Living bridge Dordrecht

A bridge across the Oude Maas

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1. Introduction

The cities of Dordrecht and Zwijndrecht have always had a strong connection to each other and to the river which separates them, the Oude Maas. Ever since 1514 some form of connection across the river between the two cities has existed, weather by bridge, by ferry or both. In the future these two cities wish to amplify their connection to each other. From this wish the idea came for a new city centre on the banks of the Oude Maas. This also solved a problem of Dordrecht. The city has many small city centres and lacks one real large city centre. The new centre on the banks of the Oude Maas should replace these centres. The two centres on the bank need to form one centre and from that the wish for a connection between the river banks came forth. The current connections between the back are either time consuming (ferry) or a large detour (bridge) making them unattractive to use when wanting to go from one river bank to another. By building a living bridge the bridge does not just form a connection between the two river banks, but becomes an integral part of the new city centre for the Drechtsteden.

The purpose of this report is to present a structural design for a living bridge across the Oude Maas between Dordrecht and Zwijndrecht. The basis for this design will be an architectural design from a student of the faculty of Architecture of the Delft University of Technology. Because the shipping channel in the Oude Maas is a very busy one, with both flammable and toxic goods being transported along the river, extra attention is paid to the risks and safety of the project.

The lay-out of the report is as followed. In chapter 2 a description of the history and future plans of the two cities are given. These plans lead to the problem definition and the report objective. To get some insight into living bridges a definition is given in chapter 3. To come to a good design the limitations for the design are required. These limitations form the basis for the list of requirements given in chapter 4. Because parts of the limitations come from the parties involved in this project also an overview of these parties is given in this chapter. Next the characteristics of the river such as transport and river discharge as well as the characteristics of the shipping channel are given (chapter 5). After this a qualitative overview of the risk relations is given in chapter 6. In chapter 7 an overview of the alternative designs for the living bridge are given. From these alternatives one design will be chosen and in chapter 8 this design will be used to discuss several variants on this design. From these variants one will be chosen and this variant will be used for the structural design in chapters 9 and 10. In these chapters the bridge across the Oude Maas (9) and the building on the Zwijndrecht bank (10) will de calculated. Also the method of erection will be discussed in this chapter, After this the possibilities for collision protections for the pylon will be discussed in chapter 11. Finally in chapter 12 the conclusions and recommendations are given.

2. Problem analysis

In this chapter the current situation is analysed by first looking at the history of Dordrecht and Zwijndrecht and after this by looking at the future plans. From these plans the problem definition and report objective for this thesis is derived.

2.1 History of the Drechtsteden

Dordrecht has always been a city with a strong connection to the water. The city came into being around 1100 on the banks of the river the Thure, under the name Thuredriht, which means fordable place in the Thure. In 1080 a start was made with the construction of the Big church and the small town grew into a living place for farmers and fisherman. The area was mainly formed by peat swamps and therefore Thuredriht was a traditional land cultivation town. At first, mainly the banks along the rivers were used. Later on, also the areas along the peat waters, which flow into the rivers, were cultivated. Eventually the hart of the peat areas were cultivated. By the end of the thirteenth century the land cultivation process was completed.

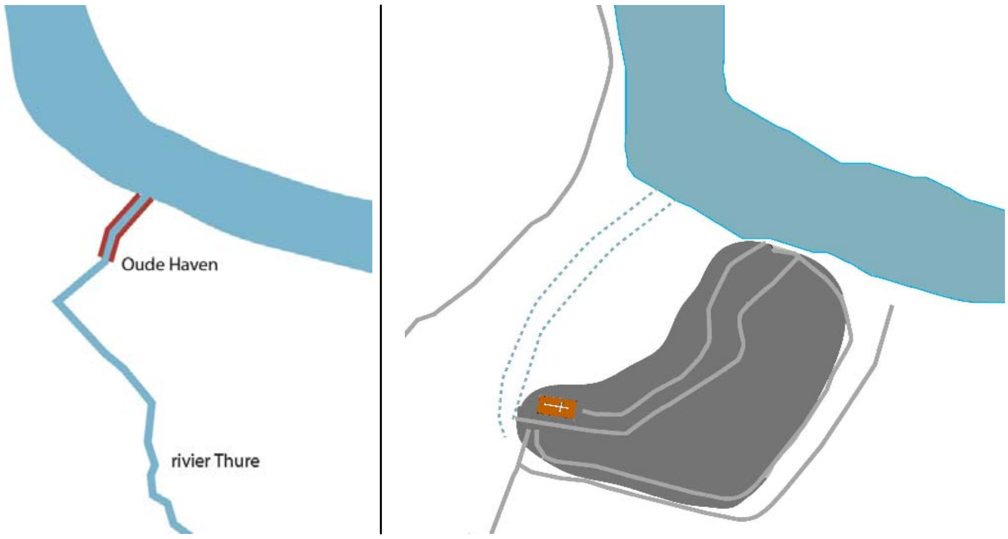


Figure 2.1: Development along the river, left harbour (around 1100) and right buildings (around 1200).

In the beginning Thuredriht was a small town with two strips of buildings on either sides of a peat creek. The houses were on individual terps, these terps were divided by separation ditches. These ditches were primarily there to dewater the peat, but by there length, they also determined the allocation of the peat mining. This pattern can still be found, because the current central water of Dordrecht can be traced back to the peat creek which, in the north, flowed into the Merwede. The two main streets of the present day Dordrecht are were the first terps used to be.
Around 1150 there were a number of floods, these floods had big consequences for the river system. The peat creek became a connection between the Rijnsystem to the north and the Maassystem to the south. The town was now in a place with a good approachableness by ship. This caused a growth of the town, the ground between the terps was raised and the building

was expanded. In spite of the growth, the majority of the buildings remained along the river banks.
In the first part of the thirteenth century the development of the town focused on the strips behind the river banks, the zones leading to the water were later on made ready for buildings. In 1220 the earl of Holland gave city rights to Dordrecht. In the second half of the thirteenth century the town developed into a city. In this period a thickening of the buildings on the main street took place. In the back area free-standing buildings were made, this leaded to the creation of side streets.
The big growth of Dordrecht happens after 1300, there are two causes for this. First of all the location and second the “stapelrecht” Dordrecht was given. Because of the location of Dordrecht on a strategic point with respect to the main shipping channels in the Netherlands, it was possible to control the trade traffic on the Rijn and Maas in east – west direction. There also was an important intersection with the important north – south route. By levying tolls on all the waters around Dordrecht, the city became an important centre. In 1299 the “Stapelrecht” was acquired. This meant that all goods transported over the shipping channels around Dordrecht had to be offered for sale at the marked. This caused a permanent trade, which brought a lot of revenues and wealth to the city.

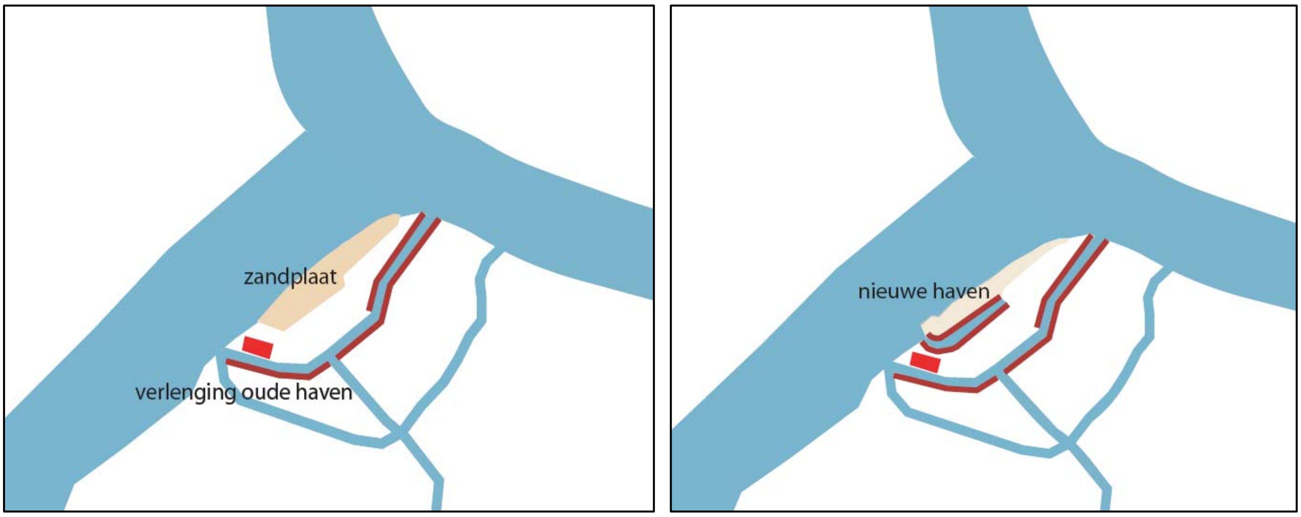


Figure 2.2: The harbour around 1282 (left) and around 1410 (right).

The increase in business lead to the question for a new arrangement of the harbour. Therefore it was decided upon to create a new harbour. Along the main street the thickening of buildings was continued and the parcels were narrowed to a minimum width of 4 m. The existing buildings behind houses were coupled to the houses in front of them, which lead to very deep-seated houses. And the streets were paved. The side streets were no longer access roads to the hinterland, but were assigned an own direct function.
In 1421 there was a big flood, the St. Elisabethflood. All the town in the vicinity of Dordrecht were flushed away and Dordrecht became an island. This caused that Dordrecht could only be reached by ship. This causes the business to increase even more. The most import city gates are located near the water, because these are the only points of entry to the city.
The development of cities in the area, like Zwijndrecht and Papendrecht, falls behind in comparison to Dordrecht, they mainly serve as a hinterland for Dordrecht. Mainly because the island itself is not suitable for agricultural purposes.

In 1457 the city gets struck by a large fire, however the structure of the city is that well developed that with the rebuilding of the city the old structure is used. Despite the setbacks the city remains an important trade city, in this century and the centuries to come.
In the sixteenth century, Dordrecht becomes one of the first cities where the reformation happens. During the 80-year war, Dordrecht becomes an even more important city. The Dutch rebels control the shipping channels, and therefore they had the possibility to block the flow of goods from and to Dordrecht. This forced Dordrecht in 1572 to side with Willem van Oranje during the great revolt.
Because of its safe location on an island, Dordrecht became the capitol in which the rebel government resided.

Figure 2.3: Bridges near Dordrecht around 1545.

In 1545 many changes were made to the infrastructure, with the construction of a number of bridges across the WIjnhaven. At the same time a number of adjustments were made to the harbour. The new harbour was made deeper and made longer. And the Wolwershaven, the Bomhaven, the Lijnbaan haven and the Maartense gat are created to expand the harbours.

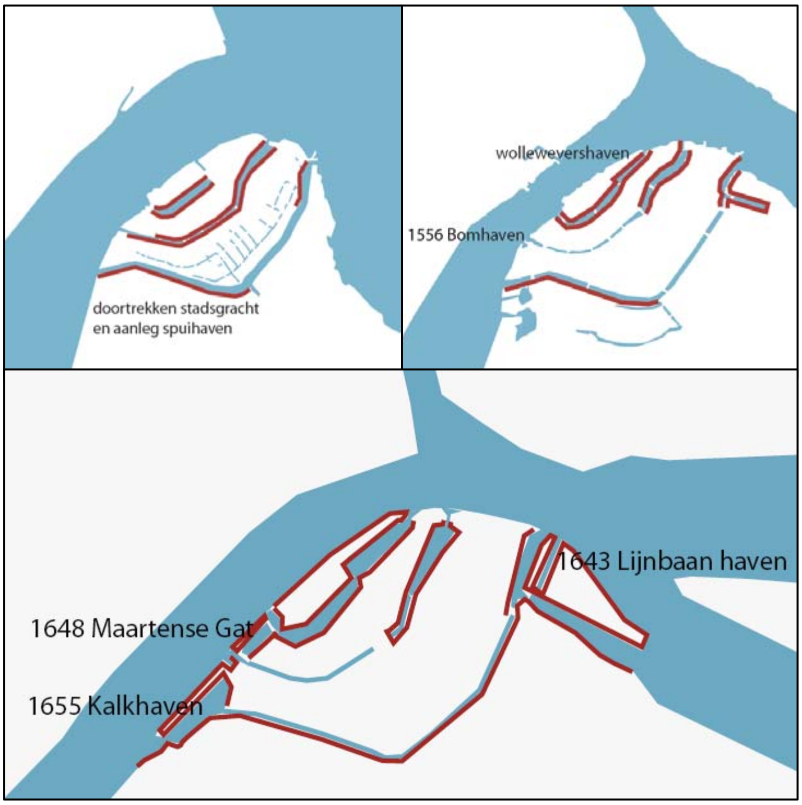


Figure 2.4: The Dordrecht harbours around 1574 (left upperpart), 1609 (right upperpart) and 1650 (bottom).

However the position of most important harbourcity is starting to decline with the rise of Rotterdam. Dordrecht fails to get a position in the international trade, causing a setback in the economical activities. In 1795 the city loses its “Stapelrecht”, causing the city to lose its economical base. By the end of the eighteenth century Dordrecht is no longer the most important harbour city, Rotterdam has taken its place.

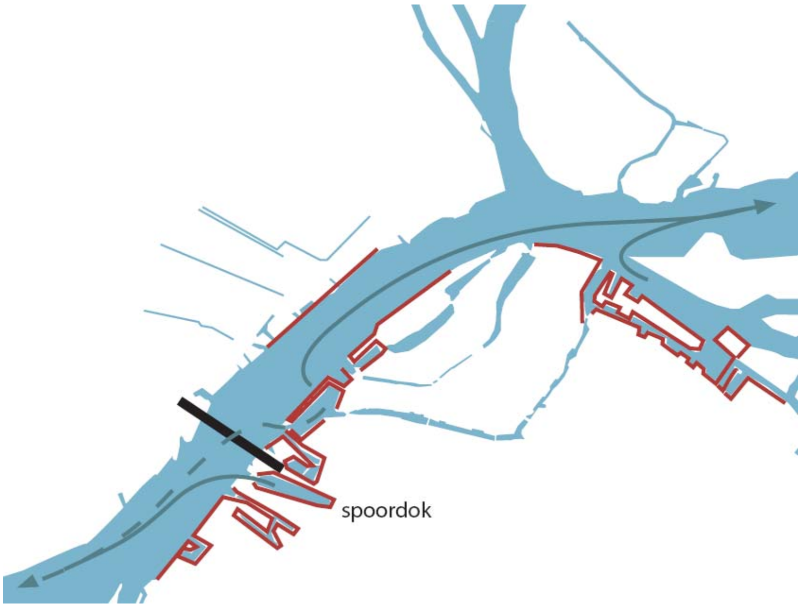


Figure 2.5: Harbours Dordrecht around 1873.

However the city did keep its function as an import city for the region. By the beginning of the nineteenth century, Dordrecht still lives of the wealth of the previous centuries, however it becomes clear that trade and wealth are declining. The city also receives funds from the government for the building of a grain exchange. From this point on the city changes from trade – harbour city to a regional distribution centre and industrial city. This also has its reflection on the image of the city. Because shipyards leave the city and metal and timber industry rise up later, unemployment and a stagnation of the city growth takes place.

An import change for the city is the building of the railway Rotterdam – Dordrecht – Antwerpen. The station is placed outside of the city, causing the ground between the city and the station to become an interesting piece of land. This also changes the orientation of the city, in the past the city used to be orientated at the river, now the city is more orientating on the land. Causing the docks to loose there important function. The new railroad also created an extra connection to Zwijndrecht. This connection was first made in 1814 by Napoleon, he created a ferry service between Dordrecht and Zwijndrecht. Because of the rise of the automobile the capacity of the new connection was quickly outgrown. At first, the capacity of the ferry was expand, but in 1920 the city council realised that the number of cars was too large for the streets of Dordrecht. To create a better situation a bridge was build in during the thirties. The ferry stayed in use for the transport of pedestrians, because Zwijndrecht was the endpoint of the Rotterdamsche Tramweg Maatschappij. In later time the ferry became redundant, and therefore was shut down. The last couple of years the connection was reinstated in the form of water trolleys. Another important expansion of the city took place in 1905, with the adding of the town of Dubbeldam to the Dordrecht area. This was the start for the city to expand in southern direction.

The city of Zwijndrecht has a much more modest history. In the beginning the area mainly served as hinterland for Dordrecht. In that period Zwijndrecht was nothing more than a line town along the dike. It remained so for centuries. It took till 1950 for the city to develop. The structure of the city differs from the one Dordrecht has. The city is much wider with large living areas and wide streets.

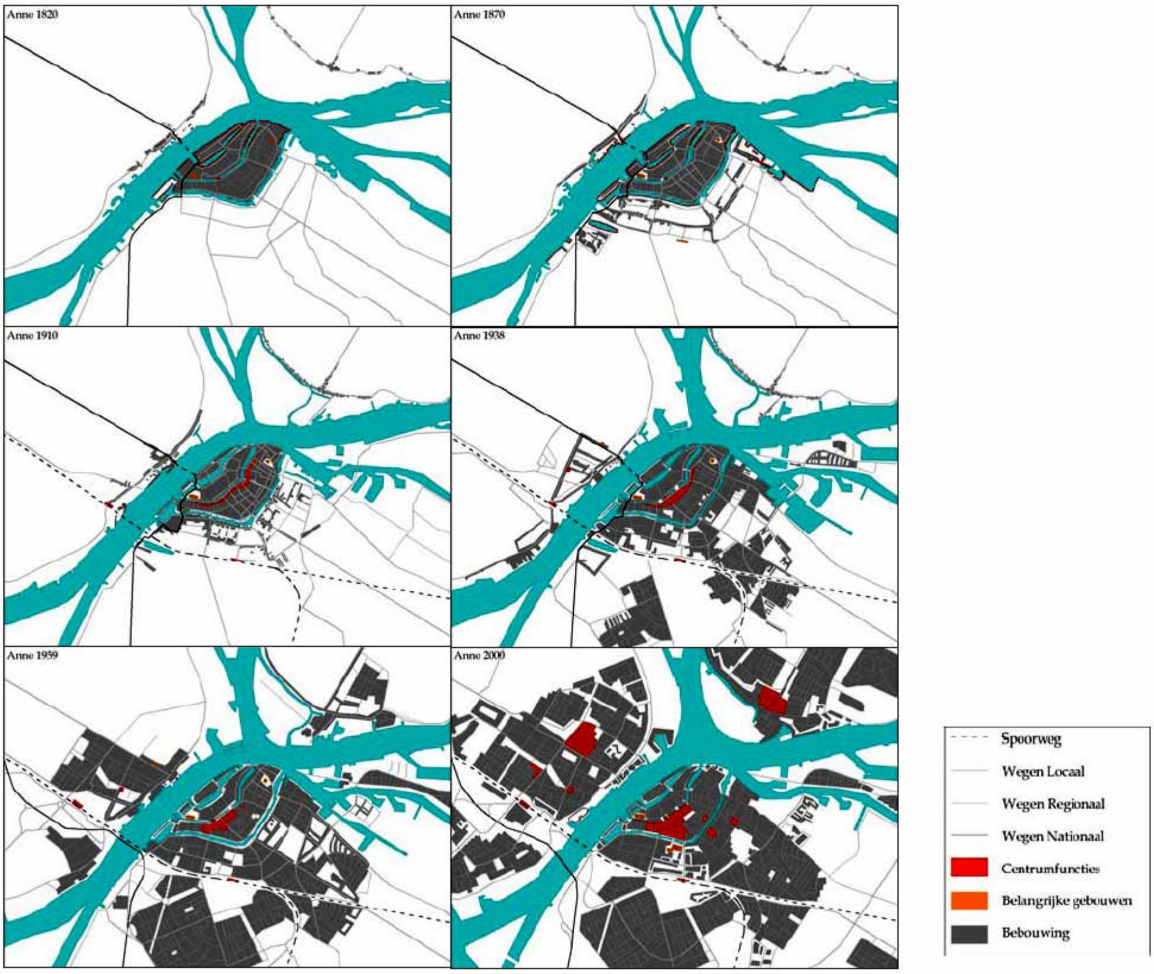


Figure 2.6: Development of Dordrecht and Zwijndrecht during the last two centuries.

2.2 Future plans

In the current situation both Dordrecht and Zwijndrecht do not have one clear area which can be marked as a city centre. In stead they both have several areas with a centre function, as can be seen in figure 2.7.

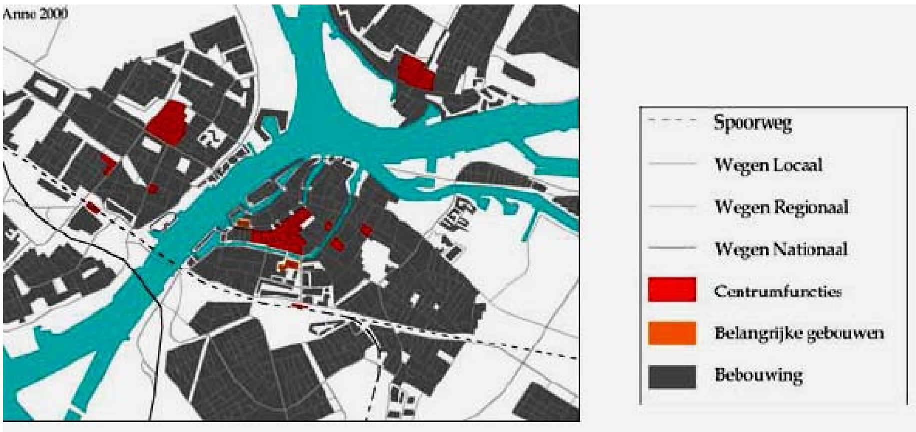


Figure 2.7: Map Drechtsteden.

Especially in Dordrecht there is the need for one area with a centre function in stead of a number of areas with a centre function. To create a stronger connection between the Drechtsteden and to give the new centre a bigger look, the question is raised for a new centre on the banks of the Oude Maas. On both the side of Dordrecht and on the side of Zwijndrecht, there is an area available for creating such a new city centre. Especially with the demolition of an industrial area on the side of Zwijndrecht, which is scheduled for the next years.

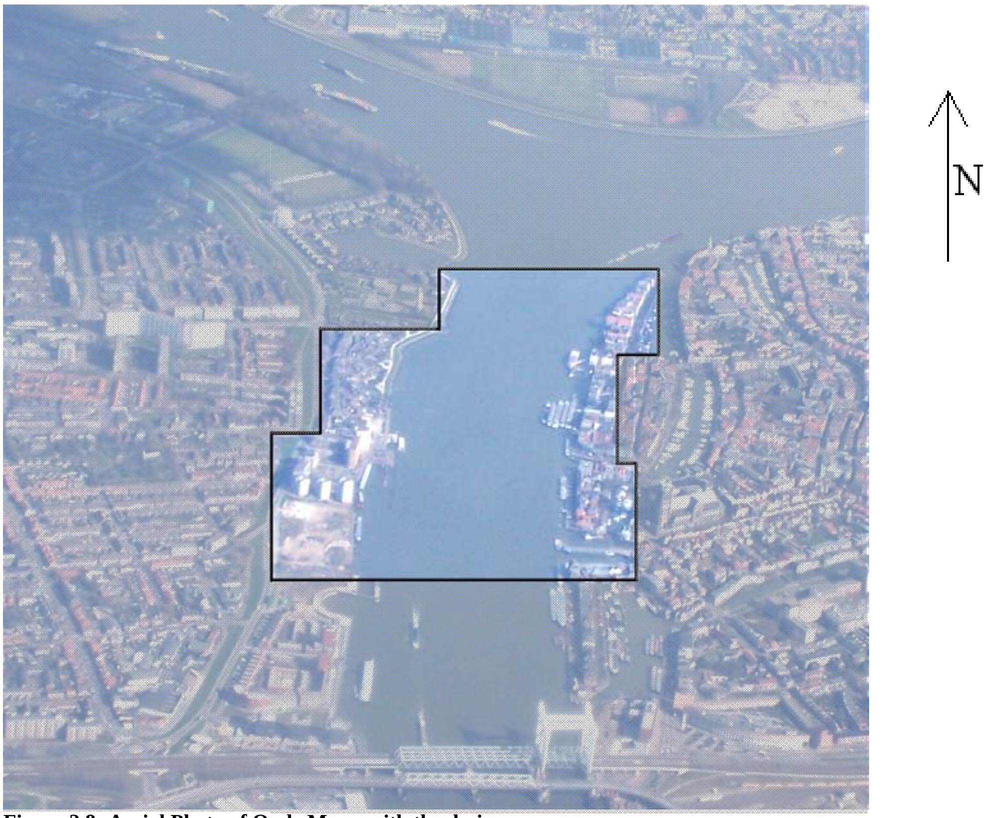


Figure 2.8: Aerial Photo of Oude Maas, with the design area.

The two areas give a great opportunity to create a new city centre for the Drechtsteden. However to unite the two areas and to give it the appearance of being one centre a connection between the two banks is needed. There are several ways to do so, a bridge or tunnel can be constructed or a small ferry service can be created. However when the cities opt for a real Eyecatcher, a bridge is the best option. Because a bridge is visible from different parts of the city and from the water. Therefore a bridge is the most visible connection between the two banks. By making this bridge a living bridge, the connection between the two parts is amplified, making it appear as one new city centre instead of two centers. This means buildings will be constructed on the bridge or as part of the bridge. And also the available space on the water can be used to construct buildings.

An other good reason to create a connection between the two banks of the river can be derived from the history of these two cities. Ever since 1814 there has been a connection between the two cities, either by ferry, bridge or both. There has always been a large use of these connections by the people of both Zwijndrecht and Dordrecht. By building a new connection between the two cities the people are given a new, faster option two travel between the two cities, without taking a detour across the current bridge or travel by the more time consuming water ferry.

2.3 Problem definition

There is the demand for a new city centre on the banks of the Oude Maas. This demand comes from both the Drechtsteden. On both the Dordrecht and the Zwijndrecht side of the river there is enough space available on the river banks to develop a new city centre. However these two banks are separated by a river, to create one city centre a connection between the two river banks needs to be realised. There are however strict limitations to the design due to the fact that this connection is a living bridge over a busy shipping channel.

2.4 Report objective

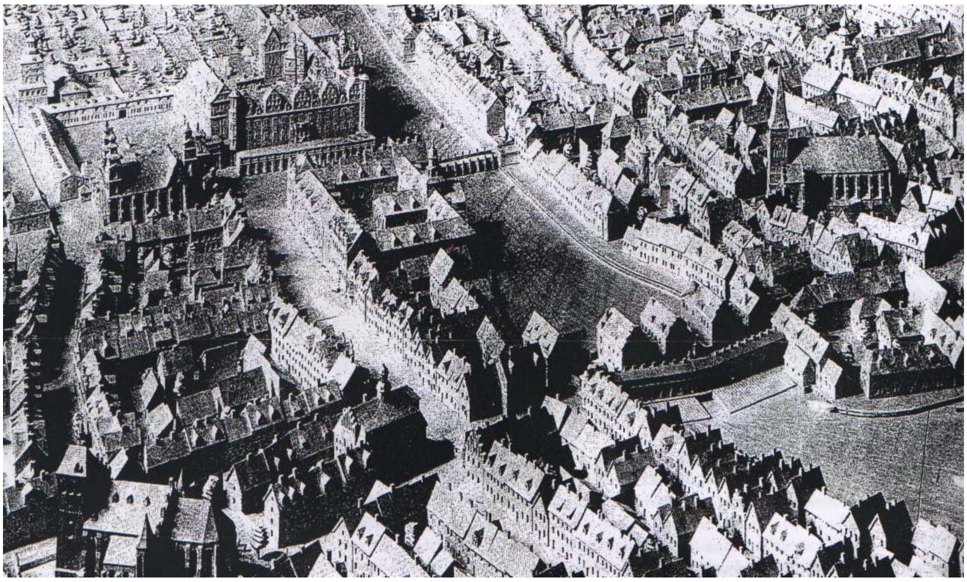
To make a structural design for a living bridge for pedestrians and cyclists between Zwijndrecht and Dordrecht. Making both the river banks come together as one new city centre.

3. Definition living bridge

To define what a living bridge is, it is best to look at its use throughout history.
When looking at the history the classical (and most simple) definition for a living bridge is: A bridge which has buildings on it. However, what did this mean for the practice?

3.1 History world wide

Europe



In Europe this type of bridge was mainly used to connect two urban areas which were divided by an obstacle, mostly a river. The buildings on the bridge formed an accommodation for social and economical activities.

Figure 3.1: Model of the centre of Berlin with the Mühlendammbrücke (1688).

So it becomes clear a living bridges consists of two parts, a bridge to overcome an obstacle and a superstructure for social and economical activities. An other secondairy function assigned to a living bridge was militairy, the bridges were integrated into the city defensive system.

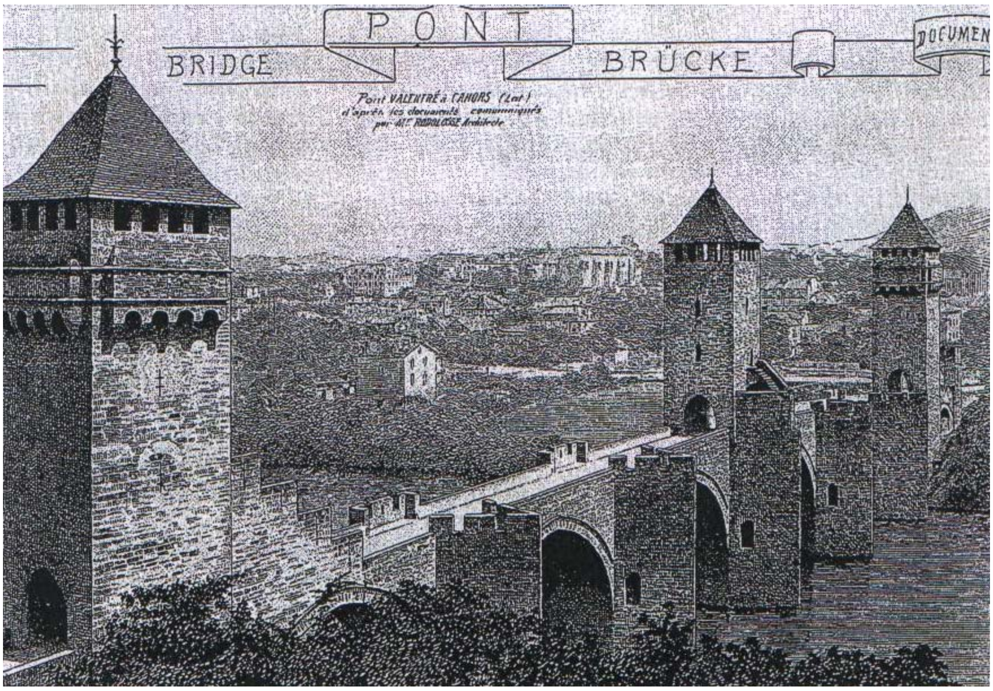


Figure 3.2: Pont Valentré, Cahors, 19th century.

The first living bridges were built in the Middle Ages (eleventh and twelfth century). People continued to build these bridges until the end of the seventeenth century. Unfortunately most of these bridges were destroyed during the eighteenth century. After this period almost no living bridges were constructed. There are a number of factors which have caused this change in attitude towards building living bridges.


Figure 3.3: Old Tyne Bridge, Newcastle, in ruinous state, 1772.

The growth of cities and with that the expansion of their economics forced the cities to grow outside their city walls. This also caused the traffic to grow, inhabited bridges formed an obstruction for the flow of this traffic, and therefore where demolished during this period. Also in this period the training of engineers and architects was separated. And because to engineers the idea of combining building and bridge was not an attractive one and because the architects mainly concentrated on triumphal bridges, only few living bridges were designed in this period.

Most bridges were constructed in England, France and Italy. To the north of these countries only a few living bridges were constructed. To the south of this line no significant evidence of living bridges was found.

U.S.A.

Outside of Europe there were only a few living bridges build. In the United States of America the first proposals for living bridges were put forward in the 1920s. Two architects came up with the idea to create skyscrapers as a support for suspension bridges (over the Hudson River and across the San Francisco bay). The skyscrapers were designed to house people and to create office space. However due to the economic crisis in 1929 these two bridges were never constructed. After WW 2, a few inhabited bridges were constructed, they were designed by F. L. Wright.

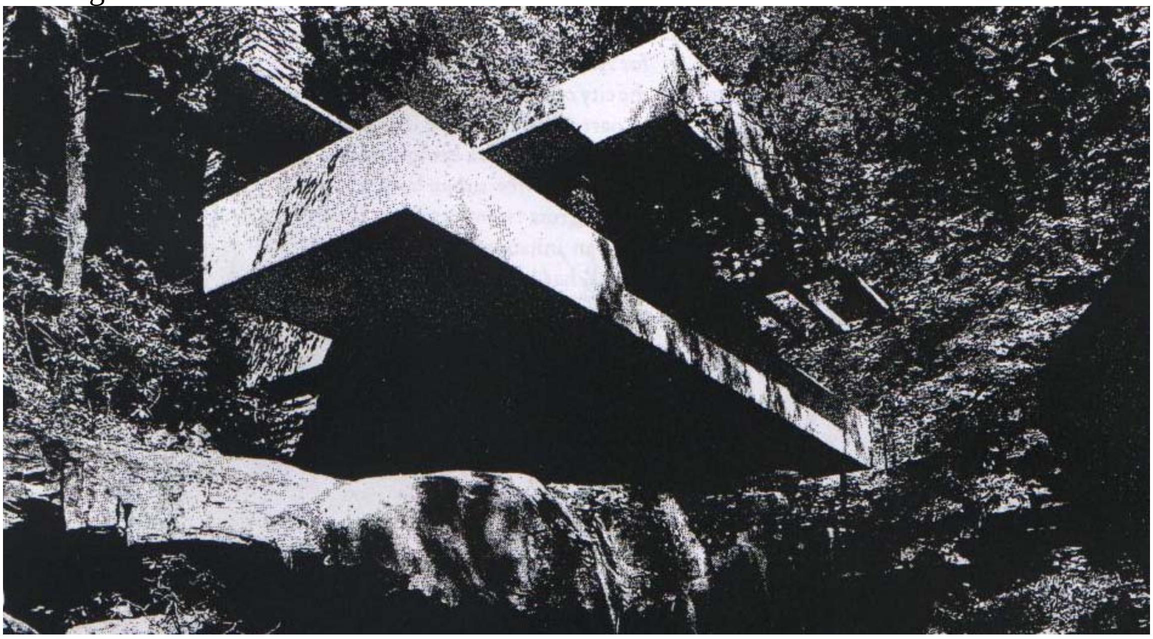


Figure 3.4: F.L. Wright, Falling Water, Bear Run, Pennsylvania, 1936.

Non-western world

In the Middle East and Asia living bridges appear to be isolated exceptions, no tradition of building living bridges seems to exist. One of the few known examples is the Iranian barrage bridge, this bridge had, in contradiction to the European, no commercial, residential or cultural facilities, but formed both a passage over the Zayandeh Roud and a dam.

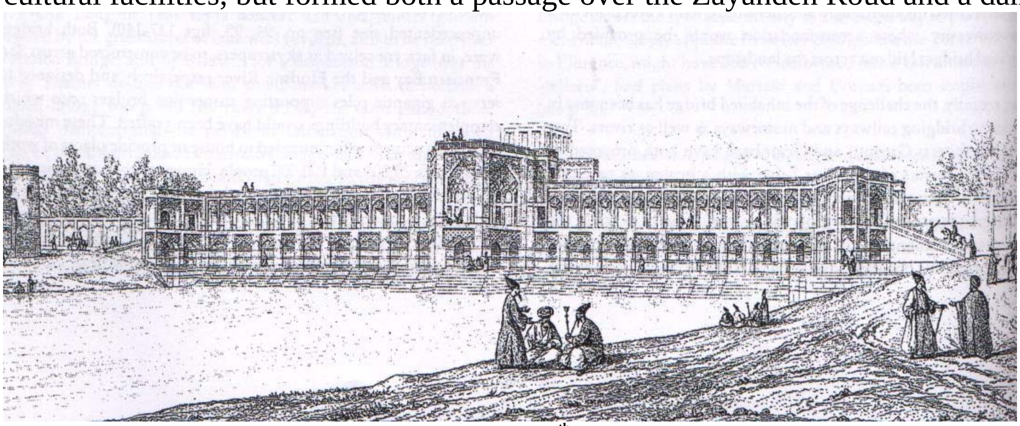


Figure 3.5: The Hasan-Beg Barrage Bridge, Isfahan, Iran, 17th century.

3.2 Conclusion

It can be concluded that the dominant functions of inhabited bridges, besides spanning a river, were commercial and residential. Most times the first commercial use was modest and gradually evolved into more prestigious nature. Because the need of safeguarding there stock most tradesmen wanted a living quarter near there shop, this lead to the creation of apartments on one or more levels above the shops.
An other, totally different function, the living bridge was used for, was military. Bridges were integrated into the defensive system of the city walls.

From all the above a more sophisticated and elaborate definition for a living bridge can be derived.
A living bridge is bridge with multiple functions. The first function is always to span a gap, being a river, a highway or a canyon. The second function is formed by a construction on or next to the bridge. This can be both residential and commercial.

4. List of requirements

In this chapter the list of requirements is given. To come to the list of requirements first an overview of the participating parties is given. Most of the limitations come from one of the participating parties. These limitations form the basis for the list of requirements.

4.1 Participating parties

There are a number of parties involved in the project. They can be divided into three groups. The initiators, they are the ones who made it possible to realize the project, the beneficials, this group consists of the people who benefit from the project, and the third group is the third party, this group is formed by the people of Dordrecht and Zwijndrecht.

Initiators

- The city of Dordrecht
  Initiator of the plan for a new city centre in combination with the city of Zwijndrecht, on the opposite site of the river. The city council determines the land use within the city limits. Therefore the city council has to change the plans for the land use, to create space on the banks of the Oude Maas.
  There is also the possibility that the infrastructure needs to be adapted to the new situation.

- The city of Zwijndrecht
  For the city of Zwijndrecht holds the same as for the city of Dordrecht, they are the initiators and are the ones who have to change the plans for land use.

- Government
  The jurisdiction of the rivers in the Netherlands is in the hands of Rijkswaterstaat (RWS). The design has going to have influences on the shipping channel and on the width and depth of the river. Therefore an approval by RWS is required.

Beneficials

- People of Dordrecht
  They are the primary users of the new city centre. They no longer have to travel across town for groceries, etc. Because of the new city centre, they will now find everything centered in one location.
  Also the traffic trough the city will be reduced, because most of it will no longer need to go trough the city, but will be going directly to the new city centre. And of course an easy connection between Dordrecht and Zwijndrecht is created, making it easier for the people to go to the other side of the river.

- People of Zwijndrecht
  They are the other group of primairy users and have the same benefits as the people of Dordrecht.

- People living near the two cities
  This group consists of people living in the small towns near Dordrecht. They also have the opportunity to go shopping in a new city centre.

- Tourists
  The new living bridge will be an object which draws people to the city for site seeing, most likely they will also go shopping in the new centre.

- Exploiters
  This category consists of people such as shop owners, restaurant holders and the cinema exploiter. They will have their business set up in the new city centre. It is very important for this group that the infrastructure is up to date, for the delivery of goods to the shops, etc.

- Cities of Dordrecht and Zwijndrecht
  Owners of the land on which the new city centre is created. By selling the land or renting it out, they can make revenues of the new centre. Also new parking space is required, by making this paid parking, an other source of revenues is created.

Third party

- Third party
  Group of people who live near the new city centre. Beside the beneficials they are also the people who live near the new centre. Therefore it must be made sure that during construction they have as little hindrance from the construction as possible.

4.2 *Limitations*

In this paragraph the limitations to the design are given. They will be quantified in the list of requirements, paragraph 4.3. Sometimes a reference is given to another chapter or paragraph, where the limitation is explained. The limitations can be divided into three groups. The first group are the boundary conditions, these are limitations that originate from demands by one of the participating parties. The party which is responsible for the condition is mentioned between brackets. The designer has no influence on these conditions. The second group are the constraints, this are limitations used by the designer to create boundaries within which a design is to be made. The third group are the assumptions, this group represents values that are uncertain. During the research and report they are legitimatised, to make sure they are correct and to make sure a change has no influence on the outcome.

The three groups are also divided into a number of categories. This is done to create a better overview of the limitations which are connected. These different categories also give the limitation a code, which is used as a reference in the list of requirements. The different categories are:

- Functional
- Technical
- Economical
- Environmental
- Societal

4.2.1 Boundary conditions

Functional boundary conditions:

FBC 1 The bridge should have a movable part (RWS)
FBC 2 One of the new buildings will have to contain a movie theatre (city councils)
FBC 3 New parking space is required (city councils)
FBC 4 The shipping channel can not be blocked during construction or when the project is finished (RWS)
FBC 5 The bridge is designed for pedestrians and cyclists (city councils)
FBC 6 Emergency services should have access to the bridge (VROM)
FBC 7 The depth of the shipping channel should be kept at a sufficient depth (RWS)
FBC 8 The width of the shipping channel should stay sufficient (RWS)
FBC 9 The turning circles of ships should be taken into account (RWS)

Technical boundary conditions:

TBC 1 The height of the not movable part of the bridge should be large enough to facilitate most ships (RWS)
TBC 2 The river discharge should be taken into account (RWS)
TBC 3 Extra safety measures are required in case a calamity occurs (VROM)

Economical boundary conditions:

EcBC 1 Lifetime should be defined (city councils)
EcBC 2 Revenues should be larger than costs (city councils)

4.2.2 Constraints

Functional constraints:
FC 1 The infrastructure should be adapted to the new situation

Technical constraints:
TC 1 The amount of dredging should be kept at a minimum
TC 2 Design should comply with NEN
TC 3 Design should be located within study area
TC 4 Difference in water level needs to be taken into account
TC 5 Water density should be given
TC 6 Soil data is required
TC 7 Gravitational acceleration should be given
TC 8 Bottom research for pipes, historical objects, etc. is required

Environmental constraints:
EnC 1 The design should not be too harmful for the environment
EnC 2 Bottom should be taken into account

Societal constraints:
SC 1 Create the least possible hindrance for people living near the construction
SC 2 Create the least possible hindrance for traffic near the construction
SC 3 The level of the groundwater can not be altered during construction

4.2.3 Assumptions

Functional assumptions:
FA 1 Land needed for the design is available

Economical assumptions:
EA 1 All licenses required, are given

Societal assumptions:
SA 1 The plans for land use are, when necessary, changed in favour of this project

4.3 *List of requirements*

4.3.1 Boundary conditions

Functional boundary conditions:

FBC 1 The bridge should have a movable part, spanning at least 16,4 m (paragraph 8.1)
FBC 2 The design will contain a movie theatre with a capacity of 2400 seats
FBC 3 A new parking space, with 500 places is included in the design
FBC 4 The shipping channel will remain open for ships class VI c
FBC 5 The bridge requires lanes for pedestrians, these lanes should comply with RONA
FBC 5 The bridge requires lanes for cyclists, these lanes should comply with RONA
FBC 5 The load on the bridge should comply with NEN
FBC 6 The bridge will be accessible for emergency services
FBC 7 The depth of the shipping channel should be kept at a minimum depth of 6,3 m (paragraph 5.3)
FBC 8 The width of the channel should be kept at a minimum width of 156,8 m (paragraph 5.3)
FBC 9 The shipping channel will be kept at a sufficient width for ships to turn

Technical boundary conditions:

TBC 1 The height of the not movable part of the bridge will be Rijnvaart-height
TBC 2 The river discharge should be taken into account (RWS)
TBC 3 Extra safety measures will be designed to prevent calamities

Economical boundary conditions:

EcBC 1 The lifetime of the bridge is 100 years
EcBC 1 The lifetime of the buildings is 100 years
EcBC 2 Revenues will be larger than costs

4.3.2 Constraints

Functional constraints:

FC 1 The infrastructure will be adapted to the new situation

Technical constraints:

TC 1 No dredging is required
TC 2 Design should comply with NEN
TC 3 Design should be located within study area
TC 4 Difference in water level needs to be taken into account
TC 5 Water density is 1 kg/m^2
TC 6 Soil data is required (paragraph 7.3)
TC 7 Gravitational acceleration is 10 m/s^2
TC 8 Bottom research for pipes, historical objects, etc. is required

Environmental constraints:

EnC 1 The design should not be too harmful for the environment
EnC 2 There should be looked at the necessity of bottom protection

Societal constraints:

SC 1	Create the least possible hindrance for people living near the construction
SC 2	Create the least possible hindrance for traffic near the construction
SC 3	The level of the groundwater can not be altered during construction

4.3.3 Assumptions

Functional assumptions:

FA 1	Land needed for the design is available

Economical assumptions:

EA 1	All licenses required, are given

Societal assumptions:

SA 1	The plans for land use are changed in favour of this project

5. The river the Oude Maas

In this chapter information is given about the transport along the Oude Maas, the discharge of the Oude Maas and about the shipping channel in the Oude Maas.

5.1 Transportation of dangerous goods

Location

The Oude Maas is a heavily sailed shipping route, therefore the river is classified as a main shipping route (Source: Risicoatlas Hoofdvaarwegen Nederland). This creates a number of risks, especially because a number of dangerous goods are shipped along this route. The dangerous goods are devided into two groups, flammable and poisonous goods. These two groups are further divided, see table 5.1.

Table 5.1: Classification dangerous goods.

Code	Category	Example	Ship type
GF3	Flammable gasses	Propane	Tanker
GT3	Toxic gasses	Ammonia	Tanker
LF1	Flammable liquids	Diesel oil	Single hull
LF2	Flammable liquids	Gasoline	60% Single hull, 40% Double hull
LT1	Toxic liquids	Acrylnitril	Double hull
LT2	Toxic liquids	Propylamine	Double hull

The Dutch rivers are divided into a number of traffic sections. The last 10 years a research was done into the traffic intensity of the different sections and to the number of accidents within a section. This was also done for the Oude Maas, section 59

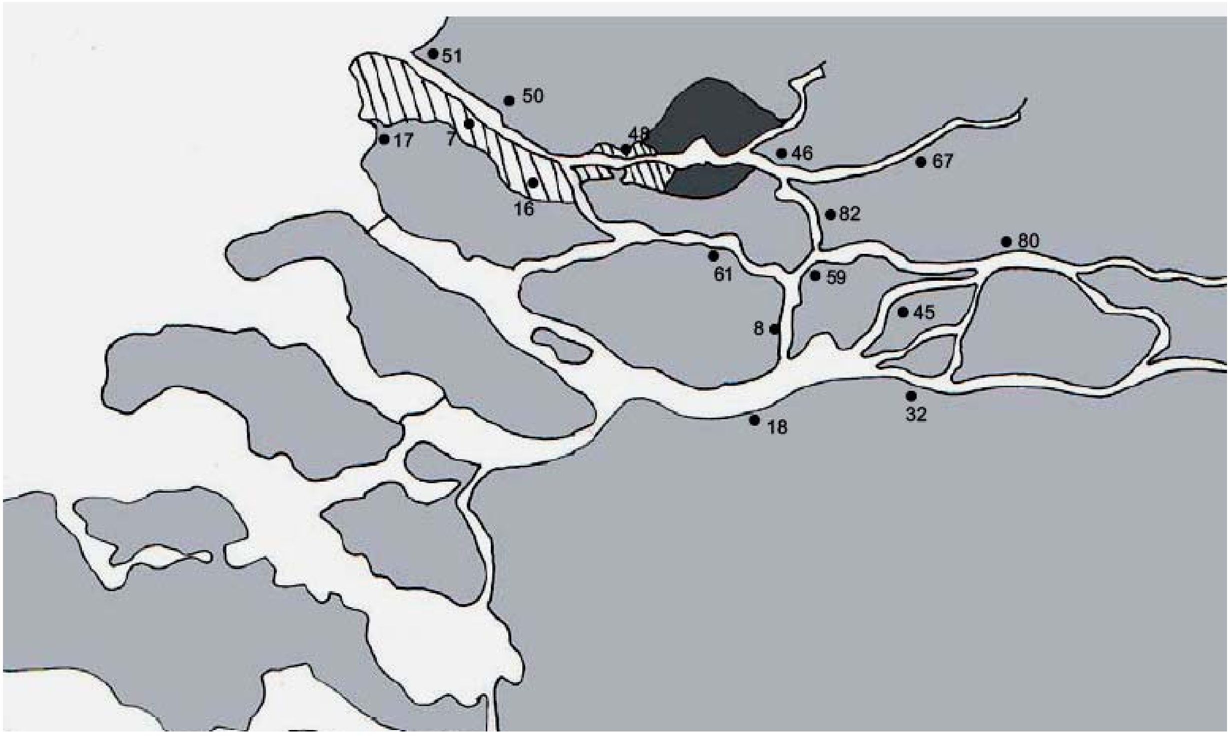


Figure 5.1: Map with traffic sections.

Transport on the Oude Maas

The research of section 59 has given the following data:

Table 5.2: Data traffic section 59.

Section	From	To	Length (m)	Width (m)
59	Noord	Dordtsche kil	4,3	300

Section	Traffic intensity (1/jaar)	GF3	GT3	LF1	LF2	LT1	LT2
59	99697	276	20	4580	4134	46	0

The data clearly shows there is transportation of both flammable and poisonous goods along the river.

To get a clear picture of the importance of the transport along the Oude Maas, a bigger section needs to be investigated. The main transport across the Dutch rivers is from west to east, or from Rotterdam and Antwerp to the Ruhr area. The main points of entry from the North Sea are the Westerschelde and the Maasvlakte.

Shipping routes of flammable liquids from the North Sea to the Ruhr area (see figure 5.2)*:*

From the Westerschelde / Hollands Diep:
It is clear that both the Nieuwe Merwede and the Oude Maas are used for the transportation of flammable liquids to the Ruhr area.

From the Maasvlakte:
The vast majority of the ships uses the Oude maas and only a small part uses the area near Rotterdam.

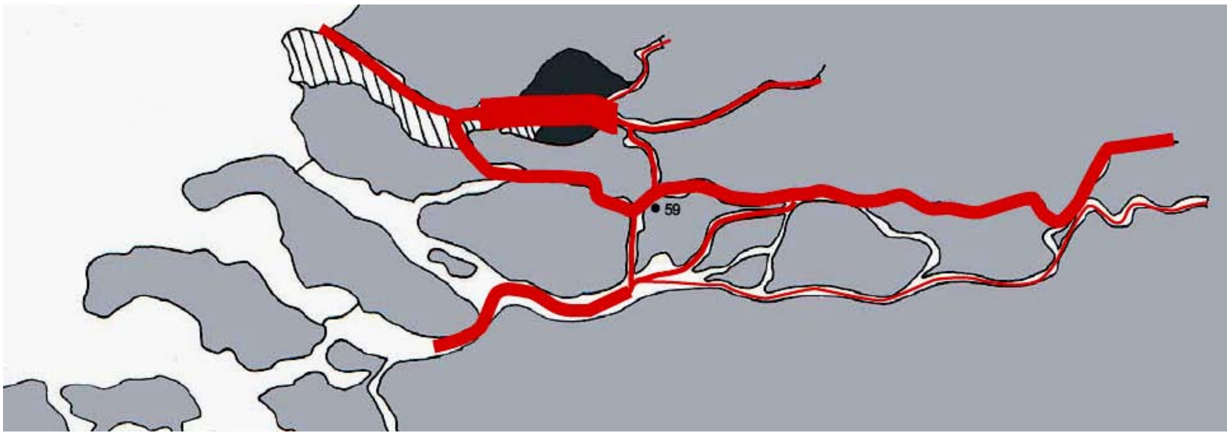


Figure 5.2: Transportation of flammable liquids.

It can be concluded that the Oude Maas plays an important role as a shipping route for the transportation of flammable liquids.

Shipping routes of flammable gasses from the North Sea to the Ruhr area (see figure 5.3):

From the Westerschelde / Hollands Diep:
A part of the ships goes directly, via the Nieuwe Merwede to the Ruhr area. Another part goes, via the Dordsche Kil, to Rotterdam. Only a small part of the ships use the Oude Maas

From the Maasvlakte:
Only a small portion of the ships containing flammable gasses enter the Netherlands via this route, this is because the majority has Rotterdam as destination. These ships use the Waal and don't use the Oude Maas

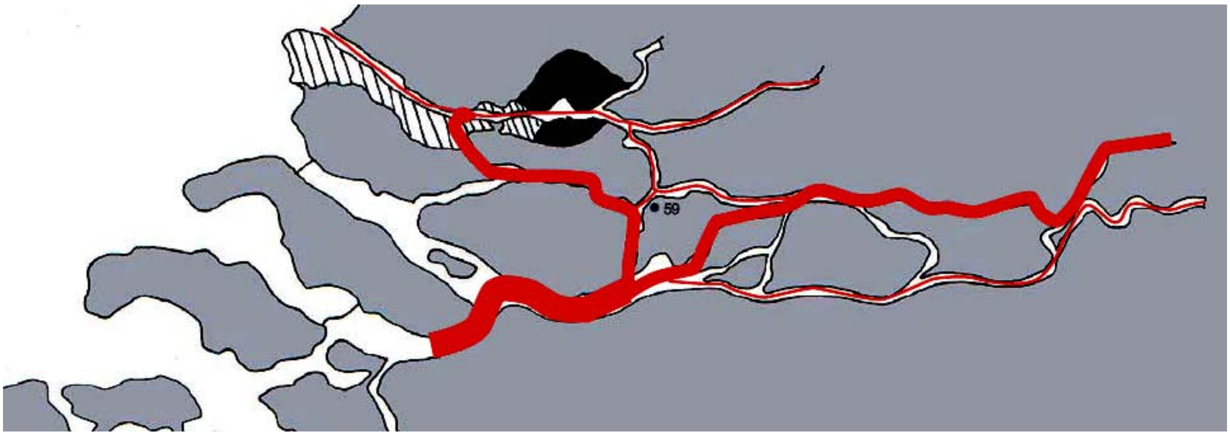


Figure 5.3: Transportation of flammable gasses.

It can be concluded that the Oude Maas only plays a small role when there is looked at the amount of ships with flammable gasses which use it as a transportation route.

Shipping routes of toxic goods from the North Sea to the Ruhr area (see figure 5.4)*:*

From the Westerschelde / Hollands Diep:
The majority of the ships entering go north trough the Dordsche Kil, one part goes to Rotterdam and the other part uses the Oude Maas to go to the Waal

From the Maasvlakte:
For this route holds the same, one part goes directly to Rotterdam, the other part uses the Dordsche Kil and the Oude Maas to go to the Waal

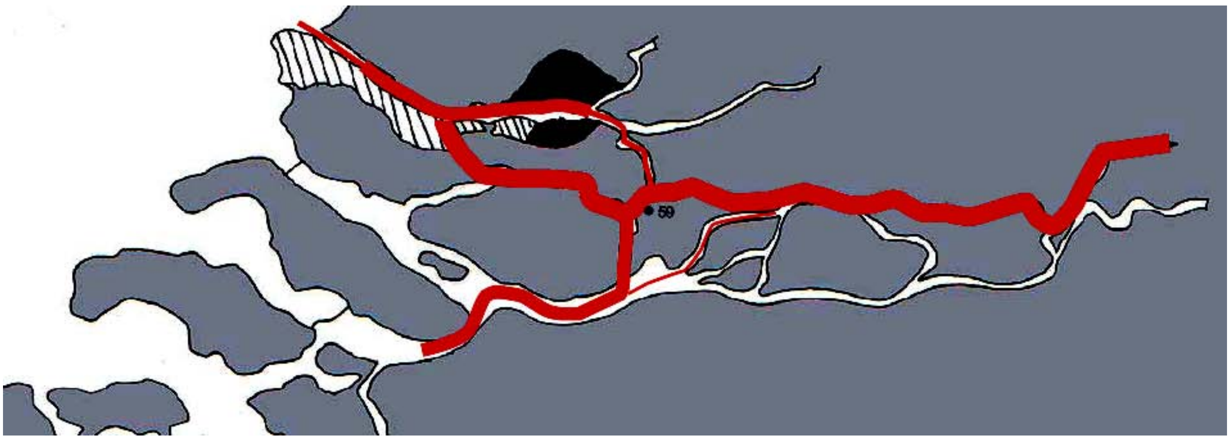


Figure 5.4: Transportation of toxic goods.

In comparison to flammable goods, the amount of toxic goods transported, is much lower. However it becomes clear that a major part of the ships carrying toxic goods use the Oude Maas. Therefore it can be concluded that the Oude Maas is an important shipping route for these goods

Other ships

Besides the ships carrying dangerous goods, there are also other ships using the river. Both commercial and non-commercial ships. By keeping the river open to the ships mentioned in the previous part, other types of ships can also use the shipping route. However there is one exception, sailing ships. Because of their high masts they need more clearance. Because of this, not all rivers in Holland are accessible for these ships. The Oude Maas however is a river at which they are allowed, because the river is a part of the so called “Standing mast”- route. This needs to be taken into account when designing a bridge over the river.

Conclusion first part

When looking at the transport on the Oude Maas, it becomes clear that the Oude Maas is a very important shipping route for flammable and toxic goods. When looking at flammable goods, especially the transport of liquids goes trough the Oude Maas, the gasses make more use of other rivers. When looking at the toxic goods, almost all ships use the Oude Maas. The absolute number of ships is relatively low, but it is clear that the Oude Maas is an import part of the toxic goods transport route.
It becomes clear that the Oude Maas should be kept open for the transportation of both flammable and toxic goods. There are alternative routes, but the Oude Maas is a too important route to shut down.
Because the river is a part of the standing mast route, the river should also stay open for sailing ships.

5.2 Discharge

The primary function of a river has been and always will be the discharge of water. The amount of water which has to be discharged depends on the amount of water which flows into the river. The amount of water flowing into the Oude Maas primarily depends on the amount of water which comes into the Netherlands from Germany.
RWS has done long term studies into the amount of water which flows into rivers and the water levels which belong to these discharges. Table 5.3 and graphs 5.5 and 5.6 give the amounts of water which come into the river system at Lobith and the amount of water which has to be discharged by the Oude Maas.

Table 5.3: Discharge figures at Lobith and of the Oude Maas.

Incoming at Lobith (m^3/s)	Oude Maas (m^3/s)	Percentage (%)
600	125	20,8
800	144	18,0
1000	175	17,5
1200	198	16,5
1700	241	14,2
2000	232	11,6
2200	227	10,3
3000	161	5,4
4000	135	3,4
5000	121	2,4
6000	111	1,9
7000	113	1,6
8000	115	1,4
9000	103	1,1
10000	105	1,1
12000	124	1,0

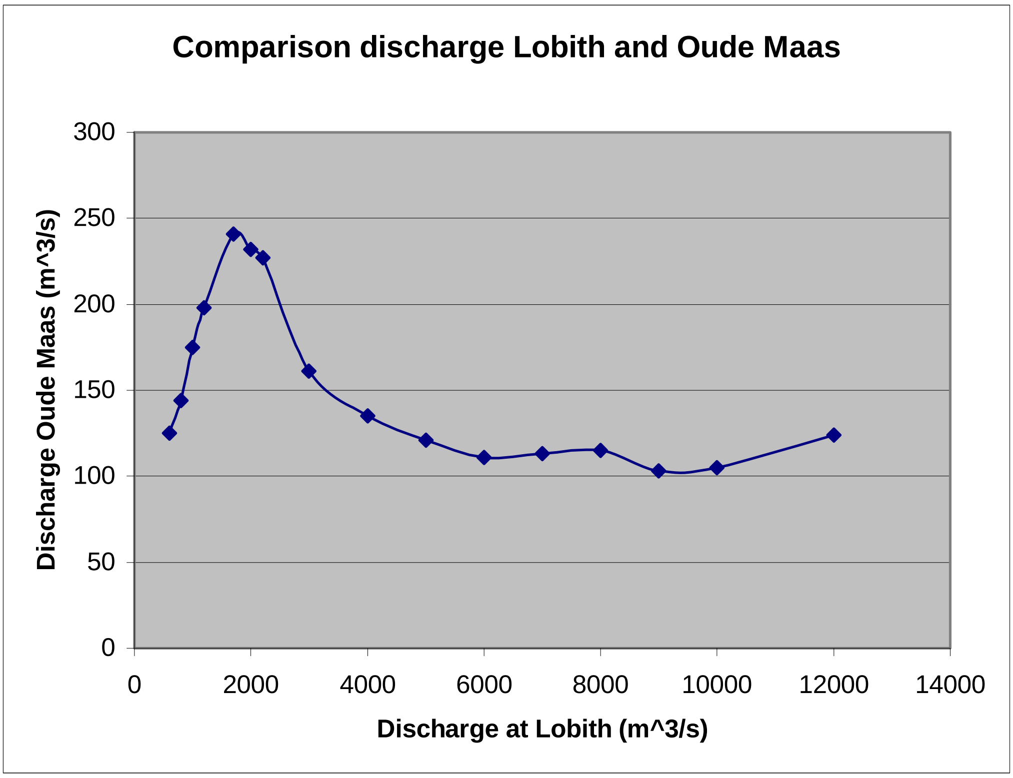


Figure 5.5: Comparison discharge Lobith and Oude Maas.

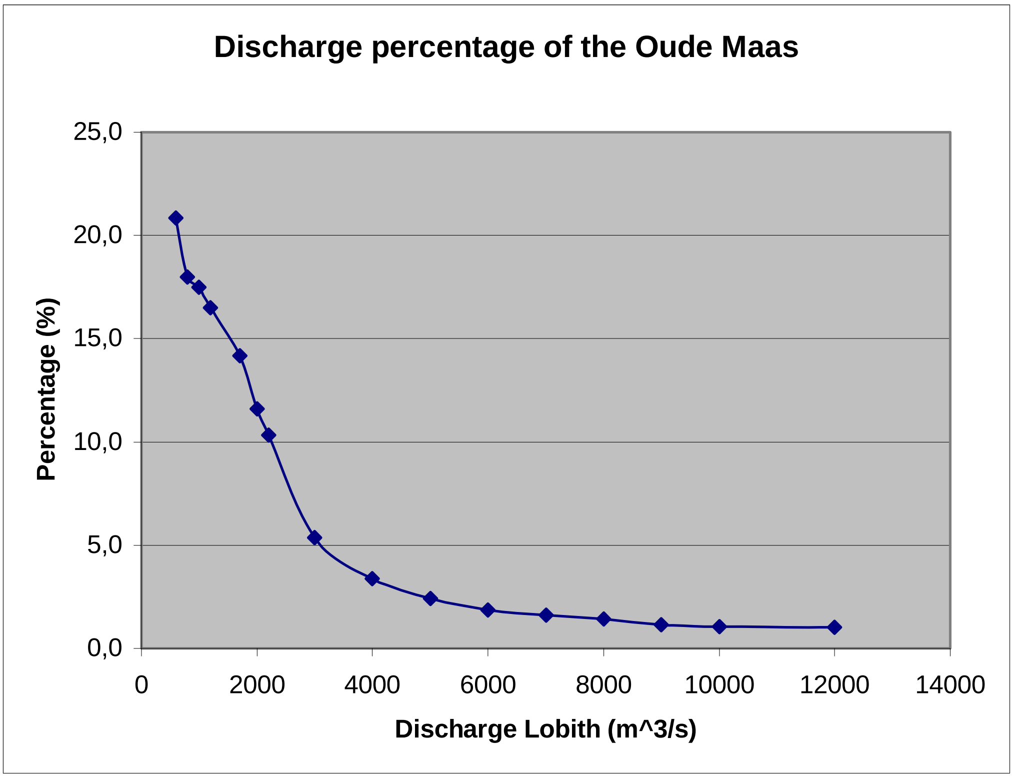


Figure 5.6: Discharge percentage of the Oude Maas.

When looking at the table and graphs it becomes clear that the Oude Maas primarily discharges water when there is a low discharge from Lobith. When the discharge at Lobith rises, the percentage of water discharged by the Oude Maas falls. This means that in flood situations, the river the Oude Maas plays a very small part in the discharging of water.
This comes in handy when looking at the design. The fewer obstacles there are in a river, the easier it becomes for a river to discharge water. However from the above it becomes clear that the Oude Maas does not have a function as a water discharging river, especially in flood situations. Therefore it will be no problem to construct in the river, however it must be kept in mind that the less obstacles in the river, the better.

Conclusion second part

When looking at the river discharges, it becomes clear that when the amount of water entering the Netherlands at Lobith increases, the percentage of water discharged by the Oude Maas decreases.
Only with low water discharges the Oude Maas functions as a discharging river. In flood situations the river looses this function, discharging only 1 – 2 % of the water.

Total conclusion

When looking at the two primary functions of a river, discharging water and acting as a means of transport, the following can be concluded:

1. The river is an important route for ships transporting dangerous goods.
2. In flood situations the river only discharges 1 – 2 % of the totally discharged water.

It must also be taken into account that the river is part of the standing mast route, which means that it will have to be possible for sailing yachts to pass the bridge.
This has consequences for the design, the river can not be blocked, because it is an important shipping route and to facilitate the sailing ships a movable part in the bridge is required. However building in the river is no problem when looking at the discharge.

5.3 Shipping channel

There are limits given by the "Richtlijnen Vaarwegen (CVB)" for the minimum dimensions of a shipping channel. The minimum dimensions depend on the size of the largest ship which has to sail trough the channel and on the location of the channel (side winds).

The general requirements are given in the figure 5.7.

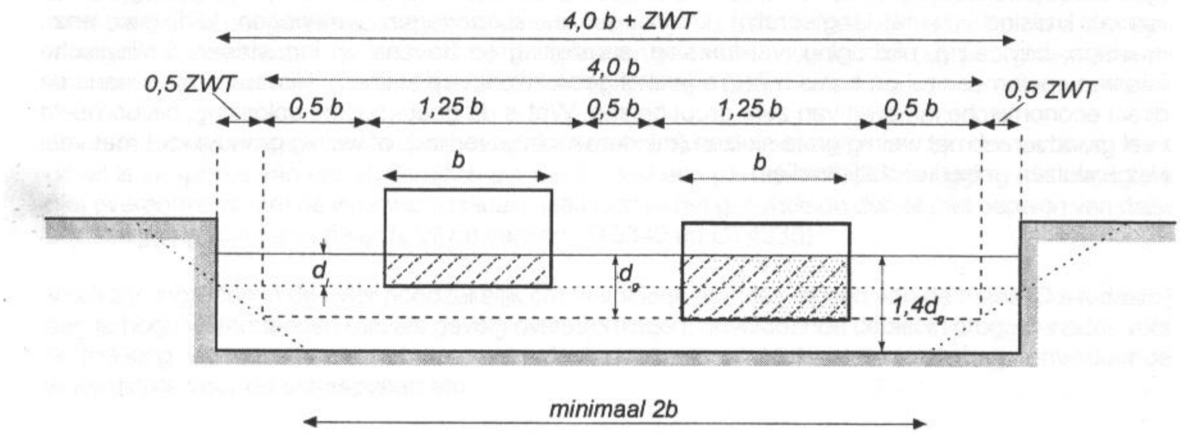


Figure 5.7: General requirements for a shipping channel

The largest ship on the Oude Maas is a "Duwstel 6 baks (wide)", this ship has the following characteristics:

Length:	200 m
Width:	34,2 m
Draught:	4,5 m
Height:	9,1 m
Weight:	18.000 ton
Mass:	27.000 ton
Block coefficient:	0.88
Speed:	18 km/h

The Drechtsteden are located in the coastal area (see figure 5.8), this means the coefficient for cross winds, ZWT = 10% * length

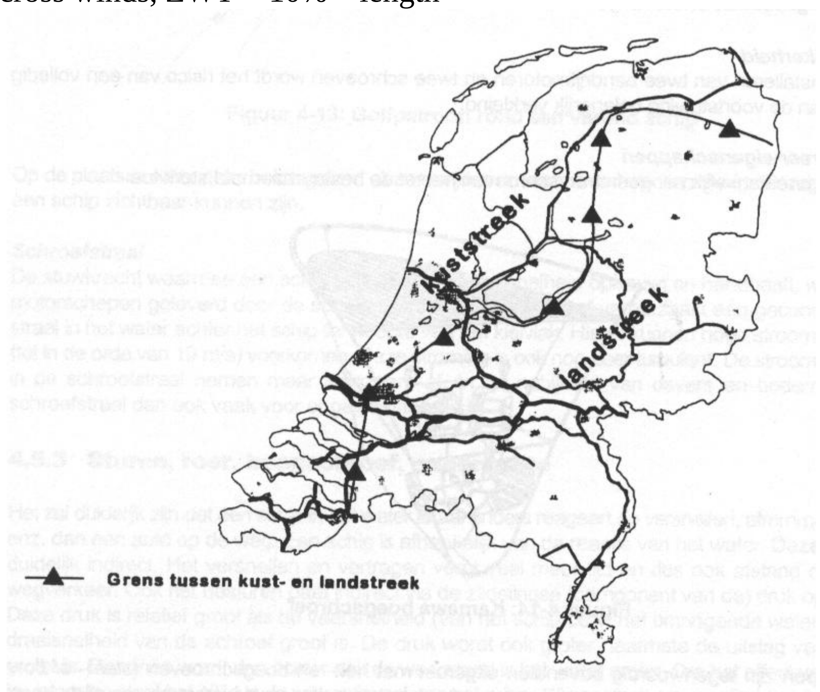


Figure 5.8: Wind area division of Holland

This leads to the minimum dimensions required for the shipping channel of the Oude Maas:

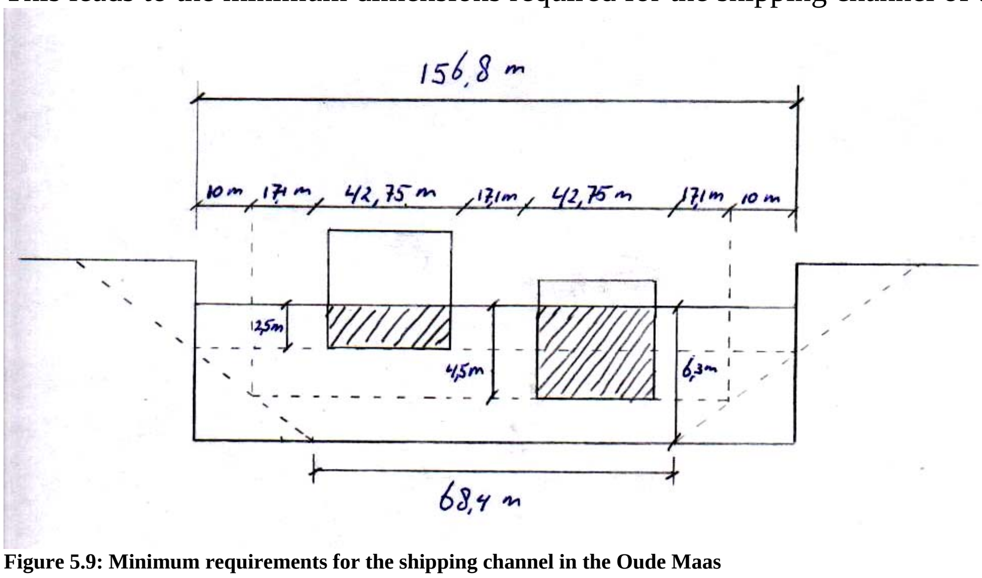


Figure 5.9: Minimum requirements for the shipping channel in the Oude Maas

The shipping channel should be at least 156,8 m wide and the depth should at least be 6,3 m.

6. Risk analysis

In this chapter an analysis will be made of the risks that occur by building on and near a river. A living bridge is an example of multiple use of space, besides advantages like the saving of space, it also has disadvantages, especially when looking at the safety. This is caused by the fact that a number of objects, with different functions, are constructed in a small area. This means that risks occurring at one construction also are a risk for the other constructions, while when creating enough space between them this risk would not hold for the other constructions.
This chapter creates insight into the risk that can occur when constructing a living bridge.

6.1 Definition of risk

A lot of definitions on risk exist, but in general it can be said they all lead to:
Risk is the chance on an event with negative consequences.
This definition itself also needs some explaining.

Chance:
This is defined as the probability of occurrence. (Van Dale dictionary)

Event:
An event / action with consequences for the construction.

Negative consequences:
There are two types of negative consequences. The first is injury or death of a person involved in the project (user, exploiter, constructer, third party). The second is material damage.
The first especially has influence on the societal acceptance of the project, because with a rising risk on injury or death people tend to make less use of an object.
The second primarily has a financial origin. In the case damage to a construction is done, which prohibits the use of the construction, the exploiter will lose revenues. And of course there are also costs for the repairing / replacing, or in case this is not possible, demolition of the construction.

Combining all the above, it can be concluded that risk in civil engineering means:
The probability of an event which prohibits safe use of the construction.
Therefore risk is often expressed as chance * consequence, the later is mostly expressed in money.

6.2 Normal bridge

Looking at a “normal” bridge there are three types of safety, internal, external and between groups of users (see figure 6.1).

Internal safety (A)
This concerns the safety of the users of the construction. For example when an accident occurs in the traffic on the bridge, the safety of the other traffic on the bridge needs to be guaranteed.

External safety (B)
This concerns the safety of people in the vicinity of the construction. For example when an accident occurs on the bridge, this should pose no risk for the people on the shore.

Safety between groups of users (C)
This concerns the safety of a second group of users in case a calamity occurs in the first group of users. For example when an explosion on a ship under the bridge occurs, the safety of the people on the bridge should be guaranteed.

