# Master Thesis

# Ultra Rapid UnderPass

## The adaptation of the URUP method for the Netherlands

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# Preface

In your hands is the final thesis "The adaptation of the URUP method in the Netherlands". URUP stands for Ultra Rapid UnderPass. With this final thesis I hope to finish my master Civil Engineering at the Delft University of Technology.

In September 2009 I started exploring this topic. Now I present the adaptation of the URUP method for the Netherlands. URUP is an innovative method of tunnel boring. The main innovations of this method are that the tunnel boring machine (TBM) starts and ends on the surface and that the TBM and the lining has a rectangular cross section.

I would like to thank Mr. Sallo van der Woude for his never ending encouragement and passion for this topic. Further I would like to thank, Mr. Johan Bosch, Mr. Cor van der Veen and Mr. Jaap Boneveld for their suggestions and recommendations during our conversations and meetings. I have learned a lot from their remarks.
Several persons from Van Hattum en Blankevoort have assisted me with parts of the research. I am very grateful for the support of Mr. Schenk, Mr. Huitema, Mr. Clephas, Mr. Bruens and Mr. Boer. I would like to pay tribute to the companies Van Hattum en Blankevoort and Obayashi for providing information for this research.
Finally I would like to thank family and friends for their support during my studies.

I have experienced this research as very interesting and hopefully this report will have a positive contribution to the development of tunneling in the Netherlands.

# Summary


With the URUP (Ultra Rapid UnderPass) method short underpasses for cyclists and pedestrians below roads and railways can be constructed. With this method a tunnel boring machine (TBM) starts on surface, bores an underpass, and ends on surface. The cross section of the underpass has a rectangular shape**.**

Contractor Van Hattum en Blankevoort (VHB) learned about this project for the first time during the ITA congress 2009. The URUP method can be useful in the Netherlands because in the coming years several hundreds of multi- level crossings will be made.. VHB invited inventor Obayashi to cooperate to adapt the URUP method for the Netherlands. This thesis is part of the adaptation plan that VHB and Obayashi explored.

The aim of this thesis is to adapt the URUP method in a technical sense, such that it can be industrially executed for short underpasses for cyclists and pedestrians below roads and railways in the Dutch circumstances.

The construction length of the enclosed part of the underpass should be shorter than 250 meters, because if this length would be exceeded additional tunnel law would come into force, which would result in additional measures. The underpass should have a socially safe character. The minimum required profile of free space for a two way cycle path and a pedestrian path is  6.00 by 2.50 meters. The maximum allowable slope for a cycle and pedestrians path is 4.0%.

Generally there is a high groundwater table present in the Netherlands. This high groundwater table causes buoyancy. A ballast layer of one meter is required to prevent buoyancy everywhere in the Netherlands.

The adaptation of URUP in the Netherlands is investigated by researching a representative case. The case is situated in Goes where an underpass below a road is made. In the recent past VHB made a design for an underpass at this location. The soil conditions can be characterized as soft and cohesive with a high groundwater table. For the construction of the road the surface is embanked with a sand layer of approximately 1 meter. Two preliminary designs are made for the Goes case. In “Goes worst situation” the groundwater table is equal to the top of the road and the maximum ballast layer is required to prevent buoyancy. In “Goes realistic situation” the groundwater table is lower and no ballast layer is required.

The elements are of prefabricated reinforced concrete, as requested by potential client ProRail. The elements are built in rings with a length of 1.0 meter. A longer length of the rings results in fewer connections; on the other hand it must be possible to transport the elements and to correct bore deviations. Two types of rings are applied, mirrored to each other –  this gains more building accuracy. The joints in the ring are situated where the bending moments are relatively low. This is roughly at ¼ and ¾ of the span in the top slab. In the Goes case a ‘steel box solution” is chosen as connection method. This box is embedded in the prefabricated segments, and two bolts can be placed to connect the elements together. In the ring joint the same principle is applied, but the connections are executed in a lighter variant. The connections are part of the definitive solution.

The costs for the URUP method are compared with the costs of the conventional methods in the Dutch circumstances. This compassion shows that the method URUP is two times more expensive as the conventional method.

Based on this research it can concluded that it is technically possible to adapt the URUP method for short underpasses for cyclists and pedestrians below roads and railways in the Dutch circumstances. But this method is only of economical interest when large number of underpasses need to be constructed in a short time and a minimal hindrance is of the essence.

# Samenvatting

Met de methode (Ultra Rapid UnderPass) kunnen korte onderdoorgangen voor fietsers en voetgangers onder wegen en spoorwegen gebouwd worden. Met deze methode start een tunnelboor machine (TBM) op maaiveld, boort een onderdoorgang, en eindigt weer op maaiveld. De dwarsdoorsnede van de onderdoorgang heeft een rechthoekige vorm.

Bouwonderneming Van Hattum en Blankevoort (VHB) nam voor het eerst kennis van deze innovatieve methode tijdens het ITA congres in 2009. De URUP methode kan de komende jaren een relevante bouwmethode worden omdat er vele spoor onderdoorgangen ongelijkvloers gemaakt zullen worden. VHB heeft Obayashi uitgenodigd samen de toepassing van de methode URUP in Nederland te onderzoeken. Deze thesis is onderdeel van het ontwikkelingsplan dat VHB en Obayashi hebben opgezet met als doel het uitvoeren van de methode URUP in Nederland.

Doel van de thesis is onderzoeken of het mogelijk is om de methode URUP geïndustrialiseerd toe te passen voor korte onderdoorgangen voor fietsers en voetgangers bij onderdoorgangen onder wegen en spoorwegen in de Nederlandse omstandigheden.

De maximale lengte van een onderdoorgang in Nederland is 250 meter, is de onderdoorgang langer dan treedt de tunnel wet in werking, Welke als gevolg heeft dat er aanvullende maatregelen genomen moeten worden. De onderdoorgang moet sociaal veilig zijn, dit betekent onder andere dat de minimale afmetingen van het profiel van vrije ruimte is 6.00*2.50 meter moet zijn. De maximale hellingen van de toeritten zijn 4.0%.

In het algemeen heeft het grondpakket in Nederland een hoge grondwaterstand. Door deze hoge grondwaterstand kan de onderdoorgang opdrijven. Er een ballast laag van een meter nodig om overal in Nederland opdrijven tegen te gaan.

De toepassing van URUP in Nederland is onderzocht door het uitwerken van een representatieve case. De case behelst het maken van een onderdoorgang onder een weg in Goes. In het recente verleden heeft VHB een ontwerp voor een onderdoorgang op deze locatie gemaakt. Het grondpakket in Goes kan omschreven worden als zachte cohesieve grond met een hoge grondwaterstand. Voor het maken van de weg is het grondpakket in het verleden opgehoogd met ongeveer 1 meter zand. Er zijn twee voorontwerpen voor deze case gemaakt. In “Goes worst situation” is de grondwaterstand naar de bovenkant van het fietspad verhoogd. In dit geval is de maximale ballast laag nodig om opdrijven tegen te gaan. In de case “Goes realistic situation” is geen ballast laag nodig om opdrijven tegen te gaan.

De elementen bestaan uit gewapend beton, dit omdat potentiële klant ProRail dit eist. Hoe langer de lengte van de ring hoe minder verbindingen noodzakelijk zijn. Aan de andere kant moet het ook mogelijk zijn om de elementen te transporteren en boor afwijkingen te corrigeren. Daarom is er gekozen voor elementen met een lengte van 1.0 meter.
De verbindingen zijn gesitueerd op plaatsen waar de momenten relatief laag zijn. Dit is ongeveer op ¼ en ¾ van de overspanning. In case Goes is er voor een “stalen box verbinding” gekozen. Deze box is ingestort in de prefab elementen. Later worden twee bouten geplaatst die de elementen verbinden. In de ring voeg is het zelfde principe gehanteerd, alleen zijn de verbindingen lichter uitgevoerd.

De kosten voor het URUP methode zijn vergeleken met de kosten voor de conventionele methode in de Nederlandse omstandigheden. Het resultaat van de vergelijking is dat de methode URUP twee keer duurder is als de conventionele methode.

Met dit onderzoek kan geconcludeerd worden dat het technisch mogelijk is om de methode URUP geïndustrialiseerd voor korte onderdoorgangen onder wegen en spoorwegen in Nederland toe te passen. Maar dat het alleen rendabel is als er veel onderdoorgangen gemaakt moeten worden en er weinig hinder gewenst is.

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**Annexes**

# Part I

# Introduction to the URUP method

# 1. Introduction to MSc thesis

## *1.1 Introduction to the URUP method*

With the Ultra Rapid UnderPass (URUP) method a tunnel boring machine (TBM) starts on surface, bores an underpass and ends on surface. The cross section of the underpass has a rectangular shape. The lining of the underpass is constructed inside the shield of the TBM. There are two main differences compared with a conventional TBM project:

- The TBM starts and ends at the surface
- The underpass has a rectangular shape

A detailed description of the URUP method and its different applications is given in chapter two.

## *1.2 Motivation*

In the coming years the Dutch rail association ProRail, responsible for building and maintenance of the Dutch railways, is intending to use the railway system with a higher frequency. One of the ideas is “Trains without a timetable”. This means that on some crowded railways a train will ride every 6 minutes in each direction. The consequence of this innovation is that the railway crossings will be closed very often. Therefore ProRail receives a subsidy to make the railway capable for trains in a higher frequency, one of the measures that will be taken is to make railway crossings multi-level [4].

When a bicycle and pedestrian crossing is made into a multi-level crossing, the traditional building method generates a lot of hindrance. The railway must be closed for a while and this causes hindrance for customers. This becomes more and more unacceptable every year. The URUP method is an alternative to prevent hindrance for civilians, since closure of the railway is not necessary anymore during the construction of the underpass.

In crowded cities an underpass below an intensively used road is sometimes requested. With a conventional building method this intensively used road must be broken for a certain period. With the URUP method obstruction of the road is not necessary anymore.

The URUP method needs less temporary structures. This results in a shorter building time compared with the conventional method. It also results in less $CO_2$ emissions [10], fact that is has become more important over the last years and whose importance will further increase in the coming years.

In conclusion, the cause for adapting the URUP method in the Netherlands consists out several reasons. The main reason is the increasing demand for multi-level bicycle and pedestrian crossings, together with the fact that hindrance during the construction time is becoming more socially unacceptable. Another important reason is that the building method is faster then the conventional method [24].

## 1.3 MSc thesis in a broader sense

The Dutch contractor Van Hattum and Blankevoort (VHB) first learned about the URUP method during the International Tunnelling and Underground Space Association (ITA-AITES) World Tunnel Congress 2009 in Budapest. It was presented by the Japanese contractor Obayashi. VHB invited Obayashi to cooperate to adapt the URUP method for the Netherlands. They made the following plan to adapt the URUP method for the Netherlands.

| Phase | Content | Start date | End date |
|---|---|---|---|
| Phase 0 | Exchange of information | May 2009 | August 2009 |
| Phase 1 | Feasibility study of URUP for the Netherlands | September 2009 | March 2010 |
| Phase 2 | market investigation | April 2010 | September 2010 |
| Phase 3 | launching customer and pilot project realization | October 2010 | May 2011 |
| Phase 4 | exploitation of URUP in the Netherlands | June 2011 | And further |

*Table 1: planning of URUP method adaptation for the Netherlands*

First there was some exchange of ideas and intentions between Obayashi and VHB. In phase 1 the feasibility of the URUP method in the Netherlands will be investigated. This MSc thesis covers a part of this feasibility study, but the thesis itself investigates only the adaptation, and not the total feasibility. For a feasibility study topics as client demand and social acceptance should be investigated in detail, which is not done in this thesis.
In phase 2 there will be a market investigation. Whether there are potential interested clients will be investigated. In phase 3 there will be a pilot project realized and in phase 4 the URUP method will be executed many times in the Netherlands. In each phase both parties have a to take a go/ no go decision, so the cooperation can be aborted if one of the parties does not see enough perspective.

## 1.4 Goals of the MSc thesis

The goal of this thesis is defined in the following way:

*Adapt the URUP method in a technical sense, such that it can be industrialized executed for short underpasses for cyclists and pedestrians below roads and railways in the Dutch circumstances*.

This results in the following sub questions:

- How does the URUP method work and in what kind of circumstances is it already applied?

- What are the required dimensions of the construction so that the URUP method everywhere in the Netherlands in an industrialized way can be applied.

- What is the most effective way to construct the lining of the underpass, related to materials shape and connections?

- What are the costs for applying the URUP method industrialized in the Netherlands and how relates this to the conventional methods that are nowadays applied?

## 1.5 Scope and limitations of the MSc thesis

**Scope**
In this research the following aspects will be investigated for adaptation of the URUP method in the Netherlands.

The Dutch circumstances
- Soil conditions
- Price level in the Netherlands
- Dutch law and regulations

Geometric aspects
- Required envelope of free space
- Requirements for the alignment

Technical aspects
- Lining of the underpass
- Determine global TBM properties
- Foundation during the start of the TBM
- Preliminary design for a project
- Executing aspects of the URUP method

**Limitations**

The following aspects may be relevant for adapting the URUP method in the Netherlands, but are not investigated during this phase. They will be investigated in a later phase of the research.

