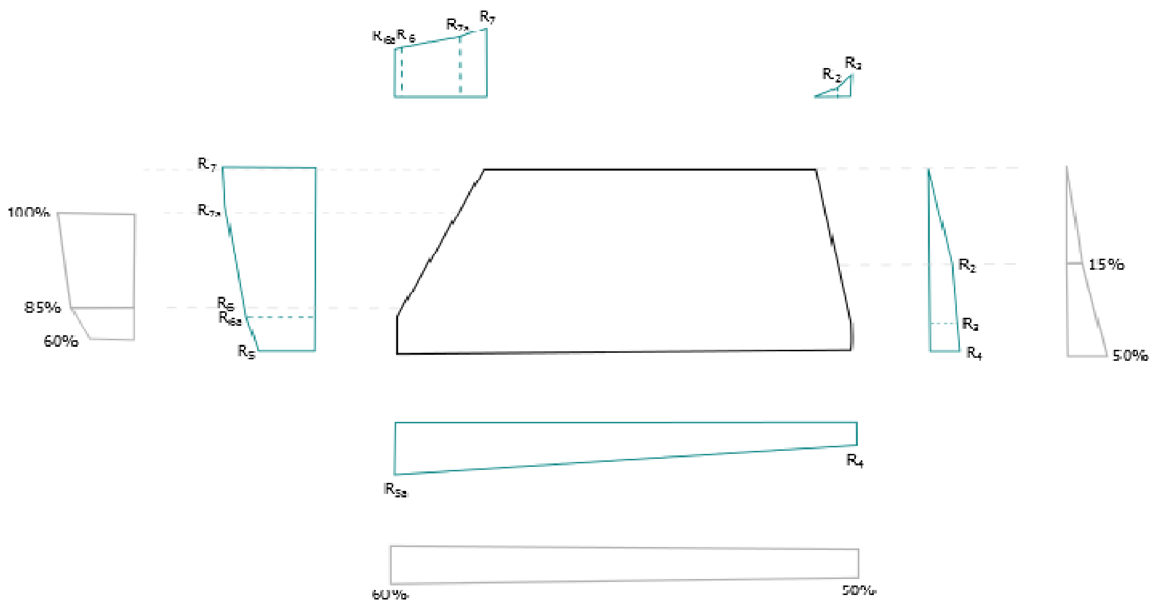
 # Totaal (statisch + golf)

	p1	p2	p3	p4	p5	p5+p5a	p6	p7	Rh	Rv
-8,675	80	80	111	119	216	211	209	180	296	-557
-9,675	90	90	121	129	224	221	217	188	292	-556
-10,675	100	100	131	139	233	230	226	197	289	-555
-11,675	110	110	141	149	242	239	235	206	286	-554
-12,675	120	120	151	159	251	249	243	215	283	-553
-13,675	130	130	161	169	260	258	252	224	280	-551
-14,675	141	141	171	179	269	267	262	233	278	-550
										-557

 # Totaal gereduceerde (statisch + golf)

Reductie door afname verval door bodembescherming

	p1	p2	p3	p4	p5	p5	p6a	P6	p7a	p7	Rh	Rv
-8,675		14	34	46	56	58	66	77	99	100	253	-321
-9,675		14	34	46	55	57	64	75	98	98	247	-318
-10,675		14	34	46	55	57	63	74	96	97	243	-314
-11,675		14	33	45	54	56	62	73	95	95	239	-311
-12,675		13	33	45	54	55	61	72	94	94	236	-308
-13,675		13	33	45	53	54	60	71	93	93	233	-305
-14,675		13	33	44	53	54	60	70	92	92	230	-302
										MAX	**253**	**-321**
										MIN	**230**	**-302**



Samenvatting belastingen

$q_{G;e.g.}$	242	kN/m^1
$q_{G;zand}$	98	kN/m^1
$q_{Q;h;w}$	253	kN/m^1
$q_{Q;v}$	-321	kN/m^1
$q_{Q;v;rock}$	100	kN/m^1

Berekening Doorsnedegrootheden - Output Bitmap Cross Sectional Analyser

Current cross section

BITMAP CROSS SECTIONAL ANALYSER RESULTS : Dorpelbalk.bmp

Sectional boundaries:
width = 320 equals to 8735 mm
height = 256 equals to 8000 mm

material properties:

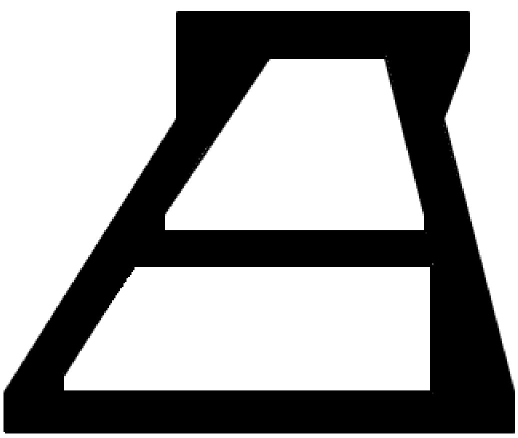
	color	E [N/mm^2]	rho [kg/m^3]
Concrete	clBlack	1	2500

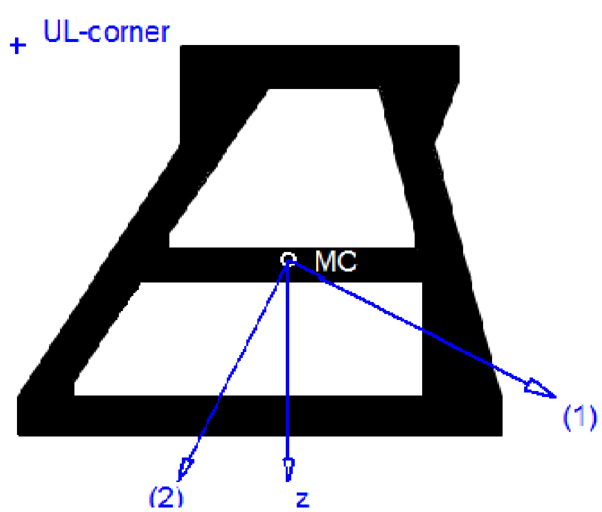
normal center NC at position:
hor = 4886.14 mm from upper left corner UL
ver = 4406.25 mm from upper left corner UL

mass center MC at position:
hor = 4886.14 mm from upper left corner UL
ver = 4406.25 mm from upper left corner UL
mass = 67123 kg/m

Cross sectional stiffness components:
A = 2.6849E7 mm^2
Iyy = 15.472E13 Nmm^2
Izz = 18.977E13 Nmm^2
Iyz = 2.409E13 Nmm^2

Principal data:
alpha= 63.0 degrees
EI1 = 20.204E13 Nmm^2
EI2 = 14.246E13 Nmm^2





Output Lowered sill beam

BITMAP CROSS SECTIONAL ANALYSER RESULTS : Dorpelbalk - lowered.bmp

Sectional boundaries:
width = 320 equals to 8735 mm
height = 123 equals to 3825 mm

material properties:

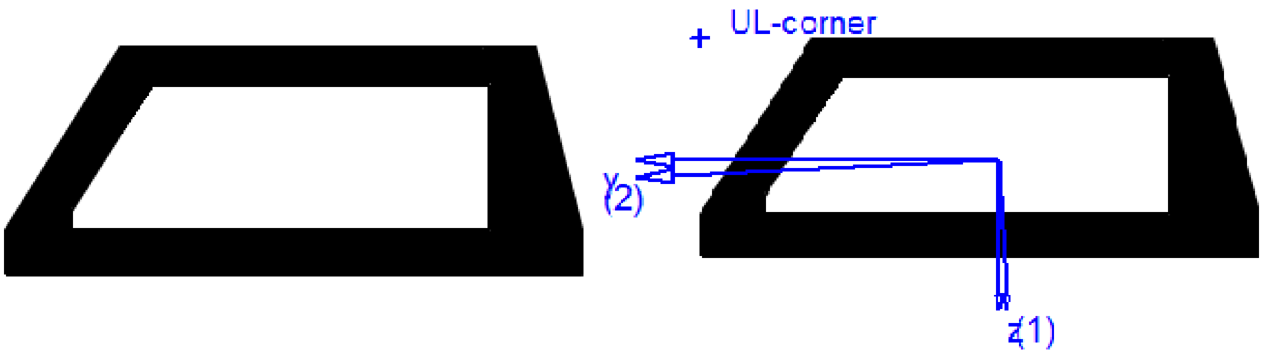
name	color	E [N/mm^2]	rho [kg/m^3]
Concrete	clBlack	1	2500

normal center NC at position:
hor = 4667.77 mm from upper left corner UL
ver = 2114.63 mm from upper left corner UL

mass center MC at position:
hor = 4667.77 mm from upper left corner UL
ver = 2114.63 mm from upper left corner UL
mass = 40563 kg/m