The relations between safety and risk can be depictured as followed:

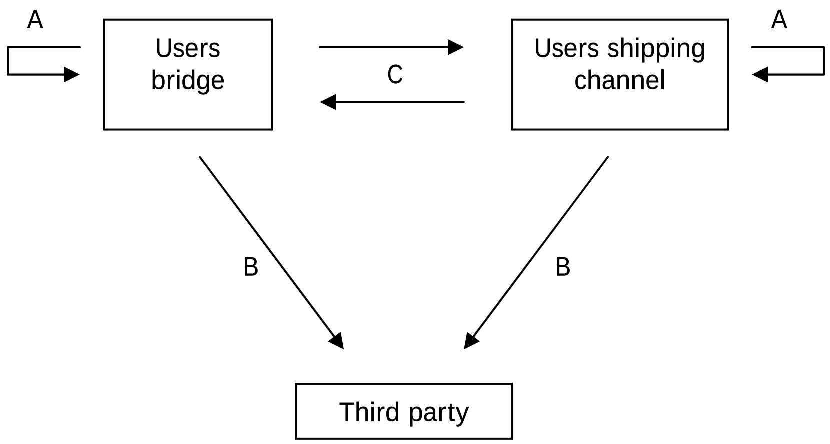


Figure 6.1: Safety relations "normal" bridge.

6.3 Living bridge

In the case of a living bridge there is an extra group of users, namely the people who reside in the therefore designated areas (shops, theater, etc.). In this case holds the same as in the previous case, there are three types of safety, internal, external and between groups of users.

The scheme can now be expanded to:

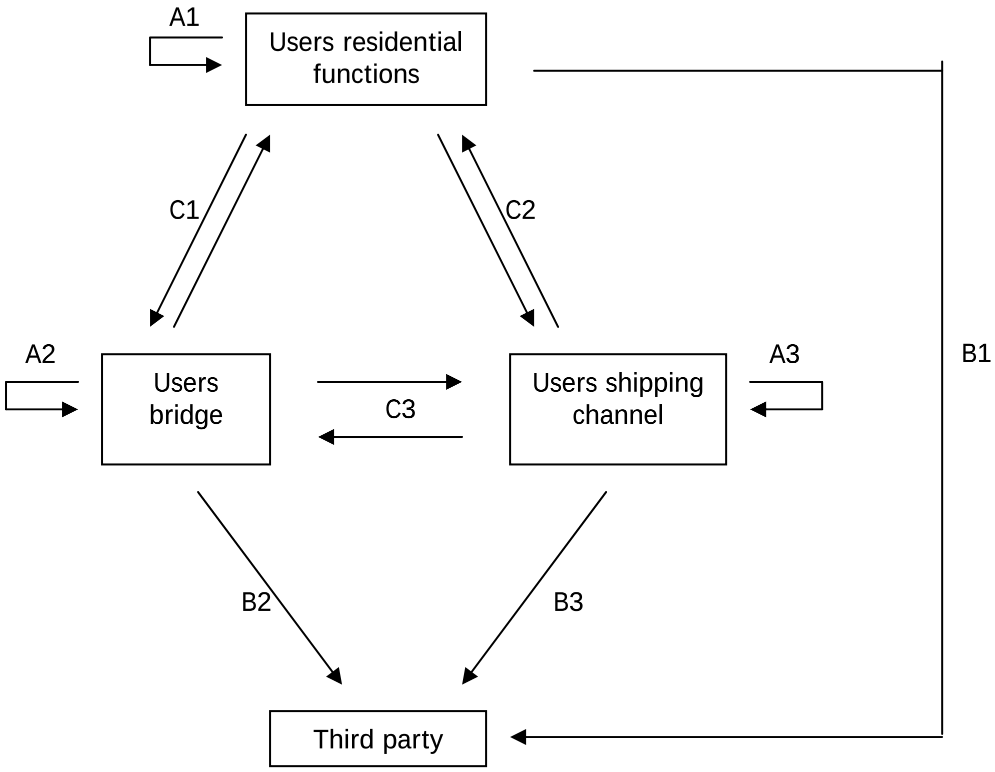


Figure 6.2: Safety relations living bridge.

Life phases

The risk also depends on the life phase of the bridge and its buildings. During the building phase there are other risks than during the exploitation phase.

Construction phase

A1. Internal safety residential areas
During the construction phase this concerns the safety of the people present at the building site. The risks depend on the design and the execution of it. However there are sufficient rules in the Netherlands to guaranty the safety of the construction workers during the construction of the bridge and buildings.

A2. Internal safety bridge
The same as for A1 holds in this case. The risks depend on the design, but there are sufficient rules to guaranty safety.

A3. Internal safety shipping channel
There is a possibility the shipping will have hindrance of the construction, for example a narrowing of the channel. It is necessary to reduce this hindrance to a minimum, because it does not only pose a treat for the construction, but also for the shipping itself. Because the hindrance may lead to a calamity.

B1 & B2. External safety
It is of course dangerous for a person from outside to come near a construction which is being build. Therefore it is necessary to make a clear separation between the construction site and the rest of the area. The placing of fences is an example.

C2 & C3. Safety between groups of users
The construction of a bridge and / or buildings across a shipping channel creates huge risks, both for construction site and the shipping, but they are also a risk for each other. For example a narrowing of the shipping channel increases the risk of a collision. Or a falling object from the construction can damage a ship or person. Therefore it is necessary to keep the shipping channel open for ships and limit the number of narrow places. Making use of prefabricated parts is an example of reducing this.

Exploitation phase

A1. Internal safety residential areas
The availability of these places for emergency services, in case of a calamity, needs to be kept in mind. Also the availability and marking of escape routes needs to be clear. Especially if a calamity in a residential area on the bridge occurs, it needs to be clear what the easiest and safest escape route is.

A2. Internal safety bridge
The bridge is for slow traffic only, this means only pedestrians and cyclists can use the bridge. Therefore a clear separation between the part for pedestrians and the part for cyclists is required. The safety is influenced by the bridge dimensions. A narrow bridge causes collisions, but a very wide bridge can cause people to stop and enjoy the view, and so create an obstacle for the rest of the traffic.
There is also a safety measurement required for the movable part of the bridge. A clear separation between the movable part and the non-movable part / river bank is required. Possible solutions are bars closing before the bridge opens and light or sound signals.

A3. Internal safety shipping channel
The new living bridge raises the risk of calamities. The channel will probably become narrower and the bridge pillars will form obstacles for the ships. By reducing the maximum speed near the bridge, the risk of a calamity is reduced.

B1. Safety third party vs. users residential areas
In the case a calamity occurs in one of the residential areas, the safety of the third party needs to be guarantied. For instance when a shop catches on fire, it can not cause a house nearby to catch fire. The Dutch government has clear regulations, by complying the constructions to these regulations the safety is guarantied.

B2. Safety third party vs. users bridge
In the case of a calamity on the bridge, this may not lead to a unnecessary risk for people living nearby. Because the bridge is a bridge for slow traffic this risk is small. Only when the traffic leaves the bridge there is a risk, by making a clear separation between the different types of traffic this risk is reduced.

B3. Safety third party vs. shipping channel
This risk is already present, because there already is a transport of dangerous goods (see chapter 5.1). Because of the narrowing and the new obstacles in the river however this risk has become larger. This leads to a change in the risk contours for this area.

C1. Safety bridge users vs. users residential area and vice versa
An accident in the traffic on the bridge could have consequences for the users of the residential areas. Therefore it is necessary that there is a variety in the escape routes. The other way around it holds that a calamity in one of the residential areas can have consequences for the traffic. Therefore not al escape routes can go via the traffic lanes of the bridge.
Because of the fact that the construction of the residential areas (partially) forms the main bearing construction, extra attention is required with regard to progressive collapse.

C2. Safety users residential areas vs. shipping channel and vice versa
In the case a calamity occurs on the shipping channel, the users of the residential areas need enough time to leave these areas and get to a safe area. The Dutch government has made a set of clear rules about the distance between buildings and a shipping channel, these rules indicate zones in which it is allowed and in which it is not allowed to build.
These same zones guaranty the safety for the shipping channel in case a calamity occurs in one of the residential areas.

C3. Safety bridge users vs. shipping channel and vice versa
Here it holds that an accident on the shipping channel may not pose progressive danger for the traffic on the bridge. Therefore enough time and space is required to clear the bridge. The other way around the same holds, an accident on the bridge should have minimal consequences for the ships in the channel.
Also the possibility of falling / throwing of objects from the bridge need to be kept in mind.

6.4 Accidents on the Oude Maas

As stated in paragraph 5.1 there is a large number of ships which use the Oude Maas. Among these ships there are a number of ships which transport flammable and toxic goods along the river the Oude Maas. This of course brings certain risks along, such as collision, fires aboard ships or explosions. Data about accidents is kept by ONOVIS. They determine the frequency of ships using the river and the number of accidents which happen on a river section. When keeping track of the accidents they classify the accidents into six groups. Depending on the amount of damage the classification is made from 0 (no damage) to 5 (Damaged hull, loss of cargo). A class 4 accident means heavy damage, but no loss of cargo.
In total 43 accidents happened on the Oude Maas in the period 2000 – 2004. 38 Of these accidents where a collision between ships or between a ship and infrastructure. 6 Accidents where classified as class 4 or 5 accidents.
With this data the risk was determined, this risk was expressed as a distance to the centre of the shipping channel. The most interesting line is the 10^{-6} contour, because this line forms the boundary within no buildings are allowed. The Risico atlas places this contour on the bank of the river, this means no buildings are allowed in the river. This has consequences for the living bridge, because this means that no buildings with a residential function can be placed in the river.

7. Alternatives for design

With the location and the limitations for the design know, alternatives for the design can be developed. In this chapter an overview of these designs is given. First however some extra characteristics of the location are given. These will serve as extra boundary conditions for the designs. At the end of the chapter a choice will be made between the alternatives.

7.1 Introduction

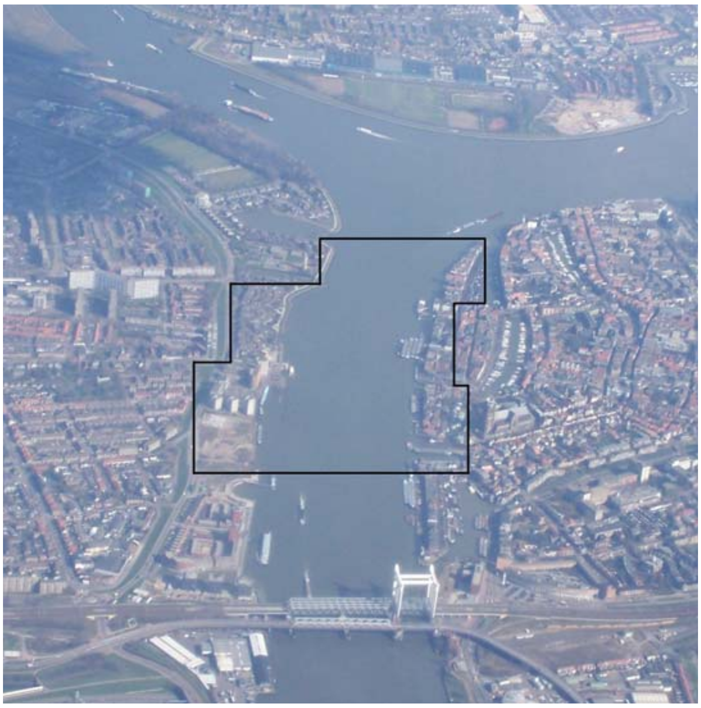


Figure 7.1: The design area.

Design area

As stated in paragraph 2.2 the design area is located on the banks of the Oude Maas, north of the railway bridge and south of the Merwede. The river has a width of about 300 meters at this location. On the river there is a lot of heavy transport especially of dangerous goods. Because the river is classified as a main shipping route, the river will have to stay open for the shipping traffic. The river also is a part of the standing mast route, which means the clearance height of the bridge will not be standard Rijn height, because this is not enough for the sailing ships. Part of the bridge will have to be movable to allow sailing ships to pass.

Harbours

On both the Dordrecht and the Zwijndrecht bank of the river, there are several harbours (see figure 7.2). These harbours still have to be accessible in the new situation. Therefore the locations of the entrances for these harbours have to be taken into account when designing the new living bridge. Also the turning circles of the ships have to be taken into account, to make sure a ship can enter and depart from a harbour in the angle required for safe shipping.

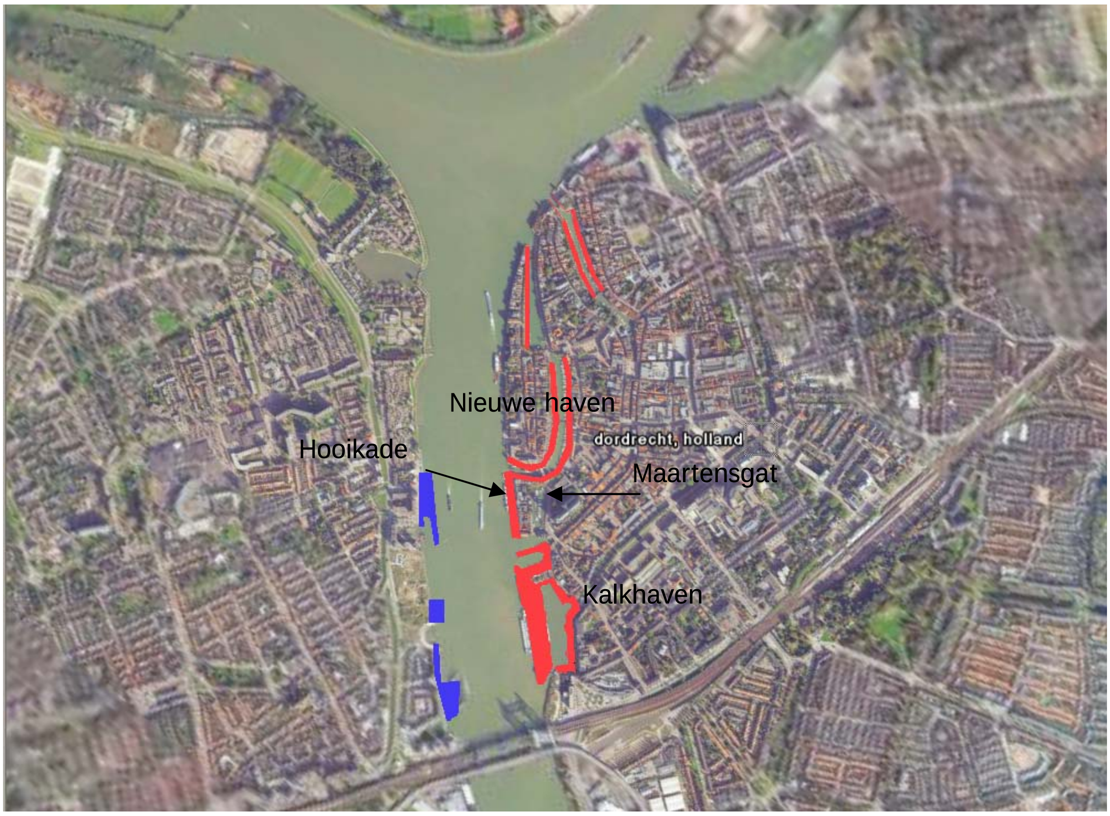


Figure 7.2: The harbours on both side of the Oude Maas.

7.2 Bottom profile

At the design location, almost the whole width is available for shipping. In the new situation with probably one or more pillars in the river, it is, for safety reasons, required to make the shipping channel smaller. In paragraph 5.3 the minimum dimensions for the shipping channel were calculated, these were:

- Width: 156,8 m
- Depth: 6,3 m

To make sure these requirements are met, it is necessary to look at the bottom profile of the river, both in the designing area and in the adjoining areas. These can best be done by making cross sections of the river. To do this, bathymetry survey’s (see figure 7.3) from RWS have been used.

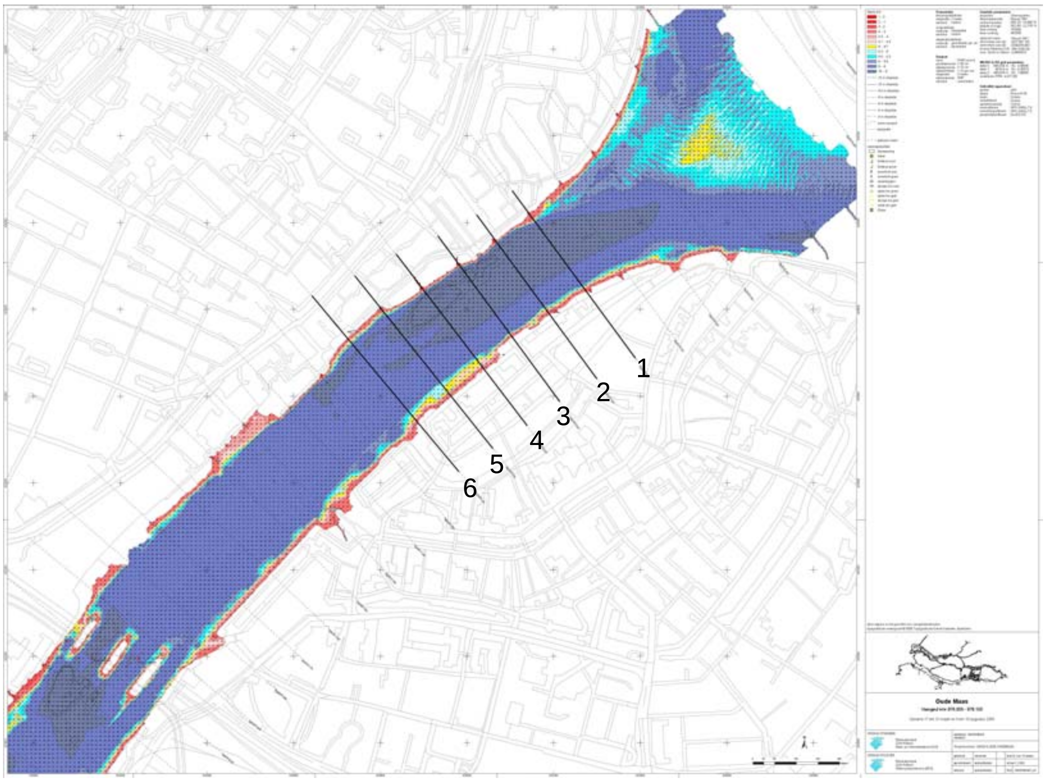


Figure 7.3: Bathymetry survey.

The following figures (7.4 to 7.9) show the bottom profiles in these areas. In each figure 1 cubic is 10 m in width and 1 m in depth.

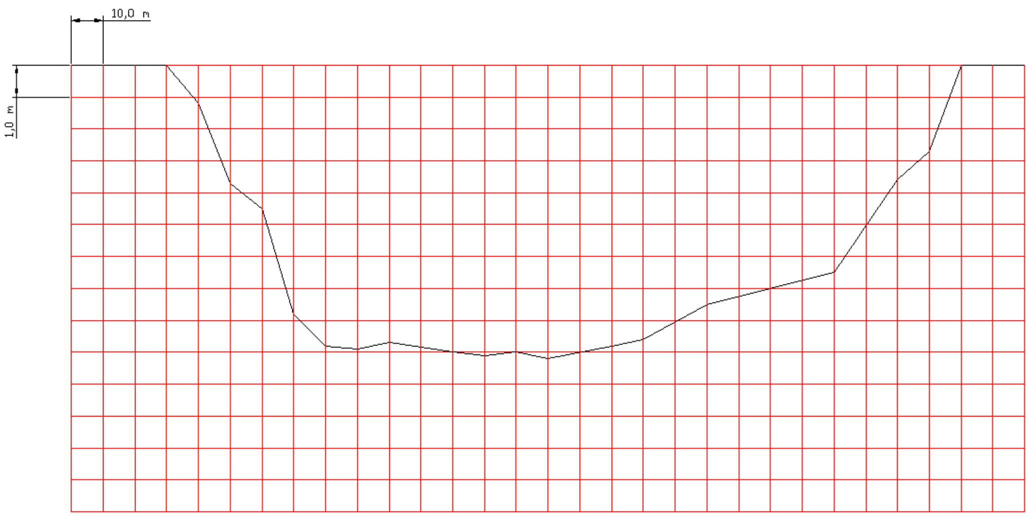


Figure 7.4: Cross section at line 1.

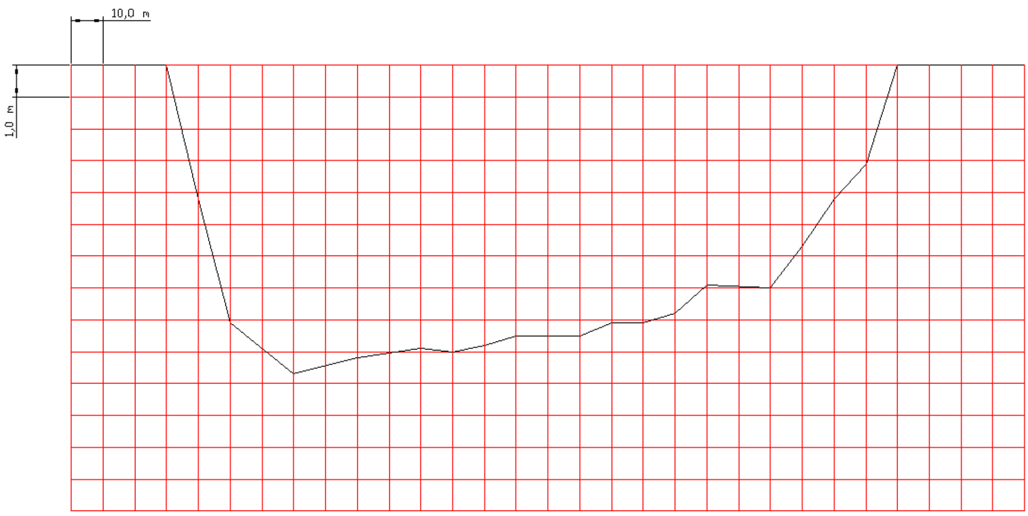


Figure 7.5: Cross section at line 2.

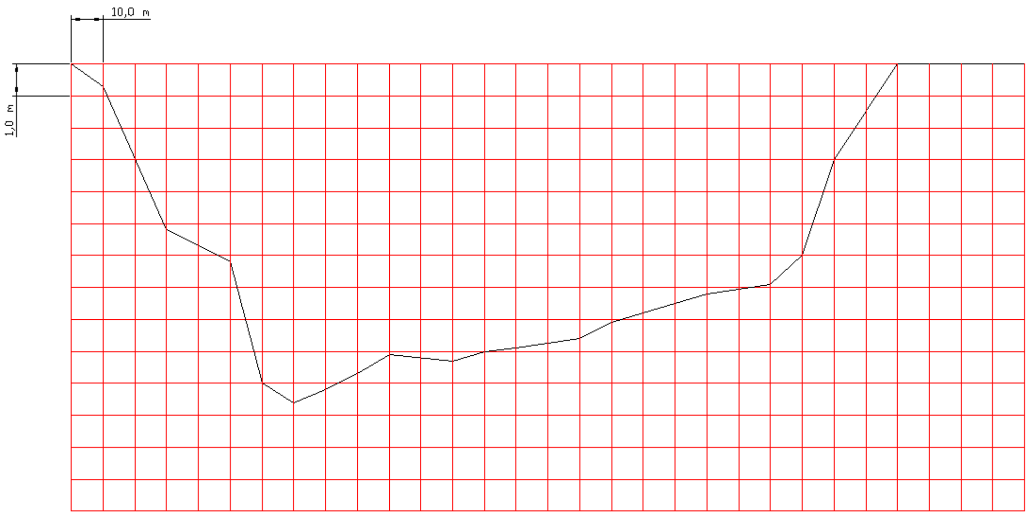


Figure 7.6: Cross section at line 3.

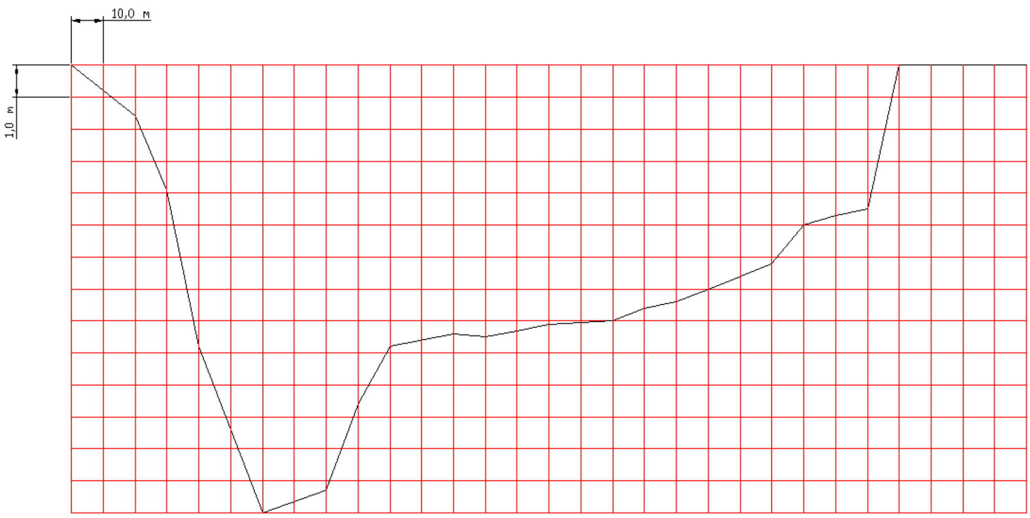


Figure 7.7: Cross section at line 4.

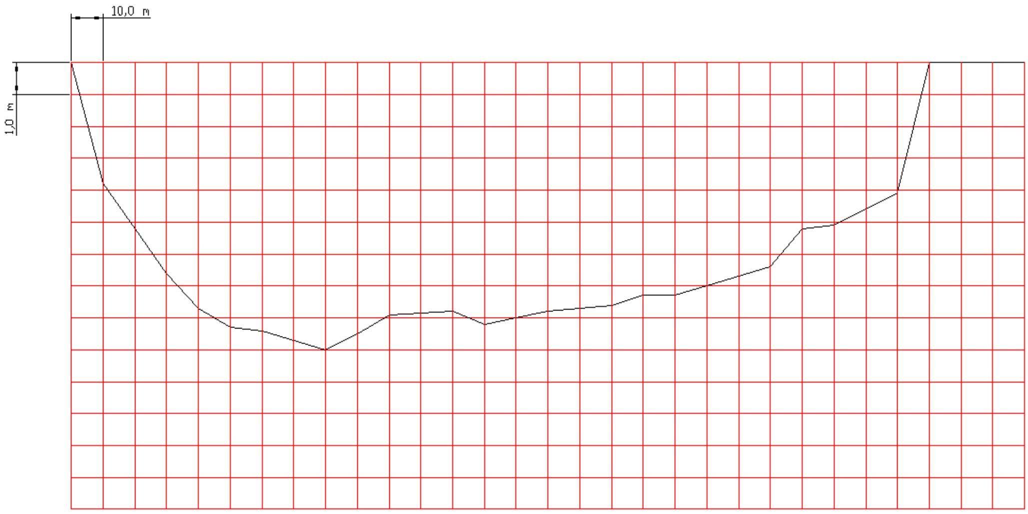


Figure 7.8: Cross section at line 5.

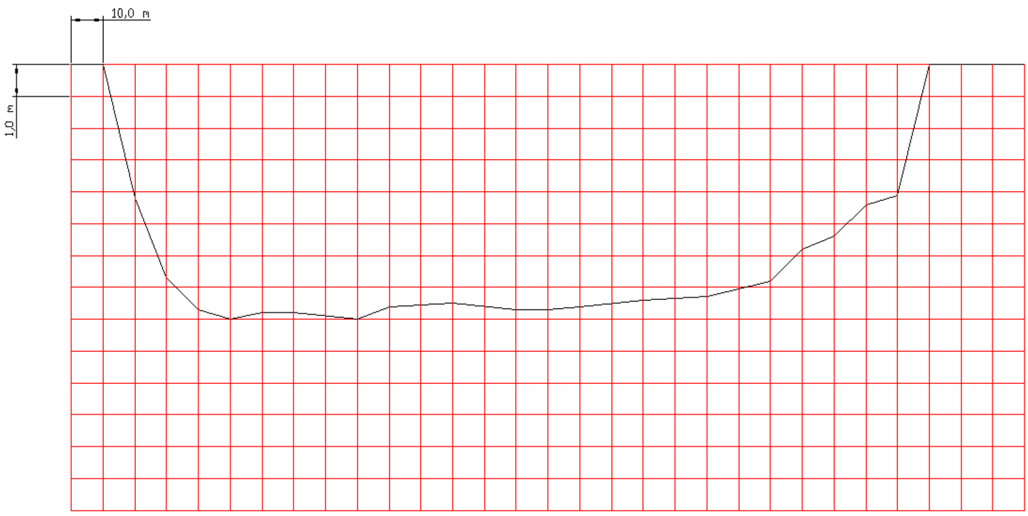


Figure 7.9: Cross section at line 6.

When looking at the surveys it becomes clear that the average depth is about 7 meter. Which is sufficient for the shipping channel. Only near the lines 3 and 4 (north of the Hooikade) the river becomes deeper on the Zwijndrecht side of the river. It is obvious that near the banks the river is shallower.
This means for the new design, that the shipping channel will have to remain as close to the centre as possible, because the river has its largest depths there, only north of the Hooikade it is possible to shift it more to the Zwijndrecht bank, because the river is deeper there.

7.3 Soil data

To determine the type of foundation required and the dept at which this should be situated soil data from the building site is required. However no data is available for the exact location is available. Therefore data from a location nearby is used to get an indication of the composition of the soil and the dept at which a layer to place the foundation on is situated. The data was acquired by Geomet in assignment of the city of Dordrecht. Two tests were done.The tests were done according to NEN 5140.
The location of the test is given in figure 7.10. In this figure the location of the building site for this thesis is also given.

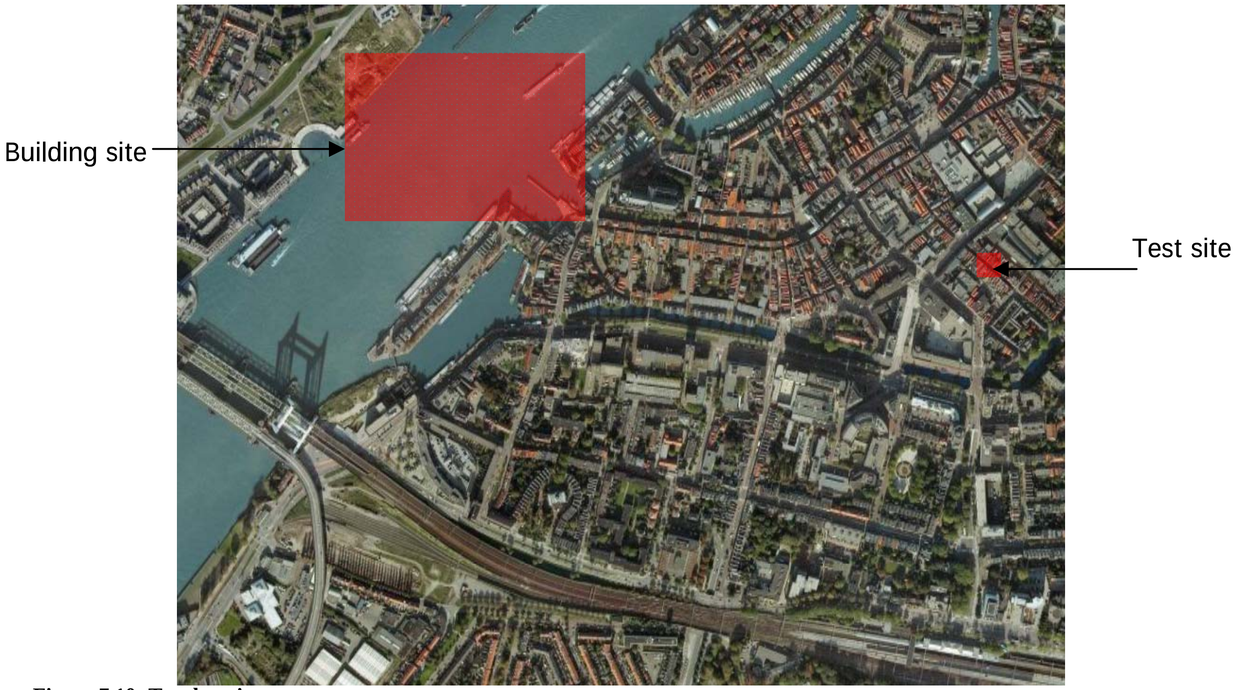


Figure 7.10: Test location.

The ground level of the tests is placed at +1,67 m NAP.

From the data (see appendix 1) it becomes clear that the first layer which is strong enough to place the foundation on is located at -12,5 m NAP. This is the depth at which the foundation for the living bridge will be placed.

7.4 The alternatives

A number of alternatives have been designed for the new living bridge over the Oude Maas. This was done by students of the faculty of Hybrid buildings and urban architecture, Delft. From these alternatives three were selected. The alternatives are presented in the next paragraph. From these alternatives the best option for further study will be selected. From the chosen alternative a number of variants will be made.

The three alternatives are:

- Alternative 1: Cable-stay bridge (original design by T. Kramer)
- Alternative 2: The Knot (original design by I. Oosterbaan)
- Alternative 3: Swimming in the Oude Maas (original design by M. van der Meulen)

Alternative 1: Cable-stay bridge

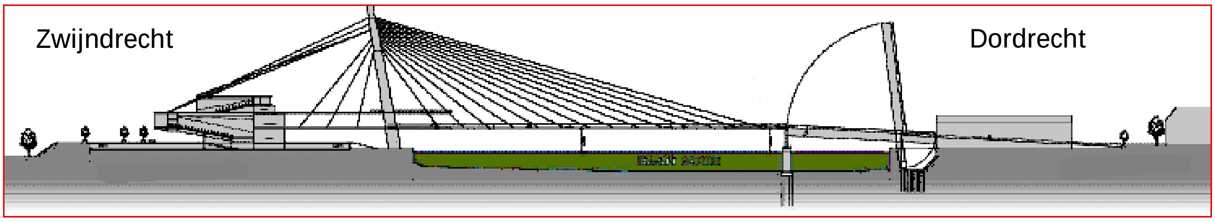


Figure 7.11: Alternative 1: Cable stay bridge

The bridge consists of two parts, a non-movable cable stay bridge and a movable bridge, a beam with a bascule basement or a cable-stay bridge on the side of Dordrecht. One pillar is constructed in the river on the Dordrecht side of this river. The cable stay bridge has Rijnvaart height, making it possible for ships in the shipping channel to sail trough under the bridge. This height is not sufficient for the sailing ships, therefore a movable bridge is made on the side of Dordrecht, by opening the bridge, no more height limitations exist and sailing ships can sail to the other side of the river crossing.

On the Zwijndrecht bank a large building with different functions is created. The most important functions are:

- Movie theatre
- Theatre
- Parking garage

On the Dordrecht bank another building is created, with the following functions:

- Shops
- Apartments
- Restaurant

By anchoring the cable-stay bridge on the building on the Zwijndrecht side, bridge and building are connected to each other and visually look as one, making this bridge a living bridge. This also has a constructional advantage, no heavy extra anchors have to be created on the bank.
On the Dordrecht side a ramp is created to gain the required height. This ramp is placed alongside the building. By not making this building a high one, most attention is drawn to the cable-stay bridge, making this living bridge an eye-catcher.

Alternative 2: The Knot

In this alternative the living bridge over the Oude Maas is part of a bigger plan, also connecting the two cities with Papendrecht, lying to the north of these two cities. Therefore not only space is created for pedestrians and cyclists, but also space is created for a tramline.

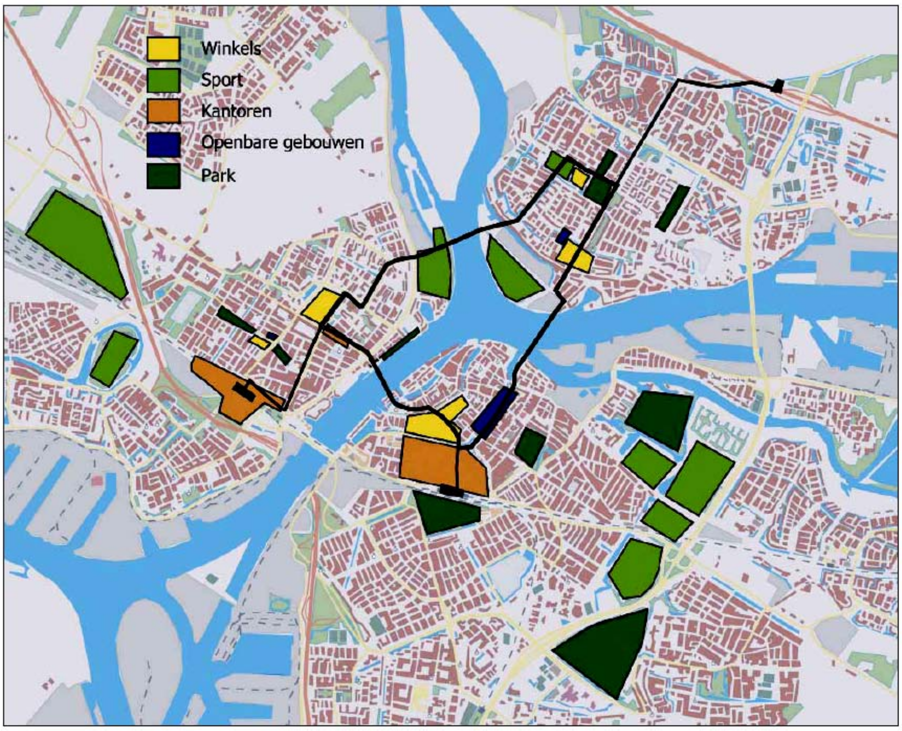


Figure 7.12: The total plan, including Papendrecht into the design.

The bridge over the Oude Maas is constructed by making a truss structure which spans the river.

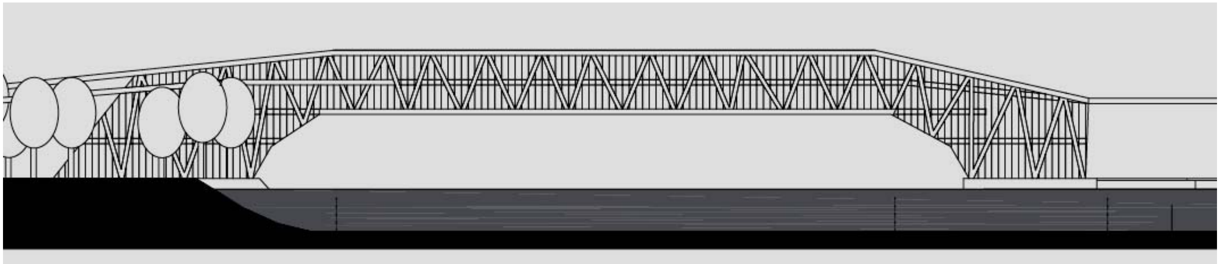


Figure 7.13: Alternative 2: The Knot.

On the bottom level the walkway for pedestrians is created. This will be spacious, making it pleasant to stay on the bridge for a while and enjoy the view.
The second level will be the level for cyclists. This level is again spacious, making it possible to create storage space for the bicycles and creating enough room, for both slow and fast cyclists.
The top level is for the tramline, also a stop will be created, making it possible for tourists to get out of the tram on top of the bridge and to enjoy the view.

The buildings are placed on islands in the river and will provide space for markets. These will be covered markets, but the sides of the buildings will be kept as transparent as possible, to visually create a unity with the environment.

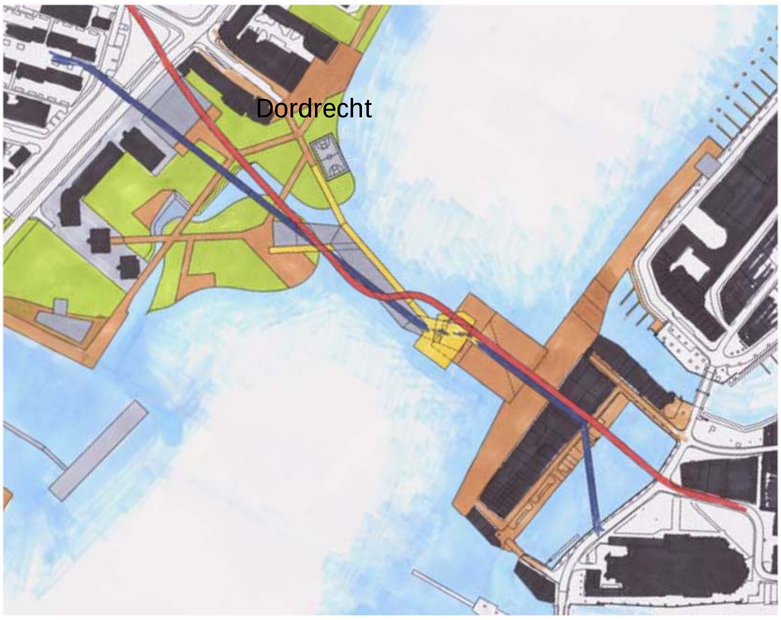


Figure 7.14: Top view of alternative 2.

The highest point near the new bridge is the old church. By making the new bridge lower than the top of the church, both objects keep there appearance as an eye-catcher.

The island on the Dordrecht side of the river is connected to the mainland by a movable bridge, making sure sailing ships can get past the bridge.

By integrating a space with commercial functions on both sides of the bridge into the design, this bridge becomes a living bridge. Forming the connection between the new city centres built on both sides of the river.

Alternative 3: Swimming in the Oude Maas

In this design the emphasis is on bringing the cities back to the water. This is done by creating new land near the banks. Between these two spaces a living bridge is build to cross the shipping channel.

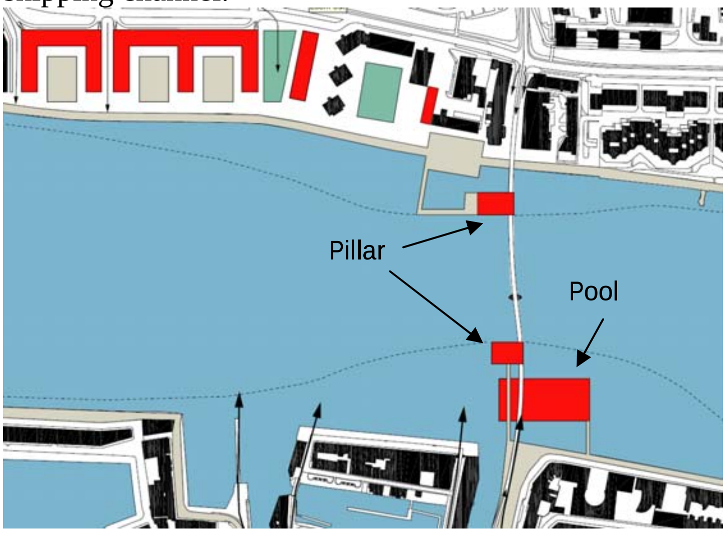


Figure 7.15: Top view area planning.

The bridge is a slender construction, which forms the connection between two of the main focus points of this design, the pillars. The other focus point is a floating pool.

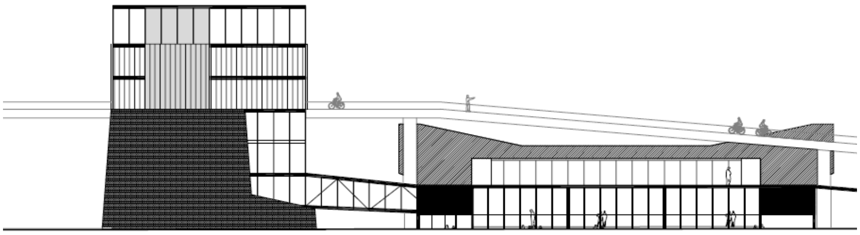


Figure 7.16: Pillar and floating pool, in the Dordrecht side of the river.

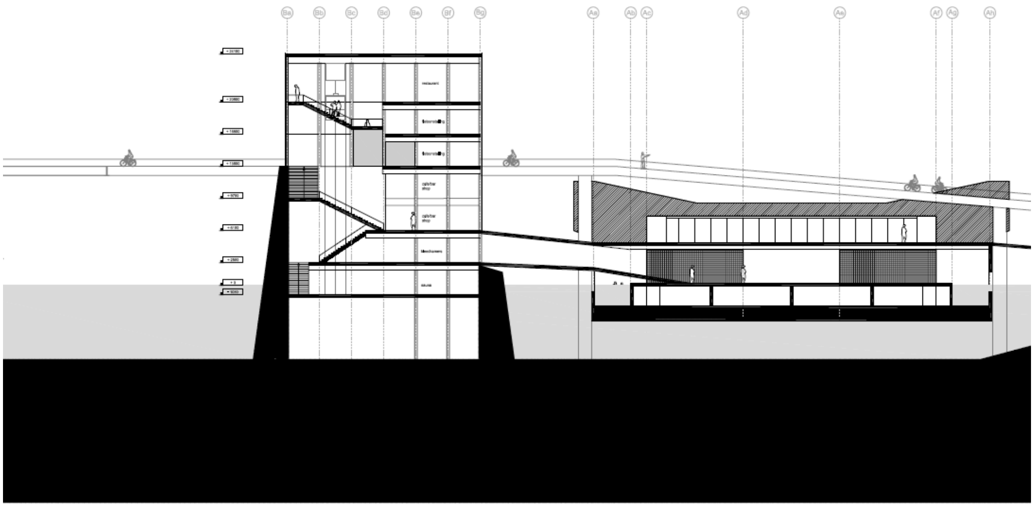


Figure 7.17: Cross-section of the pillar and pool.

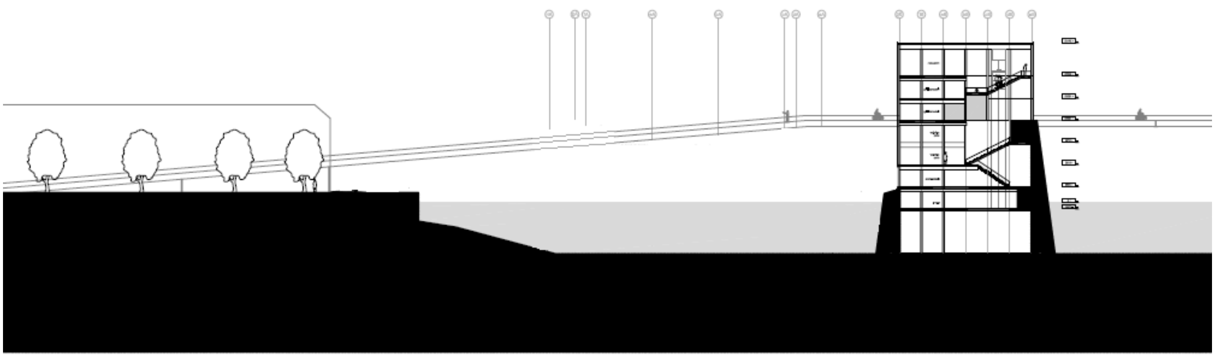


Figure 7.18: View of Zwijndrecht side of river.

The buildings on the pillar will be very transparent, giving it a contrast with the foot of the pillar. In the upper levels a restaurant will be housed, giving people a nice view over the cities while having lunch or diner. In the lower levels a health spa will be created.
The pool construction will be floating in the river, kept in place in the horizontal plane by pillars. The top of the pool will be covered, creating a park. Because this construction is floating, the water level will appear constant to people in the park. The pool itself is a 50 meter pool, with enough space around it for people to relax. Because the pool has a transparent bottom, it will appear as if people are swimming in the Oude Maas.

On the Zwijndrecht side, a same pillar will be constructed. In this pillar economical functions will be housed.

The bridge between the pillars will be a slender construction, forming a connection between the two cities. The bridge will be designed for both pedestrians and cyclists, ensuring enough space for both.

7.5 Choice of alternative

From the three alternatives presented in the previous paragraph a choice has to be made for an alternative which is used in the rest of this study. This alternative will be the basis to derive a number of variants from. One of these variants will be chosen for the definitive design for the living bridge.
The choice for the alternative which is used in further study is based on a number of considerations:

- Work done by students of the faculty of architecture
- Possibilities for further development of the alternative
- Discussion between student and the supervisors

The second alternative (The knot) has a large part, namely the connection to Papendrecht and a light rail connection, which is outside the scope of this study. It also contains plans to build islands in the river, making it very difficult to maintain a safe shipping channel in the river. Therefore this alternative is discarded for further study.
On both alternative one (Cable-stay bridge) and three (Swimming in the Oude Maas) some structural design was done, but not much. During the discussion between supervisors and the student it became clear that both alternatives were suitable for further study. However the functions dedicated to the buildings in alternative 3 are not according to the original list of requirement. And the design also contained a large part which was placed in the river, however due to the risk placing a building in the river is not allowed.
The original list of requirements (theatre, parking space, etc.) was made, to create a feasible plan in creating a new city centre. The first alternative: Cable-stay bridge complies the best with this list of requirements. This plan was also favoured by both student and supervisors as a basis for further study, therefore the first alternative: Cable-stay bridge is used for further design.

8. Variants on chosen alternative

The chosen alternative is alternative 1: Cable stay bridge. The basis of this design is a cable-stay bridge on the Zwijndrecht side of the river and a movable bridge on the Dordrecht side. The main shipping channel runs between the pylon and the pillar which supports both the movable bridge and one end of the cable-stay bridge. As stated earlier (paragraph 5.3) the minimum dimensions of the shipping channel are, width 156,8 m and the depth 6,3 m.
At the design location the river has a width of 270 m and the shipping channel at this location has a width of 192 m. This means that there is the possibility to construct in the current shipping channel.

In the original alternative there is a second cable-stay bridge as the movable bridge. However this brings a lot of difficulties along, especially when the bridge is opened. In this situation the stresses will be different from the stress in the normal closed situation, and this will not only influence the stress in the deck, but also the stress in the cables. One of the design criteria for a cable in a cable-stay bridge is that there is always a tensile stress in the cable. This can not be guaranteed, therefore other options for the movable part of the bridge will be shortly discussed in this part of the study.

8.1 Location shipping channel

Before creating the variants a closer look at the location is required.

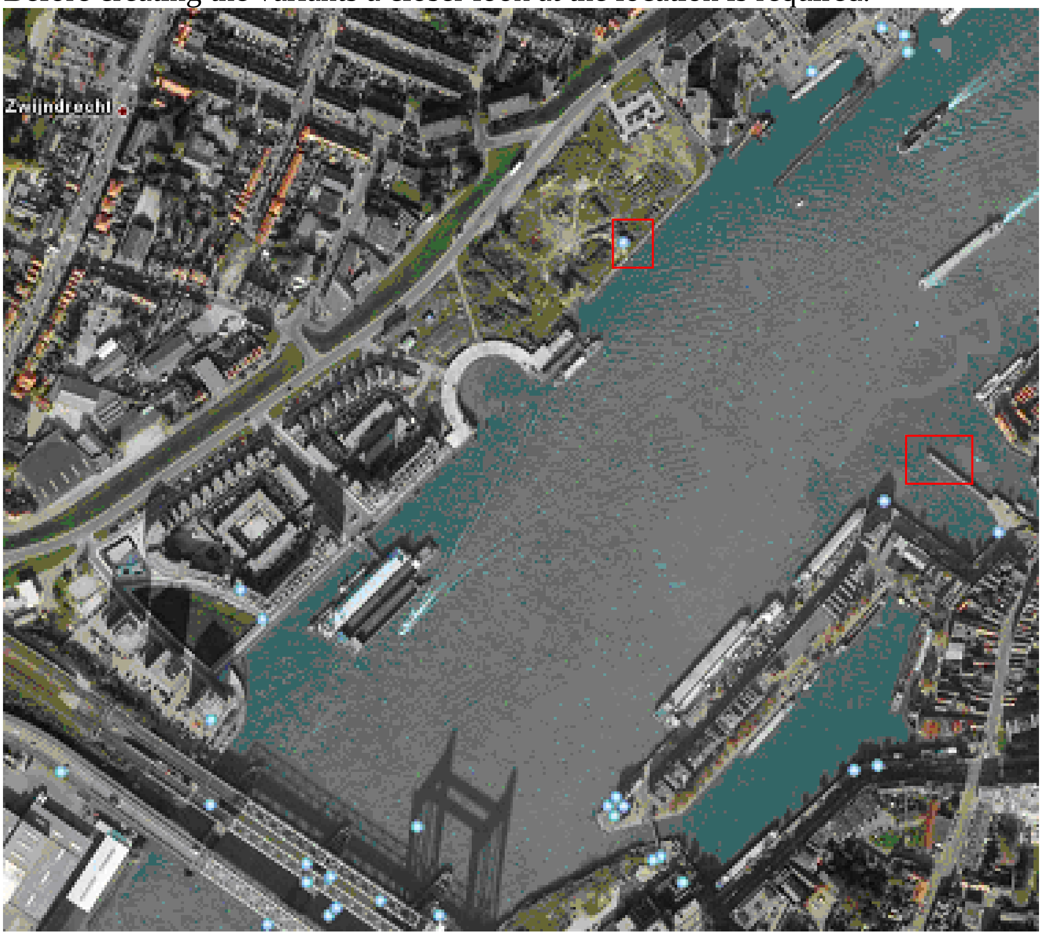


Figure 8.1: Aerial photo of the design area.

In the aerial photo both the begin and end point of the bridge are represented by the red squares. It becomes clear that on both sides there is enough space available to create both the bridges and the building. Both this only represents halve the required space for this construction. Also the space available in the river requires looking at. Before this can be done, the dimensions of the cross-section required for sailing yachts needs to be examined.

Dimensions movable part

To design the bridge for the movable part, the dimensions of the design vessel are required. The design characteristics of a sailing yacht are:
Length: 12 m
Width: 4,0 m
Depth: 1,9 m

This means that the shipping channel for the sailing ships will have to be:
$Width_{channel} = 4 * width_{yacht} + ZWT = 4 * 4,0 + 0,1 * 4,0 = 16,4$ m
$Depth_{channel} = 1,4 * depth_{yacht} = 1,4 * 1,9 = 2,7$ m

With the dimensions of both the main shipping channel and the sailing yacht channel known, a rough design of the area available for the Construction can be made.

Zwijndrecht Dordrecht

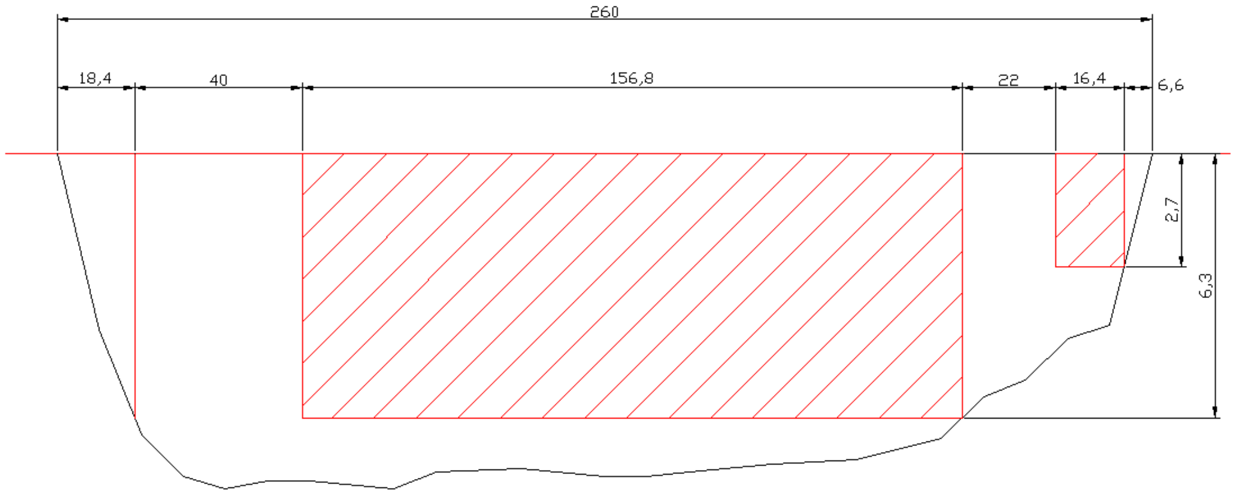


Figure 8.2: Cross-section of river bottom with the area required for the shipping channels.

As becomes clear from figure 8.2 there is enough space for both shipping channels in the cross-section of the river. However, these locations are not fixed, because they can both be moved more to the side of Zwijndrecht.
In the original design the pylon is placed on the bank on the Zwijndrecht side of the river, from the drawing it becomes clear that the space between the bank and the edge of the shipping channel is about 60 m. This makes it an interesting option to place the pylon in the river, creating a shorter span. However, there is a downside to this option, building in the river creates an extra obstacle for both the discharge of the river and for ships to come into collision with.

Now that the space available for construction both on land and in the river are known, the variants on the chosen alternative can be created.

8.2 Variants bridge

First a number of variants on parts of the bridge structure will be discussed.
When creating the variants for the cable-stay bridge, the following options need to be taken into consideration:

- There can be chosen between one row with stay-cables or two.
- There is a wide variety of shapes available for the pylon, however there are a number of main shapes, depicted in figure 8.3.

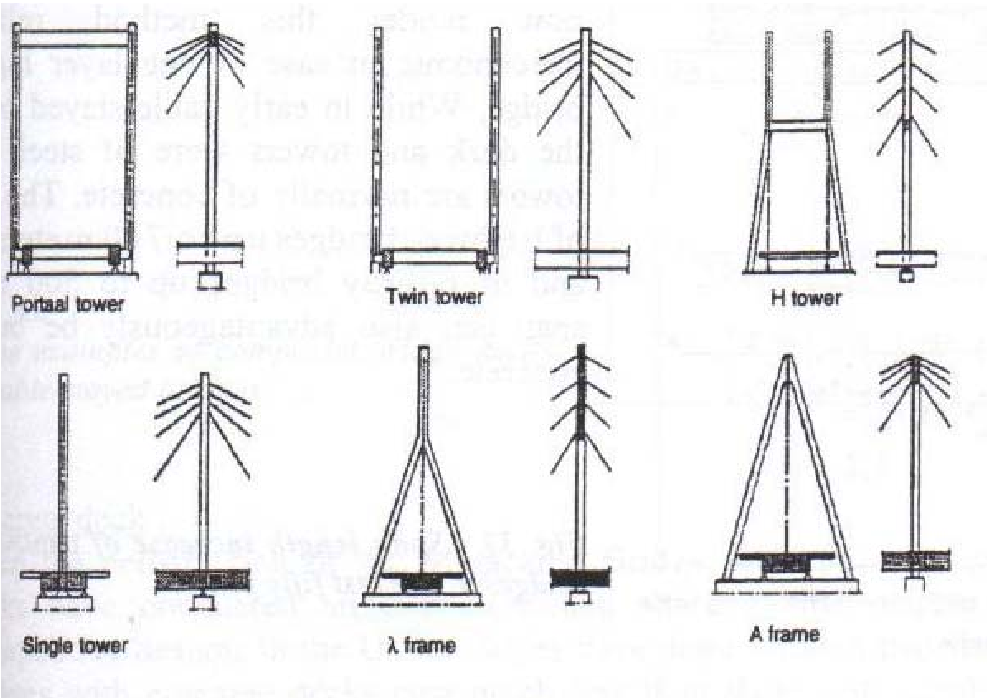


Figure 8.3: Possible pylon configurations.

- The arrangement of the stay-cables can have the following shape:

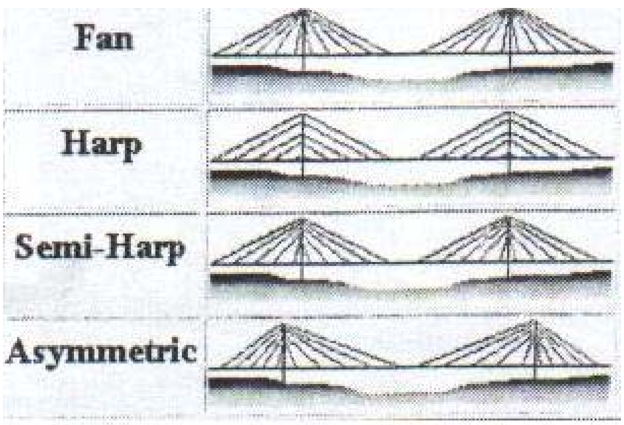


Figure 8.4: Possible arrangements of the stay-cables.

- The pylon can be placed either in the river or on the river bank.

- The anchoring point of the back stay-cables can be on the ground or on the building.

8.2.1 Pylon

The choice for the pylon type has a big influence on a lot of other design aspects. It has influence on:

- The size of the cross-section of the pylon(s)
- The number of stay-cable rows
- The shape of the cross-section of the bridge deck
- The view of the bridge

Pylon size

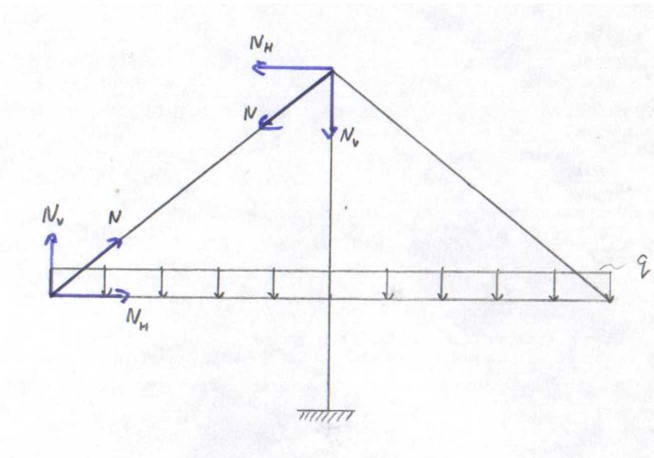


Figure 8.5: Reaction forces of a cable-stay bridge.

The loading on the pylon as a result of the loading on the bridge deck is a normal compressive stress. This makes the pylon prone to buckling. By opting for two in stead of one pylon the force in the pylon will be reduced. This means a more slender pylon can be used in case of two pylons in stead of one. This in term means a smaller cross-section can be used for the pylons.

Another advantage of using two pylons is the possibility to create a connection between the two pylons. This reduces the buckling length of the pylons considerably, making it possible to use an even more slender construction

Number of stay-cable rows

It is clear that in the case of two pylons also two rows of stay-cables will be used. The choice for one pylon leaves the option open to use one row, attached to the centre line of the deck, or two rows, each attached to one side of the deck.

The cross-section of the bridge deck

The number of stay-cable rows influences the loading on the deck.

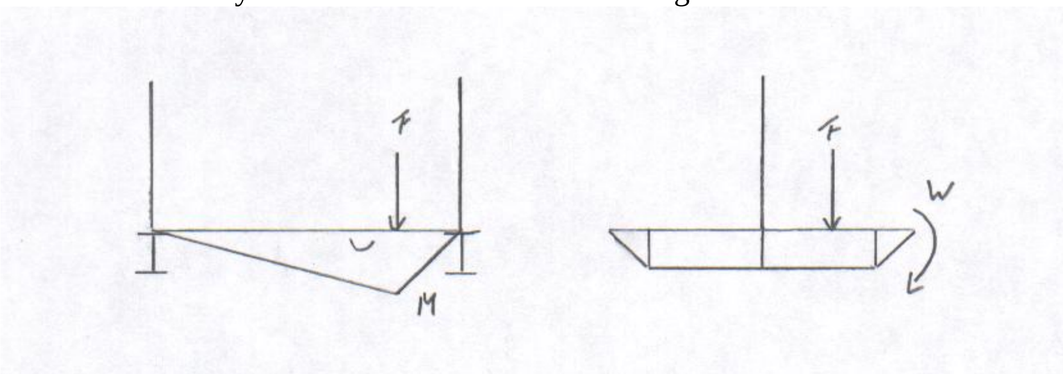


Figure 8.6: Excentric forces on a bridge deck, with two rows or one row of stay cables.

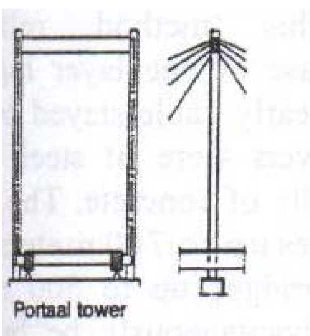
As becomes clear from figure 8.6 a non centric loading of the bridge deck gives different reactions, depending on the number of rows of stay-cables used. In case two rows are used a bending moment occurs, in case one row is used also a torsional moment occurs. These effects have to be taken into account when designing the cross-section of the bridge deck.

View of the bridge

This influence is purely an aesthetic influence. It is in the eye of the beholder, which option is the better in this case.

Pylon choice

None of the design aspects mentioned in the previous part gives a definitive reason to choose for one or two pylons. Most aspects are either an aesthetic one, leaving it up to the observer to decide which is the best, or influence the loading on the bridge, but in both cases a good design can be realised.
However one aspect of the cable-stay bridge in general, has not been taken into consideration yet. A cable-stay bridge, is a bridge prone to dynamical effects, such as wind. In this case the bridge has a small width to span ratio (about 1/25). To achieve aero dynamical stability it is best to use two planes of stay cables (Dr. A Romeijn, dictaat ct5125 Steel bridges).



Therefore the choice is made for a portal tower shape pylon. This type of pylon allows two planes of stay-cables. The reason to chose this specific one is that both the presence of two pylons and the presence of the cross beam allows for very slender pylons.

Figure 8.7: Basic form of chosen pylon style

8.2.2 Location of the pylon

The cable-stay bridge will have an asymmetric appearance, due to the fact that the shipping channel will have to remain open, without any obstacles placed in it. This means a span of at least 160 m is required. The total width of the river is 260 m. The span of the movable bridge is 40 m, leaving 220 m to be spanned with the cable-stay bridge. There are three options:

- The pylon is placed on the edge of the shipping channel creating a span of 160m on the river side of the pylon and 60 m on the bank side.
- The pylon is placed on the bank, creating a span of 220 m on one side of the pylon
- The pylon is placed somewhere between these two points, creating a large span on the river side of the pylon and a small one on the bank side of the pylon.

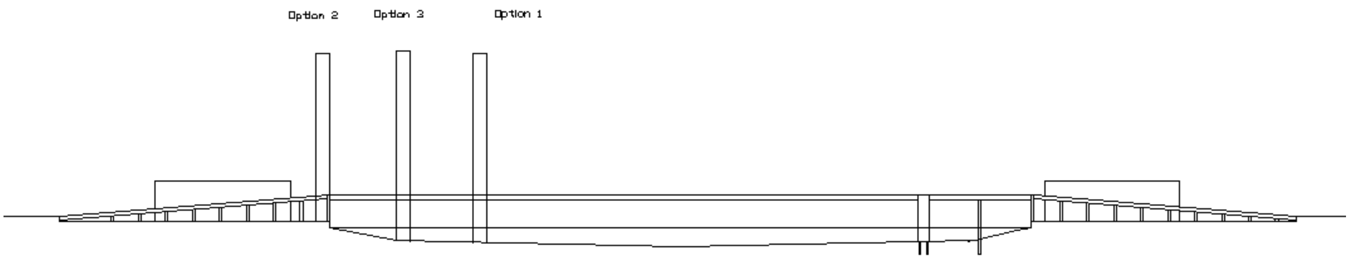


Figure 8.8: Possible pylon positions.

In this case the first option is chosen, due to the fact that the main span, in this case 160 m, needs anchoring on the river bank. By creating a small span on the bank side of the pylon, a counter weight is created, which means less anchoring is required on the bank itself. The larger this smaller span, the less anchoring is required, therefore the pylon is placed as near as possible to the centre of the total span, in this case the edge of the shipping channel.

8.2.3 Arrangement of the stay-cables

As stated before, there are four main types of stay-cable arrangements.

- Fan
- Harp
- Semi-harp
- Asymmetric

Due to the position of the pylon, automatically the last option is selected. However, this still leaves room for the three other arrangements to be taken into the design, because the place of connection between pylon and stay-cables is still open in this arrangement.

Fan arrangement:

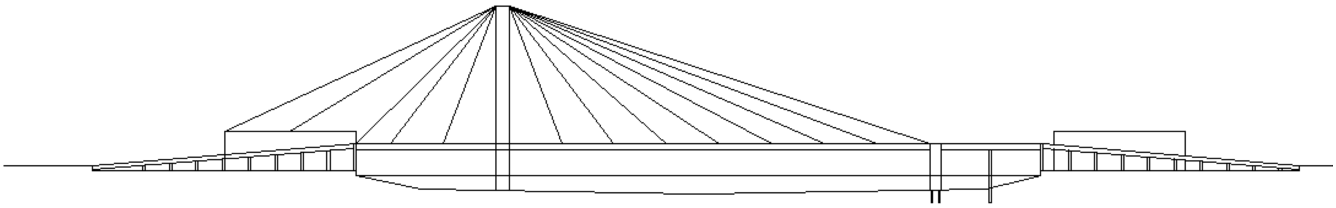


Figure 8.9: Fan arrangement.

The biggest advantage here is that the more vertical the cables are, the less strain it has to carry. Meaning that the cables near the pylon can be smaller in cross-section than the ones furthest away from the pylon. In this case, the normal force in the deck will be smaller than in case of the harp system:

$$N_{deck} = \int_0^l g \cdot dx \cdot \cot g\alpha = \frac{gl^2}{2h}$$

With:
g = Vertical loading
α = Angle between deck and cable
l = length of span
h = pylon height

The disadvantages of this system are the fact that all cables must be attached to the top of the pylon, which can be very complicating and the fact that the construction of the pylon needs to be finished before construction of the deck can be started.

Harp arrangement:

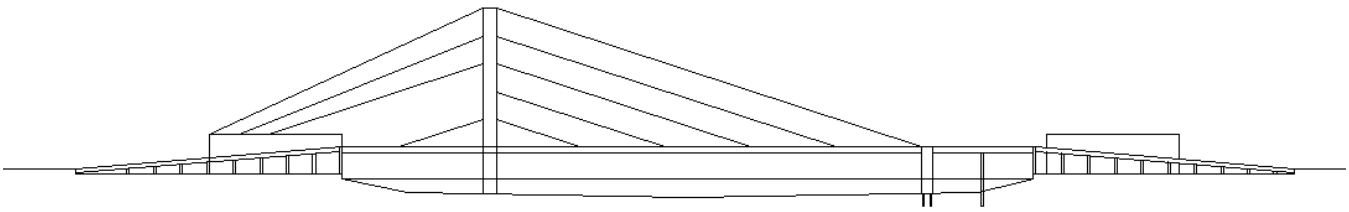


Figure 8.10: Harp arrangement.

The advantage here is that the strain is equal for all cables, making it easier for calculation. However the longer cables are more prone to stretching than the shorter cables. Another advantage is the fact that as soon as the pylon reaches the height of the first anchoring place of the cables, construction of the deck can be started. The disadvantage in this case is the normal force in the deck being twice the size of the normal force in case of a fan arrangement. Normal force in case of harp arrangement:

$$N_{deck} = \int_0^l g \cdot dx \cdot \cot g\alpha_x = \frac{gl^2}{h}$$

Another downside to this arrangement is the fact that in this case the bridge is asymmetric and the anchoring points on the bank are not known yet. As can be seen in the picture it might well be possible that when trying to create a harp arrangement, it can only be created on one side of the pylon and on the other side automatically a semi-harp arrangement is created.

Semi-harp arrangement:

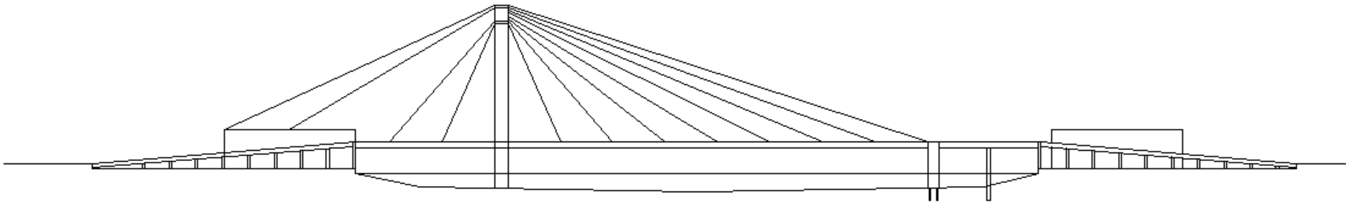


Figure 8.11: Semi-harp arrangement.

This arrangement uses the best of both previous options. Due to its harp like features it can be built quicker and gives fewer difficulties with anchoring the cables to the pylon. And due to its fan like features it reduces the normal force in the bridge deck.

Choice of arrangement

The chosen arrangement for this design is the semi-harp arrangement. Because this arrangement uses the advantages of both the fan and the harp arrangement, and thereby solves the disadvantages of these two systems.

8.2.4 Anchoring of the back stay-cables

With the cable and pylon arrangements known, only one design aspect of the stay-cable bridge needs to be taken into account. This is the anchoring location of the back stay-cables There are three main locations for anchoring the back stay-cables:

1. On top of the building

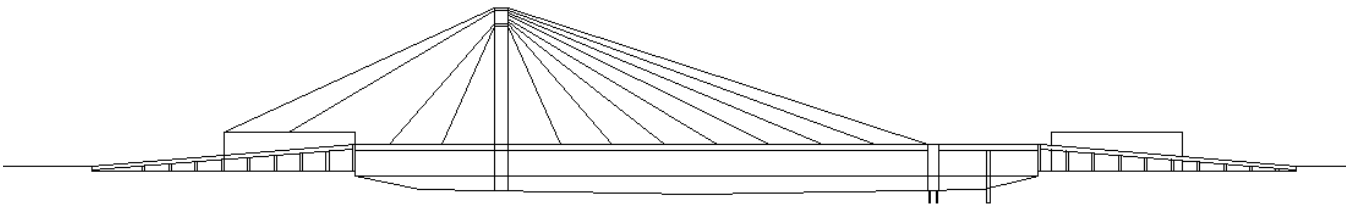


Figure 8.12: Anchoring on top of the building.