The Dutch circumstances
- $CO_2$ emissions
- Economical exploitation
- Client demand

Technical aspects
- Detailed settlement of the surface due to the boring process
- A detailed design of the TBM

## *1.6 Methodology of MSc thesis*

**Part 1 Intoduction to the URUP method**
Part 1 explores how the URUP method  works in Japan. This is done through the study of several scientific papers written about the method. The Japanese contractor Obayashi also provided information about the method.

**Part 2 Adaptation of the URUP in the Netherlands**
In Part 2 the Dutch circumstances are investigated. The following is mainly considered:

- The normative soil conditions in the Netherlands
- Dutch regulations and guidelines

From here boundary conditions and requirements are derived which are valid for all URUP models in the Netherlands. In this part soil data is analyzed and Dutch regulations are often applied.
The case study investigates how to adapt the URUP method in the Netherlands. First the alignment and detailed cross section are determined. They are calculated to the level of a preliminary design. Finally the costs of the project are calculated.

## *1.7 Outline of MSc thesis*

This chapter is an introduction to the thesis. In chapter two the URUP method is explained in detail. In chapter three the boundary conditions for URUP in the Netherlands are explained and a case is selected. This case is worked out in the next chapters where in chapter four several lining ideas and concepts are worked out. In chapter five the preliminary design for the selected case is made. In chapter six the costs of the project are evaluated. The thesis ends with a conclusion and recommendations in chapter seven.

# 2. The URUP method

## *2.1 State of the art of the URUP method*

The URUP method is a new tunnel boring technique for underpasses. The TBM starts on the surface and push off from a launching cradle. It then bores the underpass, while the lining is constructed inside the shield of the TBM. After boring the underpass the TBM ends on surface. Therefore no start and arrival shaft are required.
The face of the TBM has a rectangular shape. One of the reasons that this is possible is the shallow situation of the underpass which results in relatively small loads.
To explain the URUP method in detail, a pilot project executed by the Japanese contractor Obayashi will be discussed. The Japanese contractor presented their innovation at the ITA congress of 2009 [9] and provided some additional information [24].

**Pilot Project, Tokyo, Japan [9], [10], [24]**
This pilot project is executed in Japan on the estate of Obayashi in Tokyo, Japan. The soil can be classified as soft with cohesion. The groundwater level is far below the surface and does not have any influence on the project.

The total length of the bored part of the underpass is 100 meters. An overview of the horizontal and vertical bore path can be found in figures 1 and 2. The TBM bores with an average percentage of 8.7% downwards to the deepest point of the underpass. A vertical curve with a radius of 100 meter is applied in order to reach the horizontal direction. Below the object that is being crossed a cover of 1 diameter is reached, in this case 2.2 meter. Once the object is passed the underpass rises up with a horizontal curve with a radius of 100 meters and a vertical curve with a radius of 300 meters is also made.
The first 20 and the last 20 meters are not made with the TBM, but are made insitu afterwards, and they are not include on the present drawing.

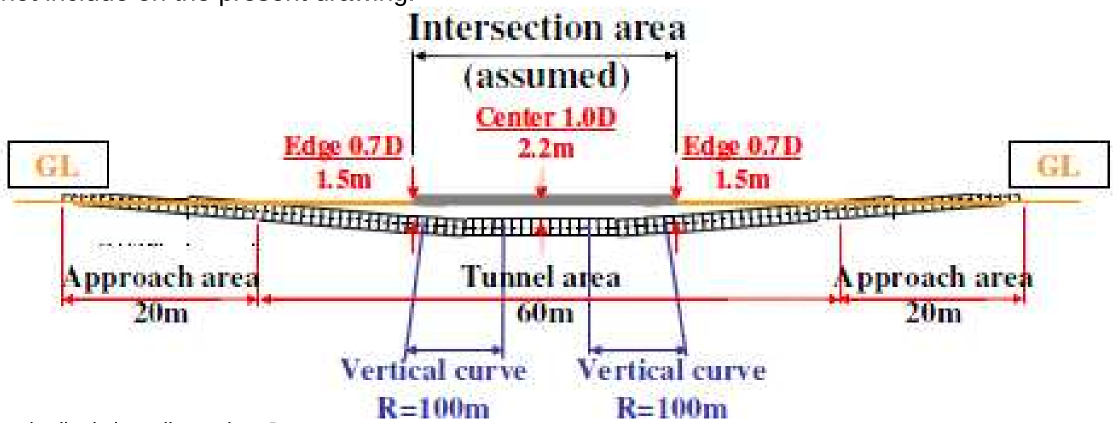



*Figure 1; longitudinal view pilot project Japan*

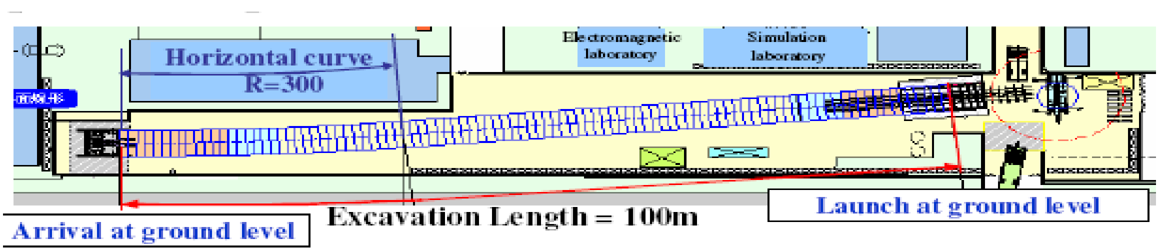



*Figure 2: top view pilot project Japan*

The TBM starts at the surface on the launching cradle. Figure 3 gives an impression of the launching cradle. The TBM will be placed on the slab at a small angle downwards. The jacks of the TBM push of at the launch cradle. The cradle is made out of steel beams which are anchored onto the concrete slab. The grout anchors are also anchored on this slab. There are in total six grout anchors that lead the forces into the soil.

Figure 3; launching cradle of pilot project

The dimension of the cross section of the steel lining is 2.00 by 4.66 meter. This is slightly smaller then the dimension of the TBM which is 2.15 by 4.80 m. The lining of this pilot project consists out of steel cassettes. An overview of the cross section of the underpass below the under passed object is shown in figure 4. In each ring a number of elements are situated, these elements have a certain length in longitudinal direction.
There is also a middle beam applied in the cross section, see figure 4. This reduces significantly the bending moments in the top slab very much and therefore a more slender top slab can be applied. The disadvantage of this solution is that the profile of free space is reduced. The cassettes are with bolts connected. The quality of the bolts is 4.6. In the longitudinal and in the lateral joint the same type of bolts are applied.

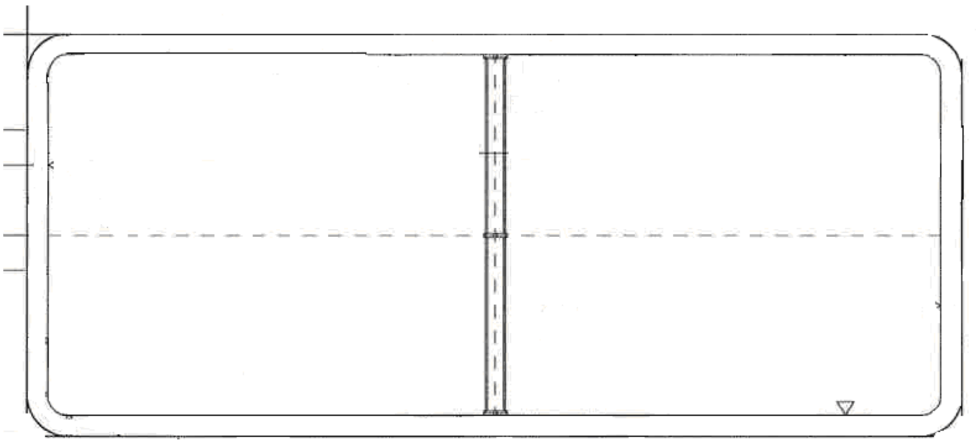


*Figure 4; cross section of enclosed part*


*Figure 5; The TBM used at the pilot project*

The TBM has a face of 2.15*4.80 meters as mentioned before, and has a length 6.00 meters. A figure of the TBM is shown in figure 5. There are two cutter heads on the front side are situated. These cutter wheels are placed on a circle that moves in a circular way. The motion of the cutter head and the circle are tuned so that a rectangular shape is bored. The TBM has two vertical bars on the outside. These are to gain stability against rotating around the x and y axis. The TBM is of the type Earth Pressure Balance (EPB) shield. The soil will be modified with foam to make it more plastic. Due to good control of the earth pressure at the bore front, the settlement of the tunnel is smaller then 10.0 millimeters.

**Advantages**
The following gives an overview of the advantages of the URUP method compared to the cut and cover method are mentioned by the inventor and executer Obayashi [9].

*Less hindrance*
The underpass is built below surface. This means less hindrance above surface for civilians. It is not necessary to close the road anymore.

*Less Temporary constructions*
There are less temporary constructions necessary. For example building pits are not necessary anymore; this allows a reduction of the cost

*Faster*
This method is faster then a conventional building method. The reduced building time can be 33% to 50% of the conventional construction time.

*Less $CO_2$ emissions*
The construction of the underpass generates less $CO_2$ emissions. Since building an underpass with the URUP method has less impact on the environment and it is more sustainable.

The first two advantages, less hindrance and less temporary constructions also exists in the Netherlands. The existence of the other two advantages is uncertain and must be proved with further research.

**Technical Issues**
Constructing the underpass in Japan also causes some technical issues. The main are mentioned below. In the Netherlands there are additional technical issues due to the high groundwater level that is present here.

*Construction of the underpass lining*
The prefab elements have to fit precisely in the lining and therefore very small tolerances are accepted. The connection methods must be strong and stiff enough to take up the loads.

*Control of support pressure of the TBM*
The exact soil conditions are uncertain, this result in the fact that steering of the TBM is difficult and not exactly predictable. If the support pressure in the shield is not well controlled this can causes settlement. Due to the thin cover the boundaries wherein the support pressure must be small. If the support pressure is not well controlled this can cause settlement.

## *2.2. Further applications of the URUP method in Japan*

The URUP method was not only invented to build short underpasses, it is also has other applications. Below some other innovations are described where parts of the URUP principles are used. Bacasue the URUP method is a new innovative method, not much detailed information is publicly available.

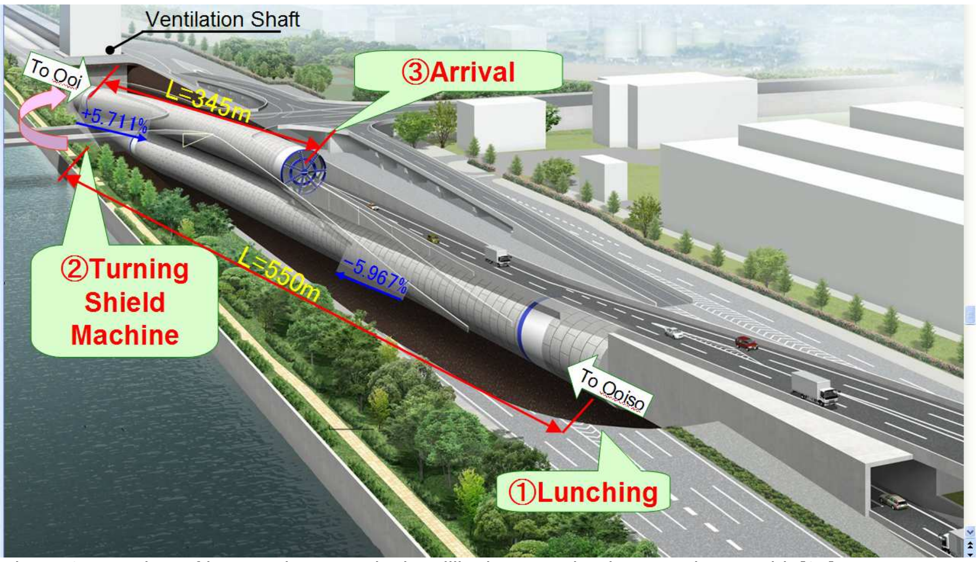


*Figure 6; overview of bore path; er wordt eigenlijk niets over het bore parth vermeld. [24]*

**Ooi Tunnel, Tokyo, Japan [9], [24]**
This project shows some of the developments of the URUP method. In this project the TBM starts on surface and bores to an existing tunnel: see figure 6. There it turns and is launched again. Then it bores up to the surface again. The total length of the tunnel is 895 meters. A large diameter tunnel is applied, the diameter of the outside of the tunnel is 12.5 meters. The average slope of the tunnel is between 5.5 and 6.0 percent, and a maximum cover of approximately two times the diameter. There are steel elements applied with a ring length of 1.7 meters. The project will be executed between June 2008 and June 2011.
This project shows that it is possible to apply the URUP principle of launching from surface with large diameter shield tunnels.

**Kawajiri Tunnel Project, Kawajiri, Japan [9], [24]**

In this project there will be boring from surface into an existing tunnel. The length of the bore path is 417 meter. The cross section is rectangular with soft edges as can be seen in figure 7. So there is some of arch working, and there is not as much space lost as in a circular cross section. The dimensions are 11.0*7.0 meter, see figure 7. The maximal cover is 4.9 meter. The tunnel is built with an open front as can be seen on the TBM in figure 8. It can therefore be concluded that the groundwater level is very low and not of any influence of the project.

