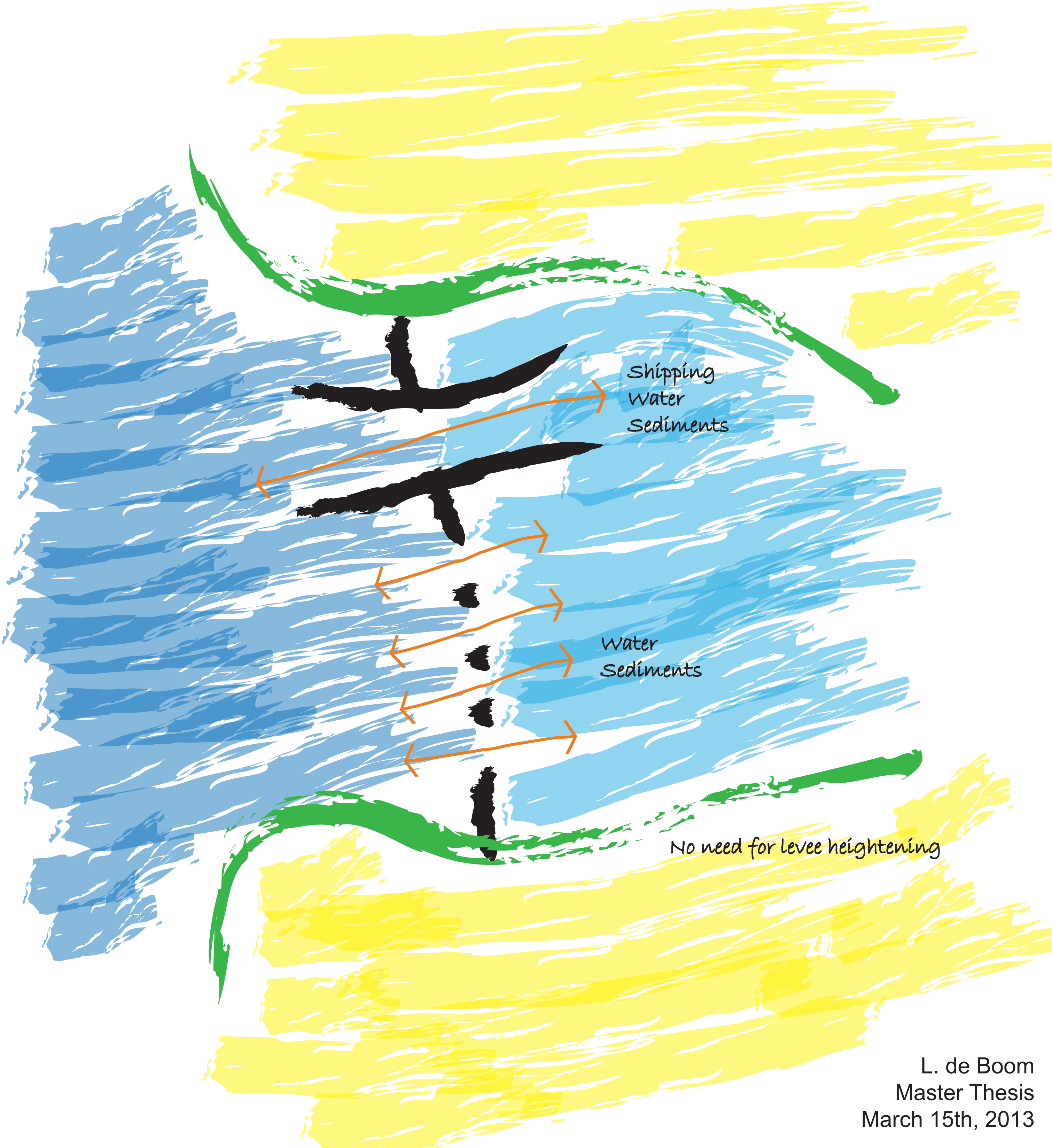
Reduction Barrier

A cost effective solution for safety, shipping and the environment?
Case Study for the Western Scheldt



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Master Thesis
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Remark
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Preface

This Master Thesis is part of the Master Hydraulic Engineering with specialisation Hydraulic Structures at the faculty of Civil Engineering and Geosciences of Delft University of Technology. This thesis has been developed in cooperation with Witteveen+Bos and was initiated in August 2012.

This thesis has been written for anyone who is interested in the subject. A lot of effort has been put into the readability of the thesis, some basic technical knowledge is however required.

Readers who are interested in the theoretical background of a reduction barrier are referred to chapter 3. Those who are more interested in the Case Study are referred to chapters 4 through 6.

I would like to thank the members of the graduation committee for their guidance and support. Also, I want to express my gratitude to Witteveen+Bos for sharing their knowledge and facilities. I would like to thank my cousin Gwen for correcting my written English. Last but not least, I would like thank my family and friends for their support during the completion of my thesis.

Delft, March 2013
Lex de Boom

Summary

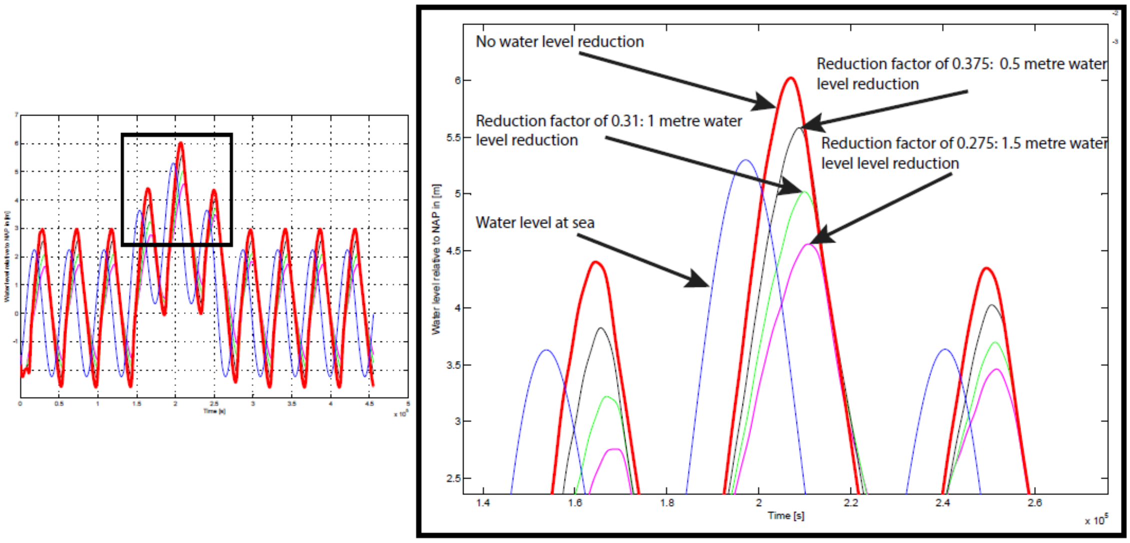
Flood protection of the land around an estuary has been important for the Netherlands since the floodings of 1953. A solution can be to improve the levees around the estuary. This can lead to expensive levee heightening over a great length. An advantage will be that the estuary itself will not be harmed. However the construction of a levee in urban areas can have a large impact. Another option is to construct a dam in the estuary. This dam can be often much shorter than the required levee length which can reduce costs. The downsides are that the dam is an obstacle for vessels and it will cause environmental changes. The environmental changes can be reduced by constructing a, semi-open, storm surge barrier, like the Eastern Scheldt barrier, instead of a dam. However, the barrier remains an obstacle for shipping traffic and the moveable parts make the storm surge barrier expensive.

Another option is a reduction barrier. This was one of the options for the Eastern Scheldt barrier, but at that time a storm surge barrier turned out to be better. A reduction barrier can provide safety by introducing additional resistance in the estuary. This can reduce the amplitude of the tide in the estuary. The reduction barrier itself can be described as a dam with some openings in it. Water can still flow in and out of the estuary which is important for both the environment and shipping traffic. The flow velocity through the shipping openings must remain acceptable for the passage of vessels.

The design water level consists of a combination of regular tide and a storm surge. The tidal wave has a smaller timescale than a storm surge wave. This makes the tidal wave much easier to reduce than the storm surge wave, since slow motions are more difficult to dampen. The reduction barrier is more effective on reducing the tidal wave in estuaries where this wave is amplified due to the shape of the estuary. The reduction of the tidal wave means that the reduction of the water level maximum is limited, since the storm surge wave is not reduced.

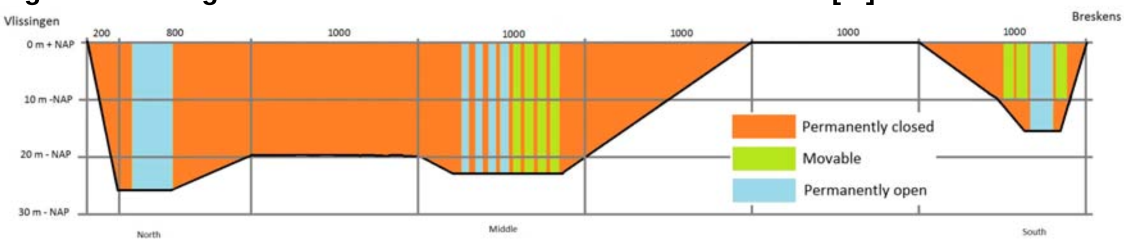
The analysis revealed that the Western Scheldt is a suitable location for a reduction barrier. The Western Scheldt requires a relatively small reduction in water level maximum, due to the relatively high quality of the levees. Belgium requires a reduction of the water level maximum of 0.5 metre in the current situation near Antwerp. The reduction barrier can be adapted for up to 1 metre sea level rise, by installing moveable gates. See Figure S.1 for the effect on the water level reduction of closed gates during a storm surge. The red line represents the situation without a reduction barrier. The black line is the effect of the reduction barrier. The green and purple lines indicate the effect of the moveable gates. The reduction factor is the factor with which the discharge area is multiplied. So a reduction factor of 0.4 means that the barrier closes off 60% of the discharge area and 40% remains open.

Figure S. 1 Water level on the Western Scheldt near Antwerp during storm surge



The shipping traffic through the barrier is an important aspect for the feasibility of the barrier, since there are four ports located along the Western Scheldt. The barrier is located just east of the line Vlissingen - Breskens. The reduction barrier has two shipping channels. The main shipping channel is located near Vlissingen and the secondary shipping channel is located near Breskens. Vessels are able to pass through the barrier during normal conditions.
It is assumed that the environment is not harmed since 80% of the original tidal prism can be maintained. See Figure S.2 for longitudinal cross-section of the reduction barrier in the Western Scheldt. The orange part is the permanently closed dam. The blue parts are always open and the green parts can be closed off by gates. The combination of blue and green areas correspond with a reduction factor of 0.375.

Figure S. 2 Longitudinal cross-section of the reduction barrier [m]



The barrier is mainly composed of concrete caissons and rubble mound. The construction costs of the reduction barrier are an estimated 4 billion euros, this is comparable with the costs of the Eastern Scheldt barrier and also comparable with the costs of levee heightening.
A reduction barrier in the Western Scheldt is technically feasible. For further research is recommended to make models of the water motion and sediment transport in 3-D. Fast and real-time shipping simulations can be used to optimize the shipping openings. The impact on the environment is another subject for further study as well.

Figure S. 3 Top view reduction barrier in the Western Scheldt
(Left is Vlissingen, right is Breskens)

Samenvatting

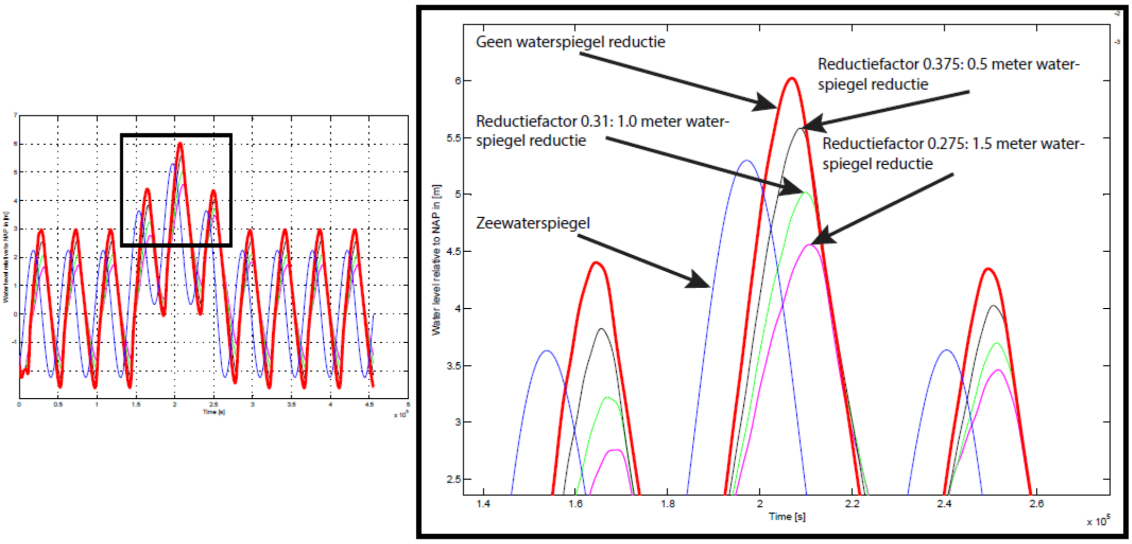
De bescherming tegen hoogwater van het land rondom een estuarium is een belangrijk onderwerp voor Nederland, vooral sinds de overstromingen van 1953. Een oplossing kan zijn om het land te beschermen met dijken. Dit kan leiden tot dijkverhogingen over een grote lengte, wat erg kostbaar is. Een voordeel is dat het estuarium zelf niet veranderd wordt door de maatregel. Een andere mogelijkheid is om de het estuarium af te sluiten met een dam. Een dam is vaak veel korter dan de dijk lengte rondom het estuarium, wat kan zorgen voor kostenbesparingen. De nadelen van een dam zijn dat de dam een obstakel is voor scheepvaart en dat het milieu in het estuarium zal veranderen. De milieuverandering kan worden voorkomen door het bouwen van een stormvloedkering in de plaats van een dam. Een stormvloedkering blijft daarentegen wel een obstakel voor de scheepvaart en de beweegbare delen van de stormvloedkering maakt deze duur bouw en onderhoud.

Een andere mogelijkheid is een reductiekering. Dit was een van de mogelijkheden voor de Oosterscheldekering, maar uiteindelijk bleek een stormvloedkering beter te zijn. Een reductiekering kan waterveiligheid creëren door extra weerstand in het estuarium te introduceren. Dit kan de getij-amplitude in het estuarium reduceren. De reductiekering kan omschreven worden als een dam met een aantal openingen erin. Water kan nog steeds in en uit het estuarium stromen. Dit is belangrijk voor zowel het milieu als de scheepvaart. De stroomsnelheid moet relatief laag blijven om scheepvaart mogelijk te maken.

De maatgevende waterstand bestaat uit een combinatie van getij en stormopzet. De getijgolf heeft een kleinere tijdschaal dan de stormopzet. Dit maakt de getijgolf makkelijker te dempen dan de stormopzet, omdat langzame bewegingen moeilijker te dempen zijn. Het is voor een reductiekering effectiever om het getij te dempen, vooral in estuaria waar de getijgolf door het beken wordt versterkt. Het reduceren van de getijgolf impliceert dat het effect op de maximale waterstand beperkt is.

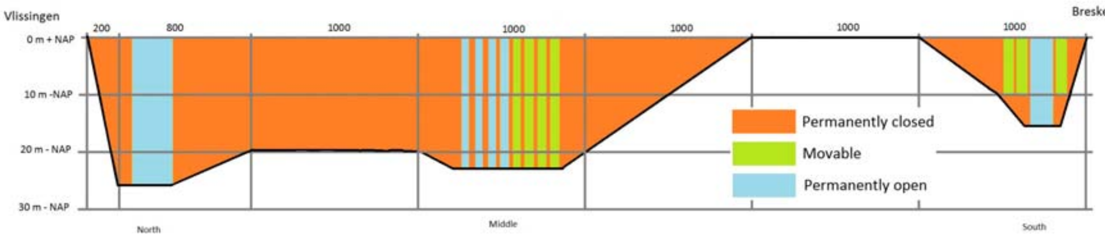
Uit de analyse kwam naar voren dat de Westerschelde een geschikte locatie is om een reductiekering toe te passen. De Westerschelde heeft maar een kleine reductie van de maximale waterstand nodig, omdat de kwaliteit van de dijken relatief hoog is. Voor Antwerpen is een reductie van 0.5 meter op de maximale waterstand noodzakelijk. De ontworpen reductiekering is aanpasbaar aan de zeespiegelstijging, door beweegbare deuren te installeren. Zie Figuur S.1 voor het effect van de deuren op de maximale waterspiegel. De rode lijn geeft de huidige waterstand weer, dus de situatie zonder kering. De zwarte lijn is het effect van de vaste kering. De groene en de paarse lijn zijn het effect van de beweegbare deuren. De reductiefactor is de factor waarmee het huidige doorstroomoppervlak vermenigvuldigd moet worden. Een reductiefactor van 0.4 betekent dat de reductiekering 60% van het huidige doorstroomoppervlak afsluit en 40% open laat.

Figuur S. 1 Waterspiegel op de Westerschelde bij Antwerpen tijdens storm



De scheepvaart door de kering is een belangrijk aspect voor de haalbaarheid van de reductiekering. Er bevinden zich vier havens aan de Westerschelde. De kering bevindt zich net ten oosten van de lijn Vlissingen - Breskens. De reductiekering heeft twee scheepvaart openingen. De hoofdscheepvaartopening bevindt zich vlakbij Vlissingen en andere scheepvaartopening ligt aan de zuidzijde bij Breskens. De schepen zijn instaat om de kering te passeren in normale condities.
Zie Figuur S.2 voor een langsdoorsnede van de kering in de Westerschelde. De oranje gedeeltes zijn altijd gesloten. De blauwe vlakken blijven altijd open en de groene stukken zijn afsluitbaar met verticale schuiven. De combinatie van blauw en groen correspondeert met een reductiefactor van 0.375.

Figuur S. 2 Langsdoorsnede van de reductiekering [m]



De reductiekering bestaat uit betonnen caissons en stortsteen. De aanlegkosten bedragen 4 miljard euro, wat vergelijkbaar is met de kosten voor zowel de Oosterscheldekering als dijkverhoging rondom de Westerschelde.
Een reductiekering in de Westerschelde is technisch haalbaar. Verder onderzoek naar de waterbeweging en sediment transport in 3-D wordt aanbevolen. Scheepvaart simulaties kunnen uitgevoerd worden om het ontwerp te verfijnen en om kapiteins te trainen. De invloed van de reductiekering op het milieu moet eveneens verder onderzocht worden.

Figuur S. 3 Bovenaanzicht reductiekering in de Westerschelde (Links is Vlissingen, rechts is Breskens)

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Glossary, List of Symbols

Glossary

Term	Explanation
Amplitude	The maximum difference with the average water level
Discharge area	The cross-sectional area of the connection between the estuary and the sea
Jetty	Piled structure for berthing; Steiger
Tidal Prism	The volume of water that is transported in or out of the estuary during the course of the tide

List of symbols

Symbol	Term	Explanation
A_e	Surface area of the water in the estuary	
A_d	Discharge plane of the opening of the estuary	
A_s	Cross-sectional area of the channels in the estuary	
B	Estuary width	
B_s	Channel width in estuary	
d	Channel depth	
k0	Wave number	The wave number is defined as: $2\pi/L$
L	Wave length	
M2	Principal lunar semi-diurnal tide	The main tide caused by the moon, with a period of 12 hours and 25 minutes
Mu	Reduction factor for the channel cross-section	
NAP	Normaal Amsterdams Peil, in English: Amsterdam Ordnance Datum	The Dutch reference level which is equal to average sea level
Q	Discharge	The volume of water which enters of leaves the estuary per unit of time
Zeta0	Amplitude of the water level in the estuary at the land side (1-D model)	
Zetae	Amplitude of the water level in the estuary	
Zetal	Amplitude of the water level in the estuary at the sea side (1-D model)	
Zetas	Amplitude of the water level at sea	
Zetasdak	Amplitude of the water level at sea in 1-D model	

1. THE CHALLENGE

Flood protection of estuaries and bays is a complicated subject all over the world. The subject is complicated since many, sometimes contradictive, aspects must be taken into account. For example; flood protection, nature, navigation, fresh water and lesure areas. The people who live along the estuaries want to be protected from flooding by the sea, but they also want to profit from the benefits of the estuary. An estuary can provide a ‘safe haven’ for vessels and an estuary can support a wide variety of wild life and diversity in vegetation. When protecting the people from floods, the most obvious solutions are the construction of levees or the construction of a dam in the estuary. These solutions have serious disadvantages. The dam will change the environment of the estuary since the sea is no longer able to shape it. The levee solution is often difficult to execute since people and structures will have to be relocated in order create the required space for the levees. The levees often have to be strengthened over a considerable length which can be seen as a disadvantage as well. The search for a solution that can provide safety while maintaining the environmental value was initiated in the Netherlands.

The Netherlands is familiar with the disadvantages of both solutions, since the implementation of the Delta Plan. The Netherlands have developed the Delta Plan as a response to the flooding in February 1953. The Delta Plan consists of the closure of the estuaries in the South West of the Netherlands and several additional measures. Work started on the smaller dams first in order to provide practice before starting on the large estuaries. The closure of the Eastern Scheldt was a turning point in the implementation of the Delta Plan. Environmentalist had argued that the closures of the other estuaries had resulted in great losses for the environment. The dams, which were part of the plan, had changed the environment from a dynamic, salty or brackish estuary into a stagnant fresh water lake, like the Haringvliet. Sailors and fishermen were other groups, besides the environmentalists, who were aginst the closure of the Eastern Scheldt. The Eastern Scheldt Barrier had to be different, since the Eastern Scheldt was considered to have a great environmental value. In order to please the opposition it was decided to close the Eastern Scheldt with a moveable barrier instead of the originally planned dam. The movable barrier was considered to be the best option for both safety and the environment.

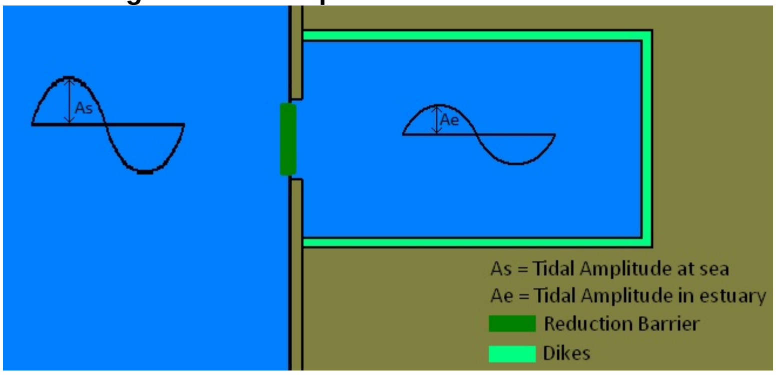
1.1. The idea

The Eastern Scheldt Barrier consists of moveable gates, see Figure 1.1. The estuary is closed during storm conditions and partly opened during normal conditions. This means, that during normal conditions, the water level in the Eastern Scheldt is influenced by the tide which is considered to be better for the environment. During normal conditions about 86% of the vertical tide is transmitted through the barrier, but only about 70% of the horizontal tide. The horizontal tide is another term for the discharge into and out of the estuary, and vertical tide is the water level fluctuation in the estuary. One of the options for an open barrier in the Eastern Scheldt was a reduction barrier. During the design it was investigated whether the gates were actually required, since the gates were very expensive in construction and maintenance. The reduction barrier will create resistance for the flow into the estuary, preferably without any moving parts since those are very expensive.

Figure 1.1 Eastern Scheldt Storm Surge Barrier (Šiman, 2008)

The reduction of the flow into the estuary reduces the amplitude of the water level elevation and can thus provide safety, see Figure 1.2. This reduction of the tidal wave height reduces the required height and strength of the levees surrounding the estuary. The construction of a reduction barrier can make increasing of the levee heights unnecessary. The idea is that the barrier is less expensive than increasing the levee height around the estuary. Such a barrier should be considered when a reltively large part of the levees in comparison to the barrier otherwise has to be raised.

Figure 1.2 Principle of the reduction barrier



There is a drive to re-introduce tide in the estuaries (Deltacommision, 2012), which were closed in the execution of the 'Delta Plan'. Apparently the Delta Plan has worked so well that the urge for safety has been exchanged to an urge for ecological development. The engineering challenge is to improve the ecology while maintaining or improving the safety level.

Sea level rise is expected to cause problems for the South West delta. The waters are closed off for sediment and therefore they will not naturally evolve with the rising water level. These arguments can be used to increase flexibility in the currently closed estuaries.

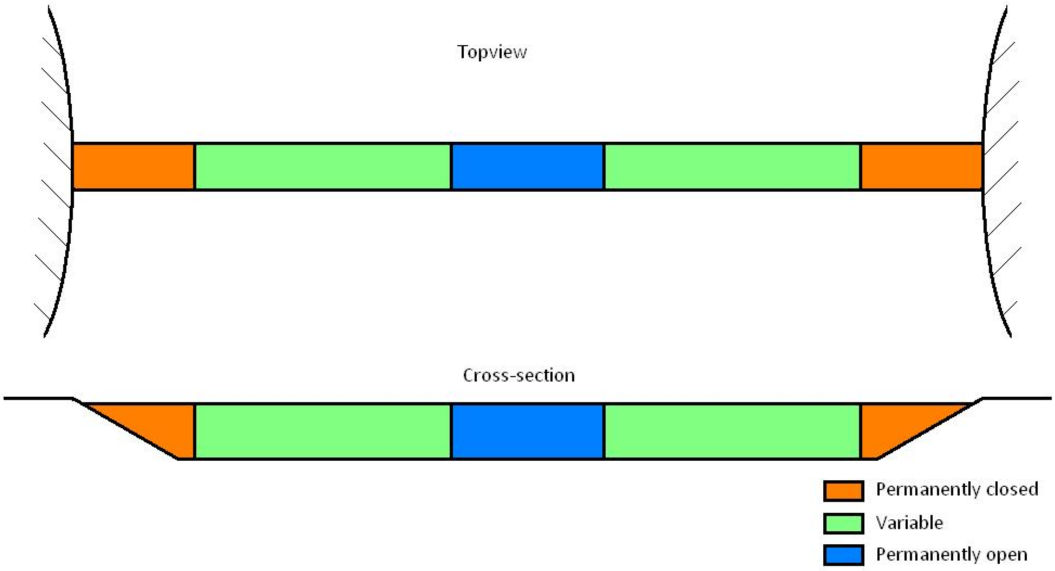
The reduction barrier should be tuned in such a way that the environmental impact is limited while additional safety during storm conditions is created. The tidal prism and water level elevation should remain relatively unharmed during normal conditions. The variable parts, if necessary, are closed only during storm conditions, see Figure 1.3. Preferably there are no moving parts at all, but moveable parts may be required to guarantee safety and to minimize the impact on the environment. The maintenance costs for the moveable parts are an import part of the total maintenance costs. The maintenance costs can be reduced by limiting the amount of moveable gates.

The transport of sediment is considered to be a problem for the Eastern Scheldt, since the sediment is not able to pass the barrier (Hoogduin, 2009). It is said that the Eastern Scheldt is open to allow the flow of water, but not to allow transport of sediments. The Eastern Scheldt is claimed to have a serious problem with the deficit of sediment due to this effect. Whether this is actually the case is questionable, the correctness of the statement is not further discussed in this thesis. A reduction barrier could have a deep opening, so there is the possibility in the design to create a barrier which is not a massive obstacle for sediment transport like the Eastern Scheldt Barrier is claimed to be. Sediment transport is considered to be important for the environment since sediments create tidal flats, these create suitable environments for different species. This means that the reduction barrier can possibly maintain the morphology while improving the flood protection. The Eastern Scheldt Barrier has taught us that the sill of the barrier is probably the cause of the reduction of sand transport between the sea and the estuary.

The open part of the barrier can be used, for example, for shipping if the flow velocity remains under a certain limit. This means that there is no need to construct complicated gates for the shipping lanes. This latter advantage can have a serious impact on the costs.

So there are several advantages for not completely closing the estuary. However these advantages do have consequences on other areas.

Figure 1.3 Concept of a reduction barrier, with closed, variable and open parts



1.2. Goal of this thesis

The goal of this Master Thesis is to investigate whether flood protection, environmental protection and shipping can go hand in hand at a competitive cost level by means of a reduction barrier.
This goal has to be achieved in two stages.

The first stage is to investigate the principle of a reduction barrier. Understanding of the principle requires insight in which parameters increase the feasibility of a reduction barrier. The goal is to develop a design chart which can be used to find a feasible location.

The second stage is to design a reduction barrier in a Case Study. The design of the reduction barrier is mainly focussed on the structural design. The Case Study gives information about the feasibility of a reduction barrier at a certain location. This should lead to a conclusion about the economic feasibility of a reduction barrier in comparison to other possible solutions. The Case Study is also used to determine whether it is possible for shipping traffic to navigate through the openings in the barrier. The planning and construction of the barrier are investigated as well.

1.3. Reader's guide

The first step in the investigation of the reduction barrier is to assess the different aspects which can influence the design of the barrier, these can be found in chapter 2. In order to investigate the principle of a 'reduction barrier', models are used. The third chapter of this Master Thesis consists of a description of the models and a discussion of the model results. The model results are used to select a proper location for the design of the reduction barrier as described in the second part of chapter 3. Chapter four contains an analysis of the selected location which results in the Terms of References. The design of the reduction barrier is the subject of chapter 5. Chapter 6 describes the construction method, gives an estimation of the costs and a preliminary planning. The seventh and final chapter contains the conclusions and recommendations about the feasibility of a reduction barrier in combination with environmental protection. Background information and more detailed results can be found in the appendices. The appendices also contain the detailed planning and drawings.

2. REDUCTION BARRIER

This chapter contains an analysis of the important aspects related to a reduction barrier. The first section discusses the main options for flood protection for an estuary on a system level. The content of the first two sections is a description of the advantages and disadvantages of the principle of the reduction barrier. The third section contains the outline of the subject which will be covered in this Thesis. Some subjects will be recommended for future research, since the thesis must be finished within a limited time frame.

2.1. Options for flood protection on system level

The flood protection of an estuary can be achieved in several different ways. The most rigorous solution is damming the estuary. The tide can no longer influence the water level in the estuary, which means that the hinterland is safe from flooding from the sea. The dam will have to have locks and sluices to let water and vessels in and out. The environmental value of the estuary changes into a fresh water system. This type of nature is considered to be less valuable, but the value of nature is difficult to put a price on. The value of the area can be described on an economic level by investigating the influence of public health for example.

The second option is the construction of storm surge barrier. Such a barrier closes off the estuary by means of moveable elements. An example of such a barrier is the storm surge barrier in the Eastern Scheldt. The shipping channels will have to be closed off by means of large gates like those used in the Maeslant barrier. In the event of a storm surge the barrier is closed to eliminate the thread of the storm surge, while in normal conditions the gates are open in order to provide the best circumstance for the environment. The construction and maintenance costs are very high due to large amount of moveable parts.

The third solution is not interfering in the estuary, but just increasing the levee height in order to guarantee safety. The shipping traffic and the environment are not influenced by this measure, but the levee works are extensive and expensive. The impact on the landscape can be considerable at certain locations, as are the costs.

The final, relatively new,option is a reduction barrier. A reduction barrier is a non moveable structure which reduces the tidal wave in order realise the desired safety level. The impact on the environment can be considerable since the tidal prism is reduced, by the barrier. This can cause a morphological instability, which means that the estuary will adapt its flow channels towards a stable situation. This can result in sedimentation in the channels, which is a problem with regard to the draft of the vessels. Although a reduction barrier is a non moveable structure, it is possible to reduce the opening of the barrier once the sea level rise becomes a thread. The construction and maintenance costs are expected to be lower since there hardly any moveable parts. The costs for the destruction of environment are also small due to the openness of the barrier.

2.2. Advantages of a reduction barrier

This thesis focuses on the reduction barrier. The main advantages of the reduction barrier are related to the open nature of the barrier. The tide can keep shaping nature behind the barrier under normal conditions. This is a major advantage for the development of nature in a special environmental area like an estuary. The open nature of the barrier enables the transport of sediments in and out of the estuary, which makes it possible for the estuary to adapt with the rising sea level by importing sediments. Estuaries can import sediments in order maintain a constant bed level with respect to the average sea level.

The length of a reduction barrier is smaller than the length of a barrier which can be closed completely since a part of the barrier remains open at all times. The smaller length can reduce costs. The opening in the barrier can enable vessels to pass the barrier during normal conditions more easily. Fish can always enter or leave the estuary. Due to the open connection with the sea, the water quality can improve and the salinity of the estuary is more natural.

A reduction barrier has preferably no moveable gates which makes it cheaper in construction and maintenance than a storm surge barrier.

2.3. Disadvantages of a reduction barrier

The major disadvantage of the reduction barrier is the openness. The estuary can still fill up with water which can lead to flooding of the land. The Eastern Scheldt Barrier can still function with a few gates failing, since the flow into the estuary can be stored by the large surface area behind the barrier. However a reduction barrier already uses this additional safety, therefore failure of part of the barrier is not allowed.

Another downside is the high flow velocity in the opening during storm conditions. These flow velocities can cause severe erosion which can threaten the stability of the barrier. The bed protection must be designed and executed very carefully, in order to prevent mistakes and serious damage. The scour holes in near the Eastern Scheldt barrier are much larger than was expected. This means that the bed protection requires serious attention.

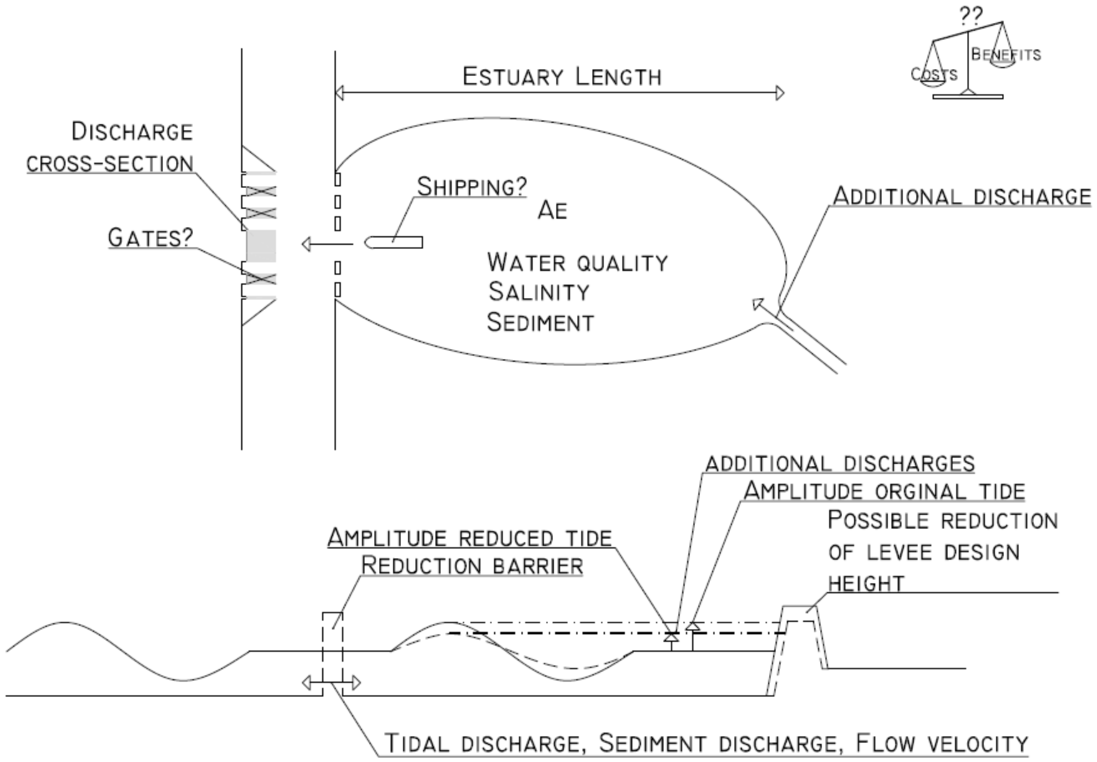
The impact on the environment is an important aspect of a barrier. The barrier is designed to have little impact on the environment, but there will always be some effect. The effects on the environment should be measured over a long period and should be put in historic perspective. The side effects of the closures in Zeeland are not well understood and are still under debate. The main effect of the barrier, flood protection, is well understood and satisfies the demands. The reduction barrier can cause morphological changes that may lead to changes in the position of the channels.

A reduction barrier was one of the options for the Eastern Scheldt storm surge barrier (finished in 1986), but it was abandoned because it failed to realise the desired results.

2.4. Subjects under investigation

The aspects which can be influenced by the barrier are essential for the final design of the barrier. A very long list of important subjects can be made, but some are more relevant than the others. The most relevant subjects are investigated in order to be able to draw relevant conclusions about the reduction barrier. Some aspects are governing for normal conditions and others for storm conditions. The selected subjects are used to select the best location for the Case Study. The important aspects are indicated in Figure 2.1.

Figure 2.1 Concept reduction barrier with relevant aspects



2.4.1. Tide in the estuary mouth

The tidal amplitude of the sea at the location of the estuary has impact of the feasibility of the reduction barrier. The larger the tidal amplitude, the larger the relative effect of the reduction of the discharge area. The water level behind the barrier is not able to adapt quickly to the water level changes at sea.

Locations with a semi diurnal tide are better suited for this solution than location with a diurnal tide, since the slow changes in water level are better able to penetrate the barrier. The faster the change in water level, the more effective the barrier due to the damping effect of the barrier.

2.4.2. Storm surge

The storm surge duration is important for the amount of reduction which can be obtained for the storm surge. For long storm surges it is more difficult to realise a reduction of the amplitude since the change in water level is very slowly and therefore the water level in the estuary can more easily follow the water level at sea, as mentioned in the previous subsection. The Delta commission has proposed to work with a storm surge duration of 35 hours (Deltaprogramma 2012). This value used to be 29 hours. However there are researchers who suggest that the storm surge duration should be even longer (Van der Westhuysen and others, 2009). The storm surge height is correlated with the design water level. The storm surge duration should be based on a analysis of the storm surge records, but for now a value of 35 hours is applied.

2.4.3. Additional discharge into the estuary

Discharge into the estuary will lead to a water level elevation when the barrier is ‘closed’. This means that the discharge into the estuary should be limited in comparison with the storage area. The source of the main discharge is of course the opening in the barrier. Additional discharges are caused by for example rivers, rain fall, pumping stations and locks. Rivers are most of the times the largest contributors. The river discharge caused by rainfall has some phase lack, but the duration of the phase lack depends on the situation. A large additional discharge would result in a low allowable discharge through the opening in the barrier, since the capacity of the estuary is exploited by the additional discharge.

2.4.4. Shipping through the barrier

Vessels need to be able to pass the barrier during normal conditions with minimal hindrance. The vessels require a certain width and depth to be able to navigate safely in the shipping channel. The shipping intensity determines whether the channel should be one-way or two-way traffic. The vessels can only pass the barrier when the flow velocity through the barrier is not very large and the water level gradient over the barrier may not be very steep either. The sea going vessels are difficult to manoeuvre, so the flow velocity must be limited. The exact limit is difficult to state, since there are no regulations on this subject. However there are locations in the world where these kinds of vessels can navigate in about 7 knots flow velocity. This value is used as an upper limit. The flow velocity and the steepness of the water level gradient should be subjects of further investigation when the reduction barrier appears to be feasible. This can be done by performing fast- and real-time simulations in order to locate problems in an early stage. The waiting time should be as small as possible for the vessels during storm conditions, since this results in high costs for both the operation of the vessels and the ports.

Vessels are assumed not to be able to pass the barrier during storm conditions, due to the increased flow velocities, the larger steepness, wind conditions and limited visibility. This means that the ports behind the barrier will experience loss when the barrier is closed. For the Port of Rotterdam has been calculated what the cost is of a closure of the Maeslant barrier. It turns out that the costs are about 1.3 million euro per day (Muntinga, 2009). These costs can be divided into waiting costs and additional wages for the shipping companies. There is no loss of reputation as long as the closures have a frequency of less than once per year. The costs are mainly dependent on the amount of cargo, so for smaller ports are the costs lower.

2.4.5. Environment in the estuary

The environment profits from minimal impact of the barrier during normal conditions on the flow patterns and the vertical and horizontal tide. An important aspect for the environment is the intrusion of tidal fluctuation, sediment and salt water. These aspects are only dealt with qualitatively, since the transport of sediments is not well understood and very complex to calculate. The effect on the environment is even more difficult. However there are some aspects which can be used to limit the impact of the barrier. For example, maintaining most of the tidal prism and designing a barrier with a low sill are useful measures to maintain the sediment transport capacity and therefore the environmental development.

The effect of the reduction of the vertical tide seems to have had little effect on the environment in the Eastern Scheldt. Therefore a reduction of the vertical tide in the order of magnitude as in the Eastern Scheldt is considered to be acceptable, 20-25%, since the tidal flats will probably still be submerged during high tide and dry during low tide. The range of the neap tide and spring tide is about 30% for the Western Scheldt. So the impact of the barrier should be less than the current difference between spring and neap tide. However for the horizontal tide a reduction factor of 0.8 is considered to be the limit. Large changes in the tidal prism can severely change the current situation, due to changes in the sediment transport. Since the development of the environment is not subject of this Thesis, the influence of the barrier is only described in quantities that could be of importance for the environment. The effect on the environment is subject to further investigations.

2.4.6. Risks induced by the barrier

The construction of a barrier in an estuary results in the creation of additional risks for vessels while reducing the flood risk for the land surrounding the estuary. The vessels can damage the barrier just before a storm surge, which, in turn, can lead to flooding of the land.

A full risk assessment can be performed once the solution of the reduction barrier proofs to be technically feasible. However the design of the barrier should be made in such a way that the possibility of flooding after a ship collision is small.

2.4.7. Levees surrounding the estuary

The levees surrounding the estuary must be able to resist the water level in the estuary during storm surge conditions. The strength and stability of the levee depends on many different loadings and parameters. For this study it is assumed that the overtopping is governing. This means that the levee height is investigated. The result of this investigation is the maximum water level in the estuary. This can be used to investigate the required reduction of the storm surge level. The other failure mechanisms of the levees are more complicated and require detailed information about the cross-section of the levees.

2.4.8. Length of the estuary

The length of the estuary has a very large impact on the feasibility of a reduction barrier. This length is formulated relative to the tidal wave length, which is dependent on the water depth and has a length of around 500-600 kilometre. For short estuaries, with a length of about $1/20^{th}$ of the tidal wave length, the water level in the estuary can be assumed horizontal. The behaviour of the tidal wave in the estuaries is more complicated in longer estuaries. The first thing to consider is the type of boundary condition at the land side of the estuary. The tidal wave can be reflected or the wave can travel land in and dampen out before getting the chance to reflect.

For estuaries with a certain length it is essential that there is no standing wave between the land in boundary and the barrier, since the barrier does not have any effect in that situation. So in long estuaries the location of the barrier must be selected carefully, otherwise the flood risk can be increased instead of reduced.

2.4.9. Shape of the opening

The shape of the opening in the barrier is of great importance for the design of the barrier. There are several types of solutions, for example the wide and shallow or narrow and deep. In case of the Eastern Scheldt barrier a compromise was made by designing a wide opening which follows the bed contour of the estuary. The wide and shallow opening is considered to be better for the flow patterns. In the other case the flow patterns are more concentrated in the channels and higher flow velocities at the bottom can be expected. However an advantage is that this opening can be used by vessels to pass the barrier in normal conditions. So when the barrier needs to be passed by vessels, the opening needs to be relatively narrow and deep. When there is no need for vessels to pass the barrier, the opening can be more shallow and wide because this leads to better conditions for the bed protection and flow patterns. In case sediment discharge through the barrier is expected to be important, the opening should be deep, since the sill can block sediment transport. Figure 1.3 is therefore a very simplified model of the barrier. It is likely that the openings are distributed more over the barrier, in order to prevent massive morphological changes, see Figure 2.2, Figure 2.3.

Figure 2.2 Barrier concept

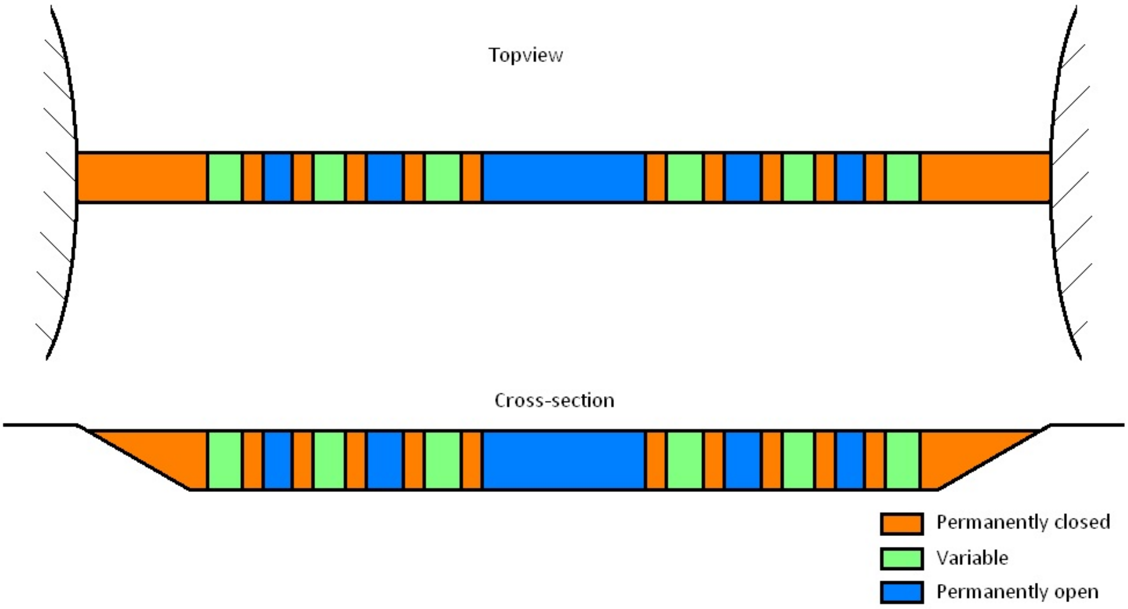
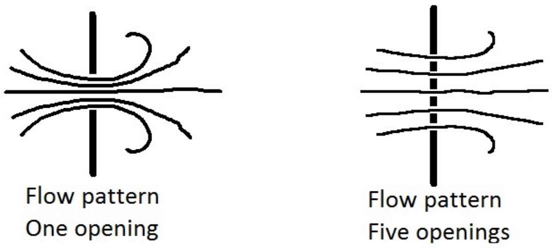


Figure 2.3 Impression impact of opening shape on flow pattern



3. A SUITABLE LOCATION FOR A REDUCTION BARRIER

The goal of this chapter is to a suitable location for the design of a reduction barrier for the Case Study. In order to do so, the effect of a reduction barrier on the water motion is studied first. The water motion is studied by using two models. The first model can be applied for situations with a short estuary length compared to the tidal wave length (1/20th) and the other model can be used for situation with a longer estuary length. The results of the calculations are used to select a suitable location for the case study, which is the subject of sections 3.7 through 3.9.

3.1. Definitions and applied boundary conditions

The symbols which are used in the calculations can be found in the Glossary on page xix. The most important variables can be found in Table 3.1.

Table 3.1 Applied parameters

Symbol	Dimensions	Term
A_e	m^2	Surface area of the estuary
A_d	m^2	Discharge cross-section at the sea side of the estuary
k0	1/m	Wave number
Mu	-	Reduction factor for the channel cross-section
Q	m^3/s	Discharge
Zetae	m	Amplitude of the water level in the estuary
Zetas	m	Amplitude of the water level at sea
Zetasdak	m	Amplitude of the water level at sea
Zeta0	m	Amplitude on the land side of the estuary
Zetal	m	Amplitude on the estuary side of the barrier

When talking about discharge areas, the area is defined as the net discharge area unless otherwise indicated. This means that the actual cross-sectional area is slightly larger, due to streamlining of the flow pattern.

The reduction factor is defined as the factor with which the original discharge cross-section is multiplied in order to obtain the discharge area of the barrier.

$$A_b = \mu \cdot A_d$$

$A_b =$ Discharge area of the reduction barrier

$\mu =$ Discharge reduction factor

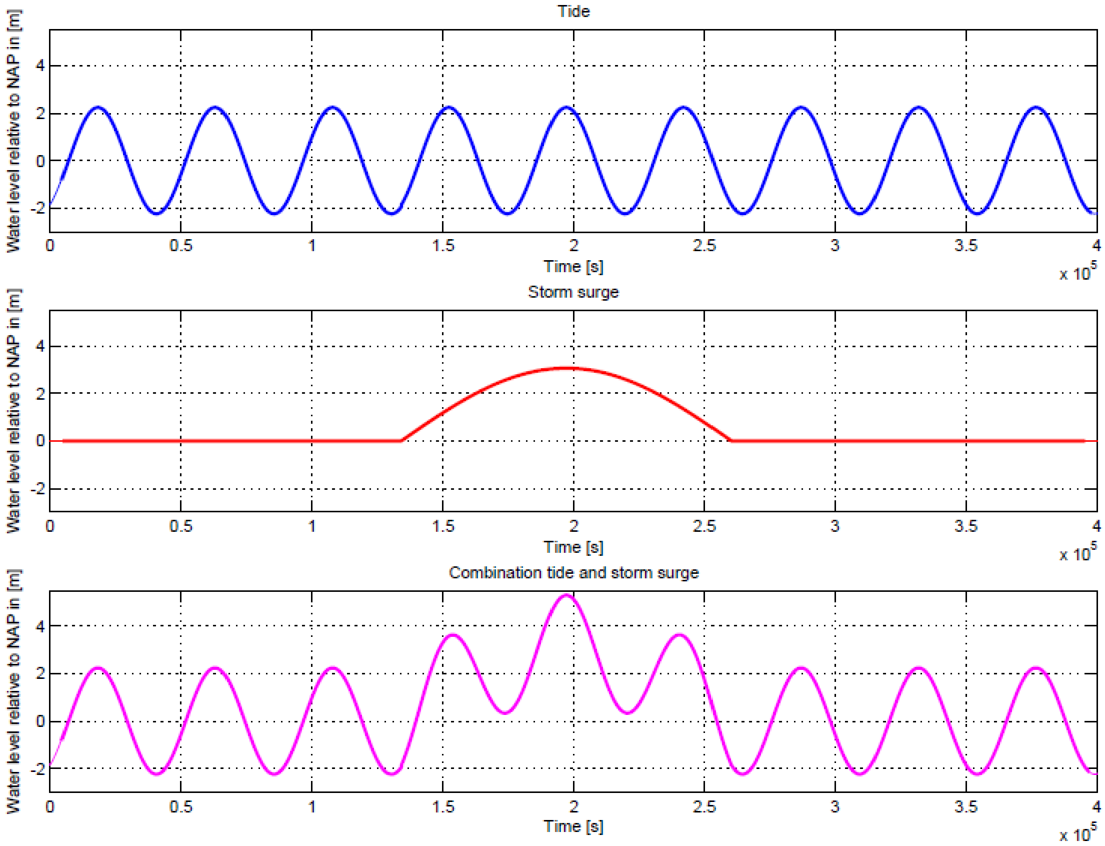
$A_d =$ Original discharge cross-section of the estuary

Explanation of the boundary conditions at sea

The boundary condition during normal tide is a sinus function with a period of 12 hours and 25 minutes. Higher order tides are not included, for simplification reasons. The amplitude is assumed constant so there is no spring and neap tide cycle. Hoever the spring tide amplitude is applied in the calculations.

The boundary conditions at sea during storm conditions are composed of two parts. The first part is the normal tide. This is modelled as a sinus function, see previous paragraph. The second part is the storm surge wave. This is modelled as a single positive half of a sinus function with a period of twice the storm surge duration. By adding up those two parts (blue + red), the boundary during storm conditions is found (purple), see Figure 3.1. The peak of tide coincides with the peaks of the storm surge in the presented figure. This leads to the highest water level at sea. It is also possible that the minimum of the tide coincides with the peak of the storm surge, this will create a double peaked boundary condition, see Figure I. 5. All intermediate combinations are also possible.

Figure 3.1 Composition of the boundary conditions at sea during storm conditions



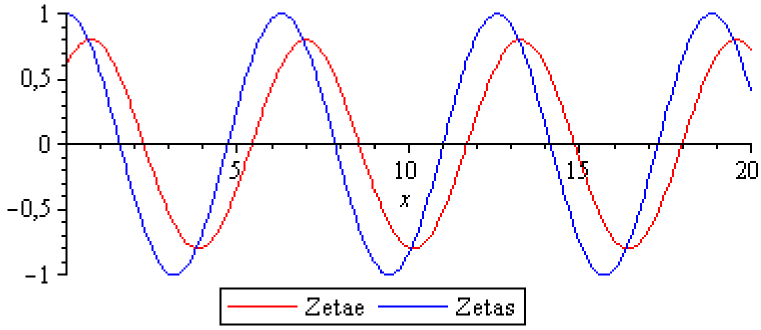
Reducing the tidal wave or the storm surge wave

This analysis focuses on the reduction of the tidal wave. The timescale of the storm surge wave is very long in comparison with the timescale of the tidal wave. This means that the response of the water level by the storm surge wave in most estuaries is quasi static. The higher frequency of the tide makes reducing the tidal amplitude much easier, since higher frequencies are easier to dampen than low frequencies. The opening in the barrier can remain relatively large. The achieved effect of the reduction on the tide is small, since the storm surge wave is able to penetrate into the estuary. In case more reduction is required, the opening should be made smaller which results in a reduction of the storm surge wave as well as the tidal wave. This opening is however very small and the dynamic nature of the estuary is affected which is undesired. So the reduction barrier should only reduce the tidal wave in order to keep the opening as large as possible to preserve the dynamic nature of the estuary.

3.2. Final gap model

The first method which is discussed is the application of the so-called ‘final gap calculation’ (Battjes, 2002). This is a type of storage calculation for a reservoir in which the water level is fluctuating due to the water level fluctuation at sea, see Figure 3.2. The amplitude reduction is related to the phase shift of the response. The response of the water level in the estuary is important for the design of the reduction barrier, since the water level fluctuations in the estuary must be restricted. The reduction barrier must be able to reduce the water level with a certain demanded amount.

Figure 3.2 Indication of the effects on water level fluctuations (sea = blue, estuary = red)



The calculation assumes that the length of the estuary is small in comparison with the length of the tidal wave. The result of this assumption is that the water level in the estuary remains horizontal. Another assumption which is made is that the inertia of the water in the connection between the estuary and the sea can be neglected. This assumption can be made when the length of the connection is small. This assumption is valid, since the length of the connection is only the width of the barrier. The method calculates the relation of water level elevation between the estuary and the sea, the magnitude of the phase shift and the flow speed of the water between the estuary and the sea.

The length limitation results in a maximum length of the estuary of about 27 kilometre for an average depth of 15 metre. Larger average water depths result in larger allowable estuary lengths.

Results of the calculation

The results of the final gap model for a normal tide with an amplitude of 2 metre are presented in Table 3.2. The second column contains information about the amplitude of the water level in the estuary relative to the amplitude of the tidal wave at sea. The impact on the tidal prism is presented in the third column. The fourth column presents the flow velocities for the normal M2 tide. The final column holds information about the relative maximum amplitude of the water level in the estuary during storm conditions. The storm conditions are modelled by increasing the tide with a wind set up of 3 metre, see section 3.1. A storm set up of three metre is used since a typical value for the Netherlands lies somewhere between 3 and 4 metre, depending on the location along the coast. The six selected values for the ratio between the storage area of the estuary and the discharge cross-section (A_e/A_d) are chosen since they cover the entire reduction spectrum.

Table 3.2 Results final gap calculation

Storage area/ discharge cross-section	Reduction factor of the amplitude of the normal vertical tide	Reduction factor tidal prism; Q	Maximum flow velocity [m/s]	Normalised reduced amplitude during storm conditions
$0.10 * 10^5$	0.99	0.99	2.6	0.99
$0.25 * 10^5$	0.77	0.79	5.4	0.91
$0.50 * 10^5$	0.46	0.51	6.4	0.77
$1.0 * 10^5$	0.24	0.30	6.7	0.64
$2.0 * 10^5$	0.12	0.16	6.8	0.48
$4.0 * 10^5$	0.06	0.08	6.8	0.34

The water level information from the table is also available in a graph, see Figure 3.3. On the x-axis of the graph the surface area of the estuary is divided by the discharge area. Keep in mind that the number on the x-axis should be multiplied with a factor 100 000. On the y-axis the tidal amplitude in the estuary is divided by the tidal amplitude at sea. So the value of 0.5 corresponds with a situation in which the tidal amplitude in the estuary is half the tidal amplitude at sea.

Figure 3.3 Design tool for normal conditions based on final gap calculation (Relative discharge area plotted against relative amplitude)

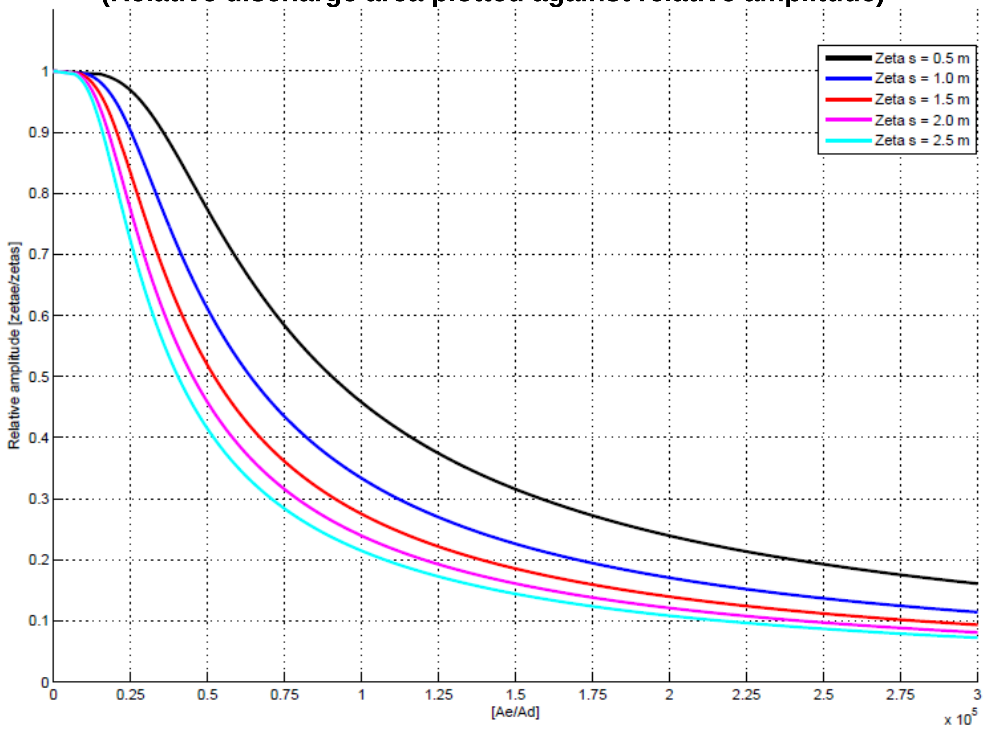
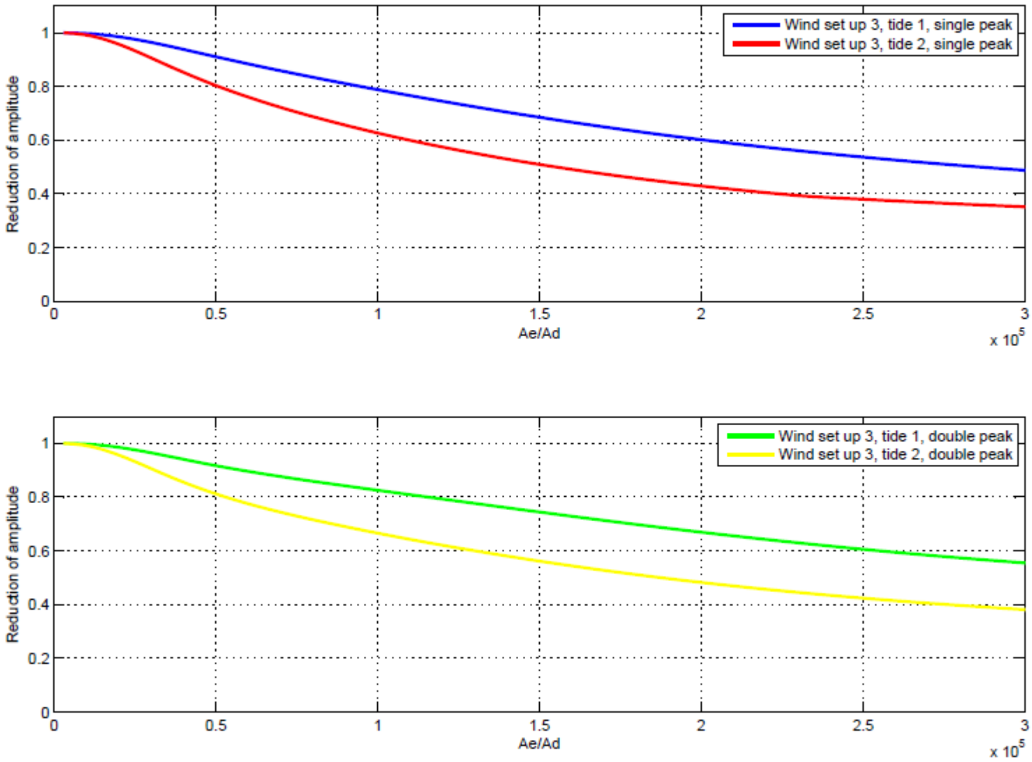


Figure 3.4 contains the information of the storm surge situation, based on the final gap model. The two graphs are for different situations. The first graph is for the situation with one very high peak, which leads to a small period of very high water. This happens because the peak of the storm surge coincides with the high tide, see Figure 3.1. The second graph is for a situation with two relatively high peaks, which results in a long period of high water. This happens because the peak of the storm surge coincides with the low tide, see Figure I. 7. Both graphs include one line for 3 metre wind set up and 2 metre tidal amplitude and one line for 3 metre wind set up and 1 metre tidal amplitude. The high water level at sea and the long duration of this high water results in a smaller reduction in comparison with the normal tide. This is as expected since more water enters the estuary

over a longer duration. In Figure 3.4 the same parameters are plotted on the axes as in Figure 3.3.

The storm surge duration has been set to 35 hours in anticipation of the advice of the Delta Commissioner. The current guideline recommends 29 hours, but there have been calls to increase the storm surge duration. The storm surge duration should be based on statistical analysis of recorded storm surges. The duration has some impact of the reduction. The storm surge duration is increased after analysing the historic storm surge duration data. Two other storm surge durations have been investigated, with a duration of 29 hours and 40 hours. In Figure I. 10 and Figure I. 11 can be seen that the difference in duration does not lead to major changes in the graphs.

Figure 3.4 Design tool for storm conditions based on final gap calculation (Relative discharge opening plotted against reduction factor)



3.3. Analytical 1-D model

A one dimensional model (1-D model) models an estuary as a prismatic rectangle with a constant width and storage width. The length of estuary is included in the model. The amplification of the tide at the end of the estuary can be calculated and the development of the tidal amplitude is modelled as well. To put it differently, the water level in the estuary is not assumed to be horizontal. The input parameters for the 1-D Model are the channel depth, channel width, storage width, resistance factor and tidal amplitude at sea.

The resistance of the tidal wave reduces when the flow velocity decreases, so when the reduction barrier reduces the discharge into the estuary, the amplitude reduction is not as much as one would expect, since the resistance decreases which results in stronger resonance effects. This model is applicable for estuaries with a more or less prismatic shape.

Results of the calculation

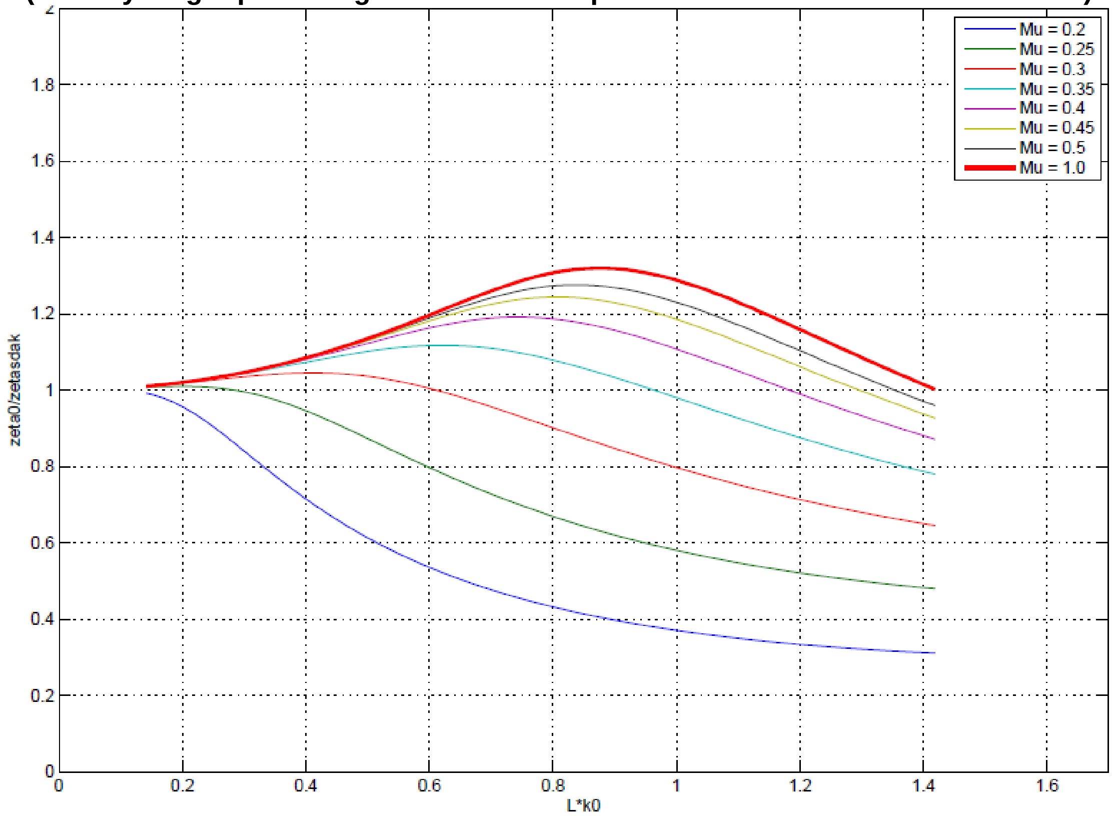
In Figure 3.5 is presented for a specific situation what the effect the reduction barrier has according to the 1-D model. In Figure 3.6 is a longitudinal cross-section presented of the schematisations in the 1-D model. The bold red line is the relation between the tidal amplitude at sea and the amplitude of the water level at the land side of the estuary without a reduction barrier. The reduction factor (Mu) is the factor between the opening in the barrier and the original opening. The x-axis is the length of the estuary times the wave number without resistance. The wave number is defined as '2π/wave length'. The graph shows that the reduction barrier is more effective for situations in which the amplitude at sea is amplified in the estuary.