2. Between the building and the river

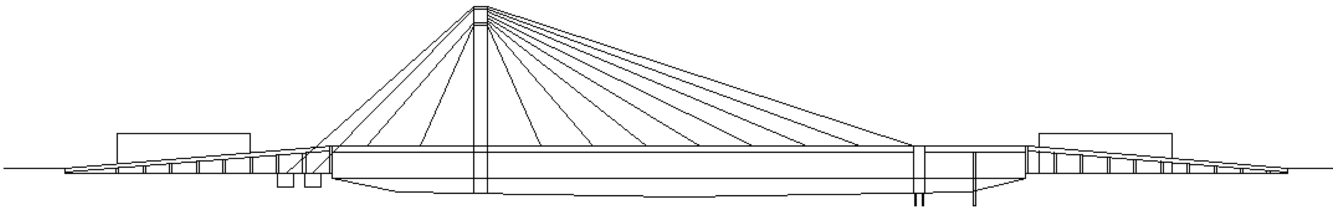


Figure 8.13: Anchoring between the building and the river.

3. Behind the building

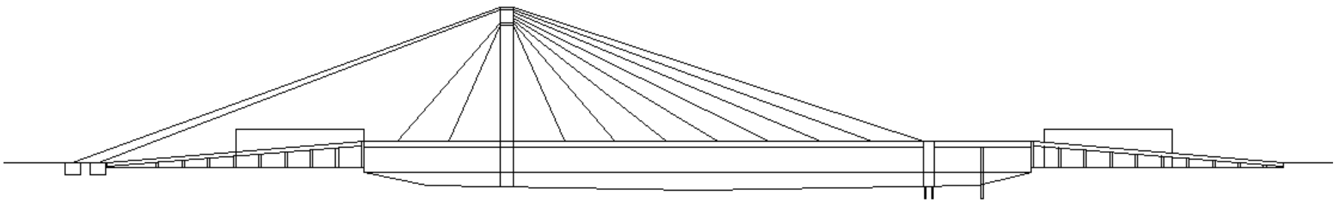


Figure 8.14: Anchoring behind the building.

When choosing between these options a number of things should be taken into account.

- The design goal was to design a living bridge, being a bridge which integrates building and bridge
- Perpendicular to this design is a road along the bank of the river.
- When placing the building not directly on the water side, but by leaving space between river and building a boardwalk can be created.

When looking at the anchoring locations with these things in mind it becomes clear that option one is the best option. This is the only option which integrates building and bridge, there is no construction of anchors needed on the boardwalk like in option 2 and there is no need to relocate the road to create space for anchoring points or use valuable space which can also be used for the building on this bank of the river like in option 3.
An extra advantage is the fact that a lot less or maybe even no extra material is required for anchoring, because the walls of the building can for the counter weight of the anchors.
A disadvantage of this choice is the fact that extra attention is required for the dampers of the bridge, because it can not be allowed for vibrations of the bridge to be transfer into the construction of the building.

8.2.5 Movable bridge

The design of the movable bridge is outside the scope of this thesis, therefore some options are only shortly discussed here to give an idea of possibilities for this part of the total project.

As stated before there are basically two options for the movable bridge.

1. Rotating bridge on a central pivoting point

Figure 8.15: Movable bridge with a central pivoting point.

2. Bridge with a contra-weight

Figure 8.16: Movable bridge with a contra-weight

A third option is a lift, where the bridge deck is hoisted into the air, creating the available space for the sailing ships to pass. This type of bridge was used for the railroad bridge just south of the design location. However, for this relatively small bridge this option is discarded, because the 4 large pylons needed for this type of bridge would become to dominating, both in view and in costs.

The two options left both have advantages and disadvantages.
For the rotating bridge the disadvantage is that an extra column needs to be built in the river to act as the central pivoting point for the bridge. This means an extra obstruction for both the discharge of the river and the passing ships will be created. However with a simple construction the column can be protected from collisions and the area of the column in the cross-section of the river is relatively small, so no major problem for the water discharge is created with this option. The advantage of this option is that there is no space required on the bank of the river contrary to the other option.
The disadvantage for the bridge with a contra-weight is the area required for the contra-weight. On the Dordrecht side of the river there isn’t much space for the buildings, therefore it is a big disadvantage, when the already scarce space is even more reduced by the area required for the contra-weight. The advantage of this option is that no extra columns will have to be built in the river.

8.3 Buildings

There are two buildings which need to be designed. One building one the Dordrecht side of the river and one on the Zwijndrecht side of the river. The design of the buildings on the Dordrecht side of the river is outside the scope of this thesis. However a simple design is given to give an indication of the possibilities for this location. Before the designs can be made the functions and the required space for these functions need to be examined. The first paragraph will be about the building on the Dordrecht side, the second paragraph about the building on the Zwijndrecht side.

8.3.1 Building in Dordrecht

In the current situation there is a small mooring facility at the location of the bridge landing, as can bee seen in figure 8.17.

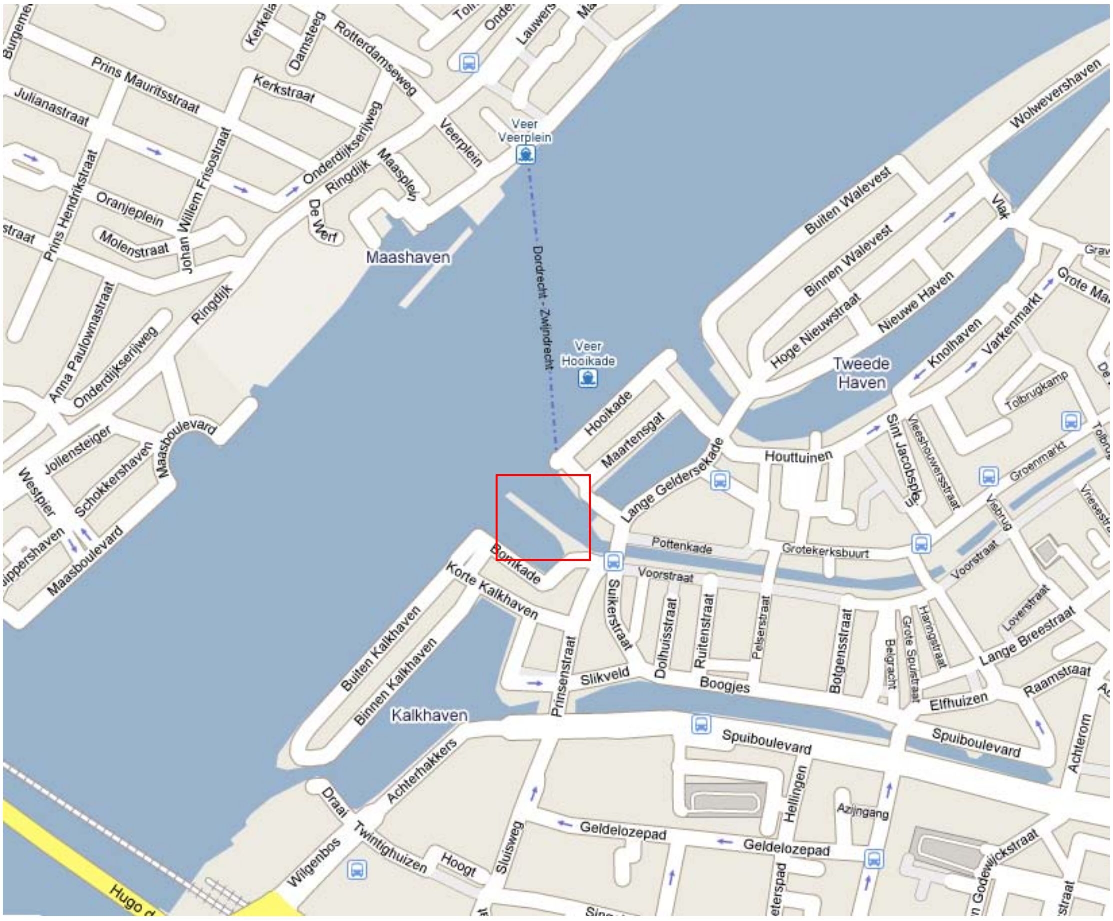


Figure 8.17: Map of present situation, with the design location in the square.

In the new situation this mooring facility will be replaced with new to create land. This new land (see figure 8.18) will house two rows of buildings with a street between them. This street will become the shopping street. The ramp towards the movable bridge will run along the southwest side of this new land, starting at ground level and rising about 10 m into the air to connect to the movable bridge.

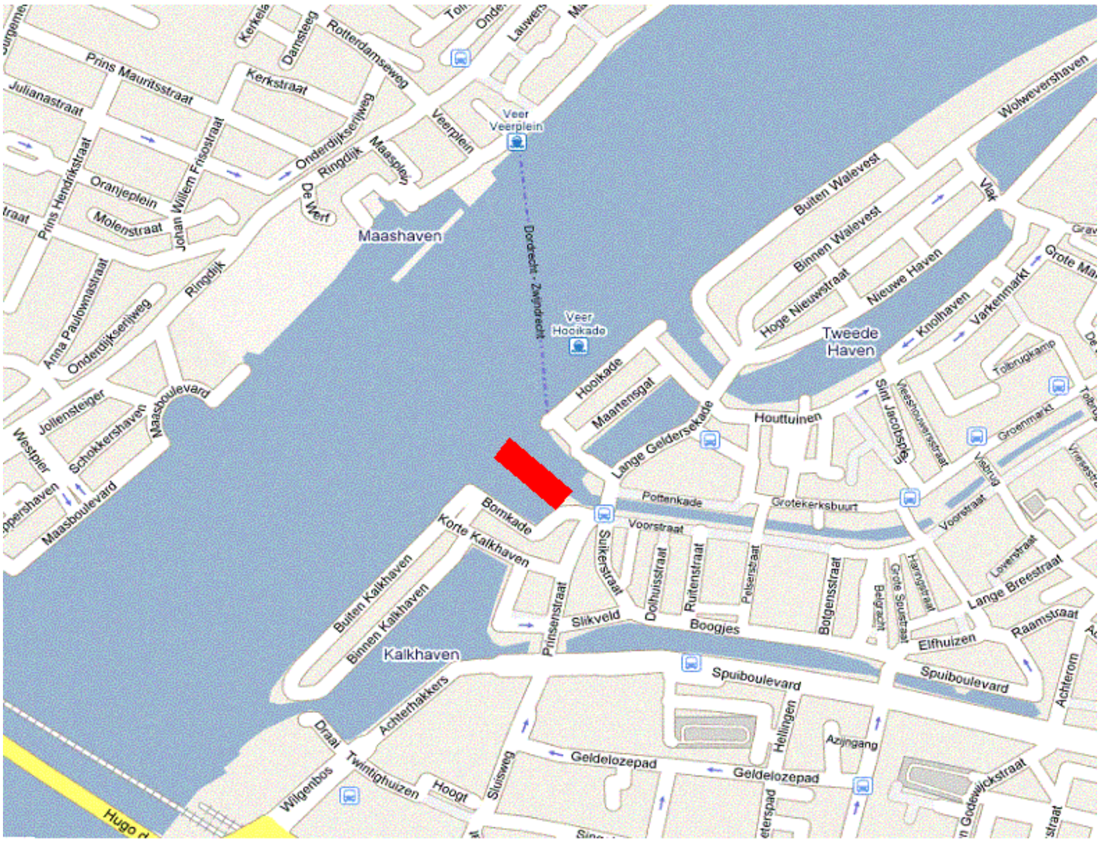


Figure 8.18: New situation

The functions of the two buildings are shops and houses above these shops. There will be room for 8 shops, 1 restaurant each with an area of about 200 m^2. The houses will be located above these shops each with an area of about 100 m^2.

The ramp towards the bridge will stretch the full length of this new land and will have a width of about 10 m. This ramp will be placed on the southwest side of the new land.

With the required areas for the different functions known a ground plan of both the bottom and top flour can be made. These can be seen in figure 8.19.

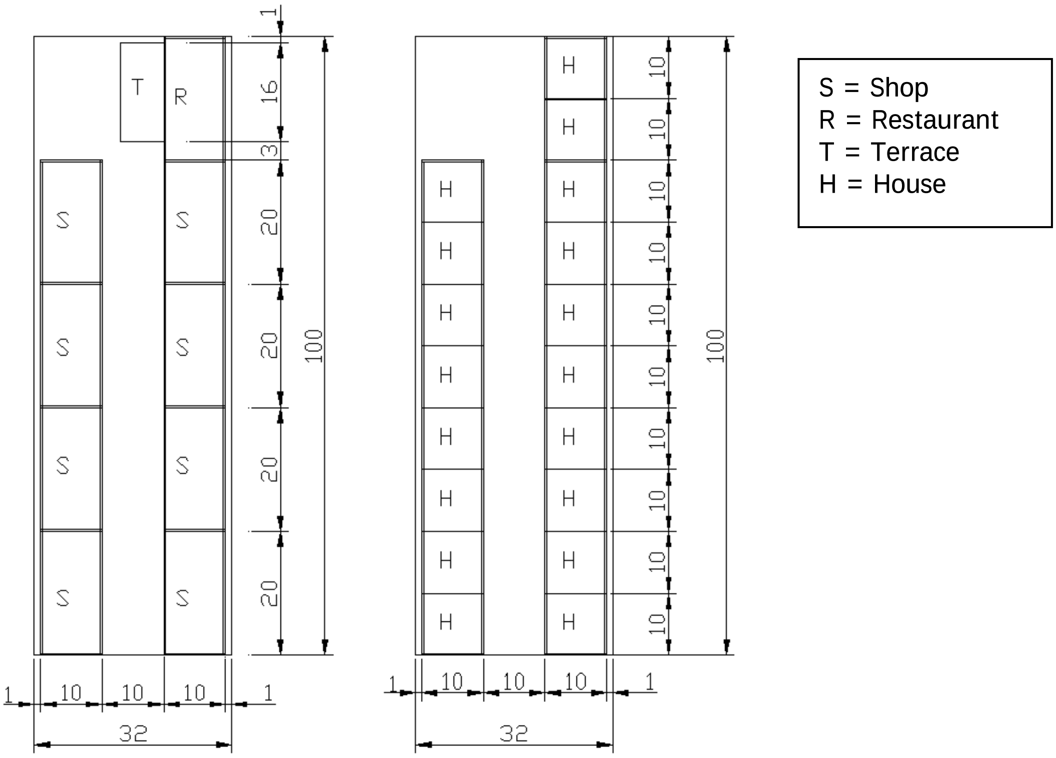


Figure 8.19: Left: plan for ground flour. Right: plan for top flour.

The row of buildings will have a rectangular shape and will be two stories tall.

8.3.2 Building in Zwijndrecht

On the Zwijndrecht side of the river, there is quit a lot of space available for constructing.

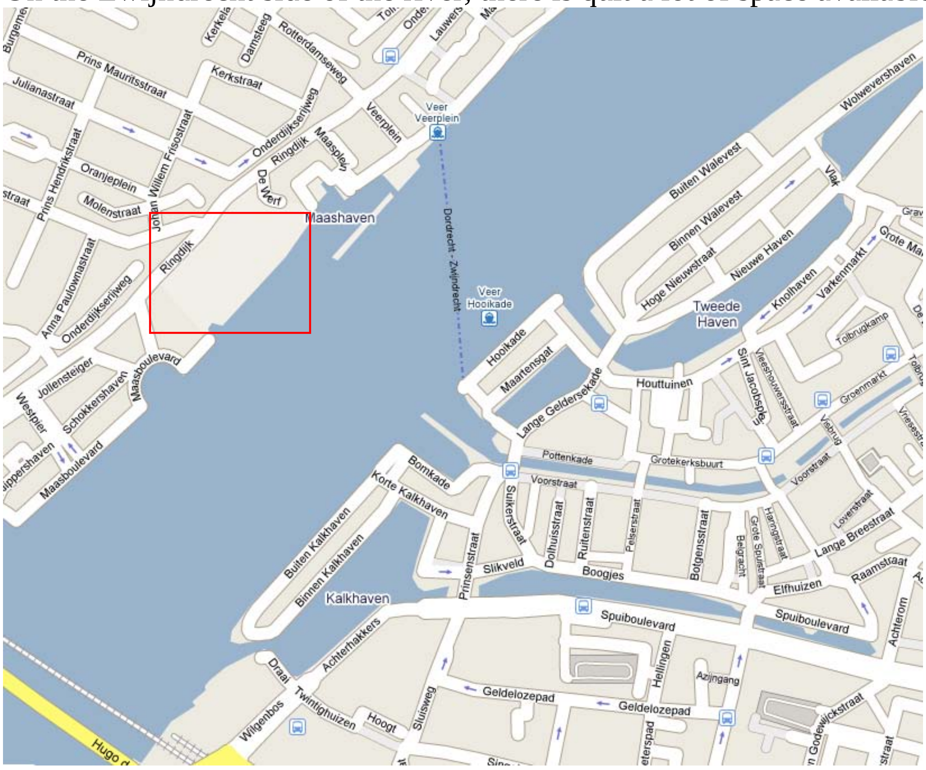


Figure 8.20: Map of design area, with the available space on the Zwijndrecht side marked.

On this side of the river the new movie theatre will arise. This theatre will have 6 rooms, each with a capacity of about 400 people. Below this theatre there will be a parking deck. On the ground level there will be four rooms and on the top level there will be 2 rooms. The top level of the two other rooms will be available for a café and a terrace. This terrace will be accessible directly from the bridge. There will be a space of 20 m wide, between the riverside and the front of the building, allowing for a boardwalk along the river. The ramps to access the bridge will be placed along the boardwalk, making it possible for both pedestrians and cyclists to enter the bridge via the boardwalk.
Each room in the movie theatre will have an area of 47 by 27 meters with a height of 15 meters allowing for the right projecting and viewing angles in the theatre.

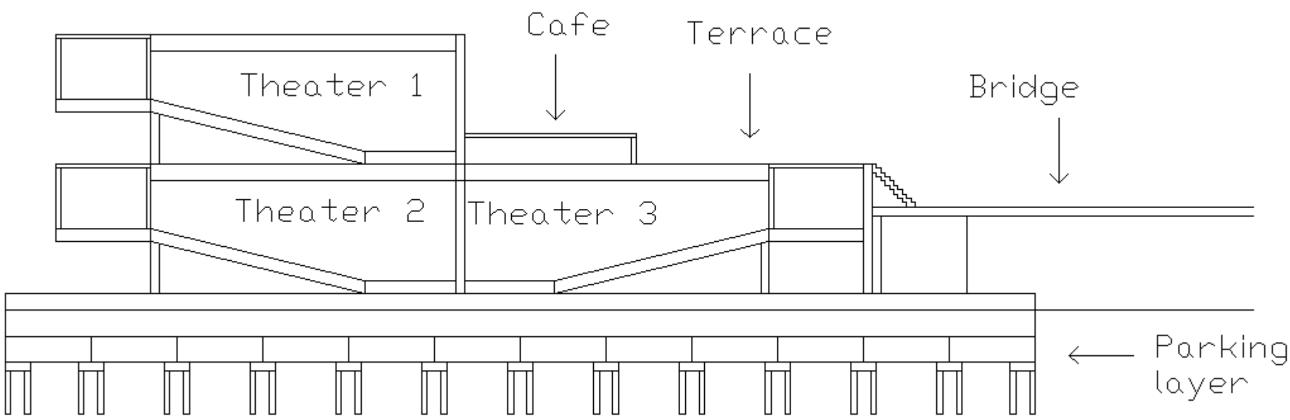


Figure 8.21: Cross-section of the theater

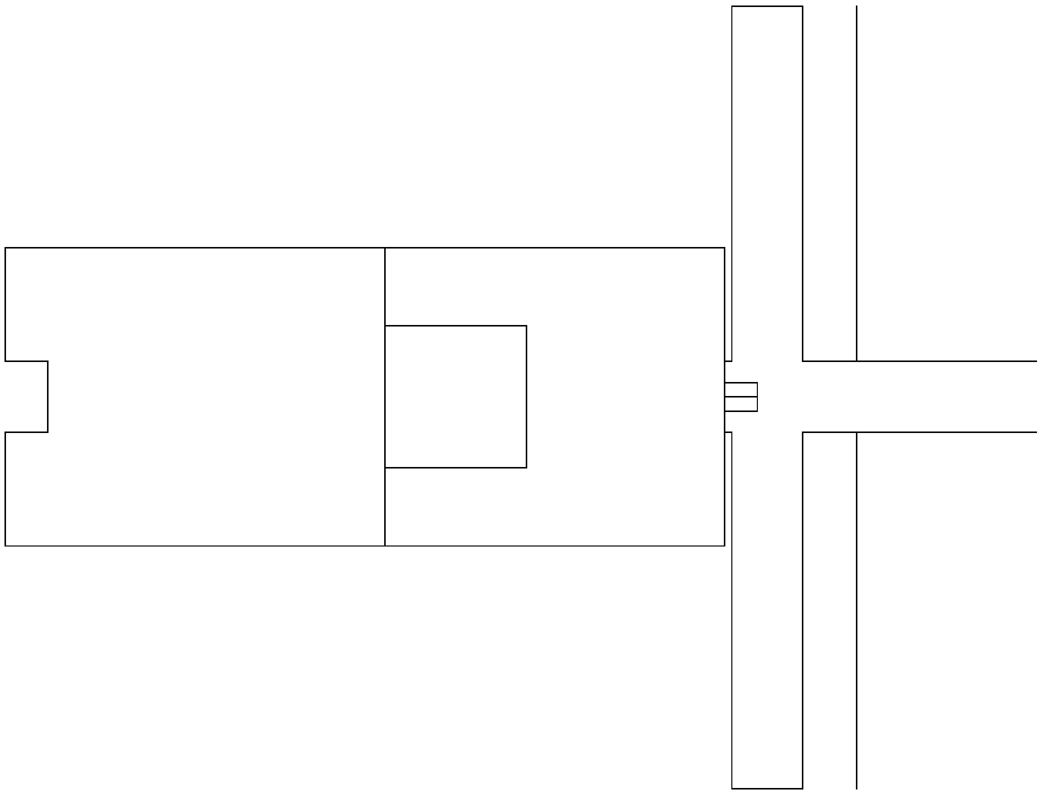


Figure 8.22: Top view of the theater and bridge ramps

As stated in the paragraph about the bridge design, the bridge will be anchored on the building. To accomplish this, the inner walls of the theatre will have to work as an anchor. This can be seen in figure 8.23.

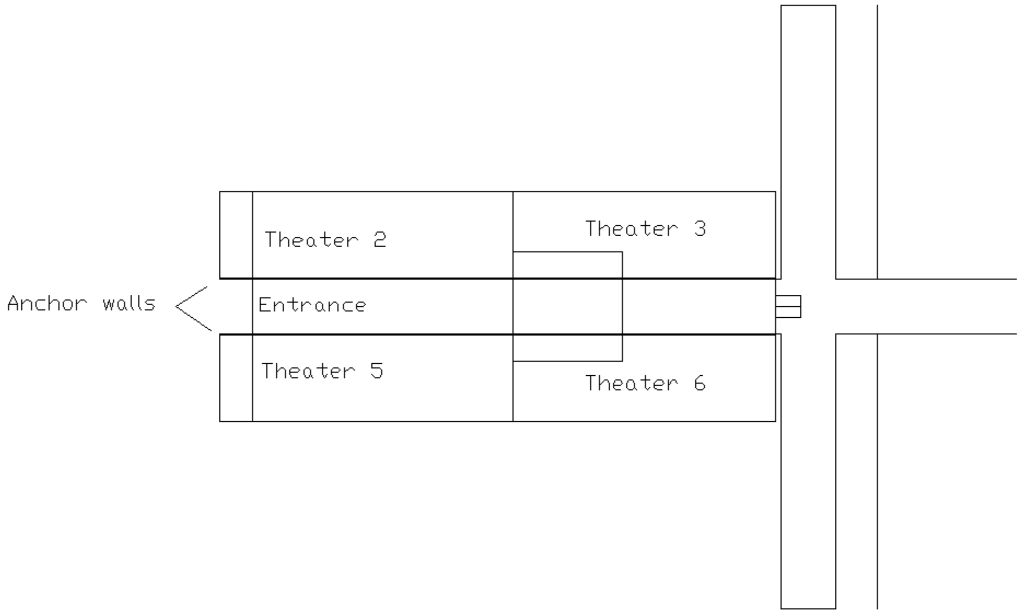


Figure 8.23: The walls of the theatre will function as an anchor for the bridge.

8.4 Total design

Now that a preliminary design of all the different parts has been made a combined side view and top view of the area can be created. In the next chapters the various structural components will be calculated and drawn, to create a definitive structural design.

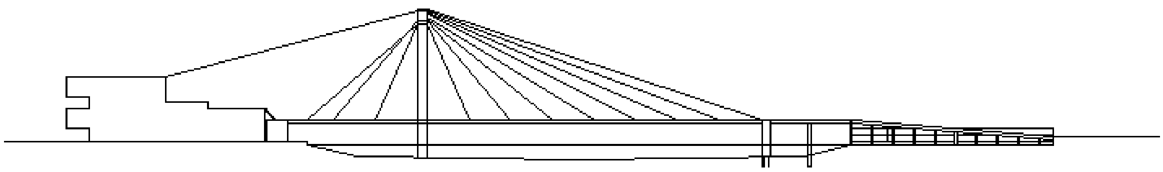


Figure 8.24: Side view.

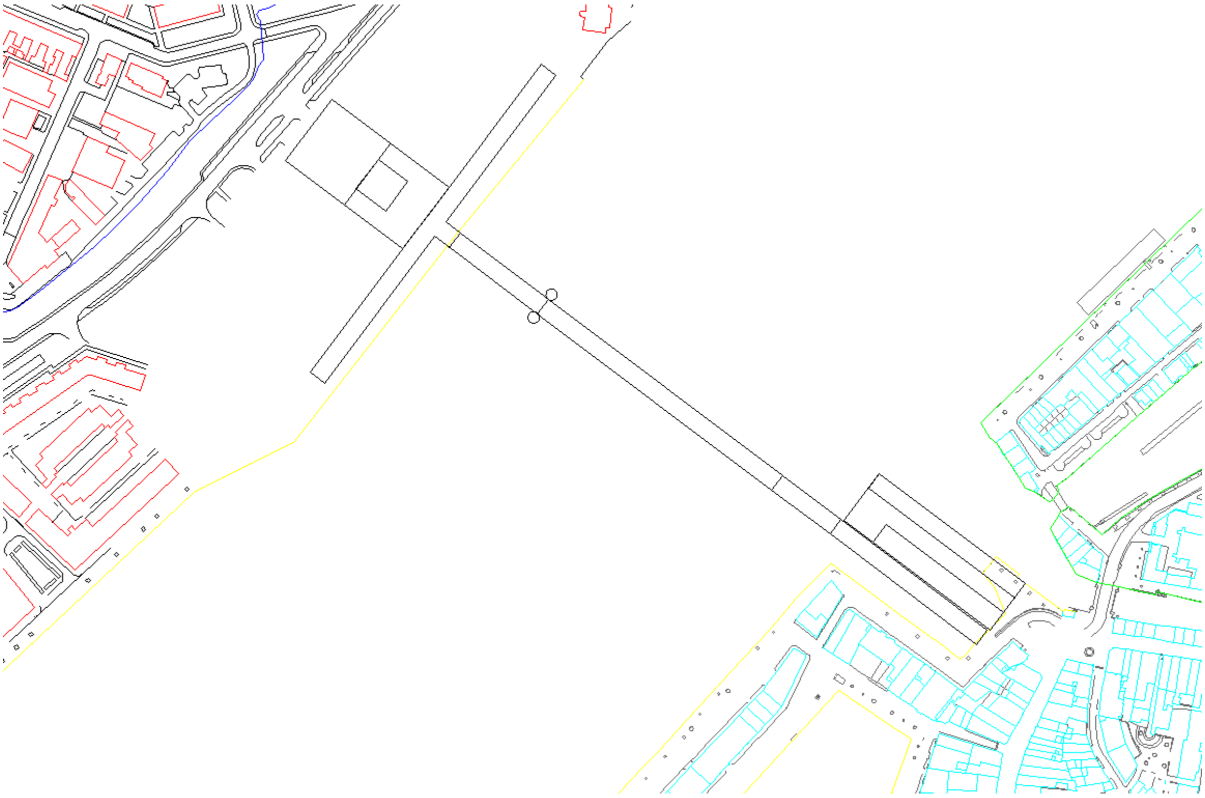


Figure 8.25: Top view of total plan

9. Calculations for bridge design

In this chapter the calculations for the different parts of the main structural components of the bridge will be done. First an overview of the different actions on the constructions will be given. Then a material choice for the different components of the constructions will be made. With both the loadings on and the strength of the structure known, calculations can be made to check weather or not the construction can withstand these actions both in strength and stiffness. The first calculations will be done by hand to find the dimensions of the different structural components. Afterwards these calculations will be checked with computer calculations. This will lead to the final dimensions of each of the parts of the main structure.

9.1 Loading actions on the bridge

Vertical loading

There are two main types of vertical loading acting on the bridges, permanent and variable. The permanent loadings are caused by the self weight of the main bearing structure and by the weight of the top layer placed on the deck. The variable loading is caused by the traffic.
The permanent loading will be discussed later, due to the fact that material choices and dimensions of these components are required to give there own weight. The variable loading is given by the Eurocode EN1991. The bridge is designed for slow traffic (cyclists and pedestrians). The loading which should be used for calculations of the bridge is a uniformly distributed load of $p_{var} = 4\ kN/m^2$.

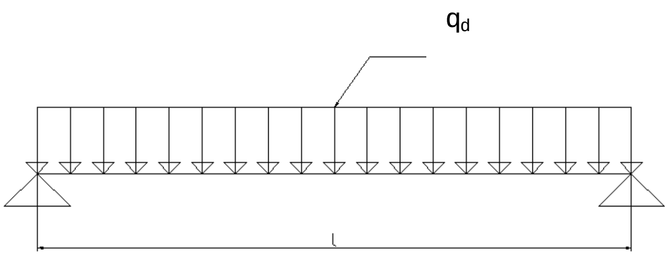


Figure 9.1: Schematisation of the loading on a beam.

The total loading on the bridge becomes:

$$q_d = \gamma_{var} * q_{var} + \gamma_p * q_p$$

With:

q_d = Total design load	[N/m]
γ_{var} = Safety factor for variable loading	[-]
q_{var} = p_{var} * width of the bridge	[N/m]
γ_p = Safety factor for permanent loading	[-]
q_p = permanent loading per meter span	[N/m]

Cable-stay Bridge

For the cable-stay bridge, there will be four cases of loading which need to be examined. One case with a uniformly distributed variable loading on the whole bridge, one case with a variable loading on only the main span of the bridge, one case with a uniformly distributed variable loading on only the side span of the bridge and one with variable loading on only the fields next to the pylon. In all four cases the permanent load will be present on the whole bridge. The bridge will be supported on both ends of the river. In between, the stay-cables act as a spring-support, with a spring stiffness of:
$k = AE/l$

with:
l = cable length
A = area of cross-section
E = Modulus of elasticity

The load is distributed from the deck to the pylons by the stay-cables. The design of a pylon is mainly based on the largest normal force acting on this structure. This is obtained considering the following load situation:

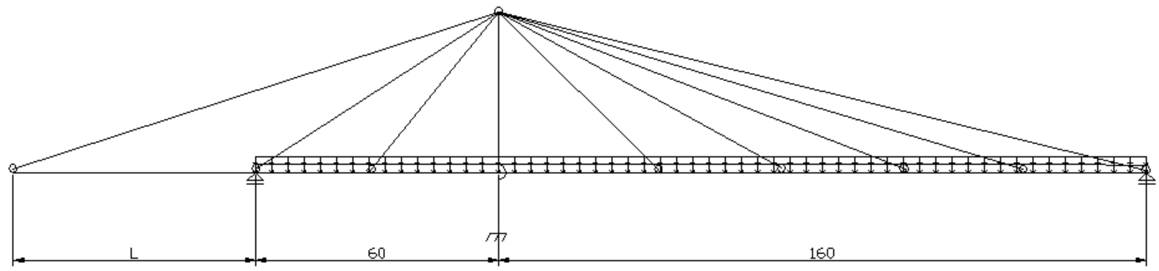


Figure 9.2: Schematisation first case.

The largest bending moment in the deck is obtained considering this structure as shown in figure 9.3:

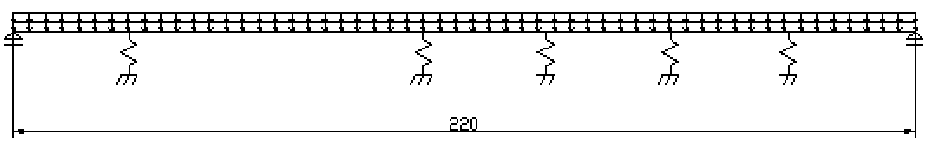


Figure 9.3: Schematisation for calculating the bending moment in the bridge deck.

The design of the anchor cable, the cable which connects the bridge to the building is based on the largest normal force acting on this part of the structure. This is obtained by considering the following load situation:

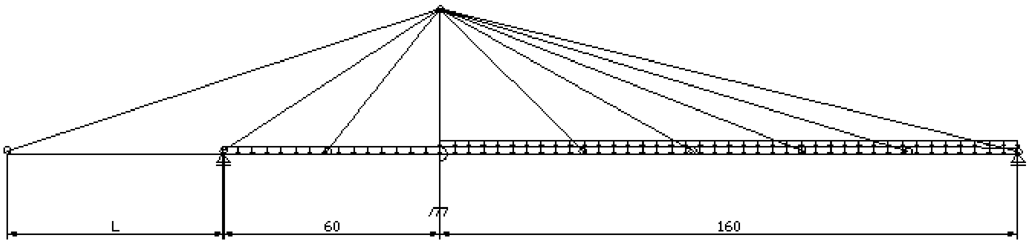


Figure 9.4: Schematisation second case.

The schematisation for the deck in this case becomes:

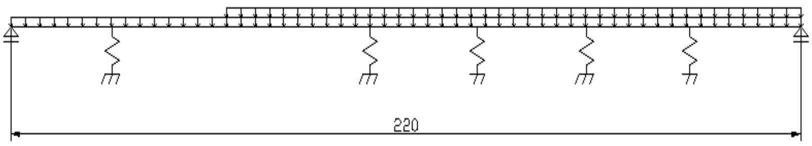


Figure 9.5: Schematisation bridge deck, second case.

The third load case is applied to check if the anchor cable is permanently loaded by a tensional force. This tensional force must be present at all times, because if this cable is loaded by compression the whole structure will become unstable. To do this check the following load case is considered:

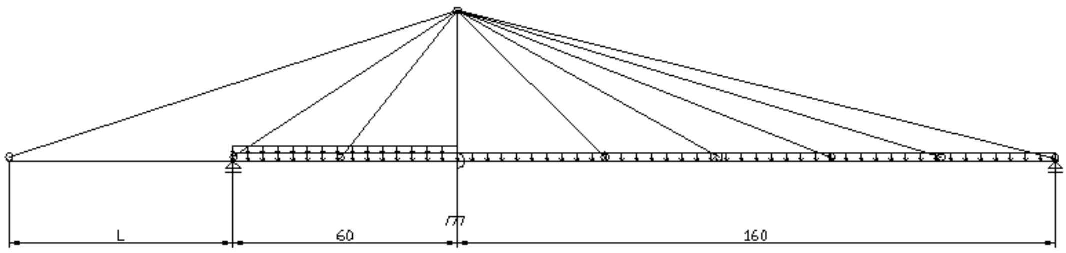


Figure 9.6: Schematisation third case.

The fourth case is used to obtain the largest negative bending moment, when this is larger than the bending moment obtained in case two, this moment must be used for the design of the bridge-deck. This bending moment is obtained considering the following load situation:

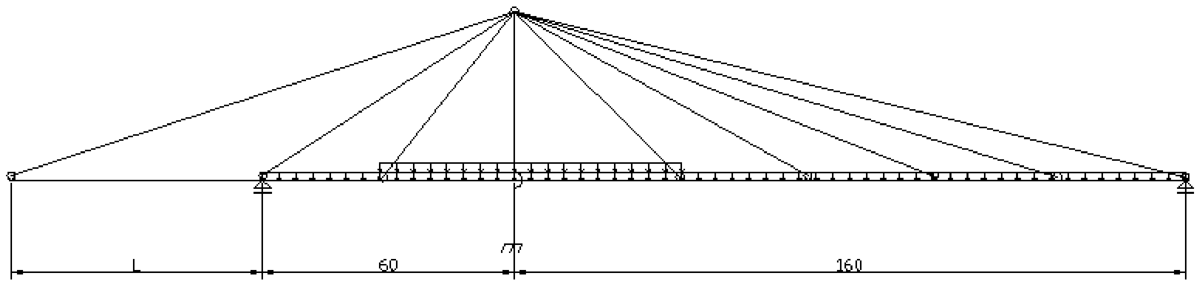


Figure 9.7: Schematisation fourth case.

With the loadings and the schematisations for the vertical loadings known, now the materials and dimensions for the construction need to be determined, because this is required to determine both the permanent vertical loading and the strength of the construction.

9.2 Material choice

Before the type of material for constructing this bridge can be chosen, the cable-stay bridge first needs to be divided into three parts: cables, pylon and the bridge deck.

For the cables the choice is a simple one, only steel can be used. For the pylon and the deck both steel and concrete are options. Even a combination of both of them can be applied.
One of the design requirements is that the bridge must be a slender bridge. The bridge is for pedestrians and cyclists only, therefore fatigue is not an issue. This means a slender design is best achieved using steel only. The type of steel which is chosen is S355. This will also be used for the pylon.

9.3 Dimensions

Before the calculations for the different components of the bridge can be done, first an estimation of the dimensions of these components needs to be done.

The spans of the bridge are a given, due to the total width of the river and the minimum width of the shipping channel. The only thing that needs to be taken into account is the ratio between the main span and the side span.
As a rule of thumb for a good design it holds that the ratio between the side and the main span should be between 0.35 and 0.45. When the ratio is larger, the situation might occur that the anchor cable is loaded by compression, which makes the whole bridge unstable.
The total width of the river which needs to be span with the cable-stay bridge is 220 m. When placing the pylon directly next to the shipping channel the following bridge lengths occur: main span = 160 m and the side span = 60 m giving a ratio of 0.375. This is within the optimal ratio, therefore these lengths are chosen.

The height of the pylon is free, however when dimensioning the height of the pylon two things need to be taken into account: the economic consequences and the consequences for

the loading of both the pylon and the rest of the bridge. The latter is caused by the fact that the angle between the cables and the deck is determinant for the horizontal loading of the deck. As a rule of thumb the ratio of the pylon height (above deck) and the main span length should be between 0.15 and 0.25 (see figure 9.8 (dr. A. Romeijn, 2005)).
For this case with a main span length of 160 m this means the height of the pylon should be between 24 and 40 m. Therefore the chosen height is 30 m.

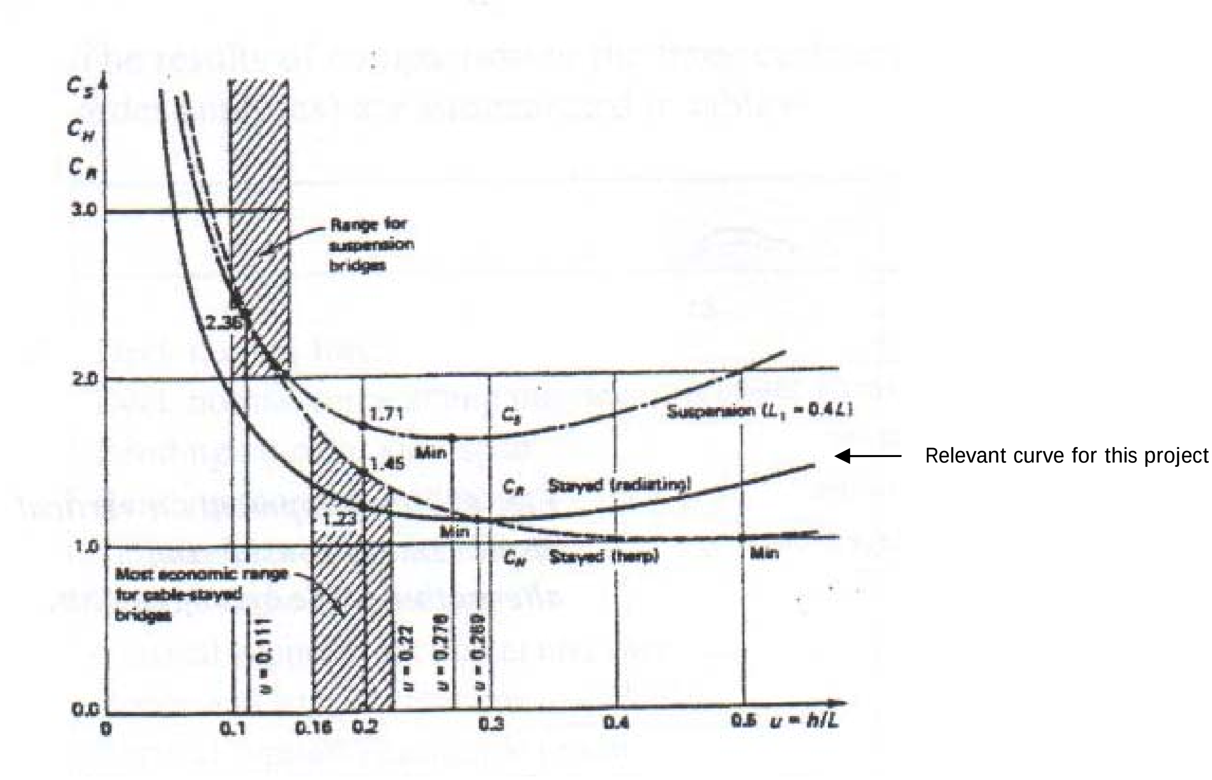


Figure 9.8: Relation ratio pylon height-main span and cost.

For the c.o.c. distance of the cable-deck connections the same considerations apply as for the pylon height. As a rule of thumb for a steel deck a distance of 16 m is the optimal distance and therefore this distance is also used in this design. This means there will be 10 cables on the mains span side of the pylon and 3 cables on the side span side + one anchor cables connecting bridge and building on each side of the bridge.

The height of the main girder is defined as a design requirement with a maximum height of 1000 mm. This maximum will be the chosen height.

With the main dimensions determined with the rules of thumb, they need to be checked, whether or not these dimensions are sufficient to resist all the loads on the bridge. First a calculation by hand will be done to check weather or not the chosen dimensions are sufficient to withstand the loading. After this check a computer calculation will be done to confirm the results of the hand calculation and to get a more accurate view on the load distribution.

9.4 Actions on the bridge

There are two types of vertical actions on the bridge: Permanent loading and variable loading. The first category consists of the self-weight of the bridge, the second category is caused by traffic and environmental effects, like wind loading. This second category is defined in the EN and depends on the type of traffic which uses the bridge. In this case it is a bridge for pedestrians and cyclists, giving a load of p_{var} = 4,0 kN/m^2.

The permanent loading (p_p) on the bridge consists of:

Weight asphalt layer:	1,5 kN/m^2
Main deck plate:	1,0 kN/m^2
Troughs:	0,6 kN/m^2
Secondary girders:	1,0 kN/m^2

And q_p:

Main girders:	5,0 kN/m

9.5 Hand calculations

First a calculation by hand is done. These calculations are done to determine the dimensions of the different parts of the structure. In the next paragraph these calculations will be checked by computer calculations.

9.5.1 Stay-cables

With the permanent and variable vertical actions on the bridge given, the loading on each of the stay cables can be determined.
The total loading on each of the stay-cables will be:
$F_{p,tui} = p_p * 0,5 * b_{bridge} * d_{tui} + q_p * d_{tui}$

With:

p_p = permanent loading per area	[kN/m^2]
b_{bridge} = width of the bridge	[m]
d_{tui} = c.o.c. stay-cables	[m]
q_p = weight of the main girder	[kN/m]

This means the total permanent loading on each stay-cable will be:
$F_{p,tui}$ = (1,5 + 1,0 + 0,6 + 1,0) * 0,5 * 10 * 16 + 5,0 * 16 = 408 kN

The total variable loading on each stay-cable will be:
$F_{var,tui} = p_{var} * 0,5 * b_{bridge} * d_{tui}$

$F_{var,tui}$ = 4,0 * 0,5 * 10 * 16 = 320 kN

For calculating the strength these actions need to be multiplied with a safety factor. These factors are:
$\gamma_{p,d}$ = 1,2
$\gamma_{var,d}$ = 1,5

This means the design loads will be:
$F_{p,tui,d} = F_{p,tui} * \gamma_{p,d} =$ 408 * 1,2 = 490 kN
$F_{var,tui,d} = F_{var,tui} * \gamma_{var,d}$ = 320 * 1,5 = 480 kN

The total design load is:
$F_{tui,d} = F_{p,tui} * \gamma_{p,d} + F_{var,tui} * \gamma_{var,d}$
$F_{tui,d}$ = 408 * 1,2 + 320 * 1,5 = 970 kN

With the vertical loading known the normal force in each of the stay-cables can be determined. To do so first the system needs to be transformed into a quasi-static system. This is done by applying hinges at the cable - main girder connections.

There are four loading combinations which need to be taken into account when calculating the stay-cables:
Case 1: The bridge is fully loaded, meaning both permanent and variable actions work on both spans.
Case two: Full loading on the main span, but only permanent loading on the side span.
Case three: Full loading on the side span, but only permanent loading on the main span.
Case four: Full loading on the first elements next to the pylon and only permanent loading on the other elements.

The case which gives the largest normal force in both the stay-cables and the deck, will be the governing case and will be used to calculate the dimensions of the cables.

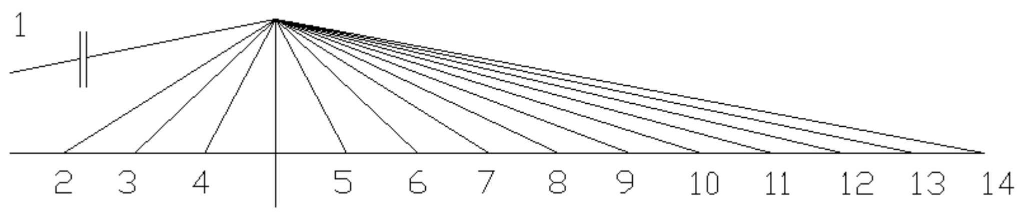


Figure 9.9: Cable numbers

In table 9.1 first the characteristics of the cables are given:

Table 9.1: Characteristics cables.

Cable number	X [m]	Cable length [m]
Back-stay-cable (1)	-130	130,6
2	-48	56,6
3	-32	43,9
4	-16	34,0
5	16	34,0
6	32	43,9
7	48	56,6
8	64	70,7
9	80	85,4
10	96	100,6
11	112	115,9
12	128	131,5
13	144	147,1
14	160	162,8

With the vertical loading and the lengths of the cables known the force in each of the stay-cables can be calculated:

$N_{tui} = F_{tui,d} / \sin \alpha$

With:

α = angle between cable and bridge deck

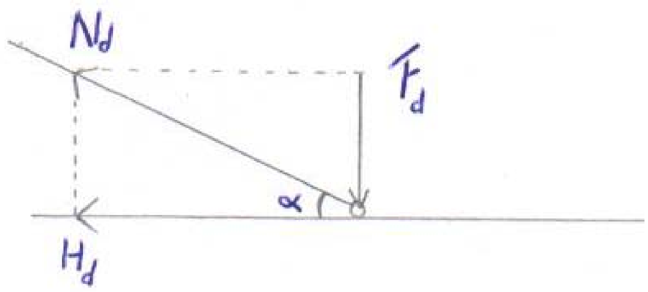


Figure 9.10: The relations between the forces on a cable – deck connection.

In table 9.2 the different vertical loadings on the stay-cables are given:

Table 9.2: Vertical loading per stay-calbe.

Cable number	$F_{p,tui,d}$ **[kN]**	$F_{var,tui,d}$ **[kN]**
Back-stay-cable (1)	Calculated later	Calculated later
2	490	480
3	490	480
4	490	480
5	490	480
6	490	480
7	490	480
8	490	480
9	490	480
10	490	480
11	490	480
12	490	480
13	490	480
14	490	480

With these and the given formula, the normal forces in the stay-cables become:

Table 9.3: Normal force per stay-cable.

Cable number	$N_{p,tui,d}$ **[kN]**	$N_{var,tui,d}$ **[kN]**
Back-stay-cable (1)	Calculated later	Calculated later
2	925	906
3	716	702
4	555	544
5	555	544
6	716	702
7	925	906
8	1154	1131
9	1396	1367
10	1643	1609
11	1894	1855
12	2147	2103
13	2402	2353
14	2659	2605

Also the horizontal forces resulting from the vertical loading can be calculated:
$H_{tui} = F_{tui,d} / \tan \alpha$

With:
α = angle between cable and bridge deck

Table 9.4: Horizontal loadings resulting form the stay-cables.

Cable number	$\mathbf{H_{p,tui,d}}$ **[kN]**	$\mathbf{H_{var,tui,d}}$ **[kN]**
Back-stay-cable (1)	Calculated later	Calculated later
2	784	768
3	523	512
4	261	256
5	261	256
6	523	512
7	784	768
8	1045	1024
9	1307	1280
10	1568	1536
11	1829	1792
12	2091	2048
13	2352	2304
14	2613	2560

With the normal forces and the horizontal reaction forces known, the four different load cases can be applied to determine the governing load case.

It is clear that the first case is governing for the normal force in the cables 2 to 14.Because the largest vertical loading will give the largest normal tensile force in the cables.
The normal force in the cables will be:
$N_{tui,d} = N_{p,tui,d} + N_{var,tui,d}$

The resulting normal forces will be given in table 9.5:

Table 9.5: Governing normal force per stay-cable.

Cable number	$\mathbf{N_{tui,d}}$ **[kN]**
Back-stay-cable (1)	Calculated later
2	1830
3	1418
4	1099
5	1099
6	1418
7	1830
8	2285
9	2763
10	3252
11	3749
12	4251
13	4756
14	5263

The same case is governing for the horizontal forces resulting from cables 2 to 14.
The horizontal force will be:
$H_{tui,d} = H_{p,tui,d} + H_{var,tui,d}$

The horizontal forces are given in table 9.6:

Table 9.6: Horizontal forces resulting from loading of stay-cables.

Cable number	$H_{tui,d}$ **[kN]**
Back-stay-cable (1)	Calculated later
2	1552
3	1035
4	517
5	517
6	1035
7	1552
8	2069
9	2587
10	3104
11	3621
12	4139
13	4656
14	5173

To determine the normal force in back stay-cable first the horizontal reaction force needs to be calculated. To determine the governing horizontal force all three load cases will be examined. Due to the fact that there must be horizontal equilibrium, in general it can be stated that:
$H_1 + \Sigma H_{2-4} - \Sigma H_{5-14} = 0$

Or

$H_1 = \Sigma H_{5-14} - \Sigma H_{2-4}$

In the first case the full loading acts on both spans, therefore the H_2 to H_{14} will be $H_{tui,d}$.
In this case the horizontal force in the back stay-cable becomes:
$H_{1,d} = \Sigma H_{5-14,d} - \Sigma H_{2-4,d}$

$H_{1,d} = (517 + 1035 + 1552 +2069 +2587 + 3104 + 3621 + 4139 + 4656 + 5173) - (1552 + 1035 + 517) = 25349$ kN

In the second case the full loading acts on the main span and only the permanent loading acts on the side span. Therefore only $H_{p,tui,d}$ will be used for H_{2-4} and $H_{tui,d}$ will be used for H_{5-14}.
In this case the horizontal force in the back stay-cable becomes:
$H_{1,d} = \Sigma H_{5-14,d} - \Sigma H_{p,2-4,d}$

$H_{1,d} = (517 + 1035 + 1552 +2069 +2587 + 3104 + 3621 + 4139 + 4656 + 5173) - (784 + 523 + 261) = 26885$ kN

In the third case the full loading acts on the side span and only the permanent loading acts on the main span. Therefore $H_{tui,d}$ will be used for H_{2-4} and $H_{p,tui,d}$ will be used for H_{5-14}.
In this case the horizontal force in the back stay-cable becomes:
$H_{1,d} = \Sigma H_{p,5\text{-}14,d} - \Sigma H_{2\text{-}4,d}$

$H_{1,d}$ = (261 + 523 + 784 + 1045 + 1307 + 1568 + 1829 + 2091 + 2352 + 2613) – (1552 + 1035 + 517) = 11269 kN

It becomes clear that case two is the governing case with a horizontal force of 26885 kN.
With the horizontal force known, the normal force in the back stay-cable can be calculated:
$N_{1,d} = H_{1,d} / \cos \alpha$ = 27000 kN

With the governing normal force in the back stay-cable known all governing normal forces in the stay-cables are known. They are given in table 9.7.

Table 9.7: Normal force per stay-cable.

Cable number	**$N_{tui,d}$ [kN]**
Back-stay-cable (1)	27000
2	1830
3	1418
4	1099
5	1099
6	1418
7	1830
8	2285
9	2763
10	3252
11	3749
12	4251
13	4756
14	5263

With the forces known, the minimal required area of each of the stay-cables can be determined.
For the cables to have sufficient strength to withstand all the loading actions, the following unity-check holds:
$\sigma_{tui,d} / f_{y,d} \leq 1$

With:
$\sigma_{tui,d} = N_{tui,d} / A_{tui}$
$f_{y,d}$ = design strength steel cable

Cable type

There are several types of cables available for stay-cables. The four main types are:

- Parallel bar, steel bars in cased in a steel sleeve combined with grout for composite bar.
- Parallel wire, a groups of wires is used to create a cable.
- Stranded cable, cable is build up of twisted strands, each formed by 7 twisted wires.
- Locked-coil cable, central core of wires is surrounded by layers of s-shaped locking-cables.

Most often nowadays the mono strand types (parallel wire and locked-coil) are used, because of the fact that they are easier to work with, when constructing the bridge. When looking at the anchors with which the cable-deck connection needs to be created, it becomes clear that the locked-coil system has an easier system of anchor plates on the deck in stead of an anchor inside the bridge deck. A disadvantage of this system is that cables will have to be replaced in whole, when damaged in stead of individual strands in case of the parallel wire system. However the easier way of constructing anchors is preferred over this and therefore the locked-coil system is chosen as the system for the stay-cables.

The design strength of the steel used in this system is, $f_{y,d}$ = 900 N/mm^2

With the governing loading and strength of the cables known, the minimal required area for each of the cables can be determined. See table 9.8.

Table 9.8: Minimum required area per stay-cable.

Cable number	**$N_{tui,d}$ [kN]**	**A_{tui} [mm^2]**
Back-stay-cable (1)	27000	30000
2	1552	2034
3	1035	1576
4	517	1221
5	517	1221
6	1035	1576
7	1552	2034
8	2069	2539
9	2587	3070
10	3104	3613
11	3621	4166
12	4139	4723
13	4656	5284
14	5173	5848

With the minimal required area of cross-section of each of the cables known, the corresponding locked-coil cable can be chosen. The characteristics of the locked-coil cables are given in appendix 2.

For easiness during construction only three different diameters will be chosen (with exception of the back stay-cable), the chosen diameters are given in table 9.9.

Table 9.9: Chosen cable diameters.

Cable number	**A_{tui} [mm^2]**	**Chosen diameter [mm] – area [mm^2]**
Back-stay-cable (1)	30000	212 - 30957
2	2034	56 - 2136
3	1576	56 - 2136
4	1221	56 - 2136
5	1221	56 - 2136
6	1576	56 - 2136
7	2034	56 - 2136
8	2539	80 - 4358
9	3070	80 - 4358
10	3613	80 - 4358
11	4166	80 - 4358
12	4723	96 - 6276
13	5284	96 - 6276
14	5848	96 - 6276

9.5.2 Pylon

To calculate the governing loading on the pylon, both the variable and permanent loading must be applied on both the side and the main span.
This results in the following forces:

$F_{p,pylon1} = (p_p * 0{,}5 * b_{bridge} + q_p) * l_{bridge} = (4.1 * 0{,}5 * 10 + 5{,}0)*160 = 4080$ kN

$F_{var,pylon1} = p_{var} * 0{,}5 * b_{bridge} * l_{bridge} = 4{,}0 * 0{,}5 * 10 * 160 = 3200$ kN

For design purpose the forces must be multiplied with the safety factors:

$F_{d,pylon1} = F_{p,pylon1} * \gamma_p + F_{var,pylon1} * \gamma_{var}$

$F_{d,pylon1} = 4080 * 1{,}2 + 3200 * 1{,}5 = 9696$ kN

There is also a vertical force as a result of the back stay-cable:
$F_{d,pylon2} = H_{1,d} * \tan\alpha = 2098$ kN

Resulting in the total loading of the pylon:
$F_{d,pylon} = F_{d,pylon1} + F_{d,pylon2} = 11794$ kN

For the buckling length the full length of the pylon, 42 m, is used. When looking at the buckling length in the plain of the stay-cables, the buckling length is smaller due to the fact of the bearing with a bending moment capacity at the bottom and the stay-cables supporting the top of the pylon (see figure 9.11), these cables can be considered as a translation spring. Out of plain the buckling length is the pylon length due to the bearing with a bending moment capacity at the bottom and the connection between the pylons at the top.

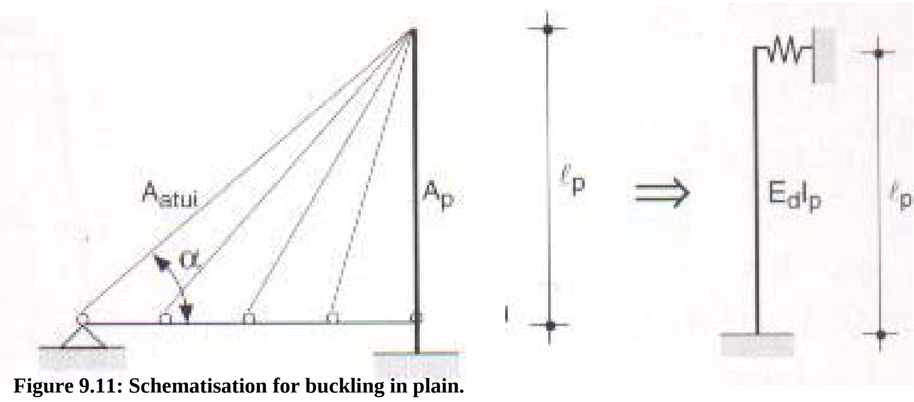


Figure 9.11: Schematisation for buckling in plain.

A square steel tube of 2000 * 2000 mm^2 is used for the pylon.
For a constant wall thickness it holds:
i_p = 0,40 * b = 800 mm

The slenderness of the pylon is given by:
$\lambda_p = l_{buc,p} / i_p$ = 42000 / 800 = 52,5

$\lambda_e = \pi * \sqrt{(E_d/f_{y,d})} = \pi * \sqrt{(210000/355)}$ = 76,4

Giving the pylon a relative slenderness of:

$\lambda_{rel} = \lambda_p / \lambda_e$ = 52,5 / 76.4 = 0,69

This leads to a buckling factor, $\omega_{buc,p}$ = 0,774 (curve b)

The required wall thickness of the pylon can be calculated now with:

$A_{min,p} = N_{d,pylon} / (\omega_{buc,p} * f_{y,d}) = 11794*10^3 / (0,774*355) = 43,1 * 10^3$ mm^2

$t_{gem,p} = A_{min,p} / 4b = 43,1 *10^3 / (4 * 2000)$ = 6 mm

9.5.3 Bridge deck

As a first assumption the following dimensions are taken:

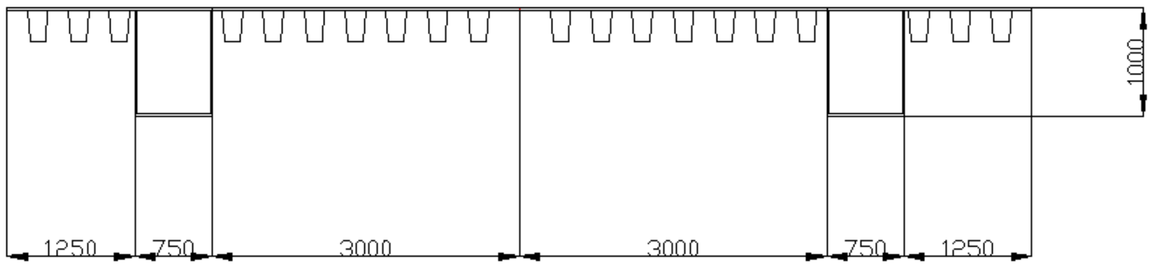


Figure 9.12: Dimensions bridge deck and main girder.

- The deck thickness is 16 mm
- The top and bottom flanges of the main girder are 28 mm
- The wall thickness of the main girder is 14 mm

This leads to the following characteristics for half the bridge width:
$A_{bridge} = A_{deck} + A_w + A_{fl,0} = 5000 * 16 + 2 * 1000 * 14 + 750 * 28 = 149 * 10^3\ mm^2$

$e_z = 241$ mm

$I_{y,bridge} = 19 * 10^9\ mm^4$

$W_{bridge,b} = 79 * 10^6\ mm^3$

$W_{bridge,o} = 25 * 10^6\ mm^3$

With these characteristics the calculations for the main girder of the bridge deck can be done. The lengthening of the longest stay-cable is a good first indication of the sag and with that the bending moment in the middle of the main girder. By tensioning the stay-cables, the vertical displacements caused by self-weight are reduced to 0, therefore the deflection is caused by only the variable loading.

$N_{var,14} = N_{var,14,d} / \gamma_{var} = 2605 / 1,5 = 1737$ kN

As further assumptions, the displacement of the top of the pylon, the lengthening of the back stay-cables and the shortening of the pylon are neglected.

$\delta_{var,14} = (N_{var,14} * l_{14}) / (A_{14} * E_d) * 1/\sin\alpha =$
$= (1737*10^3 * 162,8 * 10^3) / (6276 * 2,1*10^5) *1/\sin\alpha = 1164$ mm

With the deflection known, the bending moment can be calculated and from this bending moment the tensions in the girder can be calculated.

For the basis of this calculation a girder with a single point loading is assumed. One side is fully clamped and the other side is fully free.

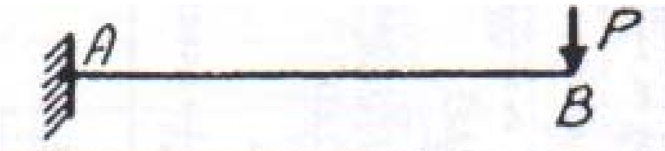


Figure 9.13: Schematisation of assumption for bending.

It holds:

$$\delta = \frac{Pl^3}{3EI} = \frac{Ml^2}{3EI}$$

This means:

$$M = \frac{3EI\delta}{l^2}$$

And with:

$$\sigma = \frac{M}{W}$$

This leads to the following stresses in the top and bottom flanges:

$$\sigma_{\text{var},b} = \frac{M}{W_b} \text{ (top)}$$

$$\sigma_{\text{var},o} = \frac{M}{W_o} \text{ (bottom)}$$

$\sigma_{var,b}$ = 7,0 N/mm^2

$\sigma_{var,o}$ = 22,1 N/mm^2

There is also a compressive force acting on the bridge deck. The maximum horizontal force is:

$N_{max,bridge,d} = \Sigma H_{p,1\text{-}4,d}$ = 26885 + 784 + 523 + 261 = 28423 kN

This leads to a compressive stress of:

$\sigma_{c,d} = N_{max,bridge,d} / A_{bridge}$ = 190 N/mm^2

To calculate the total compressive stress in the bottom flange at the location of the stay-cable – deck connection, first the stress caused by the bending moment needs to be multiplied with the safety factor.
This leads to:
$\sigma_{var,o,d} = 22{,}1 * 1{,}5 = 33{,}15 \text{ N/mm}^2$

This means that the total compressive stress in the bottom flange will be:

$\sigma_{fl,o,d} = \sigma_{var,o,d} + \sigma_{c,d} = 223{,}2 \text{ N/mm}^2$

This leads to a unity check of:

$$\frac{\sigma_{fl,o,d}}{f_{y,d}} = \frac{223{,}2}{355} = 0{,}63 \leq 1$$

This means that the dimensions chosen for the bridge deck are sufficient.

9.6 Computer calculations

The first rough analysis of the design was done by hand, to check whether or not the chosen dimensions, which were chosen with rules of thumb, were sufficient. To check the real bridge behaviour computer calculations are required, from which a better insight into the exact forces acting on the different members is obtained.

For the computer calculations Matrixframe 4.1 is used. As an input the dimensions and characteristics calculated in the previous sections are used. The connection between the cables and the deck are schematised as hinges.
There are four load cases which are examined, these cases are:
1. Variable loading on the whole bridge.
2. Variable loading only on the two fields next to the pylon.
3. Variable loading on the side span.
4. Variable loading on the main span.

For the exact parameters see appendix 3.

9.6.1 Stay-cables

As stated before there are two types of loadings acting on the stay-cables. One is a permanent, caused by the self-weight of the construction and the other is a variable caused by traffic loading.

The permanent loading was already calculated by hand. The same was done for the variable loading, however this had to be checked by a computer calculation, due to the fact that only three types of cable diameter were chosen instead of a free diameter in the hand calculation. Therefore the governing maximum normal tensile force which results from the computer calculations for the variable loading will be combined with the hand-calculated permanent loading to check if the chosen diameters of the stay-cables are sufficient. Both the results of the hand calculations and of the computer calculations are given in table 9.10.

Table 9.10: Results for both hand and computer calculation per stay cable.

Cable number	$N_{p,tui,d}$ [kN] hand	$N_{var,tui,d}$ [kN] hand	$N_{var,tui,d}$ [kN] lc1	$N_{var,tui,d}$ [kN] lc2	$N_{var,tui,d}$ [kN] lc3	$N_{var,tui,d}$ [kN] lc4	$N_{var,tui,d}$ [kN] max
1	12864	12602	8902	-3	-1196	10097	10097
2	925	906	666	12	670	-4	670
3	716	702	585	-34	573	12	585
4	555	544	427	210	461	-34	461
5	555	544	439	210	-30	470	470
6	716	702	560	-34	10	550	560
7	925	906	725	12	-3	729	729
8	1154	1131	905	-4	1	904	905
9	1396	1367	1095	1	0	1095	1095
10	1643	1609	1286	0	0	1286	1286
11	1894	1855	1494	0	0	1494	1494
12	2147	2103	1641	0	0	1641	1641
13	2402	2353	2083	0	0	2083	2083
14	2659	2605	1705	0	0	1705	1705

For further calculations the results from the computer calculations are taken as governing, due to the fact that this was a more accurate calculation than the hand calculation.
With both the normal force caused by self-weight and the maximum normal force caused by traffic loading known, the stress these forces cause in the stay cables can be calculated.

The unity-check for the stay-cables is:
$\sigma_{tui,d} / f_{y,d} \leq 1$

With:
$\sigma_{tui,d} = (N_{p,tui,d} + N_{var,tui,d,max}) / A_{tui}$
$f_{y,d} = 900\ N/mm^2$

This leads to the following results:

Table 9.11: Unity-check stay-cables

Cable number	$N_{p,tui,d}$ [kN]	$N_{var,tui,d,max}$ [kN]	A_{tui} [mm^2]	$\sigma_{tui,d}$ [N/mm^2]	Unity-check
1	12864	10097	30957	742	0,82
2	925	670	2136	747	0,83
3	716	585	2136	609	0,68
4	555	461	2136	476	0,53
5	555	470	2136	480	0,53
6	716	560	2136	597	0,66
7	925	729	2136	774	0,86
8	1154	905	4358	472	0,52
9	1396	1095	4358	572	0,64
10	1643	1286	4358	672	0,75
11	1894	1494	4358	777	0,86
12	2147	1641	6276	604	0,67
13	2402	2083	6276	715	0,79
14	2659	1705	6276	695	0,77

Now all stay-cables have been checked according to the NEN, and are found to be OK. In table 9.12 an overview of the stay-cables and their diameter and area is given.

Table 9.12: Chosen diameter per stay-cable.

Cable number	**Chosen diameter [mm] – area [mm^2]**
Back-stay-cable (1)	212 - 30957
2	56 - 2136
3	56 - 2136
4	56 - 2136
5	56 - 2136
6	56 - 2136
7	56 - 2136
8	80 - 4358
9	80 - 4358
10	80 - 4358
11	80 - 4358
12	96 - 6276
13	96 - 6276
14	96 - 6276

9.6.2 Pylon

The pylon was dimensioned with the results of the hand calculations, this lead to a square tube of 2000 * 2000 mm^2 with a wall thickness of 6 mm. Stiffeners are applied to prevent instability. The stiffeners will have the following dimensions:
t_{tr} = 10 mm
h_{tr} = 300 mm
b_{btr} = 400 mm
b_{otr} = 300 mm
l_{trw} = 403 mm

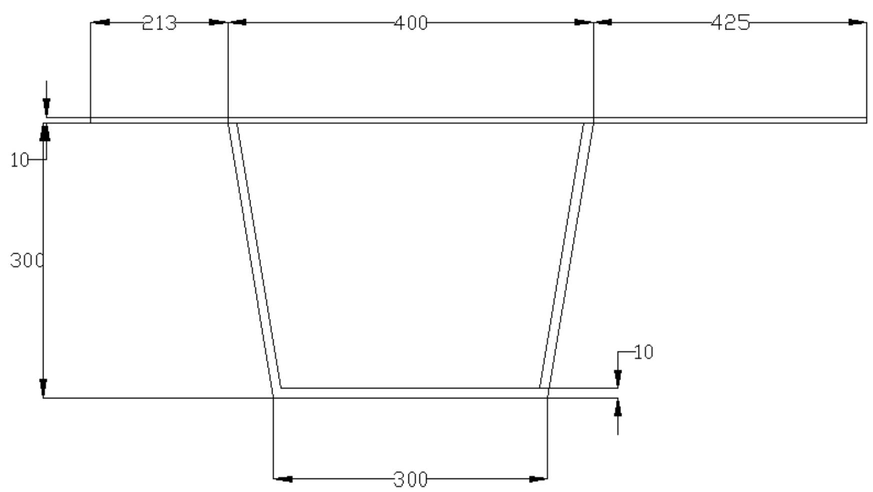


Figure 9.14: Stiffener in the pylon.

This gives the stiffener a total area of:
$A_{tr} = 9080 \text{ mm}^2$

Other characteristics:
$e_z = 200$ mm

Total area pylon:
$A_p = 4 * b_p * t_p + 8 * A_{tr} = 4 * 2000 * 6 + 9080 = 57080 \text{ mm}^2$

Other characteristics:
$I_p = 32 * 10^9 \text{ mm}^4$

$W_p = I_p / (b_p/2) = 32 * 10^9 / (2000/2) = 32 * 10^6 \text{ mm}^3$

For the calculation of the maximum stress in the pylon, two loadings need tot be taken into account. First the loading caused by the self-weight of the construction, this was calculated by hand. The second is the loading caused by traffic loading, this loading was calculated by computer. The largest load on the pylon caused by the four load cases is governing for this situation. These maximum forces are caused by load case 1 and are:
$N_{var,d} = 5599$ kN
$M_{var,d} = 7829$ kNm.

The loadings cause two forces on the pylon, a normal compressive force:

$N_{c,s,d} = N_{p,d} + N_{var,d} = 11794 + 5599 = 17393$ kN

And a bending moment:
$M_{y,s,d} = M_{p,d} + M_{var,d} = 0 + 7829 = 7829$ kNm

and

$M_{z,s,d} = 0$ kNm

The maximum compressive stress in the pylon is:

$\sigma_{c,d} = N_{c,s,d} / Ap = 17393*10^3 / 57080 = 305 \text{ N/mm}^2$

$\sigma_{m,d} = M_{y,s,d} / W_p = 7829*10^6 / 32*10^6 = 245 \text{ N/mm}^2$

$\sigma_{tot,d} = \sigma_{c,d} + \sigma_{m,d} = 305 + 245 = 550 \text{ N/mm}^2$

This leads to a unity-check of:

$\sigma_{tot,d} / f_{y,d} = 550 / 355 > 1$

This means the dimensions of the pylon are not sufficient to withstand the loading.

To make it possible for the pylon to withstand the loading, the dimensions need to be increased. The new dimensions are:
$b_p = h_p = 2500$ mm
$t_p = 10$ mm

The new characteristics are:
Total area pylon:
$A_p = 4 * b_p * t_p + 8 * A_{tr} = 4 * 2500 * 10 + 9080 = 109080 \text{ mm}^2$

Other characteristics:
$I_p = 83{,}3 * 10^9 \text{ mm}^4$

$W_p = I_p / (b_p/2) = 83{,}3 * 10^9 / (2500/2) = 66{,}7 * 10^6 \text{ mm}^3$

The maximum compressive stress in the pylon is:

$\sigma_{c,d} = N_{c,s,d} / Ap = 17393*10^3 / 109080 = 159 \text{ N/mm}^2$

$\sigma_{m,d} = M_{y,s,d} / W_p = 7829*10^6 / 66{,}7*10^6 = 117 \text{ N/mm}^2$

$\sigma_{tot,d} = \sigma_{c,d} + \sigma_{m,d} = 159 + 117 = 276 \text{ N/mm}^2$

This leads to a unity-check of:

$\sigma_{tot,d} / f_{y,d} = 276 / 355 = 0{,}78 < 1$

This means the dimensions of the pylon are sufficient to withstand the loading.

The pylon does not only have to be controlled for the stress, but also for its stability. The pylon has to be checked for buckling in two directions, one in the plain of the stay-cables and one out of the plain of the stay cables.

First the bucking in the plain of the stay-cables will be checked.

When comparing the bridge deck stiffness to that of the back stay-cable, it is clear that the stiffness of the deck is neglect able. The top of the pylon is mainly kept in place by the back stay-cable. When addressing the buckling stability, this back stay-cable can be schematised as a translation spring. The bearing at the bottom of the pylon can be schematised as a bearing with moment bearing capacity. The pylon itself can be schematised as a rod. This leads to the schematisation of figure 9.15.