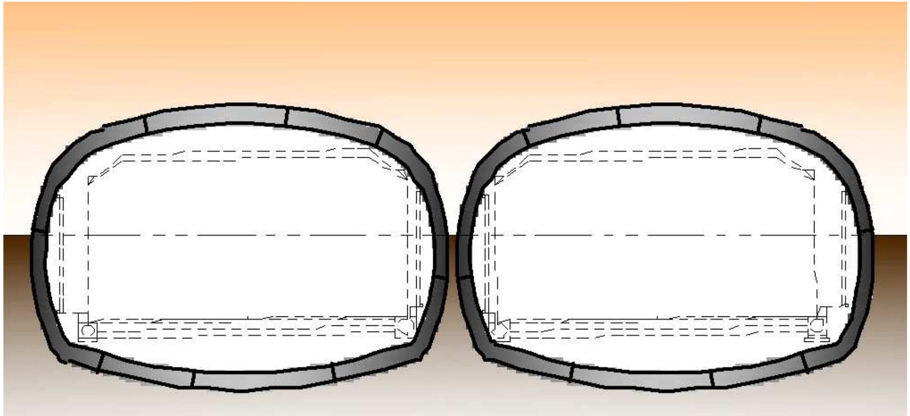


*Figure 7; cross section; of the Kawajiri tunnel project. 11.0 *7.0 meter*

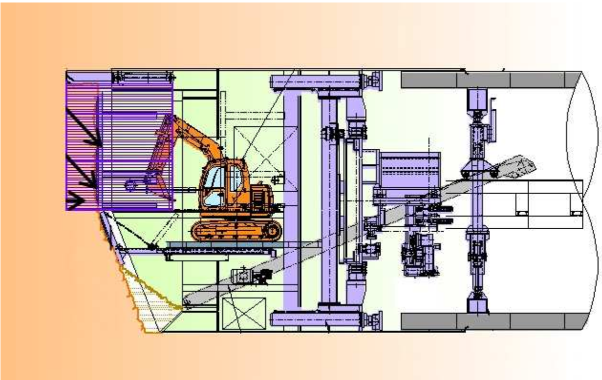


*Figure 8; overview* of the cross section

This project shows that it is possible to adapt the principle of starting on surface by large diameter tunnels. It also shows that it is possible and maybe interesting, to use shapes which are not totally circular, for bored tunnels.

**Funabashi interchange IC Project, Tokyo, Japan, [24]**
This tunnel is built with the use of the URUP TBM. The cross section is constructed in sequence shown in figure 9. First The URUP machine bores their own paths (1), and then the six different sections will be merged together by adding grout (2). After that the soil inside the grout arch will be excavated (3). Finally a tunnel results as shown below.
The lining is constructed with Steel Fiber Reinforced Concrete lining segments. This project shows that several URUP machines can be applied to build a larger diameter tunnel.

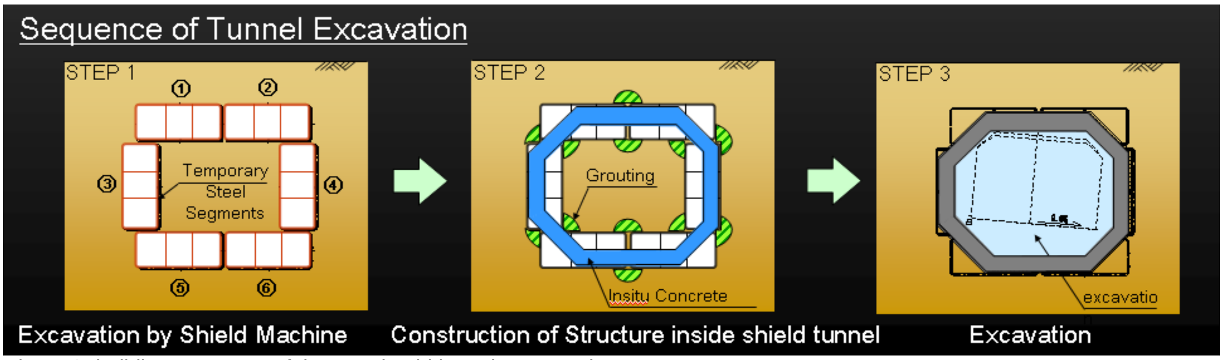


*Figure 9; building sequence of the Funabashi interchange Project*

**Evaluation**
In all the evaluated projects the soil conditions are different than in the Netherlands. The soil at the location of the projects is generally soft and with cohesion. There is also no groundwater present at most of the locations. In all the described projects some of the principles of the URUP concept are applied, below a short overview is given.

In the projects the TBM is launched from surface which is on of the remarkable innovations of the URUP methods. This is also applied by large diameter tunnels. This is an innovation that might be interesting in the future for the tunnel boring in the Netherlands.

There are different shapes applied in each project. The large diameter tunnels are circular whilst the tunnels with a smaller diameter are not. These are oval shaped or have a semi rectangle. This reduces the volume of ineffectively used space.

Another innovative idea is to merge different cross sections to construct a tunnel, this innovation seems less applicable in the Netherlands due to the joints between the different TBMS.

# Part II

# Adaptation of the URUP method in the Netherlands

# 3. Boundary conditions and case selection

## *3.1 Introduction to case selection*

To investigate if it is possible to adapt the URUP method in the Netherlands there will be a case study made. In this case study will be investigated if it is technically possible to adapt the URUP method in the Netherlands. After that the costs for making a project with the URUP method are compared with the costs for making an underpass with the conventional design.

In this chapter the boundary conditions and requirements for the case study are derived. These boundary conditions and requirements are based on the actual situation in the Netherlands regarding law, economy, culture and geotechnical situation. The following boundary conditions are determined.

- The construction must be designed with the URUP method
- The construction must be designed in a typical soil for the Western part of the Netherlands
- The Dutch economical circumstances have to taken into account
- The underpass has to fulfill the Dutch laws, standards and regulations

In paragraph 3.2 the requirements for the geometric design are derived and explained. Especially the high groundwater level in has a large influence on the design. The high groundwater level causes buoyancy and therefore preventing measures have to be taken. Therefore the magnitude of buoyancy and its influence is evaluated in paragraph 3.3. The requirements follow from the boundary conditions and the descriptions in paragraphs 3.2 and 3.3 result in the requirements that are presented in paragraph 3.5.

Base on the determined requirements a case location is selected. This case is selected out of three locations where VHB made conventional design in the past. This is done because there is more data available of these locations and the comparison of the costs gives more reliable results. The chapter ends with a detailed description of the selected case.

## 3.2 Geometric requirements

### 3.2.1 Profile of free space

The main of the profile of free space is that there is enough space for the functions in the underpass and that the underpass has a social safe character. So if the dimensions of the underpass are classified as unsafe additional measures have to be taken, for example the enlargement of the dimensions. The required space for the functions in the underpass is determined conform the CROW and ASVV regulations. The public roads in the Netherlands have to be designed conform the CROW and ASVV regulations. For the envelope of free space there are the following objects are considered.

- Cycling path two ways
- Pedestrian path

**Cyclists' path**
The CROW [5], the Dutch knowledge institute for infrastructure, traffic, transportation, and public area suggests the following profiles of free space, see table 2.

| Path | Height (m) | Width (m) |
|---|---|---|
| Cycle path, 2 ways | 2.5 | 3.5 |

*Table 2; profile of free space conform the CROW*

This is the width for the cyclists' path only. There have to be taken in account that beside both walls a "shock strip" should be taken in account. The width of this shock strip is 0.5 meter on each side.
The ASVV 2004 [2] gives the following profile of free space as showed in table 3.

| Path | Height (m) | Width (m) |
|---|---|---|
| Cycle path, 1 way | 2.5 | 2.5 |

*Table 3; profile of free space conform the ASVV*

**Footpath**
The ASVV guideline [2] is a minimum profile of free space of 1.0 *2.5 meter determined. But a width of 1.5 meter is preferred regarding to social safety.

**Emergency services**
It is an additional advantage if emergency services can take the underpass as a short cut. Therefore the height of the underpass should be enough for these services. Most of the emergency services are smaller then 2.50 meter. Except some huge fire engines that are larger then 2.50 meter. If the height is 2.50 meters or higher the majority of the emergency services can pass the underpass.

**Conclusion**
All the height requirements are fixed on 2.5 meter above the cycle path and the foot path. For the width is regarding social safety a distance of 6 meter taken in account. It is constructed as can be seen in figure 10 at the next page.

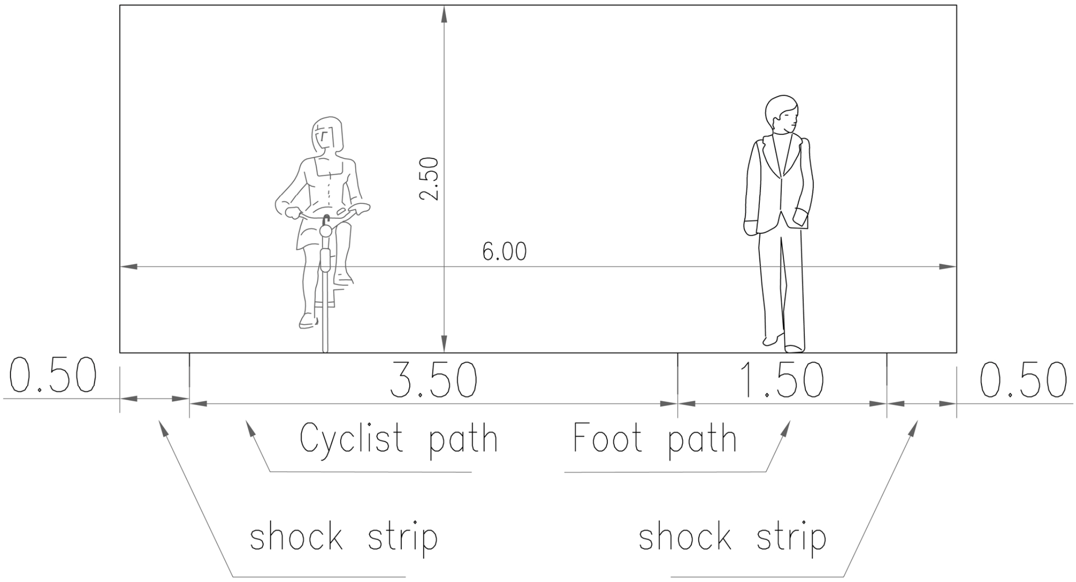



*Figure 10; cross section of the underpassing*

### 3.2.2 Longitudinal alignment

The enclosed part of the underpass must be shorter then 250 meters. Because if an object has an enclosed part longer then 250 meters it is called a tunnel and additional laws and regulations are valid [15]. Since the underpass has a smaller enclosed part, additional measures like ventilation systems and emergency routes are not necessary.

There is a certain cover necessary to prevent damage to the object that must be crossed during the construction phase. After evaluation of the pilot projects the minimal required cover is 1.5 meter in opinion of Obayashi. The cover could reduce to 1.0 meter, but then the crossed infrastructure must be closed during the construction phase [24].

The roads in the Netherlands have to fulfill the CROW and ASVV regulations as mentioned in paragraph 3.3.1. The maximum slope of alignment is determined by the ASVV 2004 [2].

**Cyclists**
The maximum allowable slope is determined by the variables presented in table 4.

| Parameter | Dependency | Assumption |
|---|---|---|
| Height | The more the height difference, the smaller the maximum allowable slope | In this case a height difference of 3.5 meter is assumed |
| Wind | If there is more wind the maximal slope reduces. | The under passing is built in an citizen environment and it is an under passing. Therefore there is assumed that there is less wind |
| Length of the ramp | If the length of the ramp enlarger a smaller slope is allowed. | There is a slope length of 100 meter assumed. |

*Table 4; longitudinal profile conform the ASVV*

The ASVV [2] gives a relation in a graph between these variables. If the assumed variables are applied a maximum allowable of slope gradient of 4.0% for cyclists is acceptable.

**Pedestrians**
For pedestrians the maximal slope is 8.3%, conform table 6.2 in ASVV 2004 [2].

**Radius**
The cycle path will be executed with a top and foot radii. These radii can be derived from the designed depth of the underpass and the chosen slope.

**Conclusion**
The normative slope is the slope of the cycle path. The maximum allowable slope for a cycle path is 4.0%.

## *3.3 Buoyancy in the Netherlands*

### 3.3.1 General

Underpasses in the Dutch soil are sensitive for buoyancy due to the soft soils and the high groundwater level. There are two important cross sections in the underpass related to buoyancy. The first cross section is on the end of the ramp where the underpass has no roof yet. The second cross section is there where the underpass is just completely below groundwater. In the first cross section it is possible to take additional measures in a later phase of the design. In the second cross section this is more difficult; therefore this situation is normative for the design and will be evaluated. In a definitive design both situations have to be evaluated and be safe against buoyancy.