The graph is valid for all estuaries with the same channel depth, channel width, storage width, resistance factor and tidal amplitude at sea, but with different lengths. In this case the input parameters for the Western Scheldt are used. The L*k0 value for the Western Scheldt is about 0.92. So only the values for L*K0 equals 0.92 should considered for the Western Scheldt. The amplification factor of about 1.3 without barrier at L*k0 is 0.92 corresponds with the measured water levels.

The tidal wave should be added to a longer, storm surge wave to create a storm situation. The response of the water level in the estuary caused by the storm surge wave is expected to be hardly influenced by the reduction barrier. The storm surge wave cannot be included in the calculation since this is the periodic solution for a periodic excitation, which does not apply to a storm surge. The reduction barrier has more effect on the tide than on the storm surge. The graph shows which estuary length is most effective for the reduction barrier.

The flow velocities through the opening in the barrier and the water level difference over the barrier are essential for the design of the barrier. The graphs for these parameters are presented in appendix I.

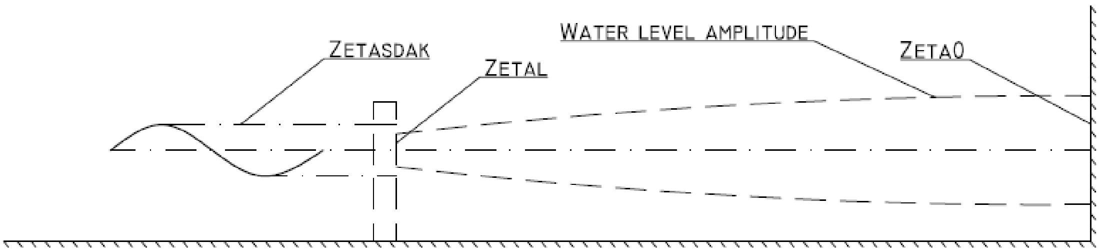
Figure 3.5 Reduction barrier in analytical 1-D model
(Estuary length plotted against relative amplitude for different reduction factors)



The graph displays the results for estuary length from 10 up to 100 kilometre. The figure can be applied for the design of the barrier, but should be used only for the specific estuary. The graph shows that for short estuary a reduction factor of the discharge area (Mu) of 0.2 is not very efficient, however for large estuary lengths is the reduction factor of 0.2 very effective (blue line in Figure 3.5). The storage area increases due to the increased length of the estuary. This leads to small reduction for the short estuaries.

The water level difference over the barrier is not indicated in the graph. In Figure 3.6 is shown that the water level difference over the barrier can be found when zetal is known. The water level difference over the barrier is important for the structural design of the barrier, since the barrier has to resist the water level difference.

Figure 3.6 Principle of the analytical 1-D model



For more information about the model and graph about flow velocity, discharge and water level difference over the barrier is referred to appendix I.

3.4. Comparison of the calculated values with measured water levels

The Eastern Scheldt Storm Surge Barrier is an excellent case to verify the results of the calculations with the models. The Eastern Scheldt had a discharge area of 80 000 m^2 in the situation before the construction of the barrier. After the construction of the barrier the gross discharge area had to be reduced to 18 000 m^2 (Visser, 2003) which would lead to a reduction factor of the tidal amplitude of about 0.86. The storage area of the Eastern Scheldt is 351 km^2. The tidal amplitude at sea is about 1.65 m.

Measured water levels in the Eastern Scheldt

The values for different locations inside and one location outside of the barrier are presented in Table 3.3. These values are used to calculate the reduction factor of the tide. 'Roompot' is the location of the barrier, 'Bergse diepsluis west' is at East side of the Eastern Scheldt and 'Stavenisse' is about in the middle.

Table 3.3 Measured water levels in the Eastern Scheldt in [cm] relative to NAP

January 1st and 2nd 1990			
Roompot outside			
High water	Low water	Total difference	
146	-177	323	
149	-131	280	
143	-167	310	
Roompot inside			
High water	Low water	Total difference	**Reduction**
122	-160	282	0.87
131	-120	251	0.90
120	-153	273	0.88
Stavenisse			
High water	Low water	Total difference	**Reduction**
147	-179	326	1.01
158	-138	296	1.06
144	-173	317	1.02
Bergse diepsluis west			
High water	Low water	Total difference	**Reduction**
177	-200	377	1.17
183	-159	342	1.22
172	-192	364	1.17

January 1st and 2nd 2010			
Roompot outside			
High water	Low water	Total difference	
163	-158	321	
195	-131	326	
194	-148	342	
Roompot inside			
High water	Low water	Total difference	**Reduction**
132	-135	267	0.83
161	-111	272	0.83
158	-125	283	0.83
Stavenisse			
High water	Low water	Total difference	**Reduction**
157	-159	316	0.98
189	-126	315	0.97
186	-140	326	0.95
Bergse diepsluis west			
High water	Low water	Total difference	**Reduction**
187	-183	370	1.15
216	-157	373	1.14
215	-170	385	1.13

As can be seen in the above tables, the barrier reduces the tidal wave in the Eastern Scheldt. Only the reduction varies along the Eastern Scheldt. For the west part of the Eastern Scheldt is the measured reduction factor indeed 0.86, but on the East side of the Eastern Scheldt is no reduction but an amplification of the tide relative to the tide at sea, which is caused by the length of the estuary. The reduction factor of 0.86 was guaranteed with a 95% certainty, which means that the amplitude reduction factor of 0.86 is a lower limit.

The presented data were measured during calm weather conditions. The maximum hourly averaged wind speed and wind direction can be found in Table 3.4. The wind direction is presented in degrees, with 0° as North, 90° as East and so on. The estuary of the Eastern

Scheldt is positioned from approximately West North West to East South East. The low wind speeds (in 1990) and the cross wind direction (2010) limit the influence of the wind on the water levels in the Eastern Scheldt. So the measured water level is mainly influenced by the tidal wave. The wind data is provided by the Royal Netherlands Meteorological Institute (KNMI). The data of the weather station of Wilhelminadorp is used, which is located halfway the Eastern Scheldt at the South side.

Table 3.4 Weather data for the measured water levels in the Eastern Scheldt

	1990		2010	
Date	January 1st	January 2nd	January 1st	January 2nd
Wind speed [m/s]	1.0	2.6	6.0	4.0
Wind direction [°]	114	170	40	220

Reproduction of the results with final gap model

According to the final gap calculation a reduction factor of 0.86, with a tidal amplitude of 1.65 metre requires an A_e/A_d of $0.21*10^5$. This means that the discharge area should be $351*10^6/0.21*10^5$=16 700 m^2. The error is limited, which is as expected. The water level in the estuary cannot be assumed horizontal, since the length of the estuary is quite long, see section 3.2.

Reproduction of the results with the analytical 1-D model

The results of the 1-D model are also checked with the measured values for the Eastern Scheldt. The 1-D model requires as input the maximum amplitude of the tidal wave in the back of the basin. For this parameter the measured value is used, so 1.85 metre.

The results are as follows. The amplitude at the land side of the estuary can be realised with a reduction factor of mu of 0.4. It is assumed that the discharge area of the channels is half the total discharge opening. So the opening in the barrier should be 0.4 * 40 000 m^2. This results in a discharge area of about 16 000 m^2. The error is about 7%. The magnitude of the error should be taken into account when calculating the discharge opening of the barrier.

The 1-D model is able to include the propagation of the tidal wave into the calculation, which is excellent for even longer estuaries. However it must be stated that the input parameters are open for debate, since the estuary is not actually prismatic. Small changes in the input can have quite large effects on the results. So the model is applicable but the results should be analysed critically. The effect of one failing gate can be quite large, as already explained in chapter 2.

Both models behave as expected, so there is no reason to expect major errors in the models.

3.5. Determination of the desired amount reduction

When designing a reduction barrier the amount of reduction of the horizontal and vertical tide has to be determined. Since the amount of reduction of the tide, both vertical and horizontal, has to be known before the opening size can be determined. The first decision which has to be made is whether the reduction should be based on the discharge or on the water level or both. The discharge is important for the sediment transport through the barrier and thus for the environment. The water level in the estuary can be important for other environmental values, like shellfish, birds and seals, but also for safety. A water level is more easy to measure than a discharge. In the case of the Eastern Scheldt barrier the demand for the reduction during normal conditions was defined as a water level near

Yerseke. Defining the minimum or maximum water level is the most practical approach in stating the desired reduction.

When talking about the normal situation, the reduction of the tide should be limited, since a reduction of the tide can harm the environment. The sediment discharge depends on the tidal prism (Bosboom and Stive, 2011), therefore the discharge through the barrier should stay in the same order of magnitude of what it was before construction of the barrier. So in this case one could demand a minimum of tidal discharge in the estuary. The sediment transport in and out of the estuary must remain possible, since the estuary should be able to evolve with the rising sea water level. The type of sediment which is important for this phenomenon is sand. Sand is assumed to be mostly transported in the bed load (Bosboom and Stive, 2011). Therefore the barrier should be designed in such a way that the sill does not cut off most of the sand transport.

For storm conditions the decision depends on different parameters. The amplitude of the tide should not lead to unsafe situations in the estuary. The height of the levees or the land around the estuary is the key factor for this decision. The bed protection should be able to cope with the high flow velocities through the barrier. In storm conditions the reduction factor doesn't depend on environmental factors but on the boundary conditions. The environment is not expected to be harmed by a short closure during storm conditions.

3.6. Conclusions about the models

This subsection presents the conclusions about the calculation and the implication for the location choice for the case study.

Final gap model

The final gap model is simple, analytical method for describing the water level fluctuation in a short estuary. The assumption concerning the inertia of the water in the connection between the estuary and the sea is valid. However the assumption about the length of the estuary does limit the applicability of the results. The estuary length must be less than 1/20th of the tidal wave length.

The results are useful for a quick estimation or a starting point for the design in a specific case. A design tool can be found in Figure 3.3. The tool is based on the final gap model with a tide period of 12 hours and 25 minutes. The reduction of the water level inside the estuary is relatively larger for situations with larger tidal amplitudes. The water level in the estuary cannot follow rapid water level changes due to the damping of the barrier. So a diurnal tide with a larger amplitude is easier to reduce because the water level change happens faster. This should be taken into account while searching for a location for the Case Study.

Analytical 1D model

The 1-D Model is also relatively simple and analytical, but is able to include the effect of the estuary length in the calculation. The 1-D model is much more realistic representation of the reality for long estuaries due to the propagation of the tidal wave. The results show that a location with a large estuary length is positive for the feasibility of the reduction barrier.

A design graph for the 1-D model for a semi-diurnal tide can be found in Figure 3.5. From the figure can be concluded that the length and resistance of the estuary can have considerable effect on the amount of reduction which can be achieved. The resistance in the estuary is dependent on the amount of reduction, since resistances depend on the amplitude of the flow velocity. This is taken into account in the model. The design graph can be used to find out what reduction factor for the discharge cross-section is required for a certain amplitude at the land side of the estuary.

The presented figure displays just the effect of the regular tidal wave. The effect of the storm surge cannot be calculated with this method, since this method calculates the periodic response of a periodic excitation. It is therefore required to make a numeric calculation of the water level response in the estuary for a combination of tide and storm surge. This can be done only for a specific location, since site specific information is required as input.

Local channels, storage areas and roughnesses should be included when a more accurate prediction of the tide is desired. The model does not include additional discharge into the estuary from rivers or rain fall. This means that the presence of a river should be taken into account by reserving sufficient buffer capacity in the estuary.

Site selection for the Case Study

The reduction of the discharge cross-section should be chosen in such a way that the environment is not harmed in order to provide flood protection while keeping the flow velocities acceptable for the vessels. The vessels require a large opening to navigate through, so a substantial reduction of the discharge opening is not desired. This means that the locations which are interesting should have a length $k0*L$ of about 0.8 - $\pi/2$, depending on the resistance in the estuary. For these lengths the effect of a reduction barrier is largest.

The estuaries which should be considered are on a sandy coast, otherwise the demand for the tidal prism with regard to morphology is irrelevant. The estuaries have to have a quite substantial flood protection scheme already in place, since the reduction which can be achieved is not more than 2 metre. The location should have a large tidal difference in normal conditions, since these shorter waves are more easily reduced than the storm surge.

Assumptions applied in the models

The calculations apply certain assumptions which make the calculation simpler, but the results less accurate. The assumptions are described below.

These calculations assume a prismatic estuary. Therefore the width does not change with the water level. The width of the storage area will change in reality due to the flooding of tidal flats.

The calculation assumes a sinusoidal tide with the period of the M2 tide which is 12 hours and 25 minutes. The real tide is composed of a combination of all kinds of sinusoidal waves. It is therefore possible that a certain component of the tide starts resonating due to the impact of the reduction barrier.

Another aspect which is not included in the calculation is the 2D and 3D effect of the flow, such as ebb and flood channels.

The reduction barrier strongly influences the flow over the width of the estuary. This may lead to dangerous currents and unexpected changes in the morphology.

The 1D calculation applies linearization of the resistance. In reality the discharge and the resistance are quadratically related.

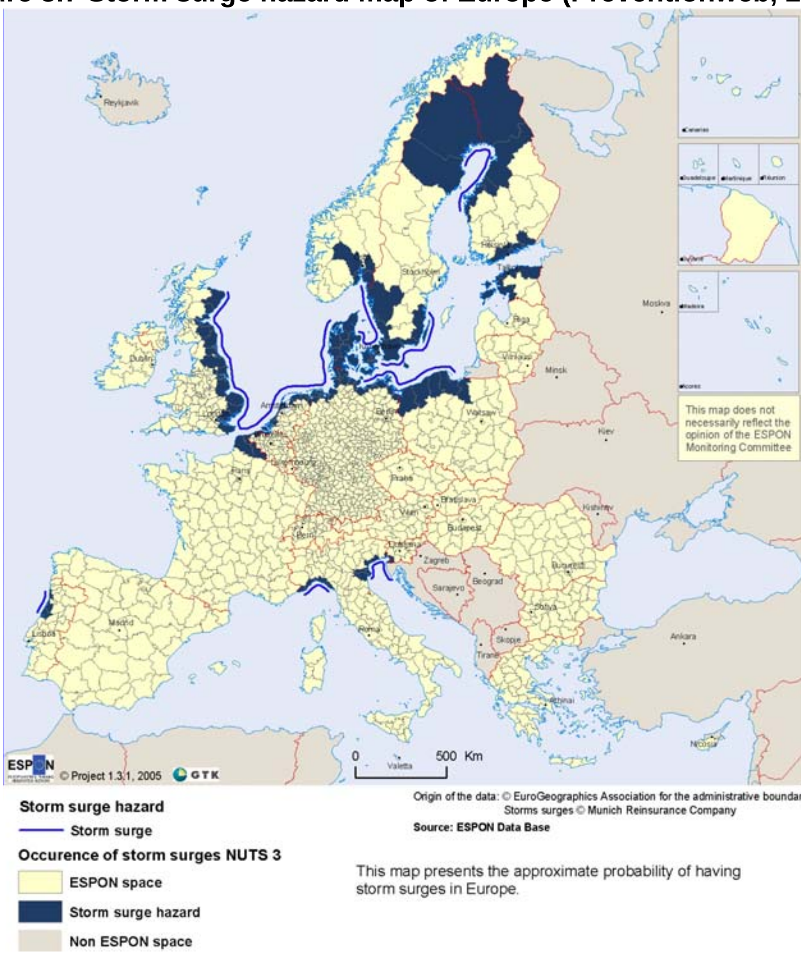
The assumptions which are described above indicate that once the reduction barrier appears to be feasible, a more accurate 2D and 3D flow model should be made.

3.7. Potential locations for the Case Study

The principle of the reduction barrier is better understood, so it is time to find location where the solution might work. In this chapter the potential locations are discussed. The potential locations must fit a number of requirements based on the results of the previous chapter. The location must have to deal with storm surges and must have a considerable surface area for storage of the water. Also large tidal amplitude is favourable. The calculations in the previous chapter showed that a large length is preferable for the effectiveness of the reduction barrier. A location in the Netherlands would increase the availability of data about the location.

The United Nations have initiated a project for disaster risk reduction. This includes a website on which information about risk is gathered and shared. The locations with risks for storm surges can be found in Figure 3.7. The ‘reduction barrier’ is only possible in a situation with a large water area behind the barrier. So bays and estuaries are the main locations which are considered. The British, Dutch, German and Danish coast are the main locations for storm surges in Europe, see Figure 3.7. The possible locations for the reduction barrier are chosen in the Netherlands since the surface levels in the Netherlands are much lower as in the other countries, which has a large impact on the size of the endangered area. Also the population density in the endangered areas is much higher in the Netherlands (Eurostat, 2008).

Figure 3.7 Storm surge hazard map of Europe (Preventionweb, 2005)



The locations which are investigated are the Eems Dollard, Waddenzee, Afsluitdijk, Lauwers Lake in the North of the Netherlands, Haringvliet, Grevelingen Lake, Eastern Scheldt and Western Scheldt in the South West part of the Netherlands. The parameters were retrieved from (Rijkswaterstaat, 2012). For detailed information about the location is referred to appendix II.

3.8. Locations for the Case Study

The selected locations are discussed in this section. The discussion leads to a decision for location of the Case Study. An overview of the geographical locations is presented in Figure 3.8.

Figure 3.8 Investigated locations for the Case Study

3.8.1. Eems Dollard

The Eems Dollard is an estuary on the border between the Netherlands and Germany. The exact position of the border is still under debate. The estuary has a high natural value. The surroundings of the estuary are not densely populated, therefore there is no economical reason for an expensive barrier. Also the border conflict will not have a positive effect on the decision making. Besides that Germany has already constructed a barrier in the Eems on German soil. Therefore this location is not considered to have any potential for the Case Study.

3.8.2. Waddenzee

In fact, the Waddenzee is already a reduction barrier. The isles are basically a sort of barrier that reduces the flow of water into the Waddenzee. By enlarging the islands and making sure that the isles remain at their position, it should be possible to reduce the water level in the Waddenzee. This could result in a situation in which the reinforcement of the Afsluitdijk and the Frisian coast would be superfluous. The Waddenzee is considered to be a very valuable environmental area. Therefore it is unlikely that this option will be considered. The hydrodynamic calculation of the system will be very complex and therefore this option is only mentioned and not considered for further study in this Thesis.

3.8.3. IJsselmeer
There are plans to create an ‘Open Afsluitdijk’ (Toekomst Afsluitdijk, 2008). This means that the Afsluitdijk is redesigned by creating tidal flats at the location of the current dam. This would create a more soft transition from the Waddenzee into the IJsselmeer. A reduction barrier is probably not possible in this situation, because the area behind the barrier has to stay fresh. The IJsselmeer is part of the fresh water reserve, therefore it is not possible to create a larger area behind the barrier.

3.8.4. Lauwers Lake
The Lauwers Lake is already an environmentally valuable area. Some of the advantages of the re-introduction of the tide can be achieved by opening the sluices in the dam at certain moments (HKV, 2005). The additional value of a more complete tidal influence is limited, since there are large costs involved, like the upgrading of the levees around the lake.

3.8.5. Haringvliet
The Haringvliet is an interesting location. The Haringvliet can be used for increasing the discharge capacity for the rivers has potential. The system is complex, but can result in large benefits for both safety and the environment. The environmental impact of re-introduction of the tide has been studied already. The storage of fresh water is a point of concern for the opening of the Haringvliet dam, since the intrusion of salt water will increase. This salt water intrusion is bad for the supply of fresh water but also has negative effects on the plant and animal life currently living in the Haringvliet. Shipping can profit from this new entrance, for both recreational and industrial purposes. The amount of vessels and the vessel size that should be expected is not as large as for example on the Western Scheldt.

3.8.6. Grevelingen Lake
The Grevelingen Lake is different to the Haringvliet because of the closed eastern boundary. This means that the advantages are more limited to the lake itself. The storage of fresh water is not an issue due to the already salt environment in the Grevelingen. The re-introduction of the tide would be positive for the water quality. The area can become less attractive for recreation activities by the re-opening of the Grevelingen, since the higher flow velocities may be dangerous to divers and the visibility under water will be reduced due to the transport of sediment.

3.8.7. Eastern Scheldt
The Eastern Scheldt will benefit from a reduction barrier, in case it is able to let more sediment pass the barrier. For sediment transport it is better to remove the sill beams for example, this would increase the discharge area and depth. This can be a solution for the deficit of sand the in the Eastern Scheldt and therefore could reduce the erosion of the tidal planes.

It might be feasible to remove one quarter of the gates. This would result in a more dynamic situation and reduces the maintenance costs of the barrier, as discussed in the introduction. However the removal of the gates will also remove the redundancy in the system.

3.8.8. Western Scheldt

A barrier in the Western Scheldt shortens the coastline significantly. The barrier must be able to let vessels pass that want to call at the Port of Antwerp and other ports in the region. The vessels that should be expected require a significant channel size. The discharge into the estuary will therefore be considerable. An advantage is the high levees around the estuary, which increases to buffer capacity. The environment will not be hindered due to the open nature of the barrier. At this location the barrier's function is not to improve the environment but to improve safety. At the other locations the function is to improve the environment while maintaining safety. The main source of discharge into the estuary is the river Scheldt.

3.9. Discussion and selection of the location of the Case Study

The two locations with highest potential are the Haringvliet and the Western Scheldt. The main difference between both locations is the purpose of the barrier. In the Western Scheldt the purpose would be to provide safety, while the barrier would be used for environmental development at the Haringvliet. Both locations are relevant with respect to sea level rise.

The required reduction of the water level at both locations is used to select the best location for the case study.

The levees surrounding the Haringvliet have not been strengthened after the flooding in 1953. So when the tide will be re-introduced in the Haringvliet it is very likely that the levees will have to be strengthened or the reduction of the tide has to be very large. When taking sea level rise into account as well, the situation gets even worse. A very strong reduced tide is not what is desired in this area. The current situation is likely to be the best option for the Haringvliet.

The Western Scheldt levees currently satisfy the criteria. However the Belgians require some reduction during storm conditions. The levees need to be maintained and possible heightened due to sea level rise. The length of the Western Scheldt is such that the reduction barrier in the mouth achieves maximum efficiency. So a reduction barrier in the Western Scheldt has more potential since it has to do less work and it can be very efficient.

This means that that the best location for the reduction barrier is the Western Scheldt. The required reduction is less large, which is beneficial for both nature and shipping since a larger opening can be maintained. In the next chapter is the Case Study presented for the Western Scheldt.

4. THE WESTERN SCHELDT

The subject of this chapter is the analysis of the Western Scheldt. The first step is to make an analysis of the relative value of reduction barrier. A major part of the analysis is the hydrodynamic analysis, which results in the required opening size for both normal and storm conditions. The entire analysis results in the terms of references in section 3. The Terms of References are used in the design of the barrier, which is presented in the next chapter.

This is the first chapter about the Case Study. The Case Study is used to show what a reduction barrier may look like, but the main function of the Case Study is that the reduction barrier can be compared with other solutions. The comparison shows whether a reduction barrier is actually a good idea or just a theoretical option.

4.1. Analysis on system level

In this section the possible solutions for the estuary are described. The next step is the decision for the best solution based on a multi criteria analysis.

4.1.1. Options for flood protection on system level

There are four basic options to protect the Western Scheldt as already discussed in section 2.1. These options are a dam, a storm surge barrier, levee heightening and a reduction barrier. The main issues for all the options are explained in section 2.1. The value for the four options is determined and explained in the following subsection.

4.1.2. Selection of the most valuable option

The value of the four options, as discussed in the previous section, is determined and compared by using a multi criteria analysis (MCA). The criteria that are used to compare the options are environmental change, shipping and redundancy. The environmental change is mainly focussed on the changes in morphology and changes in the salinity of the water. The hindrance of the solution for the shipping has to do with the hindrance of the structure itself and the hindrance due to the expected changes in morphology. Flood protection is not one of the criteria since all options should be designed in such a way that they are able to guarantee the same safety level.

The amount of criteria is relatively small and importance of the different criteria mainly based on political decisions. Therefore is decided not to differentiate in the importance of the criteria. The results are presented below.

Table 4.1 MCA on system level

	Dam	Storm surge barrier	Reduction barrier	Levees
Environmental change	1	3	4	5
Shipping	2	1	3	3
Redundancy	5	5	4	3
	8	9	11	11

The reduction barrier ties with the heightening of the levees. The importance of the criteria can influence the results of the analysis when the environmental change is considered to be less important. The power of the environmental organisations and European laws is considerable, it is therefore unlikely that the environmental change reduces its importance. The decision between the levees and the reduction barrier is completely based on the costs since both solutions have the same value.

4.1.3. Comparison of the costs between a reduction barrier and levee heightening

The reduction barrier is only economically feasible when the costs for maintaining and upgrading the current system are higher. The costs of raising the levees in the Dutch part of the Western Scheldt must be added to the costs of the flood protection in the Belgium part of the Scheldt. The estimated costs are presented in Table 4.2.

Cost estimation of levee heightening

The costs for the Dutch levees are for a levee raising of 1 metre over the entire length of the estuary. The costs for the Belgium part are the estimated costs until 2030 (Schelde communicatie, 2010). The Belgians have not taken into account the 1 metre sea level rise so the sea level rise will result in additional costs and the current safety level is still below the demanded values (Schelde communicatie, 2010). The length of the levees in Belgium is about 260 kilometre. It is assumed that 30 kilometre is located in rural areas, which results in more costs for levee heightening. The costs for the impact on the environment are not included since these costs are extremely difficult to estimate.

There are also other costs which should be expected when the sea level rises. For example the locks which provide access to the Port of Antwerp will have to be adjusted in case of sea level rise as well as other locks and pumping stations along the river Scheldt. For Port of Antwerp seven locks need to be adjusted.

Beside the ‘direct’ costs, it would be wise for the Belgians to evaluate their flood defence system. It could make sense to increase the safety level of the Antwerp area to 1/10 000th per year, since the economic properties of the area are comparable with the Rotterdam area in the Netherlands. An increase in protection level would require even more work on the levees, which would cost serious money. While a relatively small adjustment to the reduction barrier could solve the problem.

Cost for the reduction barrier

The costs for the barrier are calculated by taking the average costs of several already constructed storm surge barriers. The barrier in the Western Scheldt has to become very long, which is excellent for a repetitive design. The costs for the examples with a lot of repetition are below average, which means that the average value is probably going to be an overestimation of the costs. Another reason the costs are likely to be lower, is the absence of moveable parts in the reduction barrier.

Remarks

The nuclear power plant near Borssele could also be behind the barrier, which is positive for the safety. The barrier can be part of the plan to increase the protection level for the nuclear power plant from a flood probability of 1/4 000th to 1/10 000th per year or less (Peer Review Country Report, 2012).

The required amount of levee heightening also depends on settlements and more importantly on the wave height. The required heightening could be much larger, in case the estuary is not able to adjust with the sea level rise, due the sediment deficit or the speed of the sea level rise. A larger water depth results in larger waves, which requires higher and stronger levees.

Conclusion about costs estimation
In short, the expected costs for the maintaining and upgrading the current system are a minimum and the cost for the barrier is maximum value.

Table 4.2 Costs estimation upgrading current flood protection until 2120

	Length [km]	Costs per unit length [M€/km]	Costs [M€]
Dutch levees	120	6	720
Belgium flood protection until 2030	-	-	880
Belgium flood protection after 2030	30	18	540
	230	6	1380
		Total costs	3520

Table 4.3 Costs estimation reduction barrier

	Average retaining height [m]	Water level difference [m]	Length [m]	Costs per m^3 [€/m^3]	Total costs [M€]
Reduction barrier	17	2.0	3700	30 000	3 774

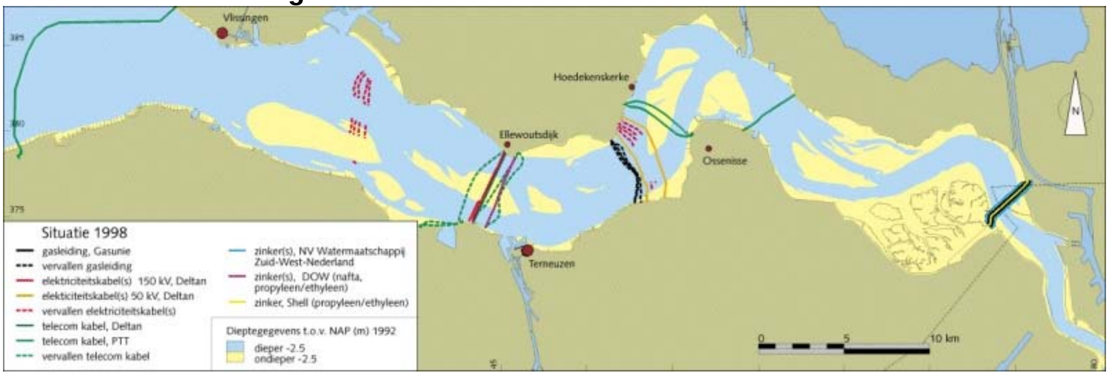
The values for the costs estimation of the barrier are rough figures. This shows that the costs of a barrier can be nearly covered by the no longer required or less extensive levee heightening. The qualities of the barrier, as mentioned before, are expected to tip the balance in favour of the reduction barrier.

4.2. Determination of the location for the reduction barrier
The selection of the barrier location in the Western Scheldt is discussed in this section. A decision is made based on a MCA. The selected location is described in more detail.

4.2.1. Possible location for the barrier in the Western Scheldt
The Western Scheldt is quite long, it is therefore possible to place the barrier at several locations. The location of the barrier in the Western Scheldt has consequences for the design of the barrier. The possible locations are presented in Figure 4.1. The sixth location is the same location as was suggested in the Belgian's Sigma Plan. Three locations in the mouth of the estuary are investigated, since the small change in location has a large effect on the wave climate and the shipping lanes. The barriers are located on different locations in the Western Scheldt where the width has a local minimum, due to a costs consideration. A major downside is that these locations are also used as a crossing point for cables and conduits, as can be seen in Figure 4.2.

Figure 4.1 Possible locations in the Western Scheldt

Figure 4.2 Locations of cables and conduits



The best location is selected based on a MCA. The criteria which are used are the wave climate, effectiveness of the reduction, environmental change, redundancy, shipping hindrance and anchorage space behind the barrier. The soil conditions are not used, since at all locations the subsoil consists of very fine sand. The subsoil information is quite restricted but the main component is, consistently, very fine or moderately fine sand.

Table 4.4 MCA for the best location for the barrier (see Figure 4.1 for the locations)

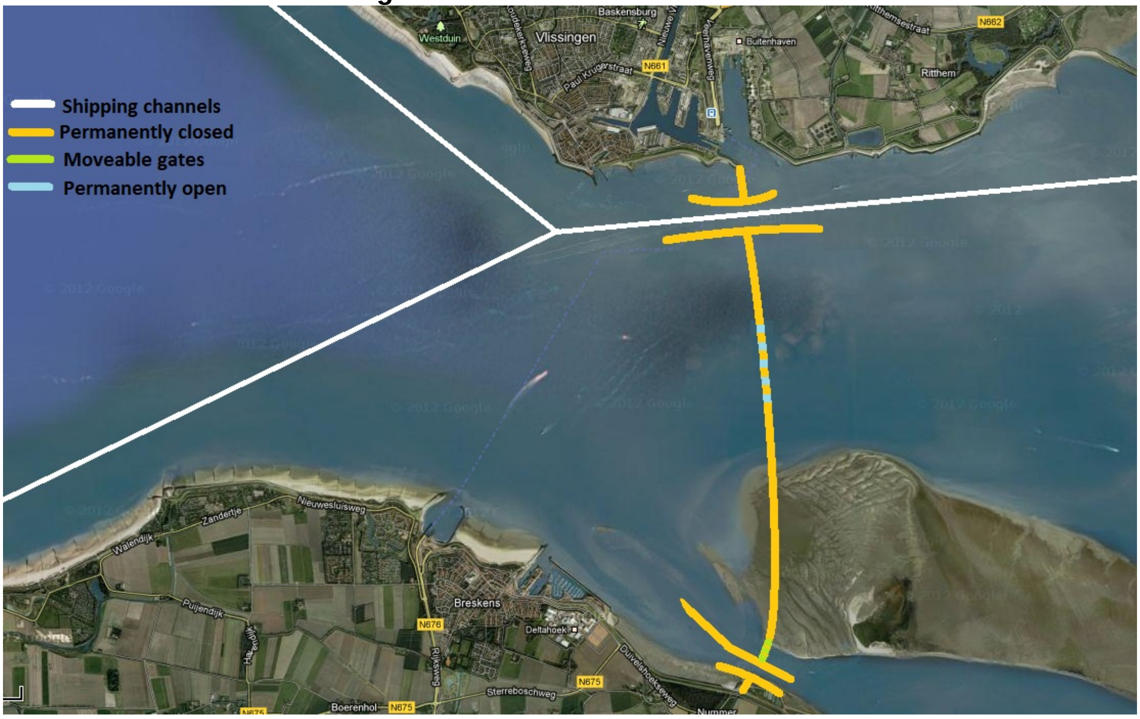
	Location 1	Location 2	Location 3	Location 4	Location 5	Location 6
Wave climate	1	1	3	2	2	5
Effectiveness of the reduction	5	5	5	3	2	1
Environment	1	1	1	2	3	5
Flood protection	5	5	4	3	2	1
Shipping	1	2	3	4	4	2
Anchorage space behind the barrier	5	4	4	3	2	1
Other infrastructure	5	5	5	3	2	1
	23	23	25	20	17	16

The result of the MCA is that location 3 is considered to be the best option. This location closes off the almost the entire estuary while it still profits from relative shelter of the land, see Figure 4.3. The length of the barriers at all locations is comparable expect for location 6. The short length of the barrier at location 6 might reduce costs, but the Netherlands do not benefit from this location. It is therefore decided to abandon this option.

4.2.2. Detailed information regarding the selected location

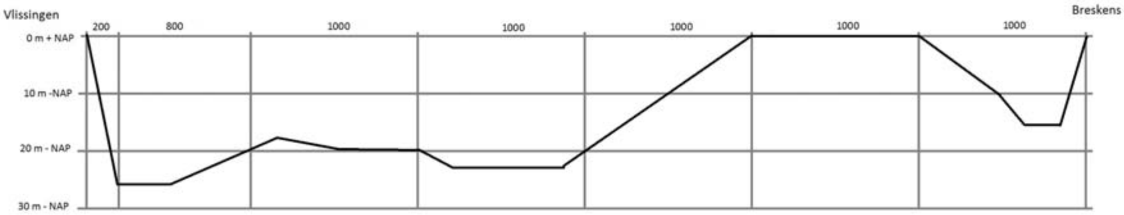
The barrier is positioned in the Western Scheldt in such a way that only one opening for the shipping channel is required. One shipping channel reduces the minimum opening size for the barrier, which is positive for the reduction capacity of the barrier. Position of the shipping lanes does not have to be adjusted. The South side the barrier is connected to the "Hoofdplaat" in order to reduce the length of the barrier. At the North side the barrier is connected to the land East of the entrance of the old port within the city of Vlissingen. The old port remains accessible without passing the barrier. A more westerly location would result in problems for the vessels, since they would have to turn, right before passing the barrier. The view from the Vlissingen and Breskens towards to North Sea is no obstructed by the position of the barrier. The beaches are also free from hindrance of the barrier. The width of the Western Scheldt at the location of the barrier is about 6000 metre. The South passages are added to increase flexibility and safety for smaller vessels. The South passages are discussed in section 5.1.

Figure 4.3 Location of the barrier



The depth profile of the cross-section of the Western Scheldt on the location of the barrier is important for the design of the barrier. This profile is obtained from a depth chart. The profile is presented in Figure 4.4. The profile consists of straight sections, this has been done for practical reasons. The bottom profile is dynamic, a more accurate depth profile can be obtained in a more definitive design stage.

Figure 4.4 Depth profile at the location of the barrier



4.2.3. Consequences of the location of the barrier

The barrier can result in an additional water level setup, which can lead to additional coastal reinforcements. A more western positioning of the barrier requires two main shipping openings instead of one. The prevention of the reinforcements in the town centre would require a much longer barrier, which has a negative impact on costs. The construction of the Eastern Scheldt barrier did not lead to significant additional set up. But this does not mean this is also the case for the Western Scheldt.

The planned strengthening, see Figure III. 2, of the levees near Breskens is not behind the barrier. The work will have to be executed even when the barrier will be build. The barrier can increase the design water level near Breskens and Vlissingen due to additional wind set up. So it is possible that the currently planned strengthening is not sufficient. The effect of the barrier can be calculated by making a 2D model which includes part of the North Sea. The planned strengthening has the form of a glass wall on top of the levee in order to reach the required height. When additional height is required, it is possible that the glass solution is not longer strong enough.

4.3. Defining the reduction situations

The location of the barrier has been selected. The next step is to determine between which boundaries the size of the opening of the barrier must lie. The storm surge situations are based on the amount sea level rise which is discussed in the next subsection.

4.3.1. Remarks about the predicted sea level rise

The barrier has to last for about 100 years. This means that in 100 years the barrier must still be functional. It is difficult to predict the change of the state of the environment over that period. It is possible that the sea level rise turns out to be much more or much less, since the effect of global warming on sea level rise is not well understood. Therefore it is preferred that the barrier is designed in such a way that it can be adjusted over time to the new requirements.

At the moment the sea level rise is measured at 3.2 mm per year (Rahmstorf, 2012). Half a metre sea level rise would take 156 year at this rate, however an acceleration of the sea level rise has not been accounted for. The highest expected sea level rise in 2100, based on current measurements is 0.78 metre (Rahmstorf, 2012).

4.3.2. The four reduction situations

There are four situations which are considered in order to determine the required reduction of the cross-section of the Western Scheldt. One of the four situations is the normal situation while the others are all in storm conditions. The three situations in storm conditions consider three different amounts of amplitude reduction near Antwerp. For these four situations is determined what opening size is either the minimum or the maximum value in section 4.4.

Normal situation

The first situation is the normal situation, so there is no storm surge and the amplitude of the tidal wave at the land side of the estuary as well as the discharge at the mouth is limited by a minimum value, because of the environmental issues. The table below contains the requirements.

Table 4.5 Reduction criteria

	Dimensions	Western Scheldt
Zeta0 minimal	m + NAP	2.25
Smallest reduction factor for the discharge	-	0.8
Maximum flow velocity	m/s	3.5

The criteria must be met by the opening size of the barrier in the normal situation. The magnitudes of the requirements are discussed in chapter 2.

Storm situation, 0.5 metre water level reduction

The second situation is related to the flood risk in Belgium. The Belgians require a water level reduction at the border between the Netherlands and Belgium of 0.5 metre in order to reach the desired level of safety (Gauderis and others, 2005). The current design water level is 6 metre + NAP. The water level must therefore have a maximum of 5.5 metre + NAP.

Storm situation, 1.0 metre water level reduction
The third situation is in anticipation of the sea level rise in 100 years. The average expected sea level rise is 1 metre. A reduction of 1 metre of the tidal wave would mean that the levees do not need heightening for the Dutch situation. So the maximum allowed amplitude at the land side of the estuary is 5 metre instead of the current 6 metre. In this case is assumed that the estuary is able to keep pace with the sea level rise, so the water depth in the estuary does not change. When the estuary bottom level remains constant over time, the required reduction factor of the discharge cross-section is slightly smaller. This should be checked by running simulations, but the morphology is a subject with a high uncertainty. The validity of the assumption can only be measured over a long period.

Storm situation, 1.5 metre water level reduction
The fourth and final situation would help keep the Belgians safe from flooding when the sea level rises 1 metre in 100 years. This requires a reduction of 1.5 metre, since the Belgians require at the moment a reduction of 0.5 metre. The current amplitude near Antwerp is about 3 metre. So the current tidal amplitude needs to be reduced by half, so a considerable reduction.

4.4. Numerical calculation of the water motion in the Western Scheldt

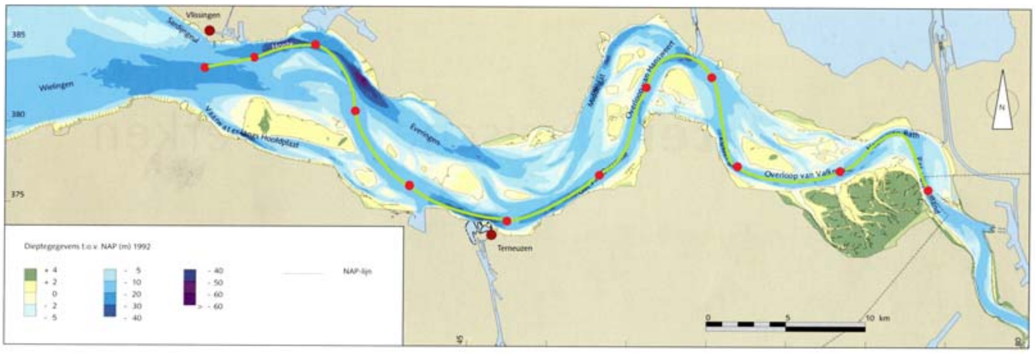
The four goals for the amounts of reduction are defined in the previous section. The response of the water level in the estuary caused by the reduction barrier is calculated with a numerical calculation, to calculate whether the above mentioned water level reduction can be achieved. The estuary has been divided into ten sections of about 6.5 kilometre and one shorter section for the barrier, see Figure 4.5. The water level at sea and the properties of the different sections are the input for the calculation. The properties of the section are the channel depth, channel width, storage width and roughness. The discharge in the green channels is calculated based on the water levels in the red points on either side of the green channel. The new water level in the red dot is calculated by calculating the difference in discharge in and out of the point. The new water levels are used to calculate the new discharges and so on.

The water level at sea is a sinusoidal tide with an amplitude of 2.24 metre and a period of 12 hours and 25 minutes combined with a storm surge of 3.06 metre which is modelled as one positive half of a sinus, see Figure 3.1.

There are also calculations made without the storm surge. For both situations has been calculated what the flow velocity and the water level difference over the barrier is.

The local wind set up is not taken into account since the barrier cannot reduce this. The goal of the barrier is to reduce the water level with a certain value, as mentioned in the previous section. The goal is reached by reducing the tidal wave. The absolute water level is not very important, only the reduction caused by the barrier. The local wind set up must be included in a 2D or 3D flow calculation in a later stage.

Figure 4.5 Principle of the numerical calculation



4.4.1. Calculation results for the normal situation

Figure 4.6 shows the water level on the border between Belgium and the Netherlands for a tide with an amplitude of 2.24 metre. The boundary conditions for the environment in normal conditions can be met with a reduction factor of 0.375, which is indicated by the black line in Figure 4.6. The discharge, flow velocity and water level amplitude are checked according to Table 4.5. The flow velocity in this situation reaches a maximum of 2.4 m/s; the water level amplitude is 2.5 and the reduction factor for the discharge is 0.8. The governing requirement is thus the discharge reduction. The reduction factor for the discharge cross-section could be smaller if it was for the water level requirement and the flow velocity requirement. The environmental requirement is based on an assumption, so the discharge cross-section during normal situations depends on the validity of the assumption.

Therefore can be decided that for normal conditions a reduction factor of 0.375 is the largest amount of reduction possible, while keeping in mind the uncertainties about the limits for shipping and the environment. The maximum water level difference over the barrier is 0.9 metre for a reduction factor of 0.375.

The water level difference is larger than expected since small discharge areas have a large discharge coefficient due to strong contraction of the flow, see formula below.

$$\xi = \left(1 - \frac{1}{\mu}\right)^2$$

$\xi =$ Discharge coefficient

$\mu =$ Discharge reduction factor

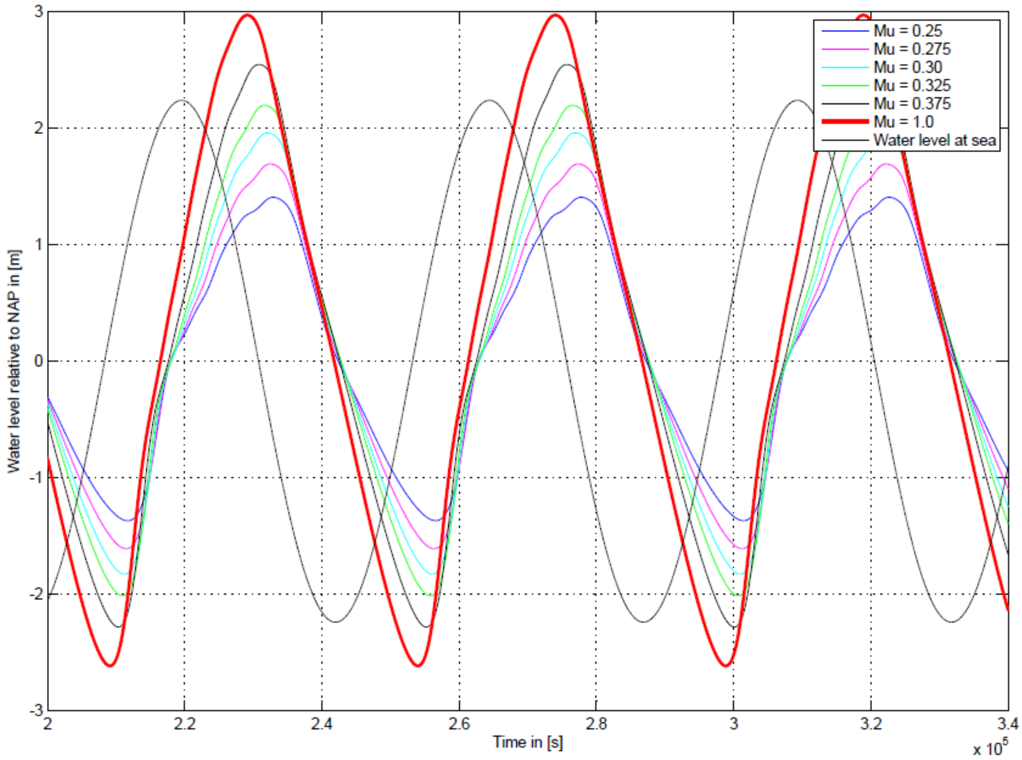
$$A_b = \mu \cdot A_d$$

$A_b =$ Discharge area of the reduction barrier

$\mu =$ Discharge reduction factor

$A_d =$ Original discharge cross-section of the estuary

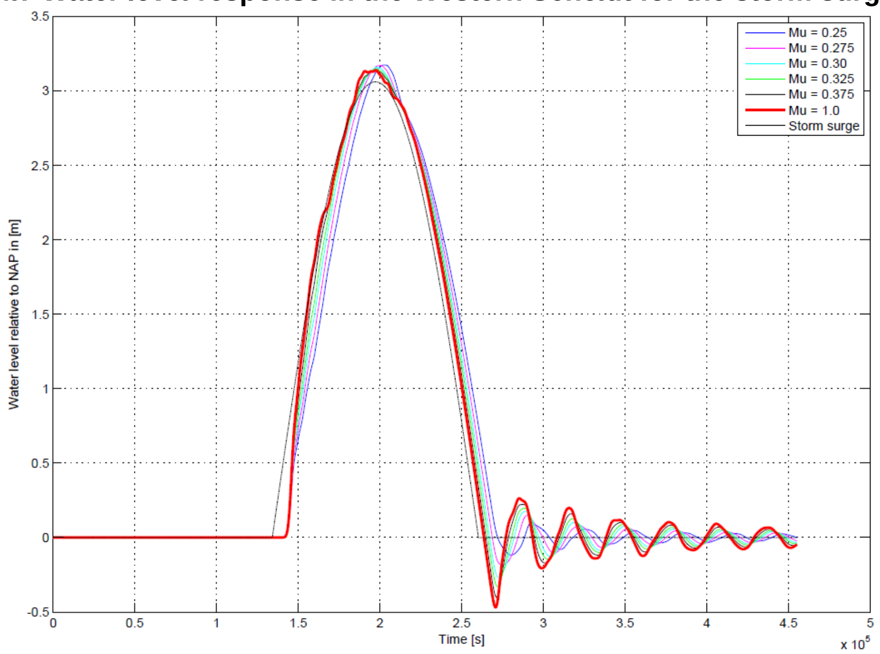
Figure 4.6 Reduction of the water level in the Western Scheldt for the spring tide



4.4.2. Calculation results storm surge

The effect of the barrier on the storm surge wave is presented in Figure 4.7. The storm surge is modelled as the positive half of a sinus. The duration of the storm surge is 35 hours. It is clear that the reduction barrier has no effect on the storm surge. The reason is the large time scale of the storm surge. The irregularities after the storm surge have to do with the sharp transition in the sea water level. This does not influence the results since the relevant value is the maximum water level.

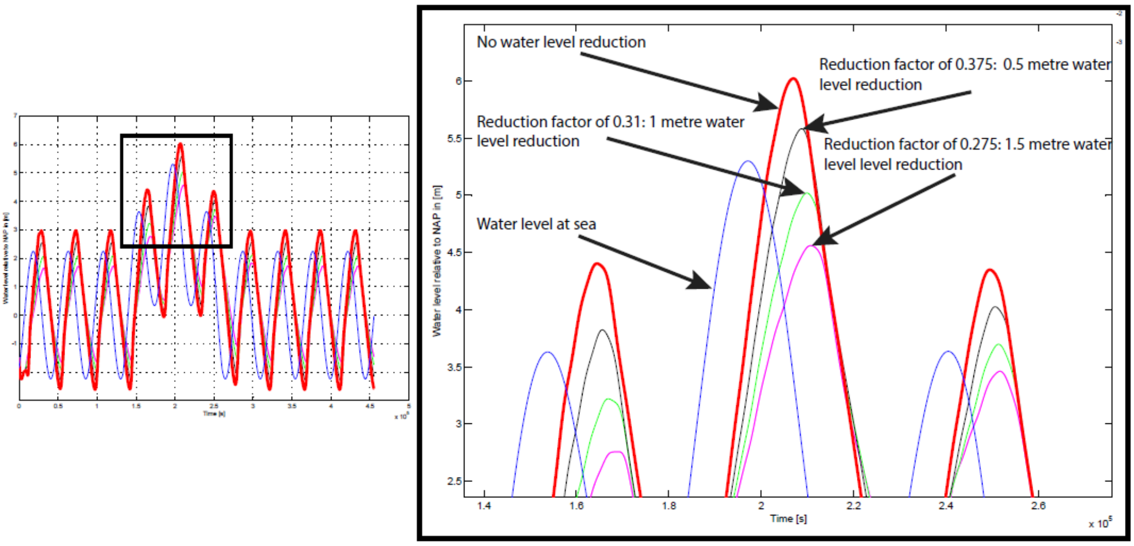
Figure 4.7 Water level response in the Western Scheldt for the storm surge wave



4.4.3. Calculation results storm surge and tide combined

The figure below shows the effect of the barrier in case of a storm situation. The absolute value of the water level near Antwerp should not be obtained from the graph above, since the local wind set up and other effects are not included. The graph should be used to determine the difference in water level near Antwerp for a certain reduction factor relative to the current situation. The red line represents the water level in the Western Scheldt near Antwerp. The reduction factor is the factor with which the discharge area is multiplied. So a reduction factor of 0.4 means that the barrier closes off 60% of the discharge area and 40% remains open. The black, green and purple line indicate the effect on the water level of the corresponding reduction factor.

Figure 4.8 Reduction of the water level in the Western Scheldt with storm surge



The local wind set up was assumed to be not influenced. The magnitude and the duration of the local wind set up is such that there will be no difference with or without the barrier.

4.4.4. Conclusions based on the numerical calculations

The situation during storm conditions shows that the reduction factor of 0.375 is able to reduce the water level by 0.5 metre. So the first situation during storm conditions can be achieved without installing moveable gates. The second and third storm situation requires a reduction factor of respectively 0.31 and 0.275. So in order to guarantee the safety for 100 years with sea level rise the barrier must be able to reduce the opening in storm conditions from a reduction factor of 0.375 to 0.275.

The flow velocity during storm conditions is largest for a reduction factor of 0.375 at 2.8 m/s. The water level difference over the barrier is largest for a reduction factor of 0.275 at 2.6 metre.

Table 4.6 Maximum size of the openings in the barrier

Reduction factor	[-]	0.375	0.31	0.275
Original discharge cross-section	[m^2]	70000	70000	70000
Maximum opening size	[m^2]	26250	21700	19250

Note that the effect of the reduction factor is very sensitive between 0.4 - 0.2. Small changes in the reduction factor can lead to large changes in the maximum water level. While for reduction factors between 1.0 - 0.5 the maximum water level is almost unaffected.

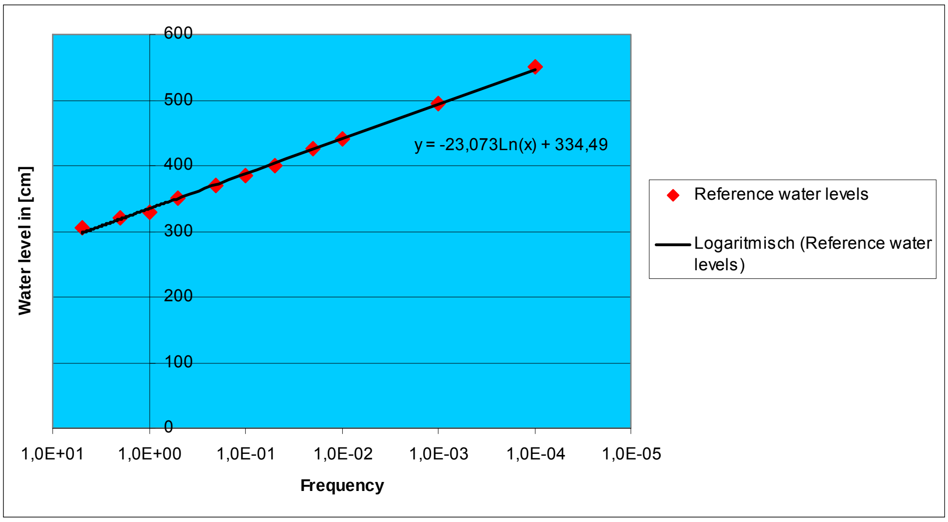
4.4.5. Remarks about the numerical calculation

The numerical calculation solves the entire motion equation. It is therefore more accurate than the 1D-model as described in chapter 3. The resistance is for example not linearized. The properties of the Western Scheldt are also more accurate described for every section. However the channels are still assumed to be prismatic, which means that the width does not change with the rising or falling water level. The effects of the higher order tides are not included in the boundary conditions nor are any 2D and 3D effects. Nevertheless, the results are comparable with the measured water levels and the reduction of the normal tide corresponds with the results of the 1D-model calculations in chapter 3.

The tidal prism is reduced with a factor 0.69 for discharge reduction factor of 0.31. The tidal prism is reduced with a factor 0.64 for the discharge reduction factor of 0.275. All these values are calculated for the normal conditions.

The barrier reduces the maximum water level with 0.5 metre in all conditions. The closing regime of the moveable gates depends on sea level rise. If the sea level rises with 0.5 metre some of the gates will have to be closed in case a water level of 4.8 metre above average sea level is expected. In case the sea level rises with 1 metre, the gates have to be closed with an expected water level of 4.3 metre above average sea water level. This means that the gates will be closed more frequently as the sea level rises. The closing frequency is 1/63th per year for 1.5 metre reduction. Assuming the water level frequencies do not change with respect to the average water level.

Figure 4.9 Reference water levels near Vlissingen



The closing frequency of the gates is very low. It might be more costs effective to close some openings permanently instead of constructing gates. The impact on the environment must be less than the costs of the gates in order for this to be feasible. The costs or the value of the environment are extremely difficult to determine.

4.5. Impact of shipping on the barrier design

The Western Scheldt is part of the approach channel for four main ports, which are the Port of Antwerp, Port of Gent, Port of Terneuzen and the Port of Vlissingen. The largest vessels and the largest amount of vessels have the Port of Antwerp as their destination. The vessel sizes which are used for the calculation of the channel width are presented in Table 4.7. The average vessel is larger than the current average vessel, since the vessel size is expected to grow in the future.

Table 4.7 Average and maximum vessel size that pass the barrier

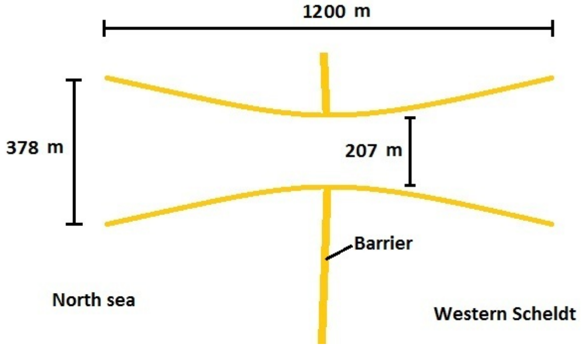
	Dimension	Average vessel	Maximum vessel
Length	m	240	397
Width	m	30	56
Draft	m	11.5	16
DWT	ton	30 000	157 000

4.5.1. Width of the shipping opening

The required width is calculated by assuming the approach channel guideline is applicable to the barrier. The approach channel guideline has been made for approach channel which end in a port. This is not exactly the case here since the port is much further away. However the guideline does give a suitable estimation of the width which required for sea going vessels. A more accurate determination of the acceptable width is subject for further investigations.

It is assumed that the average vessels have to have a two lane opening while for the maximum vessel one lane is sufficient. The amount of vessels that has to pass the barrier has been stable over the last couple of decades and is about 140 vessels per day. For the two lane situation for the average vessel the minimum width of the shipping channel is 378 metre and for the one lane situation for the maximum vessel is 308 metre (Ligteringen, 2009). This width can be reduced to 207 metre, by introducing funnel structures which eliminate the cross currents and waves, for an example see Figure 4.10. The length of the funnel structure needs to be long enough, since the vessels need to be able to adjust their course to the less severe conditions. This length is about 2.5 times the vessel length. In case the funnel structure is made shorter, the minimal width of the opening needs to be somewhere between 378 and 207 metre. It is possible to reduce the waves by selecting a sheltered location of the opening.

Figure 4.10 Minimum dimensions funnel structure for shipping



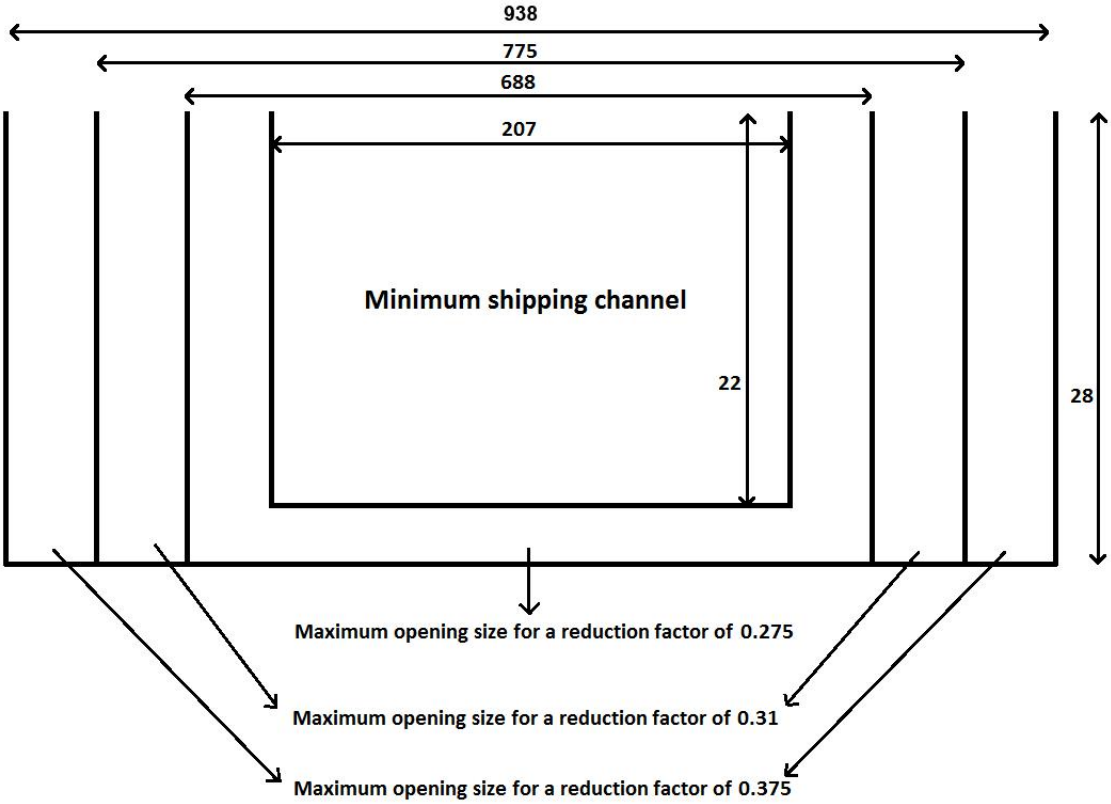
The minimum water depth for the largest vessels is calculated to be 22 metre (Ligteringen, 2009). This depth guarantees the accessibility of the Western Scheldt even during spring low water. This is deeper than the Western Scheldt is at certain locations, because the barrier may not become the bottle neck with regard to shipping.

The dimensions of the shipping channel result in an opening of 22 * 378, without funnel structure, 22 * 207 with funnel structure. This means that the reduction of the water level of

1.5 metre still enables vessels to pass the barrier, since the maximum opening in the barrier for a reduction of 1.5 metre is 688 * 28, see Figure 4.11. A more narrow opening might require the aid of pilots and other restrictions like vessel speed or tug assistance.

In the figure below is indicated how the barrier openings for the aimed reduction correspond to the required cross-section for the shipping channel. The shipping channel can fit within the smallest opening. The additional width for the smaller reduction scenarios can be used to provide an additional shipping lane for smaller vessels as well as openings in the barrier at the locations of the other channels in the estuary mouth. Changes in the morphology are less likely when maintaining the current flow pattern.

Figure 4.11 Overview of the opening cross-sections boundaries



An offshore company is located in the port of Vlissingen. This company works on for example drilling rigs and oil platforms. These rigs have a larger width than the vessels. However it is expected that the platform have a smaller width than 207 metre. These rigs can wait for optimal circumstances for the passage of the barrier, for example at very low flow velocities. It is assumed, for now, that these platforms are able to pass the barrier.

4.5.2. Remarks about shipping in storm conditions

Wind sensitive vessels are not allowed to sail on the Western Scheldt in case the windspeed exceeds 7 Beaufort. Wind sensitive vessels are for example car carriers. Also vessels with a large draft are not allowed in case of 7 Beaufort or more. The pilot service remains in operation until 9 Beaufort and if the wave height remains below 3.5 metre. Manoeuvring simulations show that it is very difficult to moor container vessels in wind speeds exceeding 8 Beaufort. This means that it is very unlikely that cargo vessels want to enter the Western Scheldt during design storm conditions, since the design storm conditions are 11 or 12 Beaufort.

4.5.3. Water level difference over the barrier

The vessels have to overcome a water level difference at the location of the barrier. The water level difference should be as gradual as possible. Vessels are better able to pass a gradual increase in water level and flow velocity. This can be achieved by increasing the width of the barrier. During normal conditions the maximum flow velocity is 2.4 m/s, which leads to a water level difference of 0.9 metre. The slope on a river is about 0.1‰. When assuming a maximum slope of 1‰, the width of the barrier has to be about 900 metre, so an additional 450 metre on either side of the barrier, see appendix III for implications for the required vessel power. The demand with regard to manoeuvrability of the vessels results in a slightly larger width.

The flow velocities during storm conditions do not cause trouble for the vessels. Sea going vessels are capable of travel much faster than the flow velocities in the openings, for instance; the flow velocities reach 7 knots or 3.6 m/s at the river Mersey in Liverpool. Sea going vessels can enter the river Mersey without assistance. The flow velocities in the opening of the reduction barrier are in the same order of magnitude. The sea going vessels do not experience any trouble as long as they can move parallel to the flow direction and the steepness is limited.

The load on the barrier itself is larger when vessels are allowed to pass during storm conditions. The larger loads require a stronger structure, which is more expensive. The costs for blocking the route is about 0.7 million per day for the Port of Antwerp (Muntinga, 2010). These costs are caused by the additional labour fees which are required to tranship the cargo after a blockage. It is assumed that an open shipping route is more valuable than the costs for a stronger barrier.

4.5.4. Water level depression in the funnel structure

Vessels cause a water level depression when entering the funnel. The amount of water level depression depends on the speed at which a vessel travels. The funnel structure must be able to handle this additional water level difference. The bed protection must be checked for this load as well. The return current is about 1.8 m/s. So the total flow velocity becomes 4.6 m/s.

It is recommended to train pilots to navigate through the opening by slowly increasing the flow velocity in the opening. A load reducing measure can be to allow passage of the barrier only during periods of low flow velocity. Extensive fast- and real-time simulations for vessel manoeuvrability are required when the reduction barrier appears to be feasible.

4.6. Wave conditions

The significant wave height for the levees near Vlissingen is 1.6 metre and for the levees near Breskens is 1.75 metre (Hydraulische Randvoorwaarden Primaire Waterkeringen, 2007). The larger value on the South side is because of the less favourable location in the governing wind conditions. The wave height can be measured using buoys, since the complex situation needs to be monitored for a long time to get reliable results. The estuary mouth has a lot of diffraction, reflections and shoaling, which makes the wave height prediction difficult. The barrier is located in such a way that it is protected from the very high waves from sea. The shoal in the estuary mouth will reduce the wave height, but an accurate wave height can only be determined by modelling the entire estuary mouth.

4.6.1. Significant wave height

The wave height in the middle of the Western Scheldt is unknown. However it is expected that the wave height is larger than previously mentioned wave height of 1.75 metre. In case the wind direction is due west, the fetch is over 150 kilometre. The shoal in the estuary mouth will cause the largest waves to break. It is expected that the wave height in the middle of the Western Scheldt can be twice as large as the maximum wave height for the levees. The wave height of 3.5 metre corresponds with the maximum wave height for the water depth on the shoal. The significant wave height is about 6 metre in case there is no shoal. The wave period is assumed to be 9 seconds. The exact wave climate should be determined by modelling the entire estuary mouth, as mentioned before.

4.6.2. Wave overtopping

The waves can overtop the barrier which leads to additional discharge into the estuary. A large amount of overtopping would reduce the opening in the barrier during storm conditions. Since the length of the barrier is considerable, the overtopping per running metre is limited.

The maximum allowed water level rise due to wave overtopping is set to 0.1 metre. This leads to a maximum overtopping discharge of 0.27 $m^3/s/m^1$, which results in a wave overtopping height of 2.7 metre (TAW, 2003), see appendix III. The top of the barrier should have a height of 9.5 metre when a settlement of 0.5 metre and sea level rise of 1 metre is taken into account.