Cross sectional stiffness components:
EA   = 1.6225E7 N
EIy = 2.890E13 Nmm^2
EIz = 11.506E13 Nmm^2
EIyz = 0.408E13 Nmm^2

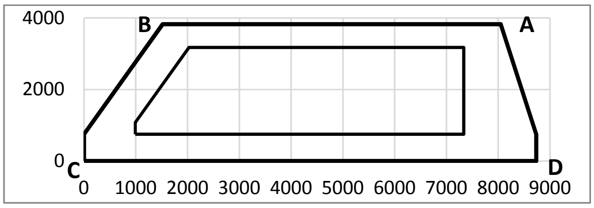
Principal data:
alpha= 2.7 degrees
EI1 = 11.525E13 Nmm^2
EI2 = 2.871E13 Nmm^2



Herberekening dorpelbalk

Hoofdafmetingen

L	33,0	m
H_t	3825	mm
b_{max}	8735	mm
A_c	16,2	m^2

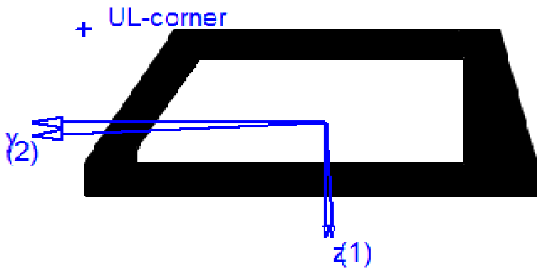


Berekening normaalkrachtencentrum (NC) t.o.v. UL-corner

Nc_y	4667,77	mm
	2114,63	mm

*Eigenschappen doorsnede in hoofdrichting: *)*

α	2,7	°
I_1	1,15E+14	mm^3
I_2	2,87E+13	mm^3



*Eigenschappen doorsnede *)*

I_{yy}	2,890E+13	mm^3
I_{zz}	1,151E+14	mm^3
I_{yz}	4,080E+12	mm^3

*) De eigenschappen van de doorsnede zijn op basis van Bitmap Cross Sectional Analyser bepaald.

Bitmap Cross Sectional Analyser is een door Hans Welleman ontwikkeld programma.

Aanwezige voorspanning

strand	19	-
$Ø_{strands}$	15,7	mm
A_{strand}	139	mm^2

Kabel Nr.		Voorspankracht	A	Kabel Nr.		Voorspankracht	A [mm^2]
16	1089	1296,3 N/mm^2	2641	15	1083	1289 N/mm^2	2641
6	1103	1313,6 N/mm^2	2641	13	1083	1289 N/mm^2	2641
17	1089	1296,3 N/mm^2	2641	11	1083	1289 N/mm^2	2641
				10	1083	1289 N/mm^2	2641
19	1098	1306,7 N/mm^2	2641	9	1083	1289 N/mm^2	2641
8	1098	1306,7 N/mm^2	2641	7	1083	1289 N/mm^2	2641
30 **)		907,41 N/mm^2		35 **)		907 N/mm^2	
31 **)		907,41 N/mm^2		33 **)		907 N/mm^2	
32 **)		907,41 N/mm^2					
34 **)		907,41 N/mm^2		1089			

Totaal	Pm;0	**37650,1 kN**
	Atot	29051

**) De spanning in deze strengen is maar voor 70% meegenomen, omdat de strengen tijdens de bouw hebben gediend om krimpspanningen op te vangen.