First the loads and calculation model are presented, and then some possible measures are evaluated. These potential measures are:

- Ballast concrete
- Additional sand layer above the tunnel
- Heavier concrete lining
- Tension anchors

The first three suggested measures will be evaluated calculated how much of these measures are required to fulfill the safety requirements regarding buoyancy in the Netherlands. Detailed assumptions and calculations of the buoyancy phenomena can be found in annex A "Buoyancy calculation of an underpass under typical Dutch conditions".
The fourth suggested measure, the tension anchors, is not suitable in practice. The anchors should be bored trough the concrete lining into the soil. This would reduce the building speed and increase the costs. Beside that there is also an enlarged risk on leakages due to the installing.

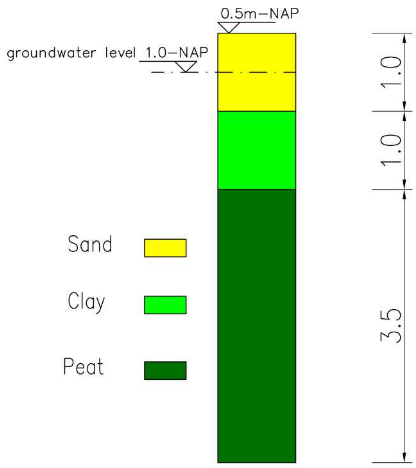



*Figure 11; typical Dutch soil profile*

The following soil profile is chosen as normative for the Netherlands, see figure 11. It is a profile that often appears in the Netherlands and the soil conditions are difficult for constructing an underpass. The profile can be described as soft soil with a high groundwater table. The top layer consists of sand and is added as an embankment for e.g. a road.

## 3.3.2 Loads, factors and calculation model for buoyancy

The following loads are presented for evaluating buoyancy, see figure 12.

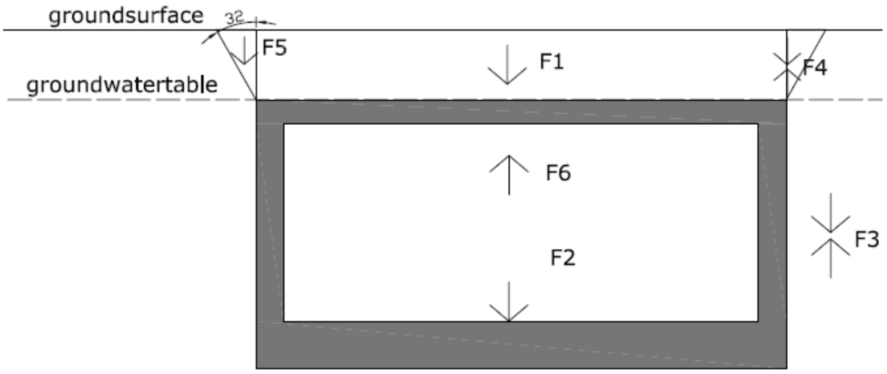



*Figure 12; graphical overview of loads*

**Loads**

F1 Weight of the soil above the structure of the under passing
F2 Weight of the under passing
F3 Friction between the soil and the under passing
F4 Friction between soil layer above the under passing and soil beside the under passing
F5 Weight of the wedges
F6 Upward force due to buoyancy

To transform representative values to calculation values load factors are applied. The following factors according to NEN 6702 [18] are applied, see table 5.

| | | | |
|---|---|---|---|
| positive force | $\gamma_p$ | 0.9 | [-] |
| negative force | $\gamma_n$ | 1.2 | [-] |
| Angle of internal friction positive working | $\varphi_p$ | 1.2 | [-] |
| Effective cohesion, positive working | $C_p$ | 1.5 | [-] |
| Required safety of friction | $Y_f$ | 1.4 | [-] |
| Total required safety | $\gamma_s$ | 1.1 | [-] |

*Table 5; applied safety factors*

The underpass is safe against buoyancy if the following requirement is fulfilled.

$$\frac{F1+F2+F5}{1.1}+\frac{F3}{1.4}>1.0*F6$$

### 3.3.3 Evaluated models

First the original situation will be evaluated on the effect of buoyancy. After that following three measures are evaluated.

- sand embankment
- heavy concrete lining
- ballast layer

**Original situation**

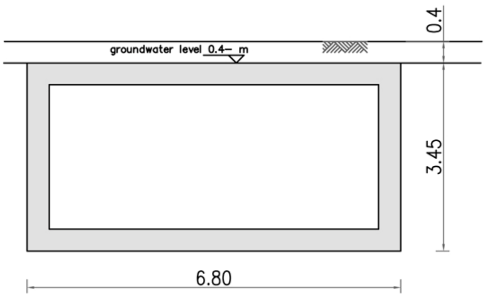



*Figure 13; original normative situation of the underpass*

This is the original situation if no additional measures are taken. The normative cross section is there where the underpass is just below the groundwater level, see figure 13. The wall thickness is assumed on 0.4 meter.

**Sand Embankment**

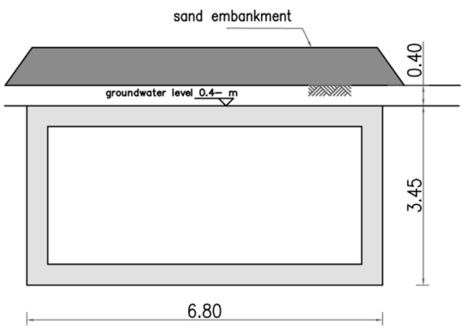



*Figure 14; sand embankment to prevent buoyancy*

In this model a sand embankment is added to make the original situation safe against buoyancy, see figure 14. The sand will be placed on the surface. There have to taken in account that not everywhere a sand embankment can be situated. This is potential measure during the construction phase.

**Heavy concrete lining**

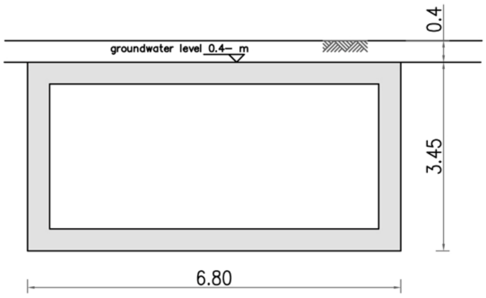



*Figure 14; heavy lining construction to prevent buoyancy*

In this model, see figure 14, the volume weight of the concrete in the original situation will be enlarged. In this more theoretical model a density of 3600 kg/m$^3$ is applied. The practical maximum volume weight of heavy concrete is 2800 kg/m$^3$. When higher weights are applied the costs will increase rapidly and the use is economical of interest. [25].

**Ballast layer**

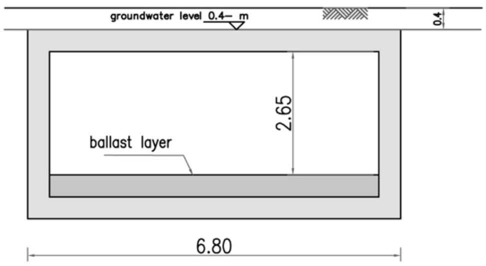



*Figure 15; lining with ballast layer*

In this model, see figure 15, a ballast layer is situated on the floor of the underpass. This increases the total height of the underpass. In the normative cross sections heavy concrete can be applied. In other sections normal concrete and sand will be applied.

## 3.3.4 Results

After evaluation of the suggested measures the following quantities of these suggested measures are required to reach a safety factor of 1.1, see table 6.

| Model | Additional measures | Safety against failure |
|---|---|---|
| Original Situation | No measures | 0.80 |
| sand embankment | Add 0.6 meter sand on surface | 1.11 |
| Heavy concrete lining | Increase the volume weight to 3600 kg/m$^3$ | 1.10 |
| Model normal lining with heavy concrete ballast layer | Add a ballast layer with heavy concrete with a thickness of 1.05 meter | 1.10 |

*Table 6; summary preventing measures buoyancy*

The following observations are notable:

- In the original situation the underpass is not safe against buoyancy.
- Using ballast concrete is a measure that can fulfill the safety requirement.
- Using a sand embankment is a measure that can fulfill the safety requirement.
- The level of the groundwater level is very important. It is assumed on 0.4 meter below surface.
- Using heavy concrete in the ballast layer and normal concrete in the lining seems a realistic solution.
- In each individual project an optimal combination of the measures will be chosen. A concept solution can be to use sand in the construction stage and heavier ballast concrete in the definitive stage.

The following risks are observed in relation to buoyancy.

- Tension water from deeper layers pushes against the under passing and generates a force upward.
- A piezometric difference between the layer where the under passing is constructed and the layer above. This can also result in an upward force.
- Due to an accident with a sewer which breaks can lead to a temporary rise in the groundwater level.
- For reasons like constructing a sewer or it is possible that the soil temporary will be removed.

.
The first two risks has to be investigated when a real case is designed. The remaining risks can be qualified as incidents and have to be mentioned in management and maintenance procedures to prevent buoyancy.

## *3.4 Description loads*

The loads that are acting on the underpass during his life cycle are described. There are two important phases during the life cycle of the underpass, the construction phase and the definitive phase.
In this stage of the design an upper boundary approach related to the loads will be made to prevent problems in a later stage of the design. This means that the loads and calculation methods will be taken conservative during the case study. Below an overview of the loads is given. The loads are only mentioned, quantization will be done in the chapter where the calculations are presented.

The following loads act on the outside of the underpass:

- Traffic over road above under passing
- Dead weight of the road over under passing
- Dead weight of the soil above under passing
- Buoyancy due to the groundwater level
- Lateral soil pressure and buoyancy

The underpass crosses a road or a railway, for both cases the magnitude of the acting force is written in the codes. The loads for roads are described in Euro code 1991-2 [16]. The loads for railways are described in NEN 6706 [19]. Comparing these values shows that the magnitude of the force on by underpassing railways is much more then for roads. In table 7 a rough comparison of the loads in the design phase between an underpass below a road and a railway is shown.

| Load | Underpass below road | Underpass below railway |
|---|---|---|
| Traffic, including point load (kN/m$^2$) | 30 | 90 |
| Construction (kN/m$^2$) | 5 | 5 |
| Soil (kN/m$^2$) | 15 | 15 |
| Own weight top slab (kN/m$^2$) | 10 | 10 |
| Total (kN/m$^2$) | 70 | 120 |

*Table 7; rough estimation loads below road and railway*

This estimation shows that the load on an underpass below a railway is much heavier then the load on an underpass below a road.

The effect of buoyancy may have a large influence on the design. The expected influence of the lateral soil pressure is marginal.

The following loads are acting on the inside of the underpass:

- Dead weight of the asphalt and other equipment
- Dead weight of users and vehicles
- Dead weight of emergency vehicle

In the Dutch code, 1991-2 [16], the magnitude of these loads is presented. The total magnitude of these loads together will be less then 20 kN/m$^2$ on the floor of the underpass. This loads are relatively low and do not have much influence on the design.

The following loads are active during the construction phase of the underpass.

- Dead weight of the TBM on the surface
- Pressure of the Jacks on the lining

The TBM has a ground surface of approximately 7*7 meter. The estimated weight is 300 ton. This results in a load of 80 kN/m$^2$ on the surface. In the definitive design the settlement due to the presence of the TBM must be investigated.

The total trust force can be up to 10,000 kN. This has to be divided over several jacks. The jacks transfer this load to the lining. On jack positions additional reinforcement for the lining is required.

**Extra ordinary situations**
Extraordinary can always occur. The two main are described below.

**Fire**
Because in the under passing only pedestrians and cyclists are allowed the risk on a crash of big vehicles is very low. Since there is not much that can burn the risk of a big fire is neglect able. Additional measures on the structure are not necessary.

**Earthquake**
The KNMI [26] measures all earthquakes in the Netherlands. From the beginning of the measurements since 1904 in a circular of 50 km of Goes is no earthquake measured. Therefore the risk of earthquakes can be neglected by designing the underpass.

So in this phase of the design the effect of this phenomena can be neglected, but in a definitive design the codes and standards should be checked.

## *3.5 Requirements*

### 3.5.1 Stated requirements

From the boundary conditions and previous paragraphs the following requirements are derived for applying URUP in the Netherlands, see table 8.

| # | Requirements |
|---|---|
| **R1** | **General** |
| R1.1 | The URUP method can be executed in soft soils |
| R1.2 | The URUP method must can be executed in soils with a high groundwater level |
| R1.3 | The underpass is social safe |
| R1.4 | Design of lifetime is at least 100 years |
| R.1.5 | The underpass is economical readable |
| R.1.6 | The underpass is built conform the Eurocode and other relevant standards and regulations. |
| **R2** | **Geometric** |
| R2.1 | Minimal height of free space is 2.5 meter |
| R2.2 | Width of free space is 6.0 meter |
| R2.3 | Maximum alignment: 4% |
| R2.4 | The maximum enclosed part is 250 meters |
| R2.5 | The soil cover below the object that will be crossed must be minimal 1.5 meter. |
| R2.6 | The shape of the outside must be of a rectangular form. |
| **R3** | **Technical** |
| R3.1 | The underpass should be safe against buoyancy |
| R3.2 | The lining must be mainly from reinforce concrete |
| R3.3 | The lining must be water tight |
| R.3.4 | The connections should be able to take up all the forces. |
| R3.5 | The maximum absolute settlement on surface is 20 mm |

Table 8; stated requirements

During the case study it is possible that additional requirements will be extracted and added.