4.7. Soil conditions

The soil conditions are important for the foundation of the structure. In the end the forces on the structure have to be transferred to the subsoil. The soil conditions were improved before the construction of the Eastern Scheldt barrier, so it is likely that the soil conditions are not excellent in the Western Scheldt either. The conditions of the soil may force the design in a certain foundation method or massive soil improvement measures have to be performed in the Western Scheldt as well.

The soil is composed of layers of marine sand and clay layers. In the appendix III can be seen that there is a sand layer which starts at -25 metre NAP. The layers can differ over the cross-section of the Western Scheldt due to the continuously changing locations of the channels during history. However the presence of sand layers is quite certain only the depth can change of the layer can change. The graphs from the Breskens are used for the design, since the graphs cover a larger depth, it is assumed that those conditions are valid for the entire location of the barrier. In a later stage more accurate soil conditions can be determined.

4.8. Terms of References

The barrier has to fulfil several requirements in order to achieve its goal. These requirements follow from the analysis in the previous section and are summarised below. The requirements are followed by the boundary conditions and assumptions.

4.8.1. Functional requirements

- The barrier must be able to resist a storm with a probability 1/4000th per year
- The barrier must be able to reduce the water level on the border with Belgium with 0.5 metre
- It must be possible to upgrade the barrier for increased reduction of 1 and 1.5 metre
- The barrier must be able to function after a ship collision with the funnel structure
- It must be possible to close and open the barrier when there is a water level difference over the barrier
- Vessels must be able to safely pass the barrier during normal conditions

- The barrier may not increase the flow velocity to more than 3.5 m/s at the cross-section of the barrier during normal conditions
- The barrier must be able to close and open relatively quickly, which means a closing opening time of less than two hours
- The minimum required opening size of the shipping channel is 22 * 378 metre, when a funnel is applied the width can be reduced to a minimum 207 metre
- The barrier must be relatively easy to maintain
- Inspections must be possible with relative ease
- The reduction factor of the tidal prism must be equal or larger than 0.8 in order to maintain sediment transport
- The barrier must be executed in such a way that it can contribute to the environmental value of the area

4.8.2. Technical requirements

- The barrier must be able to resist a positive and negative water level differences
- Design water level at North Sea is 5.30 m + NAP
- The barrier must have a height of at least 9.5 metre + NAP
- The barrier must have a life time of 100 years
- The design water level difference over the barrier is 2.6 metre (positive water level difference)
- The negative water level difference of 2.6 is assumed to be the maximum
- Design discharge of the river Scheldt is 300 m^3/s
- The significant wave height is 3.5 metre

4.8.3. Boundary conditions

- The geographical locations of the channels as presented in Figure 4.4
- The possibility for power generation is not further investigated

4.8.4. Assumptions

- For the average vessels two way traffic is applied and for the maximum size vessel one way traffic is sufficient
- Vessels should pass the barrier during normal conditions and in storm conditions during periods of small flow velocities
- There is no sill placed in the channels to facilitate sediment transport
- The barrier does not influence the water level at sea during storm conditions
- The water level rise is assumed to be 1.0 metre in 2120, assuming the construction of the barrier can be finished in 2020
- The leakage through barrier is taken into account in the total allowed discharge through the barrier
- 2000 m^2 of the total allowed discharge area is applied for leakage
- The maximum allowed opening size for a reduction factor of the discharge cross-section of 0.375 is 24 250 m^2 taking into account additional leakage
- The maximum allowed opening size for a reduction factor of the discharge cross-section of 0.31 is 19 700 m^2 taking into account additional leakage
- The maximum allowed opening size for a reduction factor of the discharge cross-section of 0.275 is 17 250 m^2 taking into account additional leakage
- The soil conditions as presented in Figure III. 11 and Figure III. 12 are applicable over the entire length of the barrier
- The barrier does not have to be part of the road network, since the shipping opening has to have unlimited vertical clearance and the Westerscheldetunnel is located close by and has sufficient capacity

4.9. Remarks about the analysis

The water motion calculation shows that the demand for environment is governing for the normal situation. This demand is based on an assumption. In case the reduction factor for the tidal discharge can be smaller without harming the environment, the permanently closed part of the barrier can be larger. So this result in more reduction of the water level and reduces the amount of moveable gates. In case the reduction factor for the environment must be larger in order not to harm the environment, the amount of moveable gates has to increase. This will lead to a barrier that is more like a storm surge barrier instead of a reduction barrier. This will also affect the construction and maintenance costs negatively.

5. DESIGN OF THE BARRIER

This is the second chapter about the Case Study of the Western Scheldt. The design of the barrier is based on the analysis of the Western Scheldt in the previous chapter. The first section is about the layout of the barrier, which follows mainly from the, in the previous chapter, calculated opening sizes. The second section explains the main design choices in the rough design. This includes the selection of the gate type for the moveable parts of the barrier. Section 3 is about the design of the main components of the barrier. The dimensions of the main elements are required for the costs calculation in the next chapter.

5.1. Lay out of the barrier

All alternatives have the same basic lay out. The main shipping channel is located on the North side of the barrier near Vlissingen. The minimum dimensions of this channel have been discussed in the previous chapter. For optimal practicality is decided to make opening on the South side of the barrier as well. This opening consists of three channels of about 60 metre wide and 10 metre deep and one channel of 140 metre wide and 15 metre deep. The three channels of 60 metre width can be closed with gates when the reduction needs to be increased to 1.5 metre. The 140 metre wide channel can be used by the smaller vessels and pleasure craft. Smaller vessels can avoid the large shipping channel for the passage of the barrier. This will improve safety and reduce the hindrance of the barrier for commercial vessels. The width of 140 metre is sufficient to let vessels up to 25 metre width pass.

For an impression of the new cross-section at the location of the barrier is referred to Figure 5.1. The permanently open channels are blue and the variable channels are green. The permanently closed dam is orange. At the locations of the shipping channels is chosen for one large opening. In the middle of the barrier is chosen for 8 narrower openings over a larger width. In this way is the flow in and out of the estuary less concentrated, which is considered to be better for the morphology.

The area of the channels is not the total maximum opening size, since 2000 m^2 of opening is reserved for the leakage around the moveable elements and through the dam. If the leakage turns out to be less, the shipping channels could be widened.

Figure 5.1 Longitudinal cross-section of the barrier [m]

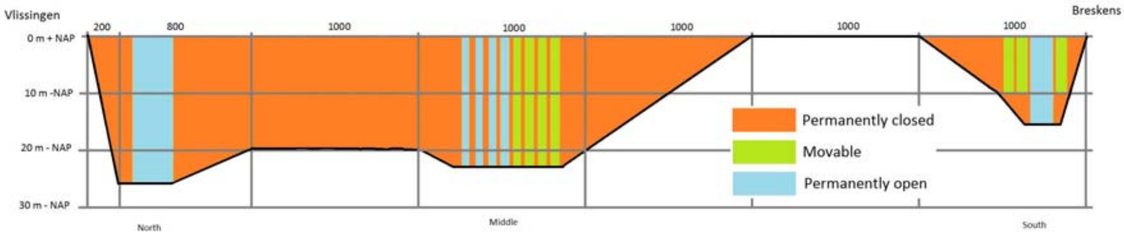


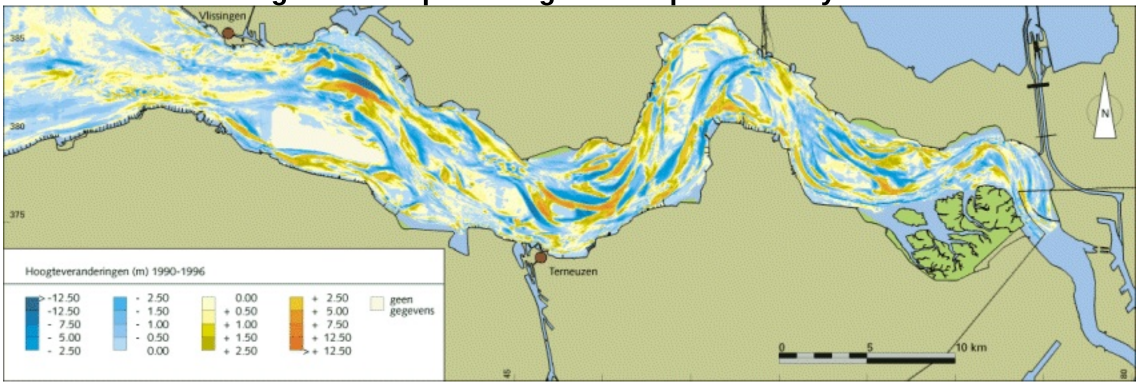
Table 5.1 Dimensions openings in barrier for the different amounts of reduction

	North channel	Middle channels	South channels	Total
Reduction of 0.5 metre	374 * 25	(4 * 50 + 4 * 60)* 25	140 * 15 + (55 + 2 * 60) * 10	24 200 m^2
Reduction of 1.0 metre	374 * 25	(3 * 50 + 3 * 60)* 25	140 * 15	19 700 m^2
Reduction of 1.5 metre	374 * 25	(3 * 50 + 1 * 60)* 25	140 * 15 + 55 * 10	17 225 m^2

The openings in the barrier are chosen in such a way that the discharge will take place at the location of the current channels. It is expected that in this way large morphological changes are prevented. The moveable parts of the middle section are placed all on the left side in order to be able to reach all gates with relative ease. The impact on the morphology of the closed gates is expected to be limited, since the duration of the closure is very limited, in relation with the duration of the open stage.

The change of the channel system between the barrier and Antwerp is difficult to determine. The system is very dynamic, as can be seen in Figure 5.2. This figure shows that the channels cannot be fixated. The morphological changes at the location of the barrier are limited, but the barrier can distort the equilibrium. The impact of the barrier on the morphology is a subject for further investigation. The minimal change of the bed in the mouth does not mean that there is no sediment transport at that location, however the majority of the transport happens further upstream.

Figure 5.2 Depth change over a period of 6 years



It is possible not to make a gate for the South channel with a width of 55 metre, since this gate is not required for the final situation. However an extra gate can improve flexibility and redundancy of the barrier. Therefore is decided to install a gate in the 55 metre channel as well. Further investigation may reveal that the gate is not necessary or too expensive.

5.2. Rough design of the barrier

There are three basic design choices that should be made for the design of the barrier. These choices are the related to the design of the permanently closed part, the moveable part and the funnel structure. The choices are discussed below.

5.2.1. Design of the permanently closed parts

The permanently closed part of the barrier can be made of two types of structures which are caissons and rubble mound. Caissons require more labour, less material and a stronger foundation. The caissons require accurate construction and installation. The advantage of the caisson is that very deep and narrow channels are possible. The rubble mound requires a lot of material, but it is relatively easy to construct. However the rubble mound is expected to be more expensive in construction than the caisson but still much cheaper than the moveable gates of a storm surge barrier. The rubble mound is more redundant, since it is more flexible than the caissons. The rubble mound barrier can be increased width and height without much trouble. Maintenance on the rubble mound is much better possible than with the caissons. The rubble mound dam is more attractive from an environmental point of view as well. The dam can initiate the growth of shells and other under water animals, like some sort of reef. The barrier is an ideal place to hide from large predators. The rubble mound is more open, so there will flow water through the dam, but over the course of time, the vegetation and sediments will block most of the flow through the dam. This is observed in the sill of the Eastern Scheldt barrier (Visser, 2003). By creating rough surfaces on the elements, in case artificial elements are used, the marine growth can be accelerated. The marine growth will attract animals like birds. In this way the environmental impact can be, partially, compensated.

In order to exploit the benefits of both solutions is decided to construct the barrier of a combination of both structures. The caissons are required to create the deep channels and for example the transfer to the funnel structure. The rubble mound dam is used to fill in the remaining stretches. The transition between the caissons and the rubble mound dam requires attention.

5.2.2. Design of the funnel structure

The funnel structure can also be made of rubble mound or caissons. However the rubble mound will require too much space which is not available. Therefore is decided to use caisson like structures to construct the funnel structure. The caissons also prevent the vessels from hitting the actual barrier. It is allowed that the caissons sustain damage when hit, as long as the vessels are not able to reach the barrier structure. It is expected that the size and weight of the caisson is large enough to prevent the vessels from the reaching the barrier.

5.2.3. Design of the moveable gates

The choice for the moveable gates cannot be made as easily as the previous two. There are a lot more possibilities. The possibilities are described below. The moveable gates are used to restrict the discharge cross-section during storm conditions. The gate with the highest value is selected by means of a MCA in the next subsection.

Rising sector gate

This type of gate is applied in the Thames Barrier in London and the Ems Barrier in the Ems in Germany. The gates are usually positioned at the bottom and are rotated into a vertical position during storm surges. The gates can be rotated in such a way that they are completely above water. This feature can be use to apply maintenance. The length of the gate is limited to about 80-100 metre.

Figure 5.3 Rising sector gate

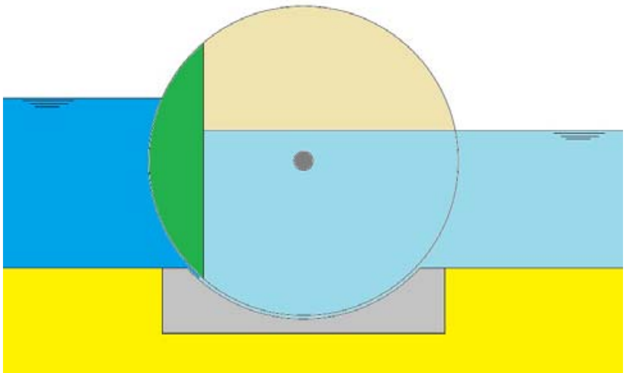


Table 5.2 Evaluation rising sector gate

Advantages	Disadvantages
Easy maintenance	Deep structure
Applied technology	Relatively complicated structures
Inconspicuousness	

Inflatable barrier

An inflatable barrier is a barrier which consists of a rubber sheet which connected to a concrete sill and can be inflated with both water and air or a combination both. The largest inflatable barrier is the barrier near Kampen and has a retaining height of about 10 metre.

The water level difference over the barrier is about 4.5 metre. For application in the Western Scheldt serious scaling is required, since the height will have to measure over 25 metre. An advantage is the small water level difference over the barrier in the Western Scheldt. The system can be applied over a large length however the size of the rubber elements is not unlimited. So it is likely that the Western Scheldt can only be closed off using elements of about 150 metre length. The rubber sheet lies on the bottom in normal situations. This can lead to serious wear due to sediment transport.

The scaling results in much larger tension force in the rubber sheet and also in much larger peak stresses. The rubber sheet is composed of several strips, of about 3 metre wide, joined together. The strength of the rubber sheet depends on the number of layers nylon or aramid reinforcement and thus on the thickness of the rubber sheet. It is possible to make very strong rubber strips, for example for conveyor belts. However it is very difficult to connect these strips in order to make a sheet. The application of the aramid makes the sheet besides stronger also stiffer, which means that the peak stresses also increase. The current state of the art is the Ramspol barrier. Larger barrier require innovation in the fabrication of the rubber sheet.

Figure 5.4 Cross-section inflatable barrier

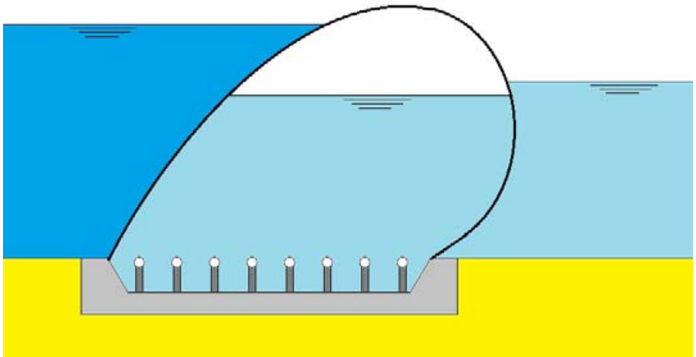


Table 5.3 Evaluation inflatable barrier

Advantages	Disadvantages
Easy maintenance	High safety factors for the rubber sheet
Applied technology	Accumulation of sand in the sheet in the open state
Functions in both directions	Manufacturing sheet requires innovation

Hydraulic flap barrier

The flap barrier can function with two different mechanisms. The first is with the use of hydraulic rams. The hydraulic rams make the system relatively quick. The hydraulic rams lie beneath the gate during normal situation. The gates protect the rams from wear caused by sediment transport. However sediment can accumulate behind the barrier in open state, which makes the lowering of the gate difficult or even impossible. The hydraulic rams need quite a lot space in the structure in the lowered position. The mechanical equipment is all located under water which is a disadvantage with regard to maintenance and emergency repairs.

Figure 5.5 Hydraulic flap gate

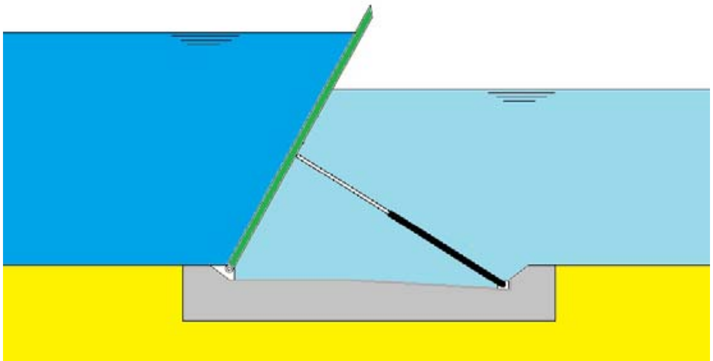


Table 5.4 Evaluation hydraulic flap gate

Advantages	Disadvantages
Applied technology	Accumulation of sand under the gate during closing procedure
Functions in both directions	Maintenance
High open and close speed	

Floating flap barrier

The floating flap barrier uses the buoyancy of the gate to make it rise. The gates lie on the bottom filled with water under normal circumstances, but during storm the water is pumped out and replaced with air which causes the gate to rise. The flaps can move independently, which makes it flexible for the absorption of wave impact, but the flexibility results in more leakage through the structure. The system is being applied in the barrier which will protect Venice. The system can only work in one direction, but it can be lowered and raised very quickly. So in theory it is possible to lower the barrier in case the water level difference approaches zero and raise it again when there is a positive water level difference, but this creates many practical issues. The growth of vegetation and shells in the gate is another point concern. This has to be prevented in order to guarantee the functioning of the gate.

Figure 5.6 Floating flap gate

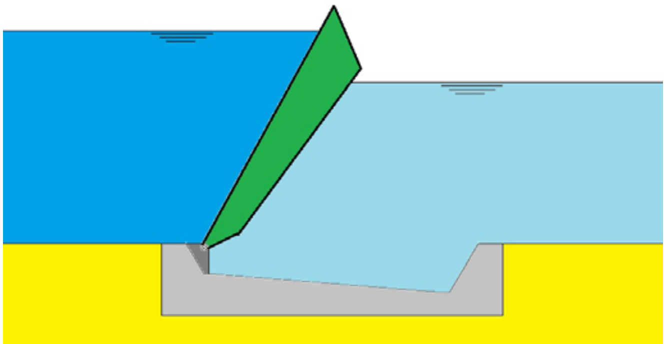


Table 5.5 Evaluation floating flap gate

Advantages	Disadvantages
High open and close speed	Accumulation of sand behind the gate in close state
Efficient for relative small water level differences	Does not function in both ways
	Expensive

Sliding door

The sliding door is a system which is applied regularly in locks. The doors can be very heavy and thus strong. The door requires a lot of space since the door is positioned next to the channel in the open conditions. It is possible to design curved gates, which should save space and increase the load bearing capacity. The door is difficult to close when there is a water level difference over the door. The rail over which the door slides has to be very stable and clean. This can be a problem with regard to sediment transport and marine growth. The doors are capable of spanning about 60 metre.

Figure 5.7 Cross-section sliding door

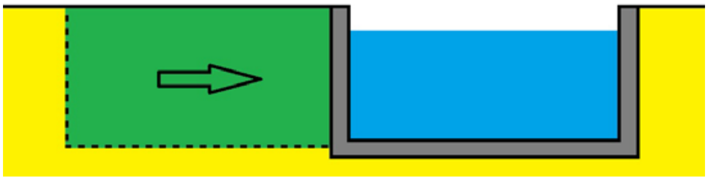


Table 5.6 Evaluation sliding door

Advantages	Disadvantages
Strong and heavy doors	Required space
Inconspicuous	Does not close in case of water level difference
Field experience	A large part of the construction needs to done under water

Vertical lift gate

This type of gate has been applied in the Eastern Scheldt barrier with success. The gate can span a large length. The gates are stored above water which makes them easily accessible for inspection and maintenance. The gates are lowered into position, which means that no power is required to close the gates. This is a mayor advantage with regard to reliability. The gates can be closed in case of water level difference over the barrier, although the gates are susceptible for vibrations. A downside of these gates is that they can be considered as destruction of the landscape. The gates could form an obstacle for shipping, however a separate shipping channel is available in this situation.

Figure 5.8 Vertical lift gates

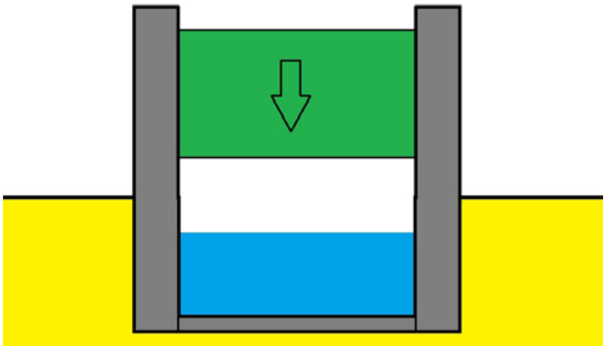


Table 5.7 Evaluation vertical lift gate

Advantages	Disadvantages
Required space	Destruction of the landscape
Maintenance and inspection	
Field experience	

Visor gate
Visor gates are applied in the Netherlands as weirs in rivers, near the villages Driel, Maurik and Hagestein. However the principle can function as a barrier as well. The visor gates are material efficient and they have spans of about 45 metre. The visors are quite large and they will destruct the landscape in upright positions. They can be closed and opened when there is a water level difference. Large part of the visor is above water in the opened situation. However there are also part under water which is a disadvantage for inspection and maintenance.

Figure 5.9 Visor gate

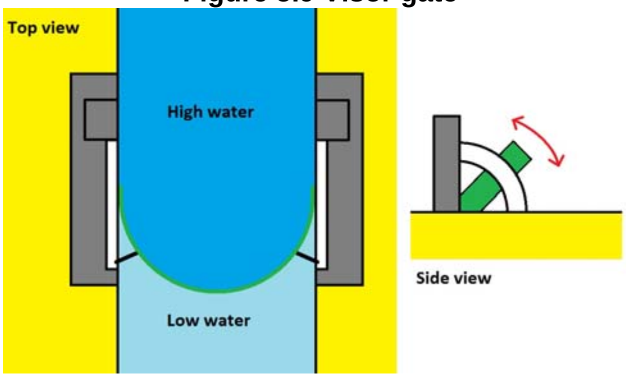


Table 5.8 Evaluation visor gate

Advantages	Disadvantages
Material efficient	Destruction of the landscape
Field experience	Not possible for large spans
	Maintenance and inspection

Radial gate
The radial gates are applied in the Haringvliet dam. The loads delivered in full compression or tension towards the supporting axis. The gates can be closed over a water level difference, although vibrations can be an issue. The gates are stored above water which is advantageous for inspection and maintenance. The gates will cause some destruction of the landscape however not as much as the vertical lift gates.

Figure 5.10 Radial gate

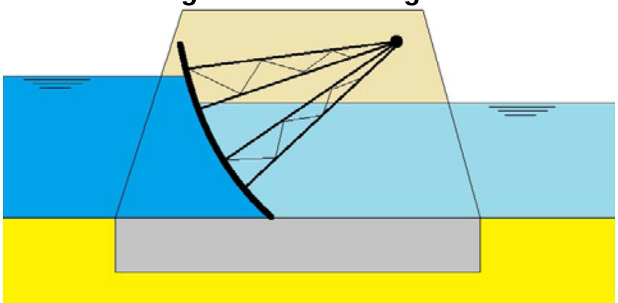


Table 5.9 Evaluation radial gate

Advantages	Disadvantages
Material efficient	Destruction of the landscape
Maintenance and inspection	Stiff support axis is required
Field experience	

Mitre gates
Mitre gates are applied primarily in navigation locks, so there is a lot of experience. The span which can be reached is not very large. The door can only function in one way, which means that two sets of doors will have to be installed or the doors should open with high water in the Western Scheldt. The doors are always partly under water which makes inspection and maintenance more difficult. On the other hand, the closing procedure is relatively simple.

Figure 5.11 Top view of mitre gates

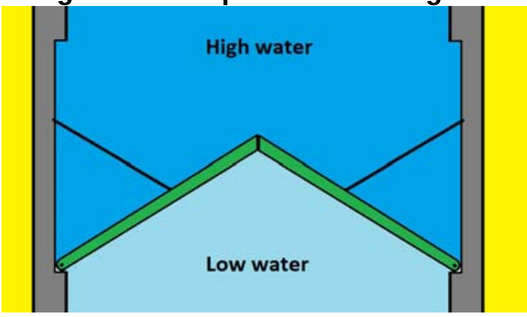


Table 5.10 Evaluation mitre gate

Advantages	Disadvantages
Field experience	Small spans
	Maintenance and inspection
	Functions in one way

5.2.4. Multi Criteria Analysis for the selection of the gates
The possible solutions for the gates are compared using the requirements which are described in the Terms of References and some more general criteria. The criteria are given a relative value. The score of the type of solution times the relative value of the criterion is the number of points this solution gets for the considered criterion. The solution with the largest total number of points is the most valuable option. Since the costs of the different solutions is assumed to be the same; € 30 000 /m^3, the value is decisive for the decision making. The tables with all the given scores can be found in appendix IV.

Table 5.11 Multi criteria analysis

	Rising sector gate	Inflatable barrier	Hydraulic flap gate	Floating flap gate	Sliding gate	Vertcal lift gate	Visor gate	Radial gate	Mitre gates
Maintainability	16	8	4	4	8	20	20	20	8
Opening and closing time	8	4	10	10	6	10	10	10	8
Redundancy	21	7	14	14	21	28	21	28	28
Required space	5	4	4	3	1	4	3	5	5
Constructability	8	4	4	4	12	20	12	16	12
Inspection possibilities	24	12	6	6	12	30	24	30	12
Field experience	12	9	6	6	12	15	12	12	15
Inconspicuousness	4	5	5	5	4	2	1	3	4
	98	53	53	52	76	129	103	124	92

Conclusions about the type of gate
The vertical lift gates have the highest value and the radial gates are second. The reason these options win because of the high scores on inspection and maintenance. The vertical lift gates are chosen because of the highest score and their ability to cross large spans.

The Multi Criteria Analysis shows that the vertical lift gates have the largest benefits. The sensitivity of the analysis lies with the grading of the alternatives over the criteria. Differences in the importance of the criteria do not result in a different choice. The main advantages are the accessibility of the components and that the gates can be lowered into place without the use of electrical power.

Material selection for the gates
There is a lot of experience with such gates. This type of gate is applied in the Eastern Scheldt barrier, Hartel barrier, Stormvloedkering Hollandse IJssel and numerous locks around the world. The span of 60 metre is not considered to be a problem.

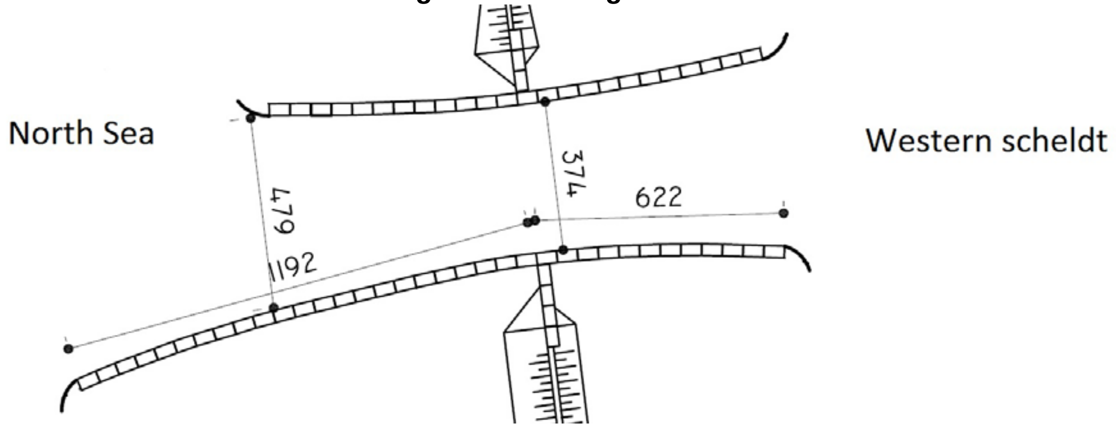
The Eastern Scheldt barrier has some problems with the maintenance of the steel gates. This is caused by poor initial coatings. Steel is a very suitable construction material however the sensitivity to corrosion is a problem. The protection of steel can be done by for example coating. It is not required to install the gates immediately after construction of the barrier. So the life time of the steel gates does not have to be as long as the life time of the barrier. It is difficult to predict the moment on which the gates are required. It is even more difficult to predict the state of the technology at that moment. It is possible that, by then, most gates are constructed from glass fibre or carbon fibre. However it is also possible that these materials turned out to be not as good as steel. It is also possible that coatings have been developed that can protect steel for a very long time. For this thesis is decided to design the gates of steel, since this is proven technology. In this way can be shown that the barrier is technically feasible. A downside of steel gates is that the coating of the gates is allowed to get into the environment. This makes the maintenance of the steel and the coating very expensive. Fibre reinforced plastics are expected to be cheaper in maintenance and could turn out to be a better overall choice.

5.3. Design of the main components
In this section the main components of the dam are designed and the design choices are explained. The dimensions and the amount of material required is essential input for the construction planning and the costs estimation in the upcoming chapter. The main aspects that are discussed are the funnel, bed protection, caissons, rubble mound and the gates.

5.3.1. Lay out of the funnel
In order to increase the navigability, the shape of the funnel is not symmetrical. The funnel has one longer arm at the sea side. The length of the south side of the funnel is twice as large. In this way the opening in the barrier is better accessible during rough conditions. The Western Scheldt side does not require such an extension because the conditions are more calm and the vessel speed is lower.

The funnel needs to be equipped with lights and radar beacons so the vessels can see the barrier and the shipping funnel in all circumstances. The visibility of the shipping channel is very important for the safety.

Figure 5.12 Design funnel



The funnel structure may seem narrow for sea going vessel, but the Western Scheldt gets narrower further upstream. The shipping channel near Bath has a width of 330 metre in combination with a bend. This shipping channel has been indicated by buoys. Nevertheless the funnel can be considered to be an obstacle. The captains and pilots can be trained to pass the barrier by proper construction phasing and by providing the assistance of tugs. Average vessels and a maximum vessel have been drawn in the shipping funnel in order to give an impression of the opening size.

Figure 5.13 Impression of the funnel with maximum vessel size

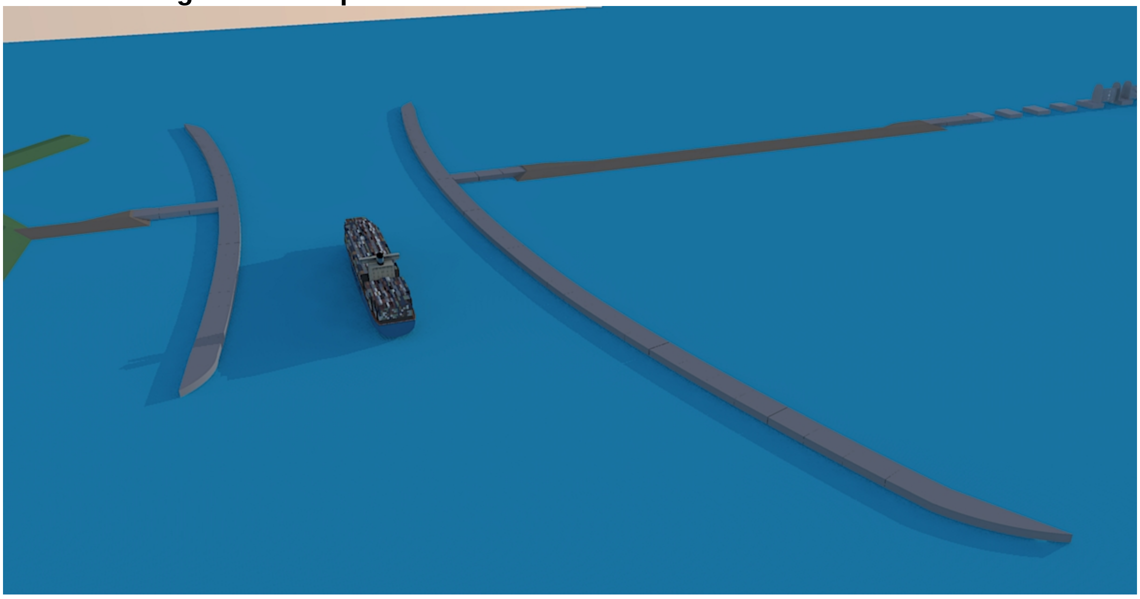
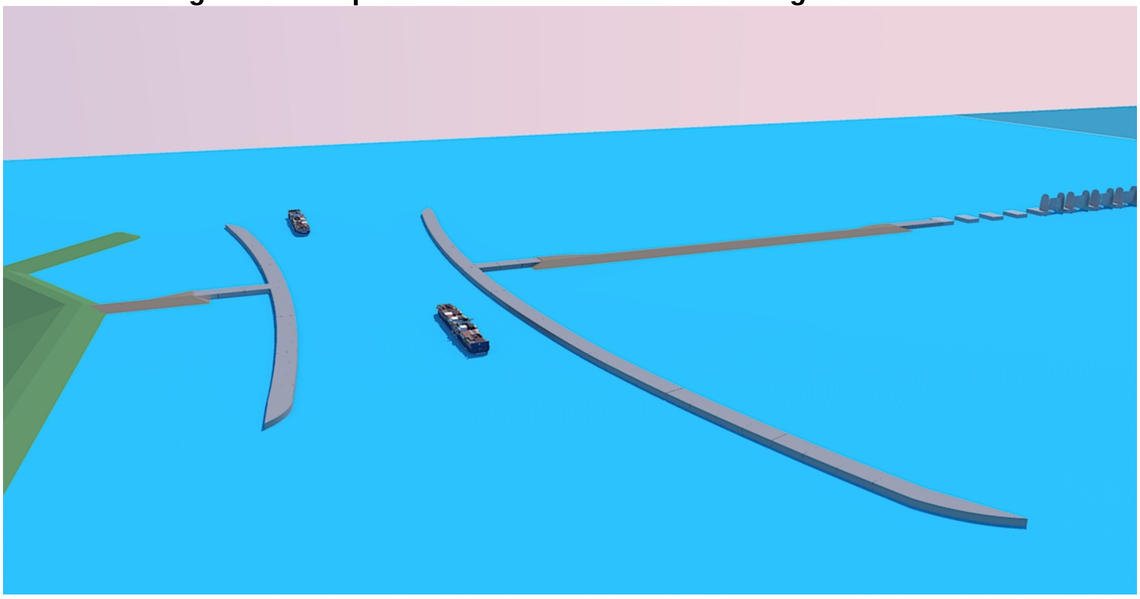


Figure 5.14 Impression of the funnel with average vessel size



Ship collision

The barrier must be able to resist ship collision or the ship collision must be prevented by other measures. It is not realistic to design a barrier to handle ship collision, since the load of ship collision during storm conditions is very large. The most likely situation in which a vessel would hit the barrier is in case of engine or rudder failure. The vessel is not under control of the captain and the speed at which the barrier is hit will be very high. The load on the barrier will be very high due to the large vessel size. This means that somehow the vessels should be kept away from the barrier. The first step in doing this is to communicate the closure of the barrier in time with the vessel. There should be sufficient capacity at the anchorage. Finally vessels that are in trouble need to be assisted in order to prevent damage to the barrier. This can be done by stationing a couple of tugs near the barrier for the assistance of the vessels in distress. The exact number of tugs needs to be calculated based on the historic accident reports. This means that the barrier does not need to be designed for ship collisions.

The funnel structure forms the obstacle between the vessel and the barrier. So it is not possible for the vessels to hit the barrier during normal usage. A collapse of the funnel does not harm the barrier, as explained in section 4.5.

Remarks about shipping opening

The design has been presented to a captain of a general cargo vessel. His mayor concern was not the flow velocity but the traffic flow. The flow velocity in the shipping opening is not a problem for the navigability. There are locations with a much higher flow velocity. For example, Pentland Firth in the north of Scotland has flow velocities of about 10 knots and vessels are able to sail through it. However the flow velocity may have a negative impact on the traffic flow. Vessels with a low maximum speed are slowed down more than high speed vessels. This may cause congestion. The average of 1 vessel every 20 minutes per direction means that there is sufficient time for the slow vessels to pass through the barrier.

The navigability should be tested by running simulations and tests.

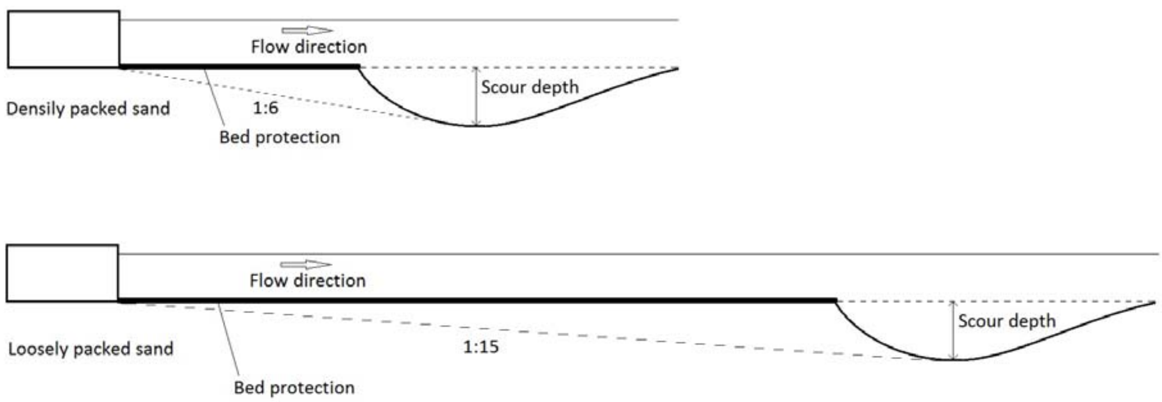
5.3.2. Design of the bed protection

The main purpose of the bed protection is the protection of the foundation of the barrier from erosion. The main sources of the erosion are the flow in and out of the estuary and the vessels that pass the barrier. The highest flow velocity caused by the tide and the storm surge is 2.8 m/s as mentioned in section 4.4. The North and South passage have to deal with the passage of vessels as well. The vessels increase the load in two ways. The first issue is the higher flow velocity in the cross-section during the passage of a vessel. This increase in flow velocity is about 1.8 m/s. The other load is caused by the propeller of the vessel. The vessels need to overcome a water level difference combined with a width restriction. The vessel may need to sail at full power through the barrier. The load on the bed protection during spring low water for the largest vessel during the largest total flow velocity is 4 m/s. This leads to a stone diameter of 0.9 metre. For the calculation is referred to appendix V. The open parts of the barrier without shipping require a stone nominal diameter of 0.12 metre and of the other locations a nominal diameter of 0.03 m is required.

The length of the bed protection is related to the depth of the scour hole. The scour hole after 100 year will be about 50 metre, for the calculation method is referred to appendix V. It is unclear how the scour hole will develop in case the sediment transport is not harmed by the barrier. This can be subject of further investigation. A smaller depth of the scour hole could reduce the length of the bed protection and thus costs. The scour holes near the Eastern Scheldt barrier are much larger than expected, so the mechanism should not be underestimated. The scour holes cannot be seen from the surface. So it is advised to check the size of the scour holes every year in order to prevent a sudden collapse.

The length of the bed protection depends on the conditions of the subsoil. It is assumed that the sand grains are loosely packed and that the sand is very fine. The length of the scour protection is related to the depth of the scour hole. For loosely packed sand, the length is 13 times the depth of the scour hole. So the bed protection must have a length of 645 metre from the toe of the barrier. Better soil conditions can reduce the required length of the bed protection. For loosely packed sand is an angle of 1:15 indicated in Figure 5.15, but the slope of the scour hole is assumed to be 1:2, so the bed protection has to be only 13 times the scour depth.

Figure 5.15 Required length bed protection

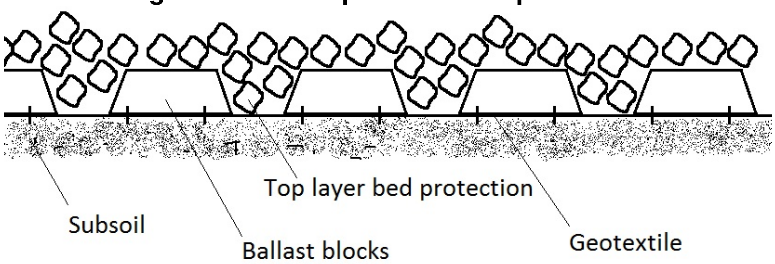


The bed protection only has to be this long near the openings in the barrier. The bed protection can be shorter behind the closed part of the barrier, since the scour hole gets less deep. The flow velocity at the location of the barrier is increased during the construction of the barrier. This will lead to erosion. It is required to prevent erosion at the location of the barrier in order to protect the foundation of the barrier. The barrier is therefore built on bed protection. The length of the bed protection is difficult to determine. It is therefore assumed that the scour hole during construction can reach a depth of 1/3rd of the depth at the openings. The length of the bed protection has to be 215 metre. It may seem a waste of materials to place extensive bed protection only for the construction stage. However the protection will also prevent damage to the foundation of the barrier due to unexpected changes in morphology.

Composition of the bed protection

The massive size of the barrier makes the construction of the bed protection very costly. The bed protection near the scour hole has to have some cohesion in order to remain functioning in case the bed protection slides into the scour hole. This can be achieved by applying geotextiles with connected ballast material, see Figure 5.16. The current state of the art is such that the lifetime 100 years should be possible for geotextiles. The geotextiles replace the filter layers under the bed protection. This means that the difficult execution of the installation of the filter is prevented. The permeability of the geotextile should be at least 10 times higher than the soil layer underneath, it is not expected that this requirement will lead to problems.

Figure 5.16 Composition bed protection



The stone size in the figure is quite small. In case a larger stone size is required for stability, a layer of smaller stones must be placed between the geotextile and the big stones in order to prevent damage to the geotextile caused by the falling rocks.

5.3.3. Design of the caissons

The caissons are used for four purposes. The first purpose is the funnel structure and the second purpose is the actual barrier, the third is the caisson that supports the gates and the fourth is the caisson that is located in the rubble mound barrier. The loads on the locations are different. This means that the caissons are different per function as well. The different loads are discussed below. For an overview of the usage of the caissons is referred to appendix VII.

General design choices for all caissons

The length of the caissons is assumed to be 50 metre. A large length would require very accurate levelling of the foundation to prevent large torsion loads for the caissons. The maximum height difference of the bed is about 25 mm. Caissons can be made in length of over 100 metre, but for this situation a length 50 is chosen. The short length may cause problems for the transportation stage. Longer caissons are more easy to transport. In a later stage can be decided to alter the length of the caissons for construction or other optimization reasons. Longer caisson could reduce the construction time, since fewer caissons have to be constructed, transported and placed.

The caissons are made of concrete. It is not efficient to make a concrete box of 34 * 50 without any inner walls. Therefore is decided to make the caissons with an inner cell structure with cells of about 5 - 6 metre. The outer walls have a thickness of 0.6 metre and the inner walls have a thickness of 0.4 metre. The outer walls must be able to resist the water level difference during transport and contain the ballast material in the permanent situation. The inner walls of the caisson are loaded in tension during normal situations but in compression during transport. The caissons are relatively high, so the load during normal conditions is also relatively high. The size of the caisson is chosen as such for good visibility and to prevent vessels from hitting the barrier by accident.

Design of the funnel caisson

The main load for the funnel structure is the water level difference over the caisson caused by the storm surges and passing vessels. The funnel structure also has to resist waves but the wave height is not as large as for the barrier itself due to the rotated position. The largest water level difference caused by storm conditions is 0.5 * 2.6 = 1.3 metre and the passing vessels will cause an additional water level depression of 2 metre. The wave height is assumed to be 1.6 metre. The top of the caisson is located at 9.5 metre + NAP. The caissons are placed on a shallow foundation. The width has to be large enough to resist the horizontal load and to make sure the foundation stress is always in compression and that the maximum soil stress is not exceeded. The allowable soil pressures are calculated using Brinch Hansens' formula. The dimensions resulting from the calculation can be found in Table 5.13.

Table 5.12 Horizontal loads funnel caisson

Load	Magnitude	Load factor	horizontal load per metre with load factor [kN/m1]
Water difference	3.3 metre	1.2	$2.2 * 10^3$
Waves	1.6 metre, 5.9 seconds	1.5	$0.51 * 10^3$

Table 5.13 Funnel caisson dimensions

	Dimension	Magnitude
Length	m	50
Width	m	34
Height	m	34.5
Maximum soil pressure	kN/m^2	507
Required water depth for transportation	m	15.5

Barrier caisson design

The caisson which is used for the barrier itself has to resist different loads. The loads are the water level difference due to reduction caused by the barrier and the wave loads caused by the storm conditions. The dimensions resulting from the calculation can be found in Table 5.15.

Table 5.14 Horizontal loads barrier caisson

Load	Magnitude	Load factor	horizontal load per metre with load factor [kN/m1]
Water level difference	2.6 metre	1.2	$2.0 * 10^3$
Waves	3.5 metre, 9 seconds	1.5	$2.3 * 10^3$

Table 5.15 Barrier caisson dimensions

	Dimension	Magnitude
Length	m	50
Width	m	38
Height	m	34.5
Maximum soil pressure	kN/m^2	566
Required water depth for transportation	m	14.2

The caisson for the barrier is composed of about the same cell dimensions as the funnel caisson for production reasons. The differences in caisson width result in small changes in the cell dimensions.

Design rubble mound caisson

A third type of caisson is used to form the core of the rubble mound dam. The caisson is used to create a more or less water tight core and make construction of the rubble mound dam easier. Once the caisson is part of the dam, the quality of the caisson is not very important any more. Cracks may form in the concrete since the water, since the stability of the dam is not threatened by some cracks.

The stability during transportation is the governing situation for this caisson. The top side of the caisson has to be at 3 metre + NAP. This implies that at deep locations the construction of a sill is required. For repetition reasons is chosen to make a sill instead of making all unique caissons.

The free standing rubble mound caissons must withstand the increased flow velocity and water level difference caused by the construction of the barrier. The free standing caissons are not designed to handle storm conditions. The caissons must be covered with rubble mound before the storm season.

Table 5.16 Rubble mound caisson dimensions

	Dimension	Magnitude
Length	m	50
Width	m	22
Height	m	21
Maximum soil pressure	kN/m^2	296
Required water depth for transportation	m	12.5

Design of the gate supporting caisson

The final type of caisson that is used is the one that has to support two gates on either side in the middle cross-section. The basis for this caisson is the one as described in Table 5.15. The caisson is rotated by 90 degrees around the vertical axis for additional stability. The load on the caisson is calculated with on either side of the caisson a gate of 60 metre in storm conditions. This means that the load on the caisson is the sum of the load on the short end of the caisson and the load on the surface of one gate. The loads are the same as in Table 5.14.

Table 5.17 Barrier caisson dimensions

	Dimension	Magnitude
Length	m	52
Width	m	38
Height	m	34.5
Maximum soil pressure	kN/m^2	600
Required water depth for transportation	m	13.6

It is assumed that two lifting structures are placed on the caissons that support the gates. These structures are quite heavy in order to prevent horizontal sliding of the caisson. These lifting structure need to be placed after the caisson has been immersed. The foundation of the lifting tower needs to be integrated in the caisson. The supports for the gates need to be prefabricated in the caisson as well.

Transportation of the caissons

The required drafts for the different caissons are acceptable with regards to the depth of the Western Scheldt. The caissons need to be transported from the construction area to the location of the barrier over the Western Scheldt.

The dynamic stability of the caisson is important with regard to the stability of the caissons during transport under wave loading. The caissons are only transported under calm conditions, so only small waves are expected. This means that the natural period of the caisson should be much larger in order to prevent an extreme response. In order to speed up the calculation caisson is simplified to an U-shaped box with no inner walls. The width of the outer walls is increased from 0.6 metre to 1 metre, to compensate for the absence of the inner walls. The natural periods for the different caissons are presented below.

Table 5.18 Natural periods of the caissons

Caisson type	Cross-section [m * m]	Natural period [s]
Funnel	34 * 34.5	93
Barrier	38 * 34.5	55
Rubble mound	21 * 21	36

The natural periods are much larger than the expected wave period. So dynamic instability is not expected. A more accurate calculation is not required at this moment.

Structural design of the caissons

The durability of the caissons must be guaranteed over the entire life time of the structure. The main thread for the caissons is the corrosion of the reinforcement due to the high salinity of the environment. It is therefore essential that cracks are prevented or that the crack width is limited. This can be achieved in two ways. The first option is prestressing the entire structure. The second option is applying a large concrete cover in combination with a high concrete quality, since high concrete qualities have a more dense structure.

Prestressing is expensive and there are always parts of the structure that are not prestressed since the prestressing force needs some length for the introduction into the concrete. Therefore normal reinforced concrete is selected, since the cell dimensions are such that prestressing is not required. The crack width is an important aspect for the design of reinforced concrete in a salt environment. The crack width of 0.2 mm is the maximum for concrete in the tidal area.

The wall thickness of the caissons has to be checked for bending moment and shear force capacity. There are two load cases that should be checked. The first case is the transportation stage in which the caisson is empty, and thus has to resist the water pressure from outside. The load depends on the draft of the caisson. The outer walls have to resist the bending moments, shear forces and normal forces, while the inner walls only have to resist the compression force. In the end situation the governing load is defined by very low water outside and soil and water pressures in the caisson. The bending moments have the opposite sign as in the transportation stage, since the resulting load has an outward direction. The shear force in the walls is very large and therefore shear reinforcement has to be applied. The draft of the caisson would make transportation impossible in case no shear reinforcement is applied, due to the larger wall thickness.

The resulting wall thicknesses and reinforcements in the inner and outer walls for each type of caisson are presented in the tables below. In the calculation is assumed that the maximum bending moment is given by (1/10) * q * l^2, due to the redistribution of bending moments. The crack width demand is governing for the reinforcement calculation.

Table 5.19 Structural design funnel caisson

	Inner wall	Outer wall
Thickness	400 mm	600 mm
Concrete cover	75 mm	75 mm
Reinforcement percentage	2%	3%
Reinforcement configuration	2 * Ø20-75	2 * Ø28-100
Shear reinforcement	none	Ø12-90
Crack width	no crack formation	0.20 mm
Concrete quality	C60/75	C60/75

Table 5.20 Structural design barrier caisson

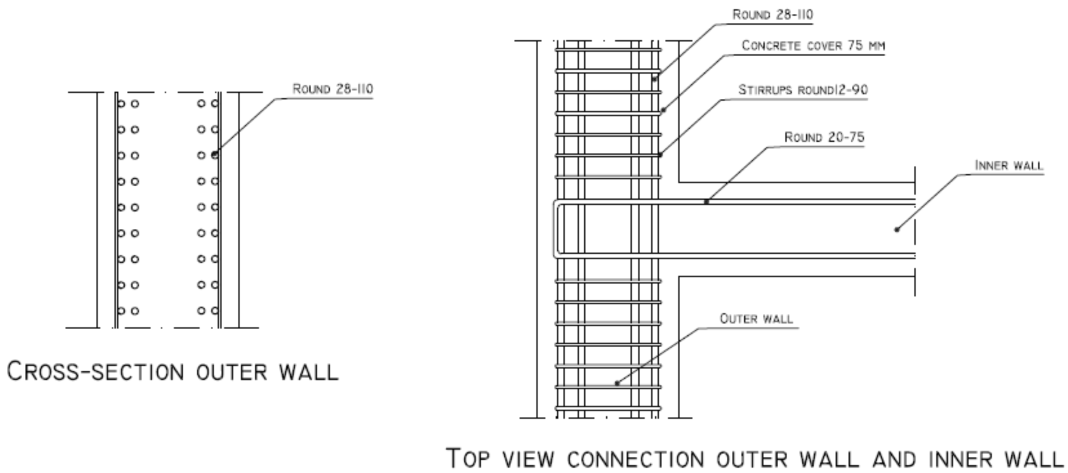
	Inner wall	Outer wall
Thickness	400 mm	600 mm
Concrete cover	75 mm	75 mm
Reinforcement percentage	2%	2%
Reinforcement configuration	2 * Ø20-75	2 * Ø28-110
Shear reinforcement	none	Ø12-90
Crack width	no crack formation	0.20 mm
Concrete quality	C60/75	C60/75

Table 5.21 Structural design rubble mound caisson

	Inner wall	Outer wall
Thickness	300 mm	500 mm
Concrete cover	75 mm	75 mm
Reinforcement percentage	2%	2%
Reinforcement configuration	2 * Ø16-65	2 * Ø28-140
Shear reinforcement	none	Ø12-140
Crack width	no crack formation	0.18 mm
Concrete quality	C60/75	C60/75

The figure below gives an impression of the amount of reinforcement in the outer wall of the funnel caisson. This reinforcement is only the calculated reinforcement. The minimum required shear reinforcement for the inner wall is not drawn. The high amounts of reinforcement are required for the crack width demand. It is possible that smaller amounts of reinforcements can suffice in the higher parts of the walls. It should be checked whether the concrete cracks. In case the concrete does not crack the amount of reinforcement can be reduced.

Figure 5.17 Impression reinforcement of the funnel caisson



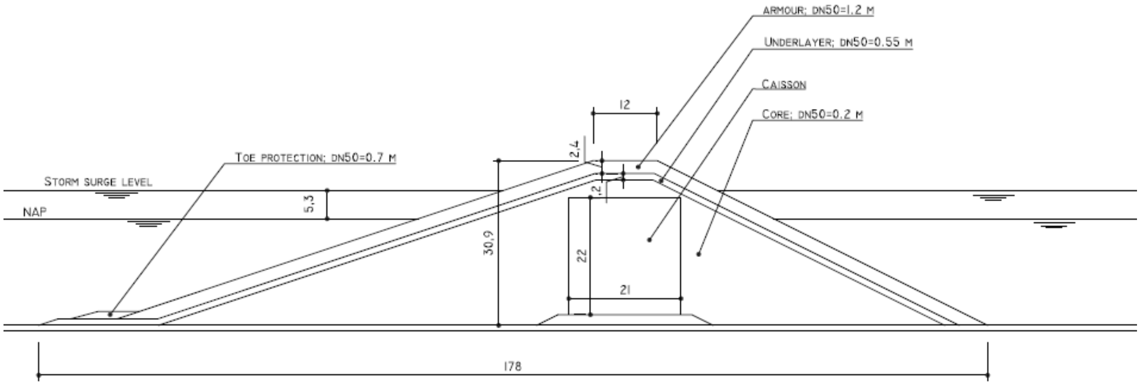
Remarks about caissons design
There are four different caisson required for the deisng of the barrier. It is desired to have few types for repetition reasons. This will result in higher material costs but the additional costs for labour and formwork will be reduced significantly.It is therefore better to have one type for the funnel, barrier and gate support and one type for the rubble mound part. The probability of human failure is also reduced, since those types have significantly different dimensions.

5.3.4. Design of the rubble mound dam

The rubble mound part of the barrier is, by itself, not as water tight as the caissons. The amount of leakage should be restricted, so the rubble mound should contain some kind of impermeable core. The construction of this core is the main problem since it has to build in relatively high flow conditions. The flow velocity increases during the construction as well as the water level difference over the barrier. This makes construction difficult. The core can be made water tight by placing a smaller caisson in it. This caisson must be able to resist normal water level difference and normal waves during construction. The caissons are most important for the construction phase, since the discharge through the dam will be stopped eventually by marine growth. The dimensions of the caisson are presented in the table below. On both sides of the caisson rubble mound slopes are place for stability and the absorption of wave energy. These slopes can be used for the environmental compensation. The sloped rubble can provide shelter for fish and allows for example shellfish growth. The stone size is calculated using the breakwater design formulas (Van de Meer, 1995), see appendix V.

The rubble mound barrier itself consists, of course, of different diameters of rock. The armour layer is the layer with the largest diameter. The diameter below the armour layer is a factor 10 smaller than the layer above. The design parameters are the outer slope, height and the width of the top. The outer slope is assumed to be 1:3, but can be increased to create more space for environmental development. The height is calculated using the run up criterion. The width is barrier is related to the width of the caisson in the core. For the detailed calculations is referred to appendix V. The design of the cross-section of the dam is presented below. The left side is the sea side and the right side is the Western Scheldt. The left slope is 1:3 and the right slope is 1:2.

Figure 5.18 Cross-section rubble mount part of the barrier at 20 metre water depth in [m]

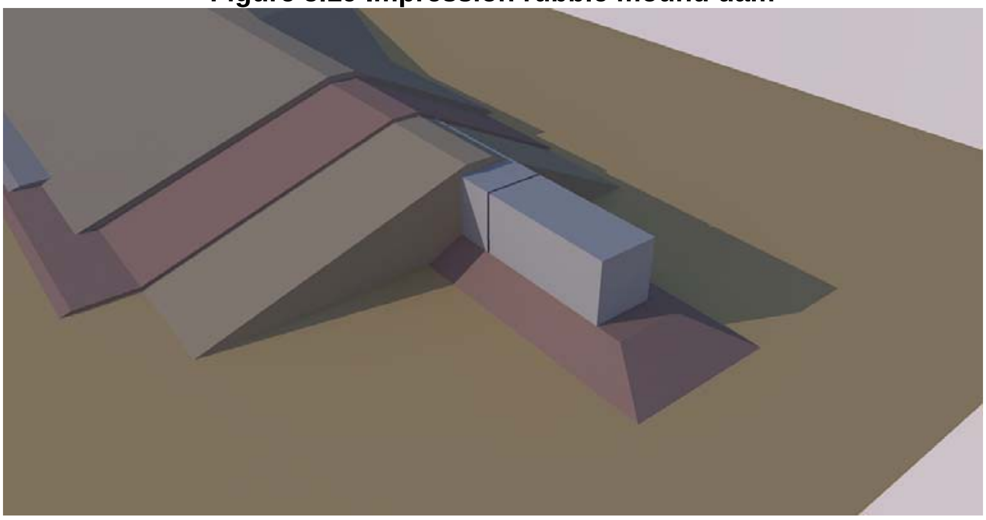


It is not realistic that the exact nominal diameters as presented in Figure 5.18 can be ordered so in the table below are the corresponding stone classes presented.

Table 5.22 Nominal diameters for rubble mound dam

Layer	Nominal diameter [m]	Stone class [kg]
Armour	1.2	5 000
Under layer	0.55	300 - 1 000
Core	0.2	10 - 60
Toe protection	0.7	300 - 1 000

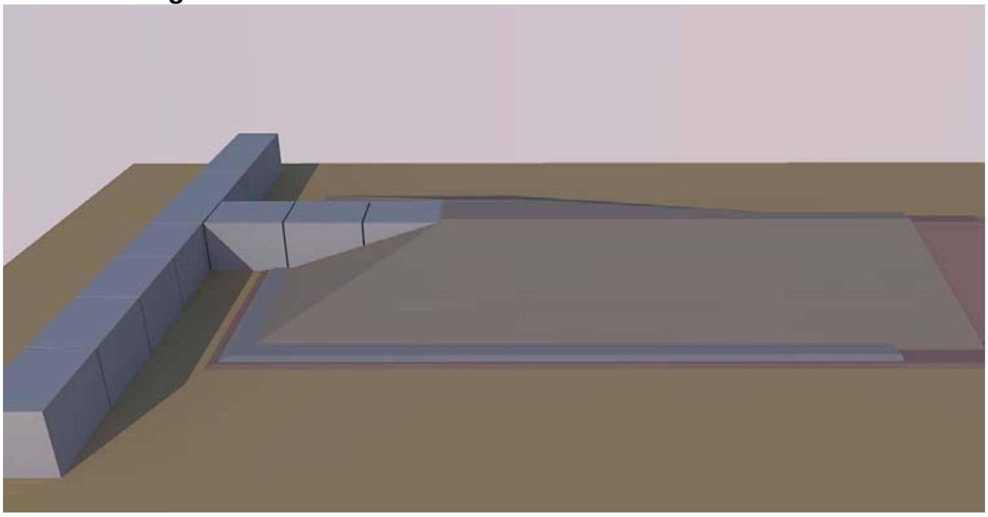
Figure 5.19 Impression rubble mound dam



Transition from caisson to rubble mound

The transition from caisson to rubble mound has to be made in such a way that the transition does not become a weak spot in the barrier. The rubble mound overlaps the large caissons for the length of one barrier caisson. The end of the rubble mound consists of slopes with the same angle as the cross-sectional slopes. This means that over the length of about 2 caissons the height of the rubble mound is reduced to zero. For an impression of the transition see figure below.

Figure 5.20 Transition from rubble mound to caisson



The rubble mound is not connected to the funnel, since the additional horizontal pressure would cause the funnel caissons to topple over.

5.3.5. Design of the vertical lift gates

In this subsection the gates for the middle openings are designed. Four different sizes of gates are applied in the barrier. It would be better for repetition to use only one type of gate but this is, for now, not possible in combination with the environmental and safety requirements. So three gates of 60*34.5 metre, one gate of 50*34.5 metre, one gate of 55*19.5 metre and two gates of 60*19.5 metre are applied. The gates can be optimized by having a standard width and only vary the height of the gates. Due to the time restriction, is decided to design only the largest type of gate, which the gate of 60*34.5 metre.

The largest gates could have severe impact on the landscape, due to their size. The vertical lift gates did not score well on inconspicuousness in MCA, see Table 5.11. However the largest gates are located in the in the middle of the barrier, which is about 2 kilometre from the coast. The large distance reduces the impact of the gates and lifting structures on the landscape. The gates can also become an icon as happened with the Eastern Scheldt barrier.

The gates are made of 3D trusses of circular hollow sections. Circular hollow section are used to prevent the collection of water in corners and the trusses applied to prevent the peak stresses caused by trapped waves and because it is an efficient way to create large spans.

Design loads

The loads on the gates are caused by waves and water level difference over the barrier. The largest water level difference is 2.6 metre and the wave height depends on the location in the barrier. The middle location has the highest significant wave height with 3.5 metre, while the South gates only have to deal with waves of 1.75 metre high. The gates have the same wave overtopping height as the caisson, which is 9.5 metre +NAP. The loads on the gates are presented in Table 5.23. The design wave load is defined by the fully reflected largest wave, which is 2 times the significant wave height.

The gates are not checked for dynamic wind loads. This can cause large loads on the supports of the gates, due to vibrations. This should be checked in a later design stage.

Table 5.23 Loads on gates

Description	Dimensions	Middle gate
Water level difference per square metre	[kN/m^2]	26
Water level difference per running metre	[kN/m^1]	7.9 * 10^2
Waves	[kN/m^1]	1.5 * 10^3
Total without load factors	[kN/m^1]	2.3 * 10^3
Total with load factors	[kN/m^1]	3.3 * 10^3
Design moment	[kNm]	1.5 * 10^6

First estimate gate dimensions

By assuming a truss width, the required cross-sectional area for the truss can be calculated. The number of connections is very important factor in the design of the gates. Labour is expensive and the connection require the most labour. The connections are more difficult to paint and corrosion will start here first. So in the design of the gates, the number of connections must be kept low. This is a first assumption for the first design in MatrixFrame.

Table 5.24 Gates dimensions

	Dimensions	Middle gate
Design moment	[kNm]	1.5 * 10^6
Truss width	[m]	8
Steel quality	[-]	S460
Required area	[mm]	4.0 * 10^5
Cross-section CHS740 * 35	[mm^2]	8.1 * 10^4
Number cross-sections require	[-]	5

The truss is located on the Western Scheldt side of the barrier in order to minimize the wave impact on the elements of the truss. The gates are designed to have a low torsion stiffness. The caissons cannot be positioned with complete accuracy, so the both guiding structures are very likely to be not parallel. So the edges of the gates will have to be rotated in order to fit between the caissons. A gate with low torsion stiffness will fit more easily without causing large stresses in the gate or in the supports.

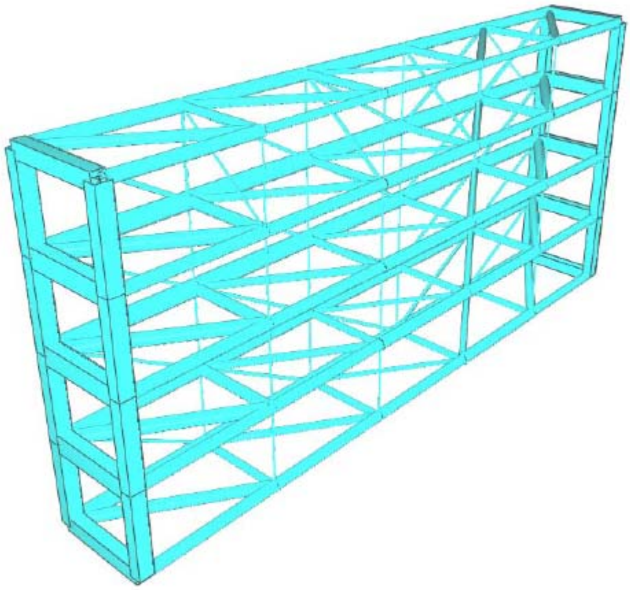
The truss supports a more-or-less water tight screen. This screen has to be able to bridge the gaps between the elements of the truss. The screen has to have a certain stiffness. Therefore is decided to use sheet pile elements for the screen. These sheet piles do not have to be manufactured especially for this purpose, which means that they are relatively cheap. An advantage for the sheet piles it that they are not continuously connected to the truss. It is therefore possible for the water to flow between the screen and the truss which reduces the water pressure. Part of the screen can be replaced, when damaged, without much trouble.

In case the truss is composed 5 sections that support the screen, the required sheet pile is AZ14-700. This sheet pile is selected for its stiffness. The steel quality must be S430 or higher. The sheet piles have been calculated as a beam on 5 supports, loaded by wave impact and water level difference. The reaction forces are loads that are applied on the truss.

Gate design

The truss is designed by making several design loops in order to fulfil the requirements. The first estimate is presented in Table 5.24. After several design loops the design is as follows. The total height of the gate is 34.5 metre so a total of 5 circular section means one section every 7 metre. The elements near the supports are rectangular in order to create more surface to deliver the load to the caissons.