Estimated loss		0,16 (De voorspanverliezen zijn in hoofdstuk 6 bepaald op 16%)	
	Pm;inf	**31626,1 kN**	34788,6887

Materiaaleigenschappen

Beton

C30/37	37	XC4
γ_c	1,5	-
	1,18	-
f_{cd}	20	N/mm^2
f_{ctm}	2,90	N/mm^2
ε_{c3} [‰]	1,75	-
ε_{cu3} [‰]	3,50	-
$E_{cm(0)}$	32837	N/mm^2
ρ_c	25	kN/m^3

Staal

FeP 1770		
f_{pk}	1770	N/mm^2
γ_s	1,10	-
$f_{p0.1k}$	1593	N/mm^2
f_{pd}	1448	N/mm^2
σ_{pmo}	1328	N/mm^2
E_p	195000	N/mm^2

Maatgevende snedekrachten & krachtswerking

Momenten t.g.v. belasting

$M_{xx;G}$	0	kNm
$M_{zz;G}$	59214	kNm
$M_{xx;Q}$	28722	kNm
$M_{zz;Q}$	-36482	kNm

Momenten t.g.v. voorspanning

$M_{xx;p}$	-10538	kNm
$M_{zz;p}$	-26734	kNm

Momenten t.g.v. voorspanning t=inf

$M_{xx;p}$	-8852	kNm
$M_{zz;p}$	-22457	kNm

Normaalkracht t.g.v. voorspanning

N (t=0)	-37650	kN
N (t=inf)	-31626	kN

Bepalen spanningverloop over de doorsnede

EA	5,31952E+11	N
EIyy	9,48977E+17	Nmm^2
EIzz	3,778E+18	Nmm^2
EIyz	1,340E+17	Nmm^2

EI-matrix

$$\begin{bmatrix} EI_{yy} & EI_{yz} \\ EI_{zy} & EI_{zz} \end{bmatrix} \qquad \begin{bmatrix} 9{,}49E+17 & 1{,}34E+17 \\ 1{,}34E+17 & 3{,}78E+18 \end{bmatrix}$$

$(\text{EI-matrix})^{-1}$

$$\begin{bmatrix} 1{,}06E-18 & -3{,}76E-20 \\ -3{,}76E-20 & 2{,}66E-19 \end{bmatrix}$$

$$\begin{bmatrix} \kappa_y \\ \kappa_z \end{bmatrix} = \begin{bmatrix} EI_{yy} & EI_{yz} \\ EI_{zy} & EI_{zz} \end{bmatrix}^{-1} \begin{bmatrix} M_{yy} \\ M_{zz} \end{bmatrix} \qquad \begin{bmatrix} \kappa_y \\ \kappa_z \end{bmatrix} = \underset{\text{Permanent load}}{\begin{bmatrix} -2{,}22E-09 \\ 1{,}58E-08 \end{bmatrix}} \quad \underset{\text{Variable load}}{\begin{bmatrix} 3{,}18E-08 \\ -1{,}08E-08 \end{bmatrix}} \quad \underset{\text{Prestressing}}{\begin{bmatrix} -1{,}02E-08 \\ -6{,}72E-09 \end{bmatrix}}$$

$\sigma(y,z)$ = N/A + E*(κ_y * y + κ_z * z)					Algemene formule
$\sigma(y,z)$ =	-7,30E-05	y	5,17E-04	z	1) Equation permanent load at t=0
$\sigma(y,z)$ =	1,04E-03	y	-3,54E-04	z	2) Equation variable load at t=0
$\sigma(y,z)$ =	-3,34E-04	y	-2,21E-04	z	3) Equation prestressing at t=0

Bepalen spanningverloop over de doorsnede

EA	5,31952E+11	N
EIyy	9,48977E+17	Nmm^2
EIzz	3,778E+18	Nmm^2
EIyz	1,340E+17	Nmm^2

EI-matrix

$$\begin{bmatrix} EI_{yy} & EI_{yz} \\ EI_{zy} & EI_{zz} \end{bmatrix} \qquad \begin{bmatrix} 9{,}49E+17 & 1{,}34E+17 \\ 1{,}34E+17 & 3{,}78E+18 \end{bmatrix}$$