### 3.5.2 Justification of requirements

In general the soil near the surface in the Netherlands can be classified as soft soil with a high groundwater level, as described in paragraph 3.2. Therefore is must be possible to construct the URUP method in these soils. R1.1 and R1.2 are for this reason stated.

The (CROW) Dutch knowledge institute for infrastructure, traffic, transportation, and public area [5] states that an underpass must give a feeling of social safety. Therefore the design should be social safe and requirement R1.3 is valid.

The design lifetime of tunnel is defined at minimal 100 years conform the Dutch regulations, conform NEN 6702 [18]. Therefore requirement R1.4 is stated.

If executing the URUP method is more expensive then the actual building methods, the benefits should be worth the price difference. Otherwise the URUP method is economical not interesting and will not be executed.

To make the URUP method profitable there has to be worked economical. This means that the TBM has to be applied several times, and that the most economical materials have to be applied. In a later phase a minimum number of projects will be defined where the same TBM should be applied. R.1.5 covers this.

To build an underpass in the Netherlands it must fulfill the Dutch laws and regulations. The main set of documents that have to be obeyed is the Eurocode. But also other regulations like zoning documents etc have to be fulfilled see R1.6.

**Geometric requirements**
The requirements R2.1 to R 2.5 are described in section 3.2 and follow from Dutch regulations and the fact that the underpass should be social safe.

**Technical requirements**
The URUP method should be safe against buoyancy. The magnitude of the buoyancy is described in paragraph 3.3. Aim is to adapt the URUP method to Dutch circumstances. The potential client, ProRail, demands that underpasses below the rail are not executed in steel. Therefore it is required that the lining will be executed in reinforced concrete. (R3.2)
The underpass will mainly lie below the groundwater level and has to be prevented against infiltrating precipitation. Therefore the underpass must be totally watertight. (R3.3)
The maximum settlement of the above situated road is 1/250 of his width [18]. The road has a width of 4.4 meter. Therefore a maximum settlement of 20 mm is permitted. (R3.5)

## *3.6 Case Selection*

Main goal of the case study is to investigate if the URUP method is technical feasible in the Netherlands. The second goal of the case study is investigating if the URUP method can be economical interesting in the Netherlands.
For the case selection, it is important that VHB made a conventional design for that location in the past. Because then there is more data relating to the soil profile, costs etc available. This increases the quality of the case study and in a later phase of the cost comparisons.

As mentioned in chapter 1, there is expected that in the future a lot of underpasses below the railroads have to be built. Therefore it should be pleasant to select a case that underpasses a railroad. The last few years VHB made several designs for short underpasses. All of them were designed below roads. Therefore a case below a road is selected. On the following locations VHB made designs for underpasses.

- Delft
- Goes
- Uden

These locations will be evaluated on the soil profile and on the length of the underpass. The soil profile must have relatively difficult conditions to construct an underpass, but must also be representative for the Netherlands. The length of the underpass has an influence on the feeling of social safety. Therefore the length of the underpass should not be to long.

For a good comparison between the cases the profile of free space will be kept equal. The dimensions are shown in figure 17. During the elaboration of the case study it is possible that this profile will be adapted.

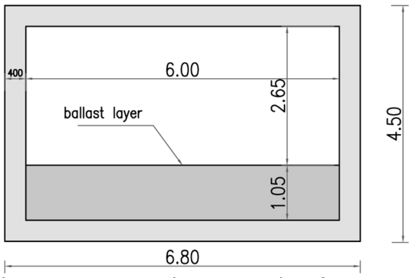


*Figure 17; assumed cross section for case selection*

**Evaluation**

The cases are evaluated on the before mentioned aspects. A detailed evaluation can be found in annex B, "Determining Boundary conditions, requirements and case selection ". The results are shown in table 9.

| Evaluated Requirements | Delft | Goes | Uden |
|---|---|---|---|
| **Soil conditions** | | | |
| Soil profile | Soft soil, mainly clay, peat | First 6 meters Sandy clay and peat, after 6 meters Sand | Mainly sand |
| GWL | 1 meter below surface | 1 meter below surface | 4 meter below surface |
| **Geometric** | | | |
| Length of the ramp (m) | 120 | 78 | 107 |
| Length of roofed ramp (m) | 54 | 30 | 40 |
| Length of horizontal part (m) | 100 | 28 | 64 |
| Total roofed (m) | 207 | 88 | 144 |
| **Social safety** | Insufficient | Good | Acceptable |

*Table 9; results evaluation case selection.*

The soil profile in **Uden** is not representative for the Western part of the Netherlands. Therefore this case is not suitable. However the length and alignment of the tunnel are good for the case.

The soil profile in **Delft** is very representative for the Western Part of the Netherlands. The underpass in Delft is too long. With more then 200 meters it has almost the length of a tunnel. In the aspect of social safety the length comparing to the cross section is problematic. Therefore the underpass in Delft is not suitable.

The soil profile in **Goes** is a representative and soil profile for the Western part of the Netherlands. Especially when in the upper layers some soft soils are assumed. The underpass is not so long, so the social safety is not a problem. Therefore the case of Goes is selected.

## 3.7 Description case Goes

### 3.7.1. General

The city Goes is located in the South West part of the Netherlands. The southwest part of the Netherlands is a delta area. The soil surface is very flat, and consists in general out of clay and has a relative high groundwater table. The land is surrounded by sea, so it can be relative windy in Goes.

The city Goes is very well equipped for bikers, like the whole Netherlands. A lot of underpasses are situated in the city to give the cyclists their own location on the road. The underpass in the case study is situated below road Ringbaan West, and is parallel to the Troelstralaan, figure 18. With this underpass the crossing will be of a multi-level. This results in more safety and less traffic congestion.

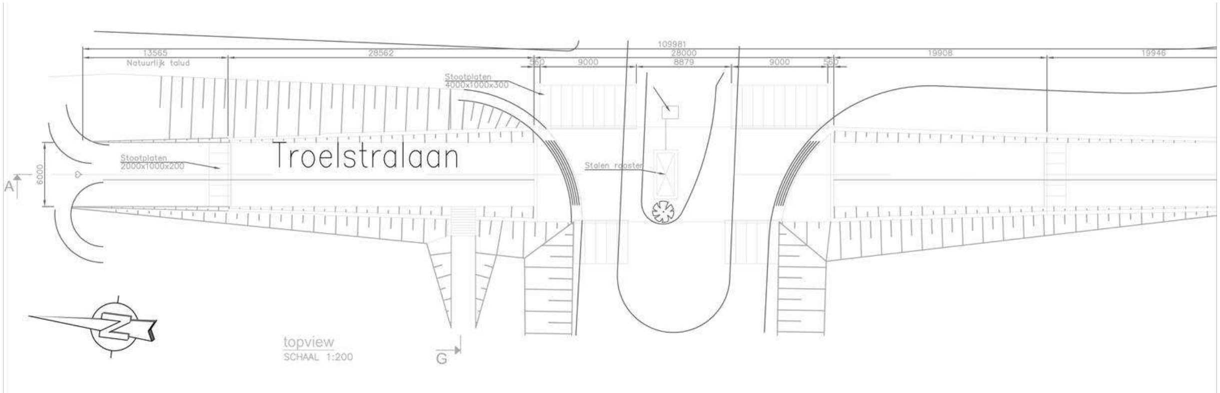



*Figure 18; overview of underpass at Ringbaan West and Troelstralaan.*

### 3.7.2. Soil profile

The crossing between the Troelstralaan and the Ringbaan West is a typical crossing that can be found in the Netherlands. The main road, Ringbaan West, is built on an embankment of sand. In general the soil consists out of soft layers. A stiff sand layer can be found approximately 10 meter below the surface. The maximum groundwater level is -0.38m NAP. The properties of the layers are as follow:

The layers have got the following properties, table 10, the shape of the layers can be found in the drawing in annex C, “soil profile case Goes” [7], [22].

| Layer | name | Admixture | Consistency | $Y_c$ | $Y_{sat}$ | $Q_c$ | N value | E | φ | c' |
|---|---|---|---|---|---|---|---|---|---|---|
| # | | | | $kN/m^3$ | $kN/m^3$ | MPa | (-) | MPa | Degree | kPa |
| 1 | sand | Clean | Loose | 17 | 19 | 5 | 18 | 25 | 30 | - |
| 2 | clay | little Sandy | Medium | 18 | 18 | 1.5 | 5 | 3 | 22.5 | 10 |
| 3 | Peat | Not preloaded | Soft | 10 | 12 | 0.1 | 0.5 | 0.2 | 15 | 5 |
| 4 | clay | clean | Medium | 17 | 17 | 1 | 5 | 2 | 17.5 | 10 |
| 5 | sand | clean | Medium | 18 | 20 | 15 | 45 | 75 | 32.5 | - |
| 6 | sand | clean | Dense | 19 | 20 | 25 | 80 | 125 | 35 | - |
| 7 | clay | clean | Medium | 19 | 20 | 2 | 9 | 4 | 17.5 | 25 |

*Table 10: overview soil properties*

Where:
$Y_c$ volume mass dry
$Y_{sat}$ volume mass saturated
$Q_c$ cone resistance from CPT test
N value derived from the CPT test
E modules of soil deformation
φ angle of internal friction
C’ cohesion

# 4 Lining concepts

## *4.1 Introduction*

### 4.1.1. General

In this chapter several lining concepts are discussed in a brainstorm session [23]. Some models are theoretical study models and suitable for practice. Others are more practical and suitable for construction.

During the evaluation of the lining concepts the upper boundary circumstances are taken in account. This means that an additional height of 1.05 meter for the ballast layer is reserved, so the total inner height is 3.70 meter. An overview of the cross section is shown in figure 17.

First some considerations and additional requirements for the lining concepts are mentioned. And then some longitudinal concepts are evaluated. After that the exact shape of the elements in the ring is determined. Followed by an evaluation of potential connection methods, to connect the elements together. The chapter ends with a summary of the selected lining concepts.

### 4.1.2. Considerations for lining concepts

**Building method**
Since the underpass will is built inside the shield of the TBM, the element supply must be through the already constructed part of the underpass. Inside the TBM the elements must be rotated and placed. So the size of the elements is limited due to the size of the TBM. To secure the building progress it is preferable to make connections between the elements that can quick and easy constructed.

**Loads and bending moments**
In the definitive phase, the loads have the shape that is presented in figure 19a. This results in the distribution of bending moments as can be seen in the figure 19b. The bending moments are relatively low at ¼ and ¾ of the span, therefore it is preferable to situate the connections between the elements at this locations.

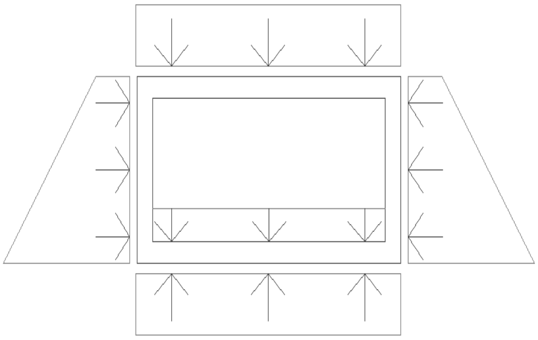


*Figure 19a; overview of the load shape*

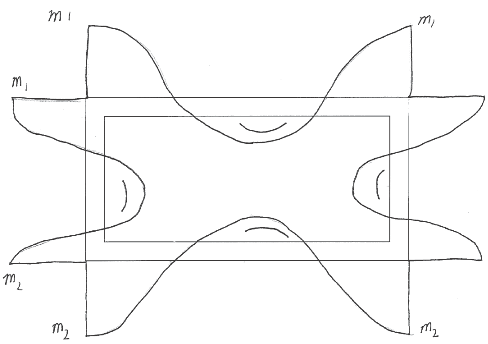


*figure 19b; overview bending moments distribution*

**Element design**
During the design of the elements the following aspects are taken in account and explanation is given below.

- Elements must have the same magnitude of weight
- As less elements as possible
- A small keystone is preferred
- The length-width relation should be well
- Small dimensions and installing tolerances are allowed

The elements are transported from the plant to the construction site and inside the TBM, the erector lifts and rotate the elements. So for logistics processes it is preferred if the elements have the same magnitude of weight.
Since less elements result in less connections and therefore lower costs, the project is served with less elements.
By prefab elements it might be hard to fit the last element, so for that reason very small dimension tolerances are allowed. If the last element is has a smaller size it can be easier to fit this element. Therefore a smaller key stone element is preferable.
If the width of the element is not large enough, it might happen that the stresses due to the loads cannot be divided well. To prevent this it is important to have a good length-width ratio.

**Water tightness**
Since the underpass lies partially below the groundwater, and infiltrating precipitation water must be kept outside the underpass, the underpass must be watertight. By making the underpass watertight the following three items have to be taken in account:

- Each individual element
- Lateral joint
- Longitudinal Joint

To secure water tightness in each individual element, a pressure zone is present. This pressure zone depends mainly on the reinforcement percentage. If there are cracks ongoing cracks, the width of this cracks should be limited. The maximal crack width for ongoing cracks is decried in “Betonconstructies onder Temperatuur- en Krimpvervormingen” [1].
In the longitudinal and lateral joints gaskets will be applied to secure the water tightness. The gaskets will be compressed due the bolts, and become then watertight. The magnitude of the required compression depends on the waterlevel and the gasket properties. On intersection points it is difficult to ensure the water tightness. Therefore it is recommended that a ring joint and lateral joint never cross each other [12].