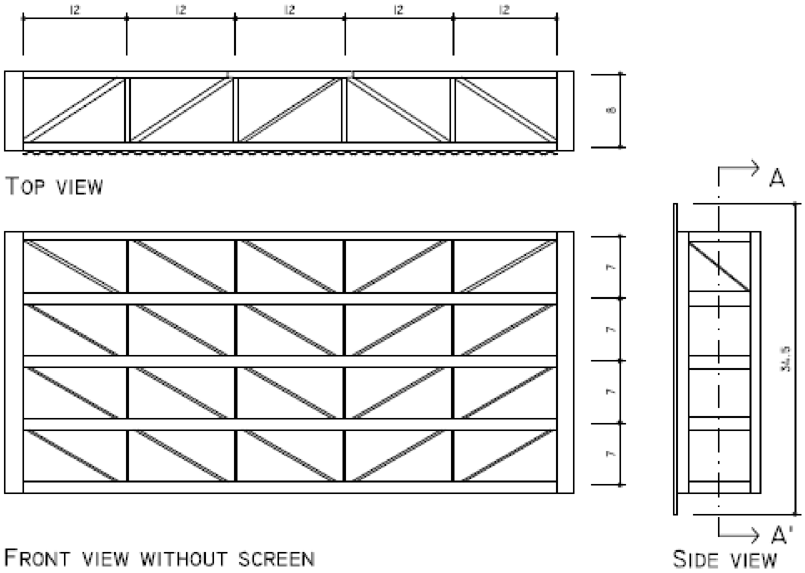
Figure 5.21 Virtual model 3D truss



A drawing of the 3D truss seems very chaotic with all the elements. In order to explain the philosophy the truss is broken up in pieces. The design of the truss is composed of 5 horizontal frameworks. The five horizontal frameworks are connected with a vertical framework one the side of the screen. This supports the self weight of the structure. The other side of the frameworks is only connected with vertical elements in order to reduce the torsion stiffness. Only the top side has diagonal element in order to reduce the deflection, see Figure 5.22. The elements that directly support the screen are loaded in bending. The elements behind the supporting element are used to increase stiffness and to reduce the local bending moments in the elements. The complete drawing is presented in appendix VII.

The truss is not completely symmetrical. It is possible to make a cross in the middle sections of the truss, however this will lead to more and complicated joints. Another option is to make a truss with 6 parts of 10 metre, but this will also lead to more connections.

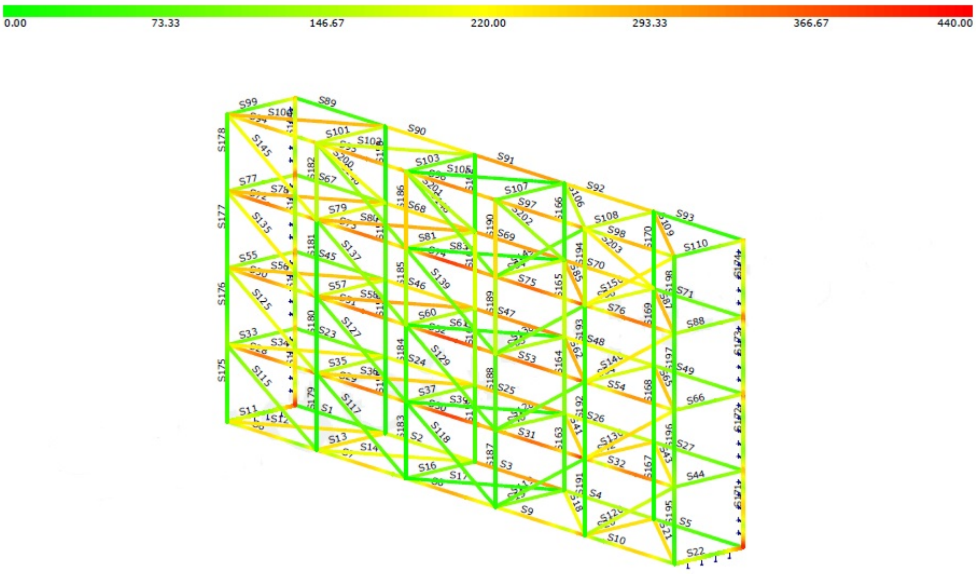
Figure 5.22 Gate design



The truss is calculated in a 3D model. The loads on the truss are the self weight of the truss itself, of the screen and the horizontal load of the waves and water level difference. The steel quality that is used is S460. The load can change directions, since the water level difference and the wave impact can be both negative and positive. This means that most elements can be in tensions as well as compression. The buckling factor is estimated on 0.60, so the highest acceptable stress is about 276 N/mm. The calculation results are presented in the figure below. For the output of the MatrixFrame calculation is referred to appendix VIII.

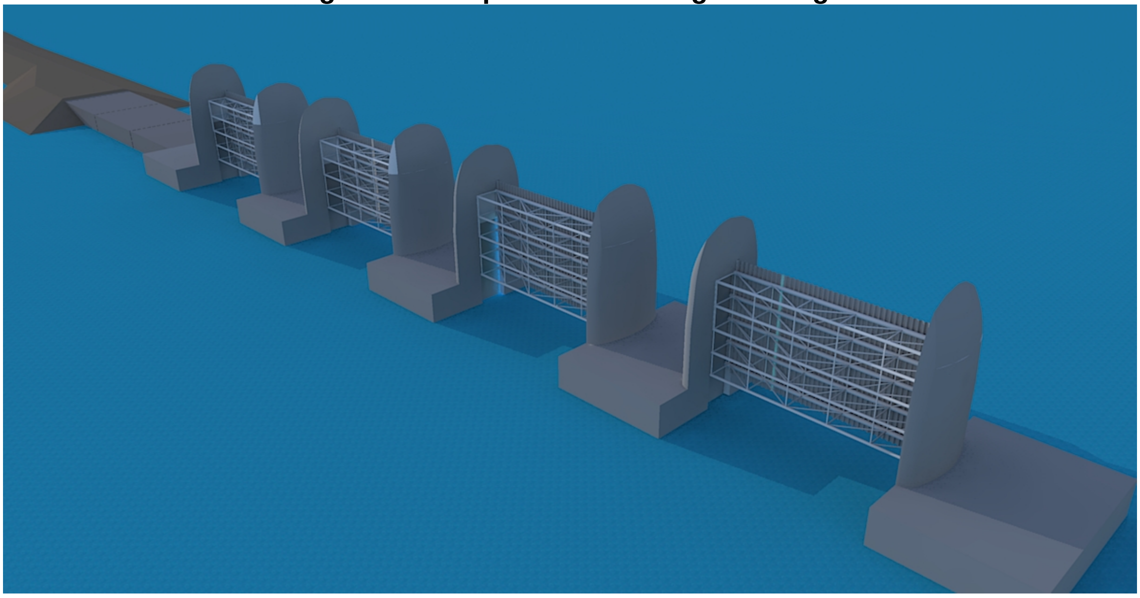
The torsion stiffness of the gates has not been calculated in this stage. The subjects of local buckling and the connection require further attention as well.

Figure 5.23 Calculation result for the 3-D truss



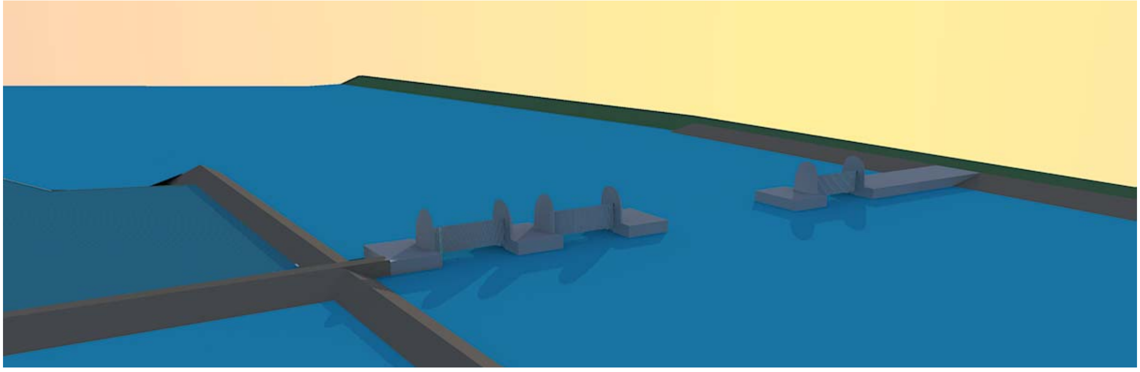
After several cycles the result above was obtained. The stress in the elements is at an acceptable level. The result of the calculation is that the amount steel per gate is known. This is input for the cost calculation in the next chapter. The figure below is an impression of the gate in opened conditions.

Figure 5.24 Impression of the gate design



In closed condition the gate looks as follows. Most of the truss in under water. The truss is located at the Western Scheldt side of the barrier. The largest waves during storm conditions are at sea and not in the Western Scheldt. For more detail on the calculation is referred to appendix VIII.

Figure 5.25 Impression southern shipping channel



The gates are not used very often so fatigue is not likely to be a problem. The number of waves that has to be resisted by the gate during 1 storm is equal to the storm duration divided by the wave period. So 35 * 60 * 60 / 9 = 14 000 cycles. When assuming 7 closures during storm conditions, the gates have to withstand 98 000 cycles. This number of cycles is not high enough to contribute to fatigue damage. Fatigue is not a problem, even with twice as many cycles.

The gates are quite expensive. It should be checked whether the moveable gates are cost effective, since it is also possible to close some of the openings permanently as the sea level rises. This will affect the environment, but the construction and maintenance costs are much lower. The failure probability is also reduced, since there are no moveable parts. In case the reduction factor of the discharge cross-section is lowered from 0.375 to 0.275 permanently, the reduction factor for the tidal prism is lowered from 0.8 to 0.64. This is a serious reduction, but the environmental demand is uncertain so it is possible that the environmental impact of the reduction factor of 0.64 is restricted. This should be the subject of an Environmental impact assessment.

5.3.6. Gates' lifting structure

The lifting structures for the gate have to support the gates during normal conditions. The large height of the gates implies a large height of the lifting structure. An advantage is that the openings are not used for shipping, so the gates do not have to be lifted high above the water level. The underside of the gates is lifted to about NAP + 7 metre. This means that waves with a significant wave height of 3.5 metre can still pas under the gate at design water level. The gates have a height of 34.5 metre, so the top of the gate is located at NAP + 41.5 metre. The mechanical equipment has to be located above the gates so the height of the lifting structure is assumed to be NAP + 50 metre. The mechanical equipment must be inspected or repaired from time to time. The lifting structure therefore needs to house vertical transport in the form of a staircase and an elevator. The lifting structures can be reached from the South side via gangways on the gates. Two gangways are located on each gate; one at the top to allow passage during closed conditions and one at the bottom to allow passage during open conditions.

The lifting structures have to be able to support the gates in raised position during storm conditions, for example when the gates do not have to be closed or when a gate fails to close. The load on a gate is about 1.2 kN/m^2. The lifting structure has to able to resist the compression load from the gate's weight in combination with the bending moment caused by wind. The weight of the gate is estimated on 1390 kN. The lifting structure has to be as wide or wider as the width of the gate. This means a width of about 10 metre.

Assuming the lift structure consists structurally of 2 walls of 10 * 2 metre. The loads, including load factor, on the lift structure are presented in the table below. For the complete drawings is referred to appendix VII.

Figure 5.26 Top view connection gate and caisson

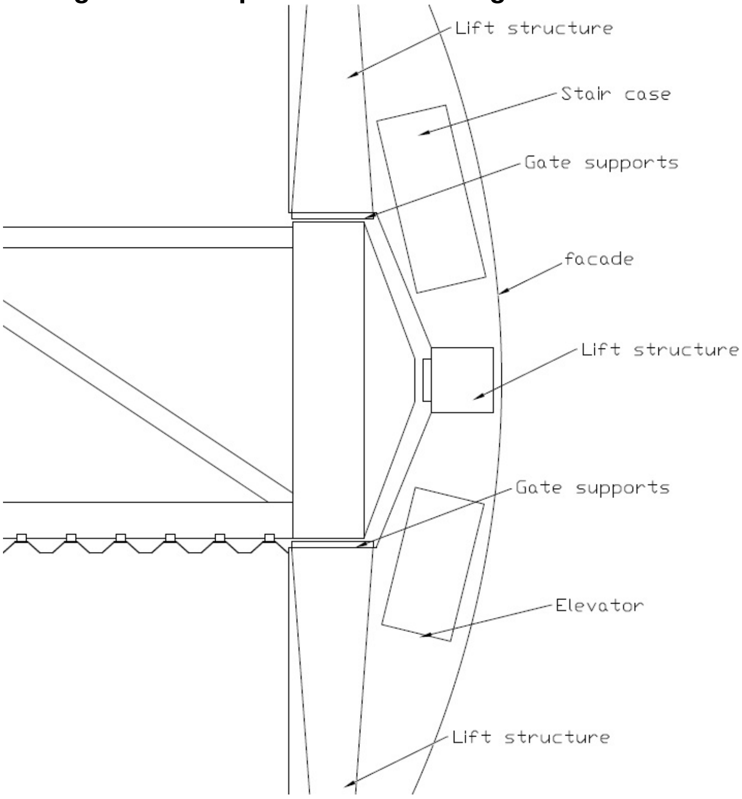


Table 5.25 Loads on the lifting structure

	Load	Arm	Moment
Self weight lift structure	48 600 kN	-	-
Self weight gate	834 kN	-	-
Wind load	1.2 kN/m^2	24.5 m	6.4 * 10^4 kNm

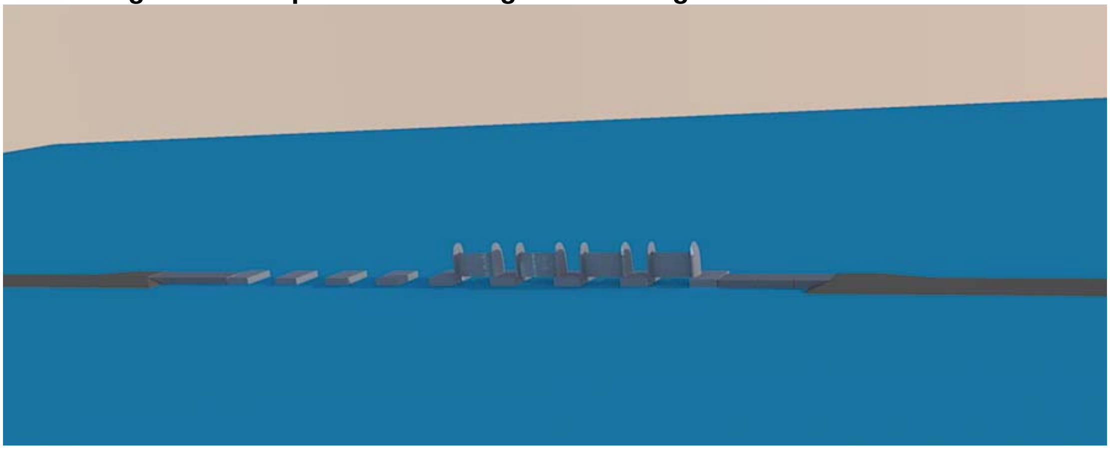
These loads are used to calculate the stress levels in the lowest part of the lift structure. The calculation shows that the lifting structures can easily be executed in reinforced concrete since the stress levels are very low.

Table 5.26 Calculation stress levels in lifting structure

Vertical load	49 * 10^3 kN
Moment	6.4 * 10^4 kNm
Moment of Inertia	4333 m^4
Area	40 m^2
Stress level maximum	1.4 N/mm^2
Stress level minimum	1.0 N/mm^2

The weight of the mechanical equipment is not included, but average stresses do not exceed 1.5 N/mm^2. So it is unlikely that this will be a problem. The lifting structures are only roughly designed since only the feasibility of the system is investigated.

Figure 5.27 Impression of the gates in lifting structure from sea side



It is advised to consult with an architect for the design of the lifting structure. An architect can help to fit the structure into the landscape. The MCA showed that the vertical lifting gates do not score well on the visual aspect. It is therefore wise to involve the architect in an early stage. For now is assumed that a circular shape is most appealing.

5.4. Remarks with regard to the design

The main elements of the barrier are designed and all the elements are technically feasible. The destruction of the horizon is an issue for the selected type of gate. For an analysis of the obstructed views is referred to appendix V. It is an option to make the gate in two or more parts. This would reduce the height of the total object and thus reduce the impact on the landscape, but it will increase the leakage through the gates and it will make the gates more sensitive to failure.

The gates cannot be accessed directly from land, because of the shipping lanes on either side. The gates can only be accessed from the south side by ship.

The closure of a gate during high wave conditions can cause damage to the gate. This can be prevented by adding stiffeners to the underside of the gate.

6. CONSTRUCTION, COSTS AND PLANNING

This chapter covers the main aspects with regard to the construction of the barrier. The first step is to find a proper location for the construction facility. This is followed by a global construction approach. The next section contains the layout of the construction facility. The construction of the caissons is explained in more detail before presenting the overall planning. The final aspect of this thesis is the calculation of the construction costs.

6.1. Construction of the barrier

In this section the construction of the barrier is explained. The first step is to find a suitable location for the construction facilities. This is followed by the layout of this facility. The construction of the caisson is given extra attention in the final subsection.

6.1.1. Location construction facility

The caissons can be constructed in or near the port of Vlissingen. This location has good transportation connections and there is space available. The water depth on the route between the barrier and the port is sufficient. The water depth in the port itself is sufficient since the maximum vessel draft is 16.5 metre. The distance between the port and the location of the barrier is about 6 kilometre, this is relatively short. It is not possible to construct the caissons in the Western Scheldt because of the environmental value and shipping traffic in the deep parts. There is a dry dock located in the port, but the width is not sufficient for caissons of 38 metre wide. The caissons of 21 metre wide could fit in the dock, but the availability of the dock is uncertain. The construction of the caissons will take a considerable amount of time. It is therefore decided to construct a dedicated construction facility somewhere else in the Port of Vlissingen. There is a site available with a maximum quay length of 650 metre and an area of about 50 hectare, indicated by the orange area in Figure 6.2. The green area is mainly used as a bulk terminal. The Port of Vlissingen is located east of the barrier location. The orange bar near the entrance of the port represents a jetty for temporary storage of the finished caissons, see Figure 6.1.

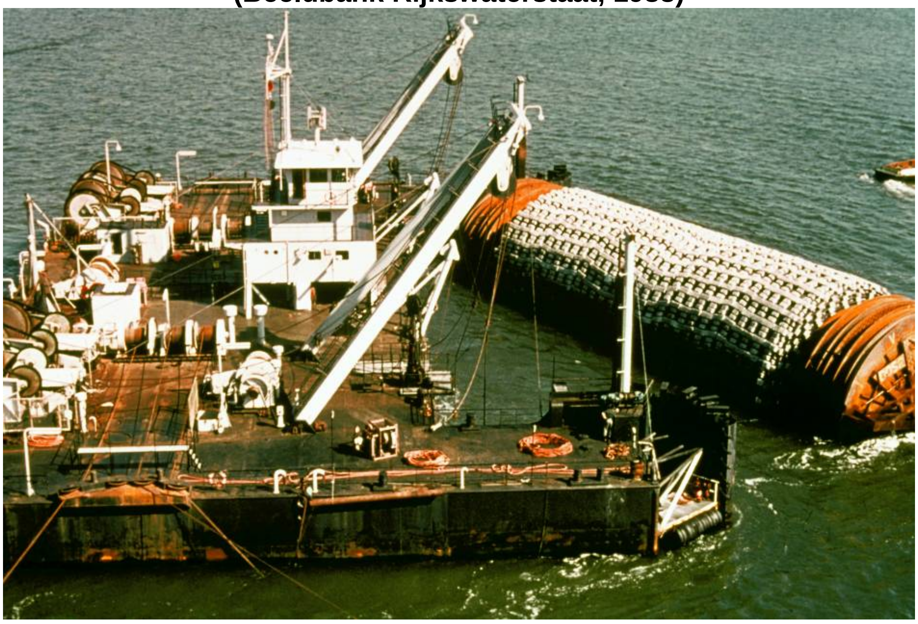
Figure 6.1 Overview of the construction areas in the Port of Vlissingen

Figure 6.2 Location of the construction site

6.1.2. Lay out construction facility

The available area in the port can be used to construct the caissons, to make the bed protection and to load the stones on the dumping vessels. The berth for the stone dumping vessels, see Figure 6.5, is located at the green area in Figure 6.2 as well as the geotextile factory. This means that the quay length of 650 metre can be used only for the construction of the caissons. The geotextile can be transported and installed on massive rolls, see Figure 6.3. During the construction of the Eastern Scheldt barrier these rolls were applied as well and with success. Assuming a width of 50 metre, the required berth is about 100 metre wide. All in all, this means 550 metre should be the minimum quay length of the green area, when assuming 450 metre berth length for the bulk terminal.

Figure 6.3 Bed protection equipment as used for the Eastern Scheldt barrier (Beeldbank Rijkswaterstaat, 1983)



The green area in Figure 6.2 is used as a bulk terminal for the transhipment of the concrete ingredients, for the geotextile and the bed protection. The caissons cannot be constructed at this location since the water depth in this part of the port is insufficient. The length of the quay wall is estimated on 450 metre for two berths for a 180 metre long Handymax vessel. The raw materials can be transported via conveyor belts to the concrete plant. The stone dumping vessels can berth at this location to receive their loads. The orange area is also used to locate the offices and other general units and equipment. The layout is presented in Figure 6.4 (the left side corresponds with the orange area and the right side corresponds with the green area).

Figure 6.4 Layout construction facility

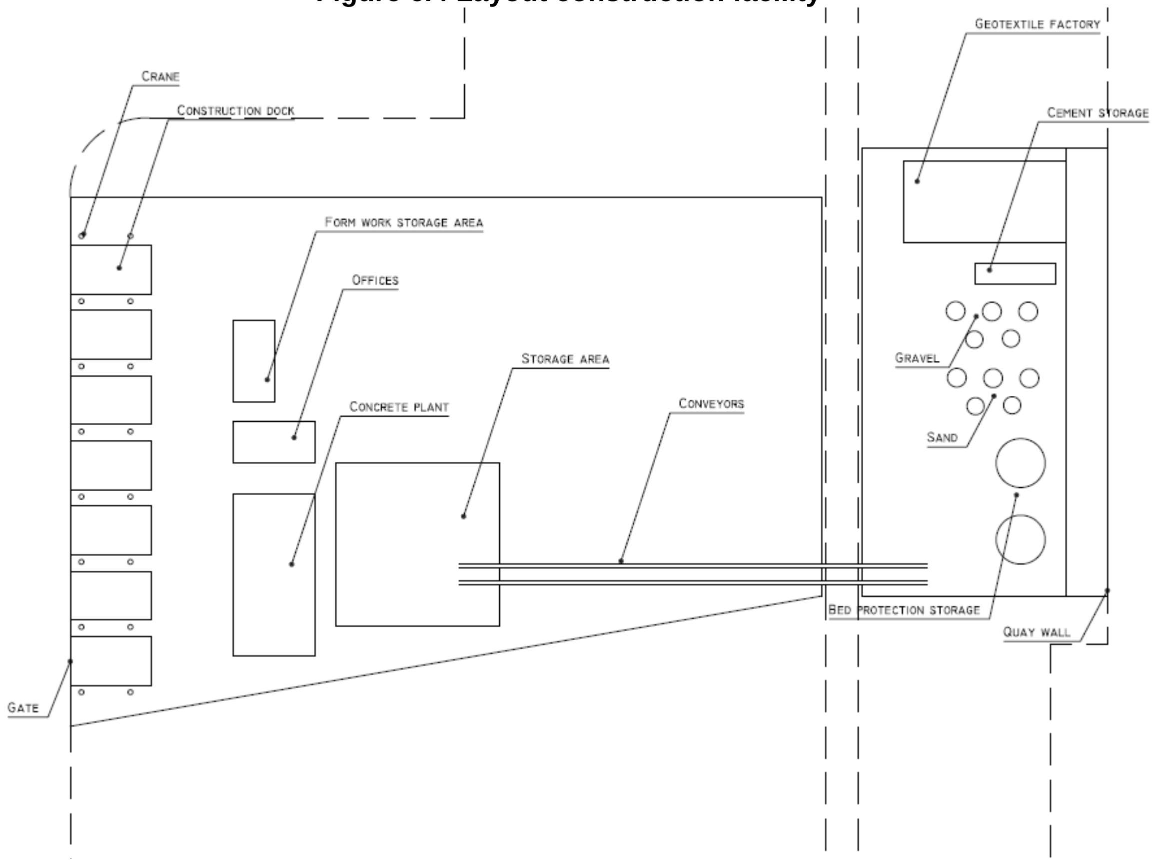


Figure 6.5 Side stone dumping vessel HAM 602

The width of 1 caisson is 38 metre. The distance between two construction docks is estimated on 38 + 40 metre. This means that 7 caissons could be constructed simultaneously along the quay with a length of about 580 metre.

A jetty needs to be constructed at a sheltered location to store the floating caissons before installation. This storage is needed since the caissons cannot always be installed at the moment they float out of the dock. One reason for the waiting time is that batches of 7 caissons are finished within 1 week and the next delivery is only after 13 weeks. The weather conditions can also make the installation of the caissons impossible. So a jetty where 7 caissons can be stored is required at a sheltered location. The caissons are transported to the final destination using tugs, see Figure 6.7.

Figure 6.6 Storage jetty caissons

Figure 6.7 Transportation of the caissons using tugs (Beeldbank Rijkswaterstaat, 1983)

The area for the management and offices is estimated on 5 000 m^2. The area for the concrete factory is estimated on 200 * 200 metre. The bulk terminal needs to be able to store the supplies for 2 weeks. The supplies for the concrete factory is 28 800 m^3 for two weeks for the caissons only. The cement needs to be stored dry, while gravel and sand can be stored in open storage. The geotextile factory has an area of 100 * 200 metre. The work area east of the caisson docks is estimated on 100 * 580 metre. The transport buffer between all objects is estimated on 20 metre wide. The form work storage area is 100 * 100 metre.

Table 6.1 Estimation areas construction facility

	Area
Management	5 000 m^2
Concrete factory	40 000 m^2
Sand	1400 m^2
Gravel	2 500 m^2
Cement	2 500 m^2
Bed protection & rubble	5 000 m^2
Geotextile factory	20 000 m^2
Form work storage area	10 000 m^2
Transport buffer	20 m

6.1.3. Caisson construction

The caissons are very large, it is therefore not possible to cast the entire caisson in one go. The maximum height for one layer of concrete is estimated on 3.35 metre. This is a common height for climbing form work. Assuming the floor is casted in a separate session, the total number of 11 casts is required to construct a caisson. The next layer of 3.35 metre can be casted after previous layer has cured for one week. So it will take 11 weeks to construct one caisson of 34.5 metre high. The total amount of large caissons is 85. The smaller caissons, with a height of 22 metre, can be constructed more quickly. For this type 7 weeks is required for the construction. The total number of small caissons is 30. The number of weeks mentioned is pure construction time, so it is estimated that 1 week of preparations and one week of cleaning needs to be added. The total time required to construct all the caissons is 202 weeks. This is for a 6 day working week and no vacations or for example delays due to frost. The time could be reduced by switching from a climbing formwork to a sliding formwork, however the amount of concrete required per hour is enormous. Another way to reduce the construction is to use longer caissons. In this way fewer caissons have to be constructed. These are optimization options at a later stage. Assuming one layer for one caisson can be made every day, a volume of about 2400 m^3 concrete is required per day. This is 200 trucks of 12 m^3 per day, or one truck every 2.4 minutes. It is not realistic to expect that this amount of trucks can come to the site every day. It is therefore decided to locate a concrete production plant at the site. The raw materials can be brought in by ship, which reduces the traffic load and guarantees the delivery of concrete.

6.2. Planning of the construction

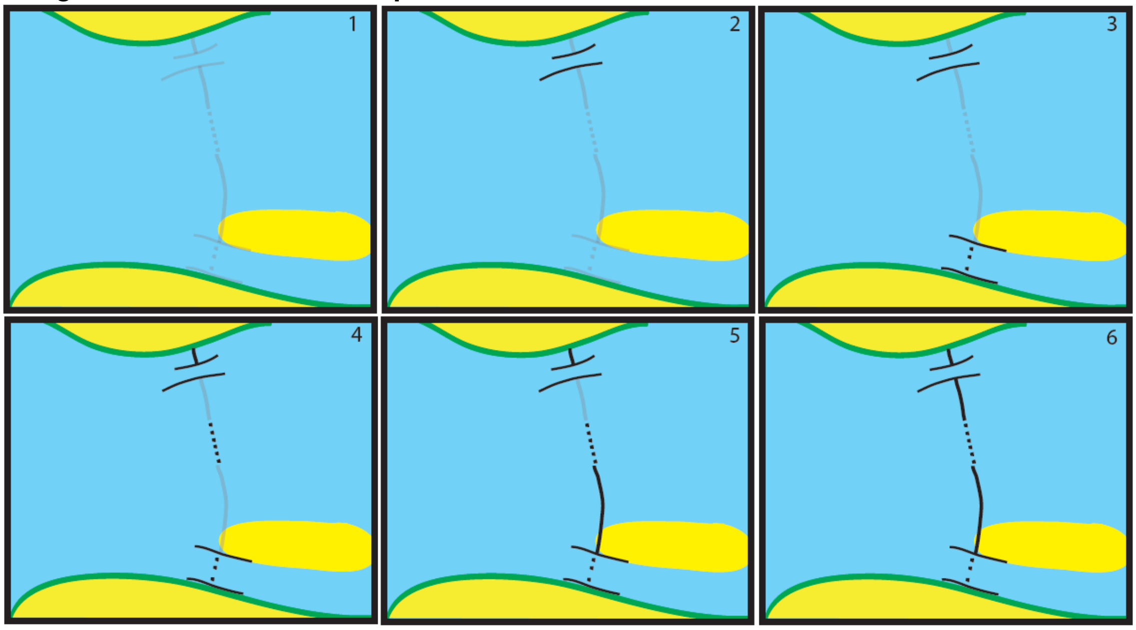
In this section the global planning is presented. The first step is to describe the construction sequence. This is followed by a more detailed list of the durations of the different activities. For the complete planning is referred to appendix VI.

6.2.1. Overall construction sequence

The construction of the barrier can be divided into several main components. The first component is the soil improvement. The bed needs to stabilised and smoothened in order to create a proper foundation for the caissons. Before the caissons are placed, the bed protection needs to be installed. By installing the bed protection the subsoil is contained at the correct location. The next step is the placing of the caissons. The caissons for the shipping funnel are placed first in order to let the captains get used to the opening and to protect the construction of the barrier from shipping hindrance. The construction of the barrier does not harm the accessibility of the ports. The southern opening can be constructed at the same time. The southern opening requires additional attention in order not to harm the environment at the 'Hoofdplaat'. The next step is to place the caissons for the middle openings. Once these openings are all fixated, the smaller caissons, which form the heart of the rubble mound dam, can be installed. The rubble mound can be installed quickly after the caissons. The rubble mound part has to be constructed during the summer and must be finished outside the storm season, since the barrier can only function when a complete stretch of dam is finished. The storm season starts in October and lasts until the end of March. The rubble mound dam between the shipping opening and the middle opening is the last part to be closed, since the impact on morphology is expected to be smallest. In case the part between the southern opening and the middle openings has to be constructed last, the stability of the 'Hoofdplaat' is endangered.

The construction sequence as explained above is presented in the figure below.

Figure 6.8 Construction sequence of the reduction barrier in the Western Scheldt



6.2.2. Estimation of the duration of the activities

The time required for the soil improvement, bed levelling and installation of the bed protection needs to be determined. About 740 hectare of bed protection needs to be placed. One roll of geotextile has an area of 1 hectare. Assuming 3 rolls can be produced and installed per week (one per 2 days), 250 weeks are required for the installation of the geotextile in total. Afterwards additional ballast needs to placed, but this can be done parallel to the installation of the geotextile. So it is estimated that about 250 weeks the entire bed protection can be installed. The parts for the openings are done first since these locations must be finished before the caissons can be placed.

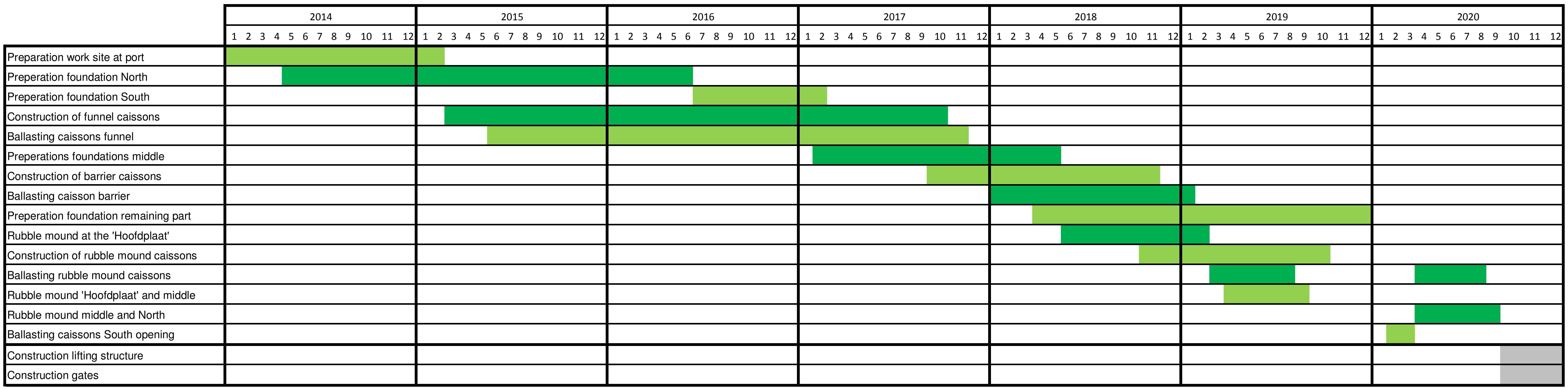
The construction of the caissons is the main item in the construction of the barrier. The duration of the caisson construction is such that this is very likely to be part of the critical path. All the other item should be planned in such a way that the construction and placing of the caissons can continue at a steady rate.

The different stages of the construction and their durations are indicated in the table below. The durations are estimates and need to be verified in a later stage. All the construction times are multiplied with a factor 1.15 in the planning to take into account vacations and other delays.

Table 6.2 Construction stages with durations

	Construction stages	Duration	
1	Preparation work site at port	52	weeks
2	Soil improvement North opening	93	weeks
3	Geotextile North opening	93	weeks
4	Additional ballast North opening	93	weeks
5	Soil improvement South openings	30	weeks
6	Geotextile South openings	30	weeks
7	Additional ballast South openings	30	weeks
8	Construction of funnel caissons	117	weeks
9	Ballasting caissons funnel	113	weeks
10	Connection funnel and levee Vlissingen	10	weeks
11	Soil improvement middle openings	53	weeks
12	Geotextile middle openings	53	weeks
13	Additional ballast middle openings	53	weeks
14	Construction of barrier caissons	52	weeks
15	Ballasting caisson barrier	47	weeks
16	Soil improvement remaining parts	74	weeks
17	Geotextile remaining parts	74	weeks
18	Additional ballast remaining parts	74	weeks
19	Rubble mound at the 'Hoofdplaat'	30	weeks
20	Construction of rubble mound caissons	39	weeks
21	Ballasting rubble mound caissons	33	weeks
22	Rubble mound between 'Hoofdplaat' and middle openings	20	weeks
23	Rubble mound between middle opening and North opening	20	weeks
24	Ballasting caissons South opening	3	weeks
25	Construction lifting structure	30	weeks
26	Construction gates	30	weeks

The construction of several components can be done parallel. The complete planning is presented in appendix VI. A summary of the planning can be found on the next page. The construction of the lifting structure and the gates can happen at a moment later in time once the sea level rise becomes problematic. An advantage is that the gates can be constructed after the opening has been finished. This means that measurement problems are avoided.


2014
2015
2016
2017
2018
2019
2020
1 2 3 4 5 6 7 8 9 10 11 12
Preparation work site at port
Preperation foundation North
Preperation foundation South
Construction of funnel caissons
Ballasting caissons funnel
Preperations foundations middle
Construction of barrier caissons
Ballasting caisson barrier
Preperation foundation remaining part
Rubble mound at the 'Hoofdplaat'
Construction of rubble mound caissons
Ballasting rubble mound caissons
Rubble mound 'Hoofdplaat' and middle
Rubble mound middle and North
Ballasting caissons South opening
Construction lifting structure
Construction gates

6.2.3. Concluding remarks about the planning

The construction of the barrier starts on January 1st, 2014 and is finished on September 24th, 2020. This is without the construction of the gates and lifting structures. The estimated construction time for the reduction barrier is comparable with the construction time of the Eastern Scheldt barrier. This means that the reduction barrier does not lead to significant shorter construction times.

The rubble mound caissons have to be constructed last since those elements are intended to form the closing section since it is more easy to adjust for deviations. The final parts can only be executed outside the storm season. This is the reason for the gap in the planning. It might be a solution to construct a floating breakwater that can be used to continue construction during the storm season. This break water can also be reused in the permanent stage. It is not certain that the floating break water is financially feasible, but it is an option for providing additional shelter and speeding up construction.

6.3. Estimation of the construction costs

The goal of the costs estimation is to calculate the cubic metre price for a reduction barrier. The cubic metre price is calculated by multiplying the length by the height by the water level difference. The costs for the barrier are calculated using unit prices for the materials. The main components are the caissons, bed protection, rubble mound and steel. The price of a storm surge barrier is about €30 000/m^3. This price was also used to calculate the price of the reduction barrier in subsection 4.1.3. It is expected that the price of the reduction barrier is lower. Now that all the main components have been designed, it is possible to calculate the price of a reduction barrier per cubic metre.

There is of course a certain range and uncertainty in the applied prices. The massive size of the barrier can influence the market prices. The size of the project means that the unit price could be relatively low, however the increase in demand might also cause an increase of the unit price. The end results should be considered as a rough price with a significant margin of about 40%.

For the explanations of the item description and the unit costs is referred to the next subsection.

Table 6.3 Cost estimation reduction barrier in Western Scheldt

Item	unit	cost per unit	amount of units	costs per item
Reinforced concrete	m3	€ 270	1 131 847	€ 305 598 690
Ballast material	m3	€ 5	4 732 703	€ 23 663 515
Small stones	m3	€ 50	20 249 595	€ 1 012 479 750
Big stones	m3	€ 100	8 220 401	€ 822 040 100
Geotextile	m2	€ 5	7 400 000	€ 37 000 000
Steel doors	ton	€ 2 000	8 620	€ 17 240 000
Mechanical equipment	-	-	50%	€ 8 620 000
Construction quay	m3	€ 20 000	40 000	€ 800 000 000
Construction area	m2	€200	300 000	€ 60 000 000
			Subtotal	€ 3 086 642 055
Other			20%	€ 617 328 411
Overhead	-	-	10%	€ 308 664 206
			Total:	€ 4 012 634 672
			m^3:	187 200
			Price per cubic metre:	€ 21 435

The price of € 30 000/m^3 is in the 2010 prices. This price has been calculated with a discount rate of 4% which is a reasonable level. In order to compare the price of this reduction barrier, the price must be converted to the prices of 2010. For this calculation a discount rate of 4 % is used as well. The result is a price of € 19 818/m^3. This seems much lower than the storm surge barrier price, but that price was an average. The price for the Eastern Scheldt barrier was much cheaper at about € 18 000/m^3.

The price for the reduction barrier is lower when the part of the rubble mound dam is replaced be barrier caissons, as is indicated in appendix V. This results in a cost reduction of about € 2 100/m^3. This would make the reduction barrier about as expensive as the storm surge barrier in the Eastern Scheldt.

The price for the reduction barrier must be adjusted in order to compare it with the Eastern Scheldt barrier, since the Eastern Scheldt barrier does not have to let vessels pass. Therefore the price for the reduction barrier is calculated as well for the situation without the shipping funnel. The costs are lower since fewer caissons have to be build and the bed protection is less heavy.

Table 6.4 Cost estimation reduction barrier without shipping funnel

Item	unit	cost per unit	amount of units	costs per item
Reinforced concrete	m3	€ 270	473 895	€ 127 951 650
Ballast material	m3	€ 5	1 988 955	€ 9 944 775
Small stones	m3	€ 50	18 399 595	€ 919 979 750
Big stones	m3	€ 100	2 172 401	€ 217 240 100
Geotextile	m2	€ 5	5 550 000	€ 27 750 000
Steel doors	ton	€ 2 000	8 620	€ 17 240 000
Mechanical equipment	-	-	50%	€ 8 620 000
Construction quay	m3	€ 20 000	40 000	€ 800 000 000
Construction area	m2	€200	300 000	€ 60 000 000
			Subtotal	€ 2 188 726 275
Other			20%	€ 437 745 255
Overhead	-	-	10%	€ 218 872 628
			Total:	€ 2 845 344 158
			m^3:	187 200
			Price per cubic metre:	€ 15 199

In the price of 2010 the price is € 14 052/m^3. This means that the reduction barrier is indeed cheaper as the storm surge barrier, as was expected. It must be stated that this is all because of the quality of the current flood protection around the Western Scheldt. In case the flood protection was of poor quality the reduction barrier is not an option. The price of the barrier without shipping could be even lower since the water flow is less fast and less turbulent due to the absence of the propeller jet. This means that the scour hole gets less deep, in turn shortening the length of the bed protection. But this is not included in the calculation.

6.3.1. Explanation unit costs

The unit costs are very important for the price of the total barrier. It is not possible to use exact prices since the prices are market driven. The explanation of the unit prices is given below.

The unit price for concrete is about €60, but this is just the concrete. So reinforcement should be added, as well as labour, form work and the costs for the onsite concrete plant.

When assuming the prices for labour, reinforcement, and formwork are also about €60 each and the remaining items are at €30 combined; the unit price for the concrete is €270.

The ballast material is sand and a common unit price for sand is €5. It is assumed that the sand can be found close by, so supply of sand is relatively simple.

Bed protection is defined as the additional ballast on top of the geotextile. These small rocks have to be transported from elsewhere. The placing is not complicated, so the unit price is estimated on €50.

The larger stones are more expensive due to the larger size and placing is more difficult. It is therefore estimated that these are twice the price of the small rocks, so the unit price is €100.

Geotextile is not very expensive at €2/m^2, however in this case ballast blocks have to be attached to the textile and the smaller strips must be connected to form one big roll. The unit price therefore estimated on €5/m^2.

The cost of the steel doors is calculated per ton of steel. The steel price is very dynamic, but the construction of the gates is not critical, so a unit price of €2 000 is estimated, including labour and other additional costs.

The mechanical equipment is estimated on 50% of the price of the gates. This is a rough estimation, but the impact on the total price is limited (about 0.2% of the total price). So for now this is accurate enough.

The construction of the several quay walls is estimated on €20 000/m^2. The square metre is defined as length time retaining height. The possible sale of the quay wall after the construction is not included in the price. However it should be kept in mind in the design of the quay wall that the quay wall could be used for other purposes after completion of the barrier.

The costs for the erection of the construction area are estimated on about €200/m^2. This is for pavement and other facilities like electricity and water.

Finally two additional items, other costs and overhead, are estimated on respectively 20% and 10% of the subtotal. These items include management and for example risks, insurance and interest.

6.3.2. Conclusion about construction costs

The construction costs of the barrier turn out to be lower than the previously estimated €30 000/m^3. The costs are also lower than the costs of levee heightening along the Western Scheldt. The mayor costs are the bed protection and the construction docks. The barrier could be a lot cheaper in case ships did not have pass the barrier, since the funnel structure and a large part of the bed protection could be deleted. The impact on the environment is not included in the costs calculation. So an environmental impact assessment or a cost benefit analysis should be able to answer this question. However the results are positive for the feasibility of the reduction barrier in the Western Scheldt.

6.4. Life cycle of the reduction barrier

The maintenance of the barrier as well as the end of the life of the barrier is discussed in this section. It is important to think about the end of the life cycle of the structure before it is built, so there are no surprises after the life cycle ended.

6.4.1. Maintenance of the barrier

The maintenance of the barrier during the entire lifetime is mainly the determined by the moveable parts. So the gates and mechanical equipment will require most attention during inspection. The corrosion of the reinforcement in the funnel and the barrier caissons is another important aspect for the inspection and maintenance. The funnel caissons need to stay intact, otherwise the stability of the caisson may threaten the navigability of the funnel.
The rubble mound part of the dam should be checked after a storm. The bottom protection and the development of the scour holes is another major element in the maintenance of the barrier.

6.4.2. What to do after 100 year

The design for the barrier has to be made for a period of 100 years. When this period is over the barrier will still be there. There are basically three options. Which option is chosen depends on the sea level rise.

In case sea level rise appears to be severe, the best option is to make the barrier into a dam. This way that the hinterland is protected and the vessels can pass the barrier through for example locks. The Western Scheldt will be turned into a lake. This will change the environment significantly, but the flood protection will be given the priority.

In case the sea level is less than expected, the barrier can still fulfil its function. It might be necessary to close some of the opening permanently, but the principle of the barrier will still stand. A problem for this scenario is the durability of the geotextiles in the bed protection. This will degrade over time and may threaten the foundation of the barrier.

The final option is to remove the barrier altogether. This will be very costly and will have a massive impact on the environment. This can be done when there is no need for flood protection due major changes in the locations of cities or other massive social changes.

7. CONCLUSIONS AND RECOMMENDATIONS

This chapter contains the conclusions and the recommendations that follow from the research done in this Master Thesis. The conclusions have been divided in a general conclusion and conclusions with regard to the designed reduction barrier for the Western Scheldt. The recommendations are all related to the reduction barrier in the Western Scheldt.

The conclusions are related to the goal as presented in the first chapter. The goal is to investigate whether flood protection, shipping and environmental protection can go hand in hand at a competitive cost level by means of a reduction barrier. The goal has been achieved by studing the principle of the reduction barrier and by designing a reduction barrier for the Western Scheldt.

7.1. Conclusions

General conclusions

- A reduction barrier is more effective for estuaries for which the estuary length is about 1/4th of the tidal wave length

Conclusions with regard to the Western Scheldt

- A reduction barrier in the Western Scheldt is technically feasible
- The reduction barrier in the Western Scheldt will reduce the tidal wave and not a storm surge wave
- The reduction barrier is feasible because of the relatively high quality of the levees along the Western Scheldt
- It is safe for sea going cargo vessels to pass the reduction barrier in the Western Scheldt during normal conditions
- The construction costs are not significantly lower than heightening of the levees
- The construction costs for the reduction barrier in the Western Scheldt are an estimated €21 500/m^3, which is significantly lower than the average unit costs for a storm surge barrier at €30 000/m^3
- The construction time of the reduction barrier in the Western Scheldt, about 7 years, is comparable to the construction time of the Eastern Scheldt storm surge barrier

7.2. Recommendations

- Wave climate, soil conditions and flow conditions investigations will have to be carried out for further design stages
- 2D and 3D hydro dynamical and morphological calculations should be done for the discharge cross-sections optimization. The morphological effect of the barrier should be investigated as well as the effect on local wind set up and wave conditions
- Fast and real time shipping simulations should be performed to optimize the shipping openings
- It should be investigated whether the caisson length can be optimized
- Environmental impact assesment must be executed. For example to investigate the impact of the tidal prism reduction

7.3. Remarks

This thesis discusses merely the technical and economic feasibility of a reduction barrier in the Western Scheldt. This thesis does not discuss the political problems of the construction a barrier which has an impact on both the Netherlands and Belgium. The recent conflict about the flooding of the Hedwigepolder indicates the difficulties managing the interests of both countries. The flooding of the Hedwigepolder has been selected as a compensation measure for the deepening of the Western Scheldt. The main difference between the reduction barrier and the deepening of the Western Scheldt is that both countries benefit from the reduction barrier. This should make the realisation of the project easier but it is still a serious political challenge. Nevertheless the reduction barrier is a serious option for the Western Scheldt.

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APPENDIX I FLOW THEORY

In this appendix the flow theory which is used to calculate the water motion in the estuaries, is discussed. Two main approaches are explained, which are the final gap model and the analytical 1D model.

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Linearization quadratic resistance term

The resistance is quadratically related to the flow velocity. In order to make the calculations simpler and faster the relation is linearized. The procedure of the linearization is described below (Battjes, 2002). This method is applied in the final gap calculation.

The total resisted consists of the friction losses and the deceleration losses.

$$W = \Delta H_f + \Delta H_d = \frac{|U| \cdot U}{2 \cdot g} + c_f \cdot \frac{l}{R} \cdot \frac{|U| \cdot U}{g}$$

$$\chi = \frac{1}{2} + c_f \cdot \frac{l}{R}$$

$$W = \chi \cdot \frac{|U| \cdot U}{g} = \chi \cdot \frac{|Q| \cdot Q}{g \cdot A_s^2}$$

$$W \sim |Q| \cdot Q \quad , \quad W = \lambda_1 \cdot |Q| \cdot Q \quad , \quad \lambda_1 = \frac{\chi}{g \cdot A_s^2}$$

$$\text{Suppose: } Q = \hat{Q} \cdot \cos(\omega \cdot t)$$

$$|Q| \cdot Q = \hat{Q}^2 \cdot \cos(\omega \cdot t) \cdot |\cos(\omega \cdot t)|$$

$$\frac{|Q| \cdot Q}{\hat{Q}^2} = \cos(\omega \cdot t) \cdot |\cos(\omega \cdot t)|$$

$$\text{Linearise: } W \approx \lambda_2 \cdot Q$$

The resistance over an entire tidal period should be the same, therefore the resistance is integrated over a tidal period.

$$\int_0^T W \cdot Q \cdot dt = \int_0^T \lambda_1 \cdot |Q| \cdot Q^2 \cdot dt = \int_0^T \lambda_2 \cdot Q^2 \cdot dt$$

$$\frac{\int_0^T \lambda_1 \cdot |Q| \cdot Q^2 \cdot dt}{\int_0^T \lambda_2 \cdot Q^2 \cdot dt} \rightarrow \frac{\lambda_2}{\lambda_1} = \frac{\int_0^{T/4} \cos(\omega \cdot t)^3 \cdot dt}{\int_0^{T/4} \cos(\omega \cdot t)^2 \cdot dt} \cdot \hat{Q} = \frac{8}{3\pi} \cdot \hat{Q}$$

$$W = \lambda_2 \cdot Q = \frac{8}{3\pi} \cdot \chi \cdot \frac{\hat{Q}}{g \cdot A_s^2} \cdot Q$$

Final gap calculation

The final gap calculation method was developed to calculate the water level during the closure of a dam or the water level in a polder after a levee breach. This calculation assumes the water level in the estuary or the storage area to be horizontal. This means that the water level only fluctuates in time and not in space. This assumption can be made for estuaries which are short in comparison with the length of the tidal wave. Once the tidal wave length is 20 times the length of the estuary the error due to the assumption is 5 % (Battjes, 2002). The inertia of the water in the connection between the estuary and the sea can be neglected, due to the small length of this connection. The discharge into and out of the estuary can be written as (Battjes, 2002):

$$Q = A_e \cdot \frac{dh_e}{dt}$$

In which:

Q = Discharge [m^3/s]
A_e = Storage area [m^2]
h_e = Waterlevel [m]

The calculation method linearizes the quadratic resistance term, as described in the previous section. The inertia of the water in the connection between the sea and the estuary is very small, due to the limited length of the connection. Therefore the resistance becomes much more important. The friction is dominated by the exit losses. The result is a first order differential equation:

$$\frac{8}{3\pi} \cdot \chi \cdot \frac{A_e}{A_d} \cdot \frac{\widehat{U}}{g} \cdot \frac{d\zeta_e}{dt} + \zeta_e = \zeta_s$$

In which:

$\chi =$ Loss factor $\left(\frac{1}{2}\right)$ $[-]$
$A_e =$ Storage area $\left[m^2\right]$
$A_d =$ Discharge cross-section $\left[m^2\right]$
$\widehat{U} =$ Flow velocity amplitude $[m/s]$
$g =$ Gravitational acceleration $\left[m/s^2\right]$
$\zeta_e =$ Waterlevel in estaury $[m]$
$\zeta_s =$ Waterlevel at sea $[m]$

The relation between the water level in the estuary and at sea can be written as (Battjes, 2002):

$$r = \frac{\zeta_e}{\zeta_s}$$

$$r = \cos(\theta)$$

$$\tan(\theta) = \omega \cdot \tau = \frac{8}{3\pi} \cdot \chi \cdot \left(\frac{A_e}{A_d}\right)^2 \cdot \frac{\omega^2 \cdot \zeta_e}{g}$$

$$\Gamma \equiv \frac{\omega \cdot \tau}{r} = \frac{8}{3\pi} \cdot \chi \cdot \left(\frac{A_e}{A_d}\right)^2 \cdot \frac{\omega^2 \cdot \zeta_s}{g}$$

In which:

$\theta =$ Phase shift $[rad]$

$\omega =$ angular velocity $[rad/s]$

$$\tau = \frac{8}{3\pi} \cdot \chi \cdot \frac{A_e}{A_d} \cdot \frac{\widehat{U}}{g} = \text{ Timescale}$$

These formulas are the input for the MatLab calculation. For several values of 'r' are corresponding values of 'θ' and 'Γ' calculated. These parameters can be used to plot the relation between the water level at sea and the water level in the estuary versus the relation between the opening of the estuary and the storage area of the estuary. The MatLab-script can be found below:

The results of the calculation are written in a general form in order to make results widely applicable. In Figure I. 1, Figure I. 2 and Figure I. 3 are three graphs presented which contain the results of the calculation. The first graph displays the relative amplitude on the y-axis and the relationship between the storage area and the discharge area on the x-axis. The second graph presents the relation between the relative amplitude on the one hand and the phase shift on the other hand. In the final graph is the flow speed connected to the relative amplitude of the water level elevation. These graphs can be used for small size estuaries, since the assumptions of the calculation method must be met. The maximum length of the estuary is 1/20th of the tidal wave length.

Figure I. 1 Relation between Surface area/Discharge area and relative amplitude

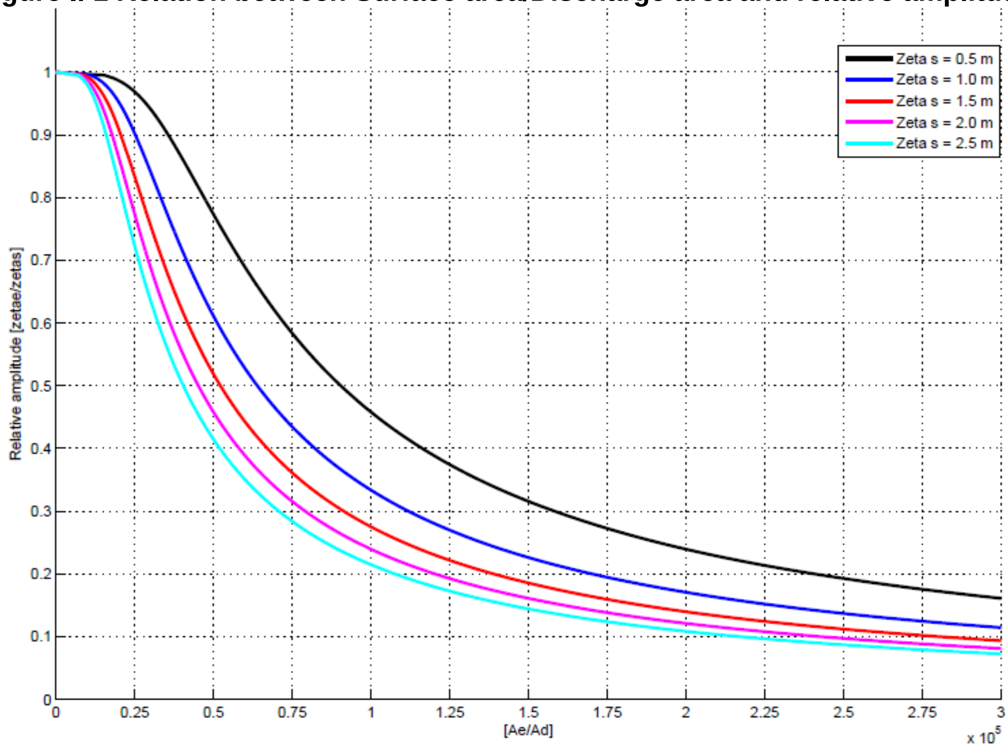


Figure I. 2 Relation between Phase shift and relative amplitude

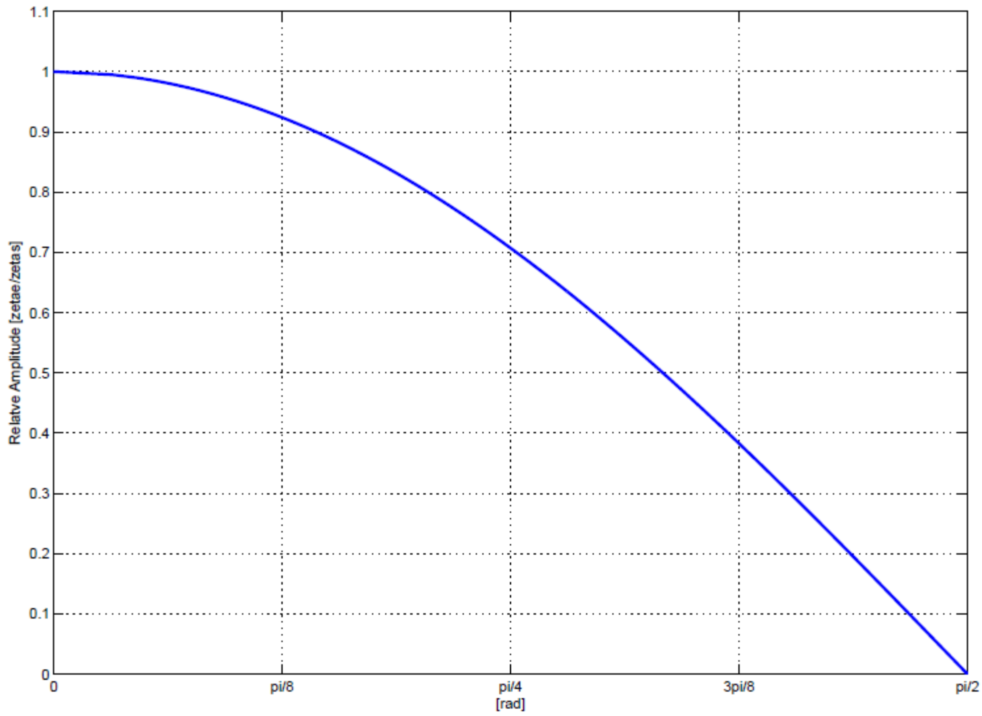
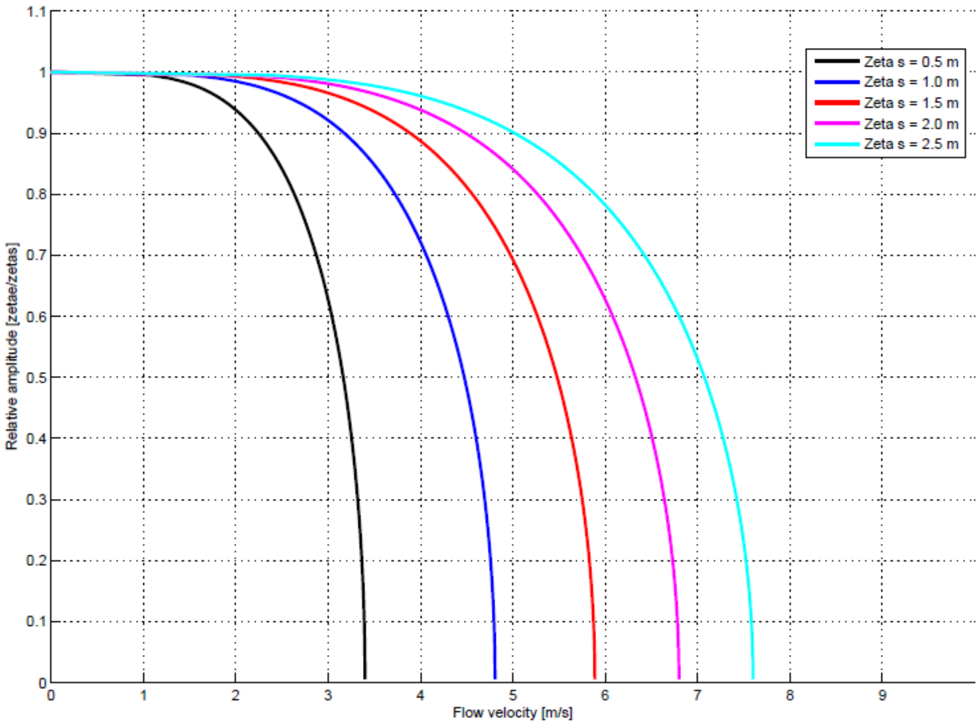


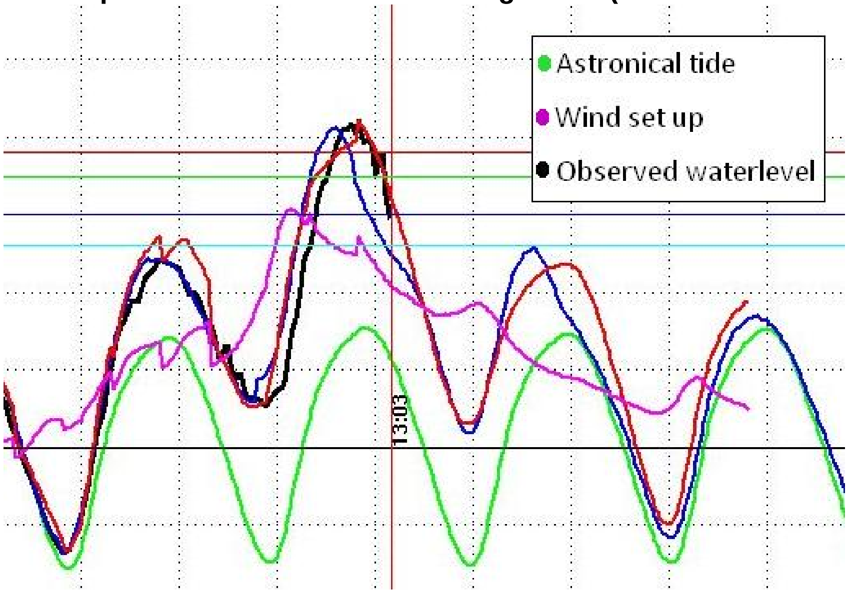
Figure I. 3 Relation between flow velocity factor and relative amplitude



The graphs as presented in Figure I. 1, Figure I. 2 and Figure I. 3 contain information about the water level elevation, phase shift and flow speed during a normal tide of 12 hours and 25 minutes. Therefore it is essential that these graphs can only be used in situations with a M2-tide.

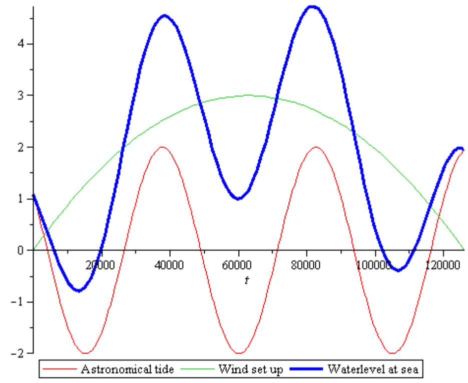
Final gap calculation for storm conditions
During storm conditions in the Netherlands, the tidal elevation is different. Due to wind set up it is common that high water lasts longer as would be expected from the tidal predictions, see Figure I. 4.

Figure I. 4 Example record of water level during storm (8 November 2007, Delfzijl)



The Deltacommision has advised (Deltaprogramma, 2012) that the wind set up duration should be extended to 35 hours while it used to be 29 hours. This has consequences for the calculations as presented above. The shape of the tidal waves is changed due to the wind set up, therefore it is no longer realistic to assume a cosine shape. The longer duration of the high water level results in a larger discharge into the estuary. The final gap calculation as presented above can only be used for a cosine shape tide. Therefore it is necessary that a different approach is made for the situation during storms. A numerical calculation has been performed for the situation with wind set up. The profile of the water level at sea is for course arbitrary, but the results give an indication of the impact of a smaller gap in the reduction barrier. For this calculation a sea water level has been assumed, see Figure I. 5. The sea water level is composed of the astronomical tide with amplitude of two metres and wind set up of 3 metres. The low tide coincides with the peak of the wind set up. The reason for the shift lies in the longer duration of high water with this configuration. Nevertheless a situation with coinciding peaks is checked as well, as can be seen in Figure I. 8 and Figure I. 9.

Figure I. 5 Assumed sea water level during storm (double peaked)



The calculations from the previous section are used to estimate the amplitude of the flow velocity, for the calculation of the parameter ‘τ’. This amplitude is merely an estimation, since the water level is not completely sinusoidal. The impact of this estimation is limited. The backward differentiation method is used to calculate the response of the water level in the estuary. The correctness of the flow velocity assumption is checked afterwards. The assumption and formulas as presented in the previous section are used in this calculation as well.

The results of the calculations can be found in Figure I. 6, Figure I. 7, Figure I. 8 and Figure I. 9. The bold line reflects the sea water level and the other lines represent the water level in the estuary for different opening sizes. The figure shows that the larger opening sizes in the barrier result in higher water levels in the estuary. On the other hand, it is clear that for larger opening sizes the water level responses much quicker to the periods of low water at sea. Figure I. 6 is the situation with a wind set up of 3 metre and a tidal amplitude of 2 metre. In Figure I. 7 are the results presented for the situation with a wind set up of 3 metre and a tidal amplitude of 1 metre. Figure I. 8 and Figure I. 9 present the results for the same situation, with the difference that the peaks of the wind set up do coincide with the peak in the tide.

The results show that the reduction of the estuary opening has relatively more effect for the cases were the tidal amplitude is larger. This should be taken in consideration while selecting a proper location for the Case Study. Since a location with a larger tide requires, relatively, less reduction of the opening size of the estuary. The figures clearly illustrate the effect of the phase shift. The smaller the opening, the larger the phase shift. For very strong reduction of the water level in the estuary can be seen that the phase shift can be considerable. This leads to relatively long periods of high water in the estuaries due to the small opening size. It is possible to open the barrier once the wind set up has disappeared, but this is not included in the calculation. This measure can be effective for reducing the duration of high water in the estuary.

The graphs below are not used in the Case Study. For the graphs which are applied in the case study, is referred to subsection 4.4.