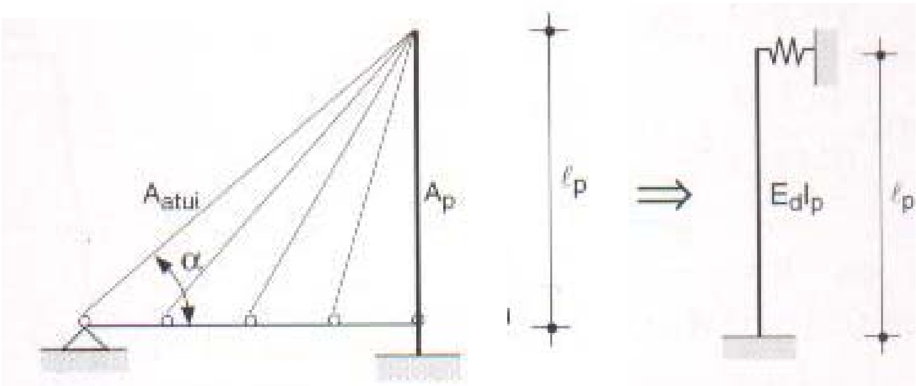


Figure 9.15: Schematisation pylon for buckling in plain of stay-cables.

When applying a horizontal unity force of, F_h = 1 kN, to the top of the pylon, this leads to a lengthening of the back stay-cable of δ_{tui} and a horizontal displacement of the pylon of δ_p.

$$\delta_{tui} = \frac{F_h}{\cos\alpha}\frac{l_1}{E_d A_1} = \frac{1}{\cos(0{,}995)} \cdot \frac{130{,}6}{2{,}1 \cdot 10^5 \cdot 10^3 \cdot 30957 \cdot 10^{-6}} = 20{,}1 \cdot 10^{-6}\, m/kN$$

$$\delta_p = \frac{\delta_{tui}}{\cos\alpha} = \frac{20{,}1 \cdot 10^{-6}}{\cos(0{,}995)} = 20{,}1 \cdot 10^{-6}\, m/kN$$

This leads to the spring stiffness, k_{spring}, at the top of the pylon:

$$k_{spring} = \frac{F_h}{\delta_p} = \frac{1}{20{,}1 \cdot 10^{-6}} = 49{,}8 \cdot 10^3\, kN/m$$

This means the factor β will be:

$$\beta = \frac{k_{spring} l_p^3}{3E_d I_p} = \frac{49{,}8 \cdot 10^3 * 42^3}{3 * 2{,}1 \cdot 10^5 \cdot 10^3 * 83{,}3 \cdot 10^{-3}} = 70 = \infty$$

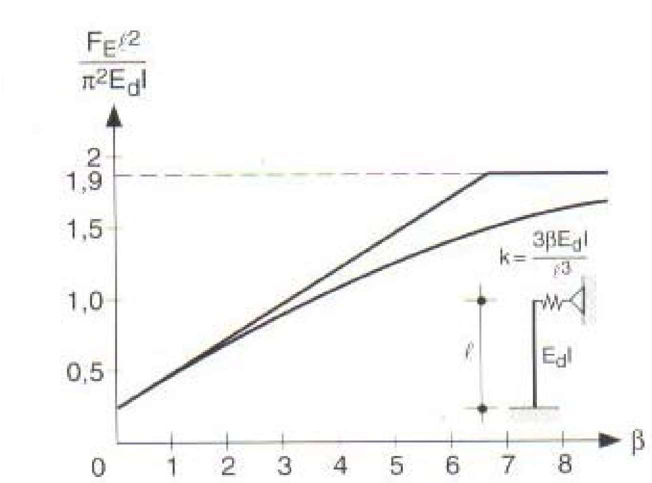


Figure 9.16: Relation buckling force and β.

This leads to the following buckling force for the pylon:

$$\frac{F_e l_{y,sys}^2}{\pi^2 E_d I} = 1{,}9 \Rightarrow \frac{\frac{\pi^2 E_d I}{l_{y,buc}^2} l_{y,sys}^2}{\pi^2 E_d I} = \frac{l_{y,sys}^2}{l_{y,buc}^2} = 1{,}9$$

$$l_{y,buc} = \frac{l_{y,sys}}{\sqrt{1{,}9}} = \frac{42000}{\sqrt{1{,}9}} = 30470mm$$

$$F_{y,E,d} = \frac{\pi^2 E_d I_p}{l_{y,buc}^2} = \frac{\pi^2 \cdot 2{,}1 \cdot 10^5 \cdot 83{,}3 \cdot 10^9}{30470^2} \cdot 10^{-3} = 186 \cdot 10^3 kN$$

$$n_y = \frac{F_{y,E,d}}{F_{tot,d}} = \frac{186 \cdot 10^3}{17393} = 3{,}9$$

When looking at the buckling out of the plain of the stay-cables it holds that the top of the pylon is kept in place by a beam which is connected to the other pylon. Therefore for buckling in this direction a portal construction has to be evaluated. By applying a beam with $I_{beam} = 2I_p$ the buckling length of the pylon is reduced.

The buckling force and length are:

$$\frac{kl_p}{\tan(kl_p)} = -\frac{6 l_p I_{beam}}{l_{beam} I_p} = -\frac{6 * 42 * I_p}{10 * I_p} = -25{,}2 \Rightarrow kl_p = 3{,}02$$

With:

$$k^2 = \frac{F_{z,E,d}}{E_d I_p} = \frac{\pi^2}{l_{buc,z}^2} \Rightarrow k = \frac{\pi}{l_{buc,z}} = \frac{\pi l_p}{l_{buc,z} l_p} \Rightarrow l_{buc,z} = \frac{\pi l_p}{kl_p}$$

This means in this case:

$$l_{buc,z} = \frac{\pi * 42}{3{,}02} = 43{,}7m$$

$$F_{z,E,d} = \frac{\pi^2 E_d I_p}{l_{buc,z}^2} = \frac{\pi^2 * 2{,}1 \cdot 10^5 * 83{,}3 \cdot 10^9}{43700^2} \cdot 10^{-3} = 90{,}4 \cdot 10^3 kN$$

$$n_z = \frac{F_{z,E,d}}{F_{tot,d}} = \frac{90{,}4 \cdot 10^3}{17393} = 5{,}2$$

The checking of the pylon on compression and bending, including second-order effects, is done conform NEN 6771.

$$i_p = \sqrt{\frac{I_p}{A_p}} = \sqrt{\frac{83{,}3 \cdot 10^9}{109080}} = 874mm$$

The slenderness is:

$$\lambda_{y,p} = \frac{l_{buc,y}}{i_p} = \frac{30470}{874} = 34{,}9$$

$$\lambda_{z,p} = \frac{l_{buc,z}}{i_p} = \frac{43700}{874} = 50$$

This means the relative slenderness becomes:

$$\lambda_{y,rel} = \frac{\lambda_{y,p}}{\lambda_e} = \frac{34{,}9}{76{,}4} = 0{,}46$$

$$\lambda_{z,rel} = \frac{\lambda_{z,p}}{\lambda_e} = \frac{50}{76{,}4} = 0{,}65$$

The maximum moment capacity of the pylon is:

$$M_{y,u,d} = W_{y,el} * f_{y,d} = 66{,}7 * 10^6 * 355 \cdot 10^{-6} = 23679 kNm$$

The maximum normal force capacity is:

$$N_{c,u,d} = A_p * f_{y,d} = 109080 * 355 \cdot 10^{-3} = 38723 kN$$

The enlargement factors for the Eulerse buckling force are:

$$\frac{n_y}{n_y - 1} = \frac{3{,}9}{3{,}9 - 1} = 1{,}34$$

$$\frac{n_z}{n_z - 1} = \frac{5{,}2}{5{,}2 - 1} = 1{,}24$$

The imperfections are applied according to NEN 6771. The total amount of welding is small relative to the total construction, therefore buckling curve B is applied for both axels.
With α_k = 0,34 and λ_0 = 0,2 it holds:

$$e_y^* = \alpha_k (\lambda_{y,rel} - \lambda_0) \frac{M_{u,d}}{N_{c,u,d}} = 0{,}34(0{,}46 - 0{,}2) \frac{23679}{38723} \cdot 10^3 = 54 mm$$

$$e_z^* = \alpha_k (\lambda_{z,rel} - \lambda_0) \frac{M_{u,d}}{N_{c,u,d}} = 0{,}34(0{,}65 - 0{,}2) \frac{23679}{38723} \cdot 10^3 = 94 mm$$

For the portal an equivalent moment needs to be calculated, this is:

$$M_{y,equ,s,d} = 0{,}6 M_{y,tot,d} = 0{,}6 * 7829 = 4697 kNm$$

The unity-check for buckling in plain now becomes:

$$\frac{N_{c,s,d}}{N_{c,u,d}}+\frac{n_y}{n_y-1}\frac{M_{y,equ,s,d}}{\omega_{kip}M_{y,u,d}}+\frac{n_y}{n_y-1}\frac{F_{tot,y,s,d}e_y^*}{M_{y,u,d}}$$
$$=\frac{17393}{38723}+1{,}34\cdot\frac{4697}{1\cdot 23679}+1{,}34\cdot\frac{17393*54\cdot 10^{-3}}{23679}$$

$$=0{,}45+0{,}27+0{,}05=0{,}77\le 1$$

The pylon can withstand buckling in the plain of the stay-cables.

$$\frac{N_{c,s,d}}{N_{c,u,d}}+\frac{n_y}{n_y-1}\frac{M_{y,equ,s,d}}{\omega_{kip}M_{z,u,d}}+\frac{n_z}{n_z-1}\frac{F_{tot,y,s,d}e_z^*}{M_{z,u,d}}$$
$$=\frac{17393}{38723}+1{,}24\cdot\frac{4697}{1\cdot 23679}+1{,}24\cdot\frac{17393*94\cdot 10^{-3}}{23679}$$

$$=0{,}45+0{,}25+0{,}09=0{,}79\le 1$$

The pylon can withstand buckling out of the plain of the stay-cables.

With both the strength and the stability of the pylon checked, it can be concluded that the chosen dimensions where sufficient and will therefore be applied.

Local instability

Besides the pylon as a whole, like in the previous part, also the stability and strength of the local systems needs to be checked. To do so, the local buckling of the plates of the pylon needs to be checked.
First the plates are checked, then the stiffeners.

Plate buckling

The forces applied to the pylon are:
$N_{c,s,d}$ = 17393 kN
$M_{y,s,d}$ = 7829 kNm
$M_{z,s,d} = N_{c,s,d} * e_z$ = 17393 * 94 * 10^{-3} = 1635 kNm

The stresses in the corners of the pylon now become:

$$\sigma_{c,d}=\frac{N_{c,s,d}}{A_p}=\frac{17393}{109080}\cdot 10^3=159N/mm^2$$
$$\sigma_{y,m,d}=\frac{M_{y,s,d}}{W_p}\frac{n_y}{n_y-1}=\frac{7829}{66{,}7}\cdot 1{,}34=157N/mm^2$$

$$\sigma_{z,m,d}=\frac{M_{z,s,d}}{W_p}\frac{n_z}{n_z-1}=\frac{1635}{66{,}7}\cdot 1{,}24=30N/mm^2$$

$$\sigma_1 = -\sigma_{c,d} - \sigma_{y,m,d} - \sigma_{z,m,d} = -159 - 157 - 30 = -346 N/mm^2$$
$$\sigma_2 = -\sigma_{c,d} - \sigma_{y,m,d} + \sigma_{z,m,d} = -159 - 157 + 30 = -286 N/mm^2$$
$$\sigma_3 = -\sigma_{c,d} + \sigma_{y,m,d} + \sigma_{z,m,d} = -159 + 157 + 30 = 28 N/mm^2$$
$$\sigma_4 = -\sigma_{c,d} + \sigma_{y,m,d} - \sigma_{z,m,d} = -159 + 157 - 30 = -32 / mm^2$$

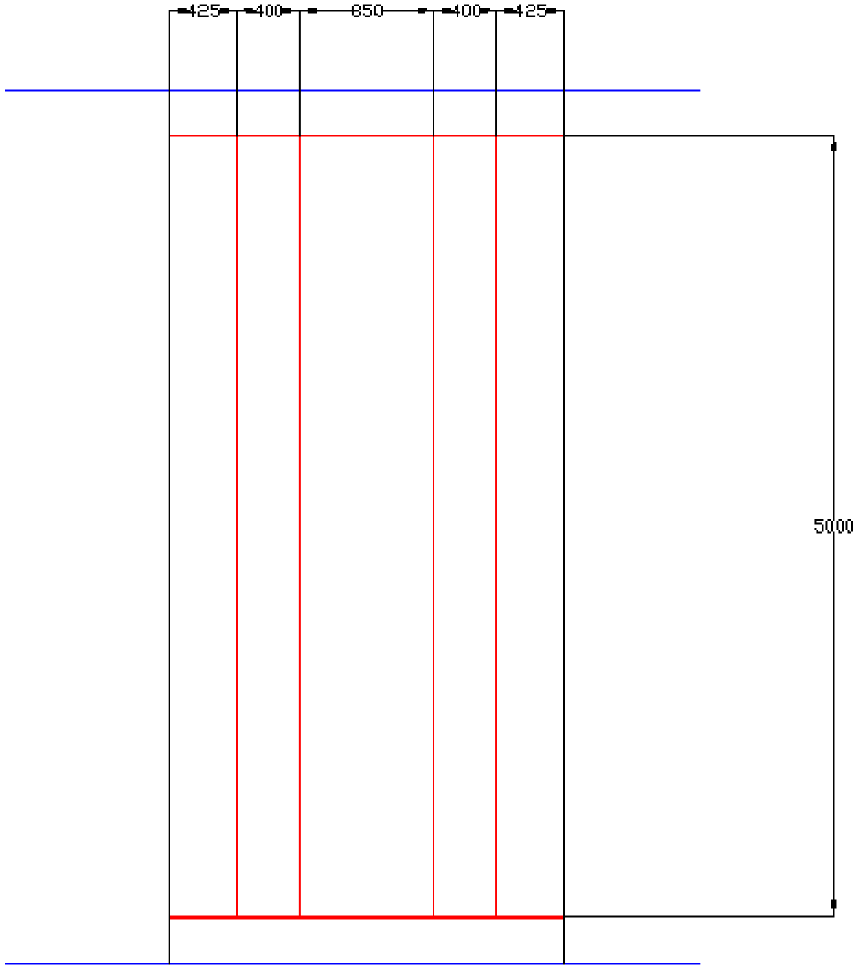


Figure 9.17: Side view pylon with stiffeners.

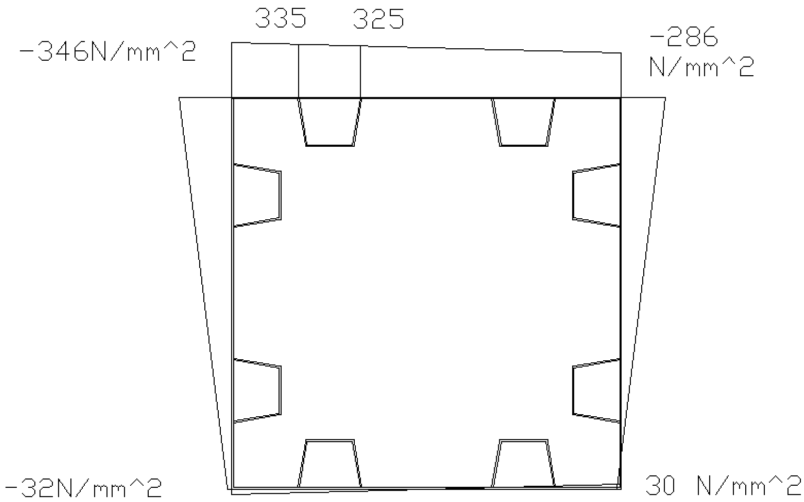


Figure 9.18: Stress distribution along the pylon.

There are two plate fields in the pylon with a width of b_1= 425 mm and b_2 = 800 mm. Also there are horizontal plates spaced c.o.c. 5000 mm.

According to NEN 6771, art. 13 it holds for plate field 1:

$$\alpha = \frac{a}{b_1} = \frac{5000}{425} = 11{,}76$$

σ_{min} = -335 N/mm^2
σ_{max} = -346 N/mm^2

This leads to:

$$\Psi_x = \frac{\sigma_{\min}}{\sigma_{\max}} = \frac{335}{346} = 0{,}968$$

This leads to the buckling coefficient:

$$k_{\sigma,x} = \frac{8{,}2}{\Psi_k + 1{,}05} = \frac{8{,}2}{0{,}968 + 1{,}05} = 4{,}063$$

$$\sigma_E = 19\left(\frac{100 t_p}{b_1}\right)^2 = 19\left(\frac{100 * 10}{425}\right)^2 = 105 N/mm^2$$

$$\sigma_{i,k,x} = \sigma_E k_{\sigma,x} = 105 * 4{,}063 = 427 N/mm^2$$

$$\lambda_{plate,rel} = \sqrt{\frac{f_{y,d}}{\sigma_{i,k,x}}} = \sqrt{\frac{355}{427}} = 0{,}91$$

$$\sigma_{plooi,rel} = 1{,}474 - 0{,}677\lambda_{plate,rel} = 1{,}474 - 0{,}677 \cdot 0{,}91 = 0{,}86$$

$$C_{\sigma,x} = (0{,}1129\Psi_x + 0{,}1129)\lambda_{plate,rel} - 0{,}0790\Psi_x + 0{,}9210$$
$$= (0{,}1129 \cdot 0{,}968 + 0{,}1129)0{,}91 - 0{,}079 \cdot 0{,}968 + 0{,}9210 = 1{,}047$$

$$\sigma_{plooi,d} = \sigma_{plooi,rel} f_{y,d} \frac{1}{C_{\sigma,x}} = 0{,}86 * 355 * \frac{1}{1{,}047} = 292 N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{x,s,d}}{\sigma_{plooi,d}} = \frac{346}{292} = 1{,}18 \geq 1$$

This means that with the currently chosen dimensions the plate will buckle locally.
There are two options to prevent this, increasing the number of stiffeners or increasing the wall thickness of the pylon. Because it is easier to increase the wall thickness of the pylon, this option is chosen. The wall thickness is increased from 10 mm to 15 mm. This increases the total area of the pylon to 159080 mm^2

This means the new compressive stress will be:

$$\sigma_{c,d} = \frac{N_{c,s,d}}{A_p} = \frac{17393}{159080} \cdot 10^3 = 109 N/mm^2$$

This leads to a maximum stress in the corner of:

$$\sigma_1 = -\sigma_{c,d} - \sigma_{y,m,d} - \sigma_{z,m,d} = -109 - 157 - 30 = -296 N/mm^2$$

σ_{min} = -286 N/mm^2
σ_{max} = -296 N/mm^2

This leads to:

$$\Psi_x = \frac{\sigma_{\min}}{\sigma_{\max}} = \frac{286}{296} = 0{,}966$$

This leads to the buckling coefficient:

$$k_{\sigma,x} = \frac{8{,}2}{\Psi_k + 1{,}05} = \frac{8{,}2}{0{,}966 + 1{,}05} = 4{,}067$$

$$\sigma_E = 19\left(\frac{100 t_p}{b_1}\right)^2 = 19\left(\frac{100 * 15}{425}\right)^2 = 237 N/mm^2$$

$$\sigma_{i,k,x} = \sigma_E k_{\sigma,x} = 237 * 4{,}067 = 964 N/mm^2$$

$$\lambda_{plate,rel} = \sqrt{\frac{f_{y,d}}{\sigma_{i,k,x}}} = \sqrt{\frac{355}{964}} = 0{,}61$$

This means:

$$\sigma_{plooi,rel} = 1$$

$$C_{\sigma,x} = 1$$

$$\sigma_{plooi,d} = \sigma_{plooi,rel} f_{y,d} \frac{1}{C_{\sigma,x}} = 1*355*1 = 355N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{x,s,d}}{\sigma_{plooi,d}} = \frac{296}{355} = 0{,}83 \le 1$$

This means the plate can withstand the forces.

Longitudinal stiffeners

The side of the trough will work as a stiffener in longitudinal direction. The average tension in a trough is:

$$\sigma_{tr} = (-286 + -276)/2 = -281N/mm^2$$

Under compression the buckling coefficient is:
$k_{\sigma,x} = 4$

Now the stiffener can be checked:

$$\sigma_E = 19\left(\frac{100t_{tr}}{l_{trw}}\right)^2 = 19\left(\frac{100*10}{403}\right)^2 = 117N/mm^2$$

$$\sigma_{i,k,x} = \sigma_E k_{\sigma,x} = 117*4 = 468N/mm^2$$

$$\lambda_{plate,rel} = \sqrt{\frac{f_{y,d}}{\sigma_{i,k,x}}} = \sqrt{\frac{355}{468}} = 0{,}871$$

$$\sigma_{plooi,rel} = 1{,}474 - 0{,}677\lambda_{plate,rel} = 1{,}474 - 0{,}677 \cdot 0{,}871 = 0{,}88$$

$$C_{\sigma,x} = (0{,}1129\Psi_x + 0{,}1129)\lambda_{plate,rel} - 0{,}0790\Psi_x + 0{,}9210$$
$$= (0{,}1129 \cdot 0{,}991 + 0{,}1129)1{,}08 - 0{,}079 \cdot 0{,}991 + 0{,}9210 = 1{,}085$$

$$\sigma_{plooi,d} = \sigma_{plooi,rel} f_{y,d} \frac{1}{C_{\sigma,x}} = 0{,}885*355*\frac{1}{1{,}085} = 290N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{x,s,d}}{\sigma_{plooi,d}} = \frac{281}{290} = 0{,}97 \le 1$$

This means the wall thickness of the trough is sufficient.

Buckling stability trough

The active width of the pylon and the trough are calculated according NEN 6771.

$$b_{e,p} = 1{,}33 t_p \sqrt{\frac{E_d}{f_{y,d}}} = 1{,}33 \cdot 15 \sqrt{\frac{2{,}1 \cdot 10^5}{355}} = 485mm$$

$$b_{e,tr} = 1{,}33 t_{tr} \sqrt{\frac{E_d}{f_{y,d}}} = 1{,}33 \cdot 10 \sqrt{\frac{2{,}1 \cdot 10^5}{355}} = 323mm$$

This leads to an effective area of the stiffener of:

$$A_{plate} = t_p \left(\frac{b_1 + b_2}{2} + b_{e,p} \right) = 15 \left(\frac{425 + 850}{2} + 485 \right) = 16838mm^2$$

$$A_{st} = A_{plate} + A_{tr} = 16838 + 9080 = 25918mm^2$$

$$z_{st} = \frac{9080(200 + 15) + 16838 \cdot 7{,}5}{25918} = 80mm$$

This leads to the following characteristics:
$I_{st} = 288{,}4 * 10^6\ mm^4$

$$W_{st} = \frac{I_{st}}{(h_{tr} - z_{st})} = \frac{288{,}4 * 10^6}{300 - 80} = 1{,}31 \cdot 10^6\ mm^3$$

This leads to a relative slenderness:

$$i_{st} = \sqrt{\frac{I_{st}}{A_{st}}} = \sqrt{\frac{288{,}4 \cdot 10^6}{25918}} = 105{,}5mm$$

$$\lambda_{st} = \frac{l_{st}}{i_{st}} = \frac{5000}{105{,}5} = 47{,}4$$

$$\lambda_{rel} = \frac{\lambda_{st}}{\lambda_e} = \frac{47{,}4}{76{,}4} = 0{,}62$$

Because of the limited amount of welding compared to the total cross-section, buckling curve B can be applied here with:
$\alpha_k = 0{,}34$
$\lambda_0 = 0{,}2$
This leads to $\omega_{buc,st} = 0{,}828$

The stress in the trough is, $\sigma_{tr} = -281 N/mm^2$

This leads to a unity-check of:

$$\frac{\sigma_{tr}}{\omega_{buc,st} f_{y,d}} = \frac{281}{0{,}828 * 355} = 0{,}96 \leq 1$$

This means the dimensions of the stiffener are sufficient to withstand buckling.

The moment bearing capacity and the normal force capacity of the stiffener are:
$M_{u,d} = W_{st} * f_{y,d} = 1{,}31*10^6 * 355 = 465$ kNm
$N_{c,u,d} = A_{st} * f_{y,d} = 25918 * 355 = 9201$ kN

The Eulerse buckling force of the trough-stiffener becomes:

$$F_{E,d} = \frac{\pi^2 E_d I_{st}}{l^2_{buc,st}} = \frac{\pi^2 * 2{,}1\cdot 10^5 * 288{,}4\cdot 10^6}{5000^2}\cdot 10^{-3} = 23910kN$$

$$n = \frac{F_{E,d}}{N_{c,s,d}} = \frac{23910}{7283} = 3{,}28$$

$$\frac{n}{n-1} = 1{,}44$$

This leads to a second order of:

$$e^* = \alpha_k(\lambda_{rel} - \lambda_0)\frac{M_{u,d}}{N_{c,u,d}} = 0{,}34(0{,}62 - 0{,}2)\frac{465}{9201}\cdot 10^3 = 7{,}2mm$$

Now the trough can be tested with a unity-check for the compression loading and bending moment:

$$\frac{N_{c,s,d}}{N_{c,u,d}} + \frac{n}{n-1}\frac{N_{c,u,d}e^*}{M_{u,d}} = \frac{7283}{9201} + 1{,}44\frac{9201*7{,}2}{465} = 0{,}79 + 0{,}20 = 0{,}99 \leq 1$$

This means the dimensions are sufficient.

The pylon and all of its parts are now checked according NEN. The governing check for the pylon was the local plate buckling. Due to the fact that plate buckling needs to be prevented, the wall thickness of the pylon was increased. This also has influence on the pylon as a whole. Due to the increase in wall thickness also the cross-sectional area of the pylon and the over-all stiffness of the pylon will increase. This has a positive influence on the stresses and the stability of the system, meaning that the stress in the pylon will be lower and the pylon will be less susceptible to buckling. Therefore the new dimensions will not be checked again.

In picture 9.19 an overview of the chosen dimensions is given.

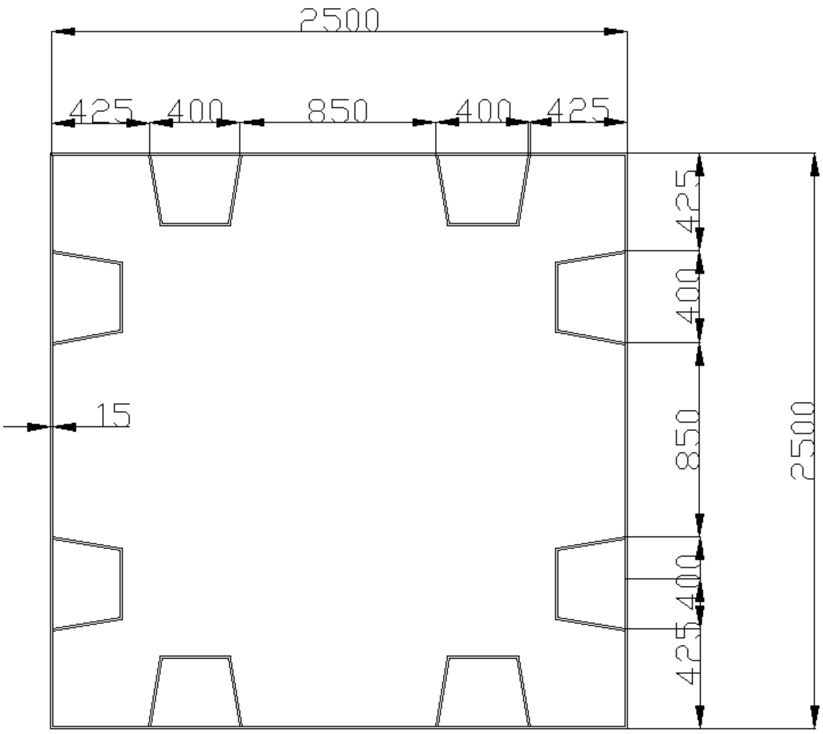


Figure 9.19: Cross-section of the pylon

Functional demand pylon

For maintenance purposes there must be a stairwell and elevator present in the pylon. To facilitate these two a minimum width and length of 3000 * 3000 mm is required. The required dimensions from a structural point of view are 2500 * 2500 mm. The functional demand is the governing in this case, therefore the final dimensions of the pylon will be 3000 * 3000 mm. No new calculations are done, because the enlargement of the dimensions has a positive effect on the stresses in the pylon.
The final dimensions are given in figure 9.20.

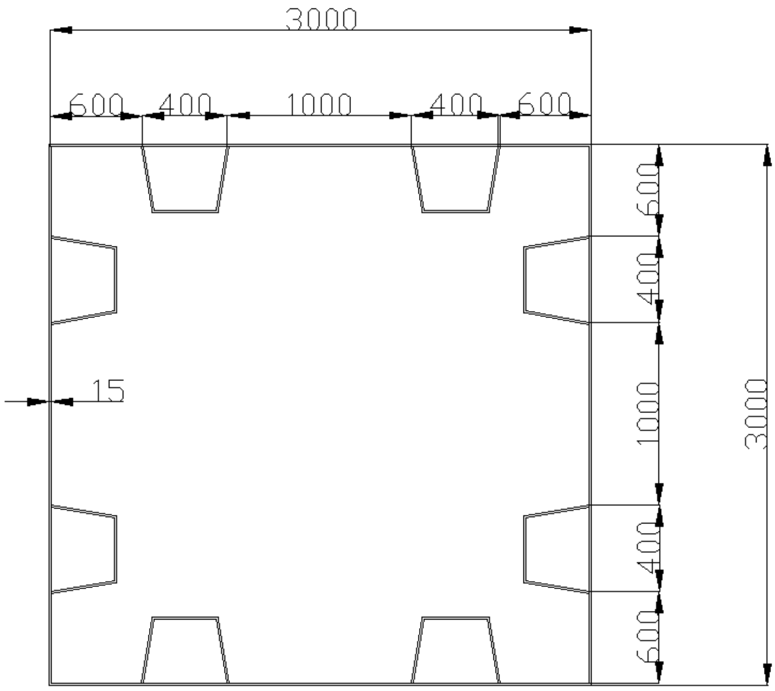


Figure 9.20: Cross-section of pylon with final dimensions.

9.6.3 Bridge deck

As a starting point for the calculations, the dimensions found with the hand calculations are taken:

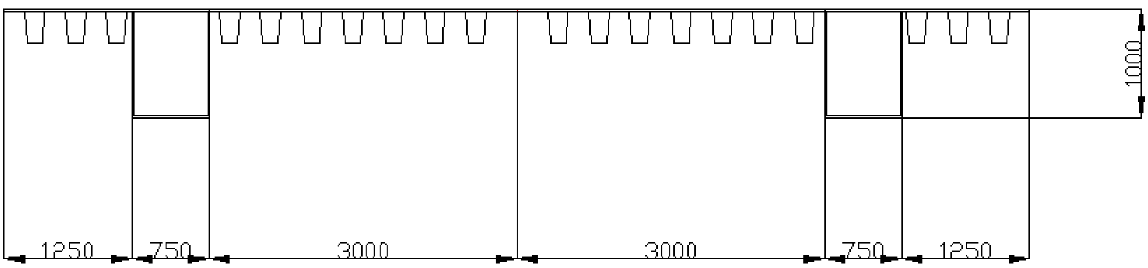


Figure 9.21: Cross-section of bridge deck

- The deck thickness is 16 mm
- The top and bottom flanges are 28 mm
- The wall thickness is 14 mm

This leads to the following characteristics for half the bridge width:
$A_{bridge} = A_{deck} + A_w + A_{fl,0} = 5000 * 16 + 2 * 1000 * 14 + 750 * 28 = 149 * 10^3$ mm^2

$e_z = 241$ mm

$I_{y,bridge} = 19 * 10^9$ mm^4

$W_{bridge,b} = 79 * 10^{6}$ mm^3

$W_{bridge,o} = 25 * 10^6$ mm^3

For the troughs under the deck the following dimensions are taken:

- b_{btr} = 300 mm
- b_{otr} = 200 mm
- h_{tr} = 300 mm
- t_{tr} = 6 mm

This leads to a cross-section of:
A_{tr} = 4850 mm^2
e_z = 189 mm

A computer analysis was used for the load distribution of the variable loading. The member forces caused by the permanent loading were already calculated when doing the hand calculations.
For the computer analysis of the bridge deck, the same 4 load cases as for the previous analysis were taken. There are two governing load cases. One with the largest negative bending moment and one with the largest normal compressive force.
Load case 1 gives the largest normal force and load case 2 gives the largest negative bending moment.

In table 9.13 an overview of the governing maximum forces is given.

Table 9.13: Overview governing forces in bridge deck

Loading	**$N_{s,d}$ [kN]**	**$M_{y,s,d}$ [kNm]**
Permanent loading	-14373	-891
Variable loading lc1	-10221	-607
Variable loading lc2	-88	-677

Checking for stability and strength of the bridge girder is done according to NEN 6771. By applying multiplier factors and eccentricities the instability problem is reduced to a strength calculation.

Due to the different loadings, the following stresses occur in the bridge deck:

Due to permanent loading:

$$\sigma_{c,p,d} = \frac{N_{p,d}}{A_{bridge}} = \frac{14373 \cdot 10^3}{149 \cdot 10^3} = 96{,}4 N/mm^2$$

$$\sigma_{m,p,b,d} = \frac{M_{p,d}}{W_{bridge,b}} = \frac{891 \cdot 10^6}{79 \cdot 10^6} = 11{,}3 N/mm^2$$

$$\sigma_{m,p,o,d} = \frac{M_{p,d}}{W_{bridge,0}} = \frac{891 \cdot 10^6}{25 \cdot 10^6} = 35{,}6 N/mm^2$$

Due to load case 1:

$$\sigma_{c,\text{var}1,d} = \frac{N_{\text{var}1,d}}{A_{bridge}} = \frac{10221 \cdot 10^3}{149 \cdot 10^3} = 68{,}6 N/mm^2$$

$$\sigma_{m,\text{var}1,b,d} = \frac{M_{\text{var}1,d}}{W_{bridge,b}} = \frac{607 \cdot 10^6}{79 \cdot 10^6} = 7{,}7 N/mm^2$$

$$\sigma_{m,\text{var}1,o,d} = \frac{M_{\text{var}1,d}}{W_{bridge,0}} = \frac{607 \cdot 10^6}{25 \cdot 10^6} = 24{,}3 N/mm^2$$

Due to load case 2:

$$\sigma_{c,\text{var}2,d} = \frac{N_{\text{var}2,d}}{A_{bridge}} = \frac{88 \cdot 10^3}{149 \cdot 10^3} = 0{,}6 N/mm^2$$

$$\sigma_{m,\text{var}2,b,d} = \frac{M_{\text{var}2,d}}{W_{bridge,b}} = \frac{677 \cdot 10^6}{79 \cdot 10^6} = 8{,}6 N/mm^2$$

$$\sigma_{m,\text{var}2,o,d} = \frac{M_{\text{var}2,d}}{W_{bridge,0}} = \frac{677 \cdot 10^6}{25 \cdot 10^6} = 27{,}1 N/mm^2$$

To determine the second order effects, first the buckling length and the Eulerse buckling force need to be determined.

To do so first the spring stiffness of the stay-cable next to the pylon (cable 5) is required. This can be determined with the formulae of Engesser:

$$k_{bridge} = \frac{E_d A_{tui} \cos\alpha}{l_{tui}} = \frac{2{,}1 \cdot 10^5 \cdot 2136 \cdot 0{,}47}{34000} = 6{,}2 \cdot 10^3 kN/m$$

When dividing this over the total length between two stay-cables this becomes:
$c_{bridge,5} = 6{,}2*10^3/16 = 388$ kN/m^2

The buckling length now becomes:

$$l_{buc,bridge,5} = \pi \sqrt[4]{\frac{E_d I_{y,bridge}}{4c_{bridge,5}}} = \pi \sqrt[4]{\frac{2{,}1 \cdot 10^5 * 19 \cdot 10^9}{4 * 388} \cdot 10^{-9}} = 22{,}37 m$$

The buckling force now becomes:

$$F_E = 2\sqrt{c_{bridge,5} E_d I_{y,bridge}} = 2\sqrt{388 \cdot 10^{-3} * 2{,}1 \cdot 10^5 * 19 \cdot 10^9} \cdot 10^{-3} = 78{,}7 \cdot 10^3 kN$$

This leads to the enlargements factors for the second order effect of:

$$n_{\text{var}1} = \frac{F_E}{N_{p,d} + N_{\text{var}1,d}} = \frac{78700}{14373 + 10221} = 3{,}2 \Rightarrow \frac{n_{\text{var}1}}{n_{\text{var}1} - 1} = \frac{3{,}2}{3{,}2 - 1} = 1{,}45$$

$$n_{\text{var}2} = \frac{F_E}{N_{p,d} + N_{\text{var}2,d}} = \frac{78700}{14373 + 88} = 5{,}4 \Rightarrow \frac{n_{\text{var}2}}{n_{\text{var}2} - 1} = \frac{5{,}4}{5{,}4 - 1} = 1{,}23$$

The relative slenderness of the bridge is:

$$i_{bridge} = \sqrt{\frac{I_{bridge}}{A_{bridge}}} = \sqrt{\frac{19 \cdot 10^9}{149 \cdot 10^3}} = 357mm$$

$$\lambda_{bridge} = \frac{l_{buc,bridge,5}}{i_{bridge}} = \frac{22370}{357} = 62{,}7$$

$$\lambda_{rel,bridge} = \frac{\lambda_{bridge}}{\lambda_e} = \frac{62{,}7}{76{,}4} = 0{,}82$$

Because the amount of welding is relatively small compared to the cross-section, buckling curve B can be applied, with $\alpha_k = 0{,}34$ and $\lambda_0 = 0{,}2$.

The imperfection parameter now becomes:

$$e^* = \alpha_k (\lambda_{rel,bridge} - \lambda_0) \frac{W_{bridge,o} f_{y,d}}{A_{bridge} f_{y,d}} = 0{,}34(0{,}82 - 0{,}2) \frac{25 \cdot 10^6 * 355}{149 \cdot 10^3 * 355} = 35mm$$

The bending moments and stresses due to this eccentricity are:

$$M_{ec1,d} = e^* (N_{p,d} + N_{\mathrm{var}1,d}) = 35 \cdot 10^{-3} (14373 + 10221) = 861kNm$$

$$\sigma_{ec1,b,d} = \frac{M_{ec1,d}}{W_b} = \frac{861 \cdot 10^6}{79 \cdot 10^6} = 11N/mm^2$$

$$\sigma_{ec1,o,d} = \frac{M_{ec1,d}}{W_o} = \frac{861 \cdot 10^6}{25 \cdot 10^6} = 34N/mm^2$$

$$M_{ec2,d} = e^* (N_{p,d} + N_{\mathrm{var}2,d}) = 35 \cdot 10^{-3} (14373 + 88) = 506kNm$$

$$\sigma_{ec2,b,d} = \frac{M_{ec2,d}}{W_b} = \frac{506 \cdot 10^6}{79 \cdot 10^6} = 6N/mm^2$$

$$\sigma_{ec2,o,d} = \frac{M_{ec2,d}}{W_o} = \frac{506 \cdot 10^6}{25 \cdot 10^6} = 20N/mm^2$$

Now both the top and bottom flange of the bridge deck can be checked:

For the bottom flange the stress from the two load cases becomes:

Load case 1:

$$\sigma_{1,o,d} = \sigma_{c,p,d} + \sigma_{c,\mathrm{var}1,d} + \frac{n_{\mathrm{var}1}}{n_{\mathrm{var}1} - 1} (\sigma_{m,p,o,d} + \sigma_{m,\mathrm{var}1,o,d} + \sigma_{ec1,o,d})$$

$$= -96{,}4 - 68{,}6 + 1{,}45(-35{,}6 - 24{,}3 - 34) = 301N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{1,o,d}}{f_{y,d}} = \frac{301}{355} = 0{,}85 \le 1$$

Load case 2:

$$\sigma_{2,o,d} = \sigma_{c,p,d} + \sigma_{c,\text{var}2,d} + \frac{n_{\text{var}2}}{n_{\text{var}2} - 1}(\sigma_{m,p,o,d} + \sigma_{m,\text{var}2,o,d} + \sigma_{ec2,o,d})$$

$$= -96{,}4 - 0{,}6 + 1{,}23(-35{,}6 - 27{,}1 - 20) = 199 N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{1,o,d}}{f_{y,d}} = \frac{199}{355} = 0{,}56 \le 1$$

This means that the bottom flange is sufficient.

The same can be done for the top flange:

Load case 1:

$$\sigma_{1,b,d} = \sigma_{c,p,d} + \sigma_{c,\text{var}1,d} + \frac{n_{\text{var}1}}{n_{\text{var}1} - 1}(\sigma_{m,p,b,d} + \sigma_{m,\text{var}1,b,d} + \sigma_{ec1,b,d})$$

$$= -96{,}4 - 68{,}6 + 1{,}45(11{,}3 + 7{,}7 + 11) = 140 N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{1,o,d}}{f_{y,d}} = \frac{140}{355} = 0{,}39 \le 1$$

Load case 2:

$$\sigma_{2,b,d} = \sigma_{c,p,d} + \sigma_{c,\text{var}2,d} + \frac{n_{\text{var}2}}{n_{\text{var}2} - 1}(\sigma_{m,p,b,d} + \sigma_{m,\text{var}2,b,d} + \sigma_{ec2,b,d})$$

$$= -96{,}4 - 0{,}6 + 1{,}23(11{,}3 + 8{,}6 + 6) = 65 N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{1,o,d}}{f_{y,d}} = \frac{65}{355} = 0{,}18 \le 1$$

This means that the top flange is sufficient.

Local check

Besides checking the bridge deck as a whole, as done in the previous part, also local checks of the structure need to be done. This needs to be done to prevent local buckling of the bottom flange of the girder and of the side plates of the girder.

Local buckling bottom flange

The largest compressive stress in the bottom flange occurs in load case 1, $\sigma_{c,fl,o,d}$ = 301 N/mm^2. The width of the flange is 750 mm. Because there is only compression in the flange it holds:
$\Psi_x = 1$
$k_{\sigma,x} = 4$

This leads to the maximum buckling stress, $\sigma_{plooi,d}$, of:

$$\sigma_E = 19\left(\frac{100t_p}{b}\right)^2 = 19\left(\frac{100 \cdot 28}{750}\right)^2 = 265N/mm^2$$

$$\sigma_{i,k,x} = \sigma_E k_{\sigma,x} = 265 \cdot 4 = 1060N/mm^2$$

$$\lambda_{pl,rel} = \sqrt{\frac{f_{y,d}}{\sigma_{i,k,x}}} = \sqrt{\frac{355}{1060}} = 0{,}58$$

For $\lambda_{pl,rel} \leq 0{,}7$ it holds:

$$\sigma_{plooi,rel} = 1{,}0$$

$$C_{\sigma,x} = 1$$

$$\sigma_{plooi,d} = \sigma_{plooi,rel} f_{y,d} \frac{1}{C_{\sigma,x}} = 1{,}0 * 355 * 1 = 355N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{c,fl,o,d}}{\sigma_{plooi,d}} = \frac{301}{355} = 0{,}85 \leq 1$$

This means the bottom flange can withstand local buckling.

Local buckling girder wall

In loading combination 1 the stress in the top flange is, $\sigma_{c,w,b,d}$ = 140 N/mm^2 and is $\sigma_{c,w,o,d}$ = 301 N/mm^2 in the bottom flange (see figure 9.22). The shear force comes from the computer calculations and is, $V_{s,tui,d}$ = 541 kN

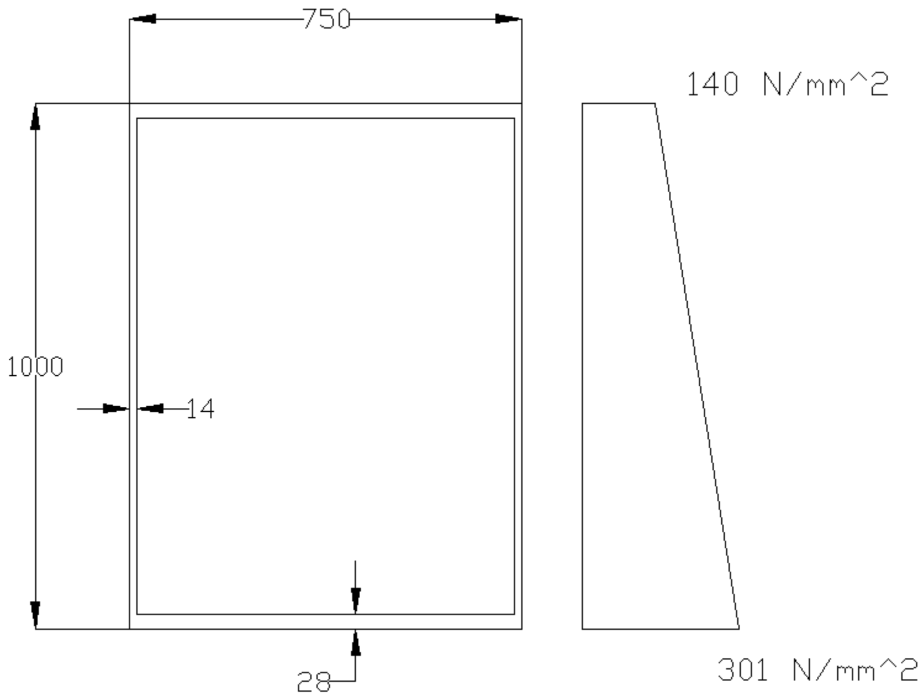


Figure 9.22: Cross-section main-girder with stresses.

This leads to a shear stress of:

$$V_{s,d} = \frac{F_{p,tui,d} + V_{s,tui,d}}{2} = \frac{490 + 541}{2} = 516kN$$

$$\tau_1 = \frac{V_{s,d}}{2A_w} = \frac{516 \cdot 10^3}{2 \cdot 1000 \cdot 14} = 18N / mm^2$$

With vertical plates with a c.o.c. of 5000 mm, the following holds for the plate field:

$$\alpha = \frac{a}{h_w} = \frac{5000}{1000} = 5,0$$

$$\Psi_k = \frac{\sigma_{c,w,b,d}}{\sigma_{c,w,o,d}} = \frac{140}{301} = 0,47$$

$$k_{\sigma,x} = \frac{8,2}{\Psi_k + 1,05} = \frac{8,2}{0,47 + 1,05} = 5,34$$

$$\sigma_E = 19\left(\frac{100t_p}{b}\right)^2 = 19\left(\frac{100 \cdot 14}{1000}\right)^2 = 37N / mm^2$$

$$\sigma_{i,k,x} = \sigma_E k_{\sigma,x} = 37 \cdot 5,34 = 198N / mm^2$$

$$\lambda_{pl,rel} = \sqrt{\frac{f_{y,d}}{\sigma_{i,k,x}}} = \sqrt{\frac{355}{198}} = 1,34$$

For $\lambda_{pl,rel} \geq 1,291$ it holds:

$$\sigma_{plooi,rel} = \frac{1}{\lambda^2_{pl,rel}} = \frac{1}{1,34^2} = 0,557$$

$$C_{\sigma,x} = (0,0468\Psi_x + 0,0468)\lambda_{pl,rel} - 0,0063\Psi_x + 1,1272$$
$$= (0,0468 * 0,47 + 0,0468)1,34 - 0,0063 * 0,47 + 1,1272 = 1,22$$

$$\sigma_{plooi,d} = \sigma_{plooi,rel} f_{y,d} \frac{1}{C_{\sigma,x}} = 0,557 \cdot 355 \cdot \frac{1}{1,22} = 162 N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{c,w,o,d}}{\sigma_{plooi,d}} = \frac{301}{162} \geq 1$$

This means that even without taking the shear force into account the walls of the girder are already prone to plate buckling. Therefore some measures need to be taken. There are two options, increasing the wall thickness or applying trough shaped stiffeners. The applying of stiffeners is a effective way of dealing with plate buckling, therefore this option is chosen.

The characteristics of the chosen troughs are (see figure 9.23):
b_{btr} = 300 mm
b_{otr} = 200 mm
h_{tr} = 200 mm
t_{tr} = 6 mm

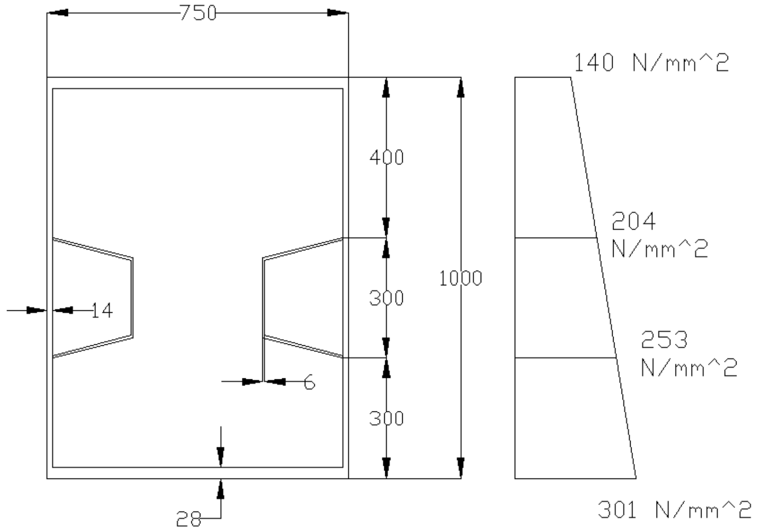


Figure 9.23: Cross-section of the main-girder with stiffeners and stress distribution.

Check of plate field 1

For this plate field it holds:

$$\alpha = \frac{a}{b_1} = \frac{5000}{400} = 12{,}5$$

$$\Psi_k = \frac{\sigma_{w,b,d}}{\sigma_{tr,b,d}} = \frac{140}{204} = 0{,}69$$

$$k_{\sigma,x} = \frac{8{,}2}{\Psi_k + 1{,}05} = \frac{8{,}2}{0{,}69 + 1{,}05} = 4{,}71$$

$$\sigma_E = 19\left(\frac{100t_p}{b_1}\right)^2 = 19\left(\frac{100 \cdot 14}{400}\right)^2 = 233N/m^2$$

$$\sigma_{i,k,x} = \sigma_E k_{\sigma,x} = 233 \cdot 4{,}71 = 1097N/mm^2$$

It holds α > 1, therefore:

$$k_\tau = 5{,}35 + \frac{4{,}0}{\alpha^2} = 5{,}35 + \frac{4}{12{,}5^2} = 5{,}38$$

$$\tau_{i,k} = \sigma_E k_\tau = 233 \cdot 5{,}38 = 1254N/mm^2$$

According to NEN 6771 art.13.6.2 it now holds:

$$\sigma_{i,k,v} = \frac{\sqrt{\sigma_{tr,b,d}^2 + 3\tau_d^2}}{\frac{1+\Psi_x}{4} \cdot \frac{\sigma_{tr,b,d}}{\sigma_{i,k,x}} + \sqrt{\left(\frac{3-\Psi_k}{4} \cdot \frac{\sigma_{tr,b,d}}{\sigma_{i,k,x}}\right)^2 + \left(\frac{\tau_d}{\tau_{i,k}}\right)^2}}$$

$$= \frac{\sqrt{204^2 + 3 \cdot 18^2}}{\frac{1+0{,}69}{4} \cdot \frac{204}{1097} + \sqrt{\left(\frac{3-0{,}69}{4} \cdot \frac{204}{1097}\right)^2 + \left(\frac{18}{1254}\right)^2}} = 1104N/mm^2$$

$$\lambda_{pl,rel} = \sqrt{\frac{f_{y,d}}{\sigma_{i,k,v}}} = \sqrt{\frac{355}{1104}} = 0{,}57$$

Because $\lambda_{pl,rel} < 0{,}7$ it holds:

$$\sigma_{plooi,rel} = 1{,}0$$

$$C_{\sigma,x} = 1$$

$$\sigma_{plooi,d} = \sigma_{plooi,rel} f_{y,d} \frac{1}{C_{\sigma,x}} = 1{,}0 * 355 * 1 = 355N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{tr,b,d}}{\sigma_{plooi,d}} = \frac{204}{355} = 0{,}57 \leq 1$$

This means the first plate field can withstand local buckling.

Check of plate field 3

For this plate field it holds:

$$\alpha = \frac{a}{b_1} = \frac{5000}{300} = 16{,}7$$

$$\Psi_k = \frac{\sigma_{tr,o,d}}{\sigma_{tw,o,d}} = \frac{253}{301} = 0{,}84$$

$$k_{\sigma,x} = \frac{8{,}2}{\Psi_k + 1{,}05} = \frac{8{,}2}{0{,}84 + 1{,}05} = 4{,}34$$

$$\sigma_E = 19\left(\frac{100 t_p}{b_3}\right)^2 = 19\left(\frac{100 \cdot 14}{300}\right)^2 = 414 N/m^2$$

$$\sigma_{i,k,x} = \sigma_E k_{\sigma,x} = 414 \cdot 4{,}34 = 1797 N/mm^2$$

It holds α > 1, therefore:

$$k_\tau = 5{,}35 + \frac{4{,}0}{\alpha^2} = 5{,}35 + \frac{4}{16{,}7^2} = 5{,}36$$

$$\tau_{i,k} = \sigma_E k_\tau = 414 \cdot 5{,}36 = 2219 N/mm^2$$

According to NEN 6771 art.13.6.2 it now holds:

$$\sigma_{i,k,v} = \frac{\sqrt{\sigma_{w,o,d}^2 + 3\tau_d^2}}{\frac{1+\Psi_x}{4} \cdot \frac{\sigma_{w,o,d}}{\sigma_{i,k,x}} + \sqrt{\left(\frac{3-\Psi_k}{4} \cdot \frac{\sigma_{w,o,d}}{\sigma_{i,k,x}}\right)^2 + \left(\frac{\tau_d}{\tau_{i,k}}\right)^2}}$$

$$= \frac{\sqrt{301^2 + 3 \cdot 18^2}}{\frac{1+0{,}84}{4} \cdot \frac{301}{1797} + \sqrt{\left(\frac{3-0{,}84}{4} \cdot \frac{301}{1797}\right)^2 + \left(\frac{18}{2219}\right)^2}} = 1801 N/mm^2$$

$$\lambda_{pl,rel} = \sqrt{\frac{f_{y,d}}{\sigma_{i,k,v}}} = \sqrt{\frac{355}{1801}} = 0{,}20$$

Because $\lambda_{pl,rel} < 0{,}7$ it holds:

$$\sigma_{plooi,rel} = 1{,}0$$

$$C_{\sigma,x} = 1$$

$$\sigma_{plooi,d} = \sigma_{plooi,rel} f_{y,d} \frac{1}{C_{\sigma,x}} = 1{,}0 * 355 * 1 = 355 N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{w,o,d}}{\sigma_{plooi,d}} = \frac{301}{355} = 0{,}85 \leq 1$$

This means the third plate field can withstand local buckling.

This means that with the applied stiffener the girder walls can withstand local buckling.

Buckling stiffener in longitudinal direction

Finally the stiffener needs to be checked in longitudinal direction for local buckling.
Again loading case 1 is the governing load case, for the shear stress, the stress found in the previous part is applied, $\tau_1 = 18 N/mm^2$.

The width of a trough wall is:

$$b_{trw} = \sqrt{200^2 + 50^2} = 206 mm$$

This leads to:

$$\sigma_{i,k,x} = k_{\sigma,x}\sigma_E = 4 \cdot 19\left(\frac{100 \cdot 6}{206}\right)^2 = 645 N/mm^2$$

The effective width of the plate and trough are:

$$b_{e,w} = \frac{1{,}33}{2} \cdot 14\sqrt{\frac{E_d}{f_{y,d}}} = 226 mm$$

$$b_{e,tr} = \frac{1{,}33}{2} \cdot 6\sqrt{\frac{E_d}{f_{y,d}}} = 97 mm$$

Other characteristics are:
A_{st} = 13582 mm^2
z = 56 mm
I_y = 60,1*10^6 mm^4
W_b = 1,1*10^6 mm^3
W_o = 0,4*10^6 mm^3

According to NEN 6771 art. 13.5.2 the whole loading should be applied to the stiffener. This leads to:

$$\sigma_{h,w,b,d} = \frac{\sigma_{w,b,d}}{2} + \sqrt{\left(\frac{\sigma_{w,b,d}}{2}\right)^2 + \tau_1^2} = \frac{140}{2} + \sqrt{\left(\frac{140}{2}\right)^2 + 18^2} = 142 N/mm^2$$

$$\sigma_{h,w,o,d} = \frac{\sigma_{w,o,d}}{2} + \sqrt{\left(\frac{\sigma_{w,o,d}}{2}\right)^2 + \tau_1^2} = \frac{301}{2} + \sqrt{\left(\frac{301}{2}\right)^2 + 18^2} = 303 N/mm^2$$

This leads to the following normal forces:

Plate field 1: $N_1 = 14 \cdot \frac{400}{2} \cdot \frac{\sigma_{w,b,d} + \sigma_{tr,b,d}}{2} = 14 \cdot \frac{400}{2} \cdot \frac{142 + 204}{2} = 484kN$

Plate field 2: $N_2 = 14 \cdot 300 \cdot \frac{\sigma_{tr,b,d} + \sigma_{tr,o,d}}{2} = 14 \cdot 300 \cdot \frac{204 + 253}{2} = 690kN$

Plate field 3: $N_3 = 14 \cdot \frac{300}{2} \cdot \frac{\sigma_{w,0,d} + \sigma_{w,o,d}}{2} = 14 \cdot \frac{300}{2} \cdot \frac{253 + 301}{2} = 582kN$

The total force on the stiffener now becomes:
$N_{st} = N_1 + N_2 + N_3 = 484 + 690 + 582 = 1756$ kN

The enlargement factor becomes:

$$F_{E,buc} = \frac{\pi^2 E_d I_y}{l_{buc}^2} = \frac{\pi^2 \cdot 2{,}1 \cdot 10^5 \cdot 60{,}1 \cdot 10^6}{5000^2} = 4983kN$$

$$n = \frac{F_{E,d}}{N_{st}} = \frac{4983}{1756} = 2{,}84$$

$$\frac{n}{n-1} = \frac{2{,}84}{2{,}84 - 1} = 1{,}54$$

An eccentricity of, e = l_{buc} / 400 = 5000 / 400 = 12,5 mm is assumed.

This lead to a stress in the stiffener of:

$$\sigma_{s,d} = \frac{N_{st}}{A_{st}} + e \cdot \frac{N_{st}}{W_o} \frac{n}{n-1} = \frac{1756 \cdot 10^3}{13582} + 12{,}5 \cdot \frac{1756 \cdot 10^3}{0{,}4 \cdot 10^6} \cdot 1{,}54 = 129 + 85 = 214 N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_{s,d}}{f_{y,d}} = \frac{214}{355} = 0{,}60 \leq 1$$

This means the dimensions of the stiffener are sufficient.

9.7 Detailing

Now the main structural components are dimensioned, they need to be connected. There are two types of connections which need to be designed. The first is the connection between the stay-cables and the main girder. The second is the connection between the pylon and the stay-cables.

First the connection between the main girder and the stay-cables will be calculated.

To connect the girder to the stay-cable, a plate will be welded to the girder. The end of the cable will be placed in a socket and this socket will be connected to the plate by welding. (see figure 9.24)

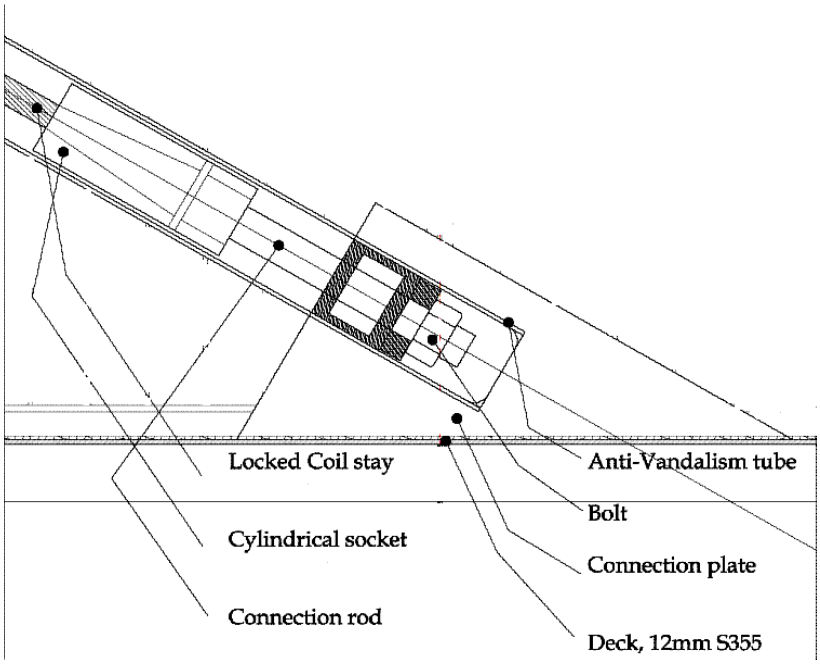


Figure 9.24: Lay-out of stay-cable – girder connection.

The size of the cylindrical socket is given by the producer of the stay-cables. The required dimensions for the socket are given in appendix 2.

The length of the connection rod is designed at the same length as the length of the socket.

For the plate connecting the cables and deck one size will be used, making it easier during erection of the bridge.
The largest normal force of a stay-cable is $N_{13,d}$ = 4485 kN. For the plate S355 is used. With these characteristics known, the minimum required dimensions of the plate are known.

According to the unity-check it holds:

$$\frac{\sigma_{pl,d}}{f_{y,d}} \leq 1$$

With:
$f_{y,d}$ = 355 N/mm^2

$$\sigma_{pl,d} = \frac{N_{13,d}}{A_{pl}}$$

This leads to:

$$\frac{N_{13,d}}{f_{y,d}} = A_{pl}$$

$$\frac{4485 \cdot 10^3}{355} = 12634 mm^2$$

The largest socket diameter used is 280 mm. When assuming a plate height of 1 1/2 this diameter, a minimum plate width of 30 mm is found. This width will be used for all plates used to connect the stay-cables and the main-girder.

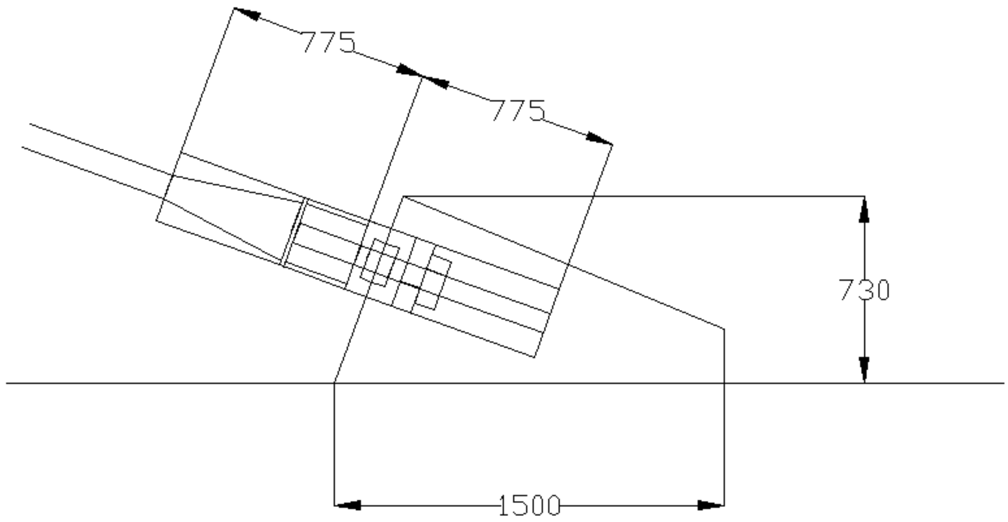


Figure 9.25: Dimensions connection plate.

For the connection two welds need to be calculated. First the weld between the socket and the plate and second the weld between the plate and the main girder.

For the connection between the socket and the plate, only the sides will be welded. According to NEN 6772 now holds for the shear stress in the cross-section:

$$\tau_2 = \frac{F_{s,d}}{4al_{ef}} \Rightarrow a = \frac{F_{s,d}}{4\tau_2 l_{ef}}$$

With:
$\tau_2 = f_{w,u,d} = 262$ N/mm^2
$F_{s,d} = N_{tui,d}$
a = width of the weld
l_{ef} = effective length of the weld

Because there are three types of sockets, there will be three calculations done, in which the governing load, is the largest load for that category.

Cable diameter of 56:
This means the socket will have a length of 460 mm. Of this length 400 mm will be used to weld the socket to the plate on both the top and bottom side and on both sides of the plate.
The maximum load is 1654 kN
This means the equation for the weld width now becomes:

$$a = \frac{1654 \cdot 10^3}{4 \cdot 262 \cdot 400} = 4mm$$

However this is only valid for l_{ef} <150a.
In this case this means l_{ef} < 600 mm. Which is true and therefore a weld width of 4 mm is sufficient.

Cable diameter of 80:
This means the socket will have a length of 645 mm. Of this length 600 mm will be used to weld the socket to the plate on both the top and bottom side and on both sides of the plate.
The maximum load is 3388 kN
This means the equation for the weld width now becomes:

$$a = \frac{3388 \cdot 10^3}{4 \cdot 262 \cdot 600} = 6mm$$

However this is only valid for l_{ef} <150a.
In this case this means l_{ef} < 900 mm. Which is true and therefore a weld width of 6 mm is sufficient.

Cable diameter of 96:
This means the socket will have a length of 775 mm. Of this length 700 mm will be used to weld the socket to the plate on both the top and bottom side and on both sides of the plate.
The maximum load is 4485 kN
This means the equation for the weld width now becomes:

$$a = \frac{4485 \cdot 10^3}{4 \cdot 262 \cdot 700} = 7mm$$

However this is only valid for l_{ef} <150a.
In this case this means l_{ef} < 1050 mm. Which is true and therefore a weld width of 7 mm is sufficient.

For the connection between the plate and the main girder both sides of the plate will be welded to the top flange of the girder. See figure 9.26. In this case only the plate with the largest loading will be calculated. All the other plates will get the same weld, making it easier during erection.

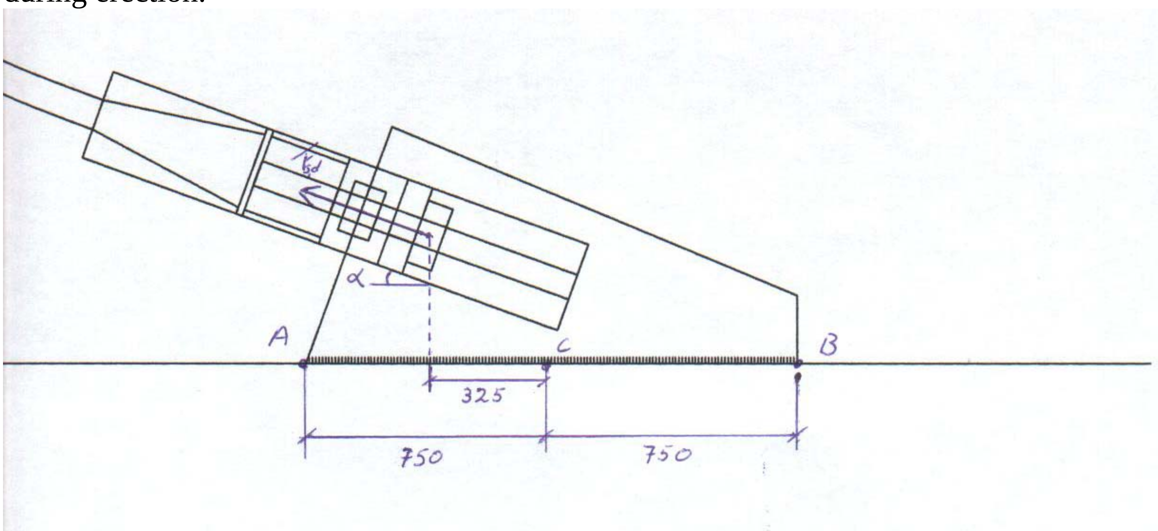


Figure 9.26: Weld calculation.

The force acting on the connection, $F_{s,d}$ can be split up into three forces:
$F_{s,d,ver} = F_{s,d} * \sin 12° = 4485*10^3 * \sin 12° = 934$ kN
$F_{s,d,hor} = F_{s,d} * \cos 12° = 4485*10^3 * \cos12° = 4390$ kN
$M_{s,d} = F_{s,d,ver} *e = 934 * 325*10^{-3} = 304$ kNm

For checking the weld, first a weld thickness of a = 8 mm is assumed, this means:
l_{ef} = l50 * 8 =1200 mm.

Now the stresses these forces cause can be calculated by dividing the load cases into three basic categories. For these categories see appendix 4.

Case 1:

$$\sigma_1^{(1)} = \tau_1^{(1)} = \frac{F_{s,d,ver}\sqrt{2}}{4al_{ef}} = \frac{934\cdot 10^3 \cdot \sqrt{2}}{4\cdot 8\cdot 1200} = 28N/mm^2$$

Case 3:

$$\tau_2^{(3)} = \frac{F_{s,d,hor}}{2al_{ef}} = \frac{4390\cdot 10^3}{2\cdot 8\cdot 1200} = 183N/mm^2$$

Case 5:

$$\sigma_1^{(5)} = \tau_1^{(5)} = \frac{2{,}12M_{s,d}}{al_{ef}^2} = \frac{2{,}12\cdot 304\cdot 10^6}{8\cdot 1200^2} = 45N/mm^2$$

The maximum stress in point A now becomes:

$$\sigma_1 = \sigma_1^{(1)} + \sigma_1^{(5)} = 28 + 45 = 73N/mm^2$$

$$\tau_1 = \tau_1^{(1)} + \tau_1^{(5)} = 28 + 45 = 73N/mm^2$$

$$\tau_2 = \tau_2^{(3)} = 183N/mm^2$$

According to NEN 6772 the unity-check now becomes:

$$\frac{\sigma_1}{f_{t,d}/\gamma_m} = \frac{73}{510/1{,}25} = 0{,}18 \leq 1$$

$$\sigma_{w,s,d} = \frac{1}{\sqrt{3}}\sqrt{\sigma_1^2 + 3(\tau_1^2 + \tau_2^2)} = \frac{1}{\sqrt{3}}\sqrt{73^2 + 3(73^2 + 183^2)} = 201N/mm^2$$

$$f_{w,u,d} = 0{,}46\frac{f_{t,d}}{\beta} = 0{,}46\frac{510}{0{,}9} = 261N/mm^2$$

$$\frac{\sigma_{w,s,d}}{f_{w,u,d}} = \frac{201}{261} = 0{,}77 \leq 1$$

Both unity-checks indicate that a weld of 8 mm is sufficient, therefore is weld will be applied in the deck-plate connection.

Now the connection between the cable and the pylon will be calculated. The same type of connection will be used as for the connection between the cables and the main-girder will be used. The cables on the main span side of the pylon will be placed in two groups of 5 on the side span the three cables connected to the bridge will be placed on one plate. The cable leading to the building will get a separate connection to the pylon. First the plate connecting the cables on the main span side will be calculated, after this the connection for the back stay-cable will be calculated.

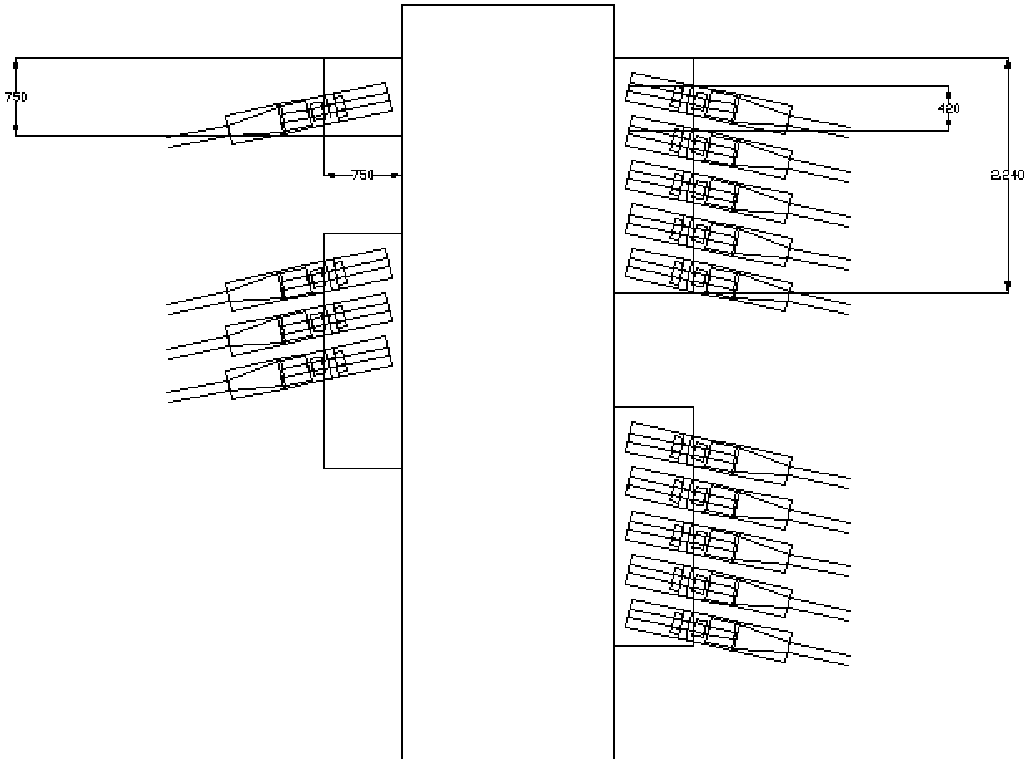


Figure 9.27: Configuration of the connection at the top.

For the largest force it hold that l = 420 mm and a = 22 mm. The eccentricity, e = 150 mm.