$(\text{EI-matrix})^{-1}$

$$\begin{bmatrix} 1{,}06E-18 & -3{,}76E-20 \\ -3{,}76E-20 & 2{,}66E-19 \end{bmatrix}$$

$$\begin{bmatrix} \kappa_y \\ \kappa_z \end{bmatrix} = \begin{bmatrix} EI_{yy} & EI_{yz} \\ EI_{zy} & EI_{zz} \end{bmatrix}^{-1} \begin{bmatrix} M_{yy} \\ M_{zz} \end{bmatrix} \qquad \begin{bmatrix} \kappa_y \\ \kappa_z \end{bmatrix} = \underset{\text{Permanent load}}{\begin{bmatrix} -2{,}22E-09 \\ 1{,}58E-08 \end{bmatrix}} \quad \underset{\text{Variable load}}{\begin{bmatrix} 3{,}18E-08 \\ -1{,}08E-08 \end{bmatrix}} \quad \underset{\text{Prestressing}}{\begin{bmatrix} -8{,}53E-09 \\ -5{,}64E-09 \end{bmatrix}}$$

$\sigma(y,z)$ = N/A + E*(κ_y * y + κ_z * z)					Algemene formule
$\sigma(y,z)$ =	-2,80E-04	y	-1,85E-04	z	3) Equation prestressing at t=inf

De totale spanning in de doorsnede is de som van de spanning door buiging en de spanning door normaalkracht. In de tabel hieronder zijn de spanningen samengevat

At t=0

	$E*\kappa_y$	$*y$	$E*\kappa_z$	$*z$	1	2	3		N/A		σ(y,z) G	G+Q
A	-7,30E-05	3380,23	5,17E-04	-1710,37	-1,13	4,13	-0,75	N/mm^2	-2,32	N/mm^2	-4,21	-0,07
B	-7,30E-05	-3147,77	5,17E-04	-1710,37	-0,65	-2,68	1,43	N/mm^2	-2,32	N/mm^2	-1,55	-4,23
C	***-7,30E-05***	***-4667,8***	***5,17E-04***	***2114,63***	***1,43***	***-5,62***	***1,09***	***N/mm^2***	***-2,32***	***N/mm^2***	***0,20***	***-5,42***
D	-7,30E-05	4067,23	5,17E-04	2114,63	0,80	3,50	-1,82	N/mm^2	-2,32	N/mm^2	-3,35	0,15

At t= ∞

	$E*\kappa_y$	$*y$	$E*\kappa_z$	$*z$	σ(y,z)				N/A		σ(y,z) G	G+Q
A	-7,30E-05	3380,23	5,17E-04	-1710,37	-1,13	4,13	-0,63	N/mm^2	-1,95	N/mm^2	-3,71	0,42
B	-7,30E-05	-3147,77	5,17E-04	-1710,37	-0,65	-2,68	1,20	N/mm^2	-1,95	N/mm^2	-1,41	-4,09
C	**-7,30E-05**	**-4667,8**	**5,17E-04**	**2114,63**	**1,43**	**-5,62**	**0,92**	**N/mm^2**	**-1,95**	**N/mm^2**	**0,40**	**-5,22**
D	-7,30E-05	4067,23	5,17E-04	2114,63	0,80	3,50	-1,53	N/mm^2	-1,95	N/mm^2	-2,69	0,81

Controle weerstand buigend moment

De totale weerstand tegen buigen is berekent met behulp van het programma MN-Kappa. De uitdraai van de berekening is op de volgende pagina weergegeven. Onderstaand zijn de resultaten samengevat

$M_{ED;xx}$ = 38540 kNm
$M_{ED;xx}$ = -5673 kNm
$M_{ED;tot}$ = 68907,5 kNm
M_{RD} = 141375,7 kNm

$M_{ED} \leq M_{RD}$ OK

Check wether the depth of the concrete compressionzone meets the requirements on the rotational capacity

Xu = 1357,5
d = 5007

$$f = \frac{((f_{pk}/\gamma_s)-\sigma_{pm;\infty})*A_p+f_{yd} * A_s}{A_p+A_s} = 520 \text{ N/mm}^2$$

Eis:

$$\frac{X_u}{d} \leq \frac{500}{500+f} = 0,49 \qquad \frac{X_u}{d} = 0,27 < 0,49 \text{ OK}$$

Project : MSc. Thesis - T. van der Aart
Onderdeel : SIll beam check ULS/crack width
Dimensies : kN;m;rad (tenzij anders aangegeven)
Datum : 04/10/2014
Bestand : C:\Users\905981\Box Sync\My Box\MSc.Thesis\cross section check.mnk
Referentieperiode: 50 jaar

Toegepaste normen volgens Eurocode met Nederlandse NB

Beton	NEN-EN 1992-1-1:2005	C2:2010	NB:2011(nl)