Figure I. 6 Water level in the estuary for different opening sizes

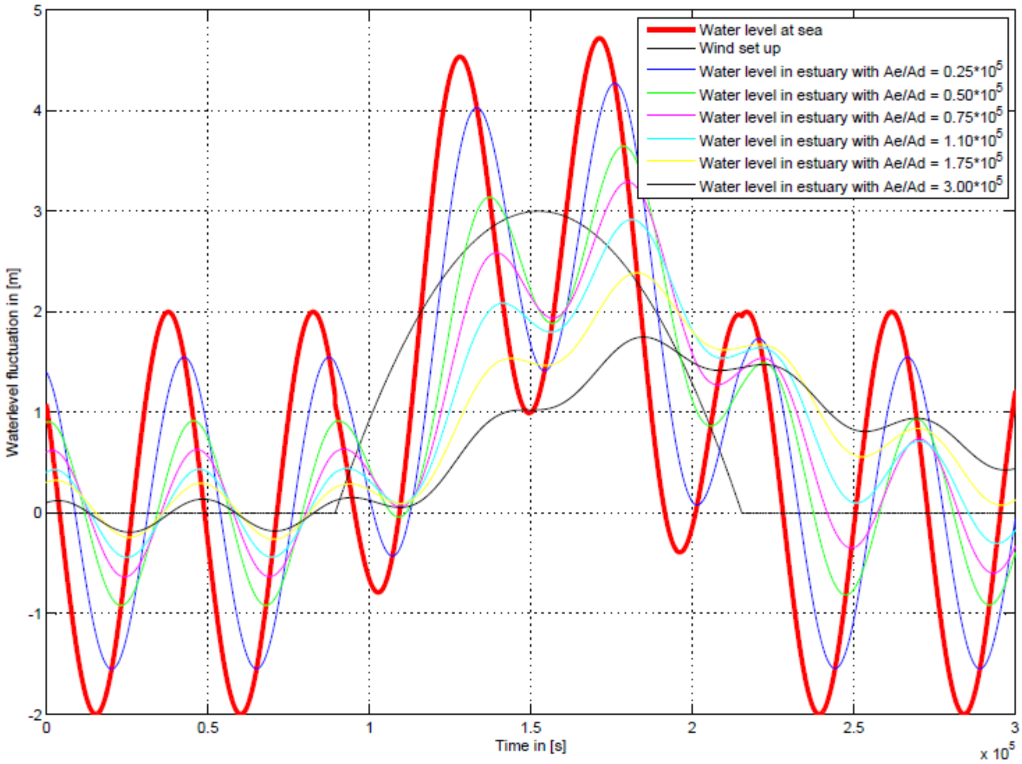


Figure I. 7 Water level in estuary for different opening sizes (2)

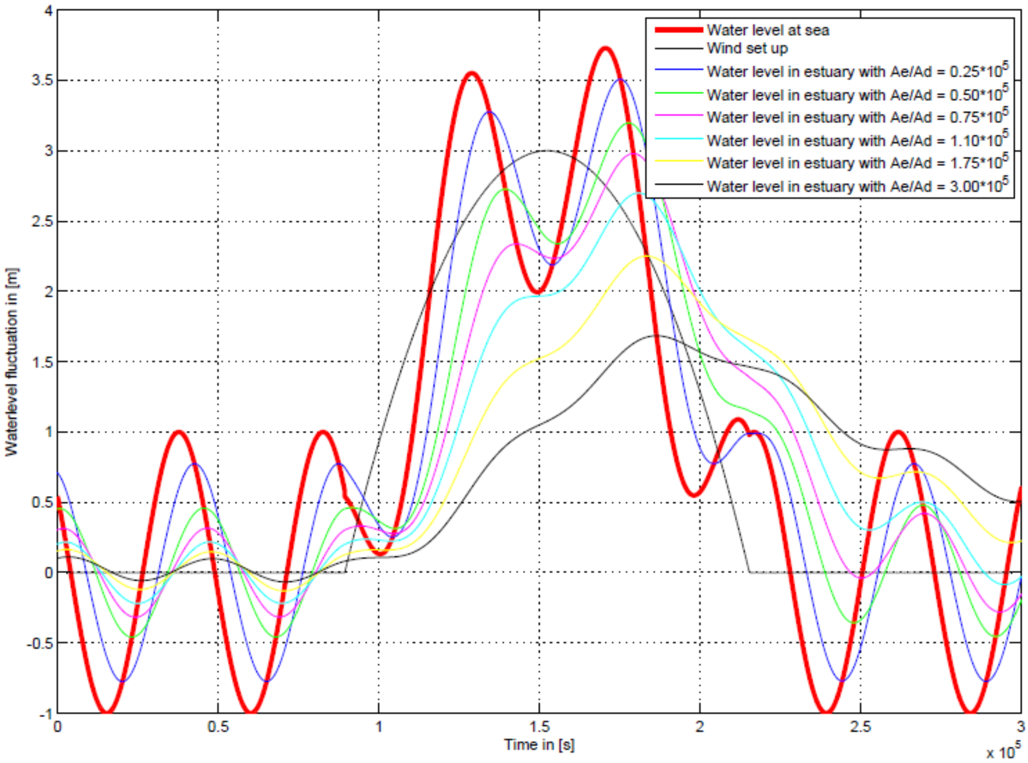


Figure I. 8 Water level in estuary for different opening sizes (3)

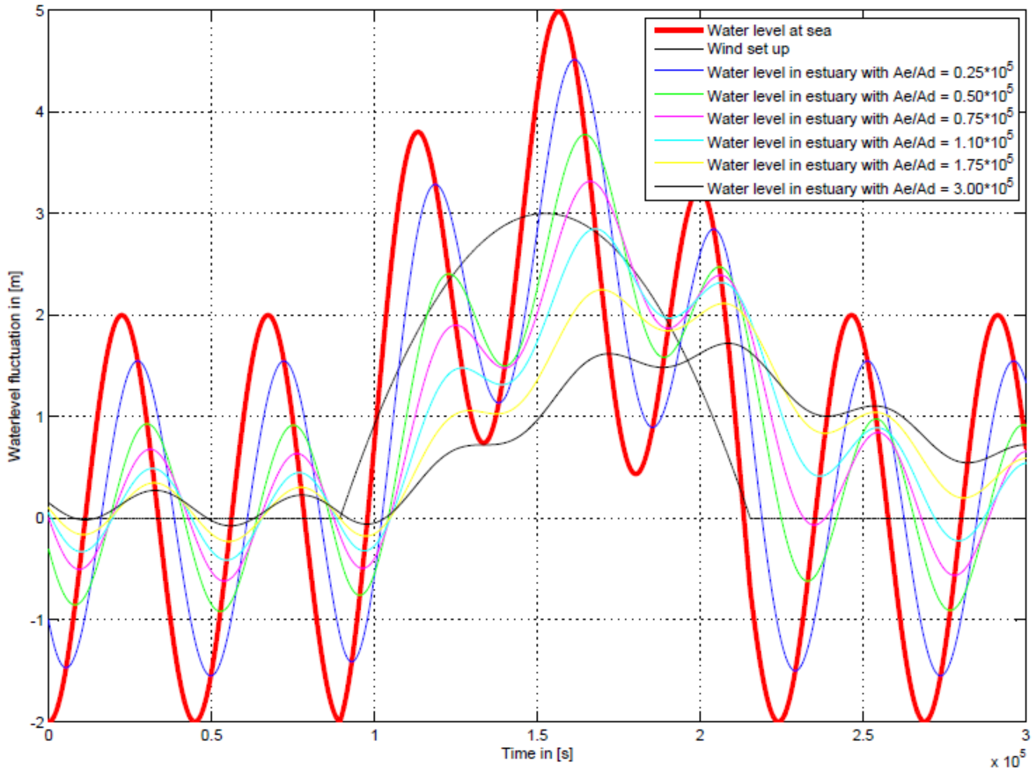


Figure I. 9 Water level in estuary for different opening sizes (4)

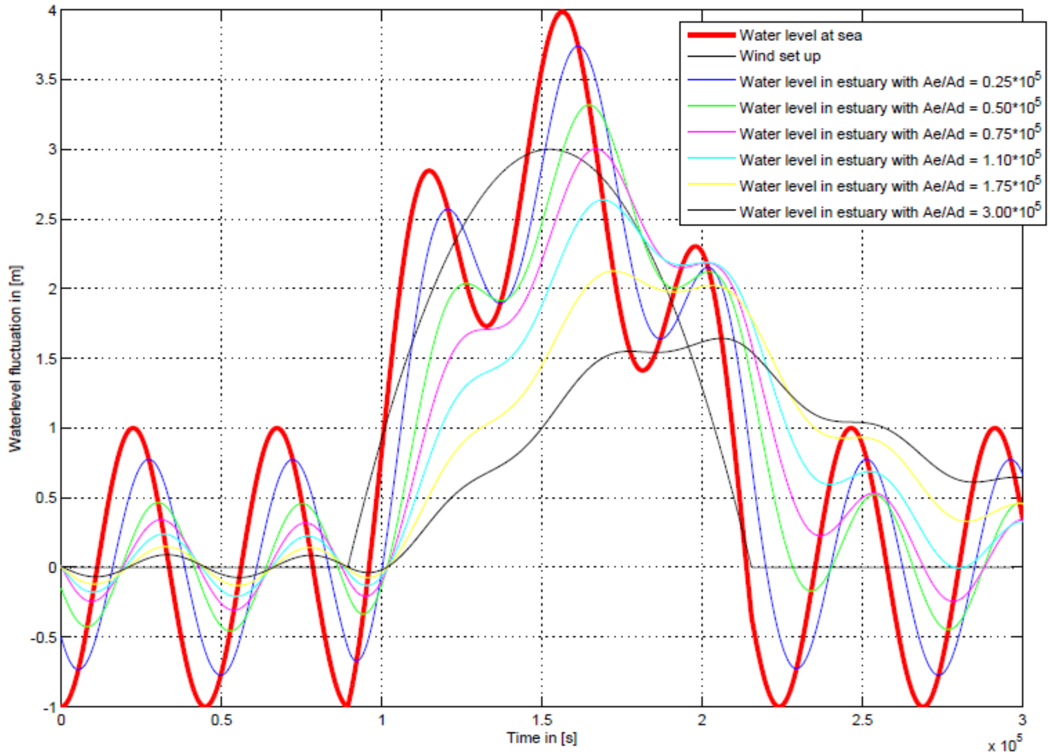


Figure I. 10 The design graph for a storm surge duration of 29 hours

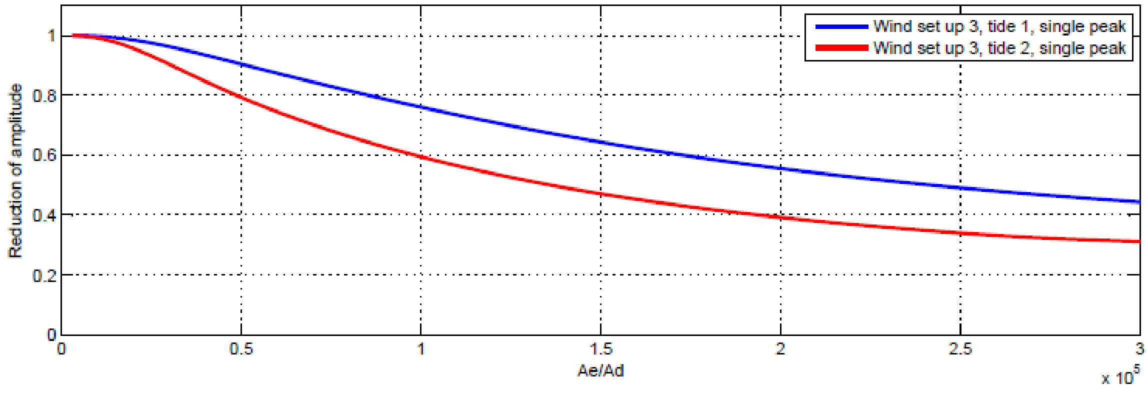
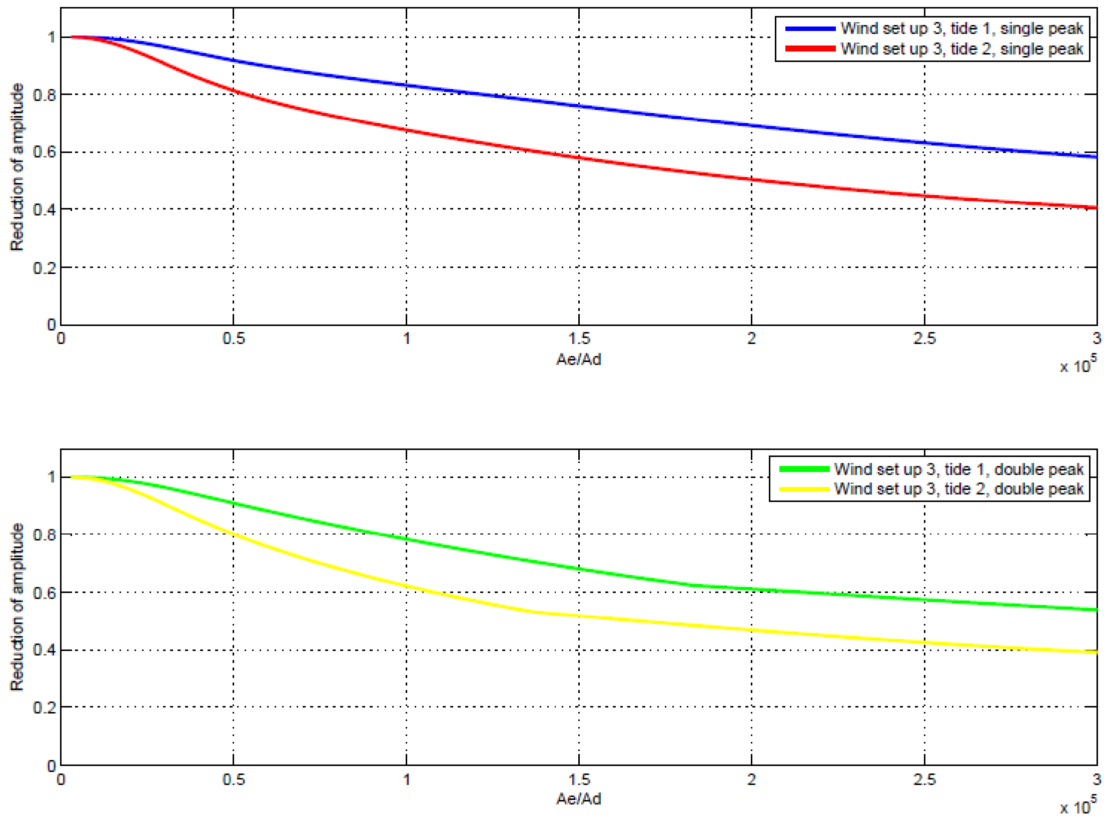


Figure I. 11 The design graph for a storm surge duration of 40 hours



1-D Model
The 1-D model is an analytical method. The model assumes the estuary to be prismatic and to have a constant cross-section over the entire length. In the calculation it is assumed that the estuary is closed at the land side of the estuary. The first step is to look at a general solution, which can be applied for several situations. The basic formulas are described below (Battjes, 2002).

$$\tilde{\zeta}(x) = \tilde{\zeta}(l) \cdot \frac{\cosh(p \cdot x)}{\cosh(p \cdot l)}$$

$$\tilde{Q}(x) = -\frac{i \cdot \omega \cdot B}{p} \cdot \frac{\sinh(p \cdot x)}{\cosh(p \cdot l)} \cdot \tilde{\zeta}(l)$$

$$\tilde{\zeta}(l) - \tilde{\zeta}_{sea} = \frac{8}{3 \cdot \pi} \cdot \frac{\left|\hat{Q}\right| \cdot \tilde{Q}}{2 \cdot g \cdot (\mu \cdot A_s)^2} \cdot \left(1 - \frac{1}{\mu}\right)^2$$

This model can only be used for regular tides, since it is the periodic solution of the periodic excitation. The storm surge is not a periodic excitation. The results do give an idea about the order of magnitude of the discharge area that is effective. The graphs for the flow velocity, water level difference over the barrier and the discharge through the barrier are presented below. The velocity graph shows that the flow velocity through the barrier is limited to a maximum for the longer estuaries. The flow velocity has a maximum of about 2.7 m/s. The tidal wave is composed of waves with different wave lengths, which means that the effects in an actual estuary can be different due to effectiveness for different wave lengths. Different wave lengths can cause unexpected resonance effects.

It must be noted that the discharge opening for all estuary lengths is constant. The storage area increases with the estuary length while the discharge area remains constant. This can explain the larger effect of the reduction barrier of the larger estuary lengths, since the storage area increases with the estuary length.

Figure I. 12 Relative flow velocity for different opening sizes

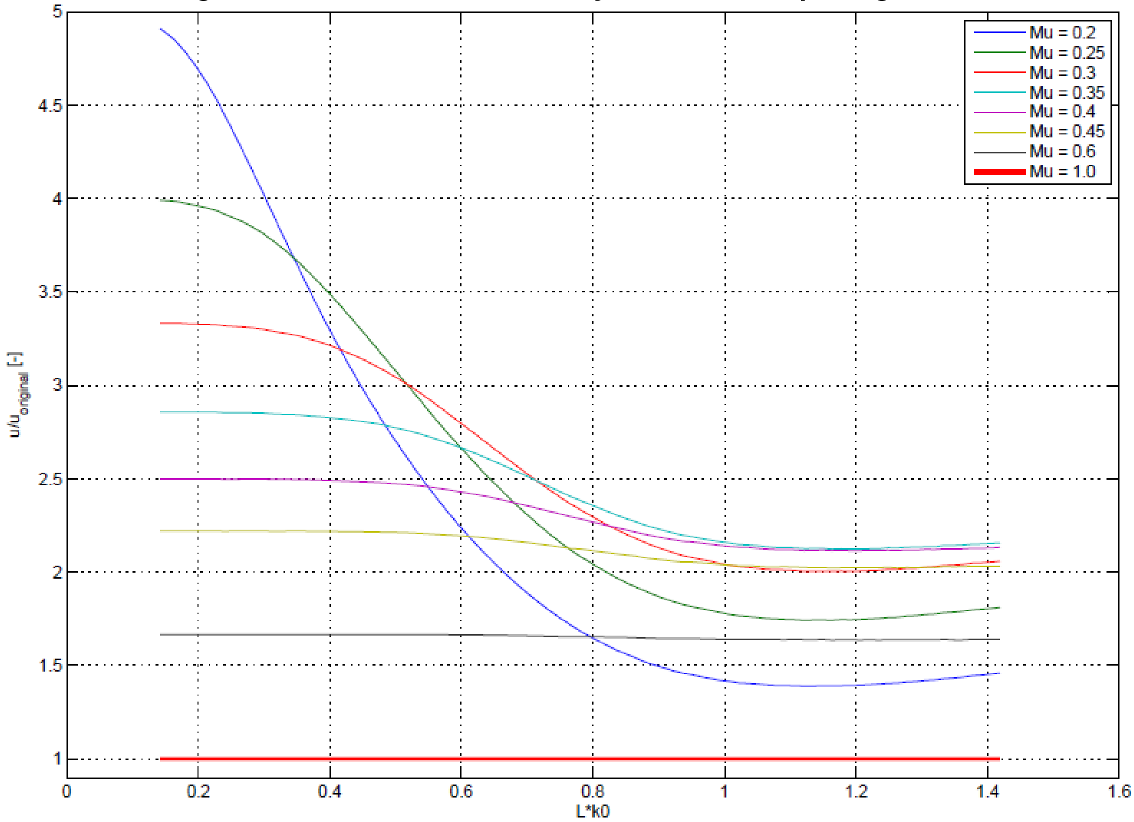
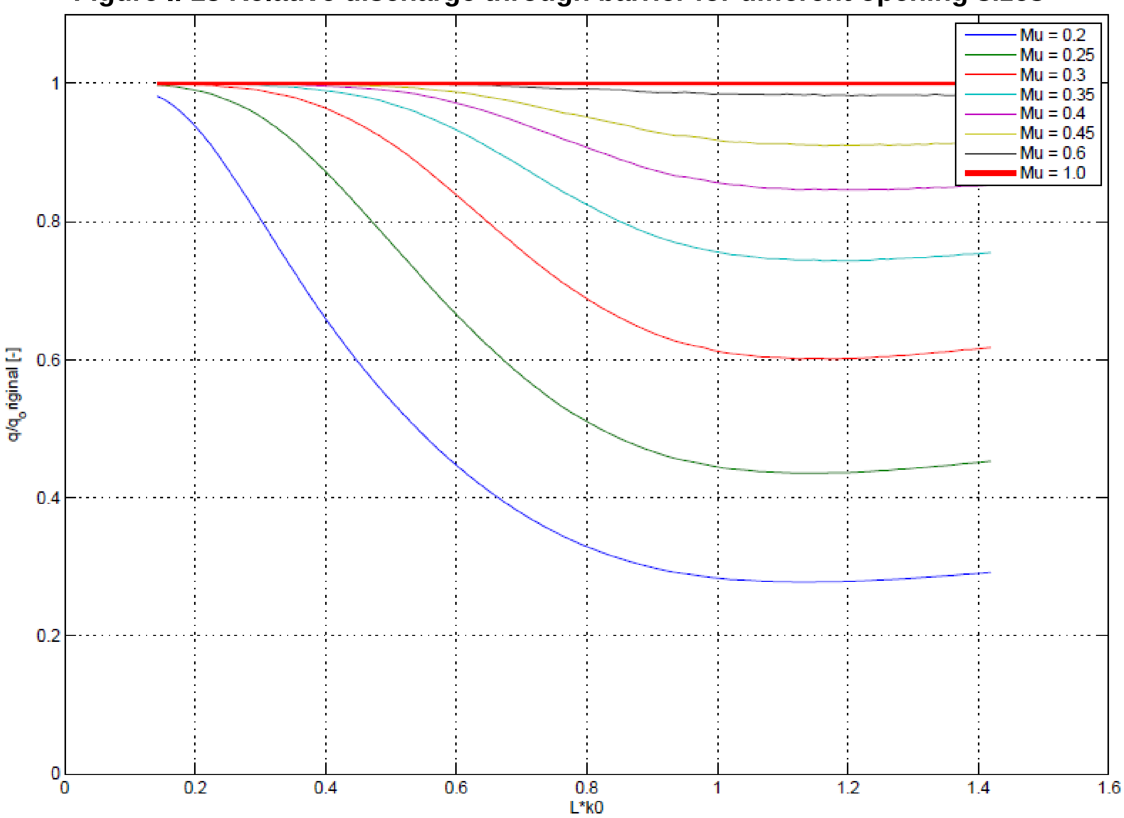


Figure I. 13 Relative discharge through barrier for different opening sizes



APPENDIX II DETAILS POTENTIAL LOCATIONS

In this appendix are the possible locations described for the Case Study. The location with the highest expected potential is selected. For each possible location is checked whether a reduction barrier could be an effective solutions for both the environment and the flood protection.

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Eems Dollard

The Eems Dollard is located in the North East part of the Netherlands on the border with Germany. The exact position of the border has been the subject of disagreements for decades and it still is. The Eems Dollard is the estuary through which the river Eems discharges into the sea. The estuary is part of the Waddenzee and is considered to be an important environmental area. The Germans have constructed a barrier a couple of hundred metre in the Eems. The tidal range in the area is quite large. Emden has a sea port and there are large cruise vessels constructed up stream along the Eems, so a barrier in the Eems should be able to let these vessels pass. The levees on the banks are able to resist the current situation, so they do not require additional attention for the design of barrier. The reduction barrier can be used to avoid the raising of the levees which might be required due to sea level rise. The length of levee which can be located behind the barrier is about 80 kilometre, of which 15 kilometre is close to buildings. So by constructing a reduction barrier, about 660 million euro can be saved. The length of the estuary is such that a reduction barrier might be feasible.

Figure II. 1 Location and lay out of the Eems Dollard (Google Maps, 2012)

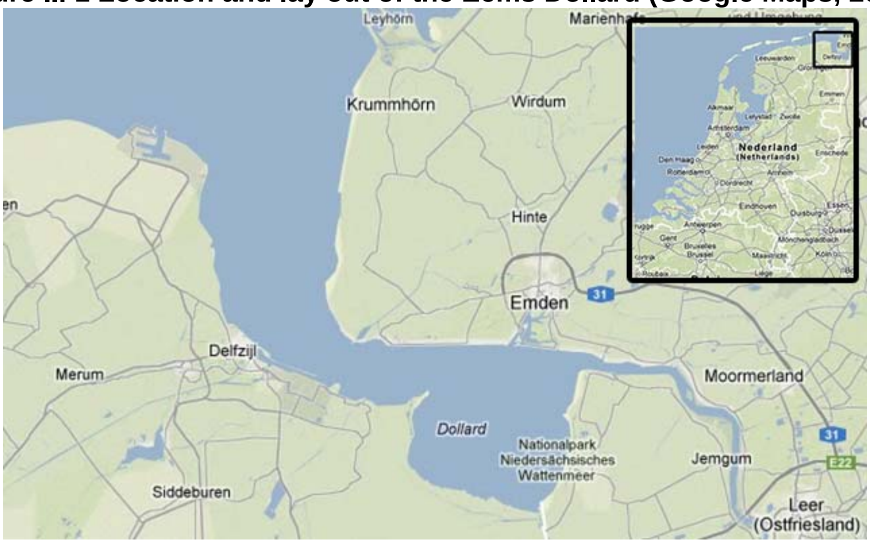

Table II. 1 Main parameters of the Lauwers Lake

	Properties
Storage area	200 km^2
Width connection	8.5 km
Length	32 km
Tidal amplitude	1.5 m
Shipping	Recreational vessels, inland vessels, sea vessels
Other discharge into the estuary	Locks, river and pump discharges

Lauwers Lake

The Lauwers Lake is a lake in the northern part of the Netherlands. The lake used be part of the Waddenzee, but was closed in order to reduce the flood risk, see Figure II. 2. The lake has mainly recreational functions. The lake and a large part of the surrounding land has become a nature reserve area. The re-opening of the lake could improve in the natural value of the lake, by re-introducing the tidal fluctuation and salt water. The deep part of the lake has large amount of stagnant water. This water can be described as dead. A forest is located at the South side of the lake. The forest is growing bigger and bigger. The tide can reduce the size of the forest and refresh the dead part of the lake. Another advantage can be the deepening of the Waddenzee due to the increased tidal volume. This part of the Waddenzee North of the Lauwers Lake, has been accreting since the closure of the Lauwers Lake. The main parameters of the lake can be found in Table II. 2.

The lake is surrounded by levees which originated from the time in which the lake was still part of the sea. The height of the levee is about 5 m. The levees are old so the levees will require special attention when this location is selected for the Case Study.

The discharge into the lake is mainly caused by pumping stations and sluices. These objects discharge the water from the Frisian and Groningse polder.

Figure II. 2 Location and lay out of the Lauwers Lake (Google Maps, 2012)

Table II. 2 Main parameters of the Lauwers Lake

	Properties
Storage area	2400 hectare water, 6700 hectare land
Width connection	Max 1900 m
Length	9 km
Tidal amplitude	1.3 m
Shipping	Recreational vessels, inland vessels
Other discharge into the estuary	Locks, leakage and pump discharges

The length of the estuary is relatively short, this means that a reduction barrier would not be very efficient for the reduction of the tidal wave. The length of the estuary can be defined as short for this location, which means that the design graph can be used to estimate the opening area. The opening size for the barrier with a reduction of 80% for storm conditions leads to $A_e/A_d=9.5*10^4$. This means that the opening can be 52 000 000/9.5*10^4= 550 m^2.
For normal conditions the tide should be as large as possible, but not so large that the levees have to strengthened a lot. The accretion or erosion of the tidal flats should be considered as well. This can result to be a serious challenge.

The Lauwers Lake will require additional measures for the levees, so the only profits which can be gained are based on the natural value. This means that there is no direct way to balance the costs of the reduction barrier, which makes realising it quite a challenge.

Haringvliet

The Haringvliet is another dammed estuary in the South western part of the Netherlands, see Figure II. 3. The dam in the Haringvliet has 17 sluices to discharge water in the North Sea. During high discharges of the Rhine and the Meuse, these sluices are used to relieve the New Waterway near Hook of Holland. The water in the Haringvliet used to be salt, but since the construction of the dam the water has turned from salt into fresh. One of the functions of the Haringvliet is therefore a fresh water buffer. The Haringvliet used to connect the Biesbosch with the North Sea. The Biesbosch is a nature reserve area, which has changed due to the disappearance of the tide. It is expected that the ecological values of the area will increase a lot once the dam is opened. The government has developed plans to open the sluices in the dam in order to re-introduce the tide. There are reports which suggest that the re-introduction of the tide has positive effects on the environment en safety (WWF, 2010). The re-introduction of the tide would give benefits of 0.5 billion euro per year. Some kind of a reduction barrier could be the solution here. The properties of the estuary are difficult to name since the estuary does not end at the East side but converts in to the rivers Rhine and Meuse. The Haringvliet can therefore not be modelled in a final gap calculation, but has to be analysed in a 1-D model. These rivers are also responsible for the main discharge into the Haringvliet.

Figure II. 3 Location and lay out of the Haringvliet (Google Maps, 2012)

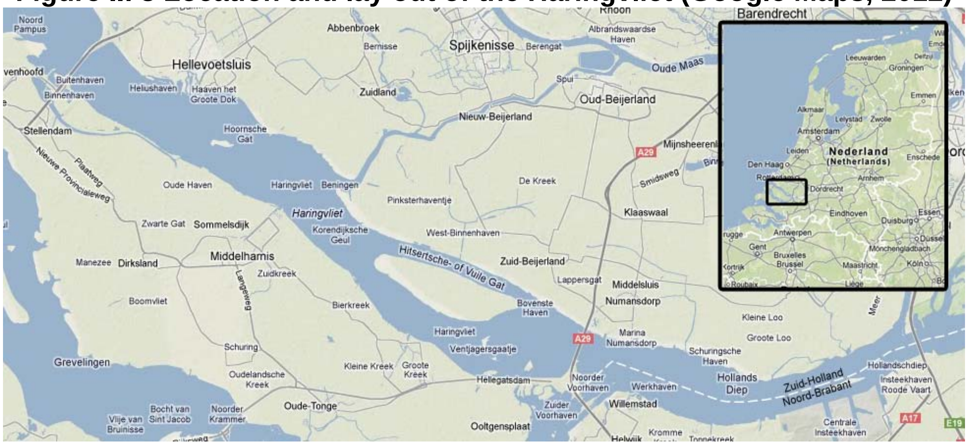

Table II. 3 Main parameters of the Haringvliet

	Properties
Tidal amplitude	1.5 m
Shipping	Mainly recreational vessels
Other discharge into the estuary	Rhine and Meuse

Grevelingen Lake

The Grevelingen Lake is located North of the Eastern Scheldt and South of the Haringvliet, see Figure II. 4. The lake was formed after the construction of the Brouwersdam in the West and the Grevelingendam on the East side. The Grevelingen is a salt water lake, which had some problems with the water quality. This was the reason for the installation of a sluice in the Brouwersdam, which made the water in the North Sea accessible again. The Grevelingen Lake is mainly used for recreation, like sailing and diving. There are no large discharges into the Grevelingen.

The re-introduction of the tide in the Grevelingen does not lead to savings elsewhere, which means that the only economic advantages are the increase of the natural value.

Figure II. 4 Location and lay out Grevelingen Lake (Google Maps, 2012)

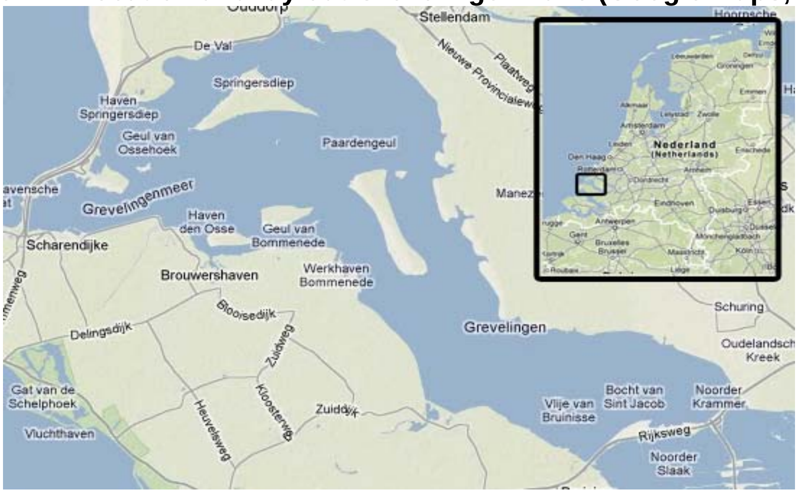

Table II. 4 Main parameters of the Grevelingen

	Properties
Storage area	11 000 - 14 000 hectares
Width connection	5 km
Length	± 24 km
Tidal amplitude	1.5 m
Shipping	Recreational vessels
Other discharge into the estuary	Lock, pump discharges

The reduction barrier in the Grevelingen is not every effective due to the limited length of the estuary. The length of the lake is just on the limit for application of the final gap calculation, assuming an average depth over the length of the lake of 10 metre. A reduction of the tide to 80% is for storm conditions leads to A_e/A_d=0.83*10^5. This means that the opening can be 130 000 000/0.83*10^5= 1 575 m^2.

For normal conditions the tide should be as large as possible. The opening size must be calculated for the new tidal area. The discharge opening should be designed in such a way that there is a significant tide and there will be no serious accretion or erosion in the lake.

Eastern Scheldt

The Eastern Scheldt is currently protected against flooding by means of The Eastern Scheldt Storm Surge Barrier, see Figure 1.1. This barrier is partly open during normal conditions. During storm conditions the gates in the barrier are lowered and the estuary is closed off. The Deltacommission has suggested that the re-introduction of the complete tide should be possible in the future. A reduction barrier could be a solution for the future. The re-introduction of the tide is believed to have positive effects on the environment. The tidal flats in the Eastern Scheldt have to deal with erosion due to changed tidal volume. The erosion has a negative effect on foraging birds.

Figure II. 5 Location and lay out of the Eastern Scheldt (Google Maps, 2012)

Table II. 5 Main parameters of the Eastern Scheldt

	Properties
Storage area	350.76 km^2
Width connection	3150 m
Length	± 42 km
Tidal amplitude	1.8 m
Shipping	Recreational vessels
Other discharge into the estuary	Locks, leakage and pump discharges

The 1-D model can be used to calculate which part of the barrier could stay open during storm conditions without having to change the height of the levee surrounding the Eastern Scheldt. The Ministry of Public Works has issued a document which holds the maximum allowed water level for all primary levees and hydraulic structures in the Netherlands. The maximum allowed water level on the East side of the Eastern Scheldt is 4 metre + NAP. The storm surge level at the entrance of the Eastern Scheldt is 3.5 metre. This storm surge wave cannot be reduced by the opening in the barrier due to the large length of the wave. In case the local wind set is assumed to be 0.30 metre the tidal amplitude has to be reduced from 1.65 metre to 0.2 metre. This would require an opening of 0.10 of the original channel cross-section. The corresponding area is 4000 m^2. With all gates open the discharge area is about 17 000 m^2. So about 25% of the gates could remain open during storm conditions. When it is decided that all the gates are removed the amplitude of the tide in the East side of the basin will hardly be reduced. This would result in the levee heightening of at least 1.65 metre.

Western Scheldt

The Western Scheldt is the only completely open estuary in the Southern part of the Netherlands, see Figure II. 6. A reason the estuary is still open, is that the estuary is part of the approach channel for the Port of Antwerp. Another reason is the environmental value of the estuary. The Belgium's have always opposed the closure of the Western Scheldt, since this could harm the development of their port. The Western Scheldt is a relatively deep estuary and the length of the estuary is large. A storm surge barrier is able to reduce the length of the coastline considerably. Therefore, a barrier is an attractive solution for reducing the flood risk. There is a possibility that the barrier reduces the tidal amplitude at the Port of Antwerp. Whether this is actually the case can be concluded after a calculation with the 1-D model. The main parameters of the Western Scheldt can be found in Table II. 6. The levees are currently part of the primary water barrier. The currently available safety can be used in the design of the reduction barrier. This means that a small reduction of the vertical tide can have a serious effect on the safety level.

Figure II. 6 Location and lay out of the Western Scheldt (Google Maps, 2012)

Table II. 6 Main parameters of the Western Scheldt

	Properties
Storage area	40 000 hectare
Width connection	4000 m
Length	± 60 km
Tidal amplitude	2.0 m
Shipping	CEMT-Vic, E-class Maersk
Other discharge into the estuary	River Scheldt, pump discharges

APPENDIX III ANALYSIS WESTERN SCHELDT

In this appendix the additional information is compiled which is required for the design of the reduction barrier. The first step is the selection of the barrier location in the Western Scheldt. This is followed by an assessment of the maximum allowable water level in the estuary. The next step is to calculate the maximum opening size in the barrier. Other subjects under investigation are the shipping intensity, shipping lanes, wave conditions, soil conditions and the discharge of the river Scheldt.

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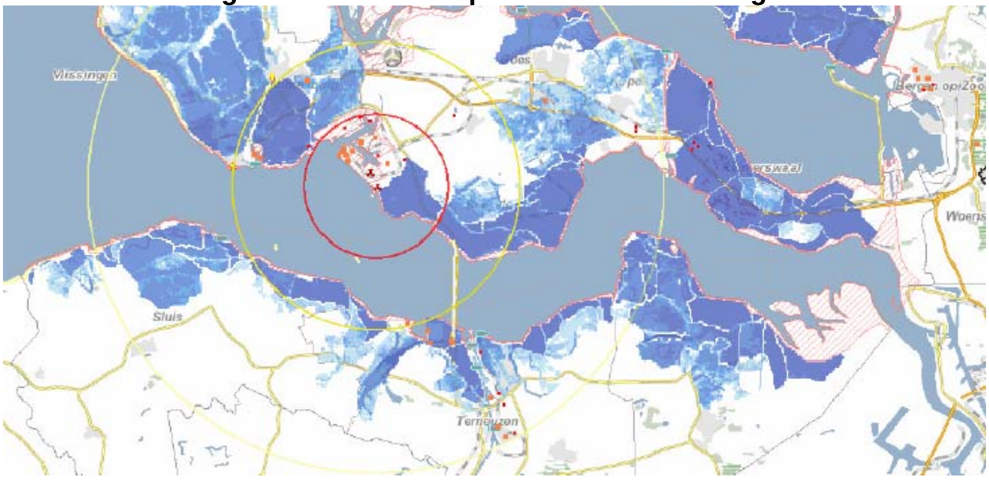
Location aspects of the barrier in the Western Scheldt
The location which is selected is near Vlissingen. This location has the best reduction of the primary water defence length. The nuclear power plant near Borssele is also behind the barrier, which is positive for the safety. In Figure III. 1 is indicated which areas can be flooded. The nuclear power plant is located in the centre of the circles. It is not possible to construct the barrier outside the danger zone of the plant, however the barrier is outside the evacuation zone which is the red circle.

Figure III. 1 Flood depths and nuclear danger



The barrier is positioned in the Western Scheldt in such a way that only one opening for the shipping lane is required, two are still possible. A more westerly location would result in problems for the vessels, since they would have to turn, right before passing the barrier. The width of the Western Scheldt at the location of the barrier is about 6000 metre.

Planned levee heightening
The levee height is one aspect which is important for the design of the reduction barrier, but this parameter results in a maximum water level along the Western Scheldt. This maximum water level is defined by Rijkswaterstaat and forms one of the boundary conditions for the check of the levees. It is essential to know what the state of the levees is currently. In Figure III. 2 is indicated which section of the levees are planned to be improved in which year. It is clear that the maintenance of the levees along the Western Scheldt is up to date.

Figure III. 2 Current state of the primary water defence (Zeeweeringen, 2012)

Check of the hydraulic boundary conditions

The storm surge wave has a very long period, which means that it can penetrate the barrier very easily. The storm surge is not really amplified in the estuary, so a storm surge of 3 metre gives a water level elevation of 3 metre in the back of the estuary. Assuming 0.2 metre water level elevation due to river and other additional discharge, a tidal amplitude of 6.7-3-0.2=3.5 is allowed. The spring tide amplitude in at sea is 2.24 metre, see Table III. 1, which results in an amplitude in the back of the estuary of about 3 metre. This means that there is 0.5 metre available for local wind set up. The fetch of the wind set up depends very much on the wind direction. The maximum fetch for North West wind directions is about 20 kilometre, due to the meanders of the Western Scheldt. With results in a local wind set up of about 0.4 metre. This means that the maximum water level can be explained and that the maximum water level does not require any reduction of the tidal amplitude for the Dutch levees.

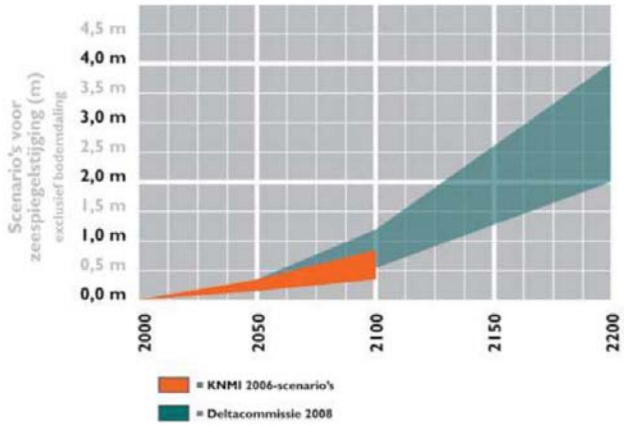
Table III. 1 Properties of the tide near Vlissingen

	High water [cm + NAP]	Low water [cm + NAP]	Tidal difference [cm]
Average tide	205	-181	386
Spring tide	243	-204	447
Neap tide	155	-147	302

Sea level rise

Over 100 years the situation is different since a sea level rise of 1 metre should be expected, see Figure III. 3. The sea level rise of 1 metre is the average of the values predicted by the Deltacommision and the KNMI. Assuming the estuary is able to evolve with the rising sea level a reduction of the tide of 1 metre is required. This means that the maximum tidal amplitude in the back of the estuary is reduced from 3 metre to 2 metre. The levees do not need to be heightened when this reduction can be achieved, since the maximum water level remains the same.

Figure III. 3 Predicted sea level rise (Deltacommision, 2008)

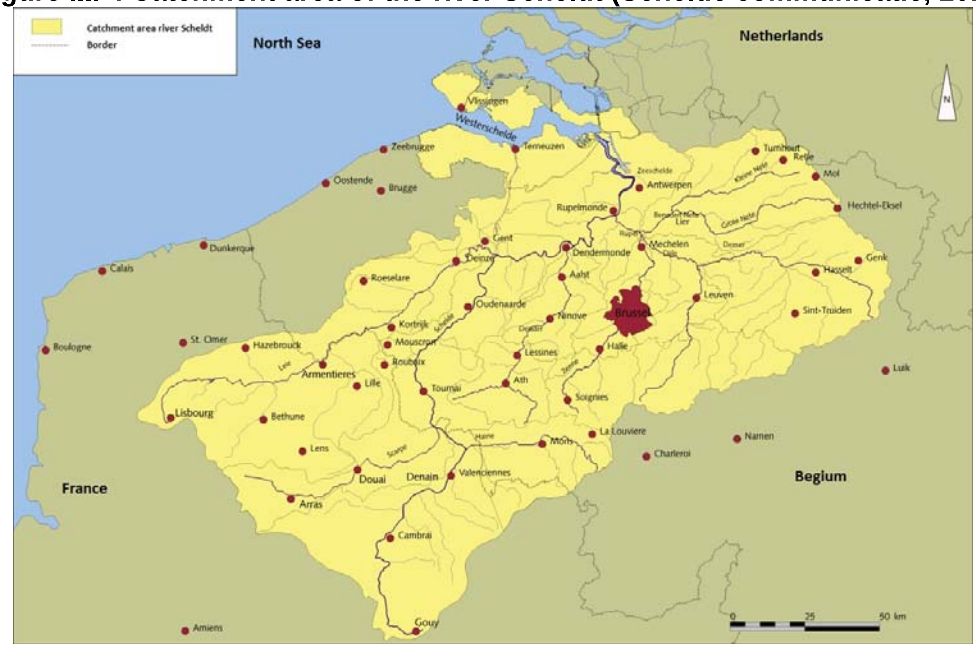


Not just the situation in the Netherlands is important for the determination of the required reduction, but also the levees and water defences in Belgium. It is very well possible that the situation near Antwerp requires a much larger reduction of the tidal wave. A costs benefit analysis for the reduction of the flood risk in Belgium states that a reduction of the water level of 0.5 metre on the border between the Netherlands and Belgium is enough to satisfy the flood risk demands for the Belgians (Gauderis, 2005). So the Belgians require a reduction of the maximum water level of 0.5 metre without sea level rise.

River Scheldt and Sigmaplan

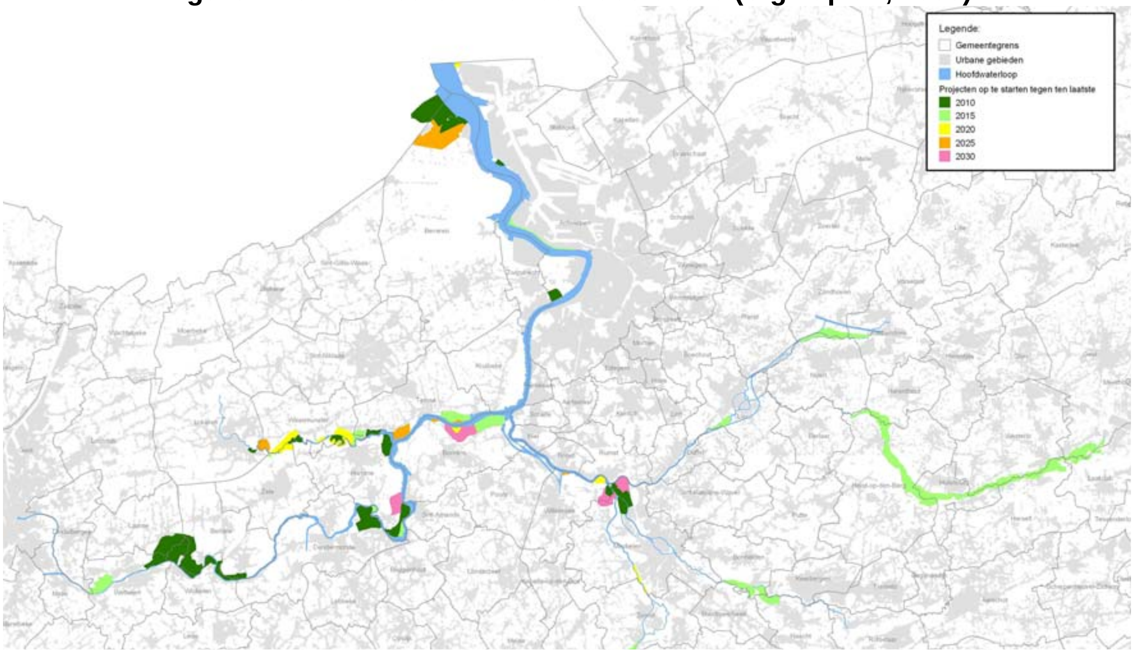
The river Scheldt flows from France through Belgium to the Netherlands to end up in the North Sea. The river Scheldt consists of several branches as can be seen in Figure III. 4.

Figure III. 4 Catchment area of the river Scheldt (Schelde communicatie, 2010)



The Belgium government has initiated a flood protection plan, the Sigmaplan, as a response to the flooding in 1976. The initial plan consisted of increasing the levee heights, constructing controlled flood areas and the construction of a storm surge barrier just north of the city of Antwerp (Sigmaplan, 2005). This plan would give the Belgians the same level of protection as the Netherlands. Later the storm surge barrier was considered to be too expensive, so it was decided to just raise the levees and make several controlled flood areas. As a result of this decision the desired level of protection could not be provided. Therefore the Belgium government decided in 2005 to further increase the levee height and to construct even more flood areas. In order to control a river with the use of flood areas, the capacity of these areas has to be huge. The total area of controlled flood area is currently set on about 1200 hectare. It is questionable whether this enough. It has been investigated that about 4000 hectare is required to reach the desired level of protection (Schelde communicatie, 2012), see Figure III. 5 for an overview of the planned flood areas. The total length of levee that has to be raised is very large, since the controlled flood areas also need to be enclosed by a levee. The Belgium government uses the development of nature as one of the most important aspects of the Sigmaplan.

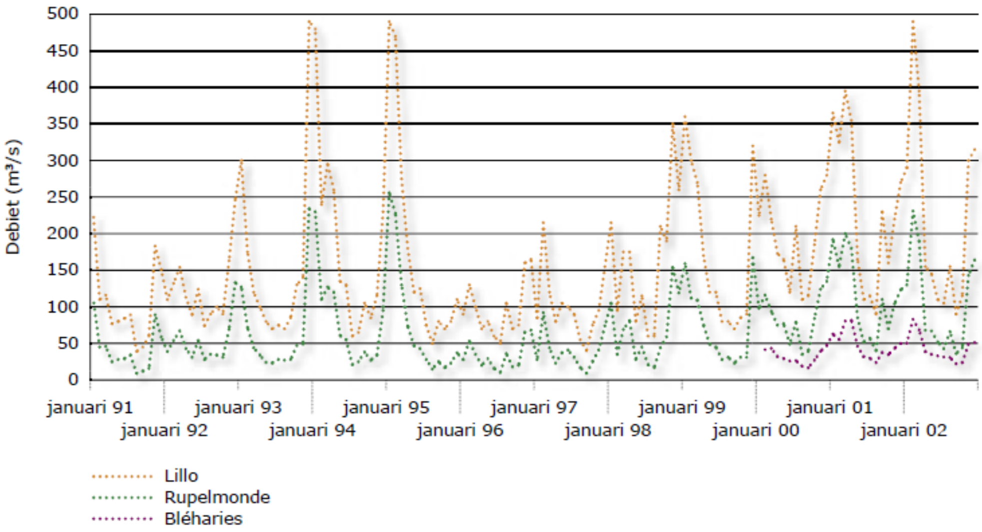
Figure III. 5 Planned controlled flood areas (Sigmaplan, 2005)



Discharge Scheldt

The discharge of the river Scheldt is relatively small. The maximum recorded discharge is about 500 m^3/s. It is not likely that the discharge of the river can become larger, since the Belgians have created several storage areas which can be used in case of large discharge. The storage areas are used reduce the maximum discharges of the river Scheldt. This is part of the Sigma Plan which is the Belgium Delta Plan. Therefore it is possible to say that the discharge of the Scheldt has a maximum of 500 m^3/s, as can be seen in Figure III. 6 (Internationale stroomgebieddistrict van de Schelde, 2007). The yellow dotted line in the figure is the estimated discharge on the border between the Netherlands and Belgium. The average discharge of the river Scheldt is 161 m^3/s (Internationale stroomgebieddistrict van de Schelde, 2007). For more information about the river Scheldt and the Belgium equivalent of the Deltaplan is referred to the previous section.

Figure III. 6 Discharge of the river Scheldt



As can be seen in Figure III. 6 the peak discharges tend to occur during the storm season, which in the winter. The probability of a peak discharge in combination with a storm surge, is not very large. Storms that cause a storm surge come from a North West direction. This means that the river discharge of the storm which is caused by the storm that caused the surge is delayed. However it is possible that two storms come close behind each other from different directions or that high river discharges are the result of melting snow. A discharge of 300 m^3/s is a discharge which has occurred 16 times over the period of 11 years as can be seen in Figure III. 6. The total storage area of the Western Scheldt depends on the position of the barrier, but has a maximum of 43 000 hectare (Van Maldegem, 2005). So a minimum water level rise during a storm of 35 hours of about 0.10 metre should be expected.

Calculation of the opening size in the barrier

In this section is calculated what the opening size of the barrier should be to reduce the water level in the estuary to an acceptable level. Four scenarios are calculated. The first scenario is the reduction of 0.5 metre without sea level rise, since this would guarantee the desired safety level for the Belgians. The second scenario is a reduction of 1 metre to satisfy the situation with sea level rise over 100 years for the Netherlands and also to satisfy the requirement for the Belgians in the current situation. The third situation is a reduction of 1.5 metre in order to satisfy the maximum water level for the Belgians over 100 years with sea level rise. The fourth and final situation is the normal situation, so just the regular tide.

The water level response is calculated numerically. The motion equation is combined with continuity, see the formulas below. The Western Scheldt is divided into 11 sections and for each section is calculated what the water level and the discharge is as a function of time. The local wind set up is not taken into account since the barrier cannot reduce this. The goal of the barrier is to reduce the water level with a certain value. The effect is only reached by reducing the tidal wave. The absolute water level is not very important only the reduction caused by the barrier. The local wind set up must be included in a 2D or 3D flow calculation in a later stage.

$$B \cdot \frac{\partial h}{\partial t} + \frac{\partial Q}{\partial s} = 0$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial s}\left(\frac{Q^2}{A_s}\right) + g \cdot A_s \cdot \frac{\partial h}{\partial s} + c_f \cdot \frac{|Q| \cdot Q}{A_s \cdot R} = 0$$

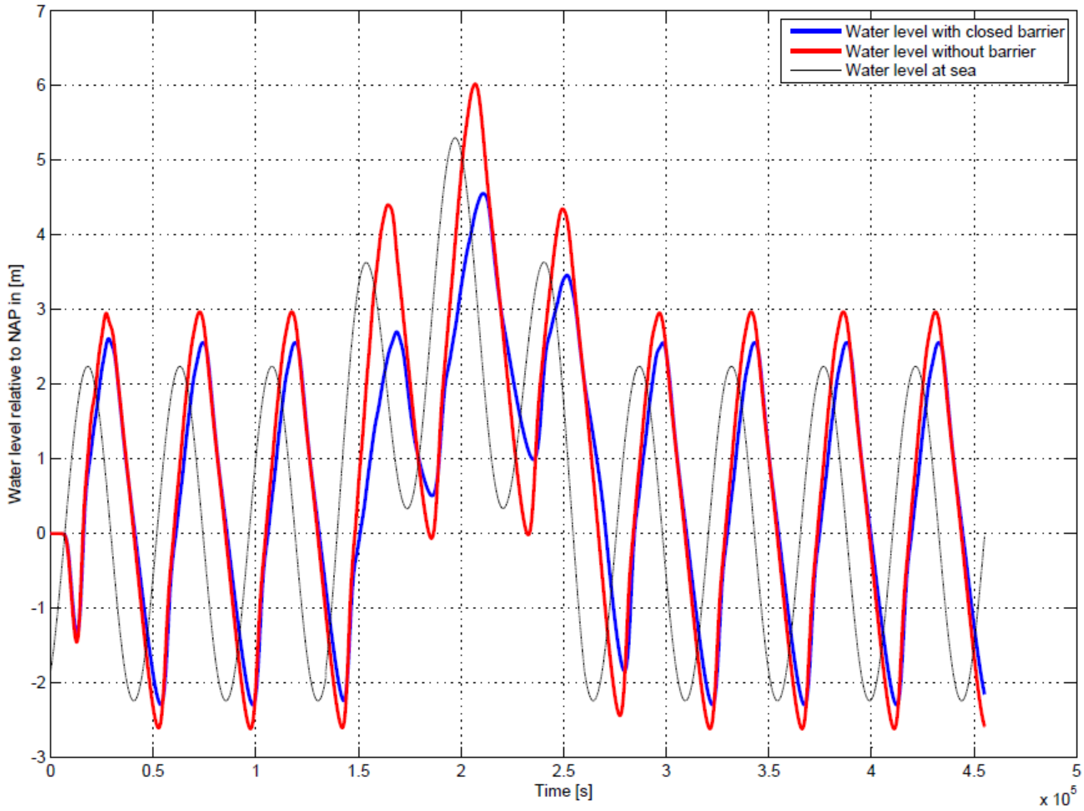
The motion equation for the section at the location of the barrier has an additional factor, which takes into account the extra water level difference over the barrier.

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial s}\left(\frac{Q^2}{A_s}\right) + g \cdot A_s \cdot \frac{\partial h}{\partial s} + c_f \cdot \frac{|Q| \cdot Q}{A_s \cdot R} + \frac{|Q| \cdot Q}{2 \cdot A_s \cdot \mu^2} \cdot \left(1 - \frac{1}{\mu}\right)^2 = 0$$

The result of the calculation is as follows. The maximum restriction of the opening with regard to shipping and the environmental aspects are sufficient to reduce the water level with 0.5 metre in the Western Scheldt near Antwerp with a mu of 0.375. The reduction of 1.5 metre can be achieved by reducing the opening from a factor 0.375 to 0.275. The closure must take place at the start of the storm surge in order to be effective. The gates can be opened when the water level in the Western Scheldt is higher than the water level at sea. The high flow velocity during storm conditions is observed during the beginning and the end of the storm surge. So with regard to the flow velocity, it is possible for vessels to enter of leave the Western Scheldt during the peak of the storm. However it is not advised to do so with regard to wind conditions.

The figure below the water level in the estuary is presented for a closed barrier. The reduction factor is 0.375 during the regular tide. The reduction factor is lowered to 0.275 as soon as the storm surge starts. The reduction factor is increased again as soon as the storm surge is over. The figure shows that the smaller reduction factor is able to reduce the water level near Antwerp with 1.5 metre.

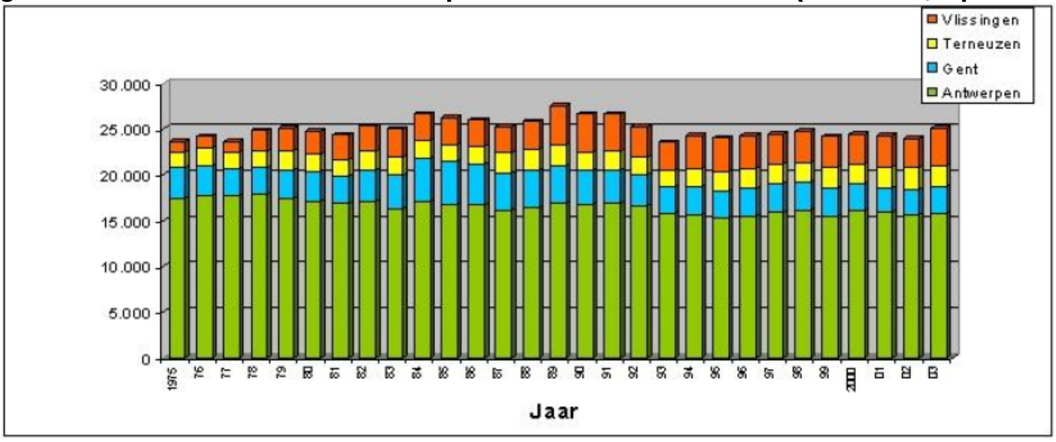
Figure III. 7 Effect of the closing of the gates on the water level in storm conditions



Shipping intensity

The number of vessels that pass the Western Scheldt on their way to the port of call is presented in Figure III. 8. The Port of Antwerp is the main contributor. These numbers are for the sea going cargo vessels, which means that there are more vessels that use the Western Scheldt, like pleasure craft. It is preferred to have an additional opening in the barrier for small vessels. A second opening can also provide redundancy with regard to shipping. Since one vessel has to pass the barrier twice when calling the Port of Antwerp, the number of traffic movements is twice as large. So the barrier has to be passed by roughly 50 000 vessels per year, which is 137 times per day. This is, on average, every 10 minutes one sea going vessel. It is clear that the number of vessels remains constant over a long period. The increased cargo flows are mainly due to the increased carrying capacity of the vessels. The ports of Vlissingen and Terneuzen are mainly importing goods. Therefore it is assumed that there are very few vessels that call at two ports in the Western Scheldt. Another reason for this assumption is the type of cargo which is predominantly handled by the ports. Antwerp handles mainly containers, while Vlissingen and Terneuzen are specialized in bulk.

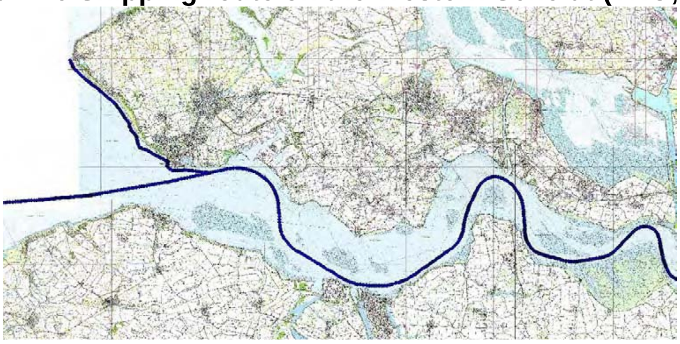
Figure III. 8 Number of vessels that pass the Western Scheldt (Voorsmit, April 2006)



Shipping channel

The vessels can follow two routes towards the North Sea, as can be seen in Figure III. 9. The vessels have to follow the channels once they enter the Western Scheldt.

Figure III. 9 Shipping route on the Western Scheldt (TNO, 2007)



The channel requires a certain width based on the vessel dimensions. The main dimensions are based on the decision whether the channel should be used for one way or two way traffic. The width is based on the average vessel size. Since the differences in size are significant in the Western Scheldt, it is better to design the channel as one way traffic for the largest vessels and two-way traffic for the average vessel. In this way the width of the channel can remain acceptable. The channel should be capable of providing access to the port over a period of 100 years, so there should be some room for future development. Especially since the vessel size is expected to grow primarily in width. Currently the average vessel has a gross tonnage of about 20 000 tons (Port of Antwerp, 2012). The average tonnage is expected to grow in the future, therefore the average vessel is estimated to be a second generation container vessel, as can be seen in Table III.2. The maximum vessel is the Maersk E-class. This type of vessel has already visited the Port of Antwerp. In order to maintain the accessibility of the Port of Antwerp the shipping channel is designed two lanes for the average vessel and single lane for the maximum size vessel. It is not advisable to a have a lot of waiting vessel near a barrier, since the probability of accident will increase. Therefore the capacity of the barrier with regard to shipping must not become the bottle neck.

Table III. 2 Average and maximum vessel size

	Dimension	Average vessel	Maximum vessel
Length	m	240	397
Width	m	30	56
Draft	m	11.5	16
DWT	ton	30 000	157 000

For the two lane situation the minimum width of the shipping channel is 378 metre and for the one lane situation for the maximum vessel is 308 metre (Ligteringen, 2009). This width can be reduced by introducing funnel structures which eliminate the cross currents and waves for example see Figure 4.10. The required distance in which the vessels can adjust to the new conditions is about 2.5 times the vessel length. In case the width is larger than 378 no length for adjustments is required. In case the width is smaller than 307 metre, 2.5 * 397 = 993 metre is required for heading adjustments. In case the width is between 307 and 378 the required adjustment length is; 2.5 * 240 = 600 metre.

The channels should be able to provide access to the port during the entire tidal cycle, so also during spring low tide. The channel depth depends on the draft of the vessel, lowest tide, sinkage due to squat and trim, vertical motions due to waves and safety margin. When all these factors are taken into account, the minimal depth for the largest vessel is 22 metre. The calculation is presented below. Currently a vessel with a daft of 13.10 metre can enter the Western Scheldt independent of the tide. For the barrier the draft of 16 metre is selected to make sure that the barrier does not become the bottle neck when the decision is made to further increase the depth in the Western Scheldt.

The depth of the shipping channel depends on several parameters, as presented in the following formula.

$$d = D - T + s_{\max} + r + m$$

In which:

$d =$ Depth shipping channel

$D =$ Draft vessel

$T =$ Low tide beneath average water level

$r =$ Vertical motion caused by waves

$m =$ safety margin

$s_{\max} =$ Maximum sinkage due to squat and trim

$$s_{\max} = \frac{C_B}{30} \cdot S_2^{\frac{2}{3}} \cdot v_s^{2.08}$$

$C_B =$ Block coefficient

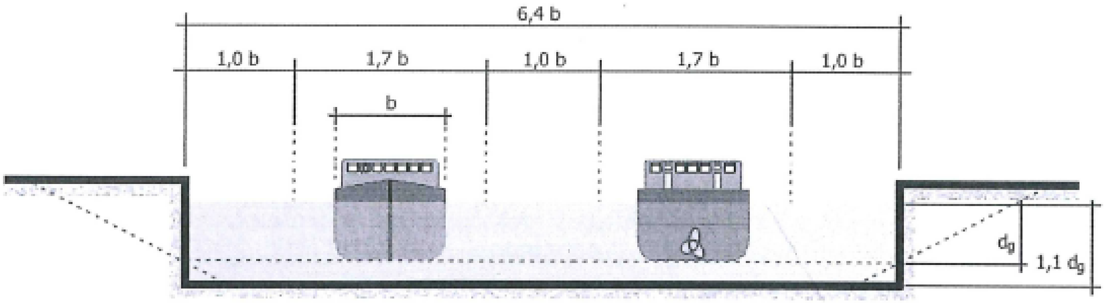
$v_s =$ Vessel speed

$$S_2 = \frac{S}{1-S}$$

$$S = \text{Blockage factor} = \frac{A_s}{A_{ch}}$$

The dimensions of the shipping channel result in an opening of (22+5) * 378 = 10 206 m^2. This means that the reduction of the water level of 1.5 metre still enables vessels to pass the barrier. A more narrow opening requires the aid of pilots and tugs. Another design rule is presented in Figure III. 10. The minimum width of the channels according to that rule is respectively 192 metre and 207 metre. This can only be achieved with a funnel structure that provides shelter.

Figure III. 10 Minimum dimensions of the shipping channel (Bezuyen, 2007)



Slope in the water level

The vessels have to overcome a water level difference at the location of the barrier, due to the restricted cross-section at the location of the barrier. The water level difference should be as gradual as possible, since vessels are better able to pass a gradual increase in water level and flow velocity. This can be achieved by increasing the width of the barrier. During normal conditions the maximum flow velocity is 2.4 m/s, which leads to a water level difference of 0.9 metre. When assuming a maximum slope of 1‰, the width of the barrier has to be about 900 metre. For storm conditions the slope can be larger, since there are no vessels navigating through the barrier. The maximum slope of the water level would become 3‰ in case of a reduction of 2.6 metre.

The slope of the water level will require additional power of the vessels. The amount of additional power can be used to check whether the slope is realistic. Assuming a vessel travels at 10 knots relative to the water, which is about 5 m/s. The maximum head current is about 3 m/s. The vessel travels thus at 2 m/s relative to the land. The required power can be calculated by multiplying the force with the velocity. The required force is, at a slope of 1‰:

$$W = F \cdot v$$

$$F = \frac{1}{1000} \cdot m \cdot g$$

The mass is assumed to be 50 000 ton. The required power is:

$$W = \frac{1}{1000} \cdot 50000000 \cdot 9.81 \cdot 2 = 981\ \mathrm{kW}$$

One horse power is about 0.75 kW, so 1 308 hp is required. Assuming an efficiency of about 50% means that 2 600 hp of the engine is needed to overcome the water level difference. These vessels have about 30 000 hp, so less than 10% of the engine power is required to overcome the water level difference. These vessels are not travelling on full power at 10 knots, so it is expected that the water level difference can be overcome.

Maximum vessel speed

The maximum physical speed is for the largest vessel through the opening in the barrier is 10 m/s relative to the water, according to the diagram of Schijf. The relation between the cross-section of the vessel and the discharge cross-section is 0.095. The water level depression corresponding with the limit speed is 0.17 times the water depth. The return flow is about 3.3 m/s.

So when the water enters the estuary with more than 10 m/s vessels are not able to navigate through the barrier. When vessels get close limit speed of 10 m/s, the water level depression around the vessel gets very large. This results in a large load on the funnel structure and the bed protection. The costs of the stronger structure must be smaller than the costs of a closed water way. This means that the flow velocity must not come frequently near the limit speed, since the each day of delay costs about 0.7 million euro (Muntinga, 2010).