The force acting on the connection, $F_{s,d}$ can be split up into three forces:
$F_{s,d,ver} = F_{s,d} * \cos 12° = 4485*10^3 * \cos 12° = 3490$ kN
$F_{s,d,hor} = F_{s,d} * \sin 12° = 4485*10^3 * \sin 12° = 934$ kN
$M_{s,d} = F_{s,d,hor} *e = 934 * 150*10^{-3} = 140$ kNm

Now the stresses these forces cause can be calculated by dividing the load cases into three basic categories.

Case 1:

$$\sigma_1^{(1)} = \tau_1^{(1)} = \frac{F_{s,d,ver}\sqrt{2}}{4al_{ef}} = \frac{3490 \cdot 10^3 \cdot \sqrt{2}}{4 \cdot 22 \cdot 420} = 134 N/mm^2$$

Case 3:

$$\tau_2^{(3)} = \frac{F_{s,d,hor}}{2al_{ef}} = \frac{934 \cdot 10^3}{2 \cdot 22 \cdot 420} = 51 N/mm^2$$

Case 5:

$$\sigma_1^{(5)} = \tau_1^{(5)} = \frac{2{,}12 M_{s,d}}{al_{ef}^2} = \frac{2{,}12 \cdot 140 \cdot 10^6}{22 \cdot 420^2} = 76 N/mm^2$$

The maximum stress in point A now becomes:

$$\sigma_1 = \sigma_1^{(1)} + \sigma_1^{(5)} = 134 + 76 = 210 N/mm^2$$
$$\tau_1 = \tau_1^{(1)} + \tau_1^{(5)} = 134 + 76 = 210 N/mm^2$$
$$\tau_2 = \tau_2^{(3)} = 51 N/mm^2$$

According to NEN 6772 the unity-check now becomes:

$$\frac{\sigma_1}{f_{t,d}/\gamma_m} = \frac{210}{510/1{,}25} = 0{,}51 \le 1$$

$$\sigma_{w,s,d} = \frac{1}{\sqrt{3}}\sqrt{\sigma_1^2 + 3(\tau_1^2 + \tau_2^2)} = \frac{1}{\sqrt{3}}\sqrt{210^2 + 3(210^2 + 51^2)} = 248 N/mm^2$$

$$f_{w,u,d} = 0{,}46\frac{f_{t,d}}{\beta} = 0{,}46\frac{510}{0{,}9} = 261 N/mm^2$$

$$\frac{\sigma_{w,s,d}}{f_{w,u,d}} = \frac{248}{261} = 0{,}95 \le 1$$

Both unity-checks indicate that a weld of 22 mm is sufficient, therefore this weld will be applied in the deck-plate connection.

Now the connection between the back-stay-cable and the pylon needs to be calculated.

The force in the back stay-cable is $F_{1,d}$ = 22961 kN. The length of the weld is 1500 mm and the a = 26 mm. The eccentricity is 150 mm

The force acting on the connection, $F_{s,d}$ can be split up into three forces:
$F_{s,d,ver} = F_{s,d}$ * cos 5,27° = 22961*10^3 * cos 5,27° = 22864 kN
$F_{s,d,hor} = F_{s,d}$ * sin 5,27° = 22961*10^3 * sin 5,27° = 2111 kN
$M_{s,d} = F_{s,d,hor}$ *e = 2111 * 150*10^{-3} = 317 kNm

Now the stresses these forces cause can be calculated by dividing the load cases into three basic categories.

Case 1:

$$\sigma_1^{(1)} = \tau_1^{(1)} = \frac{F_{s,d,ver}\sqrt{2}}{4al_{ef}} = \frac{22864\cdot 10^3 \cdot \sqrt{2}}{4\cdot 26\cdot 1500} = 207 N/mm^2$$

Case 3:

$$\tau_2^{(3)} = \frac{F_{s,d,hor}}{2al_{ef}} = \frac{2111\cdot 10^3}{2\cdot 26\cdot 1500} = 27 N/mm^2$$

Case 5:

$$\sigma_1^{(5)} = \tau_1^{(5)} = \frac{2{,}12 M_{s,d}}{al_{ef}^2} = \frac{2{,}12\cdot 317\cdot 10^6}{26\cdot 1500^2} = 11 N/mm^2$$

The maximum stress in point A now becomes:

$$\sigma_1 = \sigma_1^{(1)} + \sigma_1^{(5)} = 207 + 11 = 218 N/mm^2$$
$$\tau_1 = \tau_1^{(1)} + \tau_1^{(5)} = 207 + 11 = 218 N/mm^2$$
$$\tau_2 = \tau_2^{(3)} = 27 N/mm^2$$

According to NEN 6772 the unity-check now becomes:

$$\frac{\sigma_1}{f_{t,d}/\gamma_m} = \frac{218}{510/1,25} = 0,53 \le 1$$

$$\sigma_{w,s,d} = \frac{1}{\sqrt{3}}\sqrt{\sigma_1^2 + 3(\tau_1^2 + \tau_2^2)} = \frac{1}{\sqrt{3}}\sqrt{218^2 + 3(218^2 + 27^2)} = 253 N/mm^2$$

$$f_{w,u,d} = 0,46\frac{f_{t,d}}{\beta} = 0,46\frac{510}{0,9} = 261 N/mm^2$$

$$\frac{\sigma_{w,s,d}}{f_{w,u,d}} = \frac{253}{261} = 0,97 \le 1$$

Both unity-checks indicate that a weld of 26 mm is sufficient, therefore this weld will be applied in the deck-plate connection.

The plate at the bottom of the pylon also needs to be checked. A steel plate will be welded to the bottom of the pylon, this plate will be anchored to the concrete under it, forming the foundation of the pylon.
This check is done according to NEN 6772. It holds:

$$s_b \ge t_{col} + 2l_s$$

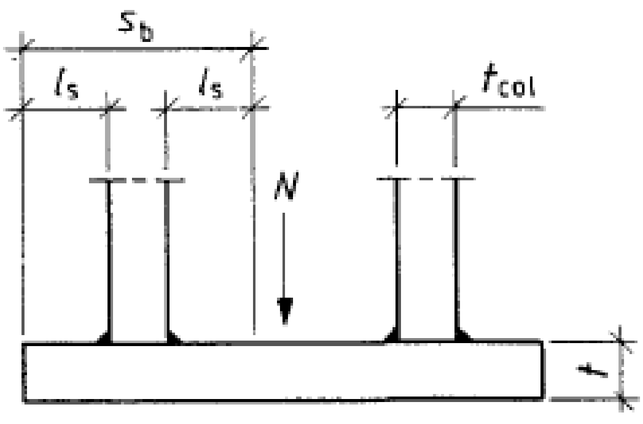


Figure 9.28: Definitions for designing bottom plate.

$$l_s = t\sqrt{\frac{f_{y,d}}{f_{j,u,d}}}$$

With:

$$f_{j,u,d} = 0{,}67 k_b f`_{b,d}$$

$$k_b = \sqrt{\frac{a_1 b_1}{ab}}$$

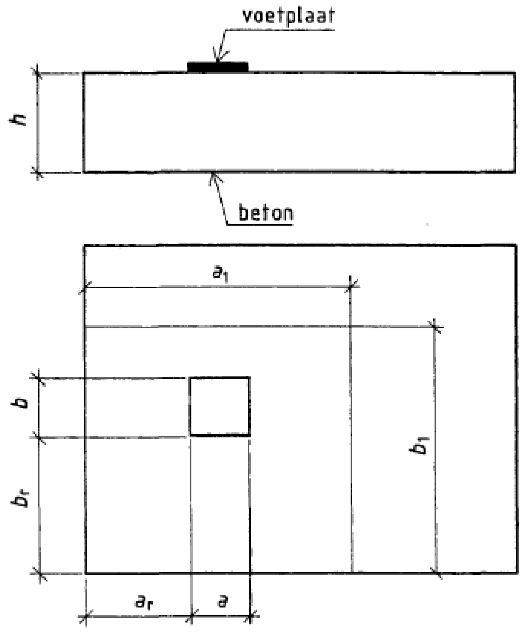


Figure 9.29: Definition dimensions footplate.

With:

a_1 is smallest value of:

- $a + 2a_r$
- 5a
- a + h
- $5b_1$

But always larger than a.

b_1 is smallest value of:

- $b + 2b_r$
- 5b
- b + h
- $5a_1$

But always larger than b.

The following characteristics are used as a starting point:
$f_{y,d}$ = 355 N/mm^2
$f`_{b,d}$ = 21 N/mm^2
a = b = 5500 mm
h = 1000 mm

This means $a_1 = b_1 = 6500$ mm

$$k_b = \sqrt{\frac{a_1 b_1}{ab}} = \sqrt{\frac{6500*6500}{5500*5500}} = 1{,}2$$
$$f_{j,u,d} = 0{,}67 k_b f`_{b,d} = 0{,}67*1{,}2*21 = 16{,}9 N/mm^2$$
$$l_s = t\sqrt{\frac{f_{y,d}}{f_{j,u,d}}} = 100\sqrt{\frac{355}{16{,}9}} = 458mm$$
$$s_b \geq t_{col} + 2l_s \Rightarrow s_b \geq 15 + 2*458 = 931mm$$

With a footplate of 5500 * 5500 mm^2 this requirement is met.
The dimensions of the total pylon foot can be seen in figure 9.30.

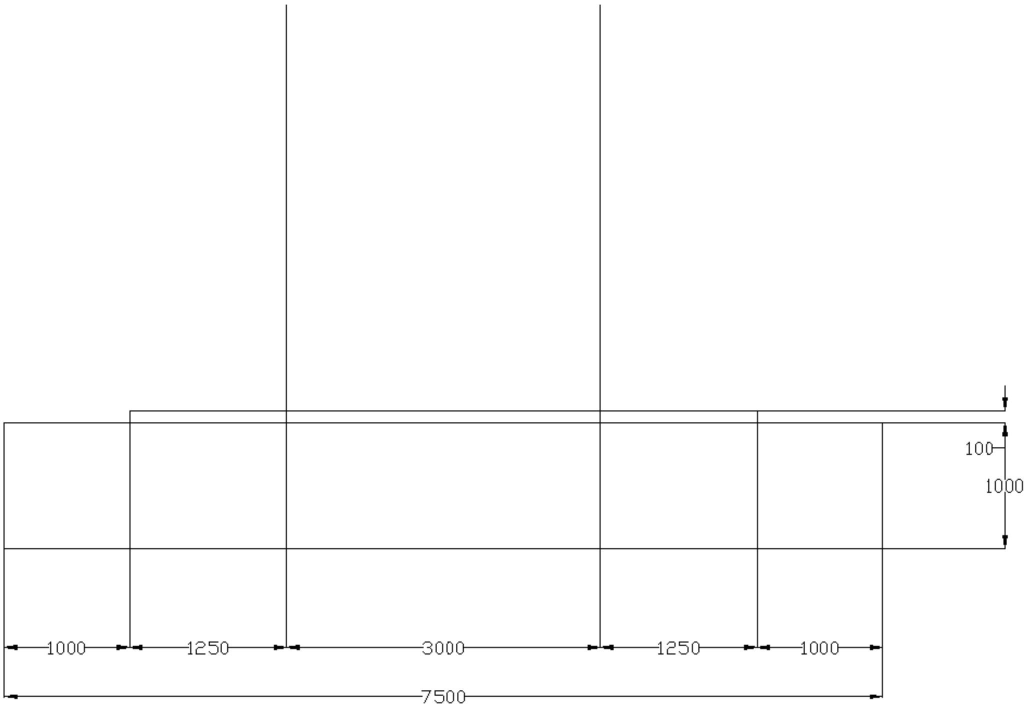


Figure 9.30: Pylon foot

Now the dimensions of the bottom plate are known, the anchor bolds required for keeping the plate in place can be calculated. The bolds used are M48 bolds 8.8. There are two forces working on the connection, a shear force due to the horizontal force and a compressive force due to the bending moment and compressive force in the pylon.

The horizontal force is 186 kN. The shear strength of a bold is:

$$F_{v,u,d} = \frac{\beta\alpha_{red,2} f_{t,b,d} A_{bs}}{\lambda_m} = \frac{0{,}25 \cdot 1 \cdot 800 \cdot \frac{1}{4}\pi \cdot 48^2}{1{,}25} = 290kN$$

Therefore with already one bold, sufficient strength for the shear stress is found. In this case the unity-check becomes:

$$\frac{F_{v,s,d}}{F_{v,u,d}} = \frac{186}{290} \leq 1$$

The normal force in the pylon is, $F_{c,d}$ = 17393 kN and the bending moment is, $M_{y,s,d}$ = 7829 kNm. With the bolds spaced at 4500 mm c.o.c. the force due to the bending moment becomes:

$$F_{m,d} = \frac{M_{y,s,d}}{x} = \frac{7829 \cdot 10^3}{4,5} = 1740kN$$

This means that on both sides of the pylon a compressive force is acting on the foundation. The larger of these two is:

$F_{v,s,d}$ = 1740 + 17393 = 19133 kN

The strength of a bold is given by:

$$F_{c,u,d} = 2a_c \alpha_{red,1} f_{t,d} d_{b,nom} t = 2 \cdot 1 \cdot 1 \cdot 355 \cdot 48 \cdot 100 = 3408kN$$

When applying a row of 6 bolds, the total compressive strength of these bolds becomes:

6 * 3408 = 20448 kN.

The unity-check now becomes:

$$\frac{F_{v,s,d}}{F_{c,u,d}} = \frac{19133}{20448} = 0,93 \leq 1$$

Therefore a row of 6 bolds is applied.

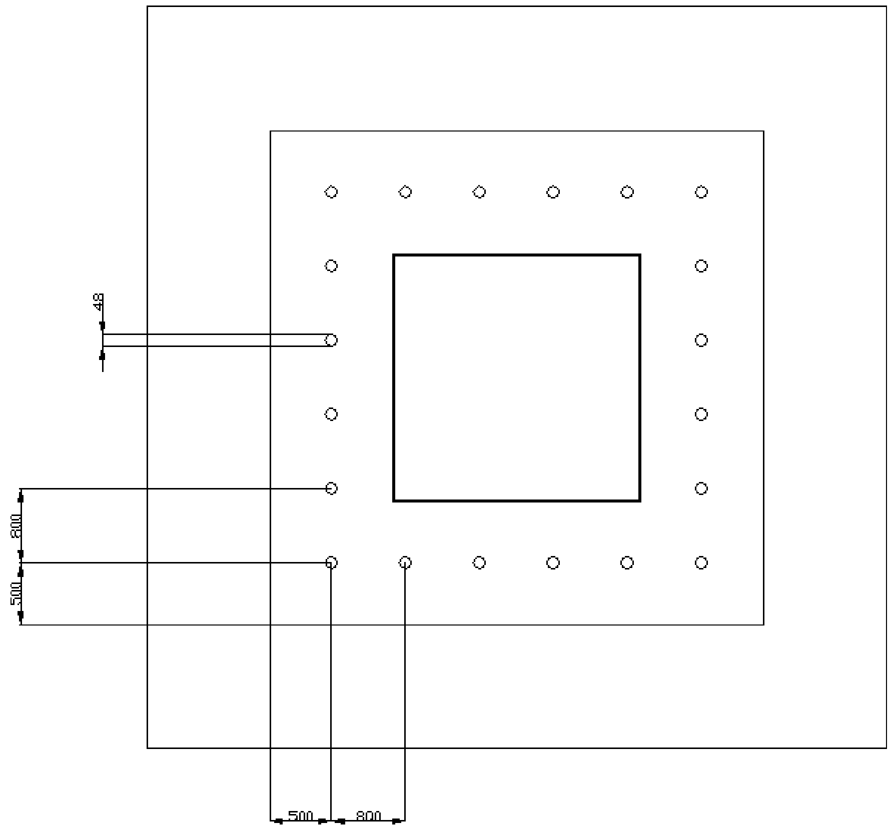


Figure 9.31: Bolt configuration

Cross-beam

As stated before, the two pylons will be connected by a cross-beam. By doing so, the buckling length of the pylon out of the plane of the stay-cables is reduced considerably. It was stated the cross-beam would have a stiffness, $I_{beam} = 2*I_p$.
The stiffness of the pylon is, $I_p = 32*10^9$ mm^4, therefore the stiffness of the cross-beam must be, $I_{beam} = 2 * 32*10^9 = 64*10^9$ mm^4.

When applying a square box-section as stiffener., it holds for the stiffness:

$$I = \frac{H^4 - h^4}{12}$$

The pylon has a width of 2500 mm, when applying the same width for the stiffener the minimum required wall thickness can be found.

$$h = \sqrt[4]{H^4 - 12I} = \sqrt[4]{2500^4 - 12 \cdot 64 \cdot 10^9} = 2488mm$$

This means the minimum wall thickness is 2500 – 2488 = 12 mm.

Therefore this wall thickness will be used for the cross-beam.

Now all dimensions have been calculated and checked, drawings of the bridge can be made. For these drawings is referred to appendix 5.

9.8 Wind loading

In the previous part the bridge was dimensioned on the basis of its permanent loading (self-weight) and its variable loading (traffic loading). These loadings were both applied as static loadings. Besides these static loadings there is another type of loading which is important for cable-stay bridges. This is dynamic loading. This dynamic loading is caused by the wind. In this chapter a short summary is given of the effects of this loading. From these effects only torsional divergence will be the only one which is calculated. The effects on the structure of the other dynamic effects has been taken into account when making the design choices and can be minimised by applying dampers to the stay-cables, however the exact calculations of these effects is outside the scope of this study.

Dynamic wind loading

When wind passes by an object, vibrations can be excited in this object due to the releasing of vortexes or due to an interaction between movement of the object and the wind. The most important forms of these vibrations are vortex-excitation, galloping and flutter.

Vortex-excitation

When a round object (like stay-cables) or a square object (like a pylon) is in a streaming medium (in this case the wind) periodically vortexes will let go. The frequency at which this happens depends on the velocity of the medium and the shape of the cross-section. This frequency is described with the Strouhal number:

$$St = \frac{f_w d}{v}$$

with:
f_w = frequency [Hz]
d = diameter or width [m]
v = velocity [m/s]

For round objects the Strouhal number is about 0,2 and for square object about 0,1. The release of the vortexes causes a horizontal force on the object. In case the frequency, f_w, is the same as the Eigen frequency of the object an enlargement of the force will occur. The wind velocity at which this happens is called the critical wind velocity.

This effect will especially occur around round cylindrical objects. This is called the lock-in-effect. This effect depends on the Scruton number, which is defined as:

$$Sc = \frac{4\pi m_e D}{\rho d^2}$$

with:
m_e = mass per meter length
D = amount of damping
d = diameter

When combining this number with the Reynolds number, the maximum excitation at the top of the construction can be found.

Galloping

The effect of galloping can only occur at rectangular objects. When this happens the object excites a self enlarging bending vibration perpendicular to the wind direction. This loading occurs when an object moves perpendicular to the flow of the medium. In this case there is also a critical wind velocity, defined as:

$$v_{cr} = Sc\frac{2f_eB}{C_{gal}}$$

with:
B = width of the object
C_{gal} = galloping stability

C_{gal} depends on the shape of the object.
By making sure the wind velocity of the design is smaller than the critical velocity galloping can be prevented.

Flutter

In case of flutter a combined bending and rotation vibration occurs. This can only happen if the Eigen frequency for bending and torsion are almost the same.

Torsional divergence

This is the most critical and governing form of aerodynamic instability for cable-stay bridges. Torsional divergence occurs when the torsional amplitude of oscillation in the wind stream increases rapidly in amplitude with small increase in wind speed at the critical wind velocity, v_{cr}.
The critical velocity is a function of the lowest torsional frequency of the bridge deck, f_t, and the overall bridge width.
For the chosen cross-section in this case it holds:

$$\frac{v_{cr}}{f_t b} = 6{,}2$$

The critical velocity is 40 m/s and the bridge width is 10 m, this leads to a torsional frequency of:

$$\frac{30}{10f_t} = 6{,}2 \Rightarrow f_t = 0{,}48$$

Due to the chosen cable configuration and pylon this is a torsionally weak system.
It can be extrapolated from figure 9.32 that the maximum span allowed with this width is 200 m. The span is 160 m, therefore no torsional divergence is to be expected.

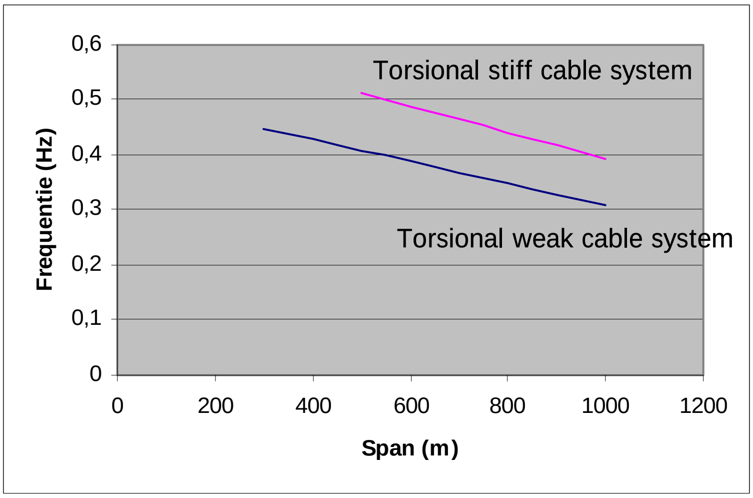


Figure 9.32: Graph for torsional divergence

10. Calculations for building design

As stated in chapter 8, the building on the Zwijndrecht side of the river will be a movie theatre. In the preliminary design the following lay-out was chosen, see figure 10.1 and 10.2.

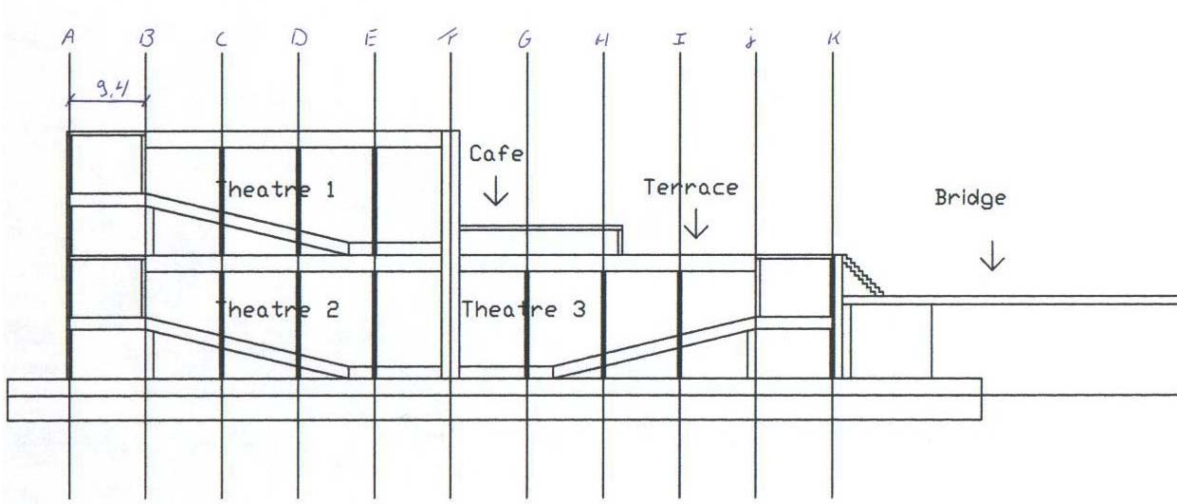


Figure 10.1: Side view of preliminary design main construction.

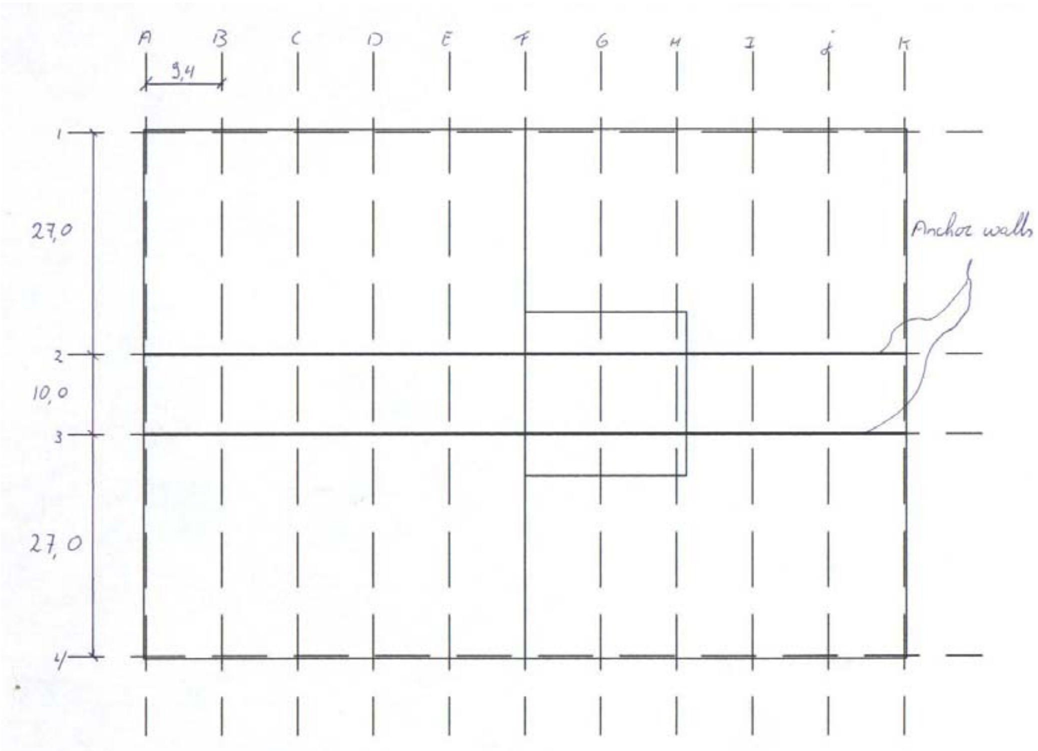


Figure 10.2: Top view of preliminary design.

First some changes are made to the preliminary design. The café will not be placed on top of theatre 3 and 6, but between theatre 1 and 4, between the anchor walls. This change is made, because there is sufficient space between the two theatres and because this way no extra construction on top of theatres 3 and 6 is required.

The new side view is given in figure 10.3.

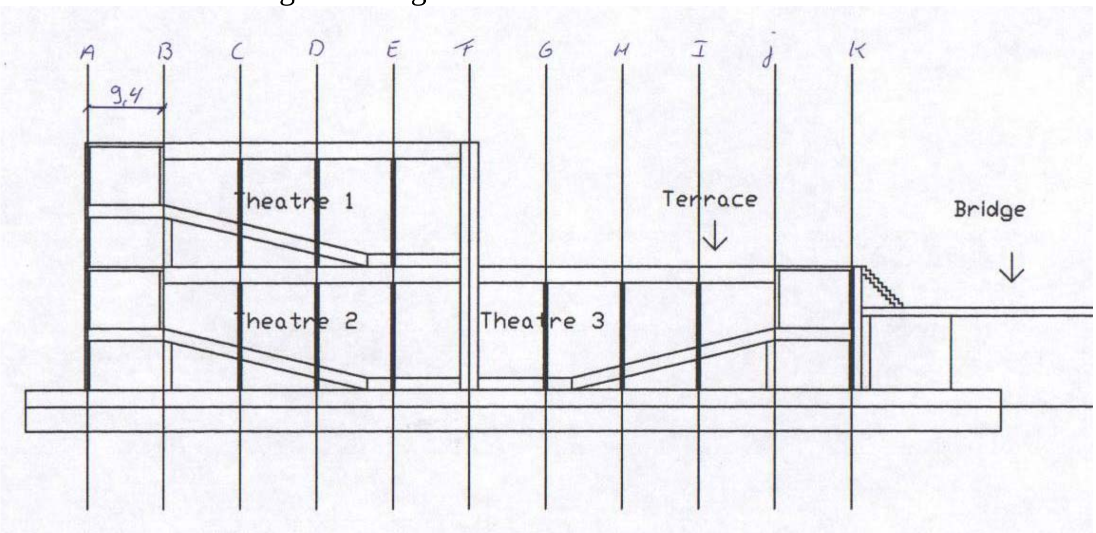


Figure 10.3: New side view preliminary design.

Now also the functional floor plans can be given (see figure 10.4 and 10.5).

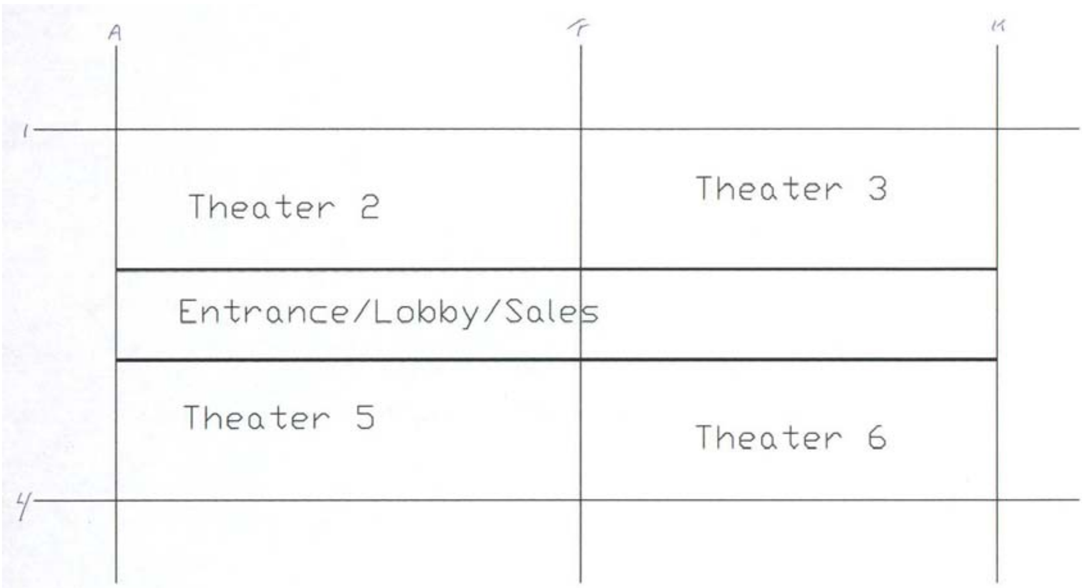


Figure 10.4: Functional lay-out bottom floor.

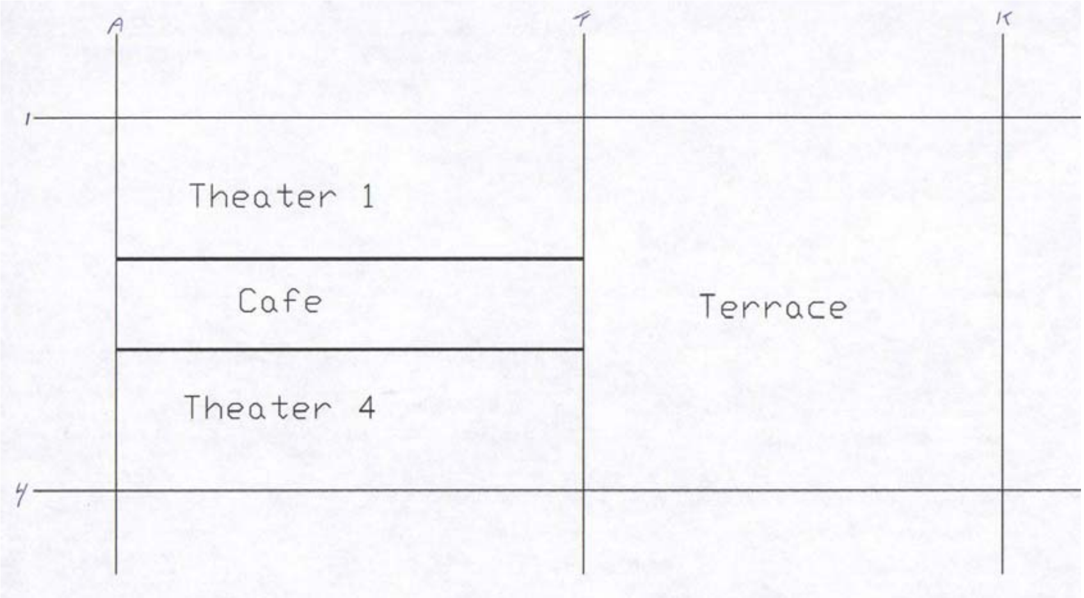


Figure 10.5: Functional lay-out top floor.

10.1 Anchor walls

The bridge will be anchored to the building. To make this possible two concrete walls will be placed in the building. These walls will have a double function, on the one side they are anchor weights for the bridge on the other side they will be walls of the building. First of all the required width of the wall needs to be calculated. The wall will have the following shape, see figure 10.4.

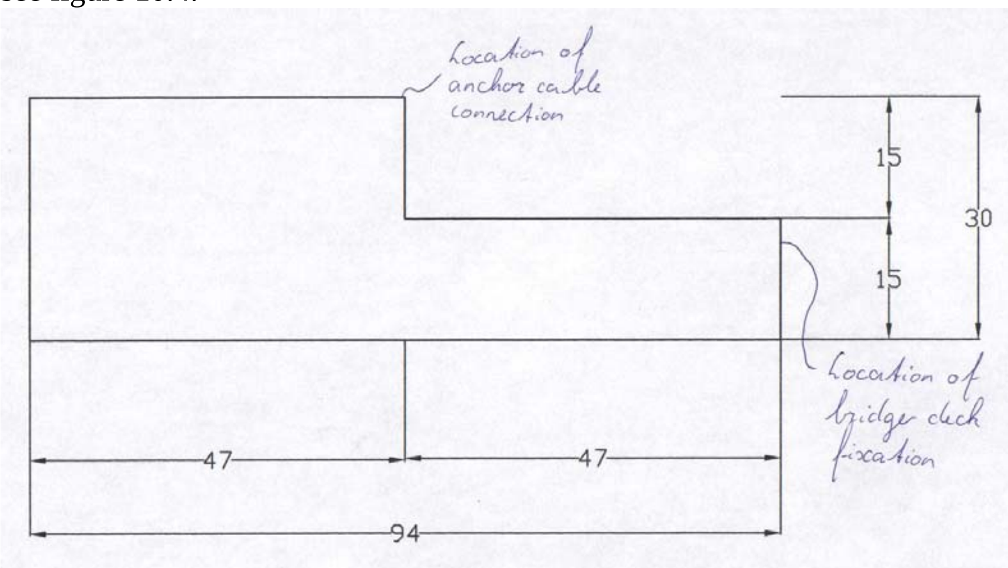


Figure 10.5: Side view concrete anchor wall.

This means the wall has an area of:
$A_{wall} = 47 * 30 + 47 * 15 = 2115\ m^2$

The weight of concrete is $p_{concrete} = 25\ kN/m^3$. That means that for this wall it holds that the weight is:
$q_{wall} = A_{wall} * p_{concrete} = 2115 * 25 = 52875\ kN/m$

The forces acting on the wall are that of the anchor stay-cable and a horizontal force from the bridge deck.
The normal force in the stay-cable is 22961 kN, this can be split up into an horizontal force and a vertical force:
$F_{1,hor,d}$ = 22864 kN
$F_{1,ver,d}$ = 2111 kN

This horizontal component is equal to the resulting horizontal force from the bridge deck, $F_{deck,d}$.

For design purpose the weight of the wall needs to be multiplied with a safety factor. Because this force is working favourably, the safety factor becomes, $\gamma = 0{,}9$.
This means the vertical force caused by the self-weight of the wall is:
$F_z = q_{wall} * d_{wall} * 0{,}9 = 47587 d_{wall}$ kN.

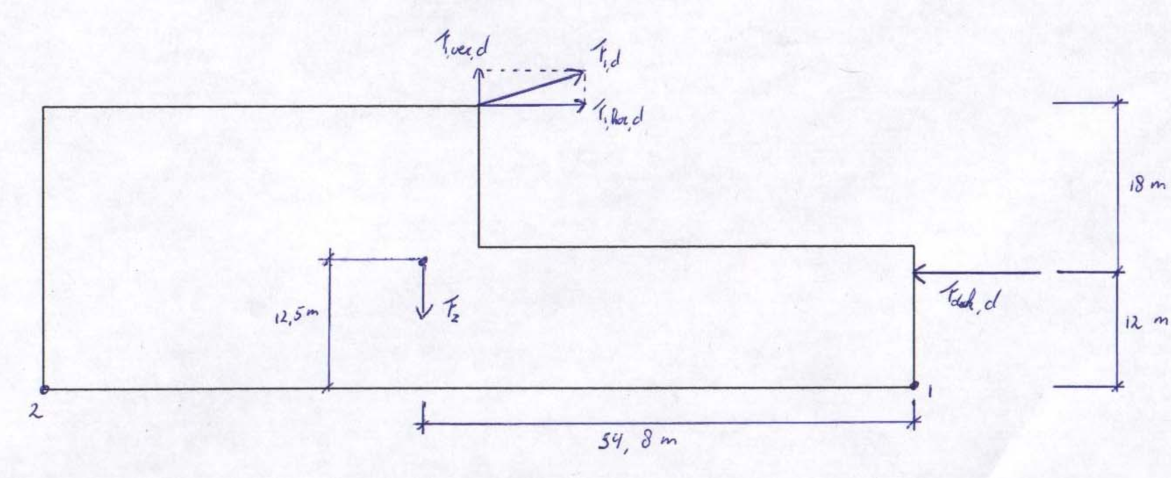


Figure 10.6: Loading on the anchor wall.

To determine the required wall thickness, d_{wall}, the moment around point 1 is taken, for the thickness to be sufficient it is required, $\Sigma M_1 = 0$.

$$\Sigma M_1 = F_{1,ver,d} * 47 + F_{1,hor,d} * 30 - F_{deck,d} * 12 - F_z * 54{,}8 = 0$$
$$= 2111 * 47 + 22864 * 30 - 22864 * 12 - 47587 d_{wall} * 54{,}8 = 0$$
$$= 510769 - 2607768 d_{wall} = 0$$
$$\Rightarrow d_{wall} = 196 mm$$

The minimum required wall thickness is 200 mm.

Besides being heavy enough to be able to act as an anchor weight, the wall must also be wide enough to connect the cable to the wall. The same kind of connection will be used here as was used for connecting the cable to the bridge.

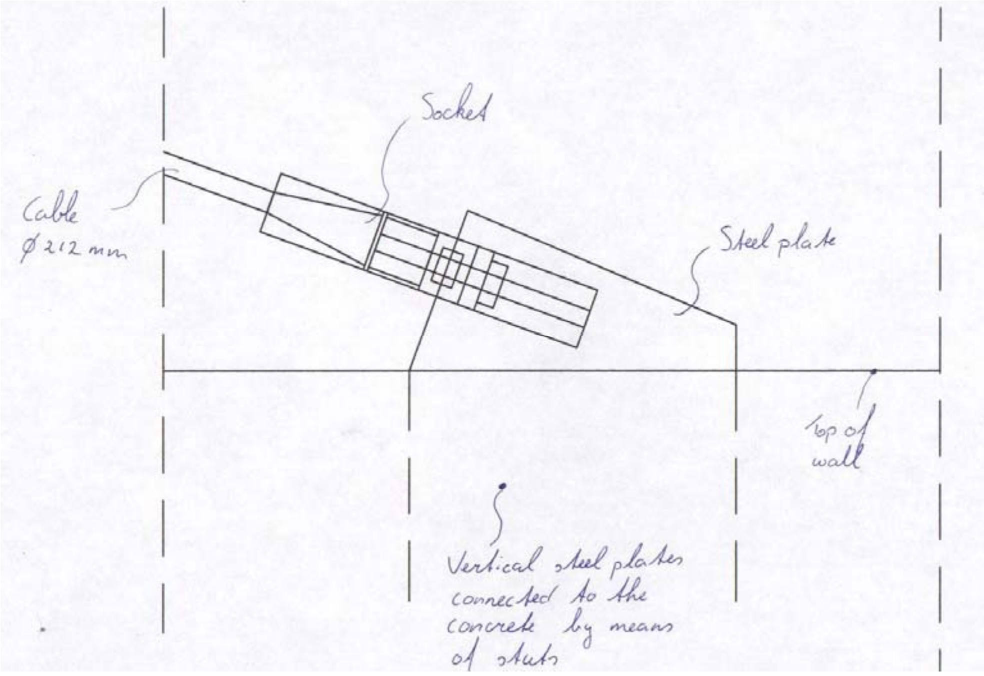


Figure 10.7: Connection between building and cable.

The cable is placed in a socket which is welded to a steel plate.
The diameter of the cable is 212 mm, this means the outer diameter of the socket is 650 mm (see appendix II).
The material chosen for the plate is steel, S355.

Now the required dimensions for the plate can be calculated.
The normal force in the cable is F_d = 22961 kN.

The required area for the plate follows from:

$$A_{pl} = \frac{F_d}{f_{y,d}} = \frac{22961 \cdot 10^3}{355} = 64679 mm^2$$

The height of the plate is assumed at 1,5 * socket diameter. This means the plate will need a width of:
Width = A_{pl} / 1,5*diameter = 64679 / 975 = 66 mm

The connection between the steel plate and the anchor wall is made by casting the plate into the concrete.

The wall has a thickness 200 mm, making the width sufficient to cast the plate into.

10.2 The roof

There are two parts of the roof, the part over theatre 1 and 4 (top floor), which will only be used as roof and the part over theatre 3 and 6(bottom floor) which will also be the terrace.

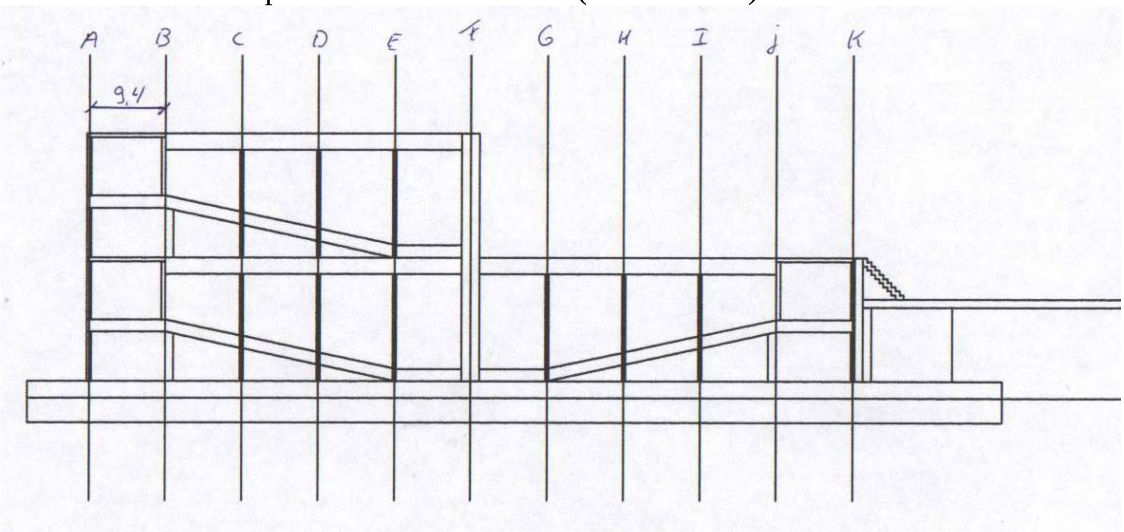


Figure 10.8: Side view theatre.

10.2.1 Roof theatre 1 & 4

The loading on this roof consists of a permanent loading, caused by the self weight of the plates used and the top layer placed over these plates and a variable loading caused by either the wind or snow. The chosen variable load depends on which of the two is the largest and thereby governing loading.
The length of one theatre is 47 m. This distance is cut into five pieces by placing girders with a c.o.c distance of 9,4 m.

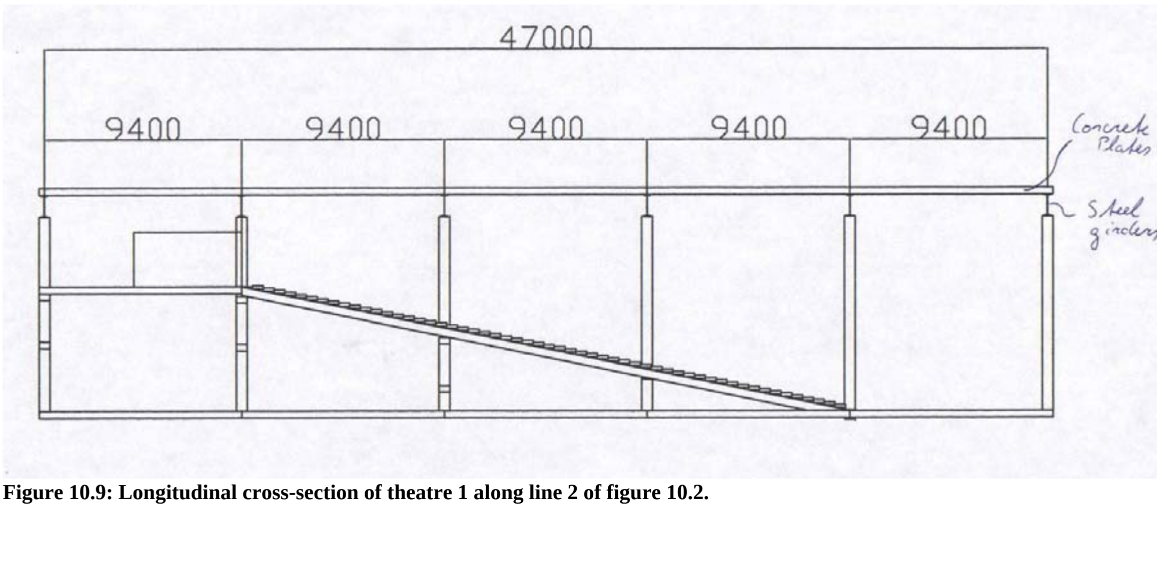


Figure 10.9: Longitudinal cross-section of theatre 1 along line 2 of figure 10.2.

The type of plates used for spanning the roof will be channel plate floors. The type chosen here is A320, this plate has the following characteristics (see also appendix 6):
h = 320 mm
b = 1200 mm
q = 450 kg/m^2 * 10 = 4500 N/m^2
A = 215850 mm^2
I = 2592,2*10^6 mm^4

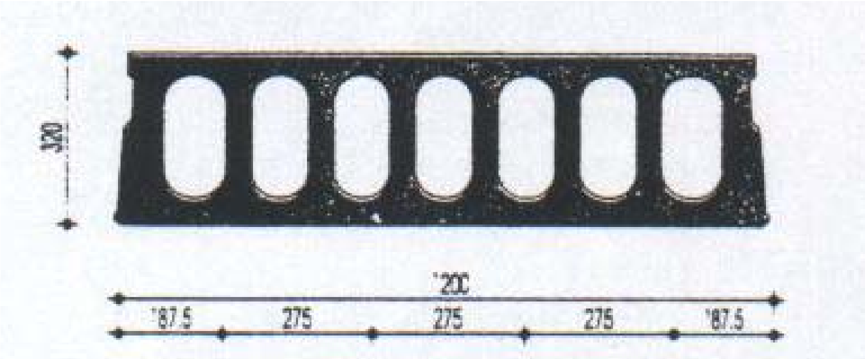


Figure 10.10: Cross-section channel plate A320.

The plate and its loading can be schematised as followed:

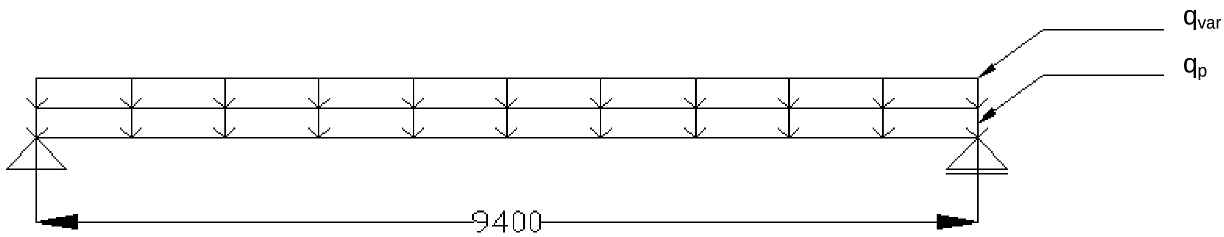


Figure 10.11: Schematisation channel plate.

The permanent loading due to the plate is:
$q_{p,pl}$ = 1,2 * 4500 = 5,4 kN/m

For the top layer a bitumen layer will be used, this leads to a loading of:
$q_{p,bit}$ = 1,2 * 0,1 = 0,1 kN/m

This gives a total permanent loading of:
q_p = 0,1 + 5,4 = 5,5 kN/m

For the variable loading the largest of the wind or snow loading is governing.
Due to the fact that the roof is a flat roof the snow load is 0,56 kN/m^2 or per plate:
$q_{var,snow}$ = 1,2 * 0,56 = 0,67 kN/m

The wind loading is defined as:
$q_w = C_{dim}C_{index}C_{eq} * p_w * A * \varphi_1$

With:
A = area [m^2]
C = wind shape factor [-]
p_w = extreme pressure caused by wind [kN/m^2]

For the location Zwijndrecht it holds that this is a area with buildings in predefined area 2 and the altitude is 30 m. This leads to a $p_w = 1{,}12$ kN/m^2.

$q_w = 1 * 0{,}4 * 1 * 1{,}12 * 1{,}2 * 1 = 0{,}54$ kN/m upward

For design purpose the loadings need to be multiplied with safety factors, these factors are:
$\gamma_p = 1{,}2$
$\gamma_{var} = 1{,}5$

$\gamma_{fav} = 0{,}9$

This means that the total loading in case of snow loading becomes:
$q_d = \gamma_p * q_p + \gamma_{var} * q_{var,\,snow} = 1{,}2 * 5{,}5 + 1{,}5 * 0{,}67 = 7{,}6$ kN/m

And in case of wind loading:
$q_d = \gamma_{fav} * q_p + \gamma_{var} * q_{var,\,wind} = 0{,}9 * 5{,}5 - 1{,}5 * 0{,}54 = 4{,}1$ kN/m

This means the snow loading is the governing loading. When looking at the figure for the loading of the plates (see appendix 6) it becomes clear that for a span of 9,4 m and a loading of 7,6 kN/m a plate type of A320 is sufficient to withstand the loading.

No only the strength but also the stiffness of the plate needs to be checked.
The maximum sag of a roof plate is defined as:
$u_{max} < 0{,}004*l = 0{,}004 * 9{,}4 = 38$ mm

The sag can be calculated with:

$$\delta = \frac{5M_{y,s,rep}l^2}{48EI}$$

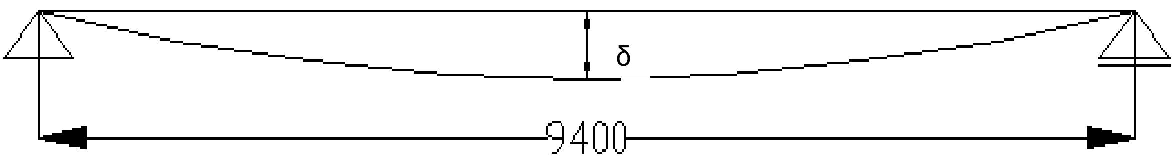


Figure 10.12: Schematisation sag.

For $M_{y,s,rep}$ it holds:

$$M_{y,s,rep} = \tfrac{1}{8} * q_p * l^2 + \tfrac{1}{8} * q_{\text{var}} * l^2$$
$$= \tfrac{1}{8} * 5{,}5 * 9{,}4^2 + \tfrac{1}{8} * 0{,}67 * 9{,}4^2 = 68kNm$$

This leads to a sag of:

$$\delta = \frac{5M_{y,s,rep}l^2}{48EI} = \frac{5 \cdot 68 \cdot 10^6 \cdot 9400^2}{48 \cdot 31000 \cdot 2592{,}2 \cdot 10^6} = 8mm$$

This leads to a unity-check of:

$$\frac{\delta}{u_{\max}} = \frac{8}{38} = 0{,}21 \leq 1$$

Therefore it can be concluded that the stiffness is large enough.
Now that the roof plates have been checked, the girders supporting these roof plates can be calculated. The theatre has a width of 27 m, because no columns can be placed in the middle of the theatres the girders supporting the roof plates will have to span the whole width in once. Therefore the girders will also have a span of 27 m.

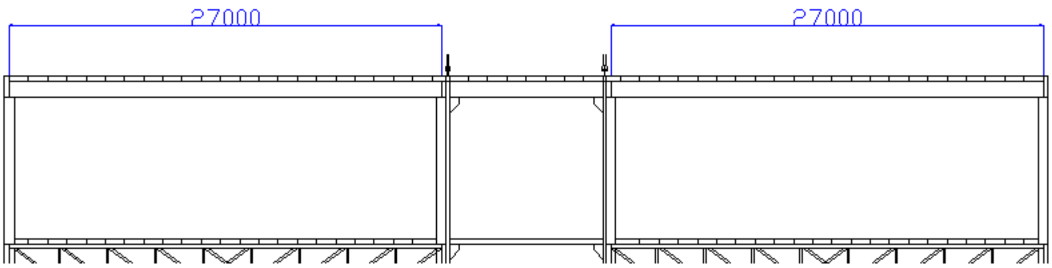


Figure 10.13: Cross-section of theatre 1 and 4 along line B of figure 10.2.

The girders can be schematised as followed:

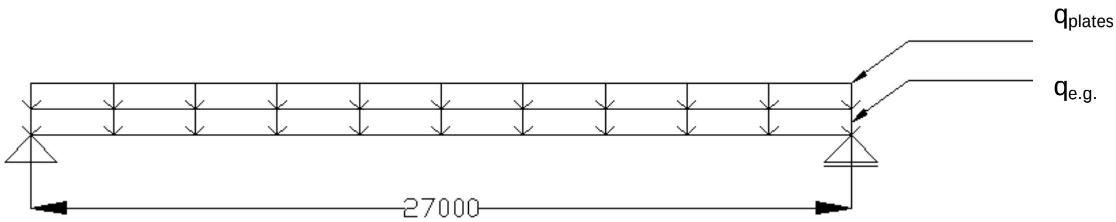


Figure 10.14: Schematisation girder.

The loading onto these girders comes from the support reactions for the plates, these are:

$$V_{pl,d} = \frac{1}{2} q_d l = \frac{1}{2} \cdot 7{,}6 \cdot 9{,}4 = 35{,}7 kN$$

This force can be spread out over the width of the plate leading to a loading of:
$q_{plates,d} = V_{pl,d} / b_{plate} = 35{,}7 / 1{,}2 = 29{,}8$ kN/m.

This leads to a maximum bending moment of:
$M_{y,s,d} = 1/8 * q_d * l^2 = 1/8 * 29{,}8 * 27^2 = 2716$ kNm

Besides this loading, also the loading caused by the weight of the girder has to be taken into account.
The chosen beam is a HE1000A. This beam has a loading of $q_{e.g.} = 2{,}72$ kN/m.
Applying safety factors, this leads to a loading of:
$q_{e.g.,d} = 2{,}72 * 1{,}2 = 3{,}3$ kN/m

This leads to a maximum bending moment of:
$M_{y,s,d} = 1/8 * q_d * l^2 = 1/8 * 3{,}3 * 27^2 = 301$ kNm

Giving a total maximum bending moment of:
$M_{y,s,d}$ = 2716 + 301 = 3017 kNm

To calculate the maximum stress in the top and bottom flange it holds:

$$\sigma_d = \frac{M_{y,s,d}}{W_z} = \frac{3017 \cdot 10^6}{11190 \cdot 10^3} = 270 N / mm^2$$

With $f_{y,d}$ = 355 N/mm^2 this leads to a unity-check of:

$$\frac{\sigma_d}{f_{y,d}} = \frac{270}{355} = 0{,}76 \leq 1$$

Making the dimensions of this beam sufficient to support this loading.

Besides the strength, also the stiffness has to be checked.

The total loading (without safety factors) on this beam is the loading caused by the roof plates plus the loading caused by the self weight.
$q_{rep} = q_{plates} + q_{e.g.}$
= ½ * 6,17 * 9,4 / 1,2 + 2,72 = 26,9 kN/m

This leads to a maximum bending moment of:
$M_{y,s,rep}$ = 1/8 * q_{rep} * l^2 = 1/8 * 26,9 * 27^2 = 2451 kNm

This leads to a maximum sag of:

$$\delta = \frac{5M_{y,s,rep}l^2}{48EI} = \frac{5 \cdot 2451 \cdot 10^6 \cdot 27000^2}{48 \cdot 210000 \cdot 6{,}64 \cdot 10^9} = 103mm$$

The maximum allowable sag is:
u_{max} < 0,004*l = 0,004 * 27000 = 108 mm

This leads to a unity-check of:

$$\frac{\delta}{u_{\max}} = \frac{103}{108} = 0{,}95 \leq 1$$

Making this girder stiff enough.

10.2.2 Roof theatre 3 & 6

Theatre 3 and 6 have the same basic lay-out as theatre 1 and 4.
The length of one theatre is 47 m. This distance is cut into five pieces by placing girders with a c.o.c distance of 9,4 m.

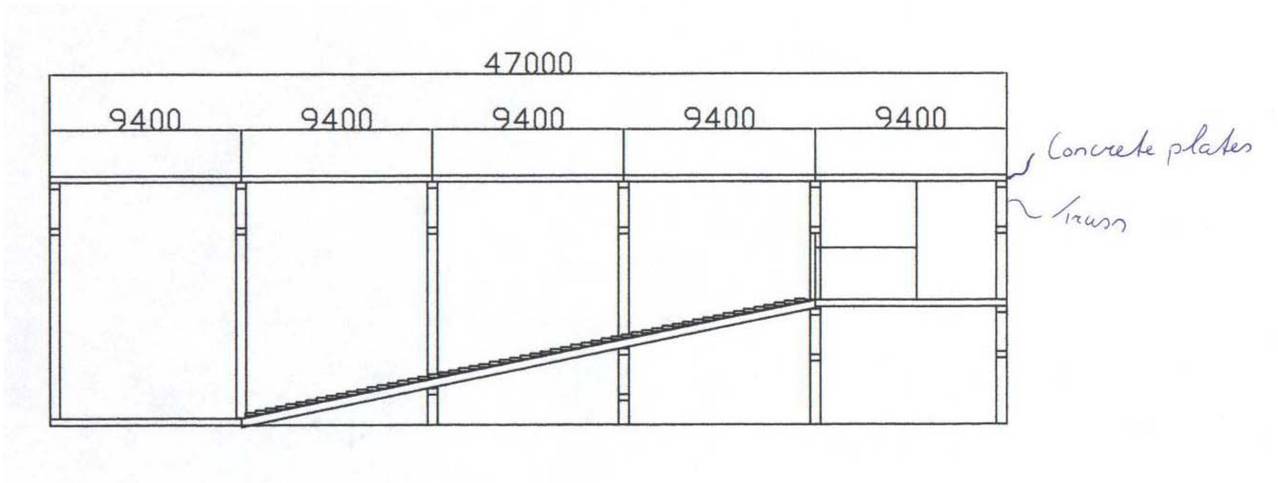


Figure 10.15: Longitudinal cross-section of theatre 3 along line 2 of figure 10.2.

Besides the loadings used in the previous part there is an extra loading which needs to be applied, because the roof of this theatre will be used as a terrace. This loading is a variable loading, $p_{var} = 4,0$ kN/m^2.
This leads to a loading on the plate of $q_{var} = 4,0 * 1,2 = 4,8$ kN/m and $q_{var,d} = 4,8 * 1,5 = 7,2$ kN/m.
This leads to a total loading of:
$q_d = \gamma_p * q_p + \gamma_{var} * q_{var,\ snow} + \gamma_{var} * q_{var} = 1,2 * 5,5 + 1,5 * 0,67 + 4,8 * 1,5 = 14,8$ kN/m

This leads to the following schematisation:

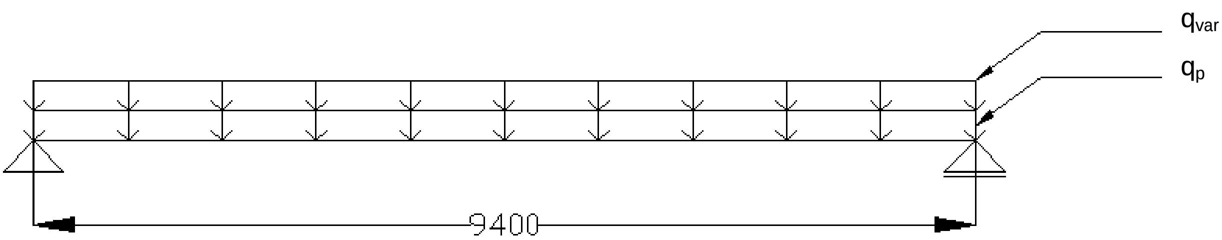


Figure 10.16: Schematisation channel plate.

From the loading diagram of the channel plates (see appendix 6) it becomes clear that with this type of plate (A320) the maximum span is 10,5 m. The span is 9,4 m, therefore it can be concluded that this plate can be used for this span.

Now also the stiffness has to be checked.

The maximum allowed sag is:
$u_{max} < 0,004*l = 0,004 * 9,4 = 38$ mm

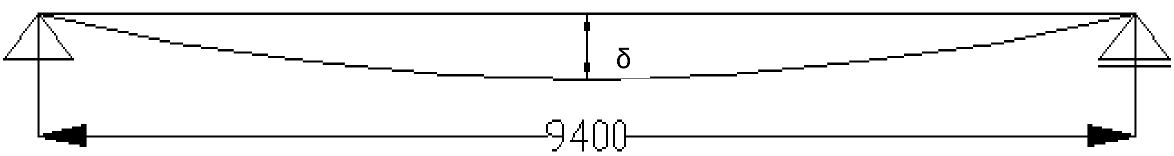


Figure 10.17: Schematisation sag.

And the maximum bending moment is:

$$M_{y,s,rep} = \frac{1}{8} * q_p * l^2 + \frac{1}{8} * q_{\text{var}} * l^2$$

$$= \frac{1}{8} * 5{,}5 * 9{,}4^2 + \frac{1}{8} * 5{,}47 * 9{,}4^2 = 121kNm$$

This leads to a sag of:

$$\delta = \frac{5M_{y,s,rep} l^2}{48EI} = \frac{5 \cdot 121 \cdot 10^6 \cdot 9400^2}{48 \cdot 31000 \cdot 2592{,}2 \cdot 10^6} = 14mm$$

This leads to a unity-check of:

$$\frac{\delta}{u_{\max}} = \frac{14}{38} = 0{,}37 \leq 1$$

Therefore it can be concluded that the stiffness is large enough.

Now the girders for the roof of theatre 3 and 6 will be calculated. For these girders a truss will be used.

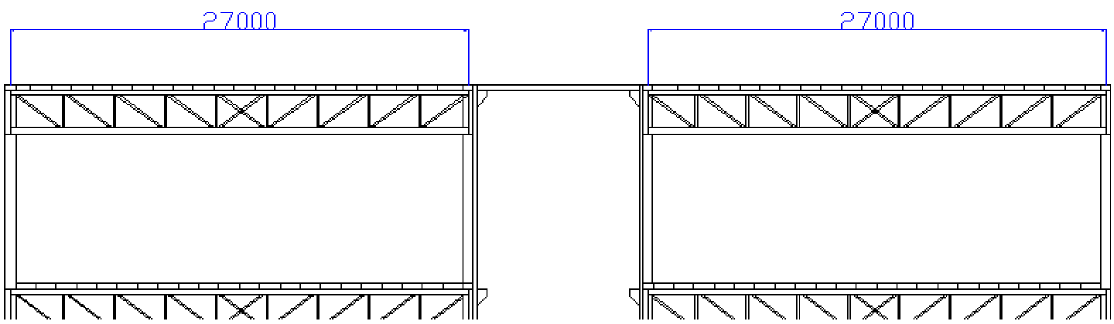


Figure 10.18: Cross-section of theatre 4 and 6 along line B of figure 10.3.

The top and bottom flange of this truss will consist of a square box section with dimensions:
b = 500 mm
h = 300 mm
t = 10 mm

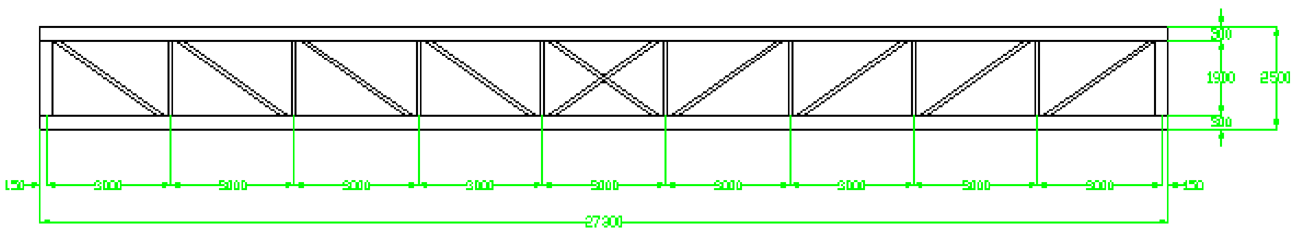


Figure 10.19: Truss used for girder, as shown in figure 10.18.

The total height of the truss will be 2500 mm.

The loading to this girders comes from the support reactions for the plates, these are:

$$V_{pl,d} = \frac{1}{2} q_d l = \frac{1}{2} \cdot 14{,}8 \cdot 9{,}4 = 69{,}6 kN$$

This force can be spread out over the width of the plate leading to a loading of:
$q_d = V_{pl,d} / b_{plate} = 69{,}6 / 1{,}2 = 58{,}0$ kN/m.

Besides this loading, also the loading caused by the weight of the girder has to be taken into account.
The loading caused by the self weight of the truss is 2,4 kN/m.
Applying safety factors, this leads to a loading of:
$q_{per,d} = 2{,}4 * 1{,}2 = 2{,}9$ kN/m

For calculating the truss the loading is schematised as point loadings acting on the nodes of the truss.

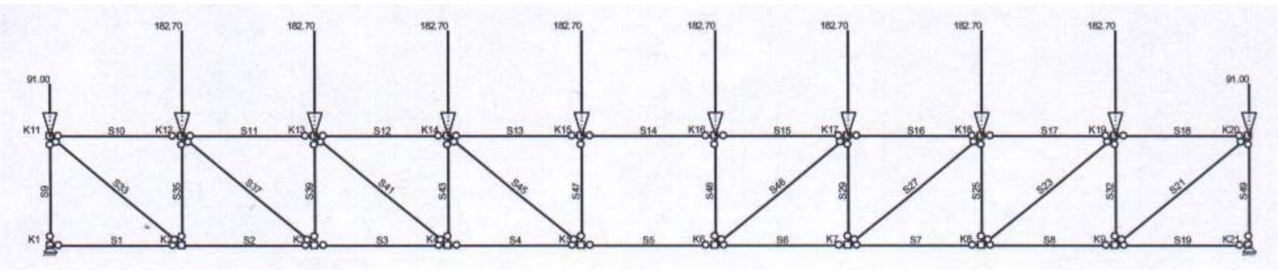


Figure 10.20: Schematisation loading on truss, as shown in figure 10.18.

The bending stiffness of the truss can be approximated by applying the Steiner-rule for the top and bottom flange.
This leads to:
$I = 24{,}7*10^9$ mm^4
$W = 19{,}76*106$ mm^3

First the top and bottom flange will be checked, after this the vertical and diagonal components of the truss will be checked.

To check the strength of the top and bottom flange of the girder, the force acting on these girders must be calculated. This force was calculated by computer and is $F_n = 2192$ kN (see appendix 7).

The bottom flange needs to be checked on tension strength:
The strength of this girder is:
$N_{u,d} = A * f_{y,d} = 15450 * 355 = 5485$ kN

This leads to a unity-check of:
$F_{fl} / N_{u,d} = 2192 / 5485 = 0{,}4 < 1$

Making the dimensions of this beam sufficient to support this loading.

The top flange has to be checked for compression:
The slenderness is:
$\lambda = l_{buc} / i = 27000 / 186 = 145$

$\lambda_{rel} = \lambda / \lambda_e = 145 / 76,4 = 1,9$

This leads to a buckling factor of $\omega_{buc} = 0,24$

The unity-check now becomes:
$F_{fl} / \omega_{buc} N_{u,d} = 2220 /(0,24*5485) = 1,69$

This means the girder will buckle.

To prevent this from happening the top flanges are connected in the middle with a beam providing stability against buckling. This measure means the buckling length is reduced to half the span is, 13500 mm. As a stabilizer a square box section will be welded to the top flange as can be seen in figures 10.21 and 10.22.

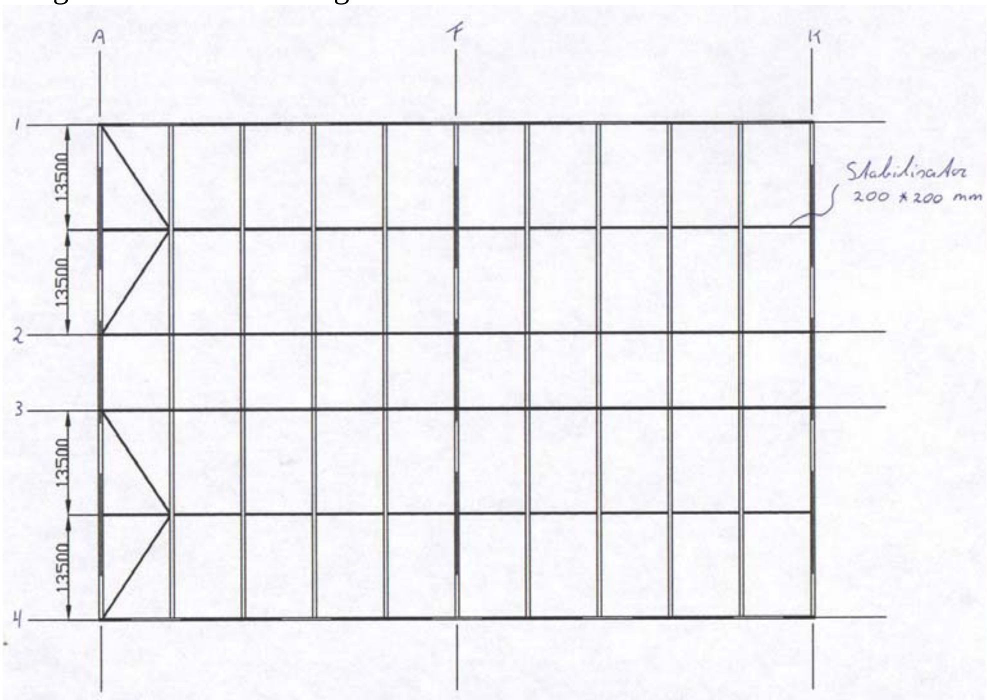


Figure 10.21: Top view cross-section of bottom floor with buckling stabilisation.

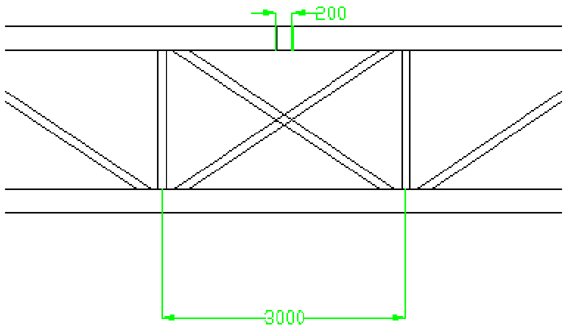


Figure 10.22: Partial front view of girder with buckling stabilisation (200 * 200 mm) welded to top flange.

This leads to a new slenderness of 72,6 and a relative slenderness of 0,95. The new buckling factor now becomes 0,7.
This leads to a unity-check of:
$F_{fl} / \omega_{buc} N_{u,d} = 2220 /(0,70*5485) = 0,58 < 1$

This means the beam can now withstand the loading.

Besides the strength, also the stiffness has to be checked.

The total loading (without safety factors) on this beam is the loading caused by the roof plates plus the loading caused by the self weight.
$q_{rep} = ½ * 11 * 9,4 / 1,2 + 2,4 = 45,5$ kN/m

This leads to a maximum bending moment of:
$M_{y,s,rep} = 1/8 * q_{rep} * l^2 = 1/8 * 45,5 * 27^2 = 4146$ kNm

This leads to a maximum sag of:

$$\delta = \frac{5M_{y,s,rep}l^2}{48EI} = \frac{5 \cdot 4146 \cdot 10^6 \cdot 27000^2}{48 \cdot 210000 \cdot 24,7 \cdot 10^9} = 61mm$$

The maximum allowable sag is:
$u_{max} < 0,004*l = 0,004 * 27000 = 108$ mm

This leads to a unity-check of:

$$\frac{\delta}{u_{max}} = \frac{61}{108} = 0,56 \leq 1$$

Making this girder stiff enough.

Now the diagonal and vertical elements of the truss need to be checked. The vertical elements are under compression and the diagonal are under tension.
First the vertical elements will be calculated.

The largest compressive force is found in the side elements, this is, $F_d = 822$ kN

This element will have the same dimensions as the top and bottom girder, 500 * 300 mm^2 with t = 10 mm.

This leads to a slenderness of:

$$\lambda = \frac{l_{buc}}{i} = \frac{2000}{125,5} = 15,9 \text{ (weak direction)}$$

This gives a relative slenderness of:

$$\lambda_{rel} = \frac{\lambda}{\lambda_e} = \frac{15,9}{76,4} = 0,21$$

This means that the buckling coefficient, $\omega_{buc} = 0,99$

This leads to a unity-check of:

$$\frac{F_d}{\omega_{buc} A f_{y,d}} = \frac{822 \cdot 10^3}{0{,}99 \cdot 15450 \cdot 355} = 0{,}15 \leq 1$$

This means the column can withstand buckling.

For easiness during assembly the rest of the vertical components will get the same dimensions as the first vertical next to the edge vertical. The closer the verticals are to the centre of the truss, the smaller the force, therefore if the second vertical is checked and found to be sufficient, all verticals will be strong enough to withstand the loading.
The normal force in the first vertical next to the edge is, $F_{2,d}$ = 731 kN
For the verticals a square box section will be used with dimensions: 150 * 100 mm^2,with t = 5 mm.