Invoer

Geometrie

Elementtype : Balk
Scheve buiging : Ja
Treksterkte fctm,fl : Nee
Doorsnedevorm : 29:Veelhoek

Oppervlak: 1 Betonkwaliteit: C30/37

nr.	y-coörd. [mm]	z-coörd. [mm]
1	0.0	750.0
2	1698.4	3875.0
3	8030.1	3875.0
4	8735.0	750.0
5	8735.0	0.0
6	0.0	0.0

Oppervlak: 2 Betonkwaliteit: C30/37

nr.	y-coörd. [mm]	z-coörd. [mm]
1	7335.0	3225.0
2	2144.0	3225.0
3	991.0	1076.0
4	991.0	750.0
5	7335.0	750.0

Doorsnedegrootheden

Grootheden exclusief wapening

A_b = 15630557 mm^2

z_z = 1717.3 mm y_z = 4690.0 mm

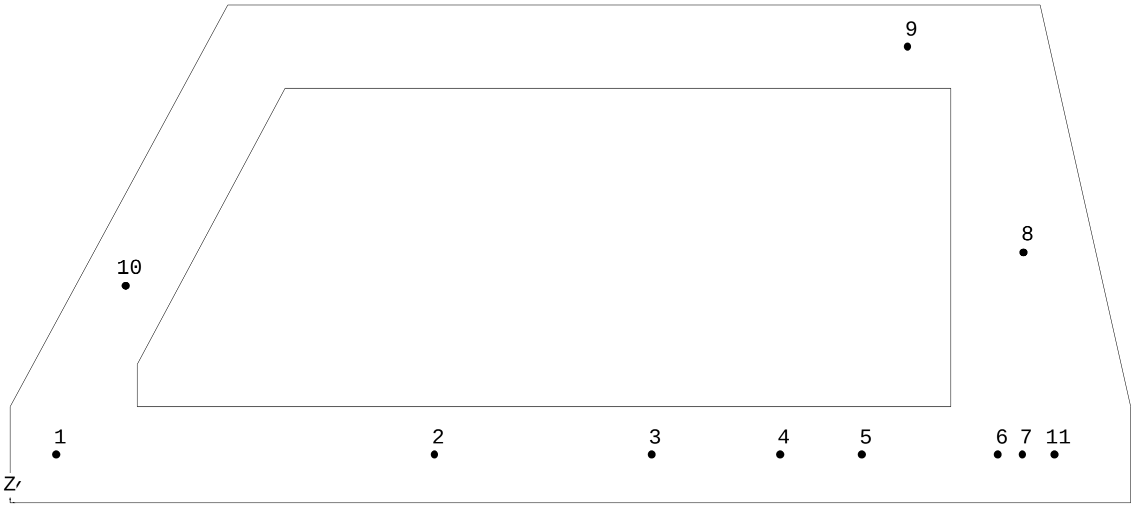
I_y = 29213487078507 mm^4 I_z = 113847623542714 mm^4