Soil conditions

There is no information available about the soil in the middle of the Western Scheldt. Therefore the soil parameters are assumed, based on the soil condition on either side of the estuary. The cone penetration tests (CPT) can be found below.

Fortunately borings are made in the middle of the Western Scheldt. These borings do not provide strength parameters, but they can be used to support the assumption of the presence of sand layers in the soil. From the borings can be concluded that the soil is mainly sand varying grain sizes and small amounts of clay and silt. At some locations there are thin clay layers or layers with shells between the sand.

Figure III. 11 Soil conditions near Breskens, CPT (TNO, 2012)

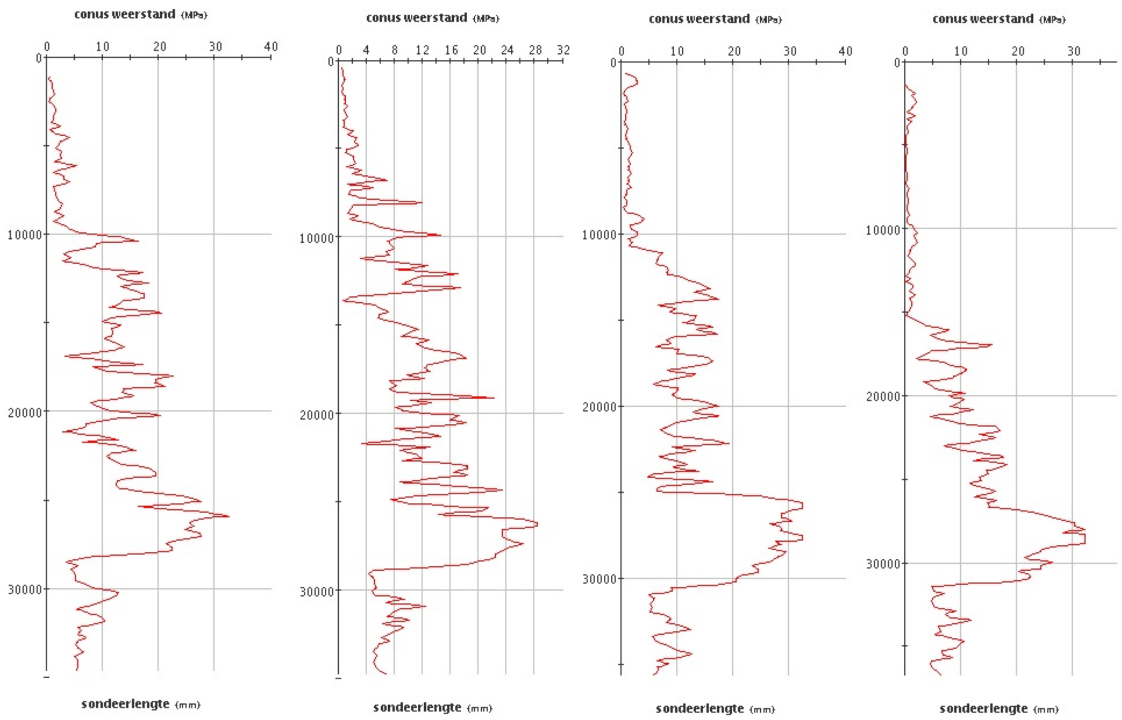
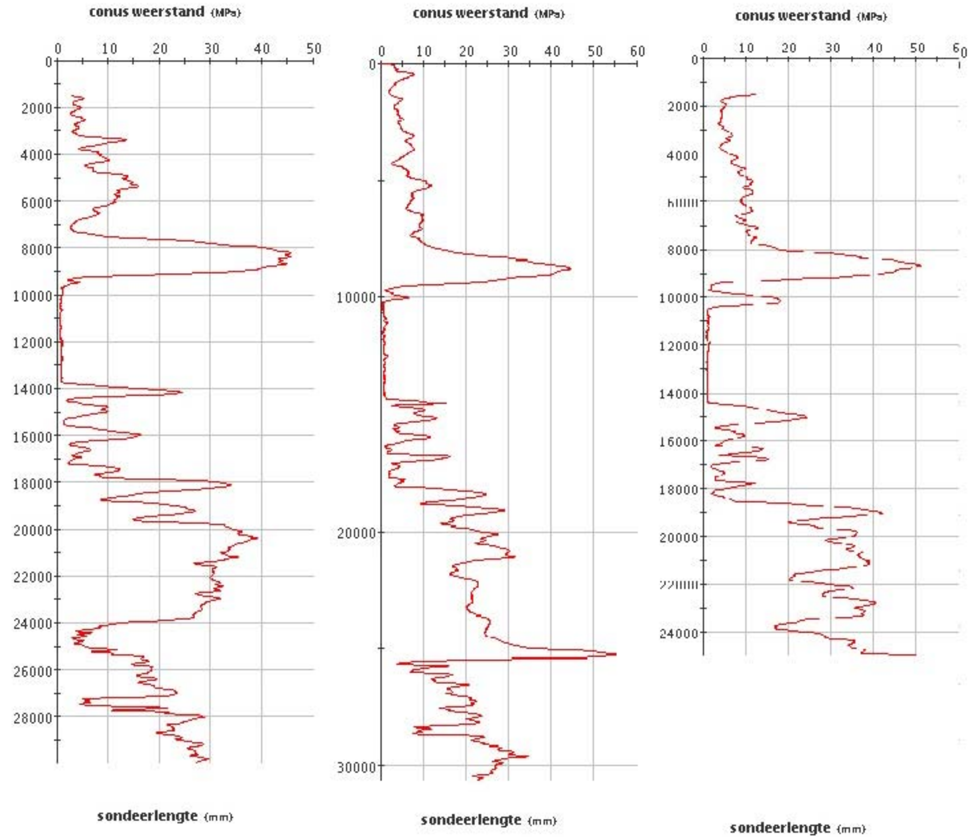


Figure III. 12 Soil conditions Vlissingen, CPT (TNO, 2012)



Wave overtopping

The waves can overtop the barrier which leads to additional discharge into the estuary. A large amount of overtopping would reduce the opening in the barrier during storm conditions. Since the length of the barrier is considerable, the overtopping per running metre has to be limited. The discharge over the barrier may only lead to a water level rise of 0.1 metre. During a storm of 35 hours the water level is not constantly at 5.30 metre above NAP. It is assumed that for 10 hours the wave overtopping should be calculated. The allowable discharge during the 10 hours is 1200 m^3/s. The length of the barrier excluding the opening is about 4400 metre. Therefore overtopping discharge has a maximum of 0.27 m^3/s. The overtopping height of the barrier should be 2.7 metre based on a significant wave height of 3.5 metre. For settlement is added 0.5 metre, which should be checked when the structure has be designed. The top of the barrier should have a height of 5.30+2.7+1+0.5=9.5 metre + NAP.

The wave overtopping height is the height which is required to make sure the overtopping discharge does not exceed a certain specified maximum. The applied formula is presented below (TAW, 2003).

$$h_{kr} = -\frac{1}{3} \cdot \gamma_{\beta} \cdot \gamma_{n} \cdot H_s \cdot \ln\left(\frac{q}{0.13 \cdot \sqrt{g \cdot H_s^{\ 3}}}\right)$$

In which:

h_{kr} = required wave overtopping height

γ_{β} = factor for angle wave attack

γ_{n} = factor for nose on structure

H_s = significant wave height

q = allowable overtopping discharge

In case the barrier has a slope, the run up height is calculated to find the required crest level. $R_{2\%}$ is 5.6 metre according to the formula below based on a reduction factor of 0.6 for a surface level of quarry stone.

$$R_{2\%} = 8 \cdot H_s \cdot \tan\left(\alpha\right) \cdot \gamma_r$$

APPENDIX IV MULTI CRITERIA ANALYSIS FOR THE GATES

This appendix contains the information about selection of the type of gate. The gate type with the highest value is selected and designed in further detail.

List of tables

MCA

The possible solutions for the gates in the barrier are compared using the requirements which are described in the Terms of References and some more general criteria. The criteria are given a relative value. The relative value is determined by comparing the importance of all the criteria against each other, see Table IV. 1. The more important criterion gets the ‘1’ the other one automatically a ‘0’. The relative importance per criterion can be determined by adding up all points in the row. Each solution is scored per criterion. The score of each solution times the relative value of the criterion is the number of points this solution gets for the considered criterion. The solution with the largest number of points is the most valuable option. Since the costs of the different solutions is assumed to be the same; € 30 000 /m^3, the value is decisive for the decision making. In case the costs of different solutions are different, the best solution is calculated by dividing the value by the costs.

The impact of the relative importance is limited. The result of the analysis does not change when a few numbers are changed in Table IV. 1.

Table IV. 1 Determination of the relative importance per criterion

	Maintainability	Opening and closing time	Redundancy	Required space	Constructability	Inspection possibilities	Field experience	Inconspicuousness	Total
Maintainability	X	1	0	1	1	0	0	1	4
Opening and closing time	0	X	0	1	0	0	0	1	2
Redundancy	1	1	X	1	1	1	1	1	7
Required space	0	0	0	X	0	0	0	1	1
Constructability	0	1	0	1	X	0	1	1	4
Inspection possibilities	1	1	0	1	1	X	1	1	6
Field experience	1	1	0	1	0	0	X	0	3
Inconspicuousness	0	0	0	0	0	0	1	X	1

The score of the solutions per criterion is done by engineering judgement and by analysing the already constructed barriers. The scores are arbitrary and therefore open for debate. However some adjustments in the scores do not lead to completely different results. It is possible that the vertical lift gate switches places with the radial gate, but the scores are always very close.

Table IV. 2 Scores of solutions per criterion

	Rising sector gate	Inflatable barrier	Hydraulic flap gate	Floating flap gate	Sliding gate	Vertcal lift gate	Visor gate	Radial gate	Mitre gates
Maintainability	4	2	1	1	2	5	5	5	2
Opening and closing time	4	2	5	5	3	5	5	5	4
Redundancy	3	1	2	2	3	4	3	4	4
Required space	5	4	4	3	1	4	3	5	5
Constructability	2	1	1	1	3	5	3	4	3
Inspection possibilities	4	2	1	1	2	5	4	5	2
Field experience	4	3	2	2	4	5	4	4	5
Inconspicuousness	4	5	5	5	4	2	1	3	4
	30	20	21	20	22	35	28	35	29

Table IV. 3 Relative scores of solutions per criterion

	Rising sector gate	Inflatable barrier	Hydraulic flap gate	Floating flap gate	Sliding gate	Vertcal lift gate	Visor gate	Radial gate	Mitre gates
Maintainability	16	8	4	4	8	20	20	20	8
Opening and closing time	8	4	10	10	6	10	10	10	8
Redundancy	21	7	14	14	21	28	21	28	28
Required space	5	4	4	3	1	4	3	5	5
Constructability	8	4	4	4	12	20	12	16	12
Inspection possibilities	24	12	6	6	12	30	24	30	12
Field experience	12	9	6	6	12	15	12	12	15
Inconspicuousness	4	5	5	5	4	2	1	3	4
	98	53	53	52	76	129	103	124	92

APPENDIX V DESIGN DETAILS

This appendix contains more detailed explanation about the calculations of the different components of the barrier. The design of the caissons is discussed first, followed by the bed protection, the design of the rubble mound part of the barrier and the moveable gates.

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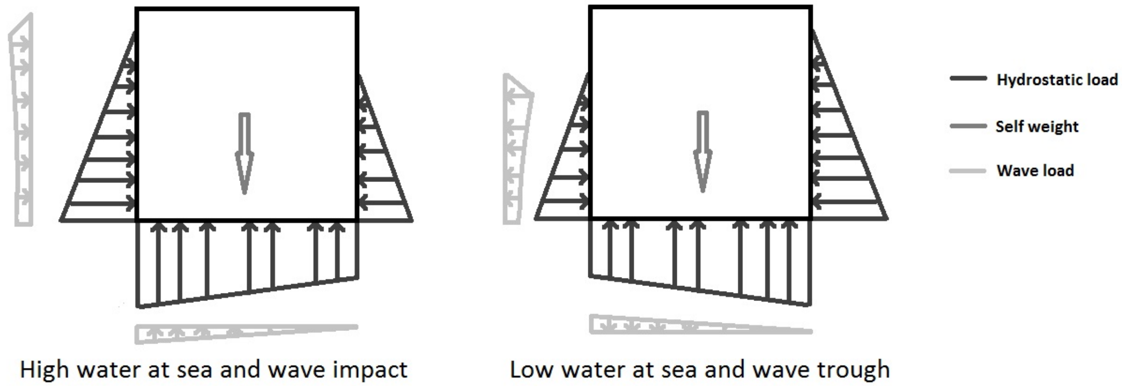
Calculation caissons

The caissons are checked on several aspects. The first step is to develop load cases. The two load cases are high water at sea with positive wave impact and low water at sea and negative load impact, see Figure V. 1. These two load cases are used to check the overall stability of the caisson according to several criteria.

The design wave height is not the significant wave height, but two times the significant wave height. This wave is reflected off the caisson, which doubles the wave height. In the calculation the maximum fully reflected wave has been taken into account. This depends on the water level. In one situation the wave height is 7 (which is the maximum for a significant wave height of 3.5 metre) and in another situation the wave height is 6.1 metre. The load on the caisson by these wave heights has been calculated using linear wave theory.

Figure V. 1 Load cases



The first criterion to check is the horizontal stability. The vertical weight of the structure must be sufficient to resist the horizontal load by friction with the subsoil. The result is a minimum weight of the structure and thus the width since the height of the structure is determined by the overtopping criterion. The friction between the subsoil and the caisson can be calculated by taking the tangents of 2/3rd of the internal friction angle. The width of the caissons can be important with regard to piping, however the caissons are placed on a geometrically closed filter and bed protection, so sand transporting wells cannot occur.

The next step is the over turning moment. The foundation of the caisson must remain in compression during all load cases. Once the foundation is completely in compression the maximum soil pressure must be calculated. This must be lower than the maximum soil bearing capacity which can be calculated using Brinch Hansens' formula. The caissons are placed on a shallow foundation.

The next subject is the floating stability of the caisson during transportation. The caisson is considered stable once the meta centric height is at least 0.5 metre. Also the draft of the caisson can be calculated for the required depth of the transport channel.

Once the outer dimensions of the caisson are known, the reinforcement in the concrete can be calculated. During the design of the caisson several loops have to be made in order to end up with a properly optimized design.

The two load combinations for the reinforcement calculations of the outer walls are presented in the figure below.

Figure V. 2 Load case during transportation

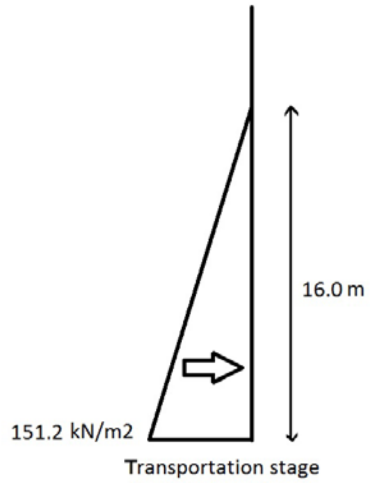
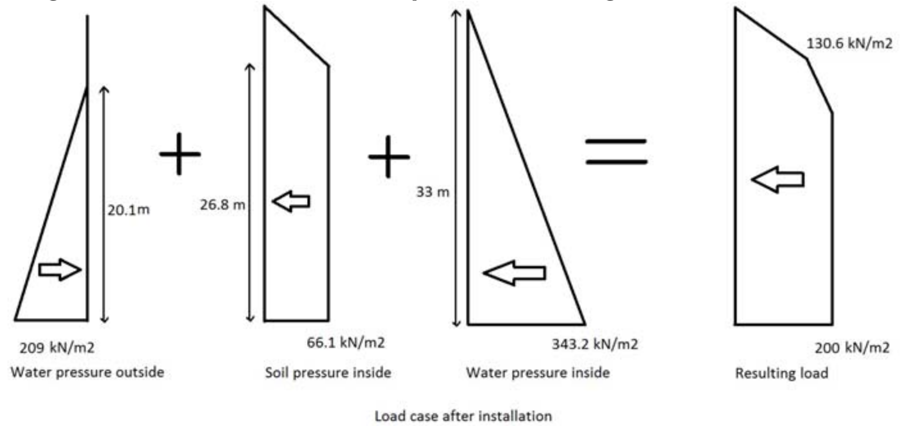
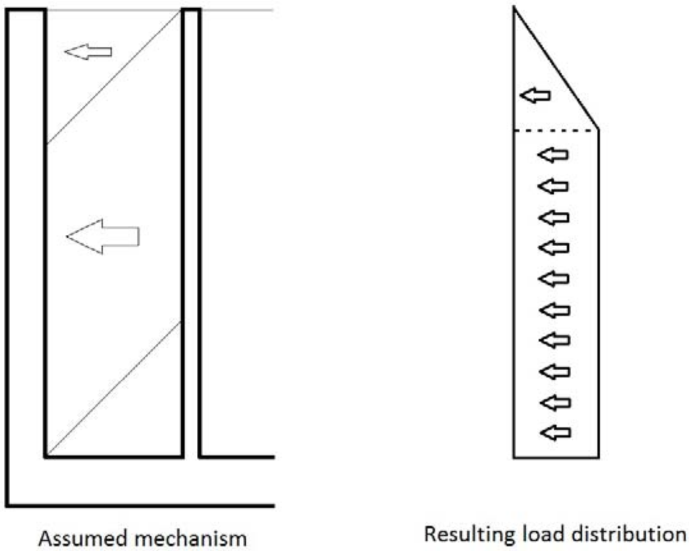


Figure V. 3 Load case for the permanent stage for a funnel caisson



The soil pressure inside the caisson is calculated as follows. It is assumed that the horizontal soil pressure of the ballast material in one cell has to be resisted by the outer wall as indicated in Figure V. 4. The horizontal soil pressure is calculated with coefficient of 1 since the repetitive movement of the water level causes additional horizontal pressures. The horizontal load on the wall is reduced by the limited cell size. It is assumed that the horizontal load does not increase further at a depth that is equal to the width of the cell, see Figure V. 4. The other load is the water level difference over the wall. In the calculation is assumed that the water level in the caisson is always at top level.

Figure V. 4 Assumed load distribution due to internal soil pressure



A load factor of 1.2 is applied before calculating the bending moment, assuming the maximum bending moment is given by (1/10) * q* l^2, due to redistribution of moments. Resulting in 716.5 kNm, since the wall has a length of 5.5 metre. The cells are not square which leads to a higher moment in the other direction, which is 991.2 kNm. The internal leverage of the wall is calculated by reducing the wall width with 150 mm and multiplying this length with 0.9. The result is an arm of 405 mm. The required reinforcement area is 5626 mm^2. The width of the wall is reduced with 150 mm in order to take into account the concrete cover, stirrups and possibility of two layers of reinforcement. The crack width requirement is not met with this reinforcement, therefore the reinforcement is increased to 12315 mm^2. The high corrosive environment demands very small crack widths. The crack width is calculated using the formula below.

$$w_{\max} = \frac{\phi}{4 \cdot \rho \cdot E_s} \cdot \left\{ \sigma_s - 0.5 \cdot f_{ctm} \cdot \left(1 + \alpha_e \cdot \rho\right) + \varepsilon_{cs} \cdot E_s \right\}$$

$\phi =$ reinforcement diameter

$\rho =$ reinforcement percentage

$E_s =$ E-modulus steel

$\sigma_s =$ steel stress

$f_{ctm} =$ average concrete tensile strength

$\alpha_e =$ E-modulus steel / E-modulus concrete

$\varepsilon_{cs} =$ steel strain

$w_{\max} =$ maximum crack width

The shear reinforcement is calculated according to the following formula:

$$V_{ED} = \frac{A_{sw}}{s} \cdot z \cdot \cot\left(\theta\right) \cdot f_{ywd}$$

$A_{sw} =$ shear reinforcement area

$s =$ reinforcement spacing

$z =$ concrete height

$\theta =$ shear angle

$f_{ywd} =$ yield stress

$V_{ED} =$ Shear strength

The inner wall must be able to resist the tension forces without cracking in order to prevent corrosion of the reinforcement. The tension force in the concrete wall has a maximum of 1400 kN/m. The wall is able to resist 1950 kN/m with a reinforcement percentage of 2% without cracking.

$$N_r = f_{ctm} \cdot A_c \cdot \left(1 + \alpha_e \cdot \rho\right)$$

$A_c =$ concrete area

$N_r =$ concrete crack force

The bending reinforcement during transportation stage has to fulfil the requirement with regard to the crack width as well. So the same amount of reinforcement has to be applied for the other load as well.

Stone diameter bed protection

The required stone diameter for the bed protection depends among others on the flow velocity. The used formula is presented below (Schiereck, 2004). The required stone diameter can be found by means of iteration.

$$d_{n50} = \frac{\overline{u_c}^2}{\psi_c \cdot \Delta \cdot \left(18 \cdot \log\left(\frac{12 \cdot R}{k_r}\right)\right)}$$

In which:

d_{n50} = Median nominal diameter

$\overline{u_c}$ = Average velocity

ψ_c = Shields parameter

Δ = Relative density of the bed protection material

R = Hydraulic radius

k_r = roughness of the bottom = $4 - 5 \cdot d_{n50}$

The local flow velocities can be higher, so the stone diameter can be higher at certain locations in the opening. The required stone diameter is not very large, due to the relatively small flow velocities and the large water depth of the openings. A larger flow velocity can be expected in case vessels pass the barrier during storm conditions. The stone diameter of 0.15 metre is required when the speed is 4.6 m/s. So with regard to the stone size the passage of vessels is possible. However the policy should be not to allow passage during storm conditions due to the combination difficult circumstances. The highest load on the bed protection is not cause by the tide or the storm surge, but by the propeller wash of the largest vessel. The calculation is presented below.

The propeller wash is calculated for the largest vessel. This vessel has a maximum draft of 16 metre and a propeller diameter is estimated to be 9 metre. The smallest water depth is the governing situation. The water depth at low spring water is 22.8 metre. The propeller axis is assumed to be 11.3 metre above bottom level. The installed power of the governing vessel is assumed to be 80 000 kW. The jet diameter is 70% of the propeller diameter which is 6.3 metre. The maximum jet velocity is:

$$u_0 = 1.15 \cdot \left(\frac{P}{\rho \cdot d^2} \right)^{1/3} = 1.15 \cdot \left(\frac{80 \cdot 10^6}{1000 \cdot 6.3^2} \right)^{1/3} = 14.5 \text{ m/s}$$

$P =$ Engine power

$\rho =$ density of the water

$d =$ propeller diameter

$u_0 =$ maximum jet velocity

The maximum velocity at the bottom is:

$$u_{b-\max} = 0.3 \cdot u_0 \cdot \frac{d}{z_b} = 0.3 \cdot 14.5 \cdot \frac{6.3}{11.3} = 2.4 \text{ m/s}$$

$z_b =$ distance propeller shaft and bottom

$u_{b-\max} =$ maximum bottom velocity

Assuming the vessel travels at 10 knots. The head current is about 2.4 m/s and the return current is 0.6 m/s. The vessel travels therefore at a velocity of 2 m/s. The actual velocity of the water near the bottom is therefore 0.4 m/s caused by the propeller wash. The total flow velocity is 2.4 + 0.6 + 0.4 = 3.4 m/s. The required stone size is:

$$d_{n,50} = 2.5 \frac{u_{b-\max}{}^2}{\Delta \cdot 2 \cdot g} = 2.5 \frac{3.4^2}{1.65 \cdot 2 \cdot 9.81} = 0.9 \text{ m}$$

Scour hole depth

The depth of the scour hole has to be calculated in order to find the required length of the bed protection. The scour depth can be calculated using the formula:

$$I = \left[0.005 \cdot \left(\cot\left(\beta_1\right) + \cot\left(\beta_2\right) \right) \cdot \Delta^{-1.4} \cdot h_0^{0.4} \cdot \left(\alpha \cdot \bar{u} - \bar{u}_c \right)^{3.4} \right] \cdot t^{0.8}$$

$$I = K \cdot t^{0.8}$$

$$S = 0.8 \cdot K \cdot t^{-0.2}$$

$$t_e = \left(\frac{0.8 \cdot K}{S} \right)^5$$

$$h_{se} = \sqrt{\frac{K \cdot t_e^{0.8} - S \cdot t_e}{\frac{1}{2} \cdot \left(\cot\left(\beta_1\right) + \cot\left(\beta_2\right) \right)}}$$

In which:

$\beta_1 =$ up stream slope of the scour hole

$\beta_2 =$ down stream slope of the scour hole

$h_0 =$ water depth

$\alpha =$ dust bin factor

$\bar{u} =$ average velocity

$\bar{u}_c =$ average critical velocity

$t =$ time

$S =$ sediment discharge

The sand is packed loose, which means that the length of the bed protection has to be 13 times the scour depth after 100 years. This is 645 metre. So at all locations with opening in the dam the bed protection has to have a length of 645 metre.

Design rubble mound

The rubble mound dam needs to be able to resist the wave impact and must be a barrier for the water flow into the estuary. The rubble mound dam is calculated by using the Van der Meer formulea. The height of the dam has to be 5.6 above high water which is 10.9 metre + NAP. An outer slope of 1:3 is assumed. The calculation is as follows:

$$\xi_m = \frac{\tan(\alpha)}{\sqrt{\dfrac{2\cdot\pi\cdot H_s}{g\cdot T_m^{\;2}}}} = \frac{1/3}{\sqrt{\dfrac{2\cdot\pi\cdot 3.5}{g\cdot 9^2}}} = 2$$

$$d_{n,50a} = \frac{H_s}{6.2\cdot P^{0.18}\cdot\left(\dfrac{S}{\sqrt{N}}\right)^{0.2}\cdot\xi_m^{\;-0.5}\cdot\Delta} = \frac{3.5}{6.2\cdot 0.4^{0.18}\cdot\left(\dfrac{2}{\sqrt{7500}}\right)^{0.2}\cdot 2^{-0.5}\cdot 1.65} = 1.2$$

$$W_{50a} = \gamma_r\cdot d_{n,50a}^{\;\;3} = 4500 \text{ kg}$$

$$W_{50a}/W_{50u} = 10 \quad \rightarrow W_{50u} = 450 \text{ kg} \quad \rightarrow d_{n,50u} = 0.55 \text{ m}$$

$$W_{50u}/W_{50core} = 20 \quad \rightarrow W_{50core} = 23 \text{ kg} \quad \rightarrow d_{n,50core} = 0.2 \text{ m}$$

$$W_{50a}/W_{50toe} = 5 \quad \rightarrow W_{50toe} = 900 \text{ kg} \quad \rightarrow d_{n,50toe} = 0.7 \text{ m}$$

$$t_a = 2\cdot d_{n,50a} = 2\cdot 1.2 = 2.4 \text{ m}$$

$$t_{toe} = 2\cdot d_{n,50toe} = 2\cdot 0.7 = 1.4 \text{ m}$$

$$t_u = 2.25\cdot d_{n,50u} = 2.25\cdot 0.55 = 1.2 \text{ m}$$

$$B_{toetop} = 5\cdot d_{n,50toe} = 3.5 \text{ m}$$

$$B_{toe} = B_{toetop} + 3\cdot d_{n,50u} = 5.2 \text{ m}$$

In which:

$\alpha =$ angle slope

$H_s =$ significant wave height

$T_m =$ wave period

$P =$ damage factor

$S =$ notional permeability

$N =$ number of waves

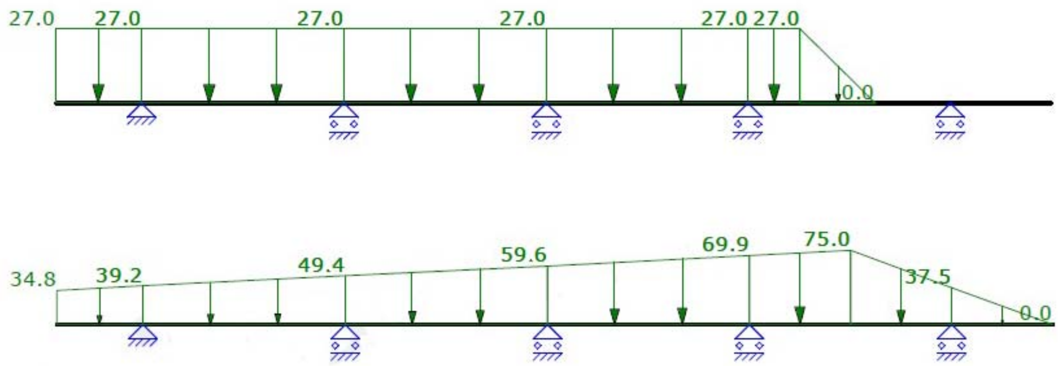
$\gamma_r =$ density rock

Gate design

The design of the 60 metre wide gate in one of the middle opening is discussed in this section. The calculations for the gate are all done in MatrixFrame 5.0.

The load on the gate is composed of a wave part and a water level difference part. The water level is assumed to 2.5 metre + NAP. The water level difference is 2.6 metre and the maximum wave height is 7 metre and this wave is fully reflected by the gate. In this situation the high water level is at sea. The wave load is modelled according to Sainflou. The loads are presented below. For now is assumed that this load is governing. The gates should also be checked for different combinations of water levels and waves.

Figure V. 5 Load case gate design



This is the load case for the screen that is supported by the 3D truss. For the calculation of the design load the water level difference is multiplied with a factor 1.2 and the wave load is multiplied with a factor 1.5. The screen is made of sheet piles; AZ14-700. These are relatively light sheet piles. These piles are selected for their stiffness a steel quality of S320 is sufficient for the moment capacity. The reaction forces of this beam model are the input values for the 3D truss model. After several design cycles is chosen for 5 supports instead of 4, in order to distribute the load better over the truss.

The loads on the truss are obtained from the reaction forces and the self weight of the truss and the sheet piles. The load factor for the self weight is 1.2. The gate is designed using circular hollow section of different diameter and wall thickness. The gate has simple supports on both sides.

The result of several design loops is presented below. The gate is designed using a steel quality of S460. The load of the water level difference and the waves can change its direction, so most elements can be in compression. For the buckling factor for most elements is about 0.6 which means that the maximum allowed stress is 276 N/mm^2. The elements are all below this value. The defection of the gate is rather limited.

The results show that it is possible to design a gate for the barrier with a span of 60 metre. Some of the elements are very green, which means that the stress level in the element is very low. It is not advised to select smaller profiles, because the elements must fit well together. The elements are also required to increase the redundancy of the gate.

Costs barrier without rubble mound
The additional costs for the rubble mound dam are estimated in this section. These costs should less than the environmental gain of the newly created habitat. Where possible the rubble mound dam has been replaced by the barrier caissons.

Table V. 1 Costs estimation without rubble mound

Item	unit	cost per unit	amount of units	costs per item
Reinforced concrete	m3	€ 270	1 385 557	€ 374 100 390
Ballast material	m3	€ 5	5 752 493	€ 28 762 465
Small stones	m3	€ 50	14 611 545	€ 730 577 250
Big stones	m3	€ 100	7 267 211	€ 726 721 100
Geotextile	m2	€ 5	7 400 000	€ 37 000 000
Steel doors	ton	€ 2 000	7 645	€ 15 290 000
Mechanical equipment	-	-	50%	€ 7 645 000
Construction quay	m3	€ 20 000	40 000	€ 800 000 000
Construction area	m2	€200	300 000	€ 60 000 000
			Subtotal	€ 2 780 096 205
Other			20%	€ 556 019 241
Overhead	-	-	10%	€ 278 009 621
			Total:	€ 3 614 125 067
			m^3:	187 200
			Price per cubic metre:	€ 19 306

The costs for the barrier with a much smaller rubble mound part is about € 400 million less, which is considerable. However on the total costs of the barrier is within the band width of the costs estimation. The larger amount of caissons will probably increase the construction time which will lead to higher additional costs.

Destruction of landscape by the vertical lift gates
The impact on the landscape of the vertical lift gates is analysed by drawing view lines around the gates. The area between the view lines behind the gates is not visible for the observer either on the beach near Breskens or on the boulevard of Vlissingen. The result is that the observers near Breskens are no longer able to the Port of Vlissingen and that the observer in Vlissingen is no longer able to see a fraction of the ‘Hoofdplaat’. The view to the sea and the opposite side of the estuary remains clear for both observers. The boulevard and beaches that are important for tourism do face the barrier, so it is unlikely that tourism will be affected.

The impact for observers at the land side of the barrier is not indicated in the drawing, since there are only a few villages located along the Western Scheldt. These villages are all positioned behind the levee, which makes it impossible to see the gates.

The impact is rather limited due to the position of the barrier. However the gates may still cause opposition from the local population.

Figure V. 6 Impact of the gates on visibility

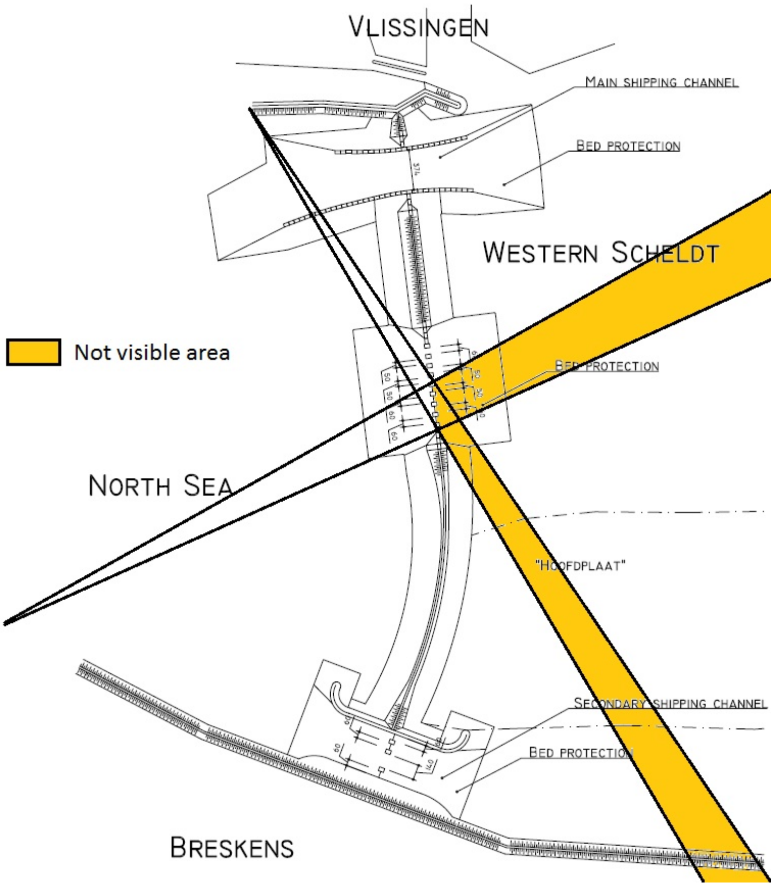


Figure V. 7 Impression of the impact on the landscape from Breskens

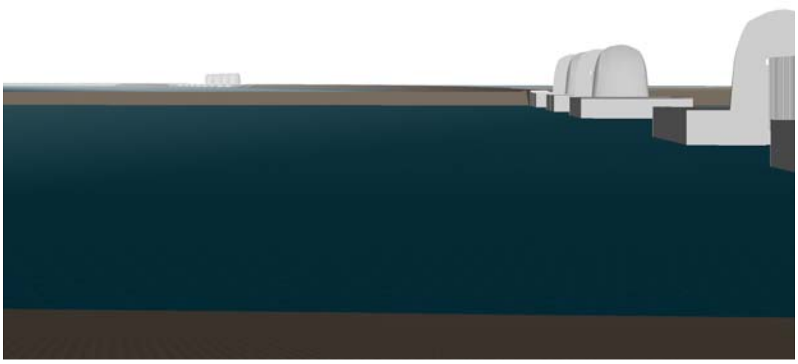
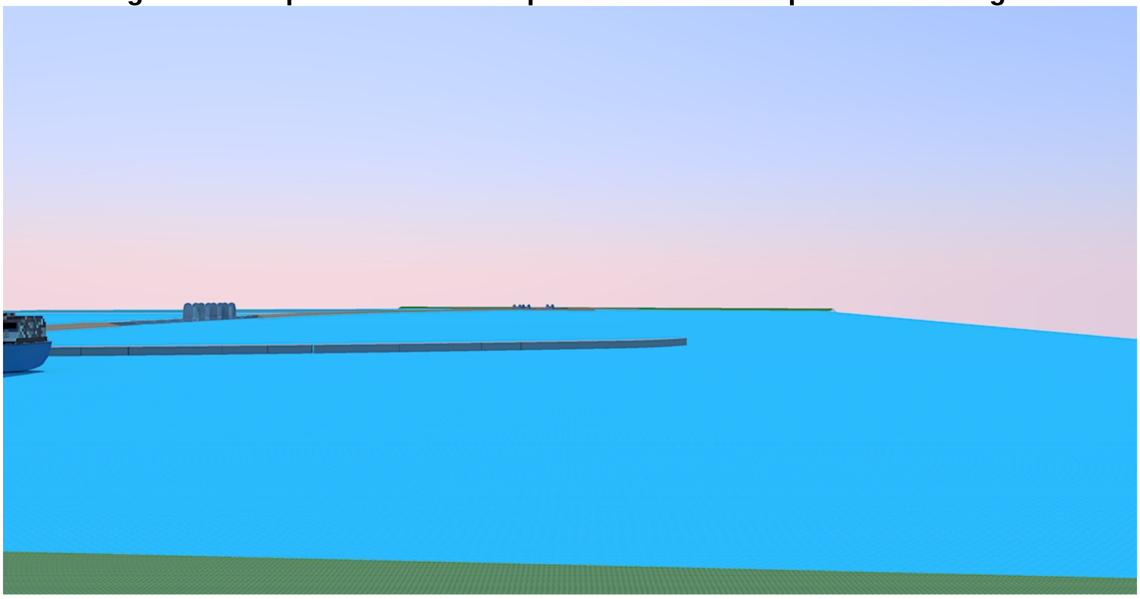


Figure V. 8 Impression of the impact on the landscape from Vlissingen

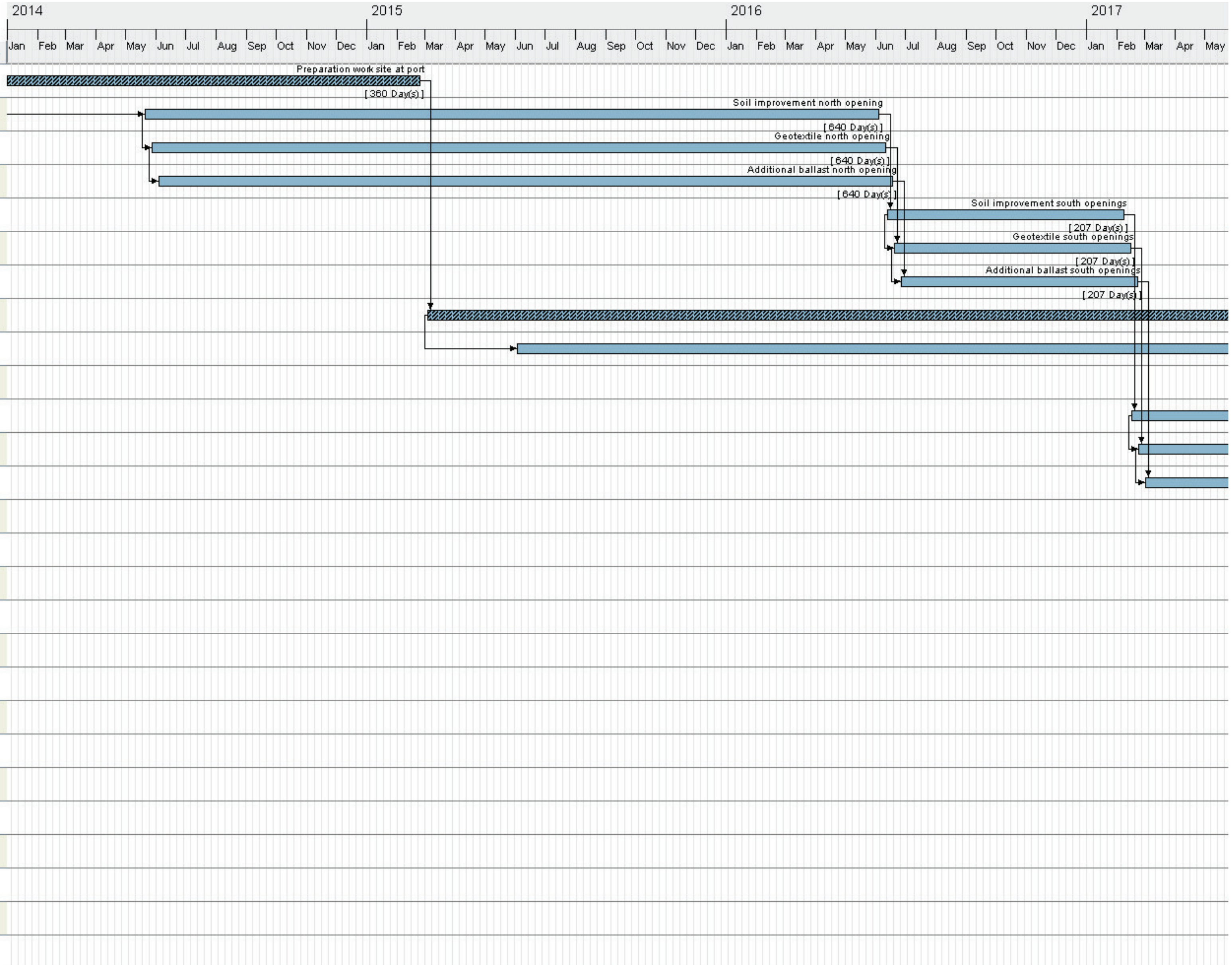


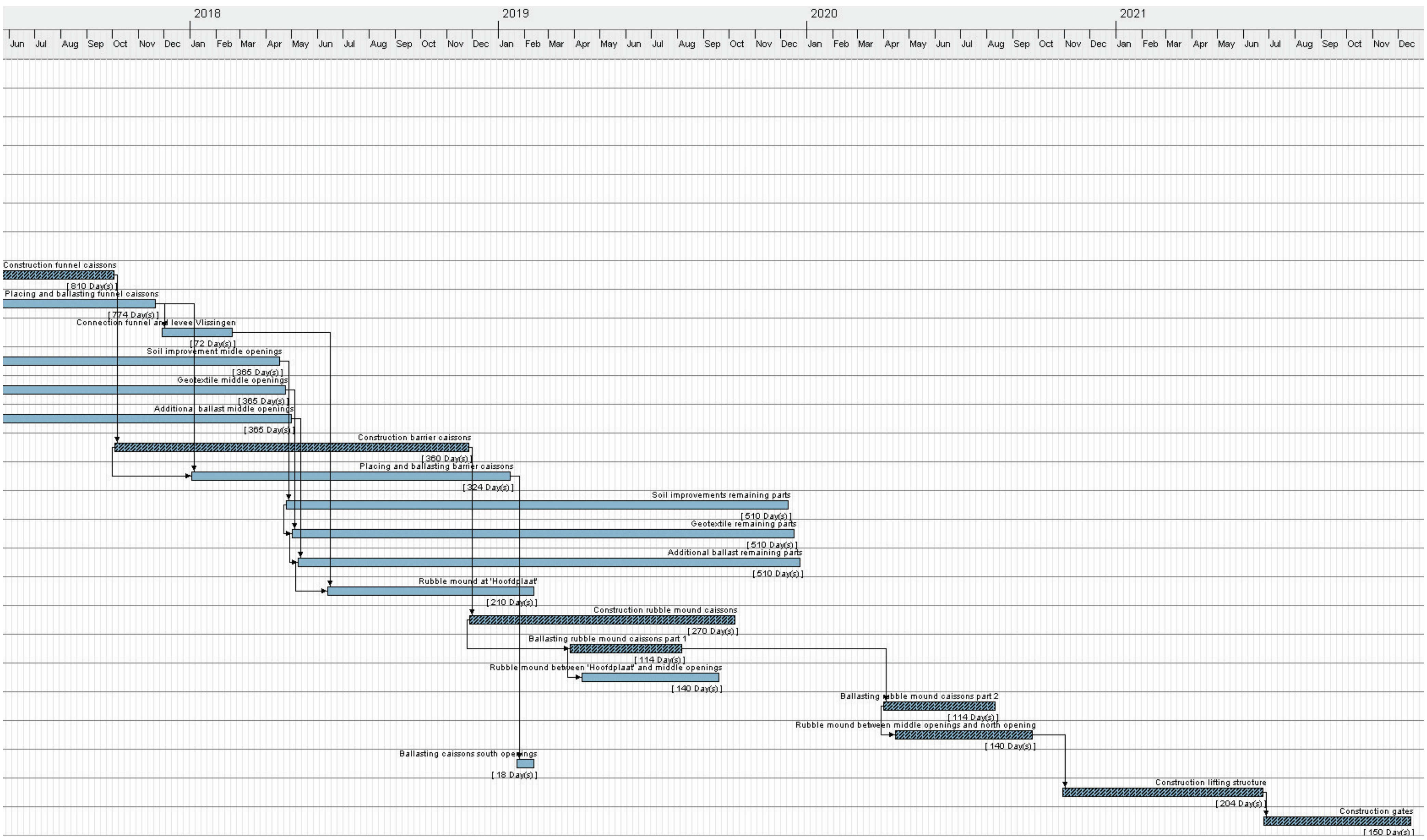
APPENDIX VI PLANNING

This appendix contains the construction planning for the reduction barrier in the Western Scheldt. The total construction time has been reduced by performing several activities similtaniously. The critical path of the planning is indicated by the dark colour.

Witteveen+Bos

Name	Begin date	End date
Preparation work site at port	1/1/14	2/24/15
Soil improvement north opening	5/21/14	6/4/16
Geotextile north opening	5/28/14	6/11/16
Additional ballast north opening	6/4/14	6/18/16
Soil improvement south openings	6/13/16	2/8/17
Geotextile south openings	6/20/16	2/15/17
Additional ballast south openings	6/27/16	2/22/17
Construction funnel caissons	3/4/15	10/3/17
Placing and ballasting funnel caissons	6/3/15	11/21/17
Connection funnel and levee Vlissingen	11/29/17	2/20/18
Soil improvement midle openings	2/16/17	4/17/18
Geotextile middle openings	2/23/17	4/24/18
Additional ballast middle openings	3/2/17	5/1/18
Construction barrier caissons	10/4/17	11/27/18
Placing and ballasting barrier caissons	1/3/18	1/15/19
Soil improvements remaining parts	4/25/18	12/10/19
Geotextile remaining parts	5/2/18	12/17/19
Additional ballast remaining parts	5/9/18	12/24/19
Rubble mound at 'Hoofdplaat'	6/13/18	2/12/19
Construction rubble mound caissons	11/28/18	10/8/19
Ballasting rubble mound caissons part 1	3/27/19	8/6/19
Rubble mound between 'Hoofdplaat' and middle openings	4/10/19	9/19/19
Ballasting rubble mound caissons part 2	4/1/20	8/11/20
Rubble mound between middle openings and north opening	4/15/20	9/24/20
Ballasting caissons south openings	1/23/19	2/12/19
Construction lifting structure	10/30/20	6/24/21
Construction gates	6/25/21	12/16/21





APPENDIX VII DRAWINGS

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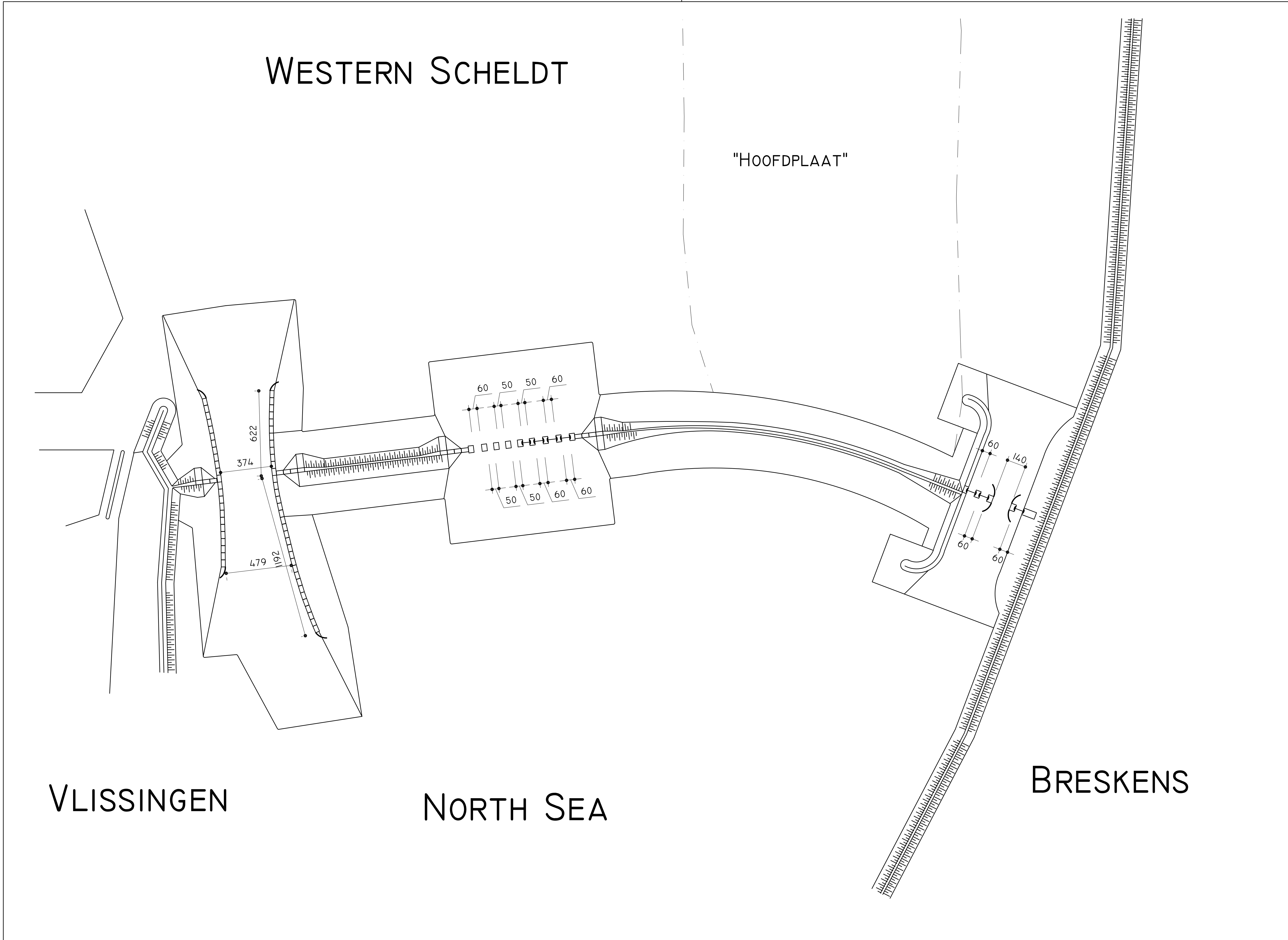
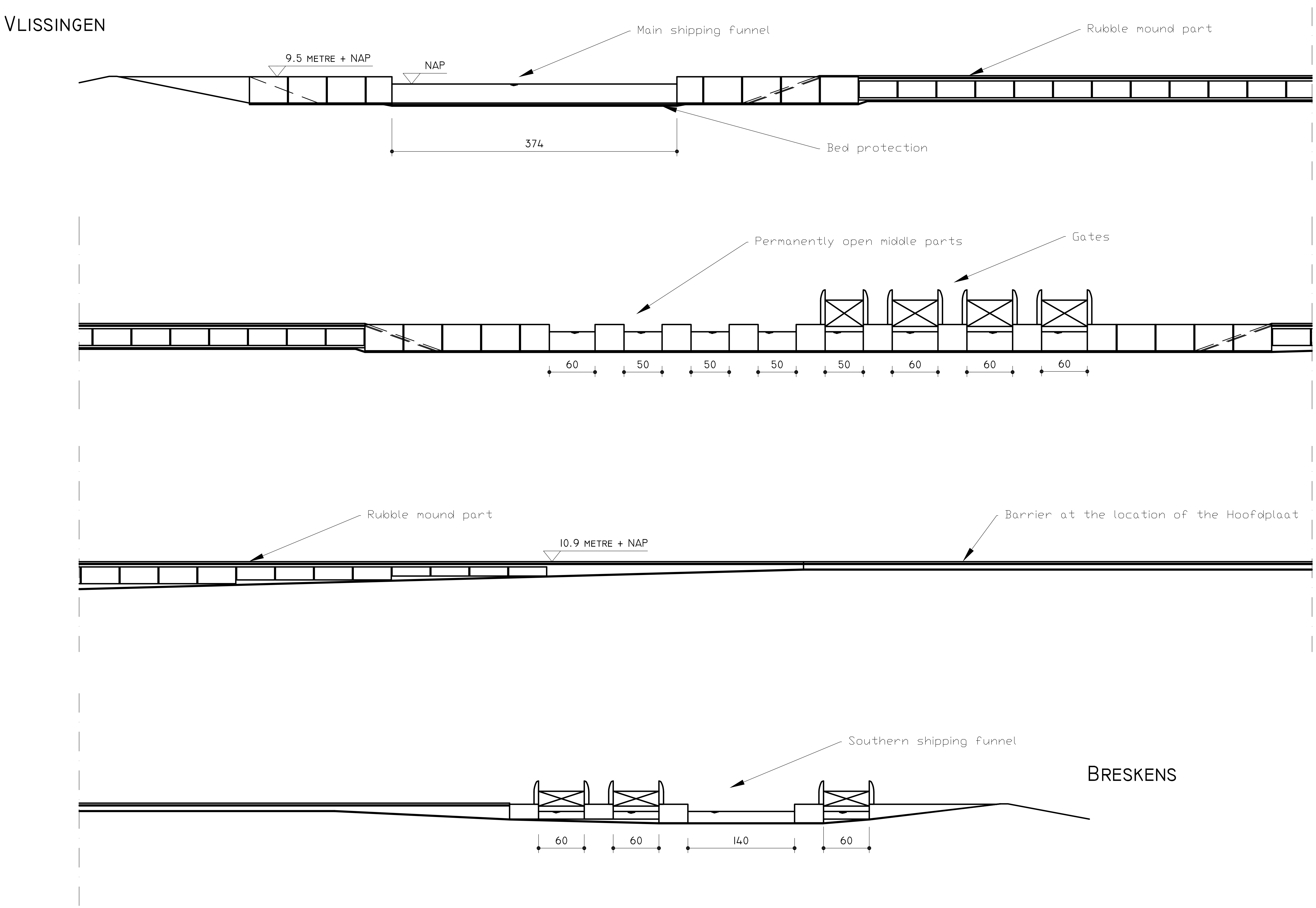
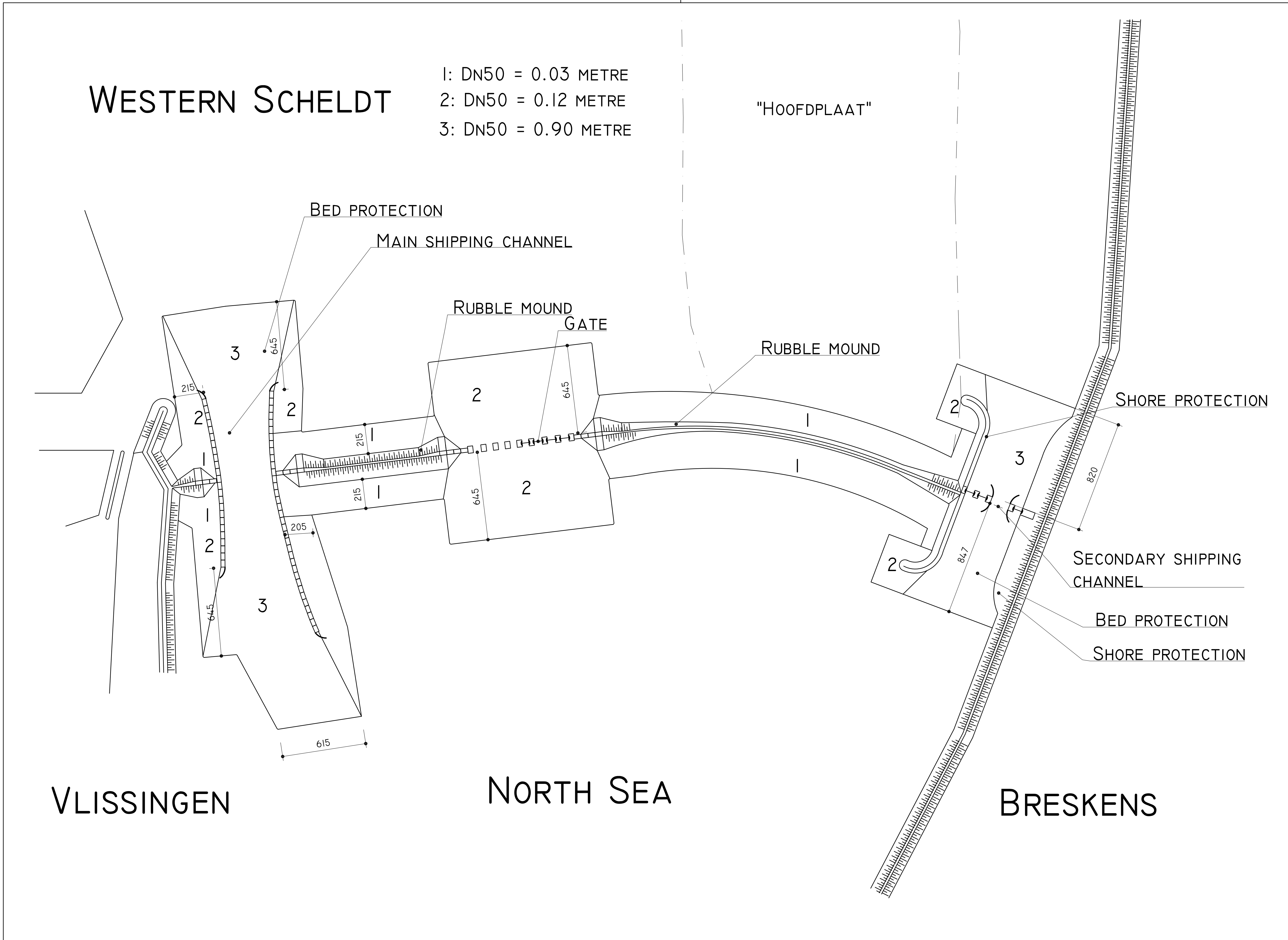


Tek.nr. 01	Naam Lex de Boom	Studienummer en projectgroep 1353764
Datum 30/01/13	Begeleider Vrijling	
Benaming	Topview reduction barrier in Western Scheldt	TU Delft, Faculteit Civiele Techniek en Geowetenschappen, Sectie GCC, Technisch Tekenen CT1112
Formaat A3	Schaal --	Eenheid m



LONGITUDINAL CROSS-SECTION OF THE REDUCTION BARRIER IN THE WESTERN SCHELDT

Tek.nr.	02
Naam	Lex de Boom
Studienummer en projectgroep	1353764
Datum	30/01/13
Begeleider	Vrijling
Benaming	Cross-section over the length of the barrier
	TU Delft, Faculteit Civiele Techniek en Geowetenschappen, Sectie GCC, Technisch Tekenen CT1112
Formaat	A3
Schaal	1:5000
Eenheid	m



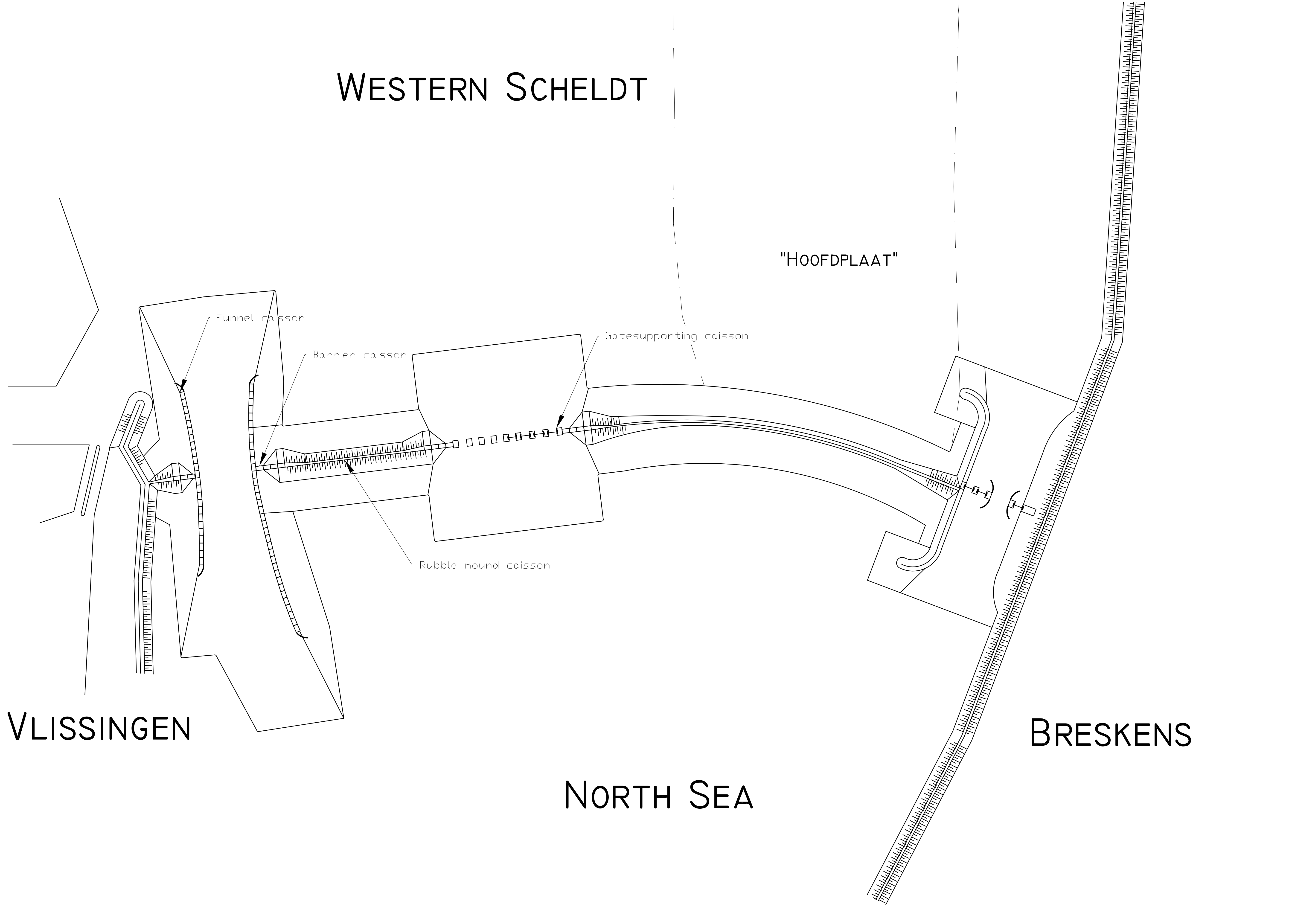
Formaat: A3
Schaal: --
Eenheid: m

Benaming: Topview reduction barrier, bottom protection

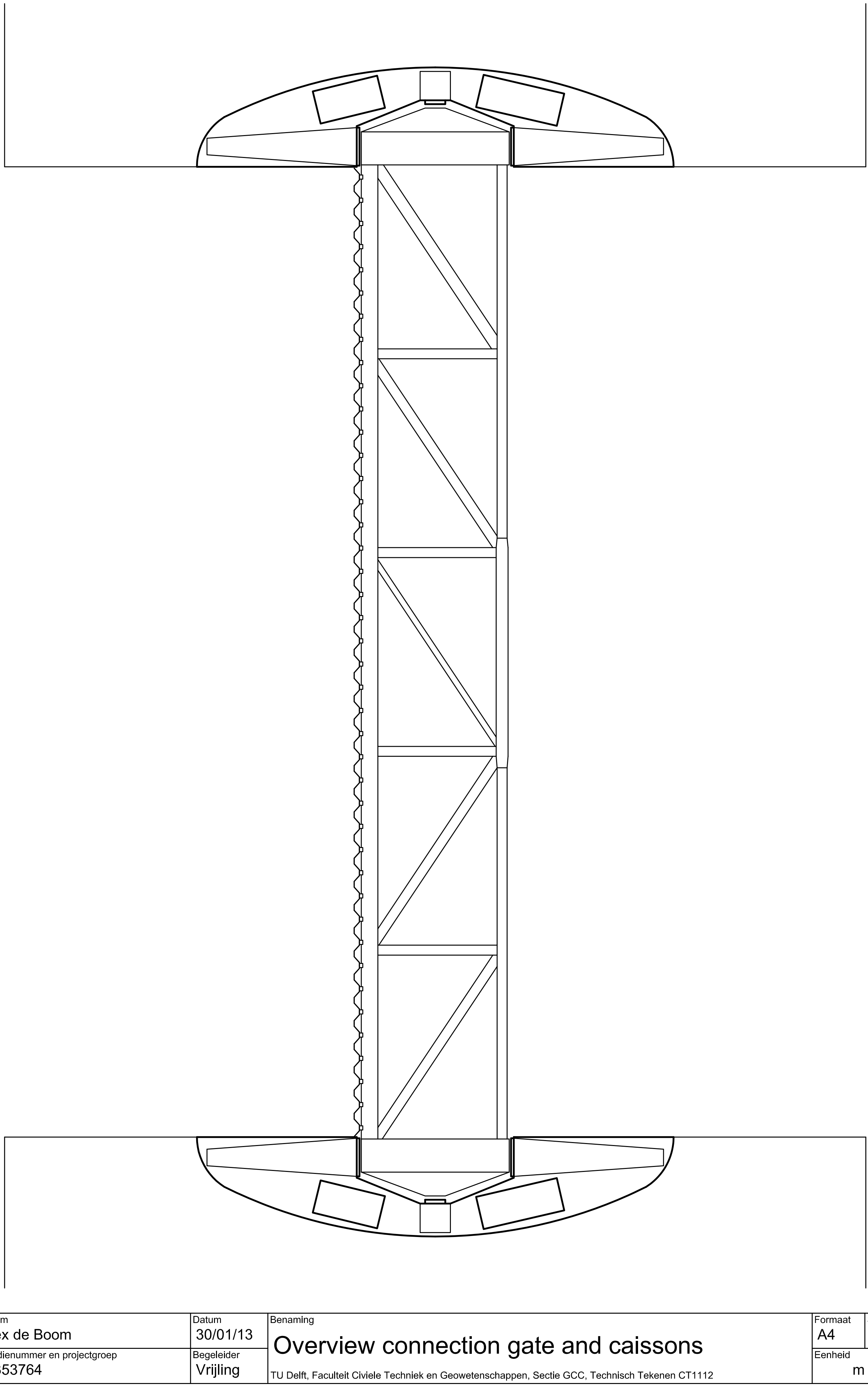
TU Delft, Faculteit Civiele Techniek en Geowetenschappen, Sectie GCC, Technisch Tekenen CT1112