**Assumptions**
During the evaluation of the lining concept the following items are assumed. In a definitive design these assumptions have to be verified.

- Each single ring stable by his own
- Concrete thickness 0.40 meter
- Jack radius 0.20 meter.

### 4.1.3 Requirements for lining concepts

In the table 11 the requirements for the lining concepts are specified.

| Requirements | Cause |
|---|---|
| Jacks at least 0.3 meter from the lateral joints | This space, 0.3 m, is required to make a connection between the elements. And it is unwanted that reinforcement for the jack location interfere with reinforcement for connection methods. |
| Try to get a shift between the lateral joints | This results in more building accuracy. If the joints are al the time at the same location the chance on de deflection is much larger. |
| The Jacks must be symmetric | There will be worked with two rings, so a symmetric jack configuration is required. |
| The lining must be watertight | The underpass lies almost in total below the groundwater level. |
| The segments must be transportable in the underpass | The underpass in constructed in the shield. So the elements must be transported through already built tunnel to the bore front. |
| The lining must be able to make some corrections for steering uncertainties | If the TBM does not follow the prescribed bore path, it is necessary to make a correction in the lining |
| Maximal width of ongoing crack is 0.05 mm | This for cracks that are ongoing from the inside to the outside. |
| The tolerance for building the element is +- 1 mm | This tolerance is required, otherwise the prefab element does not fit to each other |
| The tolerance for placing the element is +- 10 mm | This tolerance is required, otherwise the prefab element does not fit to each other |

*Table 11; requirements of the lining of the underpass*

## 4.2 Longitudinal concepts

**General**
In this paragraph it will be investigated which design in the longitudinal section is most preferable. Therefore several concepts are discussed. The paragraph ends with the selection of the most preferable concept.

**Conventional concept**

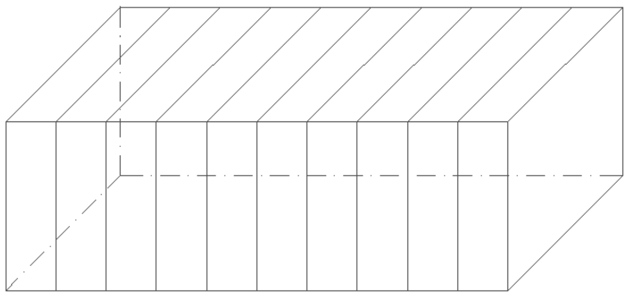


*Figure 20; Conventional ring method*

This is the most common method to build the lining of a bored tunnel, see figure 20. The tunnel is built ring by ring, all the rings have the same length. In traditional circular tunnels the elements are connected together by bolts during the construction phase. In the definitive phase these bolts are removed, this is possible due to the arch working of the circular shape. In this rectangular underpass there is no arch working, therefore the connection methods will not be removed and are part of the definitive construction.

**Long middle beam concept**

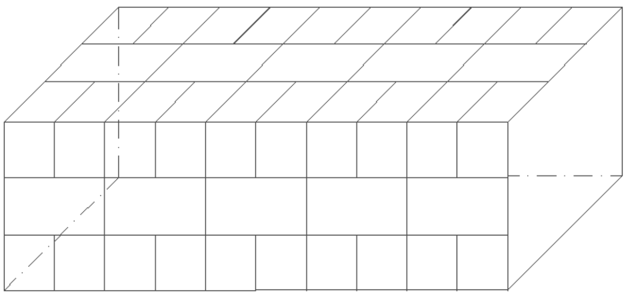


*Figure 21; Ring method with long side elements*

A modification on the previous design is to lengthen the middle beam, see figure 21. This results in more stability of the individual rings. The disadvantage of this concept is that the jacks must be twice as long as before. This results in an increase of the TBM costs. Placing of the elements can also become also more difficult due to the longer elements, and correcting steering deflections during the advance of the TBM process becomes harder.

**Half stretcher bond concept**

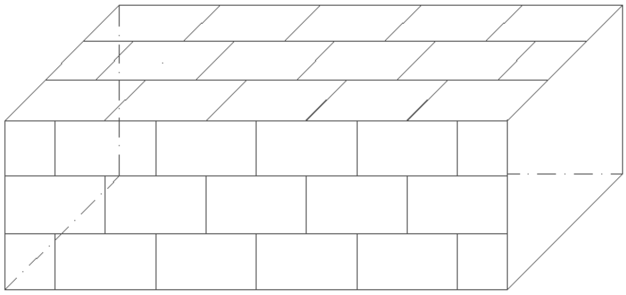


*Figure 22; half stretcher bond*

In this variant a half stretcher bond is applied, see figure 22. The disadvantage is the same as the previous concept; the TBM can only bore half of the element length before new elements have to be placed. Advantage is that the stiffness in longitudinal direction is more enlarged due to half stretcher bond. Disadvantage is that placing of the elements becomes more complex and that there are longer jacks required then in the conventional concept, and correcting steering deflections during the TBM process becomes harder.

**Selected concept**
After evaluation of the three concepts, the conventional concept will be chosen. There is chosen for this concept because the other concepts will cause too many difficulties during the construction phase. The price of the TBM would be enlarged, and installing the elements would gain practical problems.

There are as less as possible connection preferred as described in paragraph 4.1.2, therefore the length of the elements is preferred as long as possible. The elements must also be transportable, see paragraph 4.1.2., this reduces the length of the elements. First there is chosen for elements with a length of 1.0 meter. In a later phase of the design this can be changed.

By the traditional concept there is assumed that every single ring is by his own stable. Conform this assumption no there are no lateral connections between the joints required. In practice there are some small bending moments in lateral direction, and the water tightness must be secured. Therefore in the lateral direction ring joints will be applied.

## *4.3 Ring concepts*

### 4.3.1 Number of elements

In this paragraph several element configurations will be evaluated. The aim is to have as less as possible elements in each cross section. Important is that it is possible to lift and rotate, and install the elements in the available space in the TBM. Further there are as less as possible elements preferred. Each configuration is evaluated on the following aspects:

- possibility to construct the ring configuration
- Amount of connections

**Concept A**

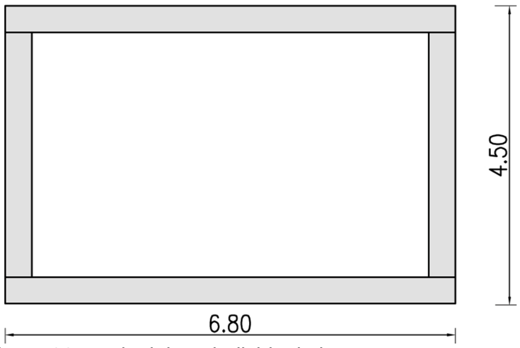



*Figure 23; each slab as individual element*

These elements are too large. After transporting into the underpass it is not possible to rotate and install the element. Unless if there are more meters excavated on the front side of the TBM. But ten large jacks and a long TBM are required. This concept is not possible due to the large dimensions of the elements.

**Concept B**

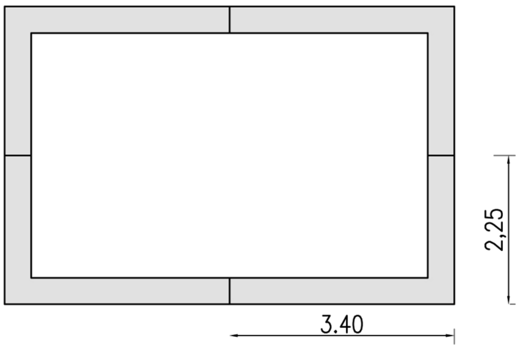



*Figure 24; ring divided into 4 equal parts*

This variant has the advantage that is a small amount of elements, so less connections. Placing the first three elements is possible, but placing the last element is not possible, unless there is one ring more excavated. Another disadvantage is that the concepts are on relative worse locations regarding, the bending moments, but stiff corners can be arranged.
Since this concept has two big disadvantages, placing of the last element, and the location of the connections, this concept is not suitable in practice.

**Concept C**

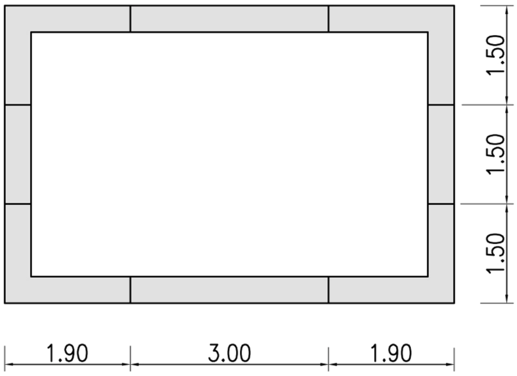



*Figure 25; joints on locations with low bending moments*

This concept contains 8 elements instead of 4 elements in the previous concepts. This results in more connections, so higher costs. But the shortcomings of the previous concepts are solved. The elements can rotate and placed, and the connections are on favorable locations related to the bending moments. Therefore a configuration of 8 elements will be selected, with roughly the sizes mentioned in the figure above. If the required space reduces the middle element can be reduced or left out and a cross section with a smaller size can be produced. It is possible that the required space reduces due to a strategic choice of the buoyancy layer.

**Conclusion**
The there will be chosen for concept C, because this model is suitable in practice and the bending moments are on relatively good locations in relation to the bending moments.

### 4.3.2 Size of elements

When the underpass has a rectangular form the concrete in the corners on the outside of the element snaps off. This happens due to loads that act on the underpass. To prevent this, the corners of each element are rounded of. On the inside this is also done to prevent snapping of the concrete. Beside the technical advantages there are also advantages relating to the social safety. The rounded corners increase the feeling of space, however it is in practice reduced. The circles that can be seen in the figures are the locations where the jacks push of on the lining. The main items where the elements will be judged on are:

- The relation between the jack position and the joints
- Building accuracy
- Possibility to construct the lining

With the first item there is tried to get an optimal distribution of the space on the lining between the joints and the jack locations. Since it is unwanted that a jack and a joint are situated at the same spot, so a “French jack” configuration is aspired. It is unwanted that a jack and a joint are close together because the reinforcement could interfere with each other. At a jack location additional reinforcement is required, and at the joints also additional reinforcement for the connection is required. In general there are two jacks per segment used, to realize an equal stress distribution over the segments. When the lateral joints are on the same position there arise building inaccuracies. Therefore it is preferred that the joint in the ring changes from ring to ring. Below several concepts will be evaluated and finally one concept will be selected.

**Concept 1: Double symmetric model**

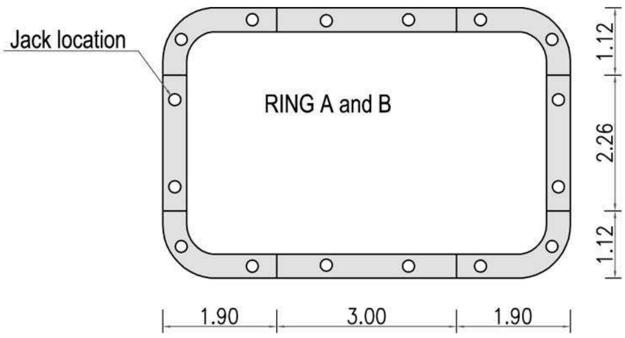



*Figure 26; jack/ element configuration of concept 1*

In this concept is chosen for symmetric segments and jacks, see figure 26. The advantage is that the locations are well placed regarding to the bending moments, and that the jacks and joints do not interfere with each other. The disadvantage is that the joints are all the time on the same location in longitudinal direction. This gives too many uncertainties during the building process that this model is not suitable in practice.

**Concept 2: partly shifted joint**

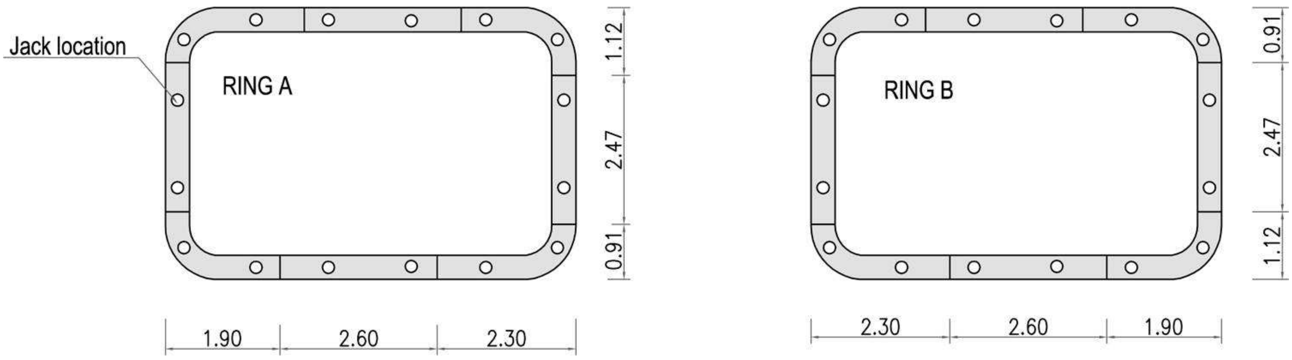



*Figure 27: jack/ element configuration of concept 2*

In this variant the longitudinal joints are shifted with 0.40 m from each other. The joint switches between two jack positions, as can be seen in figure 27. The shifts result in more building accuracy during the construction phase. But the jacks are placed relatively close to the joints, which can cause problems due for the reinforcement as mentioned before. Therefore the concept is not suitable in practice.