Wapening

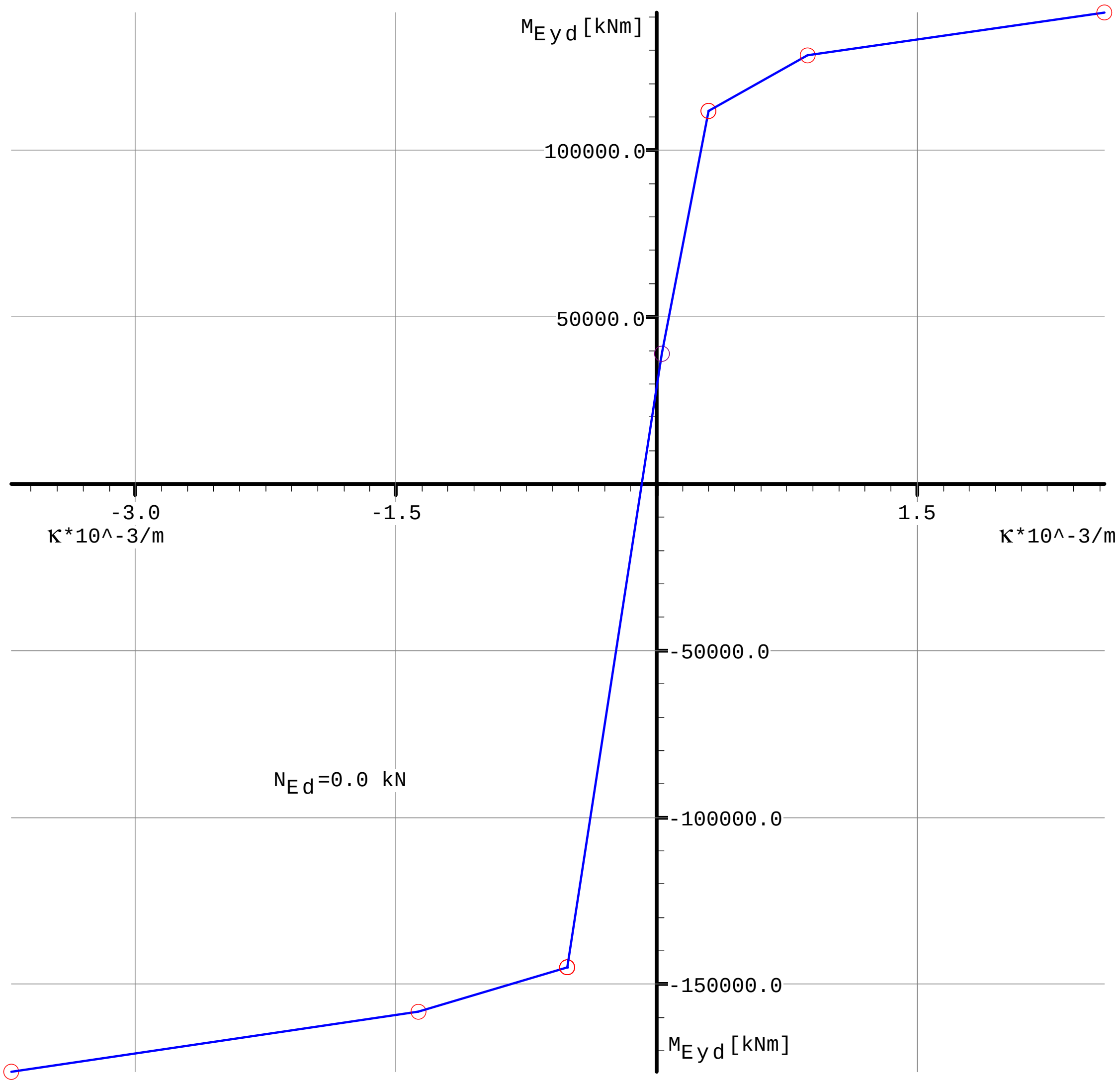
nr.	y-coörd. [mm]	z-coörd. [mm]	Diameter [mm]	Staalkwaliteit	Voorspanning [N/mm^2]
1	360.0	375.0	58.0	Y1770C	1089.0
2	3307.0	375.0	58.0	Y1770C	1083.0
3	5000.0	375.0	58.0	Y1770C	1083.0
4	6000.0	375.0	58.0	Y1770C	1083.0
5	6640.0	375.0	58.0	Y1770C	1083.0
6	7700.0	375.0	58.0	Y1770C	1083.0
7	7890.0	375.0	58.0	Y1770C	1098.0
8	7900.0	1950.0	58.0	Y1770C	1098.0
9	6994.0	3550.0	58.0	Y1770C	1089.0
10	900.0	1691.0	58.0	Y1770C	1089.0
11	8140.0	375.0	58.0	Y1770C	1083.0

Project : MSc. Thesis - T. van der Aart
Onderdeel : SIll beam check ULS/crack width

Invoer Grafisch



MN-Kappa-diagram -Sterkte-



Project : MSc. Thesis - T. van der Aart
Onderdeel : SIll beam check ULS/crack width
N_{Ed} = 0.000 kN hoek = -81.6 graden