This leads to a slenderness of:

$$\lambda = \frac{l_{buc}}{i} = \frac{2000}{40{,}7} = 49{,}1 \text{(weak direction)}$$

This gives a relative slenderness of:

$$\lambda_{rel} = \frac{\lambda}{\lambda_e} = \frac{49{,}1}{76{,}4} = 0{,}64$$

This means that the buckling coefficient, ω_{buc} = 0,87

This leads to a unity-check of:

$$\frac{F_d}{\omega_{buc} A f_{y,d}} = \frac{731 \cdot 10^3}{0{,}87 \cdot 2388 \cdot 355} = 0{,}99 \leq 1$$

This means the column can withstand buckling.

The diagonals in the truss are stressed by a normal tensile force. For the same reason as for the verticals, one size is chosen for all diagonals. The largest tensile force acting on a diagonal is, $F_{t,d}$ = 1142 kN

The minimum required area for the diagonals can now be calculated with:

$$A_{\min} = \frac{F_{t,d}}{f_{y,d}} = \frac{1142 \cdot 10^3}{355} = 3217 mm^2$$

This requirement is met by applying square box-sections with A = 150 * 100 mm^2 and t = 8 mm. This gives a cross-section of, A = 3713 mm^2

This leads to a unity-check of:

$$\frac{F_{t,d}}{A f_{y,d}} = \frac{1142 \cdot 10^3}{3713 * 355} = 0{,}87 \leq 1$$

Therefore the box-section can withstand the loading.

Stability of the hollow channel plates.

The roof of the theatre will have a stabilising factor for the building against horizontal loading. For this to work the plates will have to act as one stiff plate. However both the roofs consist of hollow channel plates type A320.

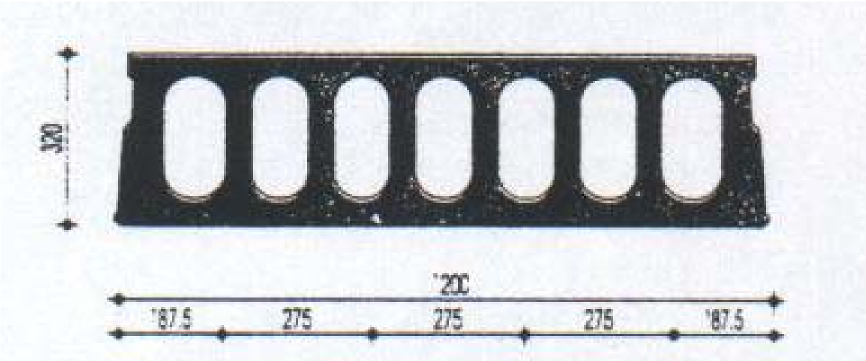


Figure 10.22: Cross-section channel plate A320.

These plates need to be connected before they can act as one stiff plate. This is done by connecting the plates together by placing rebar in the seams between the plates, these seams will then be filled with concrete. Along the edge of the roof a concrete edge beam will be cast in-situ. This ensures that the hollow channel plates act as one stiff plate.

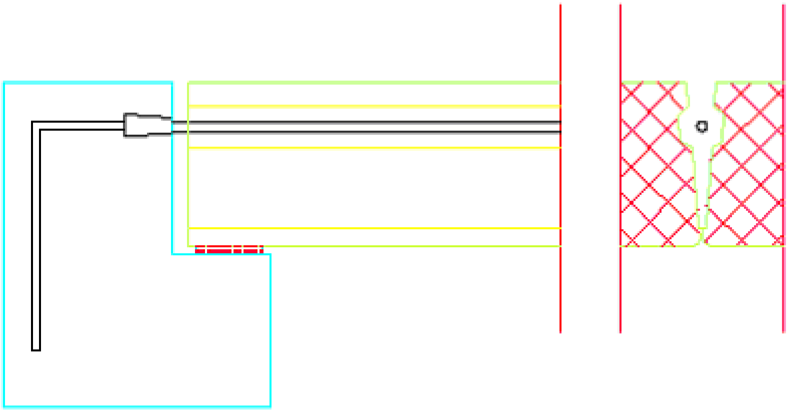


Figure 10.23: Connection between edge beam and channel plate (along line 4 of figure 10.2) and location of rebar in seam between two plates.

10.3 Floors

The floor of each of the theatres can be divided into three parts, a horizontal part in front of the screen, a diagonal part with seating and a horizontal part for the projector room.
The loading of the floors consists of two parts, a variable loading of, p_{var} = 4 kN/m^2 and a permanent loading caused by the self weight of the construction.

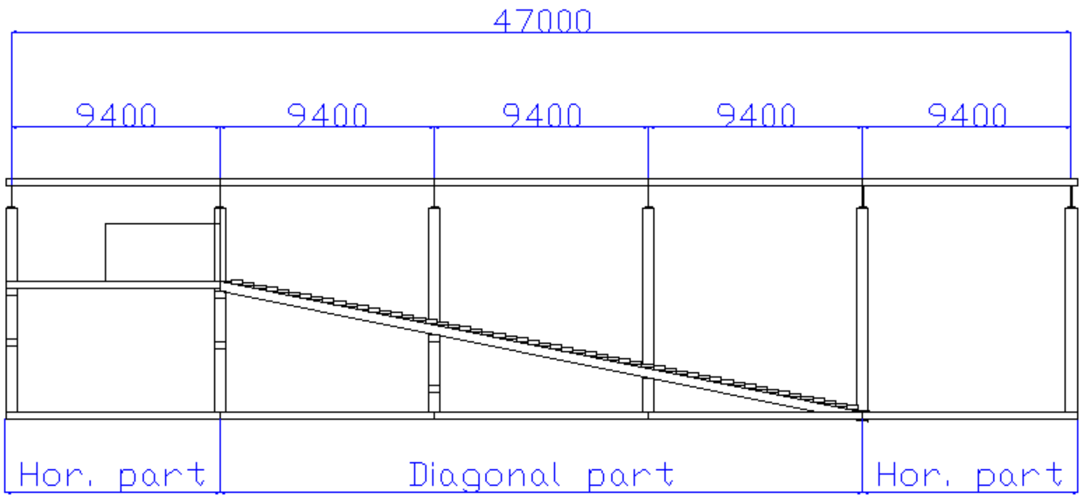


Figure 10.24: Longitudinal cross-section of theatre 1 along line 2 of figure 10.2.

Horizontal part

For the two horizontal parts of the floor the same plates as for the roof will be used, hollow channel plates, A320.
The plate can be schematised as:

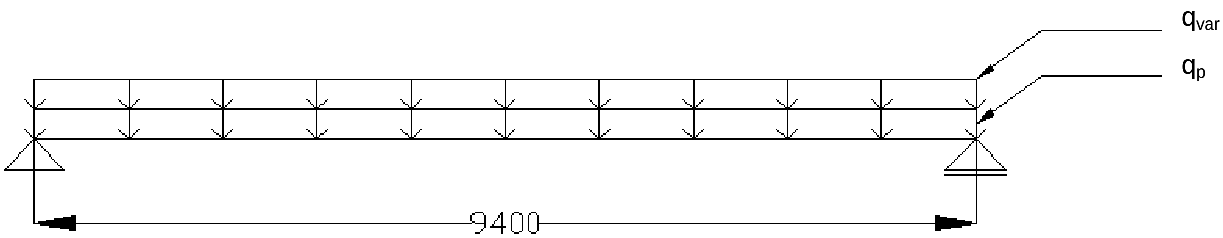


Figure 10.25: Schematisation floor plate with loading.

The total loading on the plate becomes, q_{var} = 4,0 * 1,2 = 4,8 kN/m and $q_{var,d}$ = 4,8 * 1,5 = 7,2 kN/m.
The loading caused by the self-weight is, $q_{p,pl}$ = 1,2 * 4500 = 5,4 kN/m and $q_{p,pl,d}$ = 5,4 * 1,2 = 6,5 kN/m.

The total loading now becomes:
q_{rep} = 4,8 + 5,4 = 10,2 kN/m
q_d = 7,2 + 6,5 = 13,7 kN/m

From the loading diagram of the channel plates (see appendix 6) it becomes clear that with this type of plates the maximum span is 11,0 m. The span is 9,4 m, therefore it can be concluded that this plate type can be used for this span.

Besides the strength also the stiffness of the plate needs to be checked.

The maximum sag of a floor plate is defined as:
u_{max} < 0,004*l = 0,004 * 9,4 = 38 mm

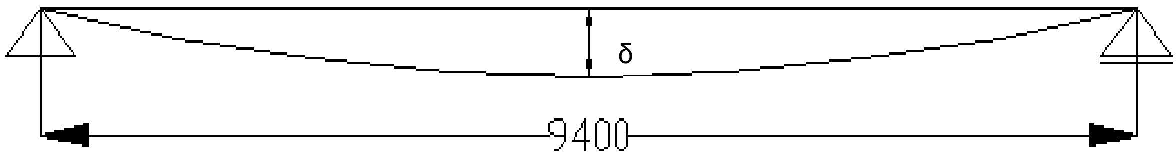


Figure 10.26: Schematisation sag.

The sag can be calculated with:

$$\delta = \frac{5M_{y,s,rep} l^2}{48EI}$$

For $M_{y,s,rep}$ it holds:

$$M_{y,s,rep} = \frac{1}{8} * q_p * l^2 + \frac{1}{8} * q_{\text{var}} * l^2$$
$$= \frac{1}{8} * 4{,}8 * 9{,}4^2 + \frac{1}{8} * 5{,}4 * 9{,}4^2 = 113kNm$$

This leads to a sag of:

$$\delta = \frac{5M_{y,s,rep} l^2}{48EI} = \frac{5 \cdot 113 \cdot 10^6 \cdot 9400^2}{48 \cdot 31000 \cdot 2592{,}2 \cdot 10^6} = 13mm$$

This leads to a unity-check of:

$$\frac{\delta}{u_{\max}} = \frac{13}{38} = 0{,}34 \le 1$$

Therefore it can be concluded that the stiffness is large enough.

For the floor in the projector room the same span and loadings hold, therefore these plates can also be used for this part of the floor

Diagonal part

To ensure that everybody in the theatre has a good line of sight, the seeds are place on a floor which is shaped like a stairway. The seeds are placed upon horizontal concrete plates, which are supported by steel girders with a c.o.c. distance of 5400 mm. These girders are supported at 6 points by trusses, which are supported by the columns of the construction.

First the concrete plates are calculated.

With rules of thumb the following height for the concrete plates is found:
l_{plate} = 5400 mm
h_{plate} =1/35 * l_{plate} ≈ 150 mm

For the calculation the width is assumed at 500 mm. The exact width will depend on the types of chairs used and the leg space required, however this is outside the scope of this thesis.

The loading onto the plates consists of a variable loading and a permanent loading, this leads to the following total loadings:
q_p = 0,5 * 0,15 * 25 = 1,9 kN/m
$q_{p,d}$ = 1,9 * 1,2 = 2,3 kN/m

q_{var} = 0,5 * 4 = 2,0 kN/m
$q_{var,d}$ = 2,0 * 1,5 = 3,0 kN/m

q_{rep} = 1,9 + 2,0 = 3,9 kN/m
q_d = 2,3 + 3,0 = 5,3 kN/m

The plates can be schematised as followed:

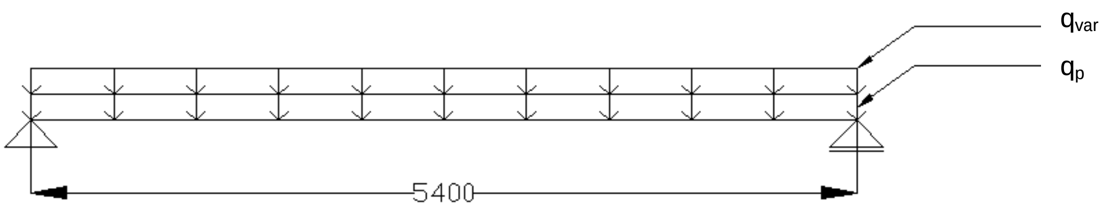


Figure 10.27: Schematisation concrete plate.

This leads to a maximum bending moment of:
$M_d = 1/8 q_d l^2 = 1/8 * 5,3 * 5,4^2 = 19,3$ kNm

To determine the required minimum amount of rebar, the following factor needs to be calculated:

$$\frac{M_d}{bd^2 f`_b} = \frac{19,3}{0,5 \cdot 0,15^2 \cdot 21} = 82$$

According to the standard table (see appendix 8) this leads to a rebar percentage, ω_0 = 0,42 %

The required minimum amount of rebar now becomes:
$A = \omega_0 * b * d = 0,42 * 10^{-2} * 500 * 150 = 315\ mm^2$

When using 5 bars with a diameter of 10 mm the area is 393 mm^2.

Now the diagonal girders supporting the plates can be calculated.

These girders can be schematised as continuous girders on four supports (see figure 10.28).

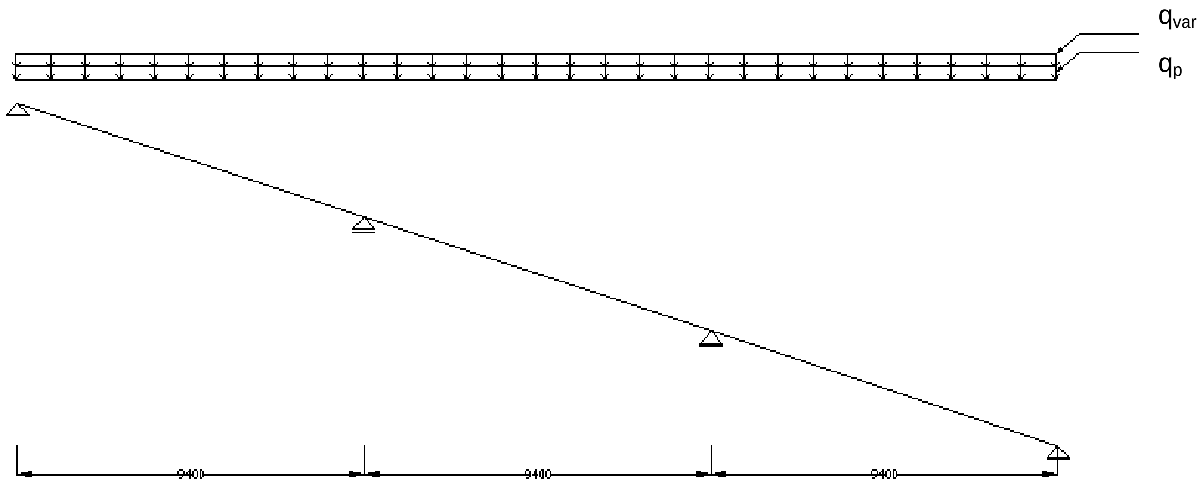


Figure 10.28: Schematisation girder.

The loading on the girders is caused by the loading from the plates and the self-weight of the girders.

The loading caused by the girders is:
V = ½ * q_d * l = ½ * 5,3 * 5,4 = 14 kN
This force is spread over 500 mm, giving a loading on the girder of:
$q_{plate,d}$ = 14*10^3 / 500 = 28 kN/m

With a rule of thumb the required minimum height is found, and this leads to a HE400A beam
This beam has a self-weight of 1,25 kN/m giving a design loading of:
$q_{beam,d}$ = 1,2 * 1,25 = 1,5 kN/m

This makes the total loading on the girder:
q_d = 28 + 1,5 = 29,5 kN/m

For the maximum bending moment in the girder is found:

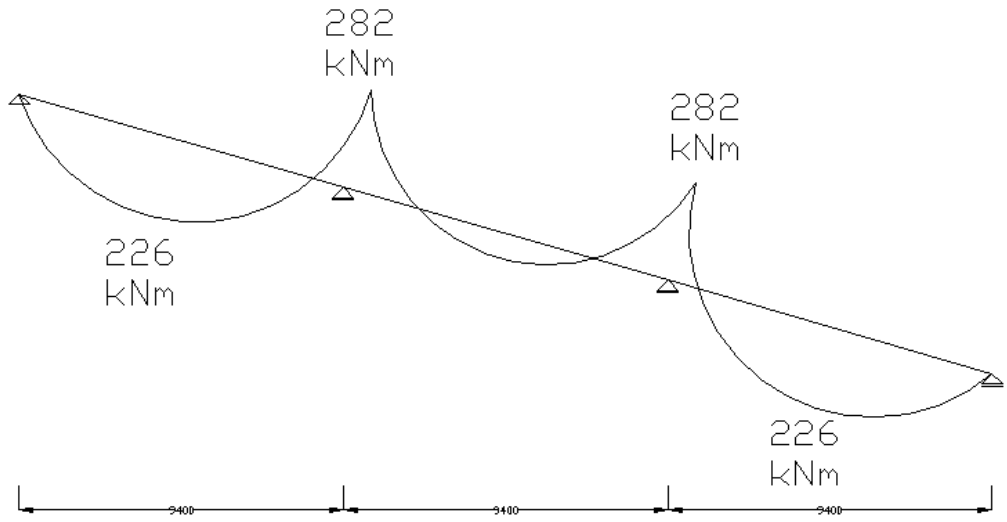


Figure 10.29: Bending moment line.

The maximum moment is found at the bearings and is:
M_d = 282 kNm

This leads to a stress of:

$$\sigma_d = \frac{M_d}{W} = \frac{282 \cdot 10^6}{2,31 \cdot 10^6} = 122 N / mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_d}{f_{y,d}} = \frac{122}{355} = 0,34 \leq 1$$

This means the girder is strong enough to support the loading.

Also the stiffness has to be checked.

The maximum allowed sag is given by:

$$u \leq 0,004 * l = 0,004 * 6000 = 24mm$$

The maximum sag is, δ = 14 mm

This leads to a unity check of:

$$\frac{\delta}{u} = \frac{14}{24} = 0,58 \leq 1$$

This means the girder is stiff enough.

The trusses used to support these girders will get the same dimensions as the girders used in the roof of theatre 3 and 6.

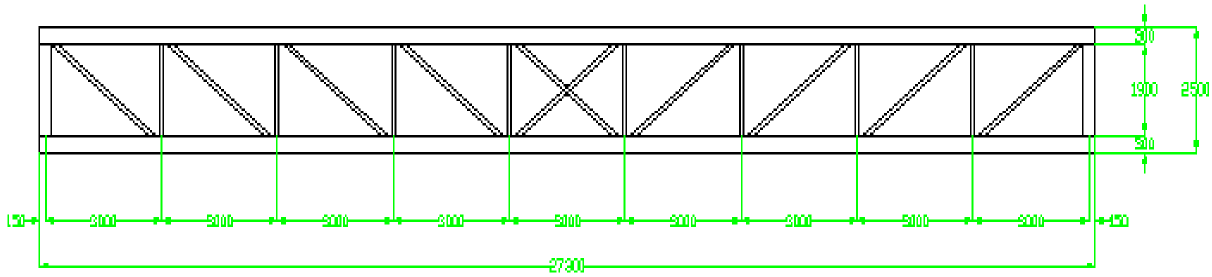


Figure 10.30: Truss.

The top and bottom flange of this truss will consist of a square box section with dimensions:
b = 500 mm
h = 300 mm
t = 10 mm

The total height of the truss will be 2500 mm.

The loading caused by the self weight of the truss is 2,4 kN/m.
Applying safety factors, this leads to a loading of:
$q_{per,d}$ = 2,4 * 1,2 = 2,9 kN/m

The bending stiffness of the truss can be approximated by applying the Steiner-rule for the top and bottom flange.
This leads to:
$I = 24{,}7 \cdot 10^9\ \mathrm{mm}^4$
$W = 19{,}76 \cdot 106\ \mathrm{mm}^3$

The truss is schematised as a girder on two supports. The distributed load is replaced by point loadings on the nodes of the truss..
The largest force caused by the girder is, F_{girder} = 330 kN (see computer calculations, appendix 9). This leads to a point load of 183 kN per node. For the self weight a point load per node of, 2,9 * 3 = 8,7 kN is found.
This leads to a total point load of 192 kN per node.

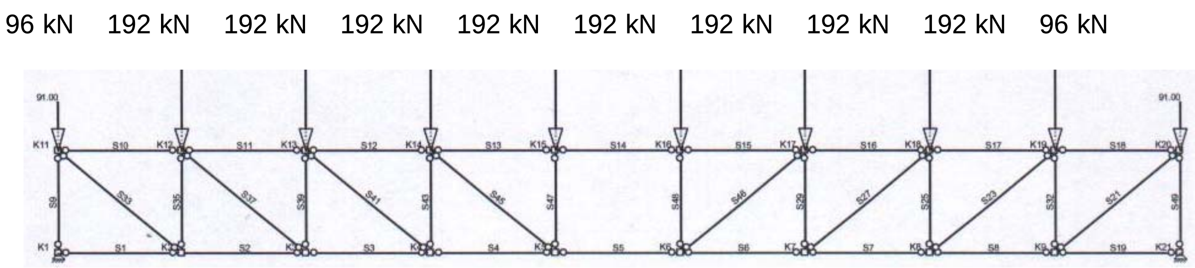


Figure 10.31: Schematisation of loading onto the truss.

For the normal force in the top and bottom flange it is found:
N_d = 2244 kN

The bottom flange needs to be checked on tension strength:
The strength of this girder is:
$N_{u,d} = A * f_{y,d} = 15450 * 355 = 5485$ kN

This leads to a unity-check of:
$F_{fl} / N_{u,d} = 2244 / 5485 = 0,41 < 1$

Making the dimensions of this beam sufficient to support this loading.

The top flange has to be checked for compression:
The slenderness is:
$\lambda = l_{buc} / i = 27000 / 186 = 145$

$\lambda_{rel} = \lambda / \lambda_e = 145 / 76,4 = 1,9$

This leads to a buckling factor of $\omega_{buc} = 0,24$

The unity-check now becomes:
$F_{fl} / \omega_{buc} N_{u,d} = 2244 /(0,24*5485) = 1,70$

This means the girder will buckle.

To prevent this from happening the top flanges are connected in the middle with a stabiliser against buckling. This measure means the buckling length is reduced to half the span is, 13500 mm.
This leads to a new slenderness of 72,6 and a relative slenderness of 0,95. The new buckling factor now becomes 0,7. This leads to a unity-check of:
$F_{fl} / \omega_{buc} N_{u,d} = 2244 /(0,70*5485) = 0,58 < 1$

This means the beam can now withstand the loading.

Besides the strength, also the stiffness has to be checked.

The total loading (without safety factors) on this beam is the loading caused by the girders plus the loading caused by the self weight.
$q_{rep} = 60.9 + 2,4 = 63,3$ kN/m

This leads to a maximum bending moment of:
$M_{y,s,rep} = 1/8 * q_{rep} * l^2 = 1/8 * 63,3 * 27^2 = 5768$ kNm

This leads to a maximum sag of:

$$\delta = \frac{5 M_{y,s,rep} l^2}{48 EI} = \frac{5 \cdot 5768 \cdot 10^6 \cdot 27000^2}{48 \cdot 210000 \cdot 24,7 \cdot 10^9} = 84 mm$$

The maximum allowable sag is:
$u_{max} < 0,004*l = 0,004 * 27000 = 108$ mm

This leads to a unity-check of:

$$\frac{\delta}{u_{\max}} = \frac{84}{108} = 0{,}78 \leq 1$$

Making this girder stiff enough.
Now the diagonal and vertical elements of the truss need to be checked. The vertical elements are under compression and the diagonal are under tension.
First the vertical elements will be calculated.

The largest compressive force is found in the side elements, this is, F_d = 831 kN

This element will have the same dimensions as the top and bottom girder, 500 * 300 mm^2 with t = 10 mm.

This leads to a slenderness of:

$$\lambda = \frac{l_{buc}}{i} = \frac{2000}{125{,}5} = 15{,}9 \text{ (weak direction)}$$

This gives a relative slenderness of:

$$\lambda_{rel} = \frac{\lambda}{\lambda_e} = \frac{15{,}9}{76{,}4} = 0{,}21$$

This means that the buckling coefficient, ω_{buc} = 0,99

This leads to a unity-check of:

$$\frac{F_d}{\omega_{buc} A f_{y,d}} = \frac{831 \cdot 10^3}{0{,}99 \cdot 15450 \cdot 355} = 0{,}15 \leq 1$$

This means the column can withstand buckling.

For easiness during assembly the rest of the vertical components will get the same dimensions as the first vertical next to the edge vertical. The closer the verticals are to the centre of the truss, the smaller the force, therefore if the second vertical is calculated, all verticals will be strong enough to withstand the loading.
The normal force in the first vertical next to the edge is, $F_{2,d}$ = 79 kN
For the verticals a square box section will be used with dimensions: 150 * 100 mm^2,with t = 6,3 mm.

This leads to a slenderness of:

$$\lambda = \frac{l_{buc}}{i} = \frac{2000}{40{,}2} = 49{,}8 \text{ (weak direction)}$$

This gives a relative slenderness of:

$$\lambda_{rel} = \frac{\lambda}{\lambda_e} = \frac{49{,}8}{76{,}4} = 0{,}65$$

This means that the buckling coefficient, ω_{buc} = 0,87

This leads to a unity-check of:

$$\frac{F_d}{\omega_{buc} A f_{y,d}} = \frac{739 \cdot 10^3}{0{,}87 \cdot 2972 \cdot 355} = 0{,}81 \leq 1$$

This means the column can withstand buckling.

The diagonals in the truss are stressed by a normal tensile force. For the same reason as for the verticals, one size is chosen for all diagonals. The largest tensile force acting on a diagonal is, $F_{t,d}$ = 1154 kN

The minimum required area for the diagonals can now be calculated with:

$$A_{\min} = \frac{F_{t,d}}{f_{y,d}} = \frac{1154 \cdot 10^3}{355} = 3251 mm^2$$

This requirement is met by applying square box-sections with A = 150 * 100 mm^2 and t = 8 mm. This gives a cross-section of, A = 3713 mm^2

This leads to a unity-check of:

$$\frac{F_{t,d}}{Af_{y,d}} = \frac{1154 \cdot 10^3}{3713 * 355} = 0,88 \leq 1$$

Therefore the box-section can withstand the loading.

10.4 Columns

Now the roofs and floors have been calculated all the columns supporting these parts can be calculated. The columns support the girders, which support the floor or roof. Therefore the loading caused by the self-weight of the roof, floor and girder and the loading caused by the variable loading form the loading onto the columns. The loads coming from the floor don’t act over the full length of the column, but have their loading point somewhere along the column, this makes it difficult to determine their exact effect when it comes to buckling of the columns. Therefore for the calculations of the columns it is assumed they do act on the top of the column. This is a safe approach, and therefore permited. The chosen material for the columns is concrete B35. This concrete has a compressive strength of, $f`_{b,d}$ = 21 N/mm^2.
To make it easier to refer to a column, they are placed at a grid, with a row and column number (See figure 10.32).

10.4.1 Columns supporting roof theatre 1 & 4

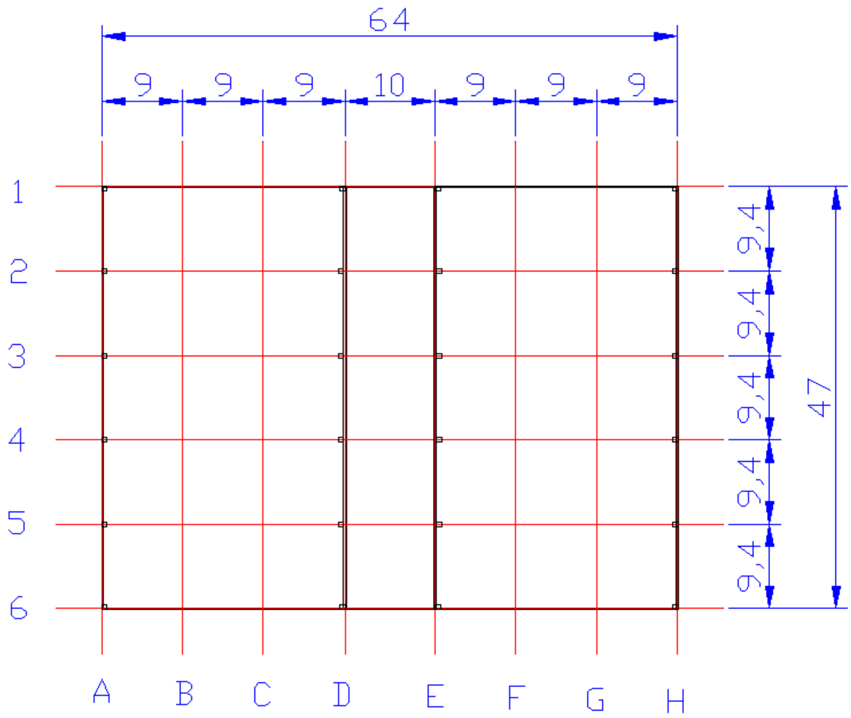


Figure 10.32: Lay-out.

Columns row 1 and 6 (A, D, E, H)
The columns in this row are loaded by:
Roof: ½ * ½ * 33,1 * 27 = 223 kN
Horizontal floor: ½ * ½ * 53,7 * 27 = 362 kN

Giving a total loading on the columns of row 1(or 6) of:
F_d = 223 + 362 = 585 kN

The minimum required area for this column is:
$A_{min} = F_d / f`_b = 585*10^3 / 21 = 27857 \text{ mm}^2$

The girder supporting the roof has a width of 500 mm, this width will also be used for the width of the column. To facilitate a good bearing a height of the cross-section of 300 mm is chosen.
However there must also be some space on the column to support the edge beam, with rules of thumb it is found that this beam will have the following dimensions:
h = 1/10 * l = 1/10 * 9000 = 900 mm
b = 1/3 * h = 1/3 * 900 = 300 mm

This means a cross-section of 500 * 600 mm is required.

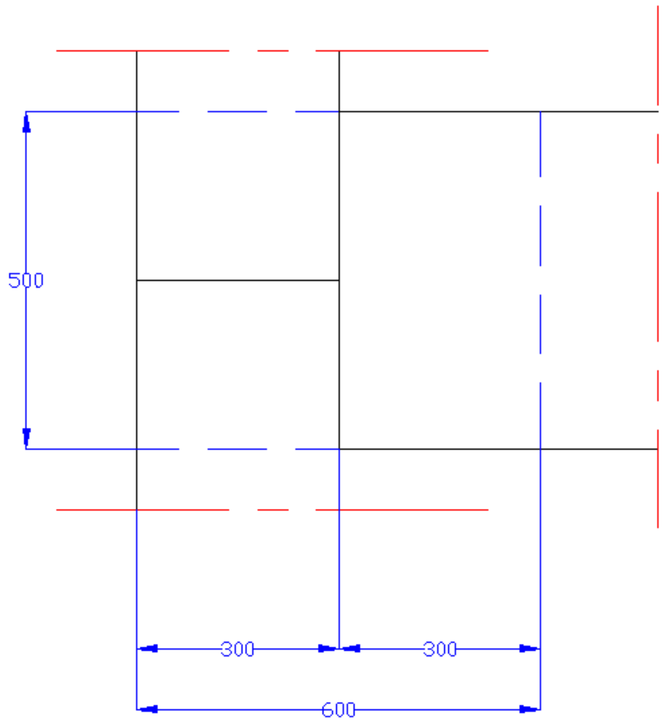


Figure 10.33: Top view column with side beams and truss.

The length of the column is determined by the height of the theatre which is required for the right viewing angles, this is 11,4 m.

Now the rebar for the columns can be calculated.

The normal force loading the column is 585 kN, however due to the shape of the column and the cross-beam the loading from the roof is an eccentric loading with an eccentricity of 150 mm. And the loading from the floor has an eccentricity of 150 mm. This means the column is also loaded with a bending moment of:
$M_d = 223*10^3 * 150 + 362*10^3 * 150 = 88$ kNm.

The required rebar can now be calculated using an interaction diagram (see appendix 10).

$$n_d = \frac{N_d}{bhf_b^`} = \frac{585 \cdot 10^3}{500 \cdot 600 \cdot 21} = 0{,}09$$

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{142 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}02$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0{,}04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}04 * 21/435 = 0{,}0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0019 * 500 * 600 = 570$ mm^2

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:

$$\alpha_n = \frac{N_d}{A_b f_b^{`} + A_s f_s} = \frac{585 \cdot 10^3}{(600 * 500 - 2 * 678)21 + (2 * 678)435} = 0{,}09$$

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For α_n < 0,25 it holds that the column does not have to be checked for second order effects if:

$$\lambda \leq \frac{5}{\sqrt{\alpha_n}} = \frac{5}{\sqrt{0{,}11}} = 15$$

This is not the case and therefore this column will have to be checked for second order effects.
The total eccentricity due to second order effects is defined as:

$$e_t = (e_0 + e_c)\xi \geq e_0$$

With:
e_0 = largest initial eccentricity

$$e_c = 3\big(1{,}5h + e_0(4\psi - 3)\big)\left(\frac{\rho l_c}{100h}\right)^2$$

This means for the roof loading:

$$= 3\big(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3)\big)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{75}{150}\right) = 0{,}75$$

This leads to a total of:
e_t = (150 + 114) * 0,75 = 198 mm

And for the floor loading:

$$= 3\big(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3)\big)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{75}{150}\right) = 0{,}75$$

This leads to a total of:
e_t = (150 + 114) * 0,75 = 198 mm

The new moment acting on the column now becomes:
$M_d = N_d * e_t = 223*10^3 * 198 + 362*10^3 * 198 = 116$ kNm

This leads to a new equivalent moment of:

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{116 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0,03$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0,04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0,04 * 21/435 = 0,0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0,0019 * 500 * 600 = 570$ mm^2

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

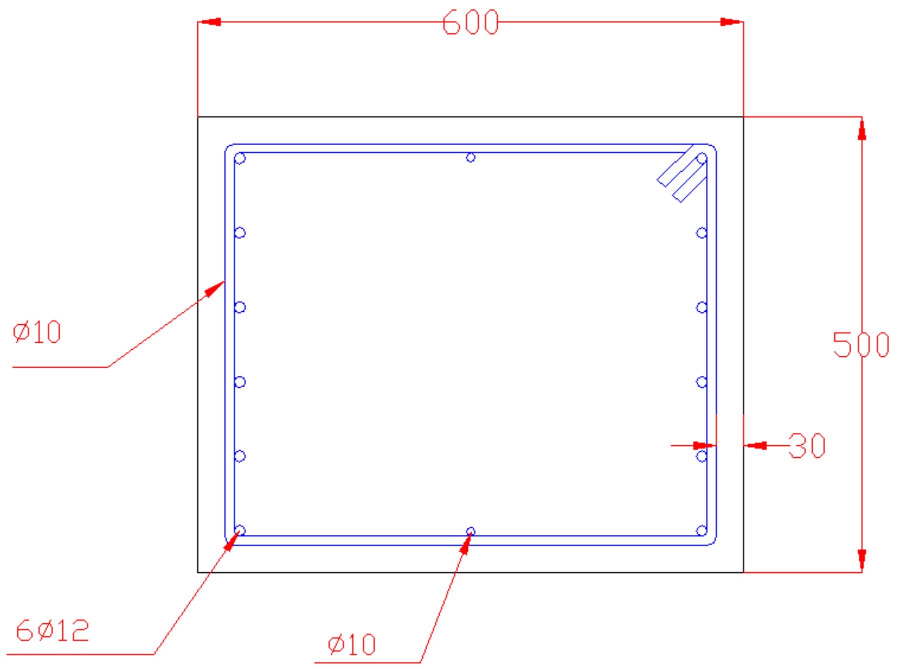


Figure 10.34: Cross-section of column with rebar.

Columns row 2 and 5 (A, D, E, H)

The columns in this row are loaded by:
Roof: ½ * 33,1 * 27 = 447 kN
Horizontal floor: ½ * ½ * 53,7 * 27 = 362 kN
Diagonal floor: 498 + ½ * ½ * 2,9 * 27 = 518 kN

Giving a total loading on the columns of row 2(or 5) of:
$F_d = 447 + 362 + 518 = 1327$ kN

The minimum required area for this column is:
$A_{min} = F_d / f`_b = 1327*10^3 / 21 = 63190\ mm^2$

The same size column as in row one is used, 500 * 600 mm.

The length of the column is determined by the height of the theatre which is required for the right viewing angles, this is 11,4 m.

Now the rebar for the columns can be calculated.

The normal force loading the column is 1327 kN, however due to the shape and cross-beam the loading from the roof is an eccentric loading with an eccentricity of 150 mm. And the loading from the floor has an eccentricity of 300 mm. This means the column is also loaded with a bending moment of:
$M_d = 447*10^3 * 150 + 362*10^3 * 300 + 518*10^3 * 300 = 331$ kNm.

The required rebar can now be calculated using an interaction diagram.

$$n_d = \frac{N_d}{bhf_b^`} = \frac{1327 \cdot 10^3}{500 \cdot 600 \cdot 21} = 0{,}21$$

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{331 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}09$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0{,}04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}04 * 21/435 = 0{,}0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0019 * 500 * 600 = 570\ mm^2$

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:

$$\alpha_n = \frac{N_d}{A_b f_b^` + A_s f_s} = \frac{1327 \cdot 10^3}{(600*500 - 2*678)21 + (2*678)435} = 0{,}19$$

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For $\alpha_n < 0{,}25$ it holds that the column does not have to be checked for second order effects if:

$$\lambda \leq \frac{5}{\sqrt{\alpha_n}} = \frac{5}{\sqrt{0{,}19}} = 11{,}5$$

This is not the case and therefore this column will have to be checked for second order effects.
The total eccentricity due to second order effects is defined as:
$e_t = (e_0 + e_c)\xi \geq e_0$
With:
e_0 = largest initial eccentricity
$e_c = 3\left(1{,}5h + e_0\left(4\psi - 3\right)\right)\left(\frac{\rho l_c}{100h}\right)^2$

This means for the roof loading:
$$= 3\left(1{,}5 \cdot 600 + 150\left(4 \cdot 1 - 3\right)\right)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$
$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{75}{150}\right) = 0{,}75$$

This leads to a total of:
e_t = (150 + 114) * 0,75 = 198 mm

And for the floor loading (horizontal floor):
$$= 3\left(1{,}5 \cdot 600 + 150\left(4 \cdot 1 - 3\right)\right)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{150}{300}\right) = 0{,}75$$

This leads to a total of:
e_t = (300 + 114) * 0,75 = 311 mm

And for the floor loading (diagonal floor):
$$= 3\left(1{,}5 \cdot 600 + 150\left(4 \cdot 1 - 3\right)\right)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{150}{300}\right) = 0{,}75$$

This leads to a total of:
e_t = (300 + 114) * 0,75 = 311 mm

The new moment acting on the column now becomes:
$M_d = N_d * e_t = 447*10^3 * 198 + 362*10^3 * 311 + 518*10^3 * 311 = 362$ kNm

This leads to a new equivalent moment of:

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{362 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0,1$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0,04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0,04 * 21/435 = 0,0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0,0019 * 500 * 600 = 570\ mm^2$

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

Columns row 3 and 4 (A, D, E, H)

The columns in this row are loaded by:
Roof: ½ * 33,1 * 27 = 447 kN
Diagonal floor: 498 + ½ * 2,9 * 27 = 537 kN

Giving a total loading on the columns of row 3(or 4) of:
$F_d = 447 + 537 = 984$ kN

The minimum required area for this column is:
$A_{min} = F_d / f`_b = 984*10^3 / 21 = 46857\ mm^2$

The same size column as in row one is used, 500 * 600 mm.

The length of the column is determined by the height of the theatre which is required for the right viewing angles, this is 11,4 m.

Now the rebar for the columns can be calculated.

The normal force loading the column is 984 kN, however due to the shape and cross-beam the loading from the roof is an eccentric loading with an eccentricity of 150 mm. And the loading from the floor has an eccentricity of 300 mm. This means the column is also loaded with a bending moment of:
$M_d = 447*10^3 * 150 + 537*10^3 * 300 = 228$ kNm.

The required rebar can now be calculated using an interaction diagram.

$$n_d = \frac{N_d}{bhf_b^`} = \frac{984 \cdot 10^3}{500 \cdot 600 \cdot 21} = 0,16$$

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{228 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0,06$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0,04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}04 * 21/435 = 0{,}0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0019 * 500 * 600 = 570\ mm^2$

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:

$$\alpha_n = \frac{N_d}{A_b f_b^` + A_s f_s} = \frac{984 \cdot 10^3}{(600 * 500 - 2 * 678)21 + (2 * 678)435} = 0{,}14$$

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For $\alpha_n < 0{,}25$ it holds that the column does not have to be checked for second order effects if:

$$\lambda \leq \frac{5}{\sqrt{\alpha_n}} = \frac{5}{\sqrt{0{,}14}} = 13{,}4$$

This is not the case and therefore this column will have to be checked for second order effects.
The total eccentricity due to second order effects is defined as:

$$e_t = (e_0 + e_c)\xi \geq e_0$$

With:
e_0 = largest initial eccentricity

$$e_c = 3\left(1{,}5h + e_0\left(4\psi - 3\right)\right)\left(\frac{\rho l_c}{100h}\right)^2$$

This means for the roof loading:

$$= 3\left(1{,}5 \cdot 600 + 150\left(4 \cdot 1 - 3\right)\right)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{75}{150}\right) = 0{,}75$$

This leads to a total of:
$e_t = (150 + 114) * 0{,}75 = 198$ mm

And for the floor loading (diagonal floor):

$$= 3\left(1{,}5 \cdot 600 + 150\left(4 \cdot 1 - 3\right)\right)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1+\frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1+\frac{150}{300}\right) = 0{,}75$$

This leads to a total of:
$e_t = (300 + 114) * 0{,}75 = 311$ mm

The new moment acting on the column now becomes:
$M_d = N_d * e_t = 447*10^3 * 198 + 537*10^3 * 311 = 256$ kNm

This leads to a new equivalent moment of:

$$m_d = \frac{M_d}{bh^2 f_b^{`}} = \frac{256 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}07$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0{,}04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}04 * 21/435 = 0{,}0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0019 * 500 * 600 = 570\ mm^2$

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

10.4.2 Columns supporting roof theatre 3 & 6 / terrace

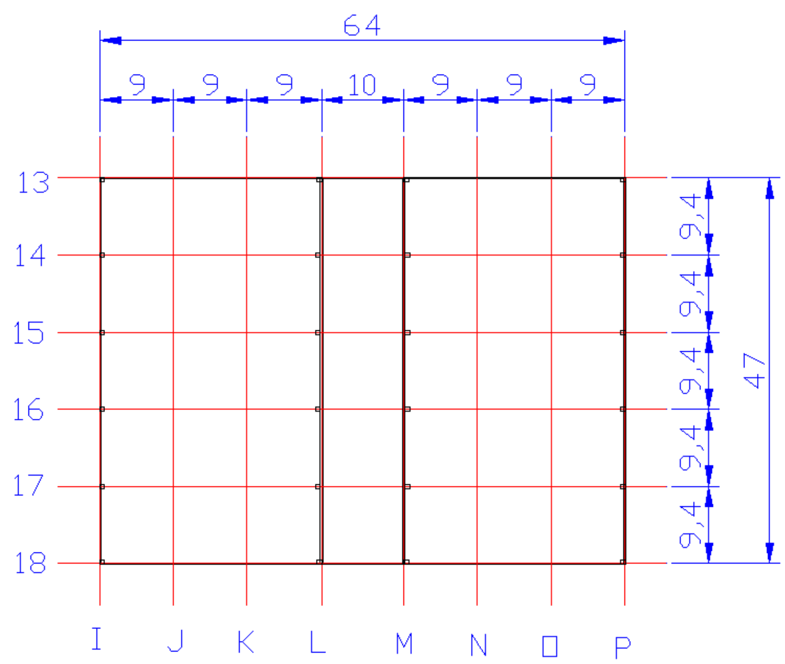


Figure 10.35: Lay-out.

Columns row 13 and 18 (I, L, M, P)
The columns in this row are loaded by:
Roof: ½ * ½ * 60,9 * 27 = 411 kN
Horizontal floor: ½ * ½ * 53,7 * 27 = 362 kN

Giving a total loading on the columns of row 13(or 18) of:
F_d = 411 + 362 = 773 kN

The minimum required area for this column is:
$A_{min}= F_d / f`_b = 773*10^3 / 21 = 36810\ mm^2$

The girder supporting the roof has a width of 500 mm, this width will also be used for the width of the column. To facilitate a good bearing a height of the cross-section of 300 mm is chosen.
However there must also be some space on the column to support the edge beam, with rules of thumb it is found that this beam will have the following dimensions:
h = 1/10 * l = 1/10 * 9000 = 900 mm
b = 1/3 * h = 1/3 * 900 = 300 mm

This means a cross-section of 500 * 600 mm is required.

The length of the column is determined by the height of the theatre which is required for the right viewing angles, this is 11,4 m.

Now the rebar for the columns can be calculated.

The normal force loading the column is 773 kN, however due to the shape and cross-beam the loading from the roof is an eccentric loading with an eccentricity of 150 mm. And the loading from the floor has an eccentricity of 300 mm. This means the column is also loaded with a bending moment of:
$M_d = 411*10^3 * 150 + 362*10^3 * 300$ = 115 kNm.

The required rebar can now be calculated using an interaction diagram.

$$n_d = \frac{N_d}{bhf_b^`} = \frac{773\cdot 10^3}{500\cdot 600\cdot 21} = 0{,}12$$

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{115\cdot 10^6}{500\cdot 600^2\cdot 21} = 0{,}03$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0{,}03$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}03 * 21/435 = 0{,}0014$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0014 * 500 * 600 = 420\ mm^2$

With on both sides 6 bars with d = 10 mm an area of 471 mm^2 on each side is found.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:

$$\alpha_n = \frac{N_d}{A_b f_b^{`} + A_s f_s} = \frac{773 \cdot 10^3}{(600*500-2*420)21+(2*420)435} = 0{,}12$$

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For α_n < 0,25 it holds that the column does not have to be checked for second order effects if:

$$\lambda \leq \frac{5}{\sqrt{\alpha_n}} = \frac{5}{\sqrt{0{,}12}} = 14$$

This is not the case and therefore this column will have to be checked for second order effects.
The total eccentricity due to second order effects is defined as:

$$e_t = (e_0 + e_c)\xi \geq e_0$$

With:
e_0 = largest initial eccentricity

$$e_c = 3(1{,}5h + e_0(4\psi - 3))\left(\frac{\rho l_c}{100h}\right)^2$$

This means for the roof loading:

$$= 3(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100*600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{75}{150}\right) = 0{,}75$$

This leads to a total of:
e_t = (150 + 114) * 0,75 = 198 mm

And for the floor loading:

$$= 3(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100*600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{150}{300}\right) = 0{,}75$$

This leads to a total of:
e_t = (300 + 114) * 0,75 = 311 mm

The new moment acting on the column now becomes:
$M_d = N_d * e_t = 411*10^3 * 198 + 362*10^3 * 311 = 194$ kNm

This leads to a new equivalent moment of:

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{194 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}05$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0{,}04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}04 * 21/435 = 0{,}0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0019 * 500 * 600 = 570\ mm^2$

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

Columns row 14 and 17 (I, L, M, P)

The columns in this row are loaded by:
Roof: ½ * 60,9 * 27 = 822 kN
Horizontal floor: ½ * ½ * 53,7 * 27 = 362 kN
Diagonal floor: 498 + ½ * ½ * 2,9 * 27 = 518 kN

Giving a total loading on the columns of row 14(or 17) of:
$F_d = 822 + 362 + 518 = 1702$ kN

The minimum required area for this column is:
$A_{min} = F_d / f`_b = 1702*10^3 / 21 = 81048\ mm^2$

The same size column as in row one is used, 500 * 600 mm.

The length of the column is determined by the height of the theatre which is required for the right viewing angles, this is 11,4 m.

Now the rebar for the columns can be calculated.

The normal force loading the column is 1702 kN, however due to the shape and cross-beam the loading from the roof is an eccentric loading with an eccentricity of 150 mm. And the loading from the floor has an eccentricity of 300 mm. This means the column is also loaded with a bending moment of:
$M_d = 822*10^3 * 150 + 362*10^3 * 300 + 518*10^3 * 300 = 387$ kNm.

The required rebar can now be calculated using an interaction diagram.

$$n_d = \frac{N_d}{bhf_b^`} = \frac{1702 \cdot 10^3}{500 \cdot 600 \cdot 21} = 0{,}27$$

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{387 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}10$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0{,}04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}04 * 21/435 = 0{,}0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0019 * 500 * 600 = 570\ mm^2$

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:

$$\alpha_n = \frac{N_d}{A_b f_b^` + A_s f_s} = \frac{1702 \cdot 10^3}{(600*500 - 2*678)21 + (2*678)435} = 0{,}25$$

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For $\alpha_n < 0{,}25$ it holds that the column does not have to be checked for second order effects if:

$$\lambda \le \frac{5}{\sqrt{\alpha_n}} = \frac{5}{\sqrt{0{,}25}} = 10$$

This is not the case and therefore this column will have to be checked for second order effects.
The total eccentricity due to second order effects is defined as:

$$e_t = (e_0 + e_c)\xi \ge e_0$$

With:
e_0 = largest initial eccentricity

$$e_c = 3(1{,}5h + e_0(4\psi - 3))\left(\frac{\rho l_c}{100h}\right)^2$$

This means for the roof loading:

$$= 3(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100*600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \ge 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{75}{150}\right) = 0{,}75$$

This leads to a total of:
e_t = (150 + 114) * 0,75 = 198 mm

And for the floor loading (horizontal floor):

$$= 3(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{150}{300}\right) = 0{,}75$$

This leads to a total of:
e_t = (300 + 114) * 0,75 = 311 mm

And for the floor loading (diagonal floor):

$$= 3(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{150}{300}\right) = 0{,}75$$

This leads to a total of:
e_t = (300 + 114) * 0,75 = 311 mm

The new moment acting on the column now becomes:
$M_d = N_d * e_t = 822*10^3 * 198 + 362*10^3 * 311 + 518*10^3 * 311 = 436$ kNm

This leads to a new equivalent moment of:

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{436 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}12$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0{,}04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}04 * 21/435 = 0{,}0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0019 * 500 * 600 = 570$ mm^2

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

Columns row 15 and 16 (I, L, M, P)

The columns in this row are loaded by:
Roof: ½ * 60,9 * 27 = 822 kN
Diagonal floor: 498 + ½ * 2,9 * 27 = 537 kN

Giving a total loading on the columns of row 15(or 16) of:
F_d = 822 + 537 = 1359 kN

The minimum required area for this column is:
$A_{min} = F_d / f`_b = 1359*10^3 / 21 = 64714\ mm^2$

The same size column as in row one is used, 500 * 600 mm.

The length of the column is determined by the height of the theatre which is required for the right viewing angles, this is 11,4 m.

Now the rebar for the columns can be calculated.

The normal force loading the column is 1359 kN, however due to the shape and cross-beam the loading from the roof is an eccentric loading with an eccentricity of 150 mm. And the loading from the floor has an eccentricity of 300 mm. This means the column is also loaded with a bending moment of:
$M_d = 822*10^3 * 150 + 537*10^3 * 300 = 284\ kNm.$

The required rebar can now be calculated using an interaction diagram.

$$n_d = \frac{N_d}{bhf_b^`} = \frac{1359 \cdot 10^3}{500 \cdot 600 \cdot 21} = 0{,}22$$

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{284 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}08$$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0{,}04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}04 * 21/435 = 0{,}0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0019 * 500 * 600 = 570\ mm^2$

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:

$$\alpha_n = \frac{N_d}{A_b f_b^` + A_s f_s} = \frac{1359 \cdot 10^3}{(600*500 - 2*678)21 + (2*678)435} = 0{,}20$$

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For $\alpha_n < 0{,}25$ it holds that the column does not have to be checked for second order effects if:

$$\lambda \leq \frac{5}{\sqrt{\alpha_n}} = \frac{5}{\sqrt{0{,}20}} = 11{,}2$$

This is not the case and therefore this column will have to be checked for second order effects.
The total eccentricity due to second order effects is defined as:
$e_t = (e_0 + e_c)\xi \geq e_0$
With:
e_0 = largest initial eccentricity
$e_c = 3(1{,}5h + e_0(4\psi - 3))\left(\frac{\rho l_c}{100h}\right)^2$

This means for the roof loading:
$= 3(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$
$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{75}{150}\right) = 0{,}75$

This leads to a total of:
e_t = (150 + 114) * 0,75 = 198 mm

And for the floor loading (diagonal floor):
$= 3(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$

$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{150}{300}\right) = 0{,}75$

This leads to a total of:
e_t = (300 + 114) * 0,75 = 311 mm

The new moment acting on the column now becomes:
$M_d = N_d * e_t = 822*10^3 * 198 + 537*10^3 * 311$ = 330 kNm

This leads to a new equivalent moment of:
$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{330 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}09$

According to the interaction diagram this leads to:
$\psi_1 = \psi_2 = 0{,}04$

This leads to:
$\omega_1 = \omega_2 = \psi * f`_b/f_s = 0{,}04 * 21/435 = 0{,}0019$

This leads to a minimum required area for the rebar of:
$A_{s1} = A_{s2} = \omega bh = 0{,}0019 * 500 * 600 = 570\ mm^2$

With on both sides 6 bars with d = 12 mm an area of 678 mm^2 on each side is found.

10.4.3 Columns supporting roof theatre 2 & 5

Now the columns supporting theatre 2 and 5 can be calculated. The loading on these columns comes from the floors, from the wall elements of the theatres above and from the columns on top of these columns. The loadings caused by the floor are the same as in the other theatres, and they cause a normal compressive force and a bending moment in the column. The loading from the columns above causes a normal compressive force in the columns.

The wall elements are not a part of the main bearing construction. Therefore the design of these elements is outside the scope of this research. However they do cause a loading onto the main bearing construction, therefore an assumption is made to make it possible to calculate the columns.
It is assumed a concrete wall is placed with $\rho = 25$ kN/m^3. The wall thickness is assumed at 20 cm and the height is 15,0 m. With this the loading on the edge beams can be calculated: $q_{wall} = 25 * 0,2 * 15 = 75$ kN/m.

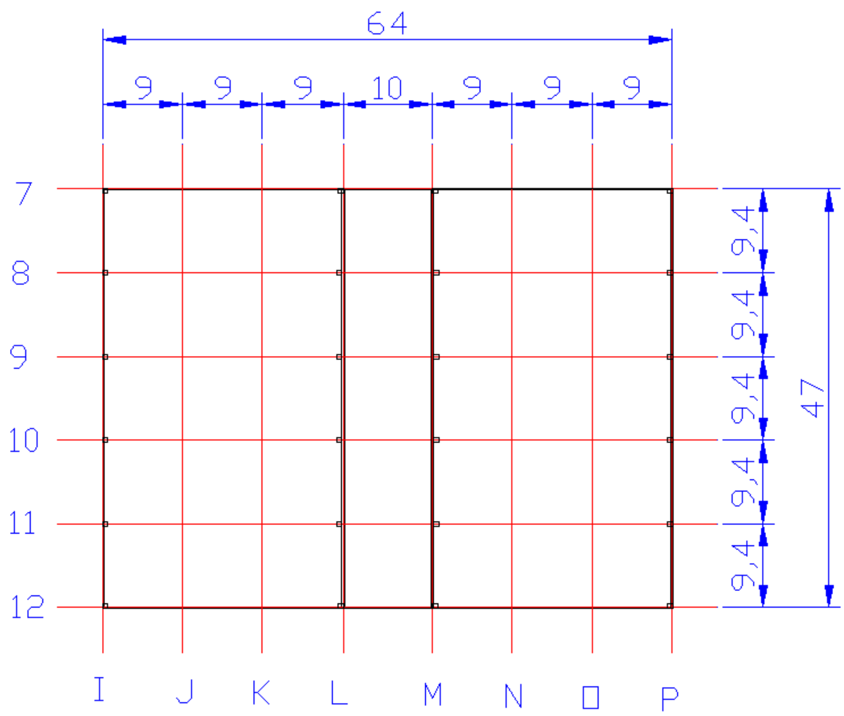


Figure 10.36: Lay-out.

Columns row 7 and 12 (I, L, M, P)
The columns in this row are loaded by:
Horizontal floor: ½ * ½ * 53,7 * 27 = 362 kN
Wall 1: ½ * ½ * 75 * 9,4 = 176 kN (H)
Wall 2: ½ * ½ * 75 * 9,0 = 169 kN (1)
Column above: 585 kN

Giving a total loading on the columns of row 7(or 12) of:
$F_d = 362 + 176 + 169 + 585 = 1292$ kN

The minimum required area for this column is:
$A_{min}= F_d / f`_b = 1292*10^3 / 21 = 61524\ mm^2$

The girder supporting the roof has a width of 500 mm, this width will also be used for the width of the column. To facilitate a good bearing a height of the cross-section of 300 mm is chosen.
However there must also be some space on the column to support the edge beam, with rules of thumb it is found that this beam will have the following dimensions:
h = 1/10 * l = 1/10 * 9000 = 900 mm
b = 1/3 * h = 1/3 * 900 = 300 mm

This means a cross-section of 500 * 600 mm is required.

The length of the column is determined by the height of the theatre which is required for the right viewing angles, this is 11,4 m.

Now the rebar for the columns can be calculated.

The normal force loading the column is 1292 kN, however due to the shape of the column, not all the loadings are centric loadings. The loading caused by the side wall (H) has an eccentricity of 150 mm in y-direction. The same holds for the back wall (1), only in the z- and y-direction. The floor loading has an eccentricity of 300 mm in y-direction, but opposite direction. The loading from the column above is a centric loading. This means the column is also loaded with two bending moments of:
$M_{y,d} = 362*10^3 * 300 - 176*10^3 * 150 + 169*10^3 * 150 = 108\ kNm$
$M_{z,d} = 169*10^3 * 150 = 25\ kNm$

The column will first be calculated in y-direction

The required rebar can now be calculated using an interaction diagram.

$$n_d = \frac{N_d}{bhf_b^`} = \frac{1292 \cdot 10^3}{500 \cdot 600 \cdot 21} = 0{,}21$$
$$m_d = \frac{M_{y,d}}{bh^2 f_b^`} = \frac{108 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}03$$

According to the interaction diagram this means the column is strong enough without any rebar, however a minimum amount is required according to TGB.

With on both sides 6 bars with d = 10 mm an area of 471 mm^2 on each side is found. This should be sufficient.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:
$$\alpha_n = \frac{N_d}{A_b f_b^` + A_s f_s} = \frac{1292 \cdot 10^3}{(600*500 - 2*471)21 + (2*471)435} = 0{,}19$$

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For $\alpha_n < 0{,}25$ it holds that the column does not have to be checked for second order effects if:

$$\lambda \leq \frac{5}{\sqrt{\alpha_n}} = \frac{5}{\sqrt{0{,}19}} = 11{,}5$$

This is not the case and therefore this column will have to be checked for second order effects.

The total eccentricity due to second order effects is defined as:

$$e_t = (e_0 + e_c)\xi \geq e_0$$

With:

e_0 = largest initial eccentricity

$$e_c = 3\left(1{,}5h + e_0\left(4\psi - 3\right)\right)\left(\frac{\rho l_c}{100h}\right)^2$$

This means for the wall loading:

$$= 3\left(1{,}5 \cdot 600 + 150\left(4 \cdot 1 - 3\right)\right)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{75}{150}\right) = 0{,}75$$

This leads to a total of:

e_t = (150 + 114) * 0,75 = 198 mm

The new moment acting on the column now becomes:

$M_{z,d} = N_d * e_t = 362*10^3 * 311 - 176*10^3 * 198 + 169*10^3 * 198 = 111$ kNm

This leads to a new equivalent moment of:

$$m_d = \frac{M_d}{bh^2 f_b} = \frac{111 \cdot 10^6}{600 \cdot 500^2 \cdot 21} = 0{,}03$$

According to the interaction diagram this means no rebar is required. Therefore the minimum amount is used.

With on both sides 6 bars with d = 10 mm an area of 471 mm^2 on each side is found.

The column will now be calculated in z-direction

The required rebar can now be calculated using an interaction diagram.

$$n_d = \frac{N_d}{bhf_b^`} = \frac{1292 \cdot 10^3}{600 \cdot 500 \cdot 21} = 0,21$$
$$m_d = \frac{M_{z,d}}{bh^2 f_b^`} = \frac{25 \cdot 10^6}{600 \cdot 500^2 \cdot 21} = 0,01$$
According to the interaction diagram this means the column is strong enough without any rebar, however a minimum amount is required according to TGB.

With on both sides 6 bars with d = 10 mm an area of 471 mm^2 on each side is found. This should be sufficient.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:
$$\alpha_n = \frac{N_d}{A_b f_b^` + A_s f_s} = \frac{1292 \cdot 10^3}{(600 * 500 - 2 * 471)21 + (2 * 471)435} = 0,19$$

The slenderness of the column is:
$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For $\alpha_n < 0,25$ it holds that the column does not have to be checked for second order effects if:
$$\lambda \leq \frac{5}{\sqrt{\alpha_n}} = \frac{5}{\sqrt{0,19}} = 11,5$$

This is not the case and therefore this column will have to be checked for second order effects.
The total eccentricity due to second order effects is defined as:
$$e_t = (e_0 + e_c)\xi \geq e_0$$
With:
e_0 = largest initial eccentricity
$$e_c = 3(1,5h + e_0(4\psi - 3))\left(\frac{\rho l_c}{100h}\right)^2$$

This means for the wall loading:
$$= 3(1,5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$
$$\xi = 0,5\left(1 + \frac{e_1}{e_0}\right) \geq 0,75 \Rightarrow 0,5\left(1 + \frac{75}{150}\right) = 0,75$$

This leads to a total of:
e_t = (150 + 114) * 0,75 = 198 mm

The new moment acting on the column now becomes:

$M_{y,d} = N_d * e_t = 169*10^3 * 198 = 33$ kNm

This leads to a new equivalent moment of:

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{33 \cdot 10^6}{600 \cdot 500^2 \cdot 21} = 0{,}01$$

According to the interaction diagram this means no rebar is required. Therefore the minimum amount is used.
With on both sides 6 bars with d = 10 mm an area of 471 mm^2 on each side is found.

Columns row 8 and 11 (I, L, M, P)

The columns in this row are loaded by:
Wall: ½ * 75 * 9,4 = 353 kN
Horizontal floor: ½ * ½ * 53,7 * 27 = 362 kN
Diagonal floor: 498 + ½ * ½ * 2,9 * 27 = 518 kN
Column: 1327 kN

Giving a total loading on the columns of row 8(or 11) of:
$F_d = 353 + 362 + 518 + 1327 = 2559$ kN

The minimum required area for this column is:
$A_{min} = F_d / f`_b = 2559*10^3 / 21 = 121857$ mm^2

The same size column as in row one is used, 500 * 600 mm.

The length of the column is determined by the height of the theatre which is required for the right viewing angles, this is 11,4 m.

Now the rebar for the columns can be calculated.

The normal force loading the column is 2559 kN, however due to the shape of the column, not all the loadings are centric loadings. The loading caused by the wall has an eccentricity of 150 mm in y-direction. The floor loading has an eccentricity of 300 mm in y-direction, but opposite direction. The loading caused by the column above is a centric loading. This means the column is also loaded with a bending moment of:
$M_d = -353*10^3 * 150 + 362*10^3 * 300 + 518*10^3 * 300 = 211$ kNm.

The required rebar can now be calculated using an interaction diagram.

$$n_d = \frac{N_d}{bhf_b^`} = \frac{2559 \cdot 10^3}{500 \cdot 600 \cdot 21} = 0{,}41$$

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{211 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}06$$

According to the interaction diagram this means no rebar is required. Therefore the minimum amount is used.
With on both sides 6 bars with d = 10 mm an area of 471 mm^2 on each side is found.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:

$$\alpha_n = \frac{N_d}{A_b f_b^` + A_s f_s} = \frac{2559 \cdot 10^3}{(600 * 500 - 2 * 471)21 + (2 * 471)435} = 0,38$$

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For 0,25 < α_n < 0,5 it holds that the column does not have to be checked for second order effects if:

$$\lambda \leq 10$$

This is not the case and therefore this column will have to be checked for second order effects.
The total eccentricity due to second order effects is defined as:

$$e_t = (e_0 + e_c)\xi \geq e_0$$

With:
e_0 = largest initial eccentricity

$$e_c = 3(1,5h + e_0(4\psi - 3))\left(\frac{\rho l_c}{100h}\right)^2$$

This means for the wall loading:

$$= 3(1,5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0,5\left(1 + \frac{e_1}{e_0}\right) \geq 0,75 \Rightarrow 0,5\left(1 + \frac{75}{150}\right) = 0,75$$

This leads to a total of:
e_t = (150 + 114) * 0,75 = 198 mm

And for the floor loading (horizontal floor):

$$= 3(1,5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0,5\left(1 + \frac{e_1}{e_0}\right) \geq 0,75 \Rightarrow 0,5\left(1 + \frac{150}{300}\right) = 0,75$$

This leads to a total of:
e_t = (300 + 114) * 0,75 = 311 mm

And for the floor loading (diagonal floor):

$$= 3(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3))\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{150}{300}\right) = 0{,}75$$

This leads to a total of:
e_t = (300 + 114) * 0,75 = 311 mm

The new moment acting on the column now becomes:
$M_d = N_d * e_t = -353*10^3 * 198 + 362*10^3 * 311 + 518*10^3 * 311 = 204$ kNm

This leads to a new equivalent moment of:

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{204 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}05$$

According to the interaction diagram this means no rebar is required. Therefore the minimum amount is used.
With on both sides 6 bars with d = 10 mm an area of 471 mm^2 on each side is found.

Columns row 9 and 10 (I, L, M, P)

The columns in this row are loaded by:
Wall: ½ * 75 * 9,4 = 353 kN
Diagonal floor: 498 + ½ * 2,9 * 27 = 537 kN
Column: 984 kN

Giving a total loading on the columns of row 9(or 10) of:
F_d = 353 + 537 + 984 = 1874 kN

The minimum required area for this column is:
$A_{min} = F_d / f`_b = 1874*10^3 / 21 = 89238\ mm^2$

The same size column as in row one is used, 500 * 600 mm.

The length of the column is determined by the height of the theatre which is required for the right viewing angles, this is 11,4 m.

Now the rebar for the columns can be calculated.

The normal force loading the column is 1874 kN, however due to the shape of the column, not all the loadings are centric loadings. The loading caused by the wall has an eccentricity of 150 mm in y-direction. The floor loading has an eccentricity of 300 mm in y-direction, but opposite direction. The loading caused by the column above is a centric loading. This means the column is also loaded with a bending moment of:
$M_d = -353*10^3 * 150 + 537*10^3 * 300 = 108$ kNm.

The required rebar can now be calculated using an interaction diagram.

$$n_d = \frac{N_d}{bhf_b^`} = \frac{1874 \cdot 10^3}{500 \cdot 600 \cdot 21} = 0{,}30$$

$$m_d = \frac{M_d}{bh^2 f_b^`} = \frac{108 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}03$$

According to the interaction diagram this means no rebar is required. Therefore the minimum amount is used.
With on both sides 6 bars with d = 10 mm an area of 471 mm^2 on each side is found.

There also has to be checked whether or not the column has to be checked for second order effects.

It holds:

$$\alpha_n = \frac{N_d}{A_b f_b^` + A_s f_s} = \frac{1874 \cdot 10^3}{(600 * 500 - 2 * 471)21 + (2 * 471)435} = 0{,}28$$

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{11400}{600} = 19$$

For 0,25 < α_n < 0,5 it holds that the column does not have to be checked for second order effects if:

$$\lambda \leq 10$$

This is not the case and therefore this column will have to be checked for second order effects.
The total eccentricity due to second order effects is defined as:

$$e_t = (e_0 + e_c)\xi \geq e_0$$

With:
e_0 = largest initial eccentricity

$$e_c = 3\left(1{,}5h + e_0\left(4\psi - 3\right)\right)\left(\frac{\rho l_c}{100h}\right)^2$$

This means for the wall loading:

$$= 3\left(1{,}5 \cdot 600 + 150\left(4 \cdot 1 - 3\right)\right)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{75}{150}\right) = 0{,}75$$

This leads to a total of:
e_t = (150 + 114) * 0,75 = 198 mm

And for the floor loading (diagonal floor):

$$= 3\left(1{,}5 \cdot 600 + 150(4 \cdot 1 - 3)\right)\left(\frac{1 \cdot 11400}{100 * 600}\right)^2 = 114mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow 0{,}5\left(1 + \frac{150}{300}\right) = 0{,}75$$

This leads to a total of:
e_t = (300 + 114) * 0,75 = 311 mm

The new moment acting on the column now becomes:
$M_d = N_d * e_t = -353*10^3 * 198 + 537*10^3 * 311 = 97$ kNm

This leads to a new equivalent moment of:

$$m_d = \frac{M_d}{bh^2 f_b^{`}} = \frac{97 \cdot 10^6}{500 \cdot 600^2 \cdot 21} = 0{,}03$$

According to the interaction diagram this means no rebar is required. Therefore the minimum amount is used.
With on both sides 6 bars with d = 10 mm an area of 471 mm^2 on each side is found.

10.5 Horizontal loading

Besides vertical loadings, there is also a horizontal loading acting on the building. This load is caused by the wind.
For the wind load it holds:
$F_{wind} = A * p_{rep}$

With:
A = area [m]
$p_{rep} = C_{dim} * C_{index} * C_{eq} * \varphi_1 * p_w$ [N/mm^2]

With:
C_{dim} = factor for building dimensions = 1
$C_{index} = C_d = 0{,}8$ or $= C_z = 0{,}4$
C_{eq} = form factor = 1
φ_1 = factor for dynamic effects = 1
p_w = predefined wind pressure, depending on altitude and location in predefined area = 1,12 kN/m^2

This means p_{rep} is:
$p_{rep,d}$ = 1 * 0,8 * 1 * 1 * 1,12 = 0,90 kN/m^2
$p_{rep,z}$ = 1 * 0,4 * 1 * 1 * 1,12 = 0,45 kN/m^2

For stabilising this building, the two concrete walls, also used as anchors for the cable-stay bridge are used. By applying the same walls to connect these two walls a stiff core formed, which should withstand the horizontal loadings.

The building has to be checked into two directions, one with the wind direction perpendicular to the front of the building and one with wind perpendicular to the side of the building.

Wind perpendicular to the front

The piles under the wall are placed at a c.o.c. distance of 9,4 m. They have a length of 8,0 m and a cross-section of 500 * 500 mm^2.
The wall has the following characteristics:
l = 30 m
b = 94 m
d = 0,2 m

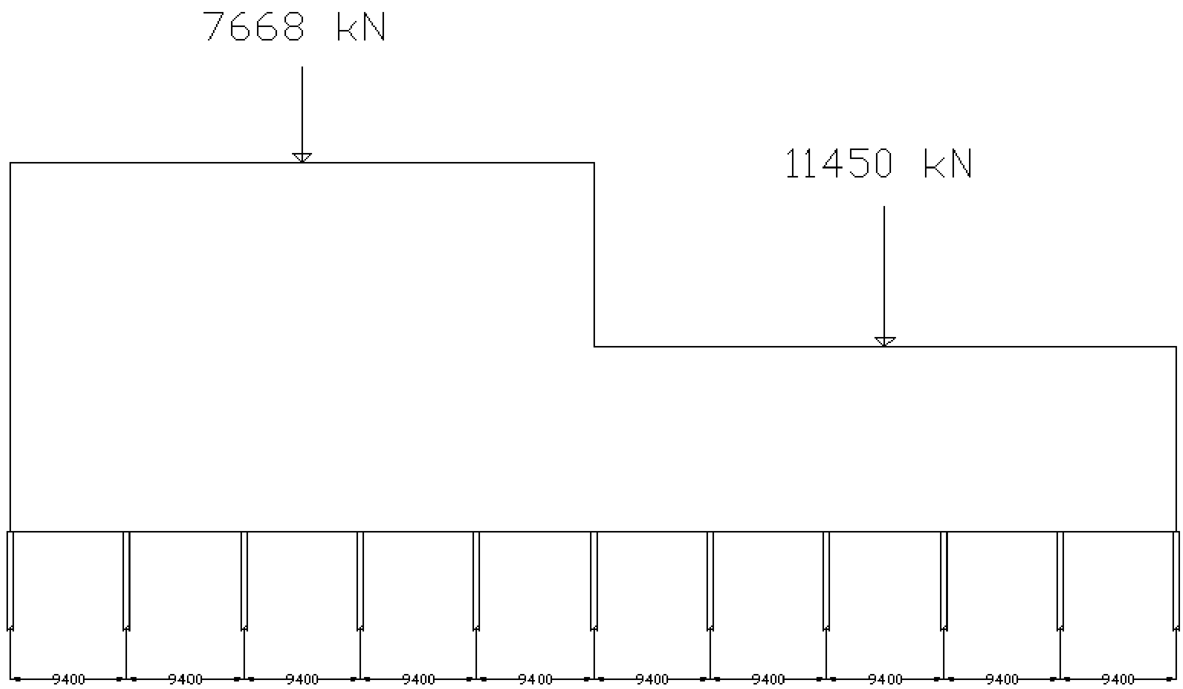


Figure 10.37: Schematisation wall with piles and self-weight.

The total loading on the wall is 19118 kN and the loading on the structure as a whole is 38236 kN.
The moment caused by the wind is, M_w = (0,9 + 0,45) * 64 * 30 * 15 = 38880 kNm
The moment caused by eccentricity of the loading is, M_e = (11450 – 7668) * 47 * ½ = 88877 kNm.