Datum: 30/01/13
Begeleider: Vrijling

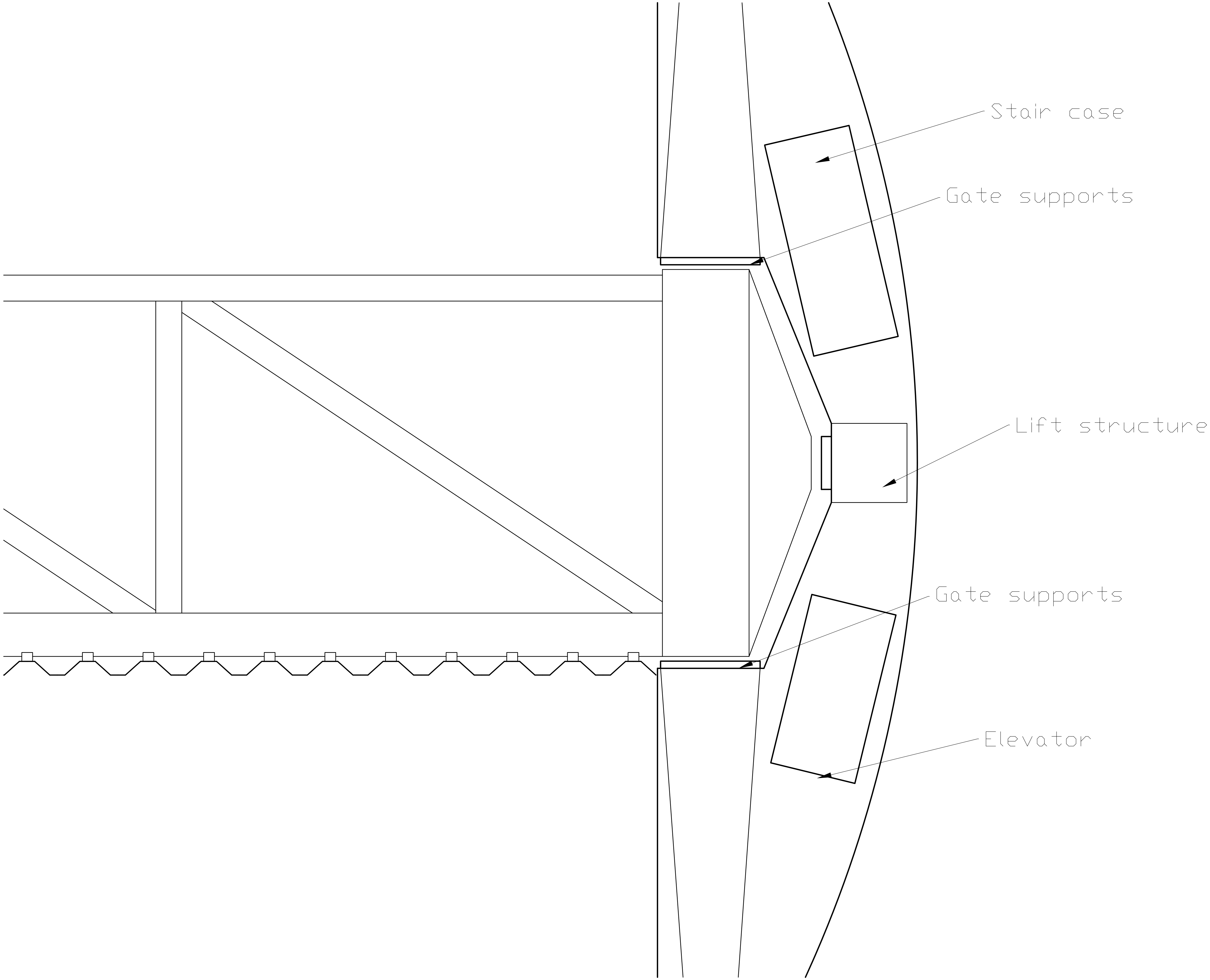
Tek.nr.: 03
Naam: Lex de Boom
Studienummer en projectgroep: 1353764



Tek.nr. 04	Naam: Lex de Boom	Datum: 30/01/13	Benaming: Topview caisson types
	Studienummer en projectgroep: 1353764	Begeleider: Vrijling	TU Delft, Faculteit Civiele Techniek en Geowetenschappen, Sectie GCC, Technisch Tekenen CT1112
Formaat: A3	Schaal: --	Eenheid: m	



Tek.nr.	Naam	Datum	Benaming	Formaat	Schaal
05	Lex de Boom	30/01/13	Overview connection gate and caissons	A4	--
	Studienummer en projectgroep	Begeleider		Eenheid	
	1353764	Vrijling	TU Delft, Faculteit Civiele Techniek en Geowetenschappen, Sectie GCC, Technisch Tekenen CT1112	m	



Tek.nr. 06

Naam: Lex de Boom

Studienummer en projectgroep: 1353764

Datum: 30/01/13

Begeleider: Vrijling

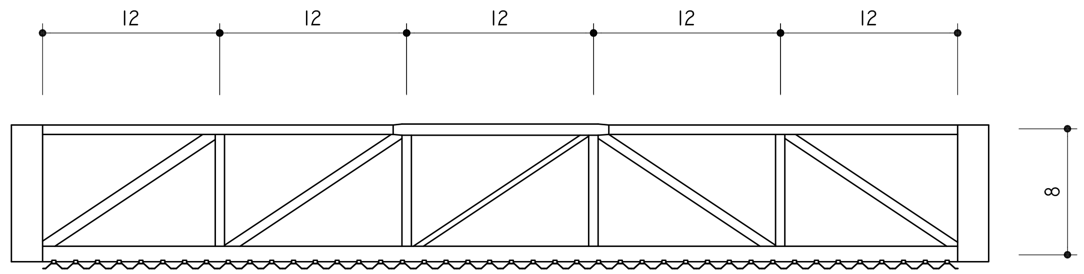
Benaming: **Connection between gate and caisson**

TU Delft, Faculteit Civiele Techniek en Geowetenschappen, Sectie GCC, Technisch Tekenen CT1112

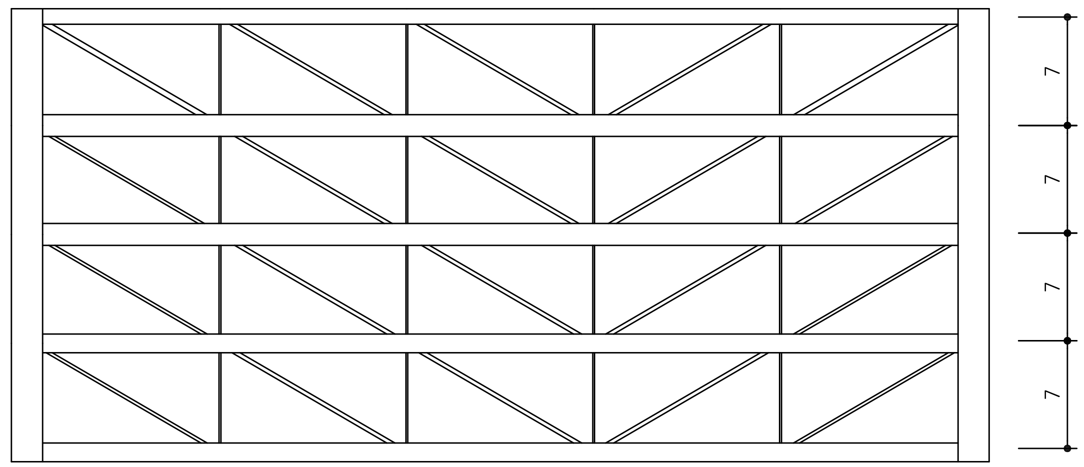
Formaat: A3

Schaal: ---

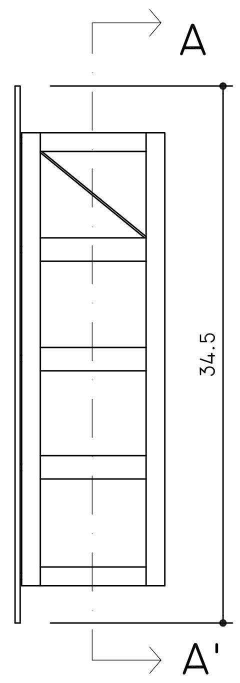
Eenheid: m



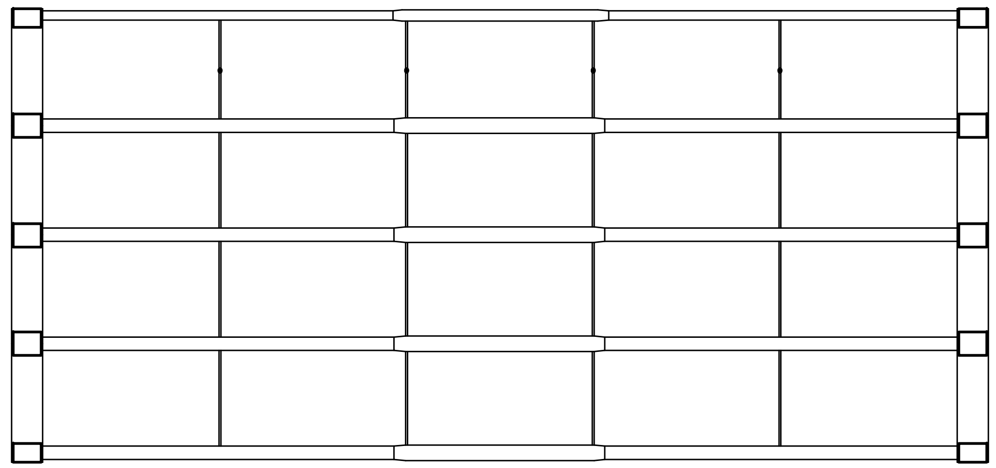
TOP VIEW



FRONT VIEW WITHOUT SCREEN



SIDE VIEW

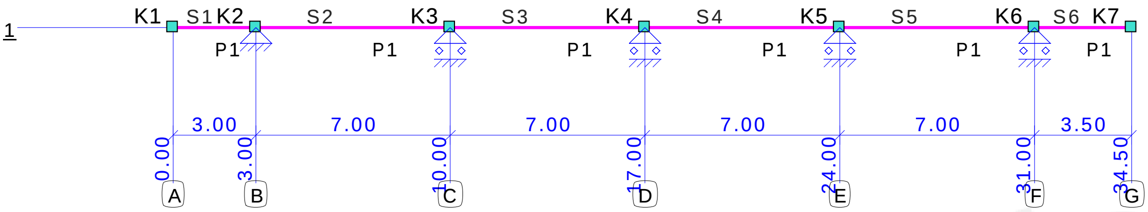


CROSS-SECTION A-A'

Formaat	A3
Schaal	---
Eenheid	m
Benaming	Gate design
	TU Delft, Faculteit Civiele Techniek en Geowetenschappen, Sectie GCC, Technisch Tekenen CT1112
Datum	30/01/13
Begeleider	Vrijling
Naam	Lex de Boom
Studienummer en projectgroep	1353764
Tek.nr.	07

APPENDIX VIII MATRIXFRAME CALCULATION GATE

Projectnaam		Projectnummer	
Omschrijving		Constructeur	
Opdrachtgever		Eenheden	m, kN, kNm
Bestand	O:\Graduation\Programs\screen met hoogste golf attempt 2 5 ondersteuningen.mxe		



Afb. Geometrie: Raamwerk

Staven

Staaf	Knoop B	B	Scharnier E	Knoop E	Profiel	X-B	Z-B	X-E	Z-E	Lengte
S1	K1	NVM	NVM	K2	P1	0.000	0.000	3.000	0.000	3.000
S2	K2	NVM	NVM	K3	P1	3.000	0.000	10.000	0.000	7.000
S3	K3	NVM	NVM	K4	P1	10.000	0.000	17.000	0.000	7.000
S4	K4	NVM	NVM	K5	P1	17.000	0.000	24.000	0.000	7.000
S5	K5	NVM	NVM	K6	P1	24.000	0.000	31.000	0.000	7.000
S6	K6	NVM	NVM	K7	P1	31.000	0.000	34.500	0.000	3.500
-	**-**	**-**	**-**	**-**	**-**	**m**	**m**	**m**	**m**	**m**

Profielen

Profiel	Profielnaam	Oppervlakte	Iy	Materiaal	Hoek
P1		1.4610e-02	2.2190e-04	S235	0
-	**-**	**m2**	**m4**	**-**	**°**

Materialen

Materiaalnaam	Dichtheid	E-Modulus	Uitzettingcoeff
S235	78.50	2.1000e+08	12.0000e-06
-	**kN/m3**	**kN/m2**	**C°m**

Opleggingen

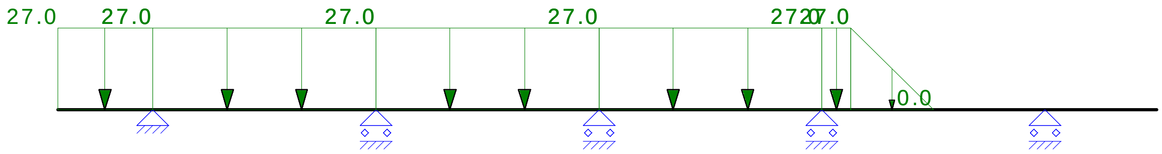
Oplegging	Knoop	X	Z	Yr	HoekYr
O1	K2	vast	vast	vrij	0
O2	K3	vrij	vast	vrij	0
O3	K4	vrij	vast	vrij	0
O4	K5	vrij	vast	vrij	0
O5	K6	vrij	vast	vrij	0
-	**-**	**kN/m**	**kN/m**	**kNmrad**	**°**

Belastingsgevallen typen

Oplegg.	Staven	B.G.Type	Gunstig/Ong.	Element	Niveau	Veld	PsiK	PsiI
B.G.1	Permanent	Permanent	-		N.v.t.	N.v.t.		
B.G.2	Permanent	Permanent	-		N.v.t.	N.v.t.		
B.G.3	Permanent	Permanent	-		N.v.t.	N.v.t.		
B.G.4	Permanent	Permanent	-		N.v.t.	N.v.t.		

B.G.1: Permanent

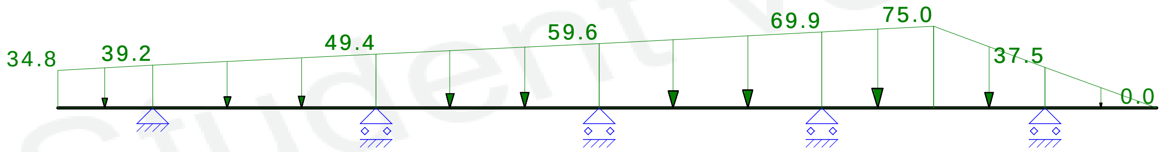
Type	Beginwaarde	Eindwaarde	Beginafstand	Eindafstand	Richting	Staaf of knoop
B.G.1: Permanent						
q	27.00	27.00	0.000	3.000(L)	Z"	S1-S4
q	27.00	27.00	0.000	0.900	Z'	S5
q	27.00	0.00	0.900	3.500	Z'	S5
Som lasten	**X: 0.00**	**kN Z: 707.40**	**kN**			
-	**-**	**-**	**m**	**m**	**-**	**-**



B.G.1: Permanent

B.G.2: Permanent

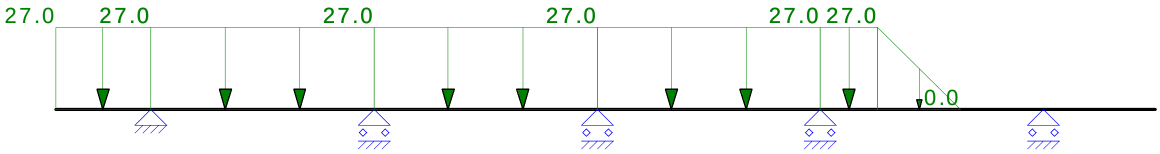
Type	Beginwaarde	Eindwaarde	Beginafstand	Eindafstand	Richting	Staaf of knoop
B.G.2: Permanent						
q	34.80	39.19	0.000	3.000(L)	Z'	S1
q	39.19	49.42	0.000	7.000(L)	Z'	S2
q	49.42	59.65	0.000	7.000(L)	Z'	S3
q	59.65	69.88	0.000	7.000(L)	Z'	S4
q	69.88	75.00	0.000	3.500	Z'	S5
q	75.00	37.50	3.500	7.000(L)	Z'	S5
q	37.50	0.00	0.000	3.500(L)	Z'	S6
Som lasten	**X: 0.00**	**kN**	**Z: 1,772.26**	**kN**		
-	**-**	**-**	**m**	**m**	**-**	**-**



B.G.2: Permanent

B.G.3: Permanent

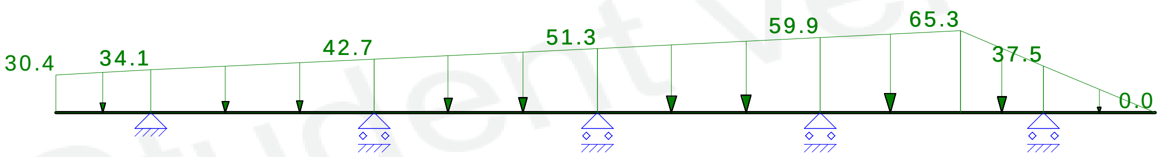
Type	Beginwaarde	Eindwaarde	Beginafstand	Eindafstand	Richting	Staaf of knoop
B.G.3: Permanent						
q	27.00	27.00	0.000	3.000(L)	Z'	S1-S4
q	27.00	0.00	1.800	4.400	Z'	S5
q	27.00	27.00	0.000	1.800	Z'	S5
Som lasten	**X: 0.00**	**kN**	**Z: 731.70**	**kN**		
-	**-**	**-**	**m**	**m**	**-**	**-**



B.G.3: Permanent

B.G.4: Permanent

Type	Beginwaarde	Eindwaarde	Beginafstand	Eindafstand	Richting	Staaf of knoop
B.G.4: Permanent						
q	37.47	0.00	0.000	3.500(L)	Z'	S6
q	65.30	37.47	4.400	7.000(L)	Z'	S5
q	30.40	34.09	0.000	3.000(L)	Z'	S1
q	34.09	42.69	0.000	7.000(L)	Z'	S2
q	42.69	51.29	0.000	7.000(L)	Z'	S3
q	51.29	59.89	0.000	7.000(L)	Z'	S4
q	59.89	65.30	0.000	4.400	Z'	S5
Som lasten	**X: 0.00**	**kN Z: 1,558.12**	**kN**			
-	**-**	**-**	**m**	**m**	**-**	**-**



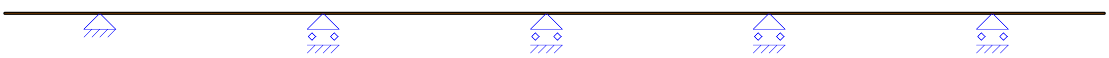
B.G.4: Permanent

Fundamenteel Belastingscombinaties

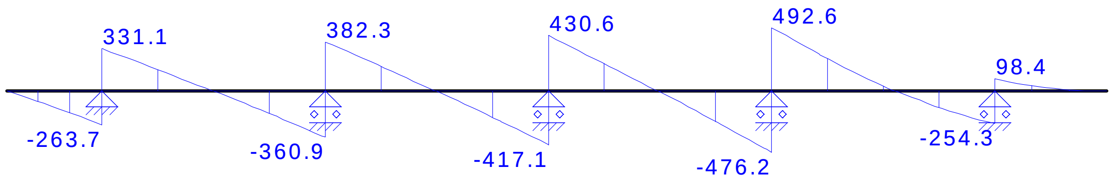
B.G.	Omschrijving	Fu.C.1	Fu.C.2
B.G.1	Permanent	1.20	-
B.G.2	Permanent	1.50	-
B.G.3	Permanent	-	1.20
B.G.4	Permanent	-	1.50

Uitgangspunten van de analyse

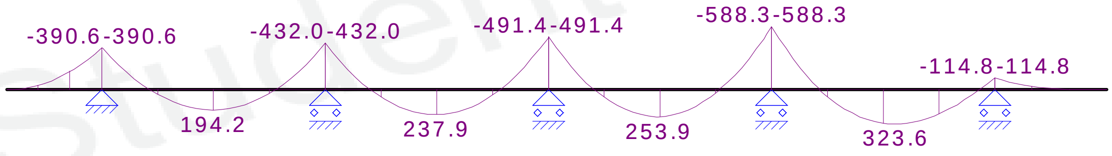
Lineaire Elastische Analyse uitgevoerd



Afb. Fu.C. Normaalkracht (Nx) Omhullende



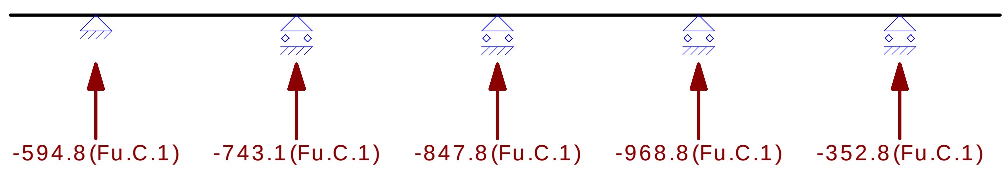
Afb. Fu.C. Dwarskracht (Vz) Omhullende



Afb. Fu.C. Momenten (My) Omhullende

Fu.C. Omhullende

Staaf	Nx Minus	Nx Plus	Vz Minus	Vz Plus	My Minus	My Plus
S1	0.00	0.00	-263.68	0.00	-390.58	0.00
S2	0.00	0.00	-360.87	331.13	-432.00	194.22
S3	0.00	0.00	-417.15	382.27	-491.40	237.86
S4	0.00	0.00	-476.22	430.62	-588.34	253.95
S5	0.00	0.00	-254.34	492.56	-588.34	323.57
S6	0.00	0.00	0.00	98.44	-114.84	0.00
-	**kN**	**kN**	**kN**	**kN**	**kNm**	**kNm**

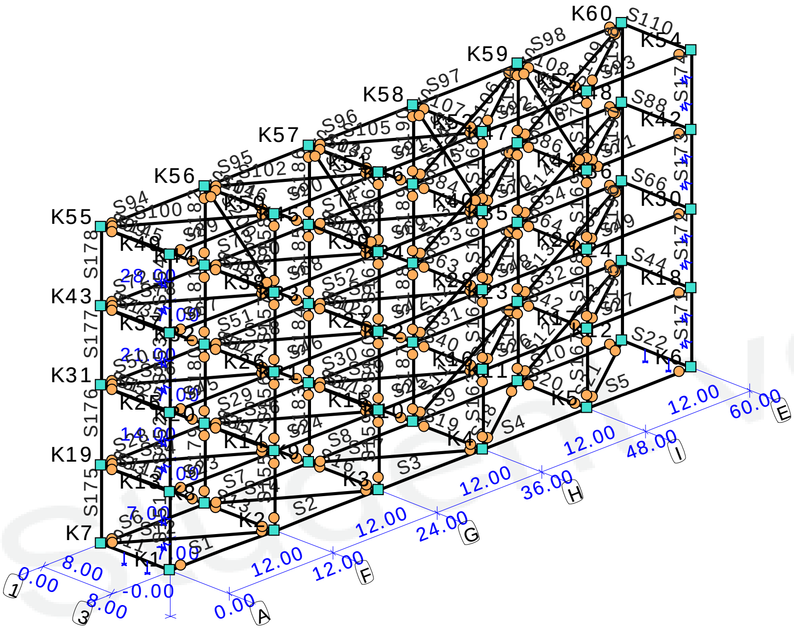


Afb. Fu.C. Oplegreacties Omhullende

Fu.C. Extreme oplegreacties

Oplegging	Knoop	B.C.	Xmax	Z	My	B.C.	X	Zmax	My	B.C.	X	Z	Mymax
O1	K2					Fu.C.1	0.00	**-594.81**	0.00				
O2	K3					Fu.C.1	0.00	**-743.14**	0.00				
O3	K4					Fu.C.1	0.00	**-847.76**	0.00				
O4	K5					Fu.C.1	0.00	**-968.78**	0.00				
O5	K6					Fu.C.1	0.00	**-352.78**	0.00				
Globale extreme waarden													
O4	K5					Fu.C.1	0.00	**-968.78**	0.00				
-	**-**	**-**	**kN**	**kN**	**kNm**	**-**	**kN**	**kN**	**kNm**		**kN**	**kN**	**kNm**

Projectnaam		Projectnummer	
Omschrijving		Constructeur	
Opdrachtgever		Eenheden	m, kN, kNm
Bestand	O:\Graduation\Programs\gate attempt 4.3.mxe		



Afb. Geometrie 1: Raamwerk

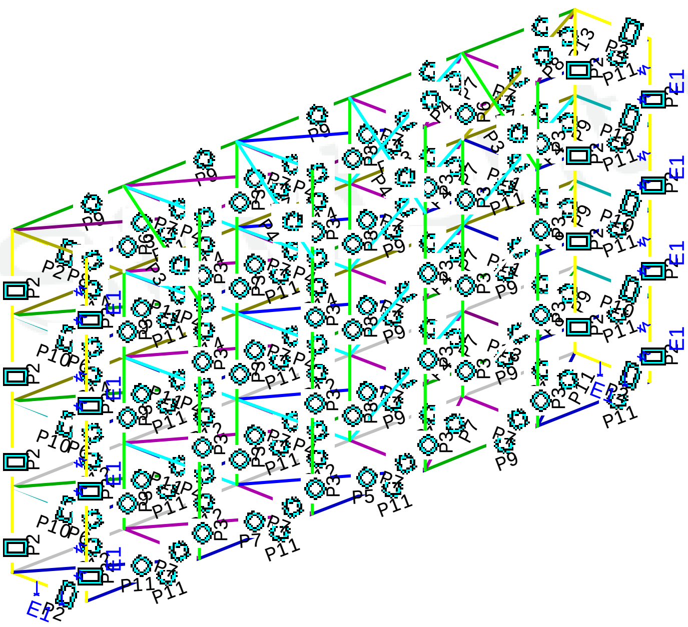
Staven

Staaf	Knoop B	Scharnier B	Scharnier E	Knoop E	Profiel	X-B	Y-B	Z-B	X-E	Y-E	Z-E	Lengte
S1	K1		XYZXrYrZr	K2	P11	0.000	8.000	0.000	12.000	8.000	0.000	12.000
S2	K2	XYZXrYrZr	XYZXrYrZr	K3	P11	12.000	8.000	0.000	24.000	8.000	0.000	12.000
S3	K3	XYZXrYrZr	XYZXrYrZr	K4	P11	24.000	8.000	0.000	36.000	8.000	0.000	12.000
S4	K4	XYZXrYrZr	XYZXrYrZr	K5	P9	36.000	8.000	0.000	48.000	8.000	0.000	12.000
S5	K5	XYZXrYrZr		K6	P11	48.000	8.000	0.000	60.000	8.000	0.000	12.000
S6	K7		XYZXrYrZr	K8	P12	0.000	0.000	0.000	12.000	0.000	0.000	12.000
S7	K8	XYZXrYrZr	XYZXrYrZr	K9	P12	12.000	0.000	0.000	24.000	0.000	0.000	12.000
S8	K9	XYZXrYrZr	XYZXrYrZr	K10	P12	24.000	0.000	0.000	36.000	0.000	0.000	12.000
S9	K10	XYZXrYrZr	XYZXrYrZr	K11	P12	36.000	0.000	0.000	48.000	0.000	0.000	12.000
S10	K11	XYZXrYrZr		K12	P12	48.000	0.000	0.000	60.000	0.000	0.000	12.000
S11	K7	XYZXrYrZr	XYZXrYrZr	K1	P2	0.000	0.000	0.000	0.000	8.000	0.000	8.000
S12	K7			K2	P11	0.000	0.000	0.000	12.000	8.000	0.000	14.422
S13	K8			K2	P7	12.000	0.000	0.000	12.000	8.000	0.000	8.000
S14	K8			K3	P7	12.000	0.000	0.000	24.000	8.000	0.000	14.422
S16	K9			K3	P7	24.000	0.000	0.000	24.000	8.000	0.000	8.000
S17	K9			K4	P5	24.000	0.000	0.000	36.000	8.000	0.000	14.422
S18	K4			K11	P7	36.000	8.000	0.000	48.000	0.000	0.000	14.422
S19	K10			K4	P7	36.000	0.000	0.000	36.000	8.000	0.000	8.000
S20	K11			K5	P7	48.000	0.000	0.000	48.000	8.000	0.000	8.000
S21	K5			K12	P11	48.000	8.000	0.000	60.000	0.000	0.000	14.422
S22	K12	XYZXrYrZr	XYZXrYrZr	K6	P2	60.000	0.000	0.000	60.000	8.000	0.000	8.000
S23	K13		XYZXrYrZr	K14	P11	0.000	8.000	-7.000	12.000	8.000	-7.000	12.000
S24	K14	XYZXrYrZr	XYZXrYrZr	K15	P11	12.000	8.000	-7.000	24.000	8.000	-7.000	12.000
S25	K15	XYZXrYrZr	XYZXrYrZr	K16	P9	24.000	8.000	-7.000	36.000	8.000	-7.000	12.000
S26	K16	XYZXrYrZr	XYZXrYrZr	K17	P9	36.000	8.000	-7.000	48.000	8.000	-7.000	12.000
S27	K17	XYZXrYrZr		K18	P11	48.000	8.000	-7.000	60.000	8.000	-7.000	12.000
S28	K19		XYZXrYrZr	K20	P12	0.000	0.000	-7.000	12.000	0.000	-7.000	12.000
S29	K20	XYZXrYrZr	XYZXrYrZr	K21	P12	12.000	0.000	-7.000	24.000	0.000	-7.000	12.000
S30	K21	XYZXrYrZr	XYZXrYrZr	K22	P12	24.000	0.000	-7.000	36.000	0.000	-7.000	12.000

Staaf	Knoop B	Scharnier B	Scharnier E	Knoop E	Profiel	X-B	Y-B	Z-B	X-E	Y-E	Z-E	Lengte
S31	K22	XYZXrYrZr	XYZXrYrZr	K23	P12	36.000	0.000	-7.000	48.000	0.000	-7.000	12.000
S32	K23	XYZXrYrZr		K24	P12	48.000	0.000	-7.000	60.000	0.000	-7.000	12.000
S33	K19	XYZXrYrZr	XYZXrYrZr	K13	P10	0.000	0.000	-7.000	0.000	8.000	-7.000	8.000
S34	K19			K14	P9	0.000	0.000	-7.000	12.000	8.000	-7.000	14.422
S35	K20			K14	P11	12.000	0.000	-7.000	12.000	8.000	-7.000	8.000
S36	K20			K15	P7	12.000	0.000	-7.000	24.000	8.000	-7.000	14.422
S37	K21			K15	P7	24.000	0.000	-7.000	24.000	8.000	-7.000	8.000
S39	K21			K16	P5	24.000	0.000	-7.000	36.000	8.000	-7.000	14.422
S40	K22			K16	P7	36.000	0.000	-7.000	36.000	8.000	-7.000	8.000
S41	K16			K23	P7	36.000	8.000	-7.000	48.000	0.000	-7.000	14.422
S42	K23			K17	P13	48.000	0.000	-7.000	48.000	8.000	-7.000	8.000
S43	K17			K24	P9	48.000	8.000	-7.000	60.000	0.000	-7.000	14.422
S44	K24	XYZXrYrZr	XYZXrYrZr	K18	P10	60.000	0.000	-7.000	60.000	8.000	-7.000	8.000
S45	K25		XYZXrYrZr	K26	P11	0.000	8.000	-14.000	12.000	8.000	-14.000	12.000
S46	K26	XYZXrYrZr	XYZXrYrZr	K27	P11	12.000	8.000	-14.000	24.000	8.000	-14.000	12.000
S47	K27	XYZXrYrZr	XYZXrYrZr	K28	P9	24.000	8.000	-14.000	36.000	8.000	-14.000	12.000
S48	K28	XYZXrYrZr	XYZXrYrZr	K29	P9	36.000	8.000	-14.000	48.000	8.000	-14.000	12.000
S49	K29	XYZXrYrZr		K30	P11	48.000	8.000	-14.000	60.000	8.000	-14.000	12.000
S50	K31		XYZXrYrZr	K32	P14	0.000	0.000	-14.000	12.000	0.000	-14.000	12.000
S51	K32	XYZXrYrZr	XYZXrYrZr	K33	P14	12.000	0.000	-14.000	24.000	0.000	-14.000	12.000
S52	K33	XYZXrYrZr	XYZXrYrZr	K34	P14	24.000	0.000	-14.000	36.000	0.000	-14.000	12.000
S53	K34	XYZXrYrZr	XYZXrYrZr	K35	P14	36.000	0.000	-14.000	48.000	0.000	-14.000	12.000
S54	K35	XYZXrYrZr		K36	P14	48.000	0.000	-14.000	60.000	0.000	-14.000	12.000
S55	K31	XYZXrYrZr	XYZXrYrZr	K25	P10	0.000	0.000	-14.000	0.000	8.000	-14.000	8.000
S56	K31			K26	P9	0.000	0.000	-14.000	12.000	8.000	-14.000	14.422
S57	K32			K26	P11	12.000	0.000	-14.000	12.000	8.000	-14.000	8.000
S58	K32			K27	P7	12.000	0.000	-14.000	24.000	8.000	-14.000	14.422
S60	K33			K27	P7	24.000	0.000	-14.000	24.000	8.000	-14.000	8.000
S61	K33			K28	P5	24.000	0.000	-14.000	36.000	8.000	-14.000	14.422
S62	K28			K35	P7	36.000	8.000	-14.000	48.000	0.000	-14.000	14.422
S63	K34			K28	P7	36.000	0.000	-14.000	36.000	8.000	-14.000	8.000
S64	K35			K29	P11	48.000	0.000	-14.000	48.000	8.000	-14.000	8.000
S65	K29			K36	P9	48.000	8.000	-14.000	60.000	0.000	-14.000	14.422
S66	K36	XYZXrYrZr	XYZXrYrZr	K30	P10	60.000	0.000	-14.000	60.000	8.000	-14.000	8.000
S67	K37		XYZXrYrZr	K38	P11	0.000	8.000	-21.000	12.000	8.000	-21.000	12.000
S68	K38	XYZXrYrZr	XYZXrYrZr	K39	P11	12.000	8.000	-21.000	24.000	8.000	-21.000	12.000
S69	K39	XYZXrYrZr	XYZXrYrZr	K40	P9	24.000	8.000	-21.000	36.000	8.000	-21.000	12.000
S70	K40	XYZXrYrZr	XYZXrYrZr	K41	P11	36.000	8.000	-21.000	48.000	8.000	-21.000	12.000
S71	K41			K42	P11	48.000	8.000	-21.000	60.000	8.000	-21.000	12.000
S72	K43		XYZXrYrZr	K44	P14	0.000	0.000	-21.000	12.000	0.000	-21.000	12.000
S73	K44	XYZXrYrZr	XYZXrYrZr	K45	P14	12.000	0.000	-21.000	24.000	0.000	-21.000	12.000
S74	K45	XYZXrYrZr	XYZXrYrZr	K46	P14	24.000	0.000	-21.000	36.000	0.000	-21.000	12.000
S75	K46	XYZXrYrZr	XYZXrYrZr	K47	P14	36.000	0.000	-21.000	48.000	0.000	-21.000	12.000
S76	K47	XYZXrYrZr		K48	P14	48.000	0.000	-21.000	60.000	0.000	-21.000	12.000
S77	K43	XYZXrYrZr	XYZXrYrZr	K37	P10	0.000	0.000	-21.000	0.000	8.000	-21.000	8.000
S78	K43			K38	P9	0.000	0.000	-21.000	12.000	8.000	-21.000	14.422
S79	K44			K38	P11	12.000	0.000	-21.000	12.000	8.000	-21.000	8.000
S80	K44			K39	P7	12.000	0.000	-21.000	24.000	8.000	-21.000	14.422
S81	K45			K39	P7	24.000	0.000	-21.000	24.000	8.000	-21.000	8.000
S83	K45			K40	P5	24.000	0.000	-21.000	36.000	8.000	-21.000	14.422
S84	K46			K40	P7	36.000	0.000	-21.000	36.000	8.000	-21.000	8.000
S85	K40			K47	P7	36.000	8.000	-21.000	48.000	0.000	-21.000	14.422
S86	K47			K41	P11	48.000	0.000	-21.000	48.000	8.000	-21.000	8.000
S87	K41			K48	P9	48.000	8.000	-21.000	60.000	0.000	-21.000	14.422
S88	K48	XYZXrYrZr	XYZXrYrZr	K42	P10	60.000	0.000	-21.000	60.000	8.000	-21.000	8.000
S89	K49		XYZXrYrZr	K50	P11	0.000	8.000	-28.000	12.000	8.000	-28.000	12.000
S90	K50	XYZXrYrZr	XYZXrYrZr	K51	P7	12.000	8.000	-28.000	24.000	8.000	-28.000	12.000
S91	K51	XYZXrYrZr	XYZXrYrZr	K52	P13	24.000	8.000	-28.000	36.000	8.000	-28.000	12.000
S92	K52	XYZXrYrZr	XYZXrYrZr	K53	P7	36.000	8.000	-28.000	48.000	8.000	-28.000	12.000
S93	K53	XYZXrYrZr		K54	P11	48.000	8.000	-28.000	60.000	8.000	-28.000	12.000
S94	K55		XYZXrYrZr	K56	P9	0.000	0.000	-28.000	12.000	0.000	-28.000	12.000
S95	K56	XYZXrYrZr	XYZXrYrZr	K57	P9	12.000	0.000	-28.000	24.000	0.000	-28.000	12.000
S96	K57	XYZXrYrZr	XYZXrYrZr	K58	P9	24.000	0.000	-28.000	36.000	0.000	-28.000	12.000

Staaf	Knoop B	Scharnier B	Scharnier E	Knoop E	Profiel	X-B	Y-B	Z-B	X-E	Y-E	Z-E	Lengte
S97	K58	XYZXrYrZr	XYZXrYrZr	K59	P9	36.000	0.000	-28.000	48.000	0.000	-28.000	12.000
S98	K59	XYZXrYrZr		K60	P9	48.000	0.000	-28.000	60.000	0.000	-28.000	12.000
S99	K55	XYZXrYrZr	XYZXrYrZr	K49	P2	0.000	0.000	-28.000	0.000	8.000	-28.000	8.000
S100	K55			K50	P13	0.000	0.000	-28.000	12.000	8.000	-28.000	14.422
S101	K56			K50	P7	12.000	0.000	-28.000	12.000	8.000	-28.000	8.000
S102	K56			K51	P7	12.000	0.000	-28.000	24.000	8.000	-28.000	14.422
S103	K57			K51	P7	24.000	0.000	-28.000	24.000	8.000	-28.000	8.000
S105	K57			K52	P5	24.000	0.000	-28.000	36.000	8.000	-28.000	14.422
S106	K52			K59	P7	36.000	8.000	-28.000	48.000	0.000	-28.000	14.422
S107	K58			K52	P7	36.000	0.000	-28.000	36.000	8.000	-28.000	8.000
S108	K59			K53	P7	48.000	0.000	-28.000	48.000	8.000	-28.000	8.000
S109	K53			K60	P13	48.000	8.000	-28.000	60.000	0.000	-28.000	14.422
S110	K60	XYZXrYrZr	XYZXrYrZr	K54	P2	60.000	0.000	-28.000	60.000	8.000	-28.000	8.000
S115	K19			K8	P6	0.000	0.000	-7.000	12.000	0.000	0.000	13.892
S117	K20			K9	P4	12.000	0.000	-7.000	24.000	0.000	0.000	13.892
S118	K21			K10	P4	24.000	0.000	-7.000	36.000	0.000	0.000	13.892
S119	K10			K23	P4	36.000	0.000	0.000	48.000	0.000	-7.000	13.892
S120	K11			K24	P6	48.000	0.000	0.000	60.000	0.000	-7.000	13.892
S125	K31			K20	P6	0.000	0.000	-14.000	12.000	0.000	-7.000	13.892
S127	K32			K21	P4	12.000	0.000	-14.000	24.000	0.000	-7.000	13.892
S128	K22			K35	P4	36.000	0.000	-7.000	48.000	0.000	-14.000	13.892
S129	K33			K22	P4	24.000	0.000	-14.000	36.000	0.000	-7.000	13.892
S130	K23			K36	P6	48.000	0.000	-7.000	60.000	0.000	-14.000	13.892
S135	K43			K32	P6	0.000	0.000	-21.000	12.000	0.000	-14.000	13.892
S137	K44			K33	P4	12.000	0.000	-21.000	24.000	0.000	-14.000	13.892
S138	K34			K47	P4	36.000	0.000	-14.000	48.000	0.000	-21.000	13.892
S139	K45	XYZXrYrZr	XYZXrYrZr	K34	P4	24.000	0.000	-21.000	36.000	0.000	-14.000	13.892
S140	K35			K48	P4	48.000	0.000	-14.000	60.000	0.000	-21.000	13.892
S145	K55			K44	P8	0.000	0.000	-28.000	12.000	0.000	-21.000	13.892
S146	K56			K45	P4	12.000	0.000	-28.000	24.000	0.000	-21.000	13.892
S148	K57			K46	P4	24.000	0.000	-28.000	36.000	0.000	-21.000	13.892
S149	K46			K59	P4	36.000	0.000	-21.000	48.000	0.000	-28.000	13.892
S150	K47			K60	P8	48.000	0.000	-21.000	60.000	0.000	-28.000	13.892
S151	K13	XYZXrYrZr	XYZXrYrZr	K1	P2	0.000	8.000	-7.000	0.000	8.000	0.000	7.000
S152	K25	XYZXrYrZr	XYZXrYrZr	K13	P2	0.000	8.000	-14.000	0.000	8.000	-7.000	7.000
S153	K37	XYZXrYrZr	XYZXrYrZr	K25	P2	0.000	8.000	-21.000	0.000	8.000	-14.000	7.000
S154	K49	XYZXrYrZr	XYZXrYrZr	K37	P2	0.000	8.000	-28.000	0.000	8.000	-21.000	7.000
S155	K14			K2	P3	12.000	8.000	-7.000	12.000	8.000	0.000	7.000
S156	K26			K14	P3	12.000	8.000	-14.000	12.000	8.000	-7.000	7.000
S157	K38			K26	P3	12.000	8.000	-21.000	12.000	8.000	-14.000	7.000
S158	K50			K38	P3	12.000	8.000	-28.000	12.000	8.000	-21.000	7.000
S159	K15			K3	P3	24.000	8.000	-7.000	24.000	8.000	0.000	7.000
S160	K27			K15	P3	24.000	8.000	-14.000	24.000	8.000	-7.000	7.000
S161	K39			K27	P3	24.000	8.000	-21.000	24.000	8.000	-14.000	7.000
S162	K51			K39	P3	24.000	8.000	-28.000	24.000	8.000	-21.000	7.000
S163	K16			K4	P3	36.000	8.000	-7.000	36.000	8.000	0.000	7.000
S164	K28			K16	P3	36.000	8.000	-14.000	36.000	8.000	-7.000	7.000
S165	K40			K28	P3	36.000	8.000	-21.000	36.000	8.000	-14.000	7.000
S166	K52			K40	P3	36.000	8.000	-28.000	36.000	8.000	-21.000	7.000
S167	K17			K5	P3	48.000	8.000	-7.000	48.000	8.000	0.000	7.000
S168	K29			K17	P3	48.000	8.000	-14.000	48.000	8.000	-7.000	7.000
S169	K41			K29	P3	48.000	8.000	-21.000	48.000	8.000	-14.000	7.000
S170	K53			K41	P3	48.000	8.000	-28.000	48.000	8.000	-21.000	7.000
S171	K18	XYZXrYrZr	XYZXrYrZr	K6	P2	60.000	8.000	-7.000	60.000	8.000	0.000	7.000
S172	K30	XYZXrYrZr	XYZXrYrZr	K18	P2	60.000	8.000	-14.000	60.000	8.000	-7.000	7.000
S173	K42	XYZXrYrZr	XYZXrYrZr	K30	P2	60.000	8.000	-21.000	60.000	8.000	-14.000	7.000
S174	K54	XYZXrYrZr	XYZXrYrZr	K42	P2	60.000	8.000	-28.000	60.000	8.000	-21.000	7.000
S175	K19	XYZXrYrZr	XYZXrYrZr	K7	P2	0.000	0.000	-7.000	0.000	0.000	0.000	7.000
S176	K31	XYZXrYrZr	XYZXrYrZr	K19	P2	0.000	0.000	-14.000	0.000	0.000	-7.000	7.000
S177	K43	XYZXrYrZr	XYZXrYrZr	K31	P2	0.000	0.000	-21.000	0.000	0.000	-14.000	7.000
S178	K55	XYZXrYrZr	XYZXrYrZr	K43	P2	0.000	0.000	-28.000	0.000	0.000	-21.000	7.000
S179	K20			K8	P3	12.000	0.000	-7.000	12.000	0.000	0.000	7.000
S180	K32			K20	P3	12.000	0.000	-14.000	12.000	0.000	-7.000	7.000

Staaf	Knoop B	Scharnier B	Scharnier E	Knoop E	Profiel	X-B	Y-B	Z-B	X-E	Y-E	Z-E	Lengte
S181	K44			K32	P3	12.000	0.000	-21.000	12.000	0.000	-14.000	7.000
S182	K56			K44	P6	12.000	0.000	-28.000	12.000	0.000	-21.000	7.000
S183	K21			K9	P3	24.000	0.000	-7.000	24.000	0.000	0.000	7.000
S184	K33			K21	P3	24.000	0.000	-14.000	24.000	0.000	-7.000	7.000
S185	K45			K33	P3	24.000	0.000	-21.000	24.000	0.000	-14.000	7.000
S186	K57			K45	P3	24.000	0.000	-28.000	24.000	0.000	-21.000	7.000
S187	K22			K10	P3	36.000	0.000	-7.000	36.000	0.000	0.000	7.000
S188	K34			K22	P3	36.000	0.000	-14.000	36.000	0.000	-7.000	7.000
S189	K46			K34	P3	36.000	0.000	-21.000	36.000	0.000	-14.000	7.000
S190	K58			K46	P3	36.000	0.000	-28.000	36.000	0.000	-21.000	7.000
S191	K23			K11	P3	48.000	0.000	-7.000	48.000	0.000	0.000	7.000
S192	K35			K23	P3	48.000	0.000	-14.000	48.000	0.000	-7.000	7.000
S193	K47			K35	P3	48.000	0.000	-21.000	48.000	0.000	-14.000	7.000
S194	K59			K47	P6	48.000	0.000	-28.000	48.000	0.000	-21.000	7.000
S195	K24	XYZXrYrZr	XYZXrYrZr	K12	P2	60.000	0.000	-7.000	60.000	0.000	0.000	7.000
S196	K36	XYZXrYrZr	XYZXrYrZr	K24	P2	60.000	0.000	-14.000	60.000	0.000	-7.000	7.000
S197	K48	XYZXrYrZr	XYZXrYrZr	K36	P2	60.000	0.000	-21.000	60.000	0.000	-14.000	7.000
S198	K60	XYZXrYrZr	XYZXrYrZr	K48	P2	60.000	0.000	-28.000	60.000	0.000	-21.000	7.000
S200	K56			K38	P3	12.000	0.000	-28.000	12.000	8.000	-21.000	10.630
S201	K57			K39	P4	24.000	0.000	-28.000	24.000	8.000	-21.000	10.630
S202	K58			K40	P4	36.000	0.000	-28.000	36.000	8.000	-21.000	10.630
S203	K59			K41	P3	48.000	0.000	-28.000	48.000	8.000	-21.000	10.630
-	-	-	-	-	-	m	m	m	m	m	m	m



Afb. Geometrie 2: Raamwerk

Profielen

Profiel	Profielnaam	Oppervlakte	It	Iy	Iz	Materiaal	Hoek
P2	K1000x1200x50x50	2.1000e-01	5.6836e-02	4.4175e-02	3.3175e-02	S460	0
P3	B120x15	4.9480e-03	1.3916e-05	6.9581e-06	6.9581e-06	S460	0
P4	B260x20	1.5080e-02	2.1865e-04	1.0933e-04	1.0933e-04	S460	0
P5	B360x20	2.1363e-02	6.1952e-04	3.0976e-04	3.0976e-04	S460	0
P6	B200x35	1.1310e-02	9.2740e-05	4.6370e-05	4.6370e-05	S460	0
P7	B600x35	6.2125e-02	4.9770e-03	2.4885e-03	2.4885e-03	S460	0
P8	B360x30	3.1102e-02	8.5374e-04	4.2687e-04	4.2687e-04	S460	0
P9	B1000x50	1.4923e-01	3.3762e-02	1.6881e-02	1.6881e-02	S460	0

Profiel	Profielnaam	Oppervlakte	It	Iy	Iz	Materiaal	Hoek
P10	K1000x1500x50x50	2.4000e-01	7.9063e-02	7.5450e-02	3.9950e-02	S460	0
P11	B860x45	1.1522e-01	1.9191e-02	9.5955e-03	9.5955e-03	S460	0
P12	B1200x50	1.8064e-01	5.9838e-02	2.9919e-02	2.9919e-02	S460	0
P13	B720x35	7.5320e-02	8.8585e-03	4.4293e-03	4.4293e-03	S460	0
P14	B1400x50	2.1206e-01	9.6751e-02	4.8376e-02	4.8376e-02	S460	0
-	**-**	**m2**	**m4**	**m4**	**m4**	**-**	**°**

Profielvormen

Profiel	Verlopende hoogte	hB	hE	tf	tw	tf2	B	bL	bR	Raatliggers	Mx
P2	Nee	1.200	1.200	0.050	0.050	0.000	1.000	0.325	0.325	Nee	0.000
P3	Nee	0.120	0.120	0.000	0.015	0.000	0.120	0.000	0.000	Nee	0.000
P4	Nee	0.260	0.260	0.000	0.020	0.000	0.260	0.000	0.000	Nee	0.000
P5	Nee	0.360	0.360	0.000	0.020	0.000	0.360	0.000	0.000	Nee	0.000
P6	Nee	0.200	0.200	0.000	0.020	0.000	0.200	0.000	0.000	Nee	0.000
P7	Nee	0.600	0.600	0.000	0.035	0.000	0.600	0.000	0.000	Nee	0.000
P8	Nee	0.360	0.360	0.000	0.030	0.000	0.360	0.000	0.000	Nee	0.000
P9	Nee	1.000	1.000	0.000	0.050	0.000	1.000	0.000	0.000	Nee	0.000
P10	Nee	1.500	1.500	0.050	0.050	0.000	1.000	0.375	0.375	Nee	0.000
P11	Nee	0.860	0.860	0.000	0.045	0.000	0.860	0.000	0.000	Nee	0.000
P12	Nee	1.200	1.200	0.000	0.050	0.000	1.200	0.000	0.000	Nee	0.000
P13	Nee	0.720	0.720	0.000	0.035	0.000	0.720	0.000	0.000	Nee	0.000
P14	Nee	1.400	1.400	0.000	0.050	0.000	1.400	0.000	0.000	Nee	0.000
-	**-**	**m**	**m**	**m**	**m**	**m**	**m**	**m**	**m**	**-**	**m**

Materialen

Materiaalnaam	Poison	Dichtheid	E-Modulus	Uitzettingcoeff
S460	0.30	78.50	2.1000e+08	12.0000e-06
-	**-**	**kN/m3**	**kN/m2**	**C°m**

Elastische bedding

Staaf	Verlopende hoogte	Type constant	Eenheden	Cz B	Cz E	Cy B	Cy E	Pasternak Instellingen: Pasternak	Cfy B	Cfy E	Cfz B	Cfz E	Breedte	Trek Verwijdering
S11	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
S22	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
S151	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
S152	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
S153	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
S154	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
S171	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
S172	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
S173	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
S174	Nee	Veer	kN/m3*(m)	00000.00	00000.00	00000.00	00000.00	Nee	0.00	0.00	0.00	0.00	N.v.t.	Nee
-	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**m**	**-**

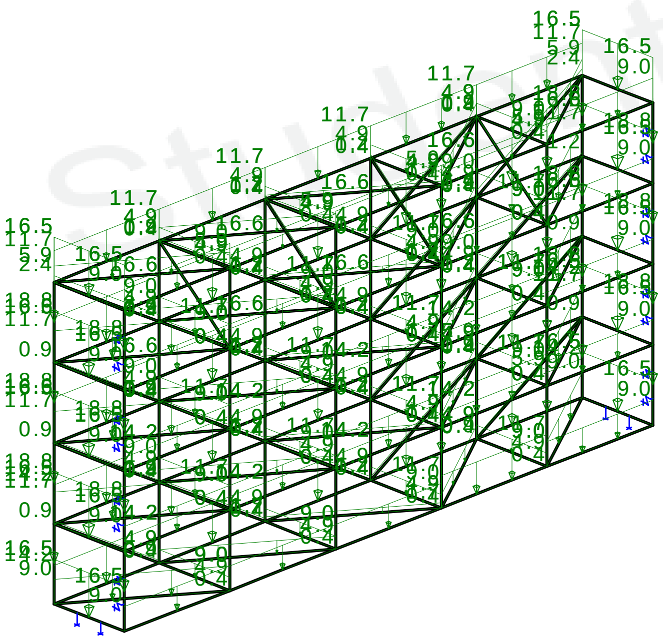
Belastingsgevallen typen

Oplegg.	Staven	B.G.Type	Gunstig/Ong.	Element	Niveau	Veld	PsiK	PsiI
B.G.1	Self weight	Permanent	-		N.v.t.	N.v.t.		
B.G.2	Load from gates	Permanent	-		N.v.t.	N.v.t.		
B.G.3	Horizontal load from gates	Permanent	-		N.v.t.	N.v.t.		

B.G.1: Self weight

Type	Beginwaarde	Eindwaarde	Beginafstand	Eindafstand	Richting	Staaf of knoop
B.G.1: Self weight						
qG	9.04 (1.00x)	9.04 (1.00x)	0.000	12.000(L)	Z"	S1-S3,S5,S23-S24, S27,S45-S46,S49, S67-S68,S70-S71, S89,S93
qG	11.71 (1.00x)	11.71 (1.00x)	0.000	12.000(L)	Z"	S4,S25-S26,S47-S48, S69,S94-S98
qG	14.18 (1.00x)	14.18 (1.00x)	0.000	12.000(L)	Z"	S6-S10,S28-S32
qG	16.49 (1.00x)	16.49 (1.00x)	0.000	8.000(L)	Z"	S11,S22,S99,S110
qG	9.04 (1.00x)	9.04 (1.00x)	0.000	14.422(L)	Z"	S12,S21

Type	Beginwaarde	Eindwaarde	Beginafstand	Eindafstand	Richting	Staaf of knoop
B.G.1: Self weight						
qG	4.88 (1.00x)	4.88 (1.00x)	0.000	8.000(L)	Z"	S13,S16,S19-S20, S37,S40,S60,S63, S81,S84,S101,S103, S107-S108
qG	4.88 (1.00x)	4.88 (1.00x)	0.000	14.422(L)	Z"	S14,S18,S36,S41, S58,S62,S80,S85, S102,S106
qG	1.68 (1.00x)	1.68 (1.00x)	0.000	14.422(L)	Z"	S17,S39,S61,S83, S105
qG	18.84 (1.00x)	18.84 (1.00x)	0.000	8.000(L)	Z"	S33,S44,S55,S66, S77,S88
qG	11.71 (1.00x)	11.71 (1.00x)	0.000	14.422(L)	Z"	S34,S43,S56,S65, S78,S87
qG	9.04 (1.00x)	9.04 (1.00x)	0.000	8.000(L)	Z"	S35,S57,S64,S79, S86
qG	5.91 (1.00x)	5.91 (1.00x)	0.000	8.000(L)	Z"	S42
qG	16.65 (1.00x)	16.65 (1.00x)	0.000	12.000(L)	Z"	S50-S54,S72-S76
qG	4.88 (1.00x)	4.88 (1.00x)	0.000	12.000(L)	Z"	S90,S92
qG	5.91 (1.00x)	5.91 (1.00x)	0.000	12.000(L)	Z"	S91
qG	5.91 (1.00x)	5.91 (1.00x)	0.000	14.422(L)	Z"	S100,S109
qG	0.89 (1.00x)	0.89 (1.00x)	0.000	13.892(L)	Z"	S115,S120,S125, S130,S135
qG	1.18 (1.00x)	1.18 (1.00x)	0.000	13.892(L)	Z"	S117-S119,S127-S129, S137-S140,S146, S148-S149
qG	2.44 (1.00x)	2.44 (1.00x)	0.000	13.892(L)	Z"	S145,S150
qG	16.49 (1.00x)	16.49 (1.00x)	0.000	7.000(L)	Z"	S151-S154,S171-S178, S195-S198
qG	0.39 (1.00x)	0.39 (1.00x)	0.000	7.000(L)	Z"	S155-S170,S179-S181, S183-S193
qG	0.89 (1.00x)	0.89 (1.00x)	0.000	7.000(L)	Z"	S182,S194
qG	0.39 (1.00x)	0.39 (1.00x)	0.000	10.630(L)	Z"	S200,S203
qG	1.18 (1.00x)	1.18 (1.00x)	0.000	10.630(L)	Z"	S201-S202
Som lasten	**X: 0.00**	**kN Y: 0.00**	**kN Z: 14,143.60**	**kN**		
-	**-**	**-**	**m**	**m**	**- -**	

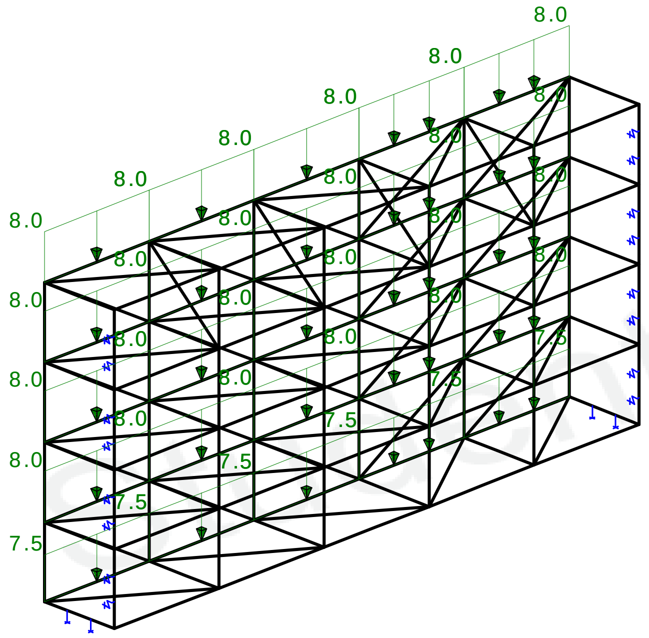


B.G.1: Self weight

B.G.2: Load from gates

Type	Beginwaarde	Eindwaarde	Beginafstand	Eindafstand	Richting	Staaf of knoop
B.G.2: Load from gates						
q	7.46	7.46	0.000	12.000(L)	Z"	S6-S10
q	8.03	8.03	0.000	12.000(L)	Z"	S28-S32,S50-S54, S72-S76,S94-S98
Som lasten	**X: 0.00**	**kN Y: 0.00**	**kN Z: 2,374.80**	**kN**		

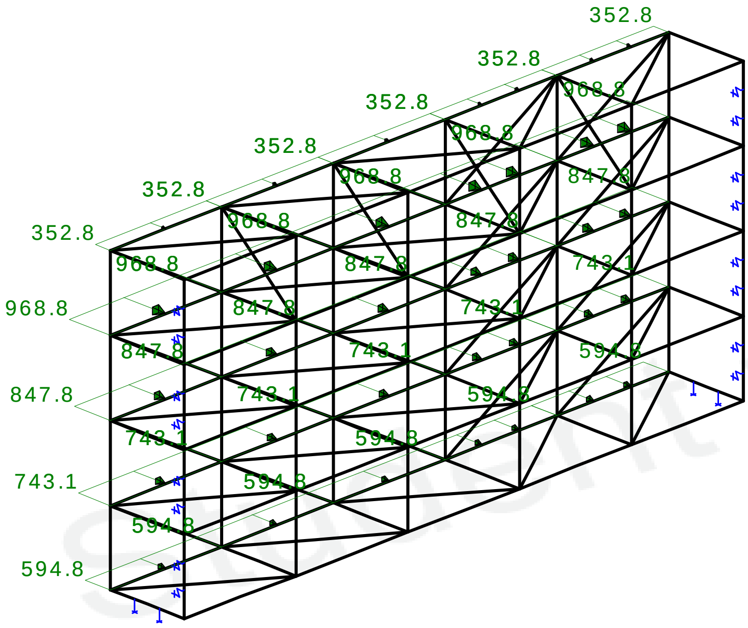
-	-	-	m	m	- -



B.G.2: Load from gates

B.G.3: Horizontal load from gates

Type	Beginwaarde	Eindwaarde	Beginafstand	Eindafstand	Richting	Staaf of knoop
B.G.3: Horizontal load from gates						
q	352.80	352.80	0.000	12.000(L)	Y''	S94-S98
q	968.80	968.80	0.000	12.000(L)	Y''	S72-S76
q	847.80	847.80	0.000	12.000(L)	Y''	S50-S54
q	743.10	743.10	0.000	12.000(L)	Y''	S28-S32
q	594.80	594.80	0.000	12.000(L)	Y''	S6-S10
Som lasten	**X: 0.00**	**kN Y: 210,438.00**	**kN Z: 0.00**	**kN**		
-	**-**	**-**	**m**	**m**	**- -**	



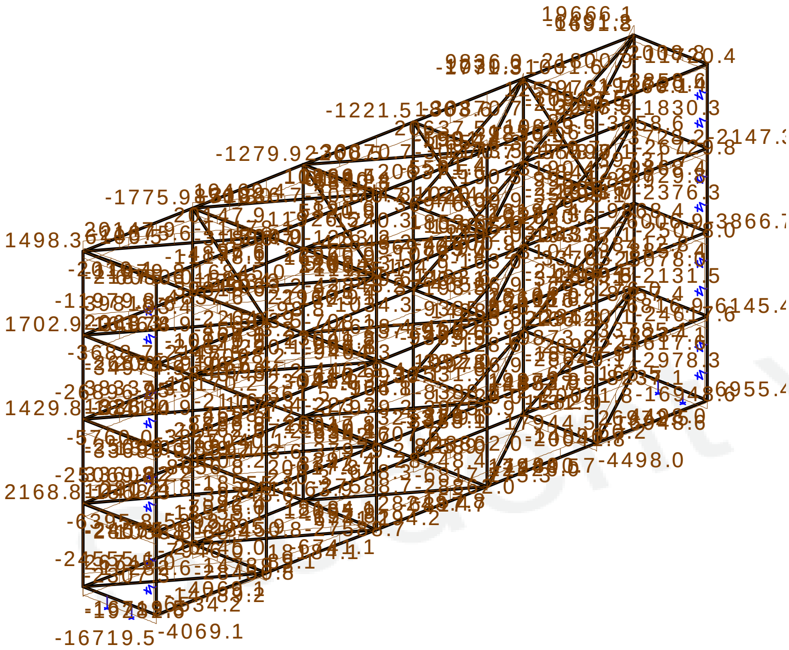
B.G.3: Horizontal load from gates

Fundamenteel Belastingscombinaties

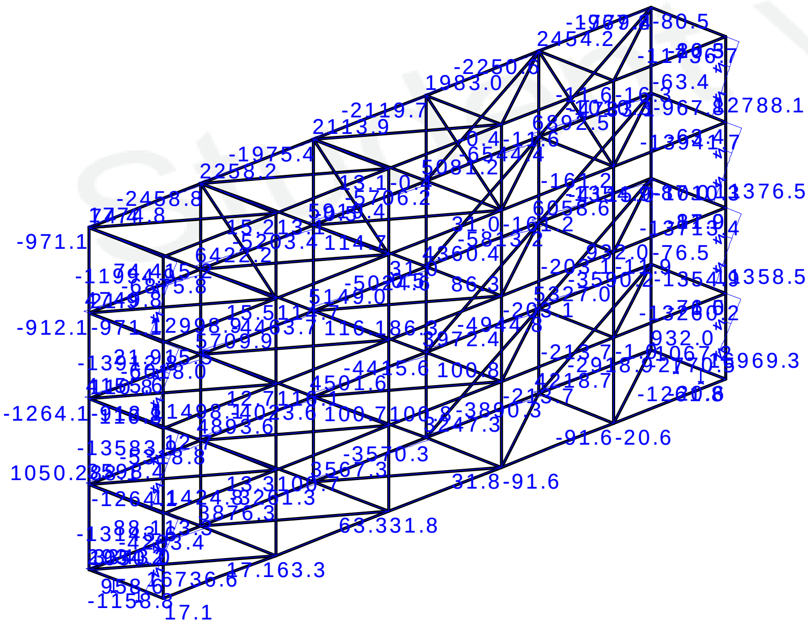
B.G.	Omschrijving	Fu.C.1
B.G.1	Self weight	1.20
B.G.2	Load from gates	1.20
B.G.3	Horizontal load from gates	1.00

Uitgangspunten van de analyse

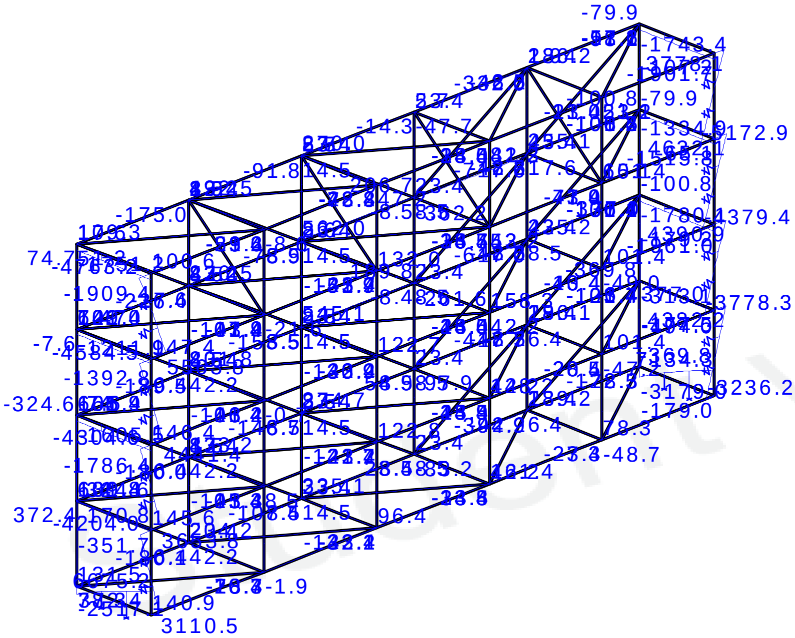
Lineaire Elastische Analyse uitgevoerd



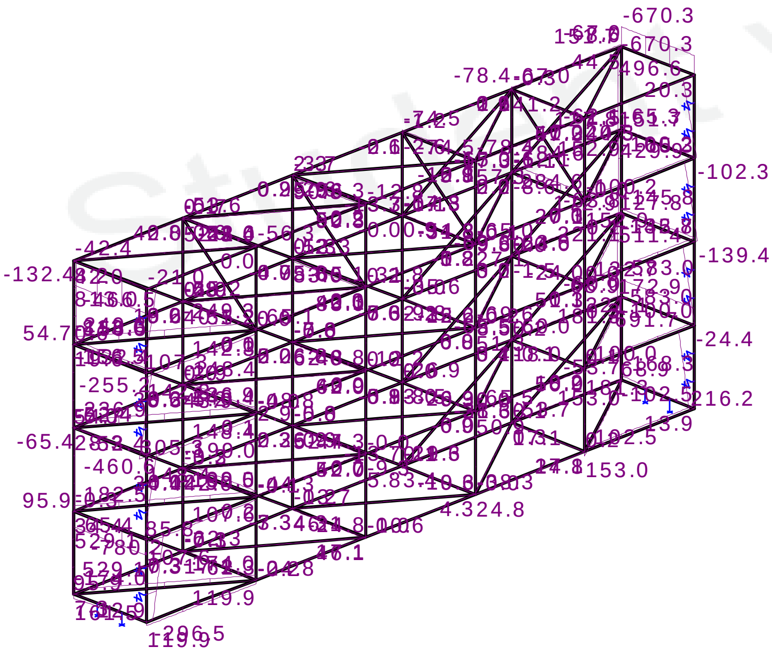
Afb. Fu.C. Normaalkracht (Nx) Omhullende



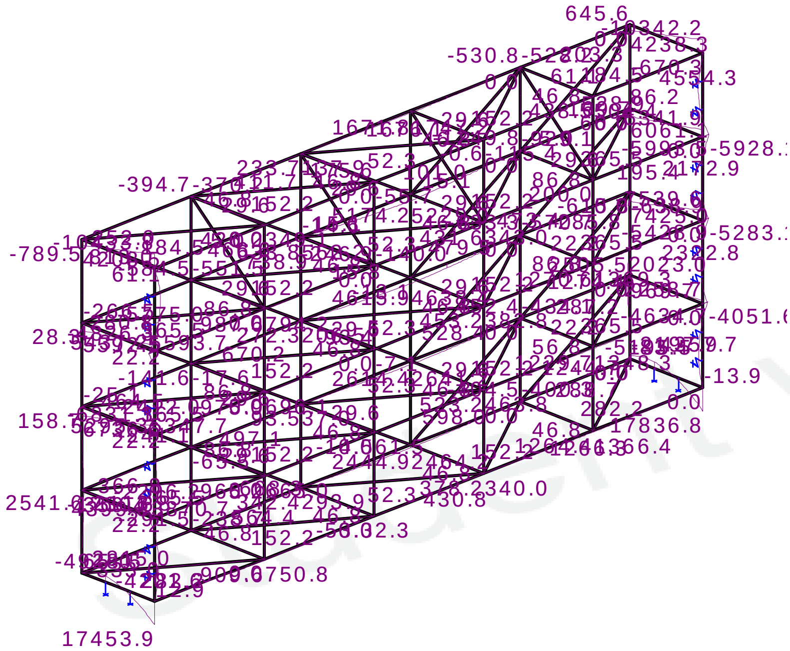
Afb. Fu.C. Dwarskracht (Vy) Omhullende



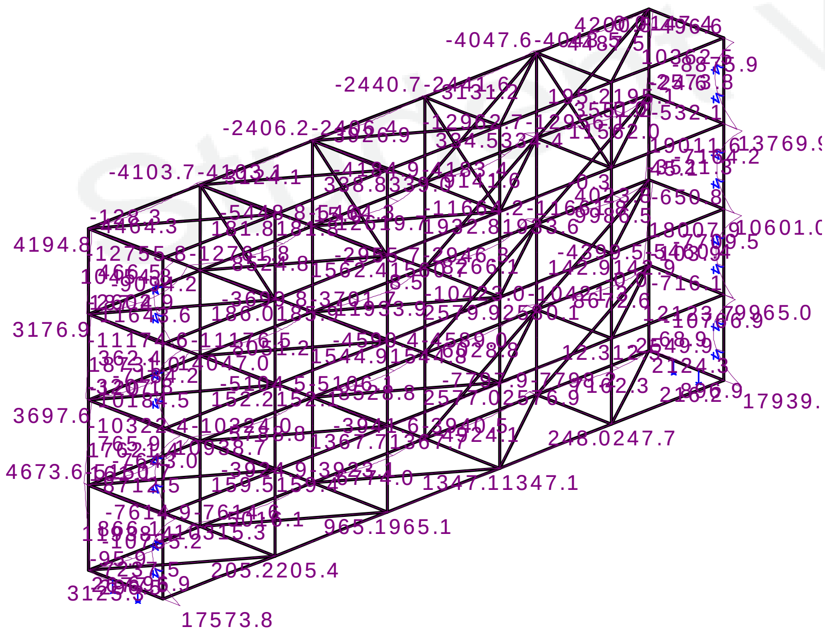
Afb. Fu.C. Dwarskracht (Vz) Omhullende



Afb. Fu.C. Torsiemomenten Omhullende



Afb. Fu.C. Momenten (My) Omhullende



Afb. Fu.C. Momenten (Mz) Omhullende

Fu.C. Omhullende

Staaf	Nx Minus	Nx Plus	Vy Minus	Vy Plus	Vz Minus	Vz Plus	Mx Minus	Mx Plus	My Minus	My Plus	Mz Minu
S1	-4069.11	0.00	0.00	17.10	0.00	140.92	0.00	119.92	0.00	909.60	0.
S2	0.00	18184.06	0.00	63.31	-132.13	0.00	-24.84	0.00	-53.33	750.77	0.
S3	0.00	28248.42	0.00	31.83	-33.80	96.45	-10.60	0.00	0.00	430.80	0.
S4	0.00	17944.47	-91.59	0.00	-7.31	161.37	0.00	24.84	0.00	1266.28	0.
S5	-4497.99	0.00	-20.65	0.00	-178.98	0.00	-102.50	0.00	0.00	1366.36	0.