Punt			z' [mm]	Δε [o/oo]	σ [N/mm²]	x [mm]	ε t.p.v. bovenkant [o/oo]	ε t.p.v. onderkant [o/oo]
1:		breekt	-6860.5	26.217	1580.9	768.4	-3.07076	29.02572
2:	C30/37	vloeit	556.7	-1.750	-20.0	1176.8	-1.75000	10.18504
3:		vloeit	-3819.3	1.428	1381.8	2067.7	-1.17824	3.38102
4:		vloeit	-3819.3	1.428	1381.8	2067.7	-1.17824	3.38102
5:		vloeit	-3819.3	1.428	1381.8	2067.7	-1.17824	3.38102
6:		vloeit	-3819.3	1.428	1381.8	2067.7	-1.17824	3.38102
7:	M_{Ed}=38955.288		-7428.1	-0.316	0.0	8022.7	-0.04670	-0.31550
8:		vloeit	-70.1	1.428	1381.8	3214.5	1.55275	-1.00441
9:		vloeit	-70.1	1.428	1381.8	3214.5	1.55275	-1.00441
10:	C30/37	vloeit	-8398.0	-1.750	-20.0	1999.8	5.77947	-1.75000
11:	C30/37	breekt	-8604.5	-3.500	-20.0	1357.5	19.01761	-3.50000

Punt			M_{yd} [kNm]	κ [10^{-3}/m]	EI [kNm²]	d [mm]	z [mm]	Voorwaarde
1:		breekt	-176370.9	-3.715	41020450	5865.8	5536.7	$_e$s=$_{eud}$t
2:	C30/37	vloeit	-158309.1	-1.370	98101926	5801.0	5373.9	$_e$c=$_{ec}$3
3:		vloeit	-144917.2	-0.514	229400604	5904.3	5237.3	eσ$_{=ey}$dt
4:		vloeit	-144917.2	-0.514	229400604	5904.3	5237.3	eσ$_{=ey}$dt
5:		vloeit	-144917.2	-0.514	229400604	5904.3	5237.3	eσ$_{=ey}$dt
6:		vloeit	-144917.2	-0.514	229400604	5904.3	5237.3	eσ$_{=ey}$dt
7:	M_{Ed}=38955.288		38955.3	0.031	1067258401	703.3	633.0	Fundamenteel
8:		vloeit	111877.3	0.298	341379286	7184.1	5821.7	eσ$_{=ey}$dt
9:		vloeit	111877.3	0.298	341379286	7184.1	5821.7	eσ$_{=ey}$dt
10:	C30/37	vloeit	128565.3	0.867	145630344	5701.1	4764.2	eχ$_{=e}$c3
11:	C30/37	breekt	141375.7	2.577	54813741	5007.0	4307.9	$_e$c=$_{ecu}$3

Sterkte

Art. 6.1 - Eurocode EN 1992-1-1
N_{Ed} = 0.0 kN
$M_{Ey;d}$ = 5673.0 kNm $M_{Ez;d}$ = 38540.0 kNm
$M_{Ed}+M_{pw}$ = (38955.3+ 29952.2) = | 68907.5 kNm| < M_R = |141375.7 kNm| voldoet

Minimum wapening

Art. 9.2.1.1 - Eurocode EN 1992-1-1
f_{ctm} = 2.9 N/mm² σ_{cp} = -0.0 N/mm²
$M_{E,min}$ = 71419.9 kNm $N_{E,min}$ = 0.0 kN
$A_{s,min1}$ = 0.0 mm2 $A_{s,min2}$ = 9907.8 mm2
$A_{s,min}$ = 0.0 mm2
M_R =|141375.7 kNm| > (M_{Ed} = | 68907.5 kNm|) voldoet

Vertical stability check

Loads

$q_{G;s.w.}$	397	kN/m^1
$q_{G;sand}$	238	kN/m^1
$q_{Q;v}$	-557	kN/m^1
$q_{Q;v;rock}$	100	kN/m^1

Load factors

g_g	0,90	g_q	1,10

Controle

	662	kN/m^1	V_up	-612	kN/m^1

U.C.	0,93	Ok!

C

Contribution parts to the cross sectional reduction

In this Appendix a calculation is made on the contribution of the different parts of the barrier to the total reduction of the cross sectional area (62100 m^2)

Table C.1: Calculation of the contribution to the cross sectional reduction

	A		Amount	A_{tot}	% of total
	l [m]	b [m]		[m^2]	
Sill beams	35	8	62	16864	27%
Foundation bed	35	4	62	8680	14%
Sill construction	35	0.5	62	1085	2%
Piers 1)	282.5		65	18363	30%
Solid barrier 2)				17109	28%
					100%