The moment of inertia of the pile group is:
$I_p = 2(9{,}4^2 + 18{,}8^2 + 28{,}2^2 + 37{,}6^2 + 47^2) * 0{,}5 * 0{,}5 = 2430\ m^4$

This means the loading on the furthest pile is:

$$P_n = \frac{M}{I_p} e = \frac{88877}{2430} \cdot 47 = 1719 kN$$

The deflection now becomes:

$$\Delta l = \frac{P_n \cdot 1{,}5 l_p}{E_p A_p} = \frac{1719 \cdot 10^3 \cdot 1{,}5 \cdot 8000}{31000 \cdot 500 \cdot 500} = 2{,}66 mm$$

The rotation is:

$$\varphi = \frac{2{,}66}{47000}$$

The stiffness now becomes:

$$C = \frac{M}{\varphi} = 1{,}57 \cdot 10^{9}\, kNm / rad$$

For the wall it holds:
I_w = 1/12 * 200 = 16,67 m^4
E_w = 31*10^6 kN/m^2
I_wE_w = 517*10^6 kNm^2

The whole building can be schematised as:

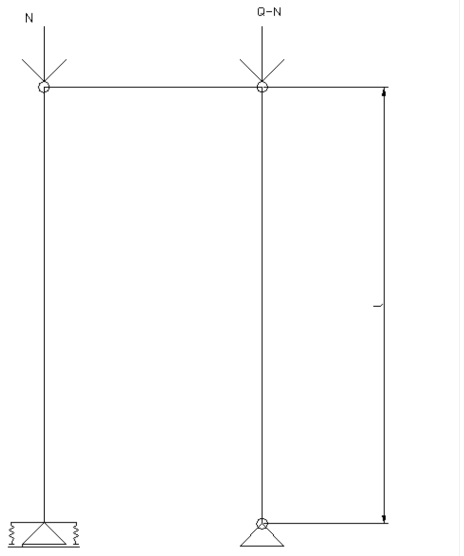


Figure 10.38: Schematisation building.

The Eulerse buckling force now becomes:

$$F_E = \frac{\pi^2 EI}{(1{,}12l)^2} + \frac{2C}{l^2} = 1126 \cdot 10^5\, kN$$

The maximum pile force can now be calculated:

$$M_t = \frac{n}{n-1}(M_w + M_e) = 127800 kNm$$

$$F_{\max} = -\frac{19118}{11} - \frac{127800 \cdot 47}{2430} = -4210 kN$$

This means the maximum stress in the pile will be:

$$\sigma_{\max} = \frac{F_{\max}}{A} = \frac{4210 * 10^3}{500 * 500} = 16{,}84 N / mm^2$$

Wind perpendicular to the side

The piles under the wall are placed at a c.o.c. distance of 2,0 m. They have a length of 8,0 m and a cross-section of 500 * 500 mm^2.
The wall has the following characteristics:
l = 30 m
b = 0,2 m
d = 94 m

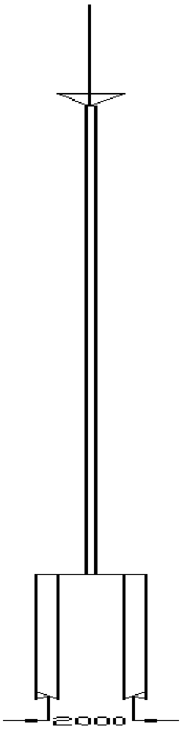


Figure 10.39: Schematisation wall with piles and self-weight.

The total loading on the wall is 19118 kN and the loading on the structure as a whole is 38236 kN.
The moment caused by the wind is, M_w = (0,9 + 0,45) * 94 * 30 * 15 = 57105 kNm
The moment caused by eccentricity of the loading is, M_e = 38236 * 0,6 = 22942 kNm.

The moment of inertia of the pile group is:
$I_p = 2 * 1^2 * 0,5 * 0,5 * 11 = 5,5\ m^4$

This means the loading on the furthest pile is:

$$P_n = \frac{M}{I_p} e = \frac{22942}{5,5} \cdot 1 = 4171 kN$$

The deflection now becomes:

$$\Delta l = \frac{P_n \cdot 1,5 l_p}{E_p A_p} = \frac{4171 \cdot 10^3 \cdot 1,5 \cdot 8000}{31000 \cdot 500 \cdot 500} = 6,46 mm$$

The rotation is:

$$\varphi = \frac{6,46}{1000}$$

The stiffness now becomes:

$$C = \frac{M}{\varphi} = 7{,}1 \cdot 10^6\, kNm/rad$$

For the wall it holds:
I_w = 1/12 * 94000 = 7833 m^4
E_w = 31*10^6 kN/m^2
I_wE_w = 243*10^9 kNm^2

The whole building can be schematised as:

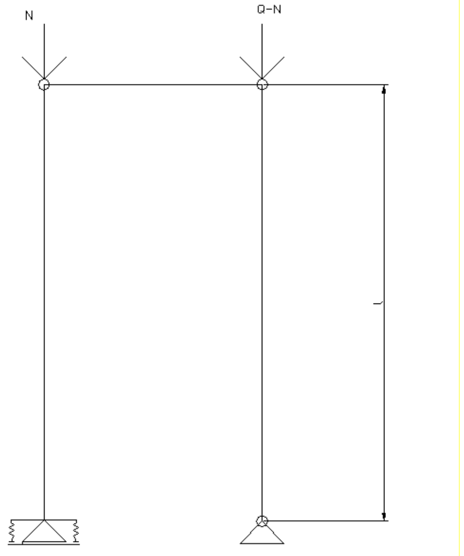


Figure 10.40: Schematisation building

The Eulerse buckling force now becomes:

$$F_E = \frac{\pi^2 EI}{(1{,}12l)^2} + \frac{2C}{l^2} = 29{,}9 \cdot 10^9\, kN$$

The maximum pile force can now be calculated:

$$M_t = \frac{n}{n-1}(M_w + M_e) = 80047 kNm$$

$$F_{\max} = -\frac{19118}{22} - \frac{80047 \cdot 1}{5{,}5} = -15423 kN$$

This means the maximum stress in the pile will be:

$$\sigma_{\max} = \frac{F_{\max}}{A} = \frac{15423 * 10^3}{500 * 500} = 61{,}7 N/mm^2$$

This means the pile can not withstand the loading, therefore its dimensions will be adjusted to 750 * 750 mm^2.

The moment of inertia for the pile group now becomes:
I_p = 12,4 m^4

This leads to a pile force of:

$$P_n = \frac{M}{I_p} e = \frac{22942}{12{,}4} \cdot 1 = 1850 kN$$

The deflection now becomes:

$$\Delta l = \frac{P_n \cdot 1{,}5 l_p}{E_p A_p} = \frac{1850 \cdot 10^3 \cdot 1{,}5 \cdot 8000}{31000 \cdot 750 \cdot 750} = 1{,}27 mm$$

The rotation is:

$$\varphi = \frac{1{,}27}{1000}$$

The stiffness now becomes:

$$C = \frac{M}{\varphi} = 18{,}1^6 \, kNm / rad$$

The Eulerse buckling force now becomes:

$$F_E = \frac{\pi^2 EI}{(1{,}12l)^2} + \frac{2C}{l^2} = 29{,}9 \cdot 10^9 \, kN$$

The maximum pile force can now be calculated:

$$M_t = \frac{n}{n-1}(M_w + M_e) = 80047 kNm$$

$$F_{\max} = -\frac{19118}{22} - \frac{80047 \cdot 1}{12{,}4} = -7324 kN$$

This means the maximum stress in the pile will be:

$$\sigma_{\max} = \frac{F_{\max}}{A} = \frac{7324 * 10^3}{750 * 750} = 13{,}0 N / mm^2$$

Now the piles can withstand the loading. Therefore the piles which will be applied will have a dimension of 750 * 750 mm^2

10.6 Parking garage

Under the theatre a new parking garage will be placed, this parking garage is both for visitors of the theatre and for tourists visiting the cities.
The ceiling of the parking garage is formed by the floor of the theatre. These floors are supported by trusses, the same as used to support the top floor of the theatre, no calculations are required due to the fact that both loading and span are the same as in the case of the top floor.
To support the trusses columns are used. The amount of columns must be minimized to create as much parking space as possible. However some are required to support the theatre above. By placing the columns directly under the columns above, the most ideal bearing construction is created.

The loading on a column comes from:

- Normal force in column above (normal force in column)
- Weight of floor (normal force in truss)
- Weight of truss (normal force + bending moment in column)
- Variable loading on the floor (normal force in truss)

The length of a column is determined by the height of the parking garage, which is 2,5 m.

The total loading on the heaviest loaded column is:

- From the column above 2559 kN (normal force)
- From the truss (including floor and variable loading) 1022 kN (normal force)
- From the truss (due to eccentricity) 1022 * 0,3 = 307 kNm

This means the total loading on the column becomes:
F_d = 2559 + 1022 = 3581 kN
M_d = 307 kNm

Now the required amount of rebar can be calculated.

$$n_d = \frac{N_d}{bhf`_b} = \frac{3581*10^3}{500 \cdot 600 \cdot 21} = 0{,}57$$

$$m_d = \frac{M_d}{bh^2 f`_b} = \frac{307*10^6}{500 \cdot 600^2 \cdot 21} = 0{,}08$$

Via the interaction diagram it is found that no extra rebar is required, therefore a minimum amount is applied of 6 bars with a diameter of 10 mm. This means the rebar has an area of, A = 471 mm^2.

Now the column also needs to be checked for second order effects.

For the slenderness it holds:
λ = l/h = 3000/600 = 5

For α_n it holds:

$$\alpha_n = \frac{N_d}{A_b f`_b + A_s f_s} = \frac{3581*10^3}{21(500*600 - 2*471) + 2*471*435} = 0{,}54$$

Because it holds 0,25 < α_n <0,5 and λ < 10, no second order effect has to be taken into account.

Garage floor

The loading on the garage floor consists of two parts, a permanent loading caused by the self weight of the construction and a variable loading which is defined as p_{rep} = 5,0 kN/m^2
The self weight consists of two parts, the weight of the plates used for the floor and the weight of the top layer.

The garage floor will be supported by the foundation. No soil data of the exact building location exists. However data of a nearby location does exist. For the purpose of this thesis the underground of the building site is assumed identical to that of the building site of the soil data.
A sand layer strong enough to support the building is found at a depth of NAP -12,5 m. The location of the building site is at NAP +1,0 m. The height of the parking garage, including the support structure for the floor above is 5,0 m. This means the top of the floor will be located at NAP -4,0 m. When a construction height of 0,5 m is assumed for the construction of the floor, this means that the piles will require a minimum length of 8 m.

The piles are located under the columns and a number of piles will be located under the parking garage floor.

The parking garage floor will consist of hollow channel plates, type A320. The length of each plate will be 9,4 m. According to the design diagram supplied by the producer of these plates, the maximum span with these plates for this loading is 10,8 m. Since the span in this case is 9,4 m these plates can be used for the garage floor.

These plates will have to be supported by a cross-beam, which is supported by the piles. The piles are placed at a c.o.c. distance of 9,0 m.

For the cross-beam a steel beam HE400A is chosen, this leads to a total loading on the cross-beam of:
q_d = 5,0 * 9,4 *1,5 + 1,25 * 1,2 = 72 kN/m

This leads to a maximum bending moment of:
M_d = 0,1 * q_d * l^2 = 0,1 * 72 * 9^2 = 583 kNm

This leads to a maximum stress of:

$$\sigma_d = \frac{M_d}{W_d} = \frac{583*10^6}{2{,}31*10^6} = 252 N/mm^2$$

This leads to a unity-check of:

$$\frac{\sigma_d}{f_{y,d}} = \frac{252}{355} = 0{,}71 \leq 1$$

Therefore the HE400A beam can be used to support the floor.

Now all calculations have been done, drawings can be made. For these drawings is referred to appendix 11.

10.7 Bridge access

The bridge has a height of 12,5 m above the water level at the location of the pylon, the river bank has a height of 1,0 m above the water level. This means there is a height difference of 11,5 m. To make it possible to access the bridge from the bank or leave the bridge onto the bank, 2 bridges are placed to space this gap. These bridges will be placed perpendicular to the cable-stay bridge. Beside these bridges an option is created for pedestrians to step onto the terrace and enter the theatre by placing steps from the bridge to the terrace.
The maximum angle at which the bridge can space this height difference is 5%. This means the total length of these bridges will be 230 m. The distance between the pylon and the side bridges is 80 m, by already placing the side span of the stay-cable bridge at an angle of 5%, the side bridges can be shortened to 150 m.
The material chosen to construct these bridges is prefabricated concrete. The chosen type of plates is VIP-plates.

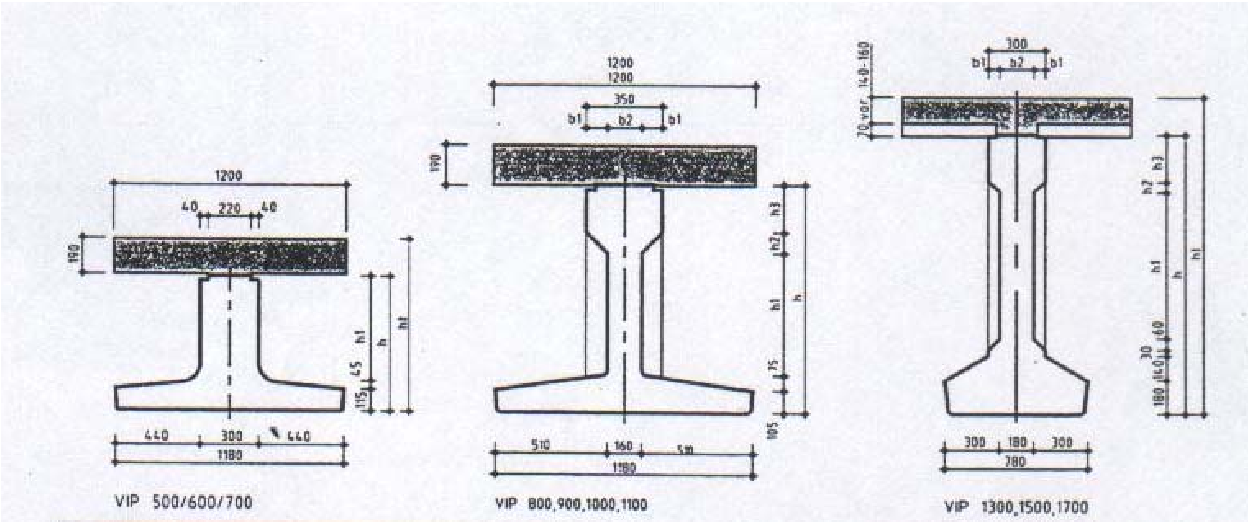


Figure 10.41: VIP-plates.

Plates

The variable loading onto these plates is the same as acting on the cable-stay bridge, p_{var} = 4,0 kN/m^2. These plates are placed at a c.o.c. 2400 mm. This means the variable loading onto a plate is:
q_{var} = 4 * 2,4 = 9,6 kN/m

The bridge is supported by portals c.o.c. 30 m. According to the design graph (appendix 12), this means VIP 1300 plates should be sufficient to span the gap.

Portal

The portal consists of a beam supporting the VIP-plates and two columns. The beam is a concrete beam, B45. The dimensions are found with rules of thumb and are:
h = 1000 mm
b = 350 mm

The loading onto the beam comes from the VIP-plates, $q_{var,d}$ = 180 kN/m and from the self weight, $q_{p,d}$ = 9 kN/m. This means the total bending moment is:
$M_d = 1/8 * q_d * l^2 = 1/8 * (180 + 9) * 10^2 = 2359$ kNm

Now the required amount of rebar can be calculated:

$$\frac{M_d}{bd^2 f_b`} = \frac{2359}{350 * 350^2 * 27} = 250$$

According to the standard table this means that the required amount of rebar is, ω_0 = 1,84%

This means the required area is:
A_{sl} = 1,84% * 350 * 1000 = 6510 mm^2

By placing 9 bars with a diameter of 32 mm an area of 7236 mm^2 is found.

The beam is supported by 2 columns. Only the longest columns will be calculated, the other columns will get the same with and height of the cross-section, making it easier during erection.
The loading onto the column comes from the beam, this leads to a vertical force of:
$V_d = ½ * q_d * l = ½ * 189 * 10 = 945$ kN

For the dimensions of the column the same width is used as the width of the beam:
b = h = 350 mm

Besides being loaded by a normal force, the column can also be loaded by a bending moment. This occurs when on one side of the column the bridge is fully loaded, while the other side is empty. The bending moment is caused by the variable loading alone. The force acting on the column as a result of this loading is $V_{m,d}$ = 450 kN. The eccentricity is ¼ * b_{beam} = 87,5 mm. This leads to a bending moment loading the column of:
$M_d = e * V_{m,d} = 39$ kNm.

Via the interaction diagram the required rebar can be found.

$$n_d = \frac{N_d}{bhf`_b} = \frac{450 * 10^3}{350 * 350 * 27} = 0{,}14$$

$$m_d = \frac{M_d}{bh^2 f`_b} = \frac{39 * 10^6}{350 * 350^2 * 27} = 0{,}03$$

This means Ψ = 0,02, and the rebar percentage is, ω = 0,001

This means the required minimum area for the rebar is:
$A_{sl} = \omega bh = 0{,}001 * 350 * 350 = 152$ mm^2

By applying 6 bars with a diameter of 6 mm 170 mm^2 is found.

Now the column also has to be checked for second order effects:

The slenderness of the column is:

$$\lambda = \frac{l_c}{h} = \frac{0{,}7 * 7500}{350} = 15$$

$$\alpha_n = \frac{N_d}{A_b f_b^{`} + A_s f_s} = \frac{450 \cdot 10^3}{(350 * 350 - 2 * 170)27 + (2 * 170)435} = 0{,}13$$

$\alpha_n < 0{,}25$ it holds that the column does not have to be checked for second order effects if:

$$\lambda \leq \frac{5}{\sqrt{\alpha_n}} = 13{,}7$$

This is not the case and therefore this column will have to be checked for second order effects.

The total eccentricity due to second order effects is defined as:

$$e_t = (e_0 + e_c)\xi \geq e_0$$

With:

e_0 = largest initial eccentricity

$$e_c = 3\left(1{,}5h + e_0\left(4\psi - 3\right)\right)\left(\frac{\rho l_c}{100h}\right)^2$$

This means for the wall loading:

$$= 3\left(1{,}5 \cdot 350 + 150\left(4 \cdot 1 - 3\right)\right)\left(\frac{1 \cdot 5250}{100 * 350}\right)^2 = 41{,}3mm$$

$$\xi = 0{,}5\left(1 + \frac{e_1}{e_0}\right) \geq 0{,}75 \Rightarrow \xi = 0{,}75$$

This leads to a total of:

e_t = (87,5 + 41,3) * 0,75 = 97 mm

This means the new bending moment becomes, M_d = 44 kNm. And m_d = 0,04

This leads to the same amount of rebar. Therefore 6 bars with a diameter of 6 mm will be applied.

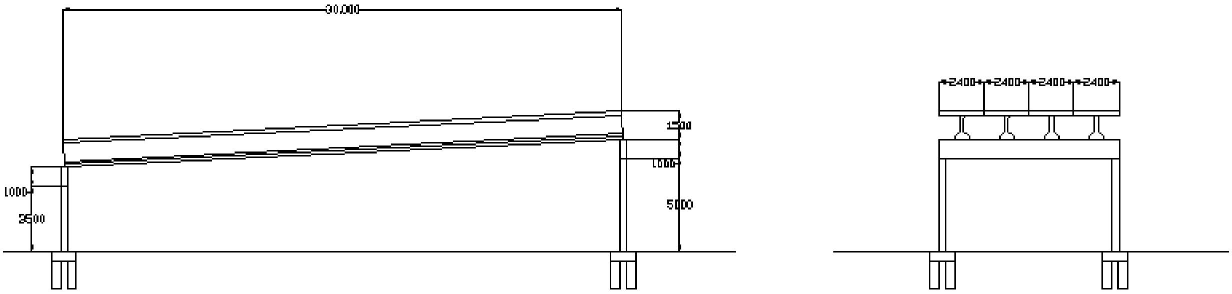


Figure 10.42: One segment of the side bridge.

10.8 Method of erection

In this paragraph the methods of erection for the different part of the project are discussed.

On the Zwijndrecht side of the river the whole project consists of three parts, the first is the theatre building, the second the stay-cable bridge and the third are the side bridges.
On the Dordrecht side there are two parts, a movable bridge and a building complex, because the design of these two parts was outside the scope of this thesis, the erection of these parts is not possible to discuss here. However it is clear that for traffic to pass along the river a part of the river must stay open, therefore both the movable and the cable-stay bridge can not be erected simultaneously.

Building

To start the building of the theatre building and parking garage, first a building excavation is required, due to the fact that the parking garage will be underground. After this is done the piles supporting the construction can be driven into the ground.

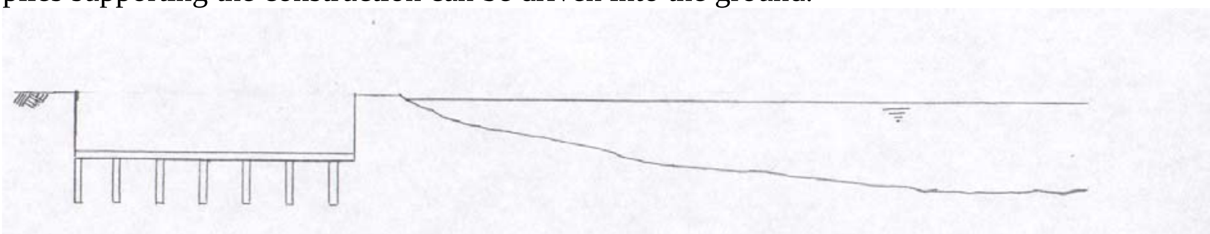


Figure 10.43: Stage 1: Piles and parking garage floor.

On top of these piles the parking garage floor can be placed and the columns supporting the theatre can be placed. Then the floor for the bottom 4 theatres and the lobby can be placed.

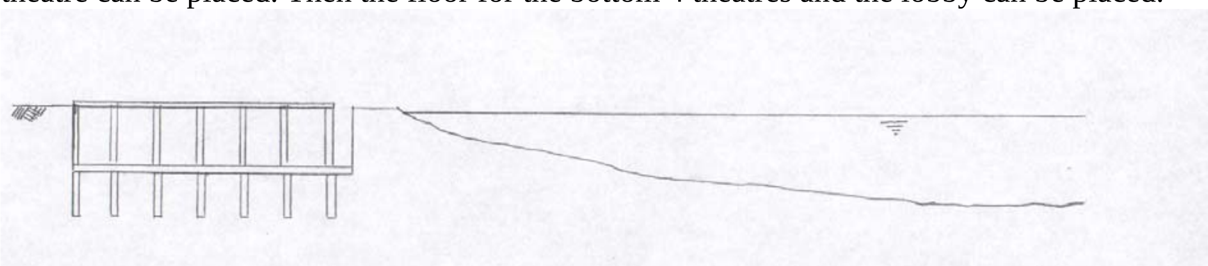


Figure 10.43: Stage 2: Placing columns on garage floor and placing theatre floor.

On top of these floors, first the concrete anchor walls must be cast, these two walls will not only form a part of the bearing construction of the building, but will also act as anchor weights for the cable-stay bridge.

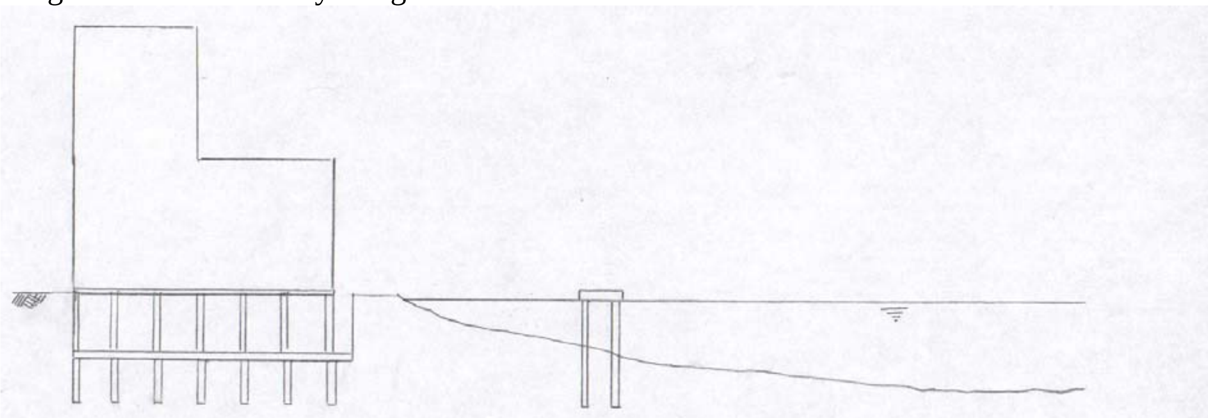


Figure 10.45: Stage 3: Placing of anchor walls and pylon foot.

While these walls harden, the prefabricated concrete columns can be placed, as well as the steel trusses which will support the top floor and the roof. The capacity of the cranes depends on the heaviest prefabricated part. The weights of the different parts are:

- The total weight of a truss is 8298 kg.
- The total weight of a roof plate is 5076 kg.
- The total weight of a floor plate is 1013 kg
- The total weight of the largest column is 8550 kg
- The total weight of a diagonal floor girder is 3525 kg

This means that the column is the heaviest part, and is therefore determining for the crane capacity.

When the main bearing construction is erected, the wall elements can be placed. When this is done, the work inside the theatre can be done, such as installing the seats, placing the projectors and screens and placing the materials to create the right acoustic environment.

Cable-stay Bridge

The first parts of the bridge which need to be erected are the anchor walls, which are part of the building, these walls are used to act as an anchor weight for the back-stay cable and will therefore stabilise the pylon.
Before the pylon can be placed first the piles supporting the pylons and the concrete bottom plates must be placed. After this is done, the pylon can be erected from prefabricated parts. Due to the chosen stay-cable configuration, the deck can only be placed when the full pylon is erected, because the anchoring points for the cables are located at the top.

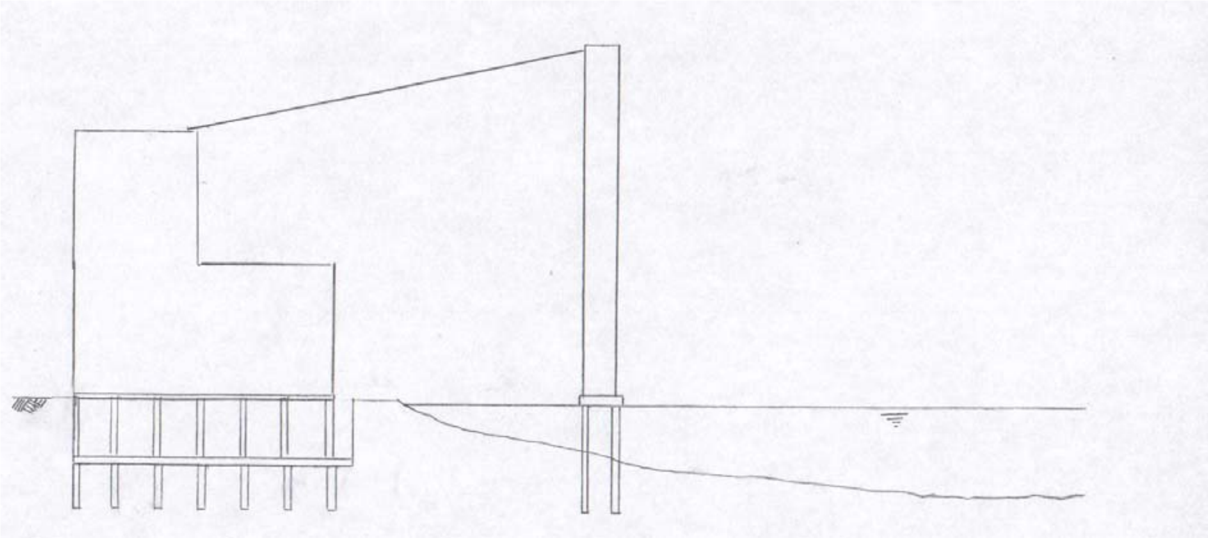


Figure 10.46: Stage 4: Erection of the pylons and anchoring them to the building.

When the pylons are erected first, the back stay-cables need to be placed and tensioned, stabilising the pylons.
Now that the pylons are placed and anchored the deck can be placed. The deck and main girders form bridge segments with a length of 16 m. The method used for erecting the bridge deck is called the balanced cantilever method. The elements can be transported over water, when they arrive at the building site, they can be hoisted in place and be connected to the elements already in place. After an element is hoisted into place and connected to the previously placed elements, the stay-cable can be placed and tensioned. During the erection of the deck, also the abutments on both the river bank and in the river need to be placed.

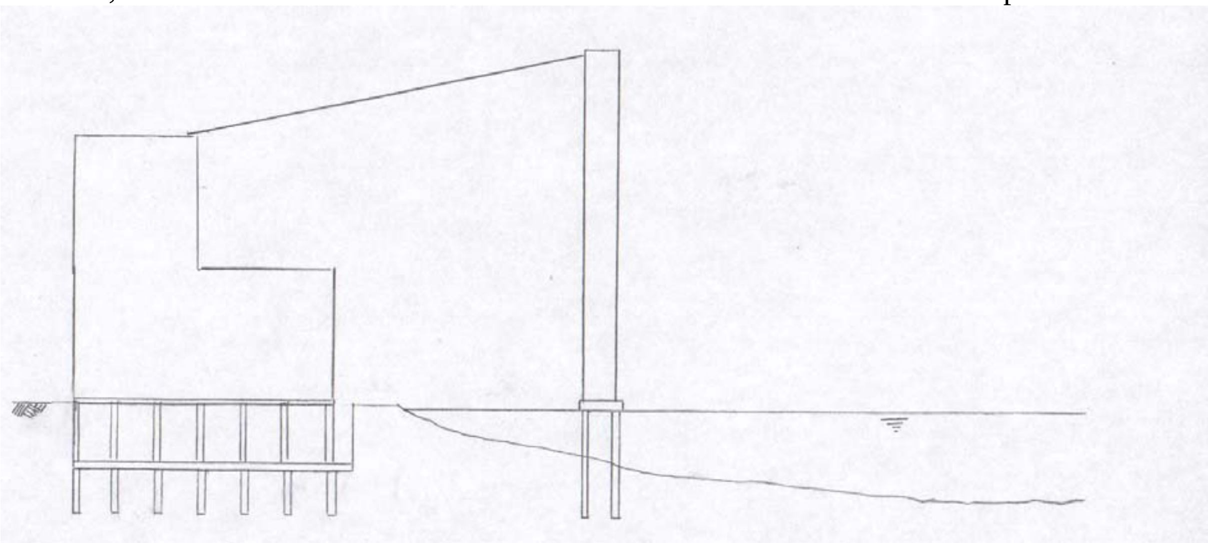


Figure 10.47: Stage 5: Placing of first deck elements and the abutments.

The first element will be placed on the river side of the pylons, the second on the bank side, the third again on the river side, etc. After 4 elements are placed on both sides of the pylons,the bridge can no longer be erected in a balanced way, due to the eccentricity of the pylons.

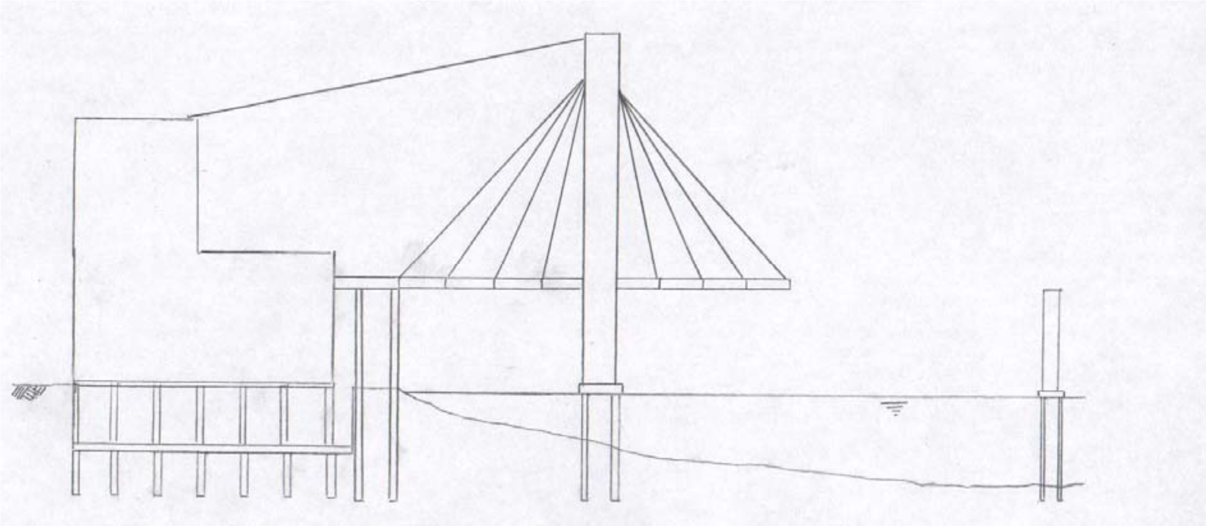


Figure 10.48: Stage 6: Placing of deck elements on both sides of the pylon

At this point only elements will be placed on the river side.

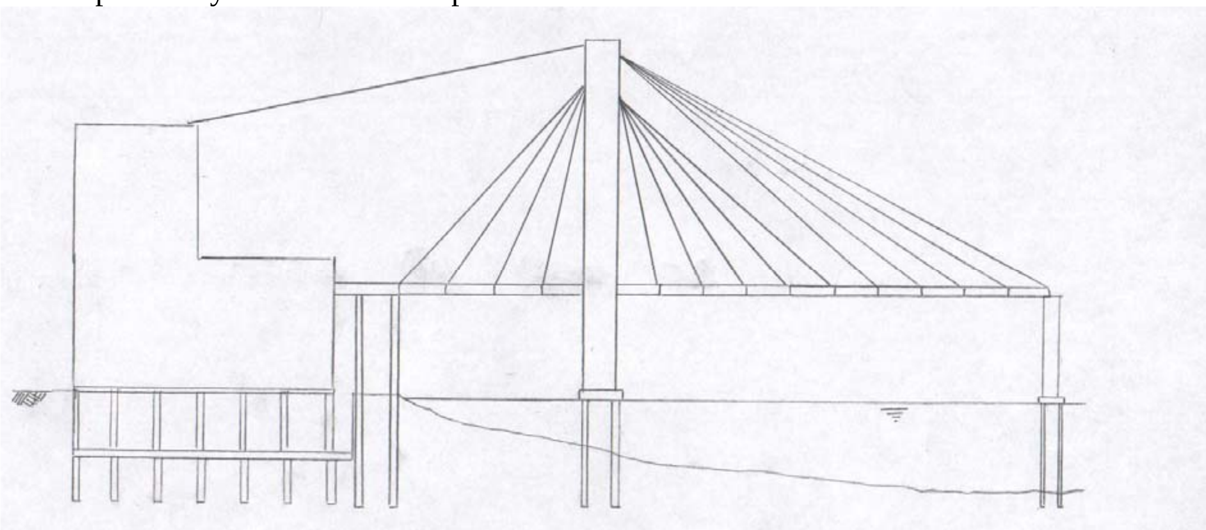


Figure 10.49: Stage 7: Placing of the remaining deck elements.

When all elements are placed and all cables are placed and tensioned, the main bearing construction of the bridge is finished and the detailing of the bridge can be started, by placing the asphalt layer, installing the safety measures and placing the lighting.

Side bridges

The side bridges consist of prefabricated concrete elements. First the piles need to be driven, after this is done the foot plates can be placed and the columns for the portal can be placed. Then the cross-beams can be placed. Now the portals are finished the VIP-plates can be hoisted into place. Now the main bearing construction is finished the asphalt layer and the safety measurements can be placed.

Surroundings

Now all the constructions have been erected, the surroundings can be addressed. Some of the options for the surroundings are:

- A promenade along the river
- A playground for children
- A park
- A shopping centre
- Office space

Beside these options also the connection between the bridge and the existing roads needs to be established.

11. Collision protections

Whenever a construction is made which crosses a channel or river, this gives the risk of collisions. This is certainly true for a living bridge over the Oude Maas between Dordrecht and Zwijndrecht. This is a very busy route for shipping, and the living bridge will cause a reduction of the width of the river in comparison to the current situation. The risk is also determined by the size of the biggest ship, which sails on the river, in this case a “6-baksduwboot”. In this chapter an overview is given of the possibilities to prevent damage to the bridge in case of a collision.

11.1 Options

There are a number of options which can help reduce the risk of a collision. However in this case also the risks for the buildings next to the bridge need to be taken into consideration.

Option 1: Do nothing
When choosing this option, nothing is done to reduce the risk of a collision. This is the cheapest option of all, however it is also the least logical option to choose. In case of a wide river with no obstacles and a bridge which spans the river in its total, this is an option. However this is not an option here because, the bridge over the Oude Maas will need more spans, due to the fact also a movable part is required.

Option 2: Prevent
In this case the measures are taken to prevent a collision or reduce the chance on a collision. It is possible to reduce the maximum allowed speed or reduce the size of the ships allowed on this route. Or a slackening structure can be created to slow a ship down and steer a ship back on course.

Option 3: Protect
Here a construction is placed around or in front of the main structure, to protect the main structure in case of a collision. The secondary structure is there to take the first blow, or correct the ships course. Damage to this secondary structure is allowed, but not preferred.

Option 4: Designing with collision loads
In this option, a collision is assumed and the load bearing capacity of the main structure is modified for this.

Normaly a combination of the options above is adapted, to enlarge the safety for both users of the river and users of the bridge and its buildings. To come to the right (combination of) options for this case, first is looked at the possibilities to bring the options mentioned above into practice.

11.2 Ships energy

All options to enlarge the safety have one thing in common, they are designed to disperse the ships energy in case of a collision. First the ships energy needs to be determined. To do this, the characteristics of the largest ship on the river are needed.

The river Oude Maas is classified as a main shipping channel, and therefore has class Vic (determined by the CEMT (Conference of European Ministers for traffic)), this means the largest ship on the river is a “6-baksduwboot (wide)”. This ship gives the characteristics needed for the design.

Table 11.1: Shipclasses

Class	Type motor ship/type “duwstel”	Length (m)	Width (m)	Draught (m)	Height (m)	Weight (ton)
0	Small vessel					
I	Spits	38,50	5,05	1,8-2,2	4	250-400
II	Kempenaar	50-55	6,6	2,5	4-5	400-650
III	Dortmund-Eemschannel ship	67-80	8,2	2,5	4-5	1.000-1.500
IV	Rijn-Hernekanaalschip /duwstel 1 bak	80-85	9,5	2,5	5,25-7	1.500-3.000
V a	Large rijnship /duwstel 1 bak	95-110	11,4	2,5-4,5	5,25-7	1.600-3.000
V b	Duwstel 2 baks (long)	172-185	11,4	2,5-4,5	9,1	3.200
VI a	Duwstel 2baks (wide)	95-110	22,8	2,5-4,5	7-9,1	3.200-6.000
VI b	Duwstel 4 baks	185-195	22,8	2,5-4,5	7-9,1	6.400-12.000
VI c	Duwstel 6 baks (wide)	193-200	34,2	2,5-4,5	9,1	9.600-18.000

For the calculations the upper boundaries of the table 11.1 are used. These measures give the biggest risk and therefore lead to the safest situation when used in calculations.

Characteristics largest ship on the Oude Maas
Length: 200 m
Width: 34,2 m
Draught: 4,5 m
Height: 9,1 m
Weight: 18.000 ton
Mass: 27.000 ton
Block coefficient: 0.88
Speed: 18 km/h

The Block coefficient represents the slenderness of a ship beneath the water level.
It can be determined with the following formula:

$$C_b = \frac{V}{lbd}$$

With: V = Volume of water displacement by ship (m^3)
l = length of a ship (m)
b = width of a ship (m)
d = draught of a ship (m)

$$C_b = \frac{27.000}{200 * 34,2 * 4,5} = 0,88$$

The maximum speed is defined by the Dutch government, it is 18 km/h or 5 m/s.

Now all characteristics of the ship are known, the ships energy can be determined. This can be done with the formula for kinetic energy:

$$E_k = \frac{1}{2} m v^2$$

With:
E_k = Ships kinetic energy (kNm)
m = Ships mass (ton)
v = Ships speed (m/s)

However in case of a collision, also a number of other matters need to be taken into account. Calculating with just the normal formula leads to unsafe and therefore unwanted situations. To take these matters into account, four coefficients need to be added to the formula. These are the hydronamic coefficient, the excentricity coefficient, the softness coefficient and de configuration coefficient,

Hydronamic coefficient
A sailing ship causes a slipstream in the water, as a consequence water gets sucked by the ship. Therefore not only the ships mass needs to be taken into account, but also the mass of the water needs to be taken into account.

The total mass which has to be taken into account in case of a collision is:

$$m = m_s + m_w$$

This is added to the formula for the kinetic energy as, C_h.

$$C_h = \frac{m_s + m_w}{m_s}$$

The water mass is defined as:

$$m_w = \rho l \frac{1}{4} \pi d^2$$

With:
ρ = Density of water (ton/m^3)
l = Ships length (m)
d = Ships draught (m)

Eccentricity coefficient
In case a ship collides with a structure it will never be at an angle of 90°. The length axis of the ship will always be at a smaller angle with the construction. During a collision a ship will rotate, this rotation costs energy, and therefore this will reduce the energy which needs to be dispersed by the construction. This reduction is taken into account with the eccentricity coefficient, C_e.

$$C_e = \frac{k^2 + r^2 \cos^2 \gamma}{k^2 + r^2}$$

With:
k = Moment of inertia ship (m)
r = Distance from the ships point of gravity to the point of collision (m)
γ = Angle between r en the ships length axis (°)

The moment of inertia can be approached with the following formula:

$$k = (0{,}19C_b + 0{,}11)l$$

Waarbij:
C_b = Block coefficient ship
l = Ships length (m)

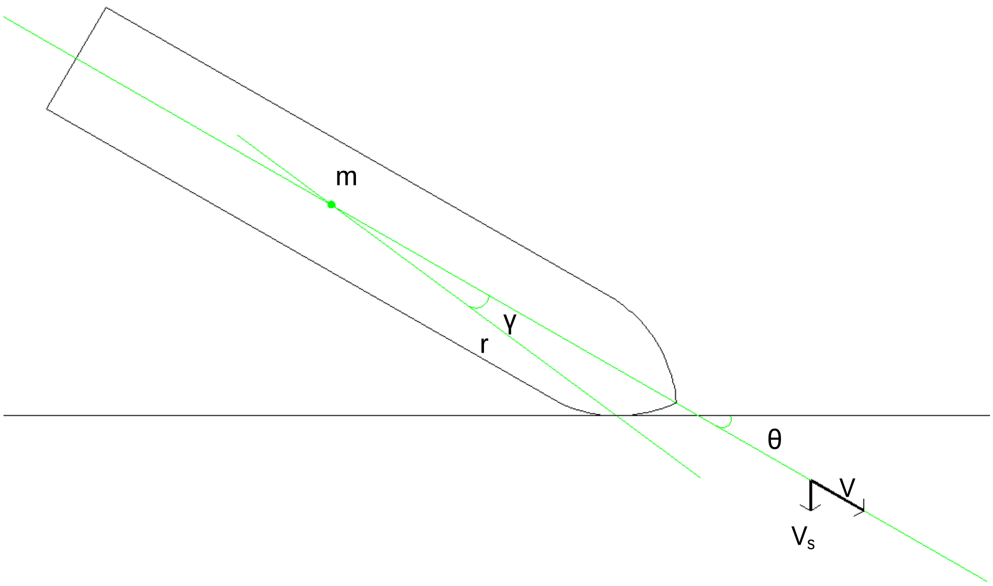


Figure 11.1: Schematisation collision.

Softness coefficient
This coefficient is used to take the relation between the ships elasticity and the constructions elasticity into account. In case a bendable construction is used, the ship can be modeled as infinite stiff, because the displacement of the construction is much larger than the ships displacement. In this case the softness coefficient is, C_s = 1. In case the construction has some bending stiffness, both the stiffness of the ship and the stiffness of the construction need to be taken into account. For this situation C_s = 0,9 can be assumed.

Configuration coefficient
This coefficient is used to take the friction in the water between the ship and the construction into account. This coefficient can especially be taken into account in case a ship is mooring at a dock. The friction reduces the energy. This reduction can be as much as 20%. This leads to:
$0{,}8 \leq C_c \leq 1{,}0$
In this case there is no mooring ship, but a construction designed to protect another construction. Therefore C_c = 1,0, this leads to a safe design.

Beside these four coefficients there is one more thing which needs to be taken into account. As stated before, the angle between a ship and the construction in case of a collision is never exactly 90°.
This also has an influence on the speed which needs to be taken into account. Because the speed at an angel of 90° is needed for calculating the energy. Therefore, for the speed holds:
$v_s = v \sin\theta$

The formula for the total energy becomes:
$$E_k = \frac{1}{2} m_s v_s^2 C_h C_e C_s C_c$$

When inserting the characteristic values for the Oude Maas only one variable remains, the angle of the collision. The energy which has to be dispersed depends on the angle, therefore the following table (11.2) and figure (11.2) give the energy at different angles.

Table 11.2: Energy that has to be dispersed per angle of collision.

Teta (°)	Energie (J)
0	0
10	2324
20	8594
30	17593
40	28011
50	38563
60	48090
70	55624
80	60443
90	62100

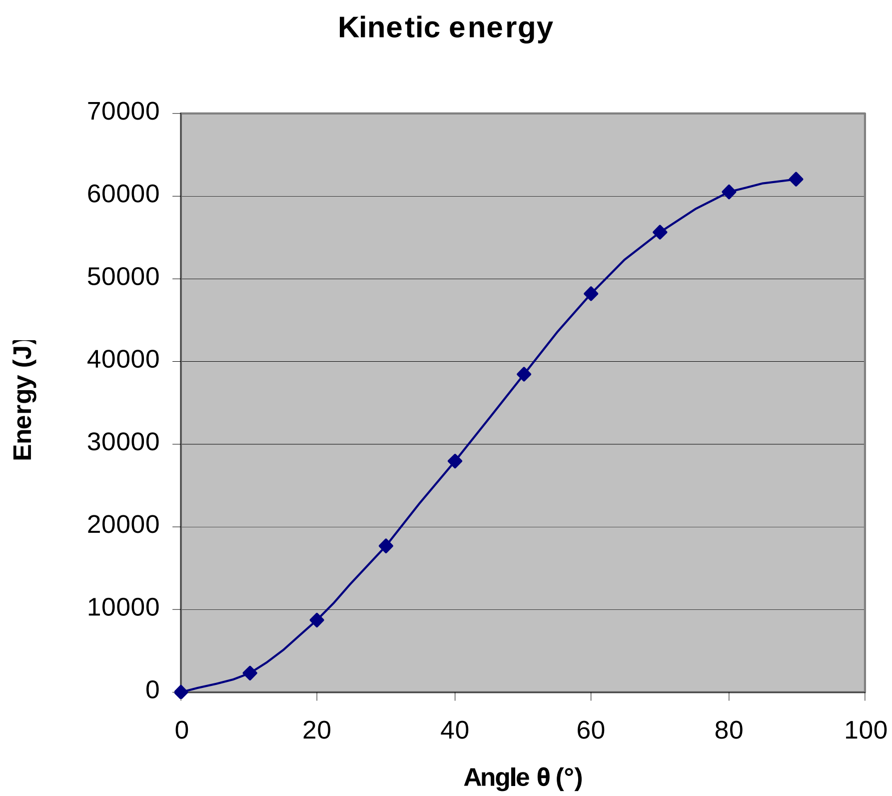


Figure 11.2: Energy that has to be dispersed per angle of collision.

As can logically be expected, the maximum energy occures in case a collison occures at an angle of 90°.

11.3 Possible safety measures

The kinetic energy released in case of a collision needs to be dispersed into another form of energy. There are three possible energy forms which the kinetic energy can be turned into:

- Displacement energy: The construction disperses the energy in case of a collision
- Potential energy: Energy needed to displace an object in the opposite direction of gravity
- Friction: Friction between two surfaces causes heat

Displacement energy

In case a collision occurs between two objects, they will deform, this deformation costs energy. The formula for this energy is as following:

$E_{work} = Fx$

The energy needed to deform an object depends on the materials stiffness. The terms F and x in the ecuation are therefore not independent. Assuming the stiffness is linear (displacement is equal to force):

$F = kx$

The total energy now becomes:

$$E_{work} = \int_0^x F dx = \int_0^x kx dx = \left[\frac{1}{2}kx^2\right]_0^x = \frac{1}{2}kx^2 - \frac{1}{2}k0 = \frac{1}{2}kx^2$$

Potential energy

This is the energy needed to move an object into the opposite direction of the gravity, therefore on earth, the energy needed to move an object upward. The energy needed depends on the gravity and the objects mass.

$$E_{pot} = mgh$$

With:
m = mass (ton)
g = gravitational acceleration (m/s^2)
h = difference in height (m)

However in this case it is not the ships mass which has to be taken into account, because the ship is in the water. This means less energy is needed to displace the ship. The mass which needs to be taken into account is the mass of the displaced water, therefore:

$$m_{water} = V_{dispwater}\rho_{water}$$

Using the ships characteristics for the displaced water gives:

$$V_{dispwater} = lbC_b h$$

For the potential energy it holds:

$$E_{pot} = lbC_b \rho_{water} g h^2$$

Friction

This is the energy which is conversed into heath in case two surfaces slide across one another. The roughness of the surfaces causes a resistance against the displacement. Therefore it costs energy to move slide the two surfaces along each other.

$$E_w = Fx$$

The size of the force depends on the shear stress of the surfaces and on the size of the surfaces. So:

$$F_w = \sigma_w A$$

The shear stress depends on the stress perpendicular to the surface. The roughness of the surface is expressed as the maximum friction angle. This leads to:

$$\sigma_w = \sigma_v \tan\phi = \sigma_v \mu$$

With:
σ_v = stress perpendicular to the surface (kPa)
Φ = friction angle (°)
μ = friction coefficient (-)

So the formula for the friction becomes:

$$E_w = \sigma_v \mu A x$$

11.4 Solutions

There are a variety of solutions for the dispersion of energy. But there are two main options. There can either be chosen to use the river bottom to steer and slow down the ship, or a (separate) construction can be made to do this

The soil

When a ship makes contact with the soil it will slow down. There are three mechanisms that are active in this case: gravity, friction and the displacement of the soil. Each of the three mechanisms plays a part in the dispersion of the energy. To get an idea of the effect of each of the mechanisms, they will be first looked at separately.

Gravity

When a ship collides with the bottom of the river, and assuming there is no friction, it will only lose energy if it is pushed upward. When assuming this is the only mechanism at work, then:

$E_k = E_{pot}$

$$\frac{1}{2}mv^2 = mgh$$

On the left side of the equation the mass is representing the mass of the ship, which is the same as the mass of the displaced water:

$$m = lbdC_b\rho_{water}$$

On the right side of the equation the mass is representing the difference in the displacement of the water:

$$m = lbC_b\rho_{water}h$$

Combining this gives:

$$\frac{1}{2}lbdC_b\rho_{water}v^2 = lbC_b\rho_{water}gh^2$$

This can be simplified to:

$$\frac{1}{2}dv^2 = gh^2$$

or:

$$h = \sqrt{\frac{v^2d}{2g}}$$

When substituting the numbers for this specific situation it gives:

$$h = \sqrt{\frac{5^2 \cdot 4{,}5}{2 \cdot 10}} = 2{,}37m$$

However there is a limitation for this situation, in the calculation above it is assumed the whole ship collides with the bottom, in most cases this won't be true. Only a part, most likely the front, will collide with the soil. This means only the part that collides will go upward, which means the energy loss will be lower than assumed in the calculation above. To correct this, a number of assumptions are required:

1. The gravitational centre is located in the geometrical centre of the ship.
2. The C_b of both the front and back half are equal.

To disperse the same amount of energy the front of the ship will be need to raised twice as high as in the previous situation.

$$h_{front} = 2h = 2\sqrt{\frac{v^2 d}{2g}}$$

Which means h_{front}=2 * 2,37 = 4,74 m

Another limitation is the water which moves with the ship. It is assumed it moves upward, but when it is assumed it only pushes the ship upward, but doesn't move upward itself the equation changes to:

$$\frac{1}{2}(m_s + m_w)v^2 = \Delta m_s gh$$

or:

$$\frac{1}{2} m_s v^2 C_h = \Delta m_s gh$$

This leads to:

$$h = \sqrt{\frac{v^2 d C_h}{2g}}$$

For this situation it gives:

$$h = \sqrt{\frac{5^2 \cdot 4{,}5 \cdot 1{,}1}{2 \cdot 10}} = 2{,}49m$$

As stated before, the front needs to be raised two times the height:

$$h_{front} = 2 \cdot h = 2 * 2{,}49 = 4{,}98m$$

Friction

When a ship touches the bottom of a river it is slowed down due to the friction between the ship and the soil of the bottom. In addition the ships bow is raised. The amount of friction depends on the vertical force on the contact area. The vertical force depends on the lifted mass and therefore on the vertical displacement of the bow.

$$F_w = \mu F_v$$

$$F_v = m_{upward} g = lbh_{grav} C_b \rho_{water} g = \frac{1}{2} lbh_{bow} C_b \rho_{water} g$$

To be able to do the calculations an assumption for the slopes of the river need to be made. When assuming no work is done on the river bottom and the only soil present is the soil naturally present there, the slopes of the river give the slopes needed for the calculations. It is assumed the river has a slope of 1:100

The formula transforms to:

$$F_v = \frac{1}{2} lb\alpha x C_b \rho_{water} g$$

with:

α = slope = $\frac{dh}{dx}$ (°)

x = length of friction path (m)

The total friction can be calculated with:

$$E_w = \int_0^x F_w dx$$

Combining the formula’s above gives:

$$E_w = \int_o^x \frac{1}{2} \mu lb\alpha x C_b \rho_{water} g dx = \frac{1}{2} \mu lb\alpha C_b \rho_{water} g \int_0^x x dx = \frac{1}{4} \mu lb\alpha C_b \rho_{water} g x^2$$

Assuming the total amount of energy is dispersed by friction, it gives:

$$E_k = E_w$$

$$\frac{1}{2} lbdC_b \rho_{water} v^2 = \frac{1}{4} \mu lb\alpha C_b \rho_{water} g x^2$$

Simplifying gives:

$$dv^2 = \frac{1}{2} \mu \alpha g x^2$$

This leads to the formula for the length of friction required for the dispersion of the ships energy.

$$x = \sqrt{\frac{2dv^2}{\mu \alpha g}}$$

With the assumed slope of 1:100 this leads to a length of:

$$x = \sqrt{\frac{2 \cdot 4{,}5 \cdot 5^2}{0{,}4 \cdot 0{,}01 \cdot 10}} = 75m$$

This means the bow of the ship will raise 0,75 m

When assuming the ship will be pushed further by the water which moves with the ship, the mass will increase, this gives:
x = 79,3 m and h = 0,79 m

Combination

In the previous, friction and gravity are looked at separately, in practice of course they work together. This means the total energy of the ship colliding with the bottom will be transformed into potential energy and friction energy.

$$E_k = E_{pot} + E_w$$

Or:

$$\frac{1}{2}mv^2 = mgh + \int_0^x F_w dx$$

This is equal to:

$$\frac{1}{2}lbdC_b\rho_{water}v^2 = lbC_b\rho_{water}gh^2 + \frac{1}{4}\mu lb\alpha C_b\rho_{water}gx^2$$

Combining with h_{front} = 2*h and simplifying gives:

$$dv^2 = \frac{1}{2}gh_{front}^2 + \frac{1}{2}\mu\alpha gx^2$$

Substituting x = 100*h, gives:

$$h_{front} = \sqrt{\frac{dv^2}{(\frac{1}{2} + 5000\mu\alpha)g}} = \sqrt{\frac{4{,}5 \cdot 5^2}{(\frac{1}{2} + 5000 \cdot 0{,}4 \cdot 0{,}01)10}} = 0{,}74m$$

Given the fact that the slope is 1:100 this means x = 74,1 m

When assuming the ship will be pushed further by the water which moves with the ship, the mass will increase, this gives:
x = 78,3 m and h = 0,78 m

In the previous a constant slope of the river is assumed, however due to the level changes of the river it will probably be necessary to create a man made slope to protect the constructions. When doing so it is possible to change the slope of the bottom, therefore the effect of different slopes will be discussed.

The effect of the slope of the river is displayed in the graphs 11.3 and 11.4. The first graph shows the rising of the front of the ship needed to stop the ship. At the x-axis the length needed to rise 1 m is displayed (slope) and at the y-axis the rising of the front needed is displayed.
The second shows the length needed to stop the ship.
It becomes clear from the graphs, that the steeper the slope, the less length is required to slow down and stop the ship.

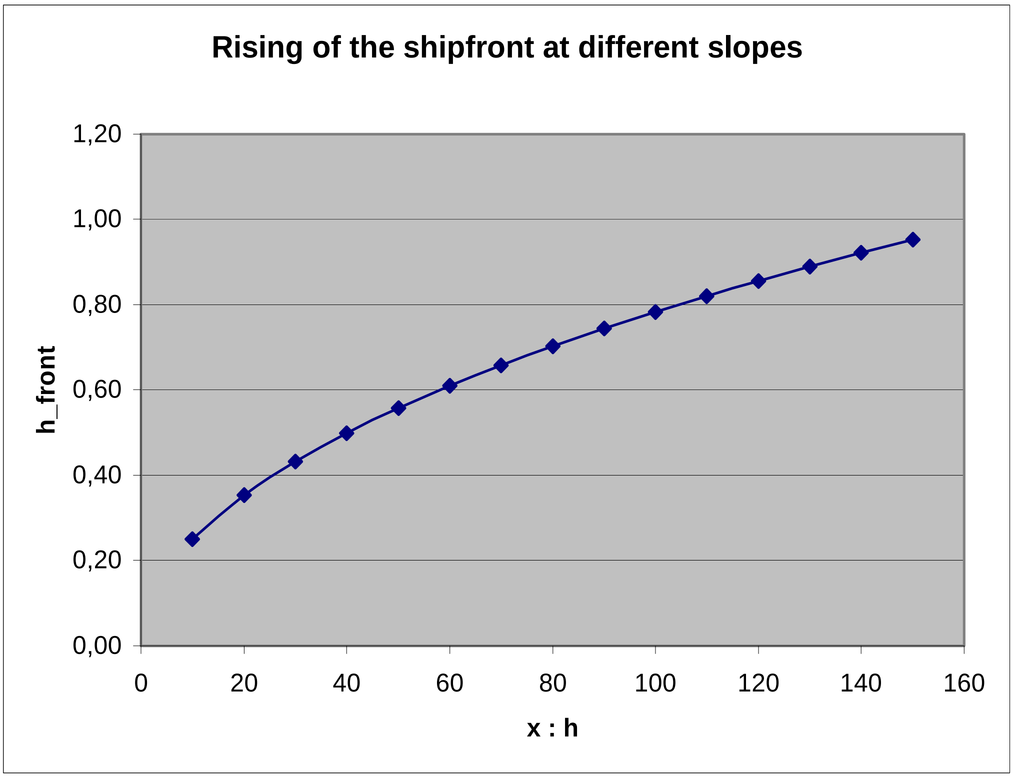


Figure 11.3: Rising of ship front in relation to slope.

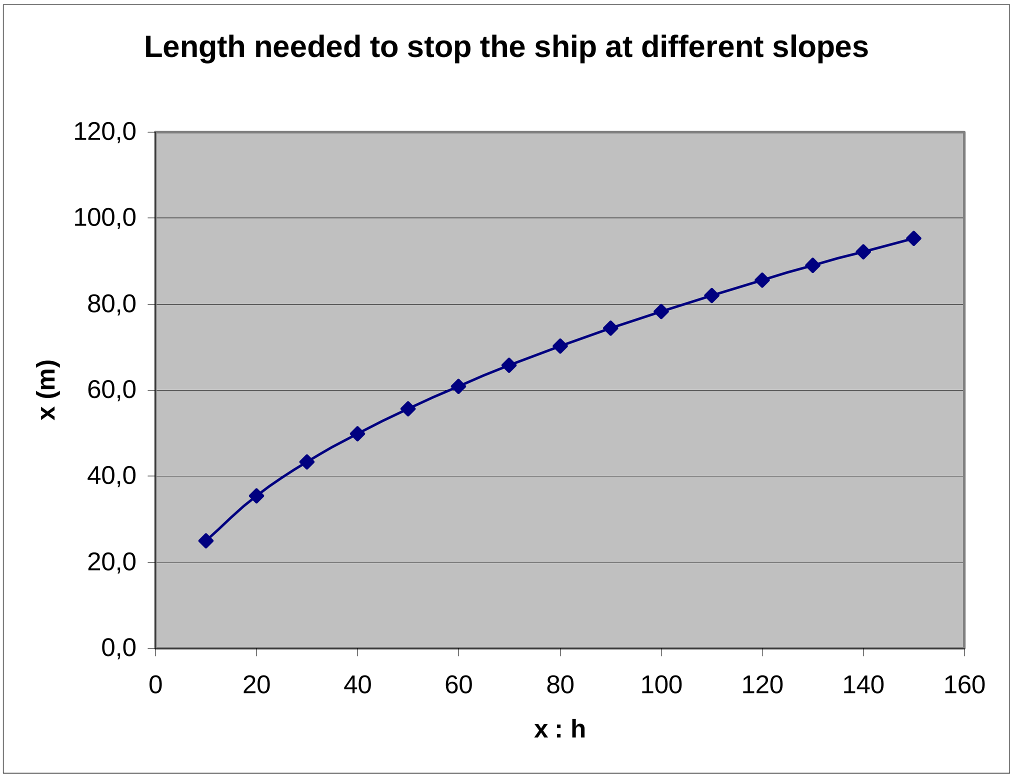


Figure 11.4: Required length in relation to slope.

It is clear that the effect of the transformation of the ships energy into potential energy has a small effect in comparison to the effect of the friction.
This effect changes however when the slope of the river changes. In the final formula the potential energy is present as 1/2g and the friction is present as 5000μαg. Since g and μ are constants it is clear the only variable is α, being the slope. So when α changes also the effect of the friction changes and therefore the effect of the potential energy plays a smaller or larger role. In graph 11.5 below the graphs for the energy dissipation with and without potential energy are displayed.

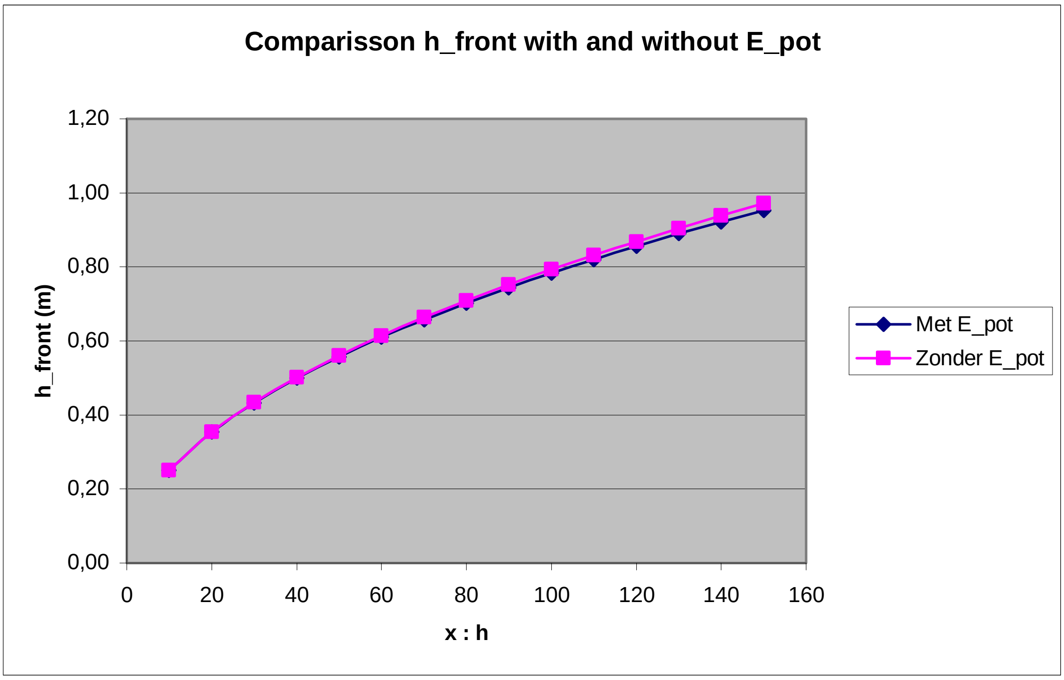


Figure 11.5: Energy dissipation with and without potential energy.

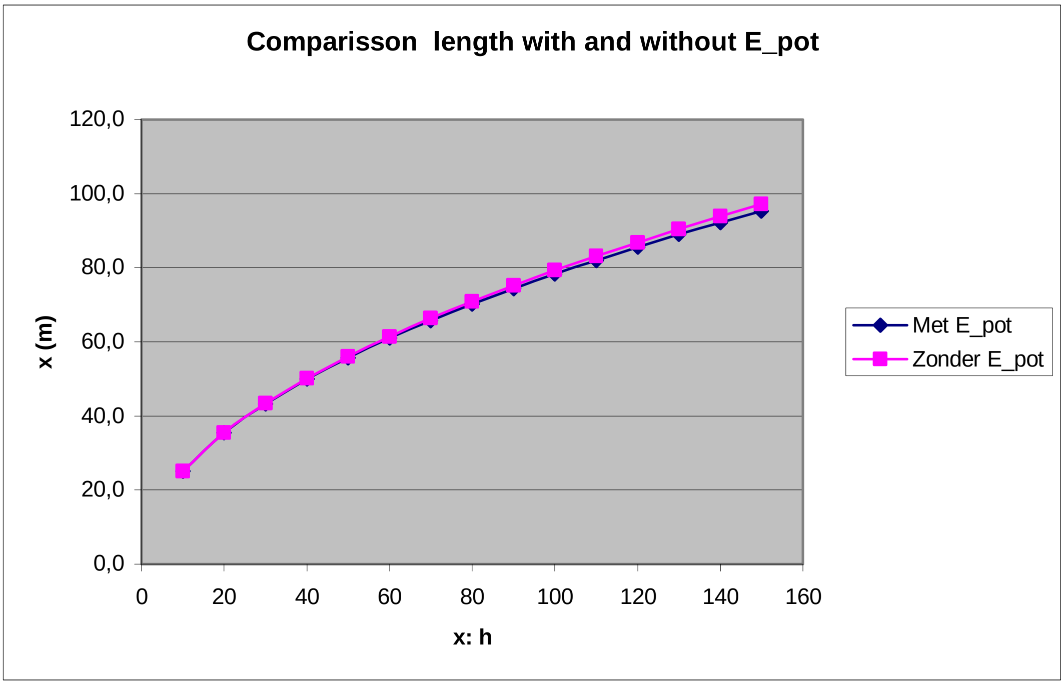


Figure 11.6: Required length with and without potential energy.

Displacement of soil

As stated before, there are three mechanisms active when a ship collides with the bottom of a river. Friction and gravity are discussed in the previous, the third mechanism is the displacement of soil.
When soil is subjected to a heavy load, it no longer acts as a linear elastic material. If the stress becomes too large, the particles start to shift. This of course costs energy, and causes the ship to slow down. However there is no consensus about the exact behaviour of soil in this case, and therefore it is also not clear what the exact relation between force and displacement is.
The Delft UT has done some research on the collision of ships with soil. With the results of this test a force – displacement diagram was created. This was used to create insight about the length needed to stop a ship. When looking at the results it becomes clear, that the length needed is larger than the length needed in case only friction and gravity are active. However, during the test the vertical displacement was kept at 0, they did measure the forces and concluded, considerable forces in vertical direction are active. In reality they will trigger the two mechanisms mentioned in the previous, and therefore create a much shorter length needed to stop a ship.
Because the exact effect of displacement of soil on stopping a ship is not clear, it will not be used in calculations in case the soil will be used to slow down a ship.

Construction

Besides using the soil to stop a ship it is also possible to create a construction to stop a ship. There are three possible constructions to do this: A slackening structure, a sliding construction and an emergence wall.

Slackening structure

Slackening structures are primarily created to slow down and give direction to a ship, without doing damage to either ship or construction. Of course when a ship is too far off course, damage is unavoidable, but the structure will still slow down the ship and therefore reduce the damage to the structure it is protecting. A slackening structure stops ships by deforming. The deformations cost energy, and therefore when a ship collides with a slackening structure, it is slowed down and eventually stopped.

There are 3 different possibilities to create a slackening structure: Single pile, a group of piles and a connected pile row. They will all be discussed separately in the following.

Single pile

It is assumed a ship does not collide with one pile, but with a row of piles. Each pile gives a small change in the ships direction. Assumed is an initial angle between the pile and a ship of $\theta = 1°$. It is assumed each pile gives a change in direction of 1°.
The characteristic ship, is the same ship as in the previous, therefore it holds:
m_s = 27.000 ton
v = 5,0 m/s

The equation for the ships (kinetic) energy remains the same:

$$E_k = \frac{1}{2} m_s v_s^2 C_h C_e C_s C_c$$

With graph 11.2 the energy of the ship can be determined.
The ships energy is:
E_k = 24,6 kNm

The piles are round steel open sections, with the following characteristics:

d =	457,0	mm
t =	40,0	mm
I =	$114949 \cdot 10^4$	mm^4
W =	$5030{,}6 \cdot 10^3$	mm^3
A =	52402	mm^2

The pile is embedded in the soil, this gives a moment bearing capacity, however this connection is not a full moment bearing connection. Therefore a fictive length is used for the calculations. It is also assumed the collision between ship and pile is 1 m above the water level.

Pile

The fictive length is given by:

$$l_f = l_{pile} + 0{,}65 d_{pile}$$

With:
l_{pile} = Length from load on the pile to the bottom of the river
d_{pile} = Length from the bottom of the river to the bottom of the pile

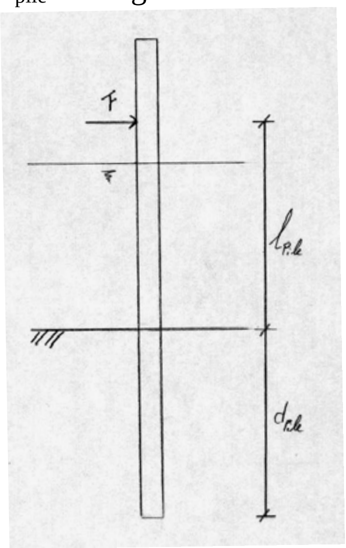


Figure 11.7: Schematisation pile.

The depth of the river is about 10,0 m, therefore l_{pile} = 11,0 m The same length, 11,0 m is assumed for d_{pile}. This gives l_f = 18,15 m.

For the bending stiffness of the pile holds:

$$k_{pile} = \frac{3EI}{l_f^3} = \frac{3 \cdot 210.000 \cdot 114.949 * 10^4}{18150^3} = 121{,}1 kN/m$$

The maximum load that can be applied on the pile is given by:

$$F_{pile} = k\Delta x = \sqrt{2kE_{k,\max}} = \sqrt{2 \cdot 121{,}1 \cdot 24{,}6} = 77{,}2kN$$

The piles displacement is:

$$\Delta x = \frac{F_{pile}}{k} = \frac{77{,}2}{121{,}1} = 0{,}64m$$

Now it is necessary to determine the stress in the pile, to make sure the pile can bear the load in case of a collision. And it is necessary to check if the passive resistance of the soil is large enough to bear the horizontal load.

The moment on the pile in case of a collision is given by:

$$M = F_{pile} l_f = 77{,}2 \cdot 18{,}15 = 1401kNm$$

This leads to a stress in the pile of:

$$\sigma = \frac{M}{W} = \frac{1401*10^6}{5030{,}6*10^3} = 278{,}5N/mm^2$$

The maximum stress of steel is 235 N/mm^2, therefore a pile will fail in the given situation.

However, this calculation was done without taking into account the fenders and there bending stiffness. When adding a fender, with a similar bending stiffness, to the construction, the total bending stiffness becomes:

$$\frac{1}{k} = \frac{1}{k_{pile}} + \frac{1}{k_{fender}} = \frac{1}{121{,}1} + \frac{1}{120} = 0{,}0166$$

And k = 60,3 kN/m

Now F_{pile} = 54,5 kN and Δx = 0,90 m

The stress in the pile becomes:

$$\sigma = \frac{989{,}175*10^6}{5030{,}6*10^3} = 196{,}6N/mm^2$$

This stress is lower than the maximum allowable stress, and therefore the pile is ok.

Soil

The maximum load for the soil is given by:

$$P = \gamma' K_p \frac{t_0^3}{24} \times \frac{t_0 + 4b}{t_0 + l_{pile}} \text{ and } t_0 = \frac{d_{pile}}{1{,}2}$$

With:
γ' = effective density (kN/m^3)
K_p = passive soil pressure coefficient (-)
t_0 = depth of 0-moment point (m)
b = with of the pile (m)

For the soil under the river the following characteristics hold:
$\gamma' = 10$ kN/m^3
$K_p = \tan^2(\frac{\pi}{4}+\frac{\phi}{2}) = \tan^2(\frac{\pi}{4}+\frac{0{,}349}{2}) = 2{,}04$
$t_0 = 9{,}17$ m

Then:

$$P = 10 \cdot 2{,}04 \cdot \frac{9{,}17^3}{24} \cdot \frac{9{,}17 + 4 * 0{,}457}{9{,}17 + 11} = 357 kN$$

The maximum load is 54,5 kN, so the soil is strong enough to bear the load.

Group of piles

When connecting a number of piles a group is created. In stead of one pile, all piles disperse the energy. Which means the load on each pile is smaller than in the case only one pile is used for the dispersion of the ships energy.

The calculations are for a group of 8 piles. The angle on which the ship collides with the structure is set at 3°. This means the ships energy E_k = 219 kNm (see graph 11.2).