**Concept 3: Design by Obayashi**

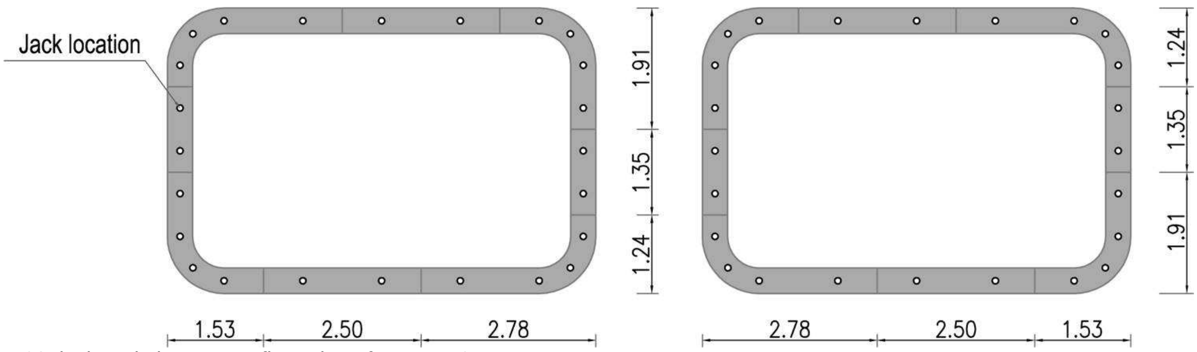


*Figure 28; jack and element configuration of concept 3*

This concept is proposed by the Japanese contractor Obayashi, see figure 28. The location of the jacks is symmetric over the x and y axis. The joint shifts between the jack positions as can be seen in the figure. Due to the size of the elements there are joints in the middle of the top slab in Ring A. This is a disadvantage since the bending moments are relatively high in the middle of the top slab.

**Concept 4: Isosceles corner elements**

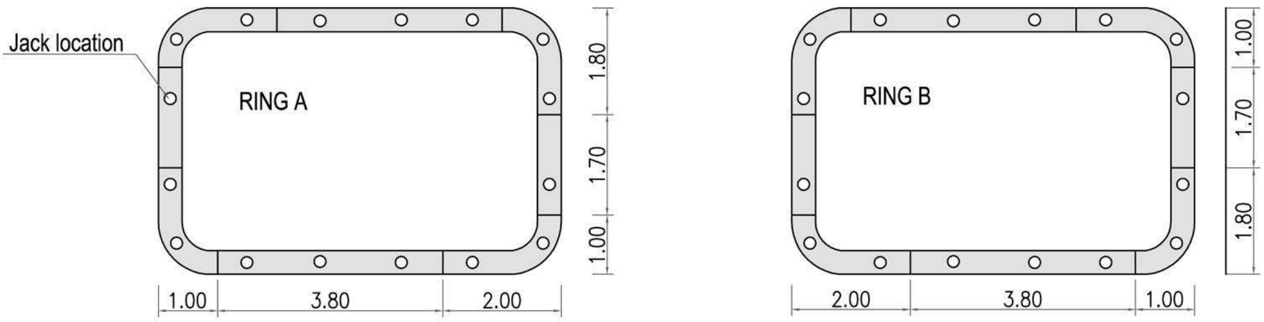



*Figure 29: jack/ element configuration of concept 4*

The concept in figure 29 is an improvement on the previous concepts. The location of the joints is more favorable in relation to the bending moments. This results in lower loads on the connections in the definitive phase.

**Conclusion**
Based on the evaluation can be said that concept 1 and concept 2 are not suitable in practice. The joints do not shift enough to gain building accuracy. In concept 3 the joints do shift enough, but this results in a worse location of the connections related to the bending moments. In concept 4 this shortcoming is optimized, therefore this is the most ideal concept and this will be worked further out in the preliminary design.

## 4.4 Connection concepts

In this paragraph the following connection principles and concepts will be discussed on their pros and cons. The concepts must be serving as connection between the elements in the ring, see figure 30.

- Oblique joints
- Longitudinal groove
- Console
- Extended rebar in ballast concrete
- Bolts in ballast floor
- Upside down console
- Box method
- Oblique bolts
- Circular bolts

After evaluation there will be one concept selected.

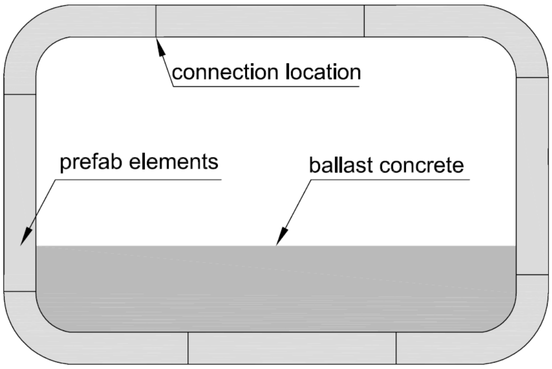



*Figure 30; schematic cross section*

**Oblique joints**

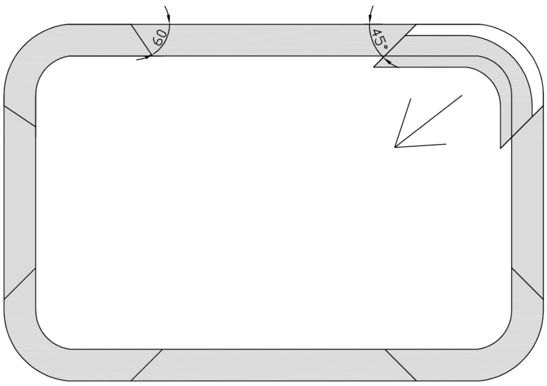



*Figure 31; oblique joints*

To increase the shear force capacity of the connection in the joint there will be oblique joints applied. A schematic overview can be seen in the figure 31. When an oblique joint of 45 degree is applied it might be possible that one of the upper corner elements is not stable and slides into the underpass. Therefore a joint with an angle of 60 degree is suggested, because with this angle the corner element will be resisted by the embracing elements and will not slide into the underpass.

**2. Longitudinal groove**

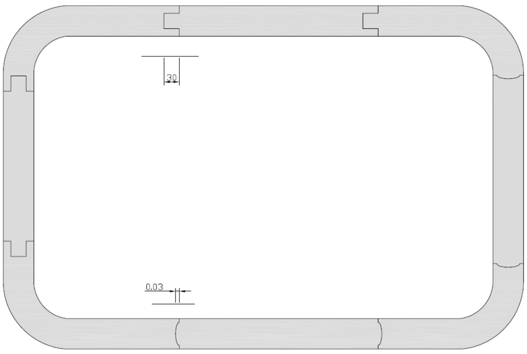



*Figure 32, longitudinal groove*

In this concept, see figure 32, some longitudinal groove variants are showed. Several types can be applied.  An element can be constructed with both grooves on the outside, for example the middle beam below. Another possibility is to construct one corbel on the inside and one on the outside, as can be seen in the beam in the top slab.  The longitudinal groove can be made of a rectangular and circular form as can be seen in the figure. To prevent cracking of the concrete the groove is in the middle of the beam situated. In all the variants the groove must be reinforced heavily.
An advantage of applying longitudinal grooves is that it introduces some set points. From these set points the positions of the underpass can be measured. This is desired for building accuracy during the construction phase. A disadvantage is that additional space is required to place the last element in the ring. Another disadvantage is that it is uncertain where the grooves exact contact each other. It can be on an unexpected location, where not enough reinforcement is present.

Due to the fact that there is additional space required for installing the elements and that it is uncertain where the contact planes between the longitudinal grooves are, this concept is not suitable for the URUP concept.

**3. Console**

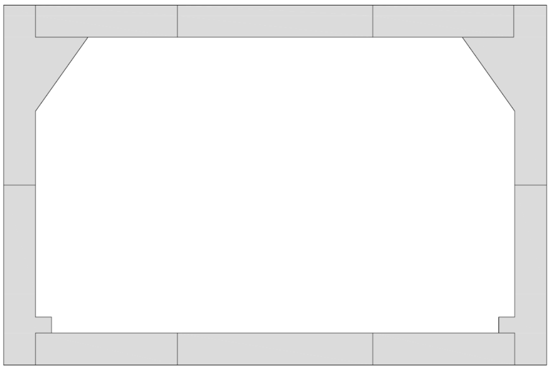


*Figure 33; model console*

A possibility to support the corner elements is to apply a console, figure 33. In this concept rectangular shapes are preferred for the supports of the console. An advantage is that there are fixed points created which increases the building accuracy in the construction phase. The consoles can only applied in the corners. In the middle beams in the walls, top, and floor slab are other methods required. This is the main disadvantage of this concept, because uniformity is desired. A second disadvantage is that the elements require additional space during the construction phase. Therefore this concept is not suitable for the URUP concept.

**Extended rebar**

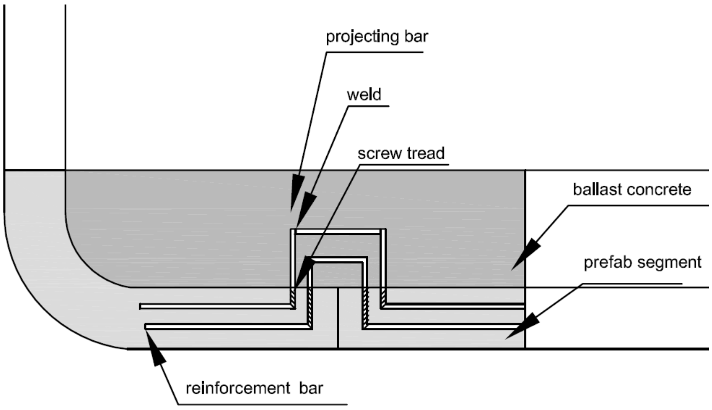



*Figure 34, concept extended rebar*

This concept, figure 34, can be applied to make a moment fixed connection in the floor. In the ballast concrete some “extended rebars ” are applied. This rebar can be screwed in the reinforcement that is placed in the prefab elements. After screwing, the rebars will be welded together, and finally the ballast concrete will be poured.
It is not preferable to transport the loads and bending moments so far out of the elements. This causes a lot of technical issues when the construction is designed in detail. Therefore this model is not suitable for the URUP concept.

### Bolts in Floor

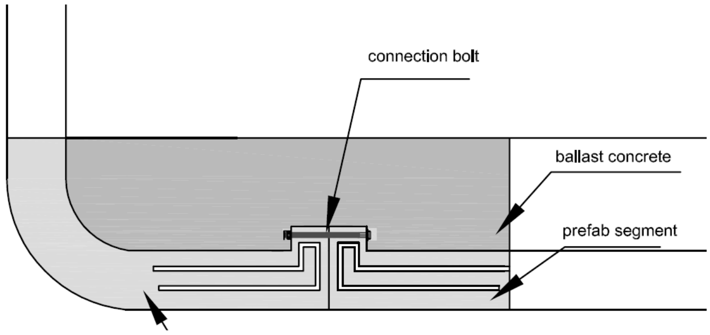
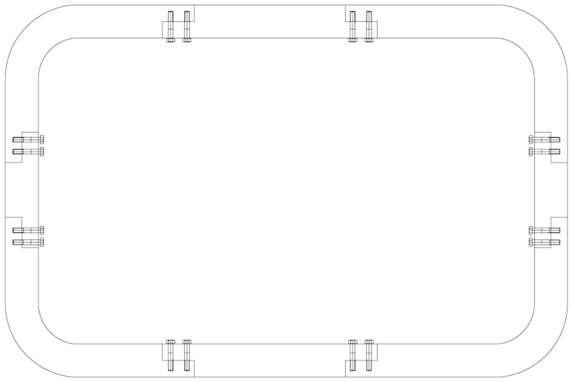



*Figure 35, model bolts in floor*

There will be bolts placed in a “cantilever, see figure 35. After that the fill concrete is be pored, and a stiff connection is realized. This connection method can be used to gain more stiffness. But it is not preferable because it cost a lot of effort and elements of other shape are required. This method is possible suitable.



**Upside down console**
In figure 36 the upside down console is shown, a detail of the bolts can be seen in figure 37. The advantage is that there is no extra space required to construct the elements. But the disadvantage is that the element in the top slab hangs on a few bolts. This is not well use of the material properties. Therefore this variant is not suitable in practice.