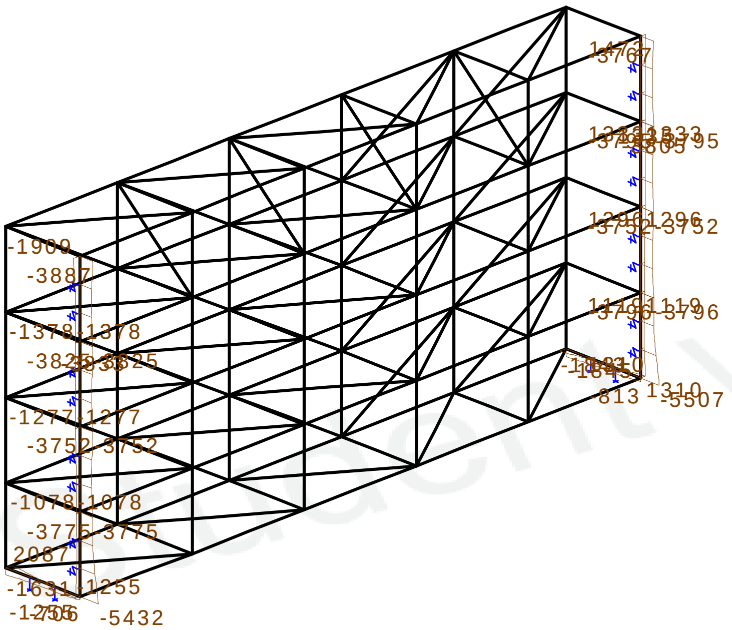
Staaf	Nx Minus	Nx Plus	Vy Minus	Vy Plus	Vz Minus	Vz Plus	Mx Minus	Mx Plus	My Minus	My Plus	Mz Minu
S6	-19282.56	0.00	-4203.38	2934.22	-180.10	131.52	0.00	7.25	-291.51	333.04	-7614.
S7	-28450.75	0.00	-3261.32	3876.28	-107.39	204.23	-7.27	0.00	-238.70	564.43	-7614.
S8	-27588.65	0.00	-3570.35	3567.25	0.00	335.06	-13.69	0.00	0.00	2444.88	-3941.
S9	-28862.00	0.00	-3890.25	3247.35	-394.86	0.00	-21.60	0.00	-404.47	2464.18	-7797.
S10	-20041.79	0.00	-2918.95	4218.65	-122.45	189.17	-53.72	0.00	-400.31	288.70	-7798.
S11	-16719.50	0.00	-1158.78	3342.98	-2517.10	6675.15	-12.89	161.51	-4928.49	17453.86	-696.
S12	0.00	26744.98	0.00	0.00	-78.27	78.27	-173.99	0.00	0.00	282.19	0.
S13	-14789.23	0.00	0.00	0.00	-23.41	23.41	-62.32	0.00	0.00	46.82	0.
S14	0.00	12095.86	0.00	0.00	-42.20	42.20	0.00	17.13	0.00	152.16	0.
S16	-6741.06	0.00	0.00	0.00	-23.41	23.41	0.00	46.12	0.00	46.82	0.
S17	-157.77	0.00	0.00	0.00	-14.51	14.51	0.00	4.28	0.00	52.32	0.
S18	0.00	12226.04	0.00	0.00	-42.20	42.20	-38.30	0.00	0.00	152.16	0.
S19	-6817.69	0.00	0.00	0.00	-23.41	23.41	-19.31	0.00	0.00	46.82	0.
S20	-14890.69	0.00	0.00	0.00	-23.41	23.41	0.00	17.09	0.00	46.82	0.
S21	0.00	26972.47	0.00	0.00	-78.27	78.27	0.00	153.04	0.00	282.19	0.
S22	-16948.61	0.00	-2770.50	1067.21	-3178.97	7134.34	-168.33	13.85	-5195.51	17836.81	-2549.
S23	-7945.98	0.00	0.00	13.29	0.00	145.56	0.00	107.55	0.00	965.22	0.
S24	0.00	20017.77	0.00	100.69	-121.75	8.50	-44.31	0.00	-14.50	668.31	0.
S25	0.00	32531.87	0.00	100.77	-45.87	122.82	-9.33	0.00	0.00	598.01	0.
S26	0.00	19873.84	-213.72	0.00	-20.52	148.17	0.00	50.86	0.00	1244.69	0.
S27	-8237.07	0.00	-1.04	0.00	-177.48	0.00	-99.96	0.00	0.00	1348.30	0.
S28	-28074.80	0.00	-5318.80	3598.40	-180.01	139.82	0.00	34.44	-241.13	366.75	-10322.
S29	-39743.64	0.00	-4023.64	4893.56	-146.67	173.16	0.00	2.31	-65.46	497.06	-10324.
S30	-37979.17	0.00	-4415.62	4501.58	0.00	374.69	-0.87	0.00	0.00	2614.35	-5106.
S31	-39756.93	0.00	-4944.77	3972.43	-417.72	0.00	-8.47	0.00	-452.39	2641.21	-10423.
S32	-28640.06	0.00	-3590.16	5327.04	-123.74	196.09	-59.96	0.00	-434.12	287.24	-10421.
S33	-24555.12	0.00	0.00	88.12	-351.69	0.00	0.00	529.11	-1730.64	359.39	0.
S34	0.00	33608.24	0.00	0.00	-101.37	101.37	-182.52	0.00	0.00	365.48	0.
S35	-18555.10	0.00	0.00	0.00	-43.41	43.41	-198.99	0.00	0.00	86.83	0.
S36	0.00	15040.08	0.00	0.00	-42.20	42.20	0.00	42.05	0.00	152.16	0.
S37	-8342.66	0.00	0.00	0.00	-23.41	23.41	0.00	52.65	0.00	46.82	0.
S39	-329.11	0.00	0.00	0.00	-14.51	14.51	0.00	6.87	0.00	52.32	0.
S40	-8388.05	0.00	0.00	0.00	-23.41	23.41	-26.86	0.00	0.00	46.82	0.
S41	0.00	14883.94	0.00	0.00	-42.20	42.20	-65.47	0.00	0.00	152.16	0.
S42	-18527.93	0.00	0.00	0.00	-28.38	28.38	0.00	18.04	0.00	56.76	0.
S43	0.00	33785.11	0.00	0.00	-101.37	101.37	0.00	181.27	0.00	365.48	0.
S44	-24617.63	0.00	-76.52	0.00	-493.98	0.00	-583.03	0.00	-2258.71	969.71	-716.
S45	-8856.76	0.00	0.00	12.68	0.00	146.44	0.00	148.36	0.00	975.86	0.
S46	0.00	23041.80	0.00	116.06	-130.91	0.00	-48.77	0.00	-93.42	696.06	0.
S47	0.00	37193.50	0.00	86.26	-45.99	122.70	-12.24	0.00	-7.06	528.43	0.
S48	0.00	23104.04	-203.11	0.00	-10.36	158.32	0.00	51.30	0.00	1274.41	0.
S49	-8857.35	0.00	-11.91	0.00	-179.23	0.00	-132.65	0.00	0.00	1369.26	0.
S50	-33680.89	0.00	-6018.01	4155.59	-189.47	165.87	0.00	28.33	-141.65	464.54	-11174.
S51	-46984.18	0.00	-4463.75	5709.85	-153.51	201.83	-5.21	0.00	-17.62	670.22	-11176.
S52	-44628.36	0.00	-5024.64	5148.96	0.00	545.12	-7.61	0.00	0.00	4615.93	-3701.
S53	-46526.87	0.00	-5813.25	4360.35	-616.98	0.00	-9.77	0.00	-633.31	4638.40	-11664.
S54	-34337.15	0.00	-4114.99	6058.61	-130.11	225.23	-63.77	0.00	-570.75	285.84	-11661.
S55	-25069.34	0.00	0.00	110.75	-1786.40	0.00	-52.41	0.00	-6831.13	6736.64	-120.
S56	0.00	38337.30	0.00	0.00	-101.37	101.37	-236.92	0.00	0.00	365.48	0.
S57	-21162.33	0.00	0.00	0.00	-43.41	43.41	-148.38	0.00	0.00	86.83	0.
S58	0.00	17008.22	0.00	0.00	-42.20	42.20	0.00	43.91	0.00	152.16	0.
S60	-9464.27	0.00	0.00	0.00	-23.41	23.41	0.00	62.01	0.00	46.82	0.
S61	-267.61	0.00	0.00	0.00	-14.51	14.51	0.00	6.79	0.00	52.32	0.
S62	0.00	16665.82	0.00	0.00	-42.20	42.20	-69.58	0.00	0.00	152.16	0.
S63	-9385.46	0.00	0.00	0.00	-23.41	23.41	-35.57	0.00	0.00	46.82	0.
S64	-21116.39	0.00	0.00	0.00	-43.41	43.41	-23.97	0.00	0.00	86.83	0.
S65	0.00	38412.81	0.00	0.00	-101.37	101.37	0.00	221.09	0.00	365.48	0.
S66	-25078.04	0.00	-86.98	0.00	-1961.01	0.00	-145.83	0.00	-7539.61	7424.99	-650.
S67	-10170.16	0.00	0.00	15.50	0.00	147.42	0.00	142.33	0.00	987.55	0.
S68	0.00	22960.94	0.00	114.71	-151.88	0.00	-65.10	0.00	-246.95	794.19	0.
S69	0.00	36976.86	0.00	31.01	-36.71	131.97	-31.78	0.00	-139.99	479.51	0.
S70	0.00	23190.37	-161.15	0.00	-77.04	53.20	0.00	27.08	0.00	473.40	-0.
S71	-9868.42	0.00	0.00	0.00	-65.12	65.12	-100.17	0.00	0.00	195.36	0.
S72	-34974.92	0.00	-6875.79	4749.81	-226.38	128.96	0.00	19.63	-584.49	280.83	-12755.
S73	-45033.21	0.00	-5203.39	6422.21	-78.89	276.45	-4.91	0.00	-551.52	738.93	-12761.
S74	-42848.19	0.00	-5706.19	5919.41	0.00	562.00	-13.28	0.00	0.00	5174.19	-5464.

Staaf	Nx Minus	Nx Plus	Vy Minus	Vy Plus	Vz Minus	Vz Plus	Mx Minus	Mx Plus	My Minus	My Plus	Mz Minu
S75	-44800.95	0.00	-6544.41	5081.19	-717.57	0.00	-17.33	0.00	-1249.81	5229.03	-12962.
S76	-35599.11	0.00	-4733.12	6892.48	-100.24	255.10	-62.06	0.00	-929.15	169.67	-12956.
S77	-26896.21	0.00	0.00	21.88	-1392.77	0.00	-156.51	0.00	-4826.26	5592.46	0.
S78	0.00	39818.62	0.00	0.00	-101.37	101.37	-249.30	0.00	0.00	365.48	0.
S79	-22641.95	0.00	0.00	0.00	-43.41	43.41	-55.18	0.00	0.00	86.83	0.
S80	0.00	16845.04	0.00	0.00	-42.20	42.20	0.00	40.05	0.00	152.16	0.
S81	-11125.08	0.00	0.00	0.00	-23.41	23.41	0.00	83.01	0.00	46.82	0.
S83	0.00	4.96	0.00	0.00	-14.51	14.51	0.00	5.75	0.00	52.32	0.
S84	-10787.38	0.00	0.00	0.00	-23.41	23.41	-54.84	0.00	0.00	46.82	0.
S85	0.00	16574.26	0.00	0.00	-42.20	42.20	-64.99	0.00	0.00	152.16	0.
S86	-22630.63	0.00	0.00	0.00	-43.41	43.41	-284.61	0.00	0.00	86.83	0.
S87	0.00	39731.72	0.00	0.00	-101.37	101.37	0.00	152.94	0.00	365.48	0.
S88	-26729.78	0.00	-63.44	0.00	-1515.80	0.00	-65.31	0.00	-5341.86	6061.08	-532.
S89	-4837.59	0.00	0.00	15.15	-29.60	100.64	-21.02	0.00	0.00	466.60	0.
S90	0.00	11926.47	0.00	13.08	-78.84	0.00	-56.33	0.00	-175.61	349.14	0.
S91	0.00	20637.54	-0.38	0.00	-37.98	47.16	-12.79	0.00	-55.74	101.00	0.
S92	0.00	12504.56	-11.60	0.00	0.00	81.27	0.00	57.35	-115.35	438.53	0.
S93	-3858.62	0.00	-16.26	0.00	-107.15	23.09	0.00	20.30	0.00	528.95	0.
S94	-21875.60	0.00	-2458.77	1774.83	-175.05	109.27	0.00	42.00	-394.66	251.97	-4103.
S95	-31886.66	0.00	-1975.39	2258.21	-91.83	192.49	-0.94	0.00	-370.24	411.66	-4103.
S96	-30870.69	0.00	-2119.66	2113.94	-14.33	269.99	-2.62	0.00	0.00	1676.11	-2440.
S97	-30870.69	0.00	-2250.64	1982.96	-331.99	0.00	-2.62	0.00	-530.77	1747.17	-4047.
S98	-21800.92	0.00	-1779.43	2454.17	-98.14	186.18	-67.03	0.00	-528.23	203.25	-4048.
S99	-11979.78	0.00	0.00	74.36	-1909.41	0.00	0.00	813.01	-10433.77	4208.53	-128.
S100	0.00	20147.90	0.00	0.00	-51.16	51.16	-42.44	0.00	0.00	184.47	0.
S101	-11178.11	0.00	0.00	0.00	-23.41	23.41	-53.56	0.00	0.00	46.82	0.
S102	0.00	10469.40	0.00	0.00	-42.20	42.20	0.00	52.34	0.00	152.16	0.
S103	-5820.83	0.00	0.00	0.00	-23.41	23.41	0.00	90.83	0.00	46.82	0.
S105	0.00	61.42	0.00	0.00	-14.51	14.51	0.00	5.91	0.00	52.32	0.
S106	0.00	9836.05	0.00	0.00	-42.20	42.20	-78.38	0.00	0.00	152.16	0.
S107	-5501.35	0.00	0.00	0.00	-23.41	23.41	-74.51	0.00	0.00	46.82	0.
S108	-10913.45	0.00	0.00	0.00	-23.41	23.41	0.00	41.16	0.00	46.82	0.
S109	0.00	19666.10	0.00	0.00	-51.16	51.16	0.00	44.52	0.00	184.47	0.
S110	-11720.41	0.00	-80.49	0.00	-1901.68	0.00	-670.29	0.00	-10342.18	4238.27	-496.
S115	0.00	1041.23	0.00	0.00	-6.39	6.39	-0.32	0.00	0.00	22.20	0.
S117	0.00	851.05	0.00	0.00	-8.52	8.52	-3.31	0.00	0.00	29.60	0.
S118	-648.30	0.00	0.00	0.00	-8.52	8.52	-5.80	0.00	0.00	29.60	0.
S119	0.00	835.81	0.00	0.00	-8.52	8.52	0.00	3.36	0.00	29.60	0.
S120	0.00	1569.49	0.00	0.00	-6.39	6.39	0.00	0.29	0.00	22.20	0.
S125	0.00	1828.36	0.00	0.00	-6.39	6.39	0.00	0.00	0.00	22.20	0.
S127	0.00	1087.36	0.00	0.00	-8.52	8.52	-2.86	0.00	0.00	29.60	0.
S128	0.00	1101.99	0.00	0.00	-8.52	8.52	0.00	3.05	0.00	29.60	0.
S129	-966.08	0.00	0.00	0.00	-8.52	8.52	-5.76	0.00	0.00	29.60	0.
S130	0.00	2301.74	0.00	0.00	-6.39	6.39	-0.10	0.00	0.00	22.20	0.
S135	0.00	2068.32	0.00	0.00	-6.39	6.39	0.00	0.60	0.00	22.20	0.
S137	0.00	1513.43	0.00	0.00	-8.52	8.52	-2.00	0.00	0.00	29.60	0.
S138	0.00	1173.50	0.00	0.00	-8.52	8.52	0.00	2.16	0.00	29.60	0.
S139	-1034.29	0.00	-0.47	0.00	-8.42	8.63	-5.28	0.00	-14.56	15.78	0.
S140	0.00	3043.55	0.00	0.00	-8.52	8.52	-1.46	0.00	0.00	29.60	0.
S145	0.00	6100.47	0.00	0.00	-17.58	17.58	0.00	8.17	0.00	61.05	0.
S146	0.00	1509.98	0.00	0.00	-8.52	8.52	-0.70	0.00	0.00	29.60	0.
S148	-1240.33	0.00	0.00	0.00	-8.52	8.52	-3.74	0.00	0.00	29.60	0.
S149	0.00	1030.34	0.00	0.00	-8.52	8.52	0.00	0.94	0.00	29.60	0.
S150	0.00	6491.24	0.00	0.00	-17.58	17.58	-8.65	0.00	0.00	61.05	0.
S151	-6534.23	0.00	-13143.61	16736.60	-4204.05	3110.51	-780.24	0.00	-4201.59	4399.77	-10735.
S152	-5898.51	0.00	-13583.94	11424.79	-4304.60	3653.82	-460.62	85.83	-2296.17	5295.34	-7642.
S153	-3827.19	0.00	-13912.81	11498.08	-4584.49	4441.41	-255.17	305.30	-2412.01	5437.24	-7184.
S154	-2148.53	0.00	-11994.93	12998.90	-4763.23	5563.79	-466.55	107.28	-5775.04	5593.75	-9094.
S155	0.00	92.37	0.00	0.00	0.00	0.00	-0.17	0.00	0.00	0.00	0.
S156	0.00	233.60	0.00	0.00	0.00	0.00	-0.10	0.00	0.00	0.00	0.
S157	0.00	364.77	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.
S158	-98.82	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.
S159	0.00	297.45	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.
S160	0.00	610.88	0.00	0.00	0.00	0.00	-0.02	0.00	0.00	0.00	0.
S161	0.00	933.36	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.
S162	-194.88	0.00	0.00	0.00	0.00	0.00	0.00	0.16	0.00	0.00	0.

Staaf	Nx Minus	Nx Plus	Vy Minus	Vy Plus	Vz Minus	Vz Plus	Mx Minus	Mx Plus	My Minus	My Plus	Mz Minu
S163	0.00	278.55	0.00	0.00	0.00	0.00	-0.03	0.00	0.00	0.00	0.
S164	0.00	555.97	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.
S165	0.00	843.67	0.00	0.00	0.00	0.00	-0.15	0.00	0.00	0.00	0.
S166	-202.63	0.00	0.00	0.00	0.00	0.00	-0.09	0.00	0.00	0.00	0.
S167	0.00	63.51	0.00	0.00	0.00	0.00	0.00	0.24	0.00	0.00	0.
S168	0.00	169.80	0.00	0.00	0.00	0.00	0.00	0.07	0.00	0.00	0.
S169	0.00	279.22	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.
S170	-89.88	0.00	0.00	0.00	0.00	0.00	-0.04	0.00	0.00	0.00	0.
S171	-6955.35	0.00	-13260.22	16969.26	-3236.21	4382.24	0.00	691.72	-4634.66	4357.73	-10766.
S172	-6145.41	0.00	-13713.42	11358.45	-3778.31	4390.95	-24.37	511.35	-5428.94	2322.84	-7709.
S173	-3866.71	0.00	-13941.68	11376.53	-4379.42	4632.13	-139.42	429.88	-5993.46	2172.92	-7164.
S174	-2147.31	0.00	-11736.67	12788.10	-5172.85	3778.13	-102.27	496.60	-5928.15	4554.27	-8875.
S175	-2307.32	0.00	0.00	1050.17	0.00	372.37	0.00	95.90	-2541.63	65.00	-4673.
S176	-1568.31	0.00	-1264.05	0.00	-324.64	0.00	-65.41	0.00	-2113.76	158.69	-5150.
S177	-1841.37	0.00	-912.09	0.00	-7.63	0.00	0.00	54.70	-25.14	28.30	-3207.
S178	-1636.78	0.00	-971.09	0.00	0.00	74.71	-132.44	0.00	-789.47	0.00	-2602.
S179	-65.42	0.00	0.00	0.00	0.00	0.00	-0.11	0.00	0.00	0.00	0.
S180	-99.17	0.00	0.00	0.00	0.00	0.00	-0.16	0.00	0.00	0.00	0.
S181	-96.62	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.
S182	-1783.31	0.00	0.00	0.00	0.00	0.00	0.00	0.84	0.00	0.00	0.
S183	0.00	67.18	0.00	0.00	0.00	0.00	-0.16	0.00	0.00	0.00	0.
S184	-223.34	0.00	0.00	0.00	0.00	0.00	-0.01	0.00	0.00	0.00	0.
S185	-708.13	0.00	0.00	0.00	0.00	0.00	-0.02	0.00	0.00	0.00	0.
S186	-1283.21	0.00	0.00	0.00	0.00	0.00	0.00	0.18	0.00	0.00	0.
S187	-158.02	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.00	0.00	0.
S188	-332.85	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.
S189	-808.09	0.00	0.00	0.00	0.00	0.00	-0.03	0.00	0.00	0.00	0.
S190	-1224.77	0.00	0.00	0.00	0.00	0.00	-0.13	0.00	0.00	0.00	0.
S191	-131.90	0.00	0.00	0.00	0.00	0.00	0.00	0.15	0.00	0.00	0.
S192	-166.25	0.00	0.00	0.00	0.00	0.00	0.00	0.15	0.00	0.00	0.
S193	-193.72	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.
S194	-1778.74	0.00	0.00	0.00	0.00	0.00	-1.11	0.00	0.00	0.00	0.
S195	-2978.26	0.00	0.00	931.97	-369.84	0.00	0.00	68.91	-83.43	2505.46	-4399.
S196	-2131.45	0.00	-1354.89	0.00	0.00	376.96	0.00	172.95	-615.75	2022.98	-5460.
S197	-2376.34	0.00	-1010.35	0.00	-100.80	0.00	0.00	127.82	-638.94	66.68	-3521.
S198	-1830.28	0.00	-967.81	0.00	-79.91	0.00	0.00	151.71	0.00	645.59	-2573.
S200	0.00	870.32	0.00	0.00	-1.86	1.86	0.00	0.14	0.00	4.95	0.
S201	0.00	2260.47	0.00	0.00	-5.68	5.68	0.00	2.30	0.00	15.10	0.
S202	0.00	1863.55	0.00	0.00	-5.68	5.68	-1.17	0.00	0.00	15.10	0.
S203	0.00	1001.65	0.00	0.00	-1.86	1.86	-0.29	0.00	0.00	4.95	0.
-	kN	kN	kN	kN	kN	kN	kNm	kNm	kNm	kNm	kN

Fu.C. Extreme oplegreacties (Momenten)

Oplegging	Knoop	B.C.	Xmax	Y	Z	B.C.	Ymax	X	Z	B.C.	Zmax	X	Y
Globale extreme waarden													
-	-	-	kN	kN	kN	-	kN	kN	kN	-	kN	kN	kN



Afb. Fu.C. Tegendruk Omhullende

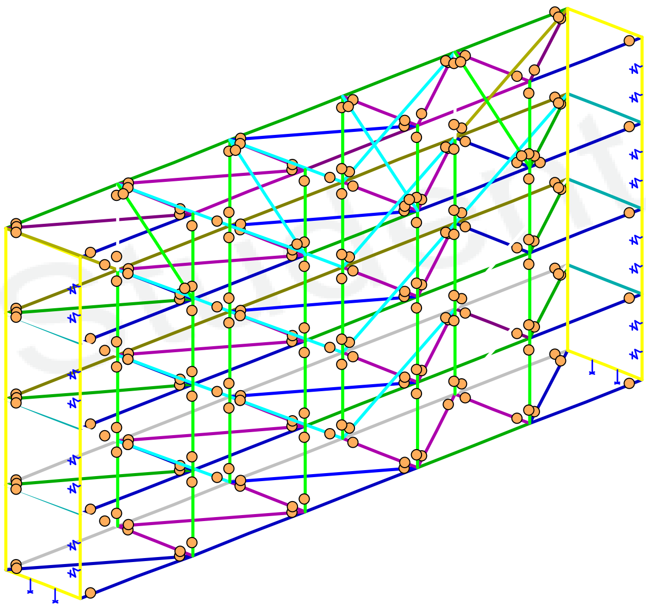
Fu.C. Bodemdruk

Staaf	B.C.	Coordinaat	Cy	Bodemdruk Y	Mx	Tegendruk Y / breedte	Cz	Bodemdruk Z	Breedte	Tegendruk Z / breedte
S11	Fu.C.1	0.000	-100000.00	2086.70	1.00	2,086.70	-100000.00	-1630.76	1.00	-1,630.76
		0.029	-100000.00	2071.41	1.00	2,071.41	-100000.00	-1624.57	1.00	-1,624.57
		0.800	-100000.00	1670.00	1.00	1,670.00	-100000.00	-1472.12	1.00	-1,472.12
		1.600	-100000.00	1270.45	1.00	1,270.45	-100000.00	-1342.83	1.00	-1,342.83
		1.750	-100000.00	1198.32	1.00	1,198.32	-100000.00	-1322.29	1.00	-1,322.29
		2.200	-100000.00	987.88	1.00	987.88	-100000.00	-1267.80	1.00	-1,267.80
		2.400	-100000.00	897.25	1.00	897.25	-100000.00	-1246.97	1.00	-1,246.97
		3.200	-100000.00	552.13	1.00	552.13	-100000.00	-1182.74	1.00	-1,182.74
		4.000	-100000.00	231.50	1.00	231.50	-100000.00	-1142.93	1.00	-1,142.93
		4.595	-100000.00	4.91	1.00	4.91	-100000.00	-1122.00	1.00	-1,122.00
		4.600	-100000.00	3.04	1.00	3.04	-100000.00	-1121.83	1.00	-1,121.83
		4.800	-100000.00	-71.50	1.00	-71.50	-100000.00	-1115.15	1.00	-1,115.15
		5.600	-100000.00	-365.09	1.00	-365.09	-100000.00	-1082.05	1.00	-1,082.05
		5.761	-100000.00	-423.85	1.00	-423.85	-100000.00	-1072.84	1.00	-1,072.84
		6.400	-100000.00	-657.11	1.00	-657.11	-100000.00	-1021.46	1.00	-1,021.46
		7.200	-100000.00	-953.22	1.00	-953.22	-100000.00	-906.53	1.00	-906.53
		8.000	-100000.00	-1255.24	1.00	-1,255.24	-100000.00	-706.01	1.00	-706.01
S22		0.000	-100000.00	-1842.97	1.00	-1,842.97	-100000.00	-1845.42	1.00	-1,845.42
		0.800	-100000.00	-1465.23	1.00	-1,465.23	-100000.00	-1667.55	1.00	-1,667.55
		1.600	-100000.00	-1102.72	1.00	-1,102.72	-100000.00	-1517.88	1.00	-1,517.88
		1.950	-100000.00	-950.93	1.00	-950.93	-100000.00	-1463.15	1.00	-1,463.15
		2.000	-100000.00	-929.61	1.00	-929.61	-100000.00	-1455.89	1.00	-1,455.89
		2.400	-100000.00	-762.32	1.00	-762.32	-100000.00	-1402.84	1.00	-1,402.84
		3.200	-100000.00	-444.38	1.00	-444.38	-100000.00	-1322.20	1.00	-1,322.20
		4.000	-100000.00	-144.77	1.00	-144.77	-100000.00	-1269.61	1.00	-1,269.61
		4.400	-100000.00	0.22	1.00	0.22	-100000.00	-1250.26	1.00	-1,250.26
		4.749	-100000.00	125.20	1.00	125.20	-100000.00	-1235.11	1.00	-1,235.11
		4.800	-100000.00	143.28	1.00	143.28	-100000.00	-1232.95	1.00	-1,232.95
		5.221	-100000.00	292.83	1.00	292.83	-100000.00	-1214.29	1.00	-1,214.29
		5.600	-100000.00	427.40	1.00	427.40	-100000.00	-1194.56	1.00	-1,194.56
		6.400	-100000.00	714.39	1.00	714.39	-100000.00	-1131.43	1.00	-1,131.43
		7.200	-100000.00	1008.52	1.00	1,008.52	-100000.00	-1015.38	1.00	-1,015.38
		8.000	-100000.00	1309.85	1.00	1,309.85	-100000.00	-813.37	1.00	-813.37

Staaf	B.C.	Coordinaat	Cy	Bodemdruk Y	Mx	Tegendruk Y / breedte	Cz	Bodemdruk Z	Breedte	Tegendruk Z / breedte
S151	Fu.C.1	0.000	-100000.00	-3775.11	1.00	-3,775.11	-100000.00	-1078.41	1.00	-1,078.41
		0.700	-100000.00	-3802.27	1.00	-3,802.27	-100000.00	-1055.23	1.00	-1,055.23
		1.075	-100000.00	-3802.59	1.00	-3,802.59	-100000.00	-1038.18	1.00	-1,038.18
		1.242	-100000.00	-3802.31	1.00	-3,802.31	-100000.00	-1030.25	1.00	-1,030.25
		1.400	-100000.00	-3802.56	1.00	-3,802.56	-100000.00	-1022.74	1.00	-1,022.74
		2.100	-100000.00	-3821.05	1.00	-3,821.05	-100000.00	-992.38	1.00	-992.38
		2.800	-100000.00	-3889.72	1.00	-3,889.72	-100000.00	-972.92	1.00	-972.92
		3.450	-100000.00	-4015.03	1.00	-4,015.03	-100000.00	-970.10	1.00	-970.10
		3.500	-100000.00	-4027.34	1.00	-4,027.34	-100000.00	-970.59	1.00	-970.59
		3.700	-100000.00	-4080.41	1.00	-4,080.41	-100000.00	-973.65	1.00	-973.65
		4.150	-100000.00	-4221.74	1.00	-4,221.74	-100000.00	-987.02	1.00	-987.02
		4.200	-100000.00	-4239.23	1.00	-4,239.23	-100000.00	-989.07	1.00	-989.07
		4.900	-100000.00	-4516.81	1.00	-4,516.81	-100000.00	-1029.54	1.00	-1,029.54
		5.600	-100000.00	-4836.83	1.00	-4,836.83	-100000.00	-1090.57	1.00	-1,090.57
		5.673	-100000.00	-4871.24	1.00	-4,871.24	-100000.00	-1098.00	1.00	-1,098.00
		6.300	-100000.00	-5160.45	1.00	-5,160.45	-100000.00	-1168.11	1.00	-1,168.11
		6.996	-100000.00	-5430.81	1.00	-5,430.81	-100000.00	-1254.71	1.00	-1,254.71
		7.000	-100000.00	-5432.17	1.00	-5,432.17	-100000.00	-1255.24	1.00	-1,255.24
S152		0.000	-100000.00	-3752.33	1.00	-3,752.33	-100000.00	-1276.98	1.00	-1,276.98
		0.300	-100000.00	-3754.82	1.00	-3,754.82	-100000.00	-1268.32	1.00	-1,268.32
		0.700	-100000.00	-3731.33	1.00	-3,731.33	-100000.00	-1250.84	1.00	-1,250.84
		1.400	-100000.00	-3645.74	1.00	-3,645.74	-100000.00	-1210.73	1.00	-1,210.73
		1.607	-100000.00	-3615.33	1.00	-3,615.33	-100000.00	-1197.88	1.00	-1,197.88
		1.689	-100000.00	-3603.14	1.00	-3,603.14	-100000.00	-1192.79	1.00	-1,192.79
		2.100	-100000.00	-3543.14	1.00	-3,543.14	-100000.00	-1167.70	1.00	-1,167.70
		2.800	-100000.00	-3458.57	1.00	-3,458.57	-100000.00	-1129.66	1.00	-1,129.66
		3.500	-100000.00	-3414.87	1.00	-3,414.87	-100000.00	-1101.51	1.00	-1,101.51
		3.650	-100000.00	-3412.08	1.00	-3,412.08	-100000.00	-1097.01	1.00	-1,097.01
		3.800	-100000.00	-3411.75	1.00	-3,411.75	-100000.00	-1093.06	1.00	-1,093.06
		4.200	-100000.00	-3422.89	1.00	-3,422.89	-100000.00	-1085.20	1.00	-1,085.20
		4.900	-100000.00	-3481.71	1.00	-3,481.71	-100000.00	-1079.85	1.00	-1,079.85
		5.600	-100000.00	-3578.60	1.00	-3,578.60	-100000.00	-1081.73	1.00	-1,081.73
		5.684	-100000.00	-3591.74	1.00	-3,591.74	-100000.00	-1082.16	1.00	-1,082.16
		5.899	-100000.00	-3625.92	1.00	-3,625.92	-100000.00	-1083.24	1.00	-1,083.24
		6.300	-100000.00	-3688.77	1.00	-3,688.77	-100000.00	-1084.35	1.00	-1,084.35
		7.000	-100000.00	-3775.11	1.00	-3,775.11	-100000.00	-1078.41	1.00	-1,078.41
S153		0.000	-100000.00	-3825.25	1.00	-3,825.25	-100000.00	-1377.95	1.00	-1,377.95
		0.250	-100000.00	-3833.25	1.00	-3,833.25	-100000.00	-1374.29	1.00	-1,374.29
		0.350	-100000.00	-3832.42	1.00	-3,832.42	-100000.00	-1371.97	1.00	-1,371.97
		0.700	-100000.00	-3814.73	1.00	-3,814.73	-100000.00	-1360.81	1.00	-1,360.81
		1.400	-100000.00	-3733.28	1.00	-3,733.28	-100000.00	-1329.83	1.00	-1,329.83
		1.538	-100000.00	-3713.21	1.00	-3,713.21	-100000.00	-1323.13	1.00	-1,323.13
		1.780	-100000.00	-3677.16	1.00	-3,677.16	-100000.00	-1311.48	1.00	-1,311.48
		2.100	-100000.00	-3629.67	1.00	-3,629.67	-100000.00	-1296.66	1.00	-1,296.66
		2.800	-100000.00	-3539.87	1.00	-3,539.87	-100000.00	-1269.54	1.00	-1,269.54
		3.500	-100000.00	-3487.28	1.00	-3,487.28	-100000.00	-1253.32	1.00	-1,253.32
		3.800	-100000.00	-3479.34	1.00	-3,479.34	-100000.00	-1250.21	1.00	-1,250.21
		4.200	-100000.00	-3483.11	1.00	-3,483.11	-100000.00	-1249.56	1.00	-1,249.56
		4.900	-100000.00	-3526.52	1.00	-3,526.52	-100000.00	-1256.60	1.00	-1,256.60
		5.450	-100000.00	-3586.04	1.00	-3,586.04	-100000.00	-1266.61	1.00	-1,266.61
		5.600	-100000.00	-3604.63	1.00	-3,604.63	-100000.00	-1269.51	1.00	-1,269.51
		5.825	-100000.00	-3633.36	1.00	-3,633.36	-100000.00	-1273.66	1.00	-1,273.66
		6.300	-100000.00	-3692.39	1.00	-3,692.39	-100000.00	-1280.12	1.00	-1,280.12
		7.000	-100000.00	-3752.33	1.00	-3,752.33	-100000.00	-1276.98	1.00	-1,276.98
S154		0.000	-100000.00	-3886.51	1.00	-3,886.51	-100000.00	-1909.37	1.00	-1,909.37
		0.177	-100000.00	-3859.28	1.00	-3,859.28	-100000.00	-1872.29	1.00	-1,872.29
		0.700	-100000.00	-3760.18	1.00	-3,760.18	-100000.00	-1763.70	1.00	-1,763.70
		1.046	-100000.00	-3686.73	1.00	-3,686.73	-100000.00	-1694.78	1.00	-1,694.78
		1.400	-100000.00	-3611.65	1.00	-3,611.65	-100000.00	-1628.57	1.00	-1,628.57
		2.100	-100000.00	-3480.45	1.00	-3,480.45	-100000.00	-1514.63	1.00	-1,514.63
		2.800	-100000.00	-3393.63	1.00	-3,393.63	-100000.00	-1428.33	1.00	-1,428.33
		2.900	-100000.00	-3385.87	1.00	-3,385.87	-100000.00	-1418.45	1.00	-1,418.45
		3.350	-100000.00	-3366.75	1.00	-3,366.75	-100000.00	-1381.67	1.00	-1,381.67

Staaf	B.C.	Coordinaat	Cy	Bodemdruk Y	Mx	Tegendruk Y / breedte	Cz	Bodemdruk Z	Breedte	Tegendruk Z / breedte
S154	Fu.C.1	3.500	-100000.00	-3366.23	1.00	-3,366.23	-100000.00	-1372.17	1.00	-1,372.17
		3.600	-100000.00	-3367.50	1.00	-3,367.50	-100000.00	-1366.58	1.00	-1,366.58
		4.200	-100000.00	-3401.55	1.00	-3,401.55	-100000.00	-1344.96	1.00	-1,344.96
		4.900	-100000.00	-3491.26	1.00	-3,491.26	-100000.00	-1341.93	1.00	-1,341.93
		5.300	-100000.00	-3559.72	1.00	-3,559.72	-100000.00	-1348.02	1.00	-1,348.02
		5.600	-100000.00	-3615.29	1.00	-3,615.29	-100000.00	-1354.83	1.00	-1,354.83
		5.658	-100000.00	-3626.16	1.00	-3,626.16	-100000.00	-1356.27	1.00	-1,356.27
		5.823	-100000.00	-3657.20	1.00	-3,657.20	-100000.00	-1360.48	1.00	-1,360.48
		6.300	-100000.00	-3741.48	1.00	-3,741.48	-100000.00	-1371.91	1.00	-1,371.91
		7.000	-100000.00	-3825.25	1.00	-3,825.25	-100000.00	-1377.95	1.00	-1,377.95
S171		0.000	-100000.00	-3796.24	1.00	-3,796.24	-100000.00	1118.54	1.00	1,118.54
		0.050	-100000.00	-3800.74	1.00	-3,800.74	-100000.00	1117.55	1.00	1,117.55
		0.700	-100000.00	-3829.95	1.00	-3,829.95	-100000.00	1095.90	1.00	1,095.90
		1.083	-100000.00	-3833.48	1.00	-3,833.48	-100000.00	1078.46	1.00	1,078.46
		1.257	-100000.00	-3834.61	1.00	-3,834.61	-100000.00	1070.15	1.00	1,070.15
		1.400	-100000.00	-3836.01	1.00	-3,836.01	-100000.00	1063.30	1.00	1,063.30
		2.100	-100000.00	-3859.93	1.00	-3,859.93	-100000.00	1032.69	1.00	1,032.69
		2.800	-100000.00	-3934.02	1.00	-3,934.02	-100000.00	1013.24	1.00	1,013.24
		3.400	-100000.00	-4052.18	1.00	-4,052.18	-100000.00	1010.46	1.00	1,010.46
		3.500	-100000.00	-4077.22	1.00	-4,077.22	-100000.00	1011.47	1.00	1,011.47
		4.200	-100000.00	-4294.90	1.00	-4,294.90	-100000.00	1031.24	1.00	1,031.24
		4.900	-100000.00	-4578.33	1.00	-4,578.33	-100000.00	1073.80	1.00	1,073.80
		5.600	-100000.00	-4903.94	1.00	-4,903.94	-100000.00	1137.73	1.00	1,137.73
		5.661	-100000.00	-4933.11	1.00	-4,933.11	-100000.00	1144.20	1.00	1,144.20
		6.300	-100000.00	-5232.36	1.00	-5,232.36	-100000.00	1218.81	1.00	1,218.81
		6.996	-100000.00	-5505.87	1.00	-5,505.87	-100000.00	1309.28	1.00	1,309.28
		7.000	-100000.00	-5507.29	1.00	-5,507.29	-100000.00	1309.85	1.00	1,309.85
S172		0.000	-100000.00	-3752.10	1.00	-3,752.10	-100000.00	1296.22	1.00	1,296.22
		0.050	-100000.00	-3754.37	1.00	-3,754.37	-100000.00	1295.31	1.00	1,295.31
		0.300	-100000.00	-3756.87	1.00	-3,756.87	-100000.00	1288.74	1.00	1,288.74
		0.700	-100000.00	-3735.73	1.00	-3,735.73	-100000.00	1272.64	1.00	1,272.64
		1.400	-100000.00	-3652.68	1.00	-3,652.68	-100000.00	1234.68	1.00	1,234.68
		1.617	-100000.00	-3621.33	1.00	-3,621.33	-100000.00	1221.86	1.00	1,221.86
		1.712	-100000.00	-3607.25	1.00	-3,607.25	-100000.00	1216.18	1.00	1,216.18
		2.100	-100000.00	-3551.16	1.00	-3,551.16	-100000.00	1193.61	1.00	1,193.61
		2.800	-100000.00	-3466.81	1.00	-3,466.81	-100000.00	1157.53	1.00	1,157.53
		3.500	-100000.00	-3423.02	1.00	-3,423.02	-100000.00	1131.44	1.00	1,131.44
		3.600	-100000.00	-3420.88	1.00	-3,420.88	-100000.00	1128.68	1.00	1,128.68
		3.850	-100000.00	-3420.36	1.00	-3,420.36	-100000.00	1122.88	1.00	1,122.88
		4.200	-100000.00	-3431.19	1.00	-3,431.19	-100000.00	1117.33	1.00	1,117.33
		4.900	-100000.00	-3490.92	1.00	-3,490.92	-100000.00	1114.26	1.00	1,114.26
		5.600	-100000.00	-3589.95	1.00	-3,589.95	-100000.00	1118.40	1.00	1,118.40
		5.662	-100000.00	-3599.85	1.00	-3,599.85	-100000.00	1118.91	1.00	1,118.91
		5.935	-100000.00	-3644.67	1.00	-3,644.67	-100000.00	1121.08	1.00	1,121.08
		6.300	-100000.00	-3703.96	1.00	-3,703.96	-100000.00	1123.03	1.00	1,123.03
		7.000	-100000.00	-3796.24	1.00	-3,796.24	-100000.00	1118.54	1.00	1,118.54
S173		0.000	-100000.00	-3795.02	1.00	-3,795.02	-100000.00	1333.08	1.00	1,333.08
		0.400	-100000.00	-3805.00	1.00	-3,805.00	-100000.00	1334.35	1.00	1,334.35
		0.700	-100000.00	-3791.09	1.00	-3,791.09	-100000.00	1330.32	1.00	1,330.32
		0.750	-100000.00	-3787.37	1.00	-3,787.37	-100000.00	1329.33	1.00	1,329.33
		1.400	-100000.00	-3714.45	1.00	-3,714.45	-100000.00	1311.03	1.00	1,311.03
		1.717	-100000.00	-3669.47	1.00	-3,669.47	-100000.00	1300.19	1.00	1,300.19
		1.807	-100000.00	-3656.47	1.00	-3,656.47	-100000.00	1297.09	1.00	1,297.09
		2.100	-100000.00	-3614.15	1.00	-3,614.15	-100000.00	1287.15	1.00	1,287.15
		2.800	-100000.00	-3526.48	1.00	-3,526.48	-100000.00	1267.27	1.00	1,267.27
		3.500	-100000.00	-3475.26	1.00	-3,475.26	-100000.00	1256.63	1.00	1,256.63
		3.550	-100000.00	-3473.37	1.00	-3,473.37	-100000.00	1256.29	1.00	1,256.29
		3.850	-100000.00	-3467.41	1.00	-3,467.41	-100000.00	1255.51	1.00	1,255.51
		4.200	-100000.00	-3472.11	1.00	-3,472.11	-100000.00	1257.17	1.00	1,257.17
		4.900	-100000.00	-3516.67	1.00	-3,516.67	-100000.00	1267.61	1.00	1,267.61
		5.428	-100000.00	-3574.78	1.00	-3,574.78	-100000.00	1279.37	1.00	1,279.37
		5.600	-100000.00	-3596.59	1.00	-3,596.59	-100000.00	1283.38	1.00	1,283.38
		5.853	-100000.00	-3629.83	1.00	-3,629.83	-100000.00	1288.97	1.00	1,288.97

Staaf	B.C.	Coordinaat	Cy	Bodemdruk Y	Mx	Tegendruk Y / breedte	Cz	Bodemdruk Z	Breedte	Tegendruk Z / breedte
S173	Fu.C.1	6.300	-100000.00	-3687.37	1.00	-3,687.37	-100000.00	1296.64	1.00	1,296.64
		7.000	-100000.00	-3752.10	1.00	-3,752.10	-100000.00	1296.22	1.00	1,296.22
S174		0.000	-100000.00	-3767.03	1.00	-3,767.03	-100000.00	1472.23	1.00	1,472.23
		0.184	-100000.00	-3742.14	1.00	-3,742.14	-100000.00	1449.79	1.00	1,449.79
		0.700	-100000.00	-3654.09	1.00	-3,654.09	-100000.00	1387.63	1.00	1,387.63
		1.061	-100000.00	-3584.28	1.00	-3,584.28	-100000.00	1346.60	1.00	1,346.60
		1.400	-100000.00	-3518.58	1.00	-3,518.58	-100000.00	1311.29	1.00	1,311.29
		2.100	-100000.00	-3399.28	1.00	-3,399.28	-100000.00	1251.73	1.00	1,251.73
		2.800	-100000.00	-3322.88	1.00	-3,322.88	-100000.00	1214.09	1.00	1,214.09
		2.850	-100000.00	-3319.53	1.00	-3,319.53	-100000.00	1212.31	1.00	1,212.31
		3.350	-100000.00	-3303.05	1.00	-3,303.05	-100000.00	1201.21	1.00	1,201.21
		3.500	-100000.00	-3304.28	1.00	-3,304.28	-100000.00	1200.22	1.00	1,200.22
		4.050	-100000.00	-3332.87	1.00	-3,332.87	-100000.00	1205.29	1.00	1,205.29
		4.200	-100000.00	-3346.95	1.00	-3,346.95	-100000.00	1208.86	1.00	1,208.86
		4.900	-100000.00	-3442.88	1.00	-3,442.88	-100000.00	1235.61	1.00	1,235.61
		5.600	-100000.00	-3572.55	1.00	-3,572.55	-100000.00	1272.93	1.00	1,272.93
		5.607	-100000.00	-3573.83	1.00	-3,573.83	-100000.00	1273.30	1.00	1,273.30
		5.661	-100000.00	-3584.51	1.00	-3,584.51	-100000.00	1276.36	1.00	1,276.36
		6.300	-100000.00	-3704.51	1.00	-3,704.51	-100000.00	1310.10	1.00	1,310.10
		7.000	-100000.00	-3795.02	1.00	-3,795.02	-100000.00	1333.08	1.00	1,333.08
-	**-**	**m**	**kN/m3* (m)**	**kN/m**	**m**	**kN/m2**	**kN/m3* (m)**	**kN/m**	**m**	**kN/m2**



Afb. Staaldefinitie

Samenstelling constructiedelen

Cdl	Staaf/staven
C1	s1
C2	s2
C3	s3
C4	s4
C5	s5
C6	s6
C7	s7
C8	s8
C9	s9

C10	s10
C11	s11
C12	s12
C13	s13
C14	s14
C16	s16
C17	s17
C18	s18
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C195	s195
C196	s196
C197	s197
C198	s198
C200	s200
C201	s201
C202	s202
C203	s203

UC's per constructiedeel

Staalcontrole volgens NEN6770/6771

Label	Toetsing	Combinatie	Formule	Max Unity Check
C1	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.08
C2	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.34
C3	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.53
C4	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.26
C5	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.10
C6	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.25
C7	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.34
C8	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.33
C9	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.35
C10	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.26
C11	Doorsnede	Fu.C.1	NEN6770(11.3-31)	0.43
C12	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.50
C13	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.52
C14	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.42
C16	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.24
C17	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.05
C18	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.43
C19	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.24
C20	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.52
C21	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.51
C22	Doorsnede	Fu.C.1	NEN6770(11.3-31)	0.44

Label	Toetsing	Combinatie	Formule	Max Unity Check
C23	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.15
C24	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.38
C25	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.47
C26	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.29
C27	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.16
C28	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.34
C29	Doorsnede	Fu.C.1	NEN6770(11.3-17)	0.55
C30	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.46
C31	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.48
C32	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.34
C33	Doorsnede	Fu.C.1	NEN6770(11.3-4)	0.27
C34	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.49
C35	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.35
C36	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.53
C37	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.29
C39	Doorsnede	Fu.C.1	NEN6770(11.3-17)	0.05
C40	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.29
C41	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.52
C42	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.53
C43	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.49
C44	Doorsnede	Fu.C.1	NEN6770(11.3-4)	0.28
C45	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.17
C46	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.43
C47	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.54
C48	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.34
C49	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.17
C50	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.35
C51	Doorsnede	Fu.C.1	NEN6770(11.3-17)	0.53
C52	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.46
C53	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.48
C54	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.35
C55	Doorsnede	Fu.C.1	NEN6770(11.3-4)	0.37
C56	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.56
C57	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.40
C58	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.60
C60	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.33
C61	Doorsnede	Fu.C.1	NEN6770(11.3-17)	0.05
C62	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.58
C63	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.33
C64	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.40
C65	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.56
C66	Doorsnede	Fu.C.1	NEN6770(11.3-4)	0.38
C67	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.19
C68	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.43
C69	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.54
C70	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.44
C71	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.19
C72	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.36
C73	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.46
C74	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.44
C75	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.46
C76	Doorsnede	Fu.C.1	NEN6770(11.3-17)	0.41
C77	Doorsnede	Fu.C.1	NEN6770(11.3-4)	0.35
C78	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.58
C79	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.43
C80	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.59
C81	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.39
C83	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.05
C84	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.38
C85	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.58
C86	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.43
C87	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.58
C88	Doorsnede	Fu.C.1	NEN6770(11.3-4)	0.35
C89	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.09
C90	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.42
C91	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.60
C92	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.44
C93	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.07

Label	Toetsing	Combinatie	Formule	Max Unity Check
C94	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.32
C95	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.46
C96	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.45
C97	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.45
C98	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.32
C99	Doorsnede	Fu.C.1	NEN6770(11.3-31)	0.26
C100	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.58
C101	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.39
C102	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.37
C103	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.20
C105	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.05
C106	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.34
C107	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.19
C108	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.38
C109	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.57
C110	Doorsnede	Fu.C.1	NEN6770(11.3-31)	0.26
C115	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.20
C117	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.12
C118	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.09
C119	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.12
C120	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.30
C125	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.35
C127	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.16
C128	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.16
C129	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.14
C130	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.44
C135	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.40
C137	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.22
C138	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.17
C139	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.15
C140	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.44
C145	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.43
C146	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.22
C148	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.18
C149	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.15
C150	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.45
C151	Doorsnede	Fu.C.1	NEN6770(11.2-10)	0.66
C152	Doorsnede	Fu.C.1	NEN6770(11.2-10)	0.54
C153	Doorsnede	Fu.C.1	NEN6770(11.2-10)	0.55
C154	Doorsnede	Fu.C.1	NEN6770(11.2-10)	0.51
C155	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.04
C156	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.10
C157	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.16
C158	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.04
C159	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.13
C160	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.27
C161	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.41
C162	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.09
C163	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.12
C164	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.24
C165	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.37
C166	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.09
C167	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.03
C168	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.07
C169	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.12
C170	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.04
C171	Doorsnede	Fu.C.1	NEN6770(11.2-10)	0.67
C172	Doorsnede	Fu.C.1	NEN6770(11.2-10)	0.54
C173	Doorsnede	Fu.C.1	NEN6770(11.2-10)	0.55
C174	Doorsnede	Fu.C.1	NEN6770(11.2-10)	0.50
C175	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.13
C176	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.14
C177	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.09
C178	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.12
C179	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.03
C180	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.04
C181	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.04
C182	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.34

Label	Toetsing	Combinatie	Formule	Max Unity Check
C183	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.03
C184	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.10
C185	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.31
C186	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.56
C187	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.07
C188	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.15
C189	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.36
C190	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.54
C191	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.06
C192	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.07
C193	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.09
C194	Doorsnede	Fu.C.1	NEN6770(11.2-3)	0.34
C195	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.12
C196	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.15
C197	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.10
C198	Doorsnede	Fu.C.1	NEN6770(11.2-5)	0.12
C200	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.38
C201	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.33
C202	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.27
C203	Doorsnede	Fu.C.1	NEN6770(11.2-1)	0.44

Gewicht staalconstructie

C25-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C26-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C34-V1 (0.000-14.422)	B1000x50	Lsys = 14.422 m	Massa = 16,894.479 kg
C43-V1 (0.000-14.422)	B1000x50	Lsys = 14.422 m	Massa = 16,894.479 kg
C47-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C48-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C4-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C56-V1 (0.000-14.422)	B1000x50	Lsys = 14.422 m	Massa = 16,894.479 kg
C65-V1 (0.000-14.422)	B1000x50	Lsys = 14.422 m	Massa = 16,894.479 kg
C69-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C78-V1 (0.000-14.422)	B1000x50	Lsys = 14.422 m	Massa = 16,894.479 kg
C87-V1 (0.000-14.422)	B1000x50	Lsys = 14.422 m	Massa = 16,894.479 kg
C94-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C95-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C96-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C97-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
C98-V1 (0.000-12.000)	B1000x50	Lsys = 12.000 m	Massa = 14,057.056 kg
Subtotaal:	**B1000x50**	**218.533 m**	**Massa = 255,994.496**
C10-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
C28-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
C29-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
C30-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
C31-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
C32-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
C6-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
C7-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
C8-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
C9-V1 (0.000-12.000)	B1200x50	Lsys = 12.000 m	Massa = 17,016.438 kg
Subtotaal:	**B1200x50**	**120.000 m**	**Massa = 170,164.379**
C155-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C156-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C157-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C158-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C159-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C160-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C161-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C162-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C163-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C164-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C165-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C166-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg

C167-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C168-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C169-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C170-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C179-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C180-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C181-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C183-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C184-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C185-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C186-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C187-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C188-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C189-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C190-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C191-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C192-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C193-V1 (0.000-7.000)	B120x15	Lsys = 7.000 m	Massa = 271.893 kg
C200-V1 (0.000-10.630)	B120x15	Lsys = 10.630 m	Massa = 412.895 kg
C203-V1 (0.000-10.630)	B120x15	Lsys = 10.630 m	Massa = 412.895 kg
Subtotaal:	**B120x15**	**231.260 m**	**Massa = 8,982.582**
C50-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
C51-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
C52-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
C53-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
C54-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
C72-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
C73-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
C74-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
C75-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
C76-V1 (0.000-12.000)	B1400x50	Lsys = 12.000 m	Massa = 19,975.817 kg
Subtotaal:	**B1400x50**	**120.000 m**	**Massa = 199,758.166**
C115-V1 (0.000-13.892)	B200x20	Lsys = 13.892 m	Massa = 1,233.391 kg
C120-V1 (0.000-13.892)	B200x20	Lsys = 13.892 m	Massa = 1,233.391 kg
C125-V1 (0.000-13.892)	B200x20	Lsys = 13.892 m	Massa = 1,233.391 kg
C130-V1 (0.000-13.892)	B200x20	Lsys = 13.892 m	Massa = 1,233.391 kg
C135-V1 (0.000-13.892)	B200x20	Lsys = 13.892 m	Massa = 1,233.391 kg
C182-V1 (0.000-7.000)	B200x20	Lsys = 7.000 m	Massa = 621.470 kg
C194-V1 (0.000-7.000)	B200x20	Lsys = 7.000 m	Massa = 621.470 kg
Subtotaal:	**B200x20**	**83.462 m**	**Massa = 7,409.894**
C117-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C118-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C119-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C127-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C128-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C129-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C137-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C138-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C139-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C140-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C146-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C148-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C149-V1 (0.000-13.892)	B260x20	Lsys = 13.892 m	Massa = 1,644.521 kg
C201-V1 (0.000-10.630)	B260x20	Lsys = 10.630 m	Massa = 1,258.346 kg
C202-V1 (0.000-10.630)	B260x20	Lsys = 10.630 m	Massa = 1,258.346 kg
Subtotaal:	**B260x20**	**201.862 m**	**Massa = 23,895.468**
C105-V1 (0.000-14.422)	B360x20	Lsys = 14.422 m	Massa = 2,418.578 kg
C17-V1 (0.000-14.422)	B360x20	Lsys = 14.422 m	Massa = 2,418.578 kg
C39-V1 (0.000-14.422)	B360x20	Lsys = 14.422 m	Massa = 2,418.578 kg
C61-V1 (0.000-14.422)	B360x20	Lsys = 14.422 m	Massa = 2,418.578 kg
C83-V1 (0.000-14.422)	B360x20	Lsys = 14.422 m	Massa = 2,418.578 kg

Student versie

Subtotaal:	**B360x20**	**72.111 m**	**Massa = 12,092.891**
C145-V1 (0.000-13.892)	B360x30	Lsys = 13.892 m	Massa = 3,391.825 kg
C150-V1 (0.000-13.892)	B360x30	Lsys = 13.892 m	Massa = 3,391.825 kg
Subtotaal:	**B360x30**	**27.785 m**	**Massa = 6,783.649**
C101-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C102-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C103-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C106-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C107-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C108-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C13-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C14-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C16-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C18-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C19-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C20-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C36-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C37-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C40-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C41-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C58-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C60-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C62-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C63-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C80-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C81-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C84-V1 (0.000-8.000)	B600x35	Lsys = 8.000 m	Massa = 3,901.450 kg
C85-V1 (0.000-14.422)	B600x35	Lsys = 14.422 m	Massa = 7,033.439 kg
C90-V1 (0.000-12.000)	B600x35	Lsys = 12.000 m	Massa = 5,852.175 kg
C92-V1 (0.000-12.000)	B600x35	Lsys = 12.000 m	Massa = 5,852.175 kg
Subtotaal:	**B600x35**	**280.222 m**	**Massa = 136,659.034**
C100-V1 (0.000-14.422)	B720x35	Lsys = 14.422 m	Massa = 8,527.266 kg
C109-V1 (0.000-14.422)	B720x35	Lsys = 14.422 m	Massa = 8,527.266 kg
C42-V1 (0.000-8.000)	B720x35	Lsys = 8.000 m	Massa = 4,730.076 kg
C91-V1 (0.000-12.000)	B720x35	Lsys = 12.000 m	Massa = 7,095.114 kg
Subtotaal:	**B720x35**	**48.844 m**	**Massa = 28,879.723**
C12-V1 (0.000-14.422)	B860x45	Lsys = 14.422 m	Massa = 13,044.317 kg
C1-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C21-V1 (0.000-14.422)	B860x45	Lsys = 14.422 m	Massa = 13,044.317 kg
C23-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C24-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C27-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C2-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C35-V1 (0.000-8.000)	B860x45	Lsys = 8.000 m	Massa = 7,235.685 kg
C3-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C45-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C46-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C49-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C57-V1 (0.000-8.000)	B860x45	Lsys = 8.000 m	Massa = 7,235.685 kg
C5-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C64-V1 (0.000-8.000)	B860x45	Lsys = 8.000 m	Massa = 7,235.685 kg
C67-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C68-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C70-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C71-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C79-V1 (0.000-8.000)	B860x45	Lsys = 8.000 m	Massa = 7,235.685 kg
C86-V1 (0.000-8.000)	B860x45	Lsys = 8.000 m	Massa = 7,235.685 kg
C89-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
C93-V1 (0.000-12.000)	B860x45	Lsys = 12.000 m	Massa = 10,853.528 kg
Subtotaal:	**B860x45**	**260.844 m**	**Massa = 235,923.504**
C110-V1 (0.000-8.000)	K1000x1200x50x50	Lsys = 8.000 m	Massa = 13,188.001 kg

C11-V1 (0.000-8.000)	K1000x1200x50x50	Lsys = 8.000 m	Massa = 13,188.001 kg
C151-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C152-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C153-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C154-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C171-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C172-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C173-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C174-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C175-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C176-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C177-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C178-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C195-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C196-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C197-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C198-V1 (0.000-7.000)	K1000x1200x50x50	Lsys = 7.000 m	Massa = 11,539.500 kg
C22-V1 (0.000-8.000)	K1000x1200x50x50	Lsys = 8.000 m	Massa = 13,188.001 kg
C99-V1 (0.000-8.000)	K1000x1200x50x50	Lsys = 8.000 m	Massa = 13,188.001 kg
Subtotaal:	**K1000x1200x50x50**	**144.000 m**	**Massa = 237,384.009**
C33-V1 (0.000-8.000)	K1000x1500x50x50	Lsys = 8.000 m	Massa = 15,072.001 kg
C44-V1 (0.000-8.000)	K1000x1500x50x50	Lsys = 8.000 m	Massa = 15,072.001 kg
C55-V1 (0.000-8.000)	K1000x1500x50x50	Lsys = 8.000 m	Massa = 15,072.001 kg
C66-V1 (0.000-8.000)	K1000x1500x50x50	Lsys = 8.000 m	Massa = 15,072.001 kg
C77-V1 (0.000-8.000)	K1000x1500x50x50	Lsys = 8.000 m	Massa = 15,072.001 kg
C88-V1 (0.000-8.000)	K1000x1500x50x50	Lsys = 8.000 m	Massa = 15,072.001 kg
Subtotaal:	**K1000x1500x50x50**	**48.000 m**	**Massa = 90,432.004**
Totaal:	**Massa = 1,414,359.801**	**L = 1,856.925 m**	