For this case smaller piles are chosen than in the previous case:
The piles are round steel open sections, with the following characteristics:

d =	457,0	mm
t =	25,0	mm
I =	$79415*10^4$	mm^4
W =	$3475{,}5*10^3$	mm^3
A =	33929	mm^2

The space between two piles is set at 3,0 m in both directions. The lengths of the piles are equal to the previous case.

For the bending stiffness of one pile, it holds:

$$k_{pile} = \frac{3EI}{l_f^3} = \frac{3 \cdot 210.000 \cdot 79.415 * 10^4}{18150^3} = 83{,}7 kN/m$$

The bending stiffness is equal to the bending stiffness of 8 single piles, so:

$$k_g = 8 \times k_{pile} = 8 \times 83{,}7 = 669{,}6 kN/m$$

$$I_g = \Sigma k_{pile} \cdot (x_i^2 + y_i^2)$$
$$= 83{,}7 \cdot (4 \cdot 1{,}5^2 + 4 \cdot 4{,}5^2 + 8 \cdot 1{,}5^2) = 9040 kNm$$

The total bending stiffness of the group now becomes:

$$\frac{1}{k_{gt}} = \frac{1}{k_g} + \frac{e^2}{I_g}$$

k_{gt} = 268 kN/m

When applying a fender (k_f = 200 kN/m) to this construction, the total bending stiffness becomes:

$$\frac{1}{k_{tot}} = \frac{1}{k_{gt}} + \frac{1}{k_f}$$

k_{tot} = 114 kN/m

The force acting on the pilegroup now becomes:

$$F_g = \sqrt{2kE_{k,\max}} = \sqrt{2 \cdot 114 \cdot 219} = 223kN$$

The force acting upon the outermost pile, can be split into two parts:

$$F_y = \frac{F_g}{n} + \frac{F_g e a_y}{I_g} = \frac{223}{8} + \frac{223 \cdot 4{,}5 \cdot 4{,}5}{9040} = 28{,}4kN$$

$$F_x = \frac{F_g e a_x}{I_g} = \frac{223 \cdot 4{,}5 \cdot 1{,}5}{9040} = 0{,}17kN$$

The total force acting on the pile is:

$$F_{pile} = \sqrt{F_y^2 + F_x^2} = 28{,}4kN$$

The bending moment in the pile is:

$$M = Fl_f = 28{,}4 * 18{,}5 = 525kNm$$

And the maximum stress equals:

$$\sigma = \frac{M}{W} = \frac{525}{3475{,}5 * 10^{-3}} = 151N/mm^2$$

So the pile is strong enough to bear the load.

Besides checking the construction, it is also necessary to check if the soil can withstand the forces.
The maximum load for the soil is given by:

$$P = \gamma' K_p \frac{t_0^3}{24} \times \frac{t_0 + 4b}{t_0 + l_{pile}} \text{ and } t_0 = \frac{d_{pile}}{1{,}2}$$

With:
γ' = effective density (kN/m^3)
K_p = passive soil pressure coefficient (-)
t_0 = depth of 0-moment point (m)
b = with of the pile (m)

For the soil under the river the following characteristics hold:
$\gamma' = 10$ kN/m^3
$K_p = \tan^2(\frac{\pi}{4}+\frac{\phi}{2}) = \tan^2(\frac{\pi}{4}+\frac{0,349}{2}) = 2,04$
$t_0 = 9,17$ m

Then:

$$P = 10 \cdot 2,04 \cdot \frac{9,17^3}{24} \cdot \frac{9,17 + 4 * 0,457}{9,17+11} = 357kN$$

The strength of the soil is much larger than the load caused by a ship colliding with the construction.
However a consideration needs to be made. Because the piles are relatively close together, a reduction on the strength of the soil needs to be made. This reduction is about 30% of the strength. This means the reduced strength of the soil is 0,7 * 357 = 250 kN. This has no consequences for the design, because the soil is still strong enough.

Connected pile row

This is the most applied form of slackening structure. A row of piles is connected by a horizontal tube. The total stiffness of the construction is a combination of the stiffness of the horizontal tube and the stiffness of the piles.

For this case the same piles as in the previous case will be used. Also the horizontal tube will be a round steel open section, with the same properties as the piles. The connection between piles and tube is assumed to be a hinged connection. For the simplification of the problem it is assumed the tube has an infinite length and the load is applied at a distance large enough to distribute the load in both directions.

There is a contradiction between the properties of the construction and the properties of a ship. Because of the linear relation between stress and displacement, for a construction it is favourable to have small displacements, because this gives low stresses. The ship however would like large displacements, because the dissipation of the energy is much larger in this case, this leads to smaller forces on the ship and therefore to smaller damage to the ship.

When looking at the construction and the calculating of it, a number of things become clear:

- The diameter of the piles has the biggest influence on the constructions displacement.
- Looking at the bending strength of the piles, the diameter has little influence. The wall thickness plays a much larger role in the bending strength.
- For the horizontal tube, both diameter and wall thickness play an important role in the bending strength.

When looking at the construction in a cost effective way, it becomes clear that it is better to strengthen the horizontal tube and use lighter piles, because stronger components mean more material, which costs more. And there is only one tube and there are many piles.

When spacing the piles at a distance of 6 meters the following combination is strong enough to bear the load.
Piles with a diameter of 457,0 mm and a wall thickness of 25 mm and a tube with a diameter of 1,50 meters and a wall thickness of 10 mm.

To reduce the size of the tube it is an option to choose for a double row of piles. This can reduce the size of the tube to a diameter of 650 mm and a wall thickness of 25 mm.

Sliding construction

The main function of a slackening structure is to steer a ship back onto its course, or bring it to a halt. Both with the objective to prevent damage to the main structure. In optimal condition it does as little damage as possible to the ship. When there is no room for a slackening structure, or due to new regulations the slackening structure is not up to date anymore, another structure is needed to stop the ship or steer it back to its course. This should be a structure which can realize this within a short distance. One of the most common forms of such a structure is a sliding construction. The ship gets lift up and steered back at its course by this construction. The mechanisms working are the same as in the case soil is used to do this, they are gravity and friction.
In this case a concrete block with an inclination 1:3 is used. The assumed angle of collision is 15°.

The ships energy is equal to:

$$E_k = \frac{1}{2} m_s v_s^2 C_h C_e C_s C_c$$

In graph 11.2 it can be found that the kinetic energy at an angle of 15° is 5038 kNm.

The friction coefficient of steel and concrete is 0,35.
The equation for the rising of the ships front is, as derived earlier:

$$h_{front} = \sqrt{\frac{dv^2}{(\frac{1}{2} + 4\frac{1}{2}\mu\alpha)g}} = \sqrt{\frac{4{,}5 * (5 * \sin 15)^2}{(\frac{1}{2} + 4\frac{1}{2} * 0{,}35 * 0{,}33)10}} = 0{,}86m$$

This means the construction needs a length of 3 * 0,86m = 2,6 m.

This leads to relative small forces on the structure. The maximum force on the structure is 19,4 $*10^3$ kN. When taking a concrete with strength class B35, this leads to f_b' = 21 N/mm^2. This leads to an area of 19,4 / 21 = 0,92 m^2 to bear the load. When looking at a ship, it becomes clear the contact area is much larger than this. So it can be concluded that the structure is strong enough to bear the load.

Emergency wall

The third and final possibility is to construct a wall in front of the structure. This wall will prevent a direct collision between ship and structure. The wall gets the first punch, this causes the ship to slow down and therefore do less damage to the main structure.

It is assumed the slackening structure has failed and the ship still has its full speed when colliding with the wall. When assuming the ship collides head on with the wall, this leads to an E_k = 62100 kNm.
The length over which the ship is damaged is assumed at 2 m.
This leads to a force:

$$F = \frac{E}{\Delta x} = 31050kN$$

The wall consists of a concrete wall with a ballast of sand. In case of a collision, the wall gets subjected to a moment. So to check the wall, the following equation needs to be solved:

$$5 * G \geq 6 * F$$
$$G \geq 37260kN$$

This means the mass of the construction should be equal to 3726 ton.

When assuming the density of concrete and sand equal this leads to a length needed of:

$$l = \frac{m}{bh\rho} = \frac{3726}{10*6*2} = 31m$$

Also the soil behind the wall needs to be checked, whether or not it can hold the wall in place in case of a collision.

The equation for the soil pressure is:

$$F_{soil} = \frac{1}{2}\gamma h^2 K_p + 2ch\sqrt{K_p}$$

With:
F_{soil} = maximum applicable force on the soil, per meter (kN)
γ = density of the soil (kN/m^3)
h = height (m)

$$K_p = \frac{1+\sin\phi}{1-\sin\phi}$$

The soil applied behind the construction is sand, which has the following properties:
γ = 17 kN/m^3
h = 6 m
Φ = 30°
K_p = 3
c = 0

This leads to:

$$F_{soil} = \frac{1}{2}*17*6^2*3 + 2*0*6*\sqrt{3} = 918kN$$

The length needed is:

$$l = \frac{F}{F_{soil}} = \frac{31050}{918}33,8m$$

The last case is determinant for the length needed for the construction.

12. Conclusions and Recommendations

The purpose of this thesis was to make a structural design for a living bridge for pedestrians and cyclists between Zwijndrecht and Dordrecht. Making both the river banks come together as one new city centre. In this chapter the conclusions from the thesis are presented. After this also the recommendations which follow from this thesis are presented.

12.1 Conclusions

Before a design could be made for the living bridge first a definition of what a living bridge is. By studying living bridges build in the past the following definition was acquired: A living bridge is bridge with multiple functions. The first function is always to span a gap, being a river, a highway or a canyon. The second function is formed by a construction on or next to the bridge. This can be both residential and commercial.

With a more clear view on the design purpose a study of the location for the living bridge was done.

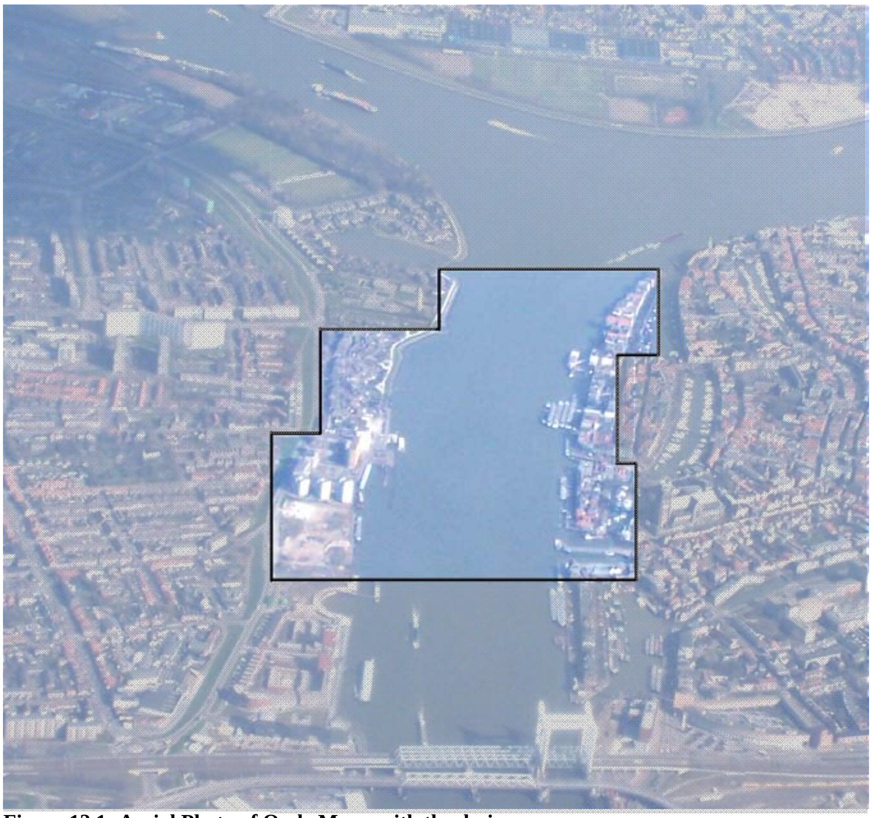


Figure 12.1: Aerial Photo of Oude Maas, with the design area.

From this study a number of conclusions were drawn:

- The Oude Maas is classified as a main shipping channel, meaning the shipping channel cannot be blocked, because it must stay open for ships.
- The Oude Maas is a heavily sailed route, among the ships is a part which transports dangerous goods, such as flammable and toxic goods.
- Due to the transport of dangerous goods no buildings with a residential function are allowed within the limits of the river banks.

- The river is part of the Standing Mast route, and therefore the bridge requires a movable part.

With this and the limitations from the list of requirements in mind, designs for the living bridge could be made. The first basic designs were made by students of the faculty of architecture. From these three designs one was chosen for further studies. The chosen design included a theatre and a café with a terrace and a parking garage beneath it on the Zwijndrecht bank of the river and a shopping centre and houses on the Dordrecht bank of the river. These two structures were connected by a cable-stay bridge and a small movable bridge.

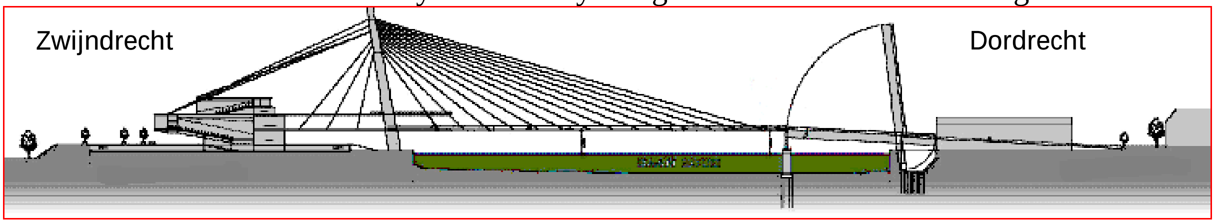


Figure 12.2: Alternative 1: Cable stay bridge

With this design as a starting point a number of design choices were made with respect to the type and location of the pylon, the cable arrangement and the anchoring point of the back stay-cables. Also a simple functional design was made to get insight into the required dimensions of the different areas. This lead to a preliminary design.

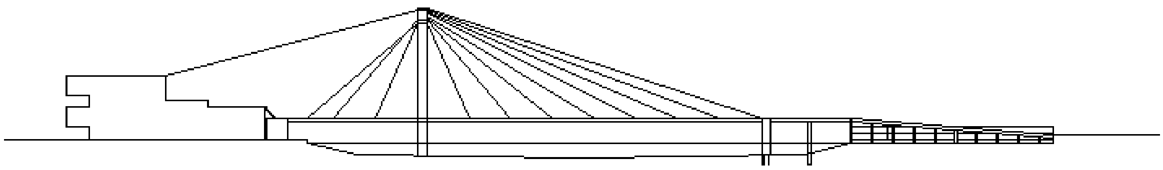


Figure 12.3: Side view preliminary design.

With this preliminary design the calculations on the main structures could start. The objective of this thesis was to make a structural design of one building and of one bridge. The theatre on the Zwijndrecht bank and the stay-cable bridge were selected for the structural design.

Before the bridge could be calculated first the choice was made for a steel bridge. After this was done the bridge was calculated with loads according to EN. This resulted in the following dimensions for the structural components of the bridge.
First the dimensions of the stay-cables are given (table 12.1) then a cross section of the pylon is given (figure 12.4), after this a cross-section of the deck and the main girders is given (figure 12.5). Finally a drawing of the whole bridge is given (figures 12.6 and 12.7).

Table 12.1: Chosen diameter per stay-cable.

Cable number	Chosen diameter [mm] – area [mm^2]
Back-stay-cable (1)	212 - 30957
2	56 - 2136
3	56 - 2136
4	56 - 2136
5	56 - 2136
6	56 - 2136
7	56 - 2136
8	80 - 4358
9	80 - 4358
10	80 - 4358
11	80 - 4358
12	96 - 6276
13	96 - 6276
14	96 - 6276

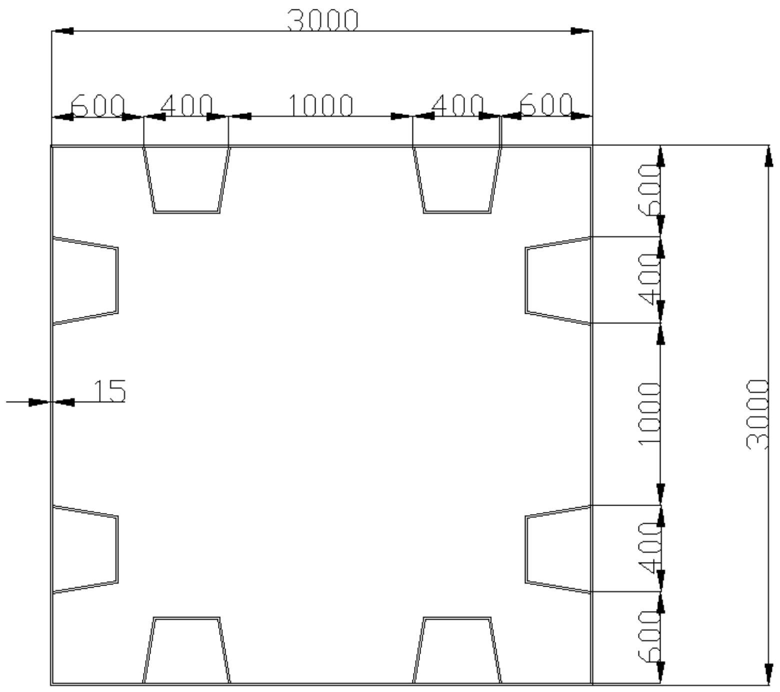


Figure 12.4: Cross-section of pylon with final dimensions.

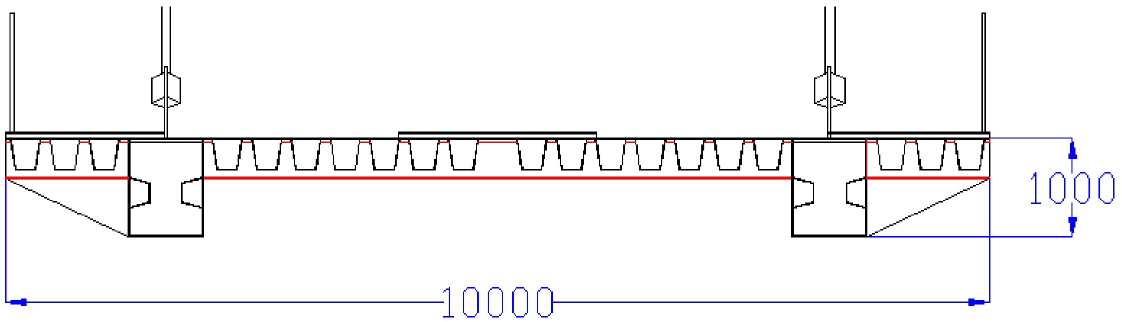


Figure 12.5: Cross-section of the bridge deck.

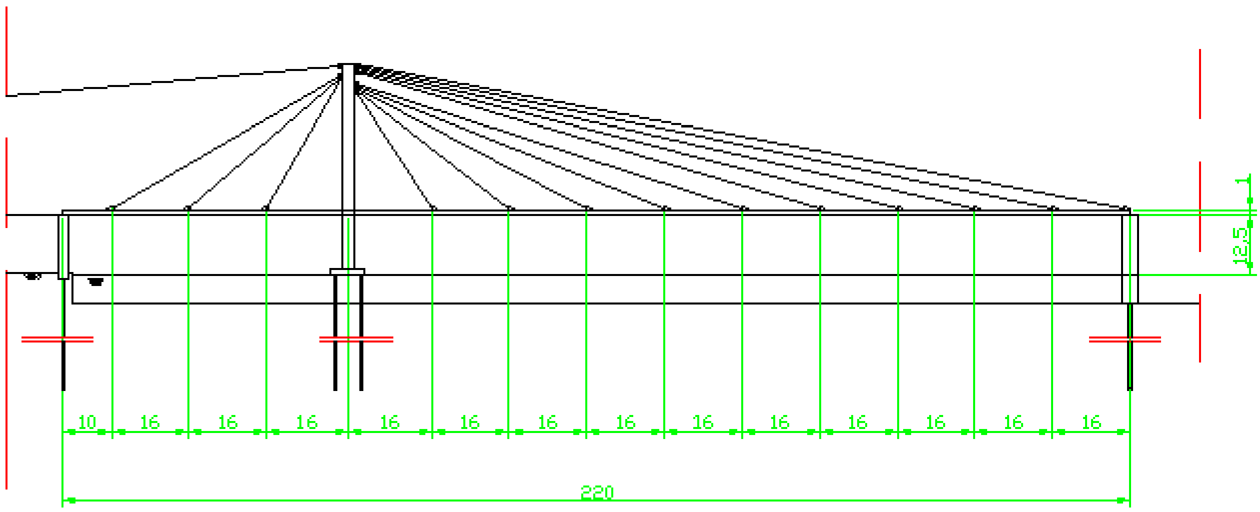


Figure 12.6: Side view final design.

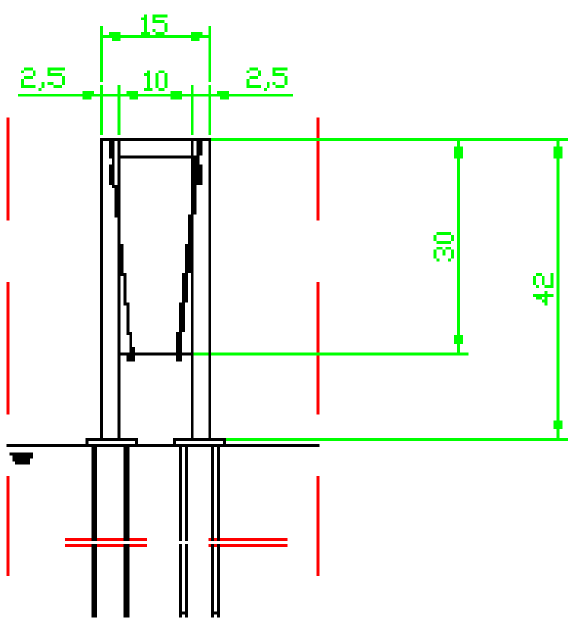


Figure 12.7: Front view final design.

For the material for the theatre building and the parking garage concrete was chosen.
After this was done the structure was calculated with loads according to EN. This resulted in the following dimensions for the structural components of the building. The roof consists of hollow channel plates, type A320 (figure 12.8) In case of the top floor these plates are supported by HE1000A girders. In case of the roof of the bottom theatres a truss is used for the girders (figure 12.9). The columns will get dimensions 500 * 600 mm.

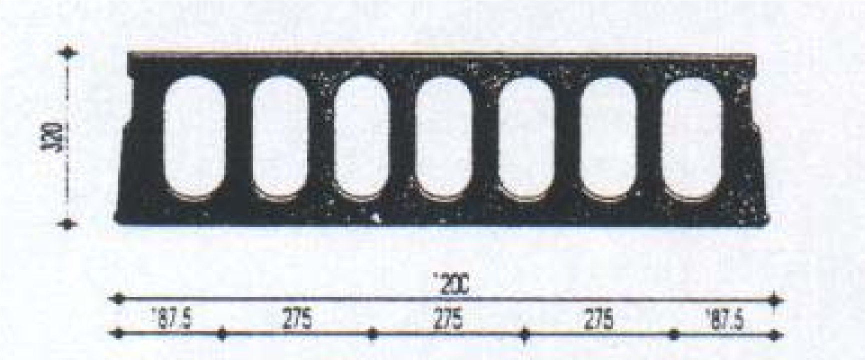


Figure 12.8: Cross-section channel plate A320.

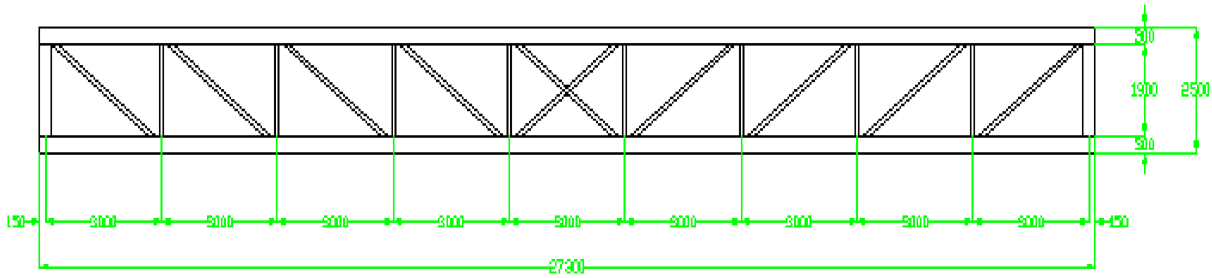


Figure 12. 9: Truss.

The diagonal floors on which the seatings will be placed will be concrete slabs placed like a stairway supported by steel HE400A beams. These beams are in turn supported by trusses (see 12.9).
A cross-section of the structure for the building is given in figures 12.10 and 12.11.

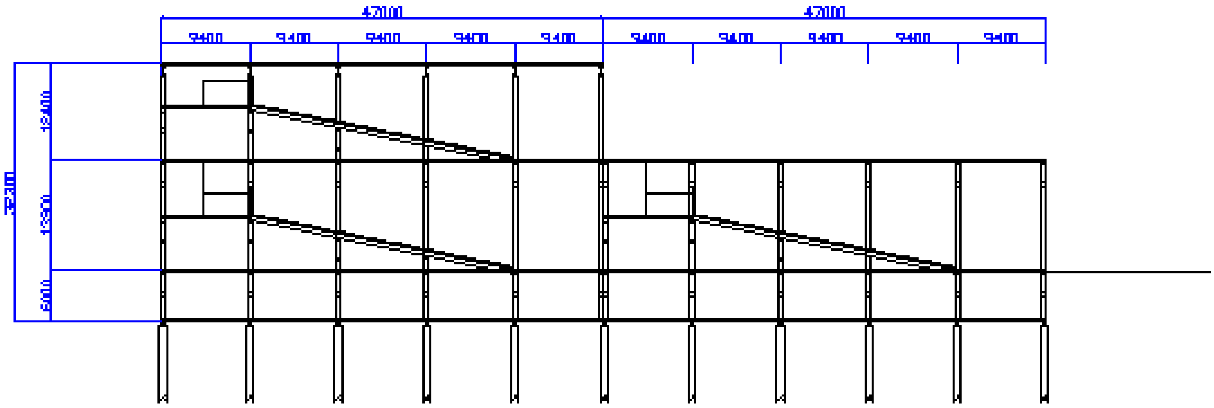


Figure 12. 10: Cross-section side view building.

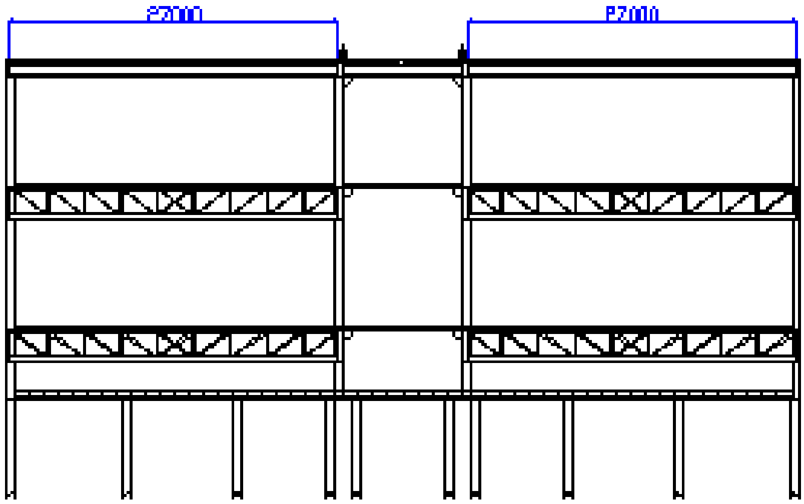


Figure 12. 11: Cross-section front view building.

There are two connections between the building and the bridge. The first is the anchoring of the back stay – cable. The second is the connection between the bridge and building for the pedestrians. Because there is a difference in height between the bridge and terrace a stair is created to make it possible for pedestrians to enter the terrace, café and theatre from the bridge.

With the different structural components and their dimensions known a description of the method of erection is given. First the parking garage needs to be build, after this the two anchor walls can be erected. With the walls ready, the erection of the bridge can start, these walls are required for the stabilisation of the pylon. With the pylon in place the deck of the bridge can be placed. This is done by method of balanced cantilever. A bridge element is placed on either side of the pylon and anchored to the pylon before the next element is placed. This can be done four times, then the part on the bank side of the pylon is finished and only elements on the river side need to be placed. Of course the erection of the rest of the building can be done simultaneously with that of the bridge.

In the final chapter an indication is given of the possibilities for creating a collision protection for the pylon and buildings on the Dordrecht bank.

12.2 Recommendations

- Testing for soil data at the building location is required.
- Structural design of the movable bridge is required
- Structural design of the buildings on the Dordrecht bank of the river is required
- An analysis of the comfort behaviour of the stay-cable bridge is required
- An analysis of the behaviour under wind loading of the stay-cable bridge is required.
- Design of the interior of the theatres is required for the right acoustic in the theatres.
- Research for bottom protection around the pylon is required.
- A choice for the protection against collision for the pylon is required.

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Surveys

Bathymetry surveys, RWS

Appendices

Appendix I

In this appendix a short summary is given of the presentation on the subject of a living bridge in Dordrecht. The presentation is a part of the MSc-thesis.

History

The city of Dordrecht came into being in the 12th century. It was located on the banks of the river the Thuredrith. Next to the city were two rivers, the Merwede and the Oude Maas. These rivers were part of the main shipping routes across Europe, making Dordrecht the ideal place for the trans-shipment of goods. This brought a lot of wealth to the city of Dordrecht. But the rivers were also important for the access to Dordrecht, because there was no infrastructure on land, the only entrance to the city was by water. Also the river running trough the city was important, because the shape of the river determined the shape of the city. And because the main social life in those days was based on the banks of the river and on the bridges across it.

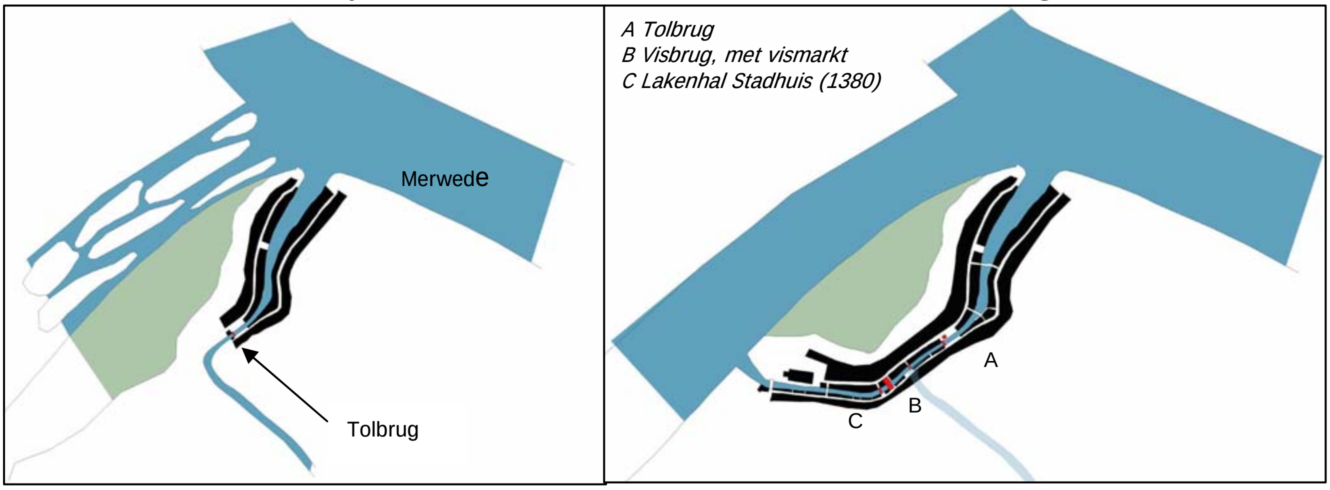

Figure I.1: Dordrecht in the 13th and 14th century.

In the 14th century due to the Elisabeths tide it was required to connect the Thuredrith and the Oude Maas. In this period also the city was expanded along the river.
The city continued to develop well into the 16th century. At this point in time the city had utilised every available space within the city limits and started to look for options to expand the city beyond this limits. At this point the city turned to the river again, because at this point new land was won by impoldering parts of the river.

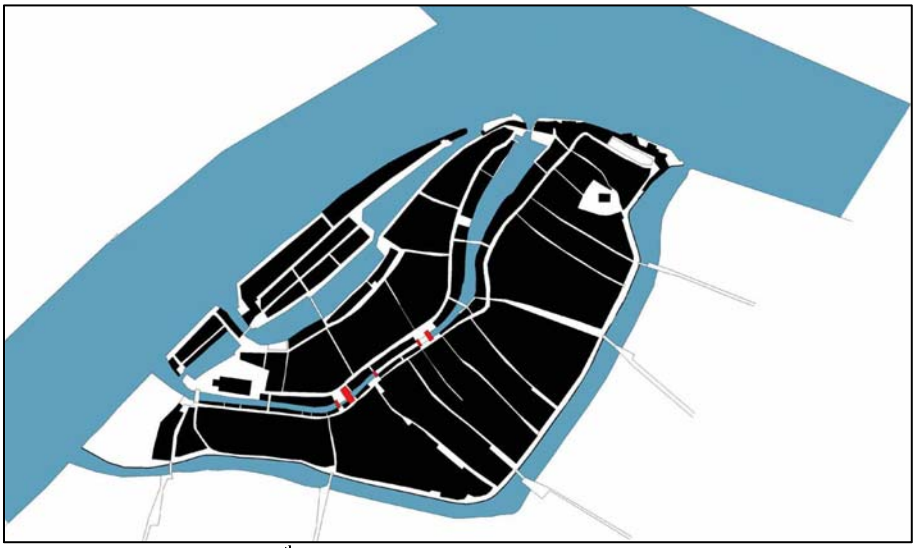

Figure I.2: Dordrecht in the 16th century.

In the centuries that followed, Dordrecht started to lose its position as an international important harbour to the city of Rotterdam. This led to major changes in the city, where for the first time in its history the city did not turn to but turned away from the water. The old city centre turned into a shopping centre and new buildings were placed on the south side of the city outside the old city limits. At this time the city was first connected to the land infrastructure. However the main entrance to the city was still by ferry across the river.

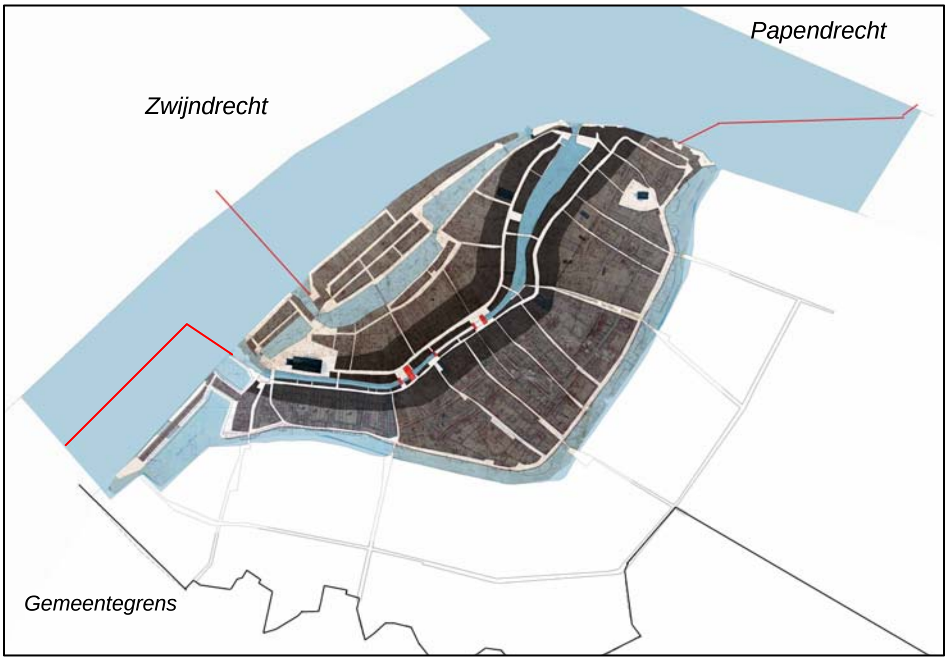

Figure I.3: Dordrecht in the beginning of the 19th century.

The main turn away of the city from the rivers came in 1872 when the railway Amsterdam – Paris was finished. This railway was the replacement of the roads which were placed under Napoleon. The railway also had a station south of Dordrecht.

Figure I.4: Dordrecht at the end of the 19th century with south of the city the railway and the station.

The realisation of this railway was only possible because there were three bridges which realisation was not possible due to lack of technology until this point in time.

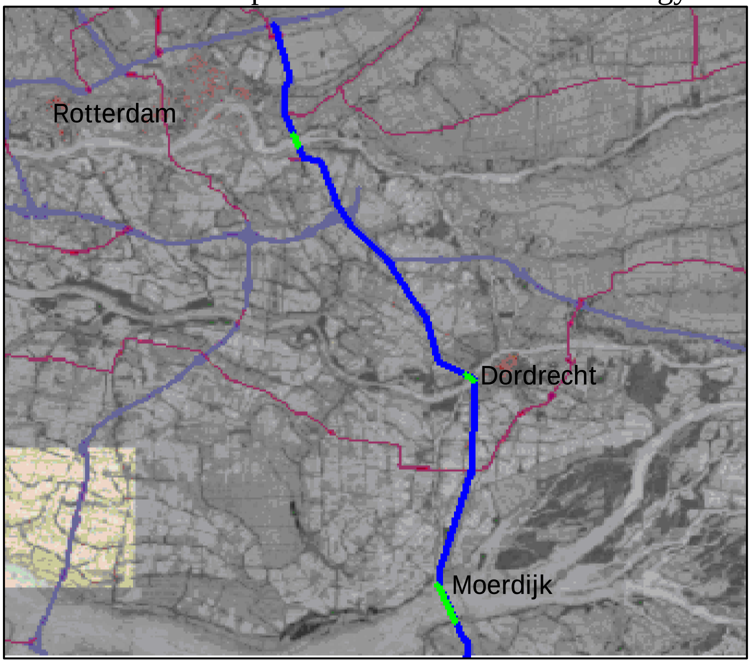

Figure I.5: Part of the railway line Amsterdam – Paris, with the location of the three bridges.

These three bridges were the top of engineering abilities in this time, because they either spanned large gaps (Moerdijk), had a great height over the river (Dordrecht) or had large towers to lift part of the bridge (Rotterdam). But besides this there was also a great architectural appreciation for these three bridges.

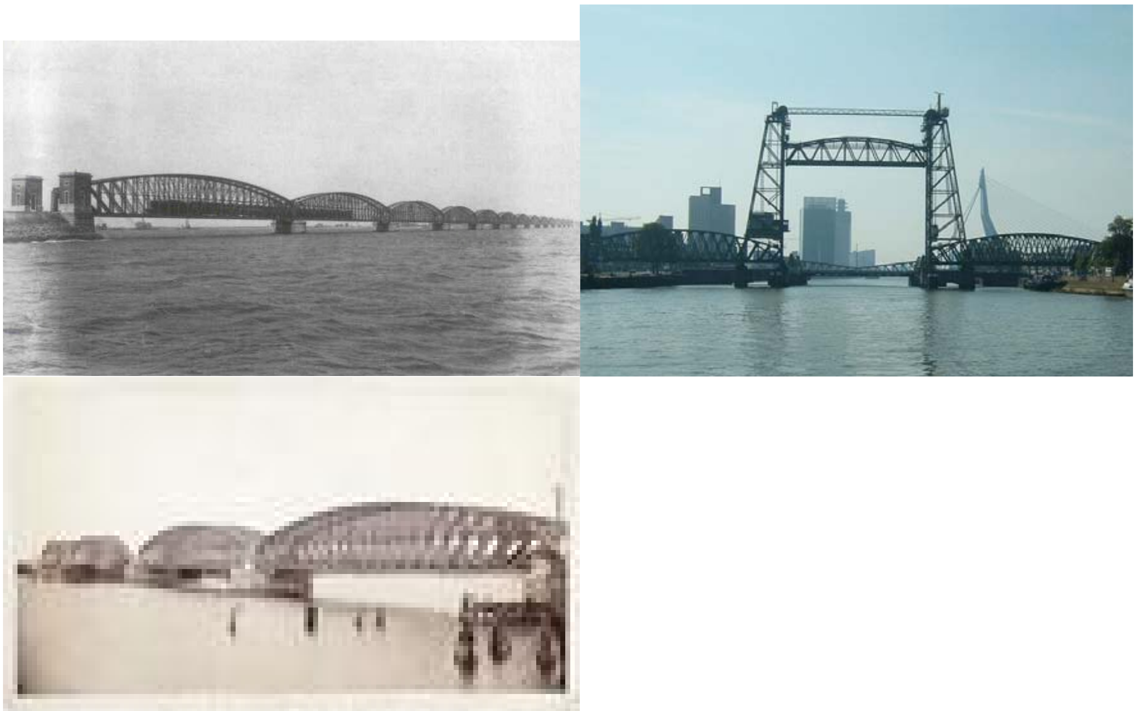

Figure I.6: The three bridges in the line Amsterdam – Paris, Moerdijk (top left), Rotterdam (top right) and Dordrecht (bottom left).

With the new railway completed, the entrance to the city shifted from the water to the land. No longer were the ferries the main entrance to the city, because now the railway took its place. This led to a further development of the city in the south beyond the old city limits. The city centre shifted away from the long line along the old Thuredrith, to the now roads which formed the connection between the station and the old centre. This led to regression in the old parts of the city centre.

Figure I.7: Dordrecht at the end of the 19^{th} century.

This process was intensified by the introduction of the automobile. The old streets of Dordrecht were to narrow for the automobile and the capacity of the ferries was limited. To solve these problems a new bridge for automobiles was placed along the railway bridge.
This meant the ferries became obsolete and the only access to Dordrecht was via land.
In the time that followed the city exploited the area between the old city and the railway, leading to a further displacement of the city centre towards the railway and further regression of the parts of the old city centre.

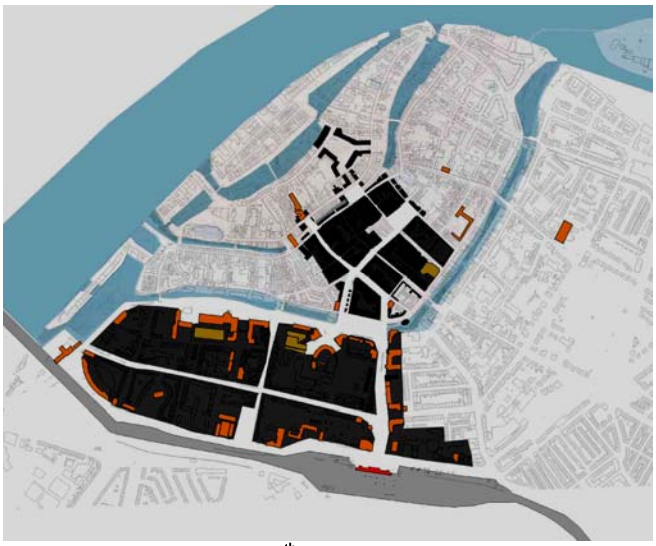

Figure I.8: Dordrecht in the 20th century, with the city centre now south of the old city.

Problem definition

Due to technological development the access to Dordrecht changed from the water to the land. This meant that Dordrecht turned away from the water, leading to regression in the old city centre and loss of contact with neighbouring cities.

Objective

To solve the problems of Dordrecht the following solution is suggested. To design a connection between Zwijndrecht and Dordrecht analogue to the situation in the old city centre. This means that a bridge will be the basis of the social life in the city and that the banks of the Oude Maas will form the new city centre. This means the bridge will be a living bridge. This concept has long been considered outdated, however due to a changed view on bridges (no longer are bridges only a means of connecting two points, but they can also be a piece of art) and a better cooperation between bridge builders and building builders it is becoming popular again.

Design

The requirements for the new design are:

- Monument
    - Visible from all around Dordrecht
    - Clearly recognisable
- Living bridge
- Connection between the river banks, not only from physical point of view
- Also buildings on the riverbanks need to be designed, because they form the basis of the use of the bridge

As a basis for the design two designs from students of the faculty of architecture are used.

First design

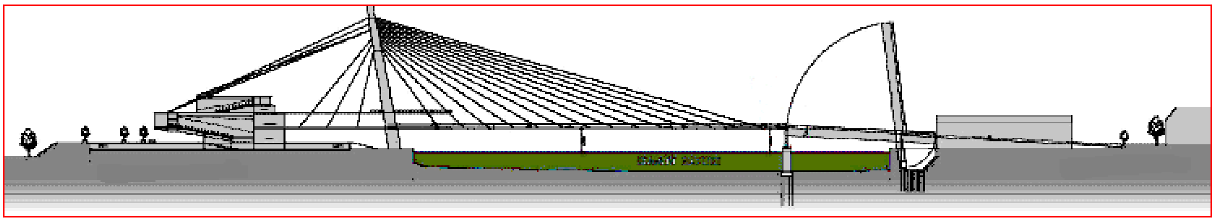

Figure I.9: Design by T. Kramer

This design is a cable-stay bridge. It places a theatre on the bank of Zwijndrecht. Under this theatre a parking garage is placed to solve the parking problems of Dordrecht. The building also acts as an anchor for the bridge. On the bank of Dordrecht a building block with shops and houses is placed. It is chosen to make this block not to high, because this would distort the characteristic view on Dordrecht.

Second design

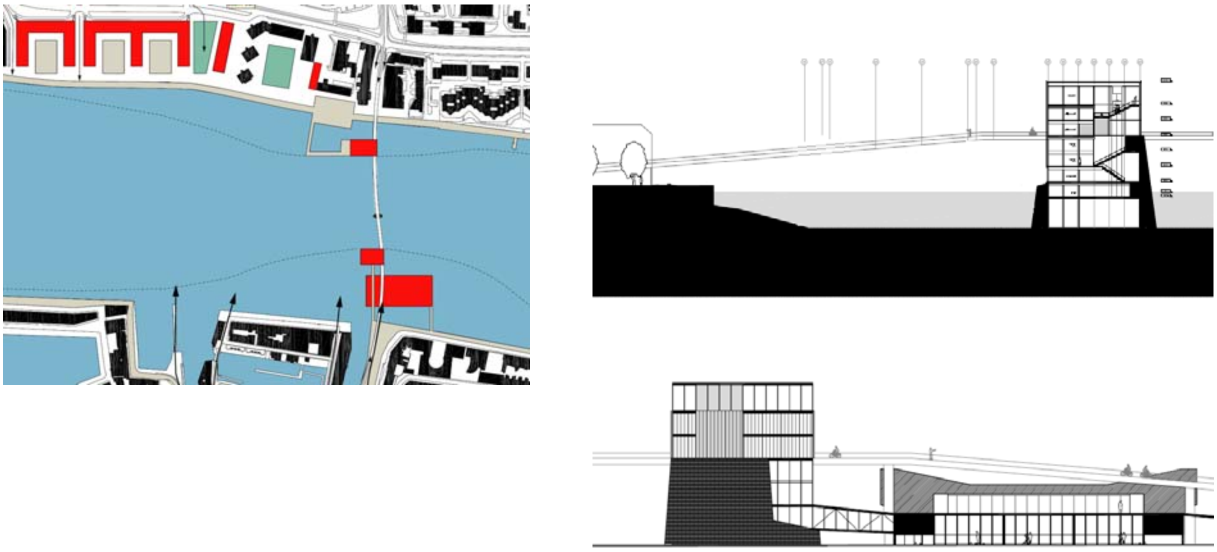

Figure I.10: Design by M. van der Meulen.

This design places buldings in the river, with a swimming pool, a health spa and a restaurant. By placing these buildings in the river, the distance between the two banks is shortened and the bridge is made an integral part of the new city centre.

Own design

The chosen design is a combination of the two designs given previously. It utilises the strong points of both designs:

- Cable-stay bridge
  - Clear distinctive shape
  - Slender construction, visually interesting
- Parking garage in Zwijndrecht
- Building as anchor for bridge
- Object in river to shorten the distance
  - Island around the pillar
  - Cuts span in three parts
- Low buildings on Dordrecht side

Also some extra choices are made:

- Wide promenade
- Activities on both banks
- Space on Zwijndrecht bank, partially green, partially shops
- Wider bridge than strictly necessary

In figure I.11 an artist impression of the final design is given.

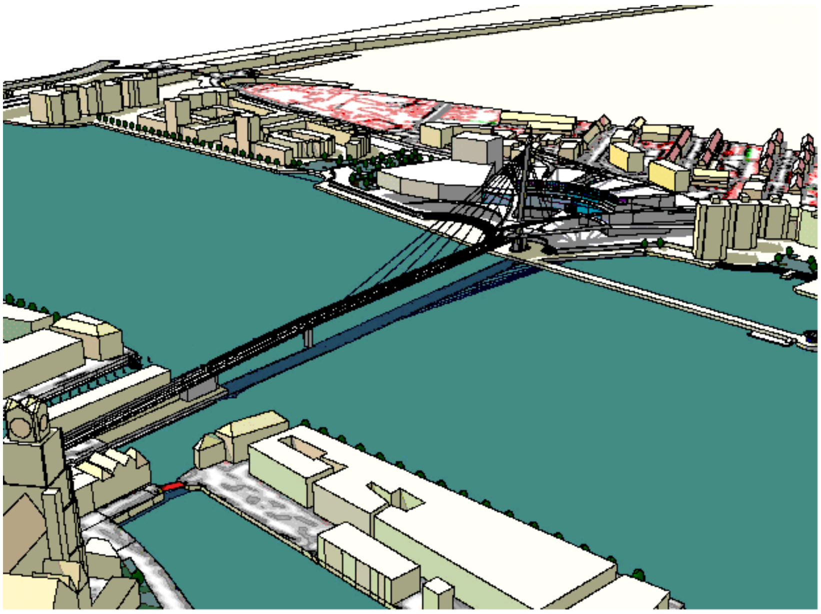

Figure I.11: Artist impression of final design.

Technical challenge

The chosen material for the bridge is steel, because this is the best material for a slender construction in this case. However this also brings some challenges into the design. Because this will be a light bridge for pedestrians the bridge will not only have to be checked for static loads, but also for dynamical loadings.
Most times the regulations for the dynamical loading will be governing over the static loading, because the regulations for dynamical loadings are very strickt.

Appendix II

In this appendix the results from the soil test are given.

The depth is given in meters -NAP.

Table I.1: Soil data.

Depth top of layer	Depth bottom of layer	Discription
Ground level	2,0 to 3,5	Sand, clay and debris
2,0 to 3,5	3,0 to 5,5	Peat with some clay
3,0 to 5,5	7,0 to 7,5	Clay with some peat
7,0 to 7,5	12,5	Clay and sand
12,5	22,0 to 24,0	Sand (Pleistocene)
22,0 to 24,0	25,5 to 26,0	Clay, silt and sand (over consolidated)
25,5 to 26,0	36,5	Clay and silt, Layer of Kedichem
36,5	-	Maximum depth

Appendix III

In this appendix tables for the locked coil cables is given.

Table II.1: Characteristics locked coil cables.

Diameter (mm)	Cross Section A (mm²)	Minimum Break Load MBL (kN)	Elastic Stiffness E-A (MN)	Weight (kg/m)
32	681	1015	112	5.6
36	862	1285	142	7.1
40	1077	1605	178	8.9
44	1303	1945	215	10.7
48	1551	2315	256	12.8
52	1841	2750	304	15.2
56	2136	3190	352	17.6
60	2452	3660	405	20.2
64	2789	4165	460	23.0
68	3149	4700	513	26.0
72	3530	5210	575	29.1
76	3933	5790	641	32.4
80	4358	6405	710	35.9
84	4805	7045	783	39.6
88	5274	7720	860	43.5
92	5764	8430	940	47.5
96	6276	9165	1023	51.7
100	6890	10050	1123	56.8
104	7452	10860	1215	61.4
108	8037	11700	1310	66.2
112	8643	12575	1409	71.2
116	9271	13480	1511	76.4
120	9922	14415	1617	81.8
124	10594	15385	1727	87.3
128	11289	16385	1840	93.0

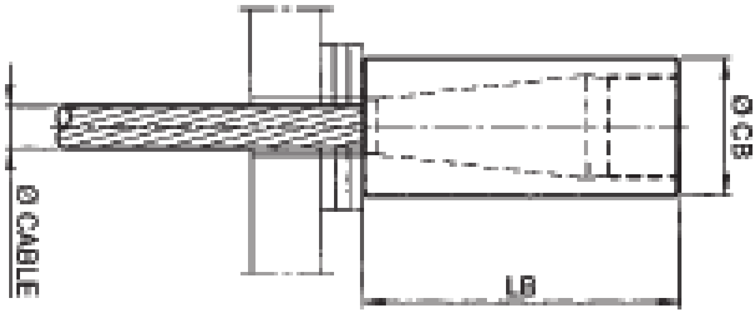

Figure II.1: Dimensions socket.

Table II.2: Required dimensions of socket

Cable Diameter (mm)	CA (mm)	LA (mm)	CB (mm)	LB (mm)
12	40	65	40	100
16	55	85	55	130
20	65	105	65	160
24	75	130	75	190
28	85	150	85	215
32	95	170	95	255
36	110	190	110	295
40	120	210	120	325
44	130	235	130	360
48	145	255	145	390
52	155	275	155	430
56	165	295	165	460
60	180	315	180	485
64	190	340	190	525
68	200	360	200	550
72	210	380	210	585
76	225	400	225	615
80	235	420	235	645
84	245	445	245	680
88	260	465	260	705
92	270	485	270	745
96	280	505	280	775
100	295	525	295	800
104	305	550	305	840
108	315	570	315	875
112	325	590	325	905
116	340	610	340	935
120	350	630	350	965
124	360	655	360	1000
128	370	675	370	1030

Appendix IV

In this appendix a figure is given showing the different weld classes.

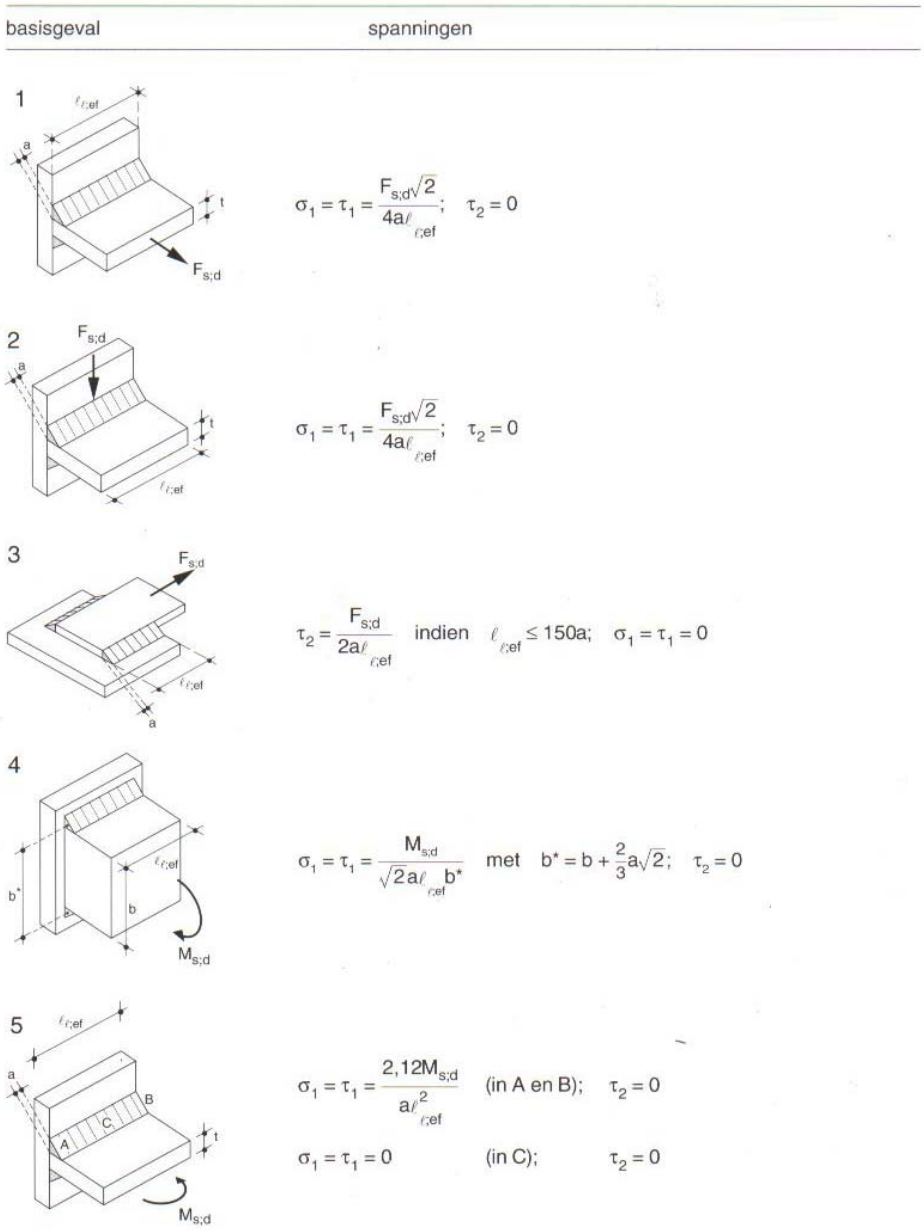

Figure IV.1: Weld types.

Appendix V

In this appendix data about the hollow channel plates is given.

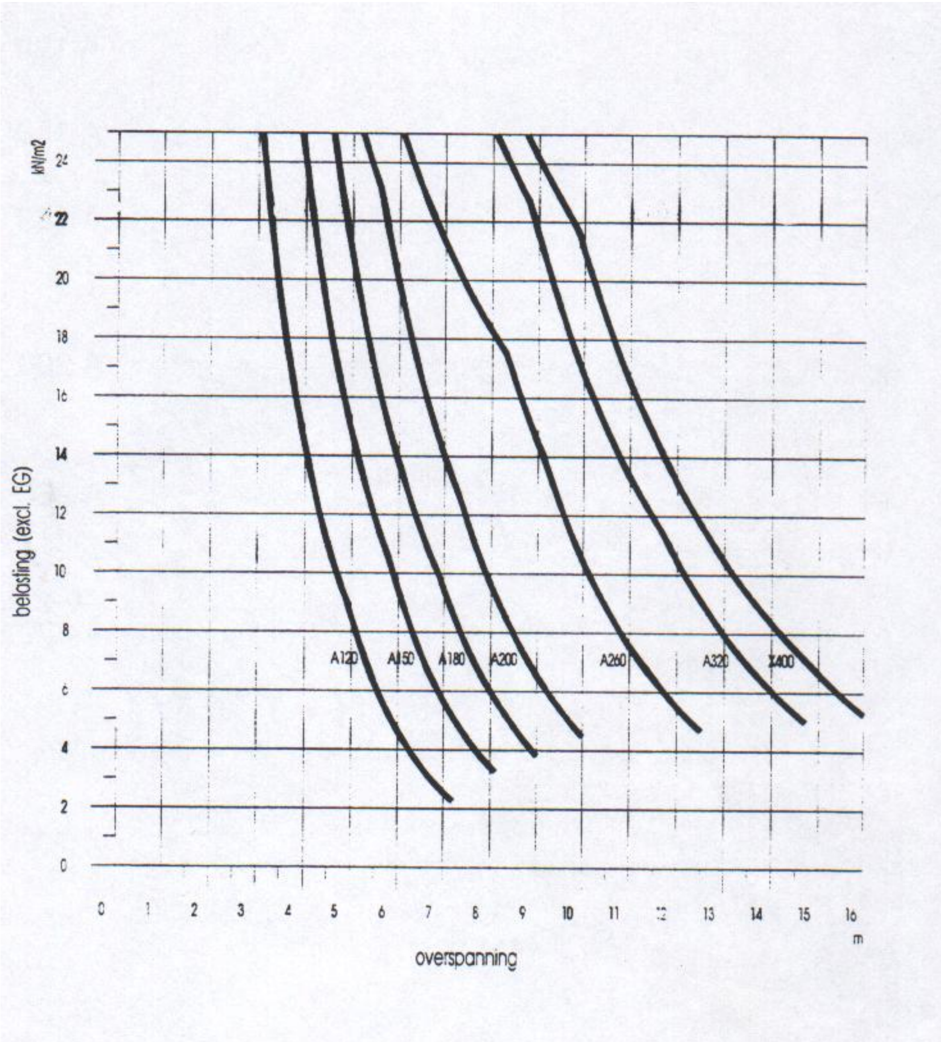

Figure VI.1:Design graph hollow channel plates

		A 120	A 150	A 180	A 200	A 260	A320	X400
dikte	mm	120	150	180	200	260	320	400
breedte	mm	1200	1200	1200	1200	1200	1200	1200
aantal kanalen	stuks	11	11	11	7	7	7	4
max. plaatlengte	m	7.20	8.10	9.00	10.00	12.50	14.70	16.00
pasplaten*	mm	300+N·100	300+N·100	300+N·100	300+N·150	300+N·150	300+N·150	462+N·275
gewicht inkl. voeg	kg/m^2	225	260	295	300	390	450	486
voegvulling	l/m	4.9	6.0	6.9	7.3	11.0	12.0	15.1
sterkteklasse*		B50	B50	B50	B55	B55	B55	B65
milieuklasse		1	1	1	1 of 2	1 of 2	1 of 2	1 of 2
brandwerendheid	min.	30 tot 60	30 tot 60	30 tot 60	60 tot 90	90 tot 120	90 tot 120	90 tot 120
betondoorsnede	mm^2	107942	124329	140230	144452	186658	215850	227600
zwaartepunt van de dsn.	mm	59.4	74.1	88.8	98.4	125.0	153.8	204.0
traagheidsmoment	10^6mm^4	161.3	301.9	498.7	668.8	1480.1	2592.2	4619.4

Figure VI.2: Technical data hollow channel plates

Appendix VI

In this appendix the design table to decide on the amount of rebar for a concrete beam is given.

Table VIII.1: Design table for rebar.

$\frac{M_d}{bd^2 f'_b}$	ψ	$\frac{x_u}{d}$	$\frac{z_u}{d}$	ω_o (%)				
				B25	B35	B45	B55	B65
10	0,010	0,013	0,99	*0,03*	*0,05*	*0,06*	*0,08*	*0,09*
20	0,020	0,027	0,99	*0,07*	*0,10*	*0,13*	*0,15*	*0,18*
30	0,030	0,240	0,98	*0,10*	*0,15*	*0,19*	*0,23*	*0,27*
40	0,041	0,055	0,98	*0,14*	0,20	0,25	0,31	0,37
50	0,051	0,068	0,97	0,18	0,25	0,32	0,39	0,46
60	0,062	0,083	0,97	0,21	0,30	0,39	0,47	0,56
70	0,073	0,097	0,96	0,25	0,35	0,45	0,55	0,66
80	0,084	0,112	0,96	0,29	0,41	0,52	0,64	0,75
90	0,095	0,127	0,95	0,33	0,46	0,59	0,72	0,85
100	0,106	0,141	0,94	0,37	0,51	0,66	0,81	0,95
110	0,117	0,156	0,94	0,40	0,56	0,73	0,89	1,05
110	0,129	0,172	0,93	0,44	0,62	0,80	0,98	1,16
130	0,140	0,187	0,93	0,48	0,68	0,87	1,06	1,26
140	0,152	0,203	0,92	0,52	0,73	0,94	1,15	1,36
150	0,164	0,219	0,91	0,57	0,79	1,02	1,24	1,47
160	0,176	0,235	0,91	0,61	0,85	1,09	1,34	1,58
170	0,188	0,251	0,90	0,65	0,91	1,17	1,43	1,69
180	0,201	0,268	0,90	0,69	0,97	1,25	1,53	1,80
190	0,214	0,285	0,89	0,74	1,03	1,33	1,62	1,92
200	0,227	0,303	0,88	0,78	1,10	1,41	1,72	2,04
210	0,240	0,320	0,88	0,83	1,16	1,49	1,82	2,16
220	0,253	0,337	0,87	0,87	1,22	1,57	1,92	2,27
230	0,267	0,356	0,86	0,92	1,29	1,66	2,03	2,39
240	0,281	0,375	0,85	0,97	1,35	1,75	2,13	2,52
250	0,295	0,393	0,85	1,02	1,43	1,83	2,24	2,64
260	0,310	0,413	0,84	1,07	1,50	1,93	2,35	2,78
270	0,325	0,433	0,83	1,12	1,57	2,02	2,47	2,91
280	0,340	0,453	0,82	1,17	1,64	2,11	2,58	3,05
290	0,356	0,475	0,81	1,23	1,72	2,21	2,70	3,19
300	0,372	0,496	0,81	1,28	1,80	2,31	2,82	3,34
310	0,388	0,517	0,80	1,34	1,87	2,41	2,94	3,48
320	0,405	0,540	0,79	*1,40*	*1,96*	*2,51*	*3,07*	*3,63*

M_d in kNm, b en d in m, f'_b in N/mm², $\psi = \omega\, f_s / f'_b$

B	25	35	45	55	65
f'_b	15	21	27	33	39

$\omega_{max} = \frac{x_u}{d} < 0{,}535$; cursief gedrukte waarden: lager dan ω_{min} of hoger dan ω_{max} !

Appendix VII

In this appendix the design table to decide on the amount of rebar for a concrete column is given.

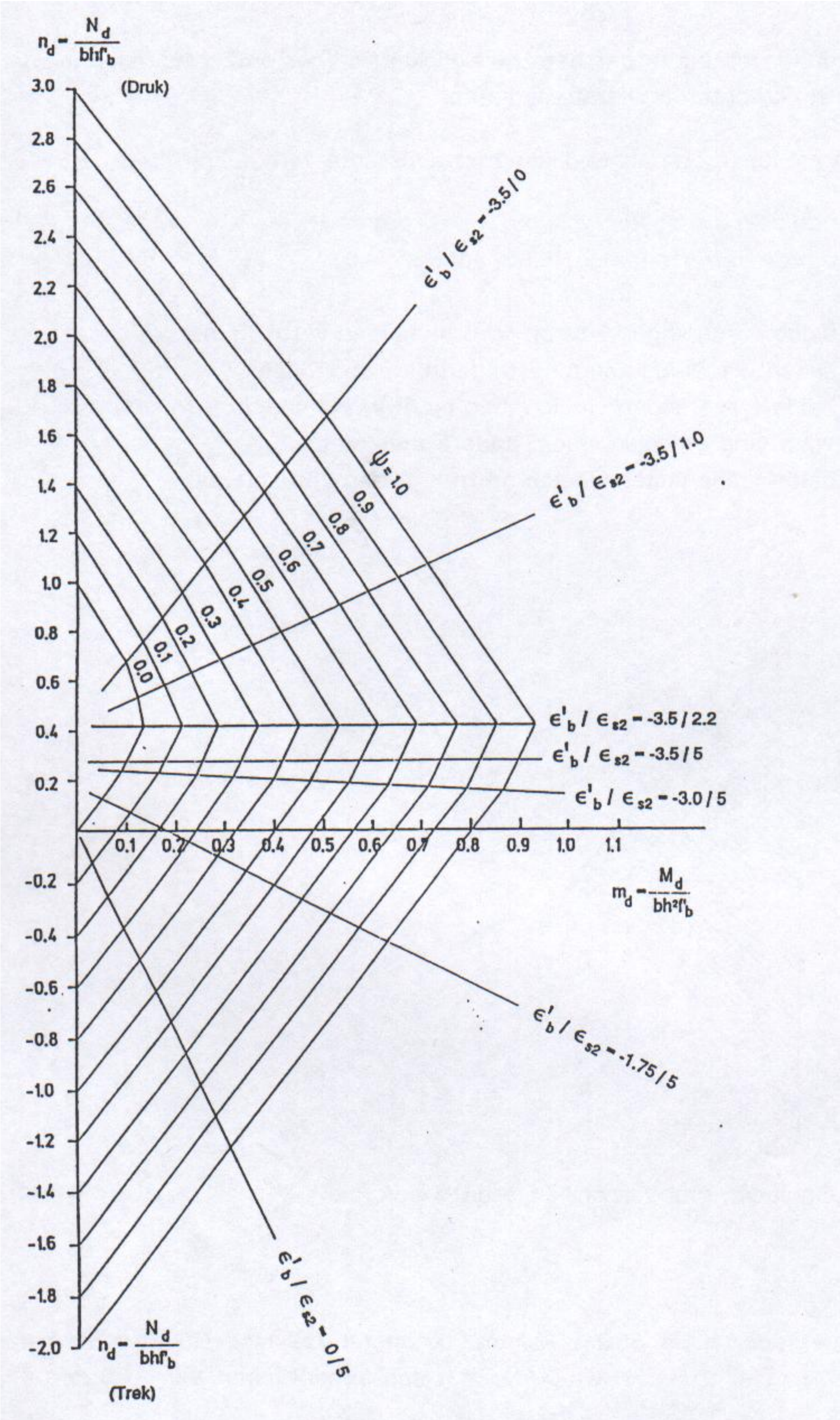

Figure X.1: Interaction diagram.

Appendix VIII

In this appendix data about the VIP plates is given.

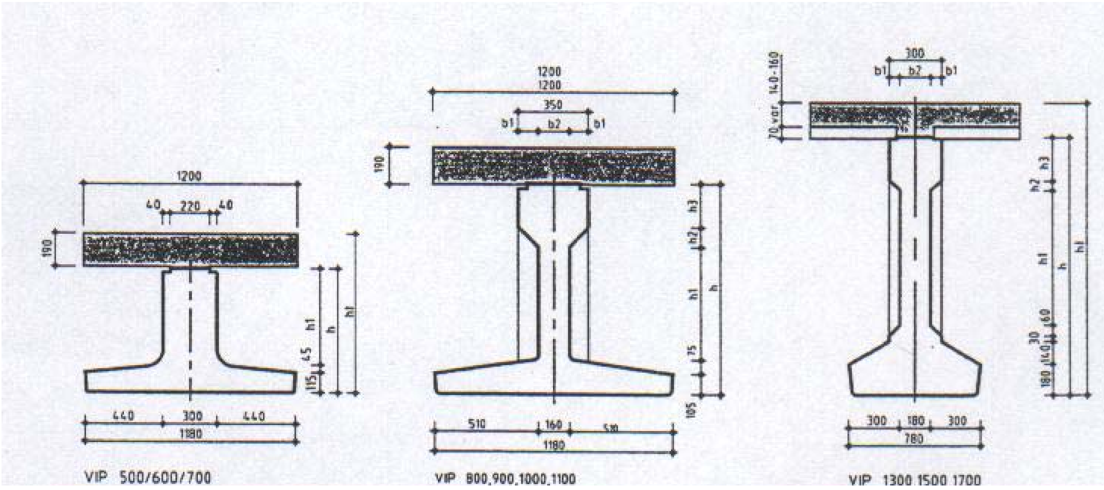

Figure XII.1: Cross-section VIP-plate.

Table XII.1: Technical data VIP-plate

Gewichten en hoogten									
VIP	eigen gewicht kN/m'		afmetingen in mm						
	midden	eind	b_1	b_2	h_1	h_2	h_3	h	h_t
500	6,75	6,75	–	–	355	–	–	515	700
600	7,49	7,49	–	–	455	–	–	615	800
700	8,23	8,23	–	–	555	–	–	715	900
800	7,76	9,93	95	160	410	100	125	815	1000
900	8,67	10,84	95	160	410	100	225	915	1100
1000	8,64	11,74	95	160	610	100	125	1015	1200
1100	9,53	12,63	95	160	610	100	225	1115	1300
1300	10,22	11,91	60	180	680	60	150	1300	1500
1500	11,11	13,87	60	180	880	60	250	1500	1700
1700	12,58	15,34	60	180	880	60	350	1700	1900
1900	12,53	26,29	160	180	1250	150	150	1900	2100
2100	17,98	28,74	160	180	1250	150	350	2100	2300
2300	19,28	31,20	150	200	1500	150	300	2300	2500
2600	19,42	29,59	100	200	2000	100	150	2600	2830

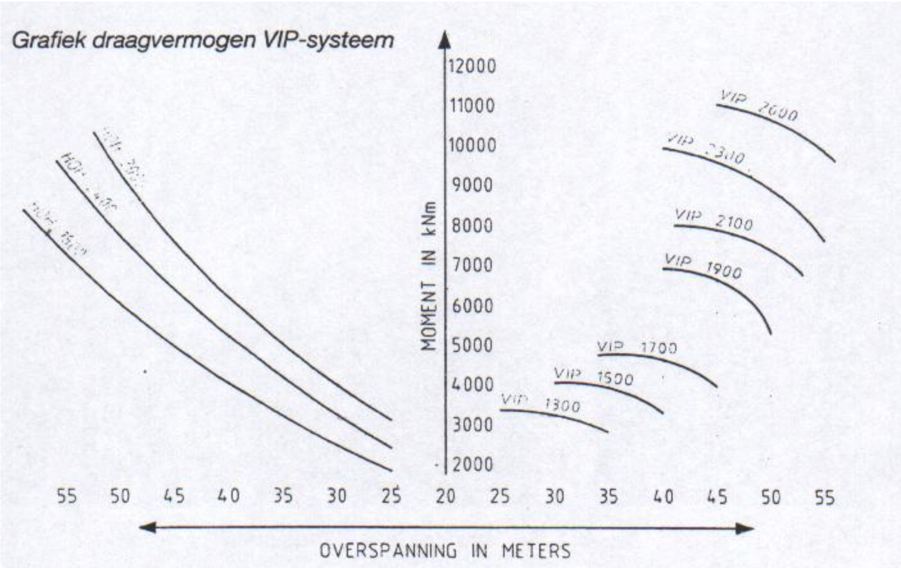

Figure XII.2: Design graph VIP-plates.