*Figure 36; overview upside down console*

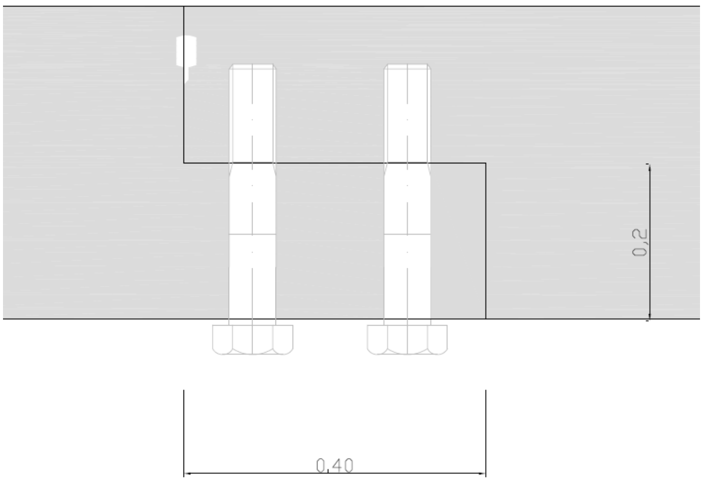



*Figure 37; detail upside down console*

**Box Method**

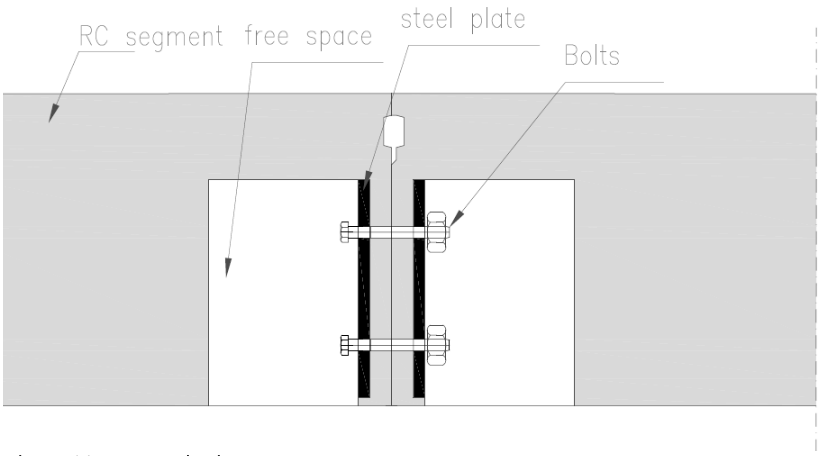



*Figure 38; Box method*

The Box method, figure 38, is often applied in the Japanese tunnel industry. A steel box is placed in these prefab elements. The boxes will be connected with rebars to the main reinforcement. These rebars are not shown on the sketch. When the elements are placed bolts are installed to connect the elements together.
An improvement could be to apply oblique joints so that a part of the shear force will be taken over by the concrete. But the positioning of the box in the concrete becomes a problem then. Another possibility is to execute this method together with the circular bolts.

Advantage of this method is that it is relatively easy to construct. After installing the elements only some bolts have to be placed. Disadvantage of this method is that is relatively expensive. Fabricating the steel boxes is expensive due to the heavy weight of these boxes.

**Oblique bolts**

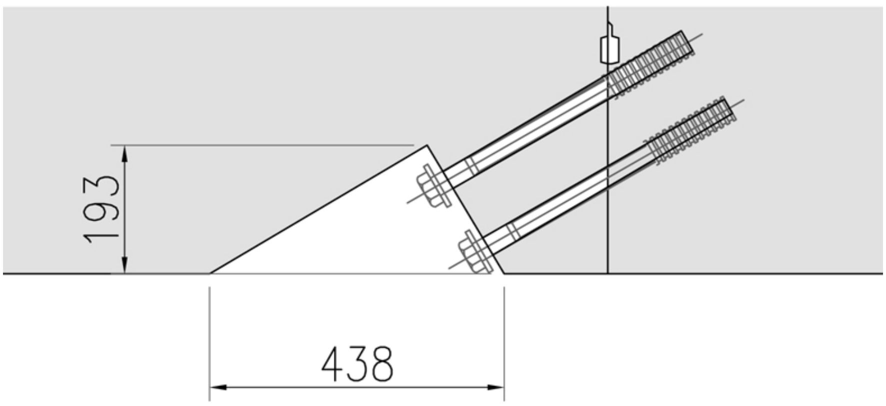



*Figure 39; oblique bolts*

In this concept, figure 39, there is in one element a box applied, and in the other elements are sockets placed. The elements are connected with bolts.
An advantage is that there is less steel required, so the connection method is cheaper. But disadvantages are that the length of the bolts becomes very long, and that it is uncertain if a socket is

strong enough to resist the loads. This method is potential suitable, but attention should be paid to the length of the bolts and the capacity of the sockets.

**Oblique bolts and oblique joints**

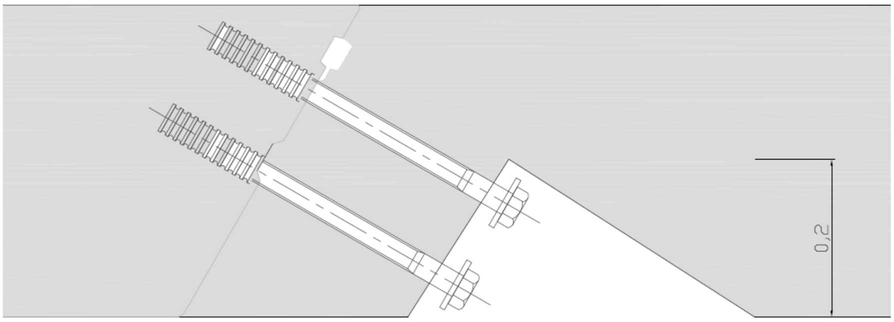



*Figure 40; oblique bolts and joints*

To reduce the length of the bolts an oblique joint can be applied, see figure 40. Additional advantage of the oblique joint is that it increases the shear force capacity of the connection, as described before. It also reduces the required the length of the bolt on the upper side. The minimum thickness of the lining is still small due to the large box. This concept is potential suitable in practice.

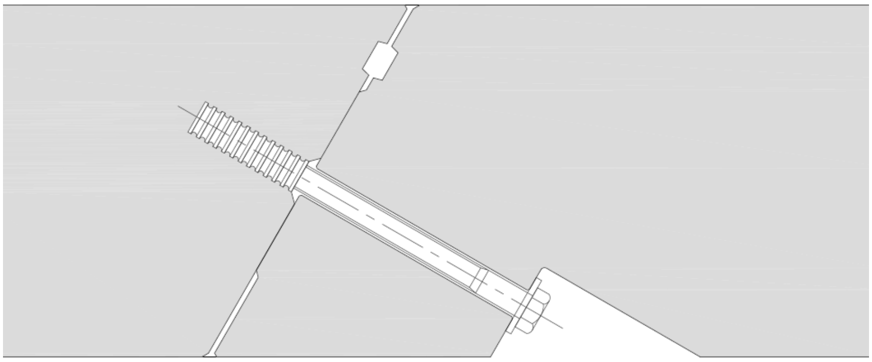


*Figure 41, oblique joint with one bolt*

To increase the minimal thickness of the lining on the box location an box with one bolt can be applied. The consequence is that the quality of the bolts must increase or there must be placed more bolts beside each other.

### Circular Bolt

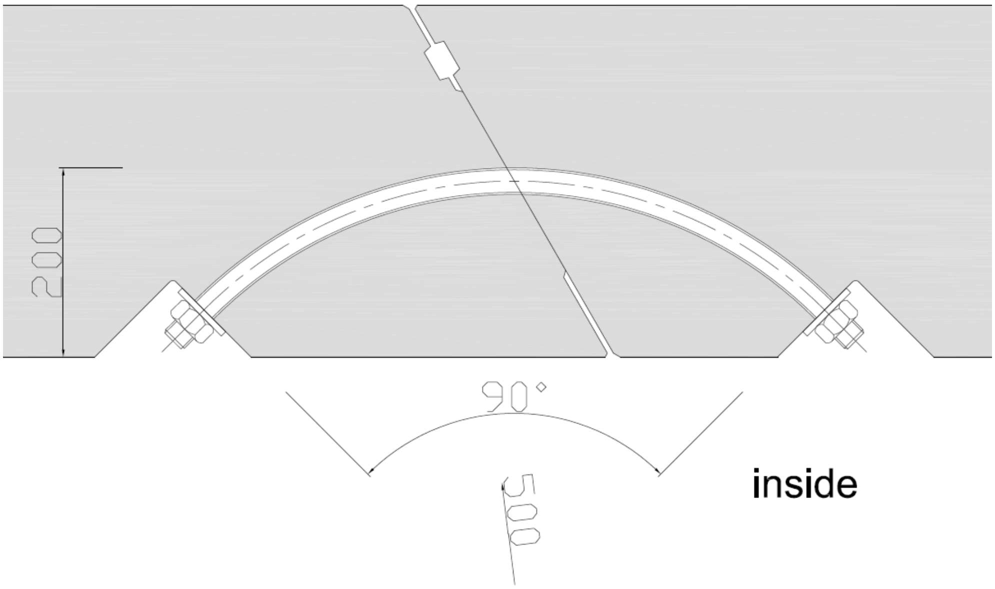



*Figure 42; circular bolts*

In figure 42 an example of the circular bolt is shown. The bolt can execute in with different radii and angles. In this sketch a bolt with 90 degree and a radius of 500 mm is showed.
The advantage is that the bolt is easy to install, so this increases the building time. Disadvantage is that the method is relatively expensive. A technical disadvantage is that the concrete could collapse. This concept has proven that it works at the Westerschelde tunnel, and therefore this concept is potential suitable in practice.

### Conclusion

The following concepts have unacceptable disadvantages and are therefore not suitable for applying in the URUP method in the Netherlands.

- Longitudinal groove
- Console
- Extended rebar in ballast concrete
- Bolts in ballast floor
- Upside down console

The following concepts are potential suitable for applying in the URUP method in the Netherlands.

- Oblique joints
- Box method
- Oblique bolts
- Circular bolts

After evaluation the box method is selected, because it is a simple concept that can take up the loads. Unfortunately it cannot combine with the oblique joints, the box would be situated to far from the surface.
In the other concepts were too many doubts about the technical feasibility. The capacity of the sockets was uncertain, therefore the oblique bolts was not possible. The capacity of the circular bolt was a problem, but it can be applied in combination with another concept, for example the box method.

## *4.5 Selected lining concept*

After evaluation [23] the following lining is selected. Chosen is for rings with an equal length of 1 meter as can be seen in figure 43 a. The joints are situated on favorable locations relating to the bending moments as shown in figure 43 b. The connections will be made with the box method, as shown in figure 43c. In later projects it is possible to extend the connections with oblique joints. In the next chapter the structural design for the lining is made.

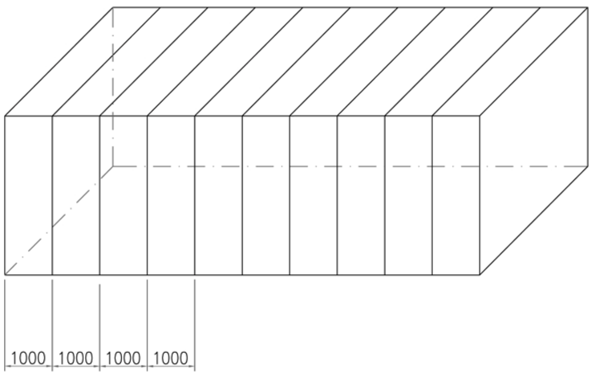



Figure 43a; overview selected longitudinal concept

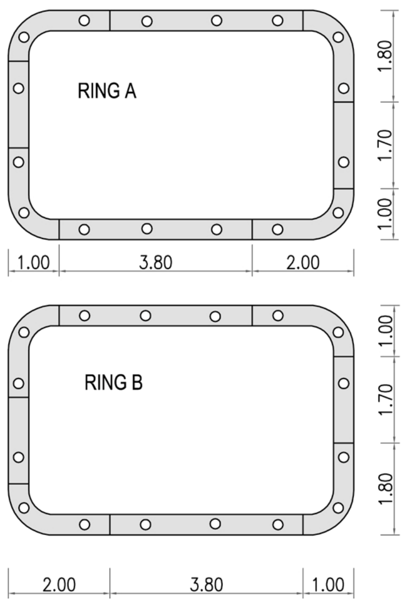
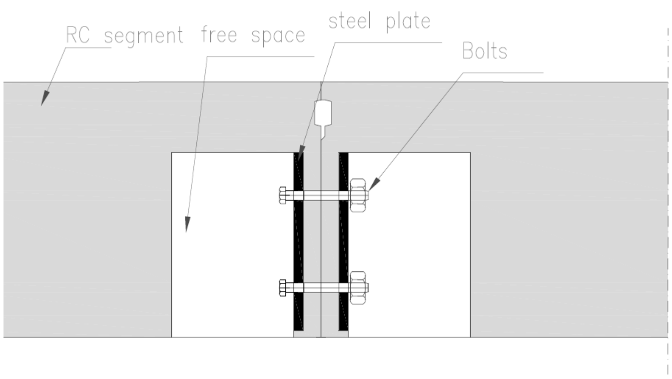



*Figure 43 b; selected element configuration*




*Selected 43c; selected connection method*

# 5 Preliminary design

## *5.1 Introduction to preliminary design*

To get a better understanding of the consequences for the costs of the URUP project, when there is a different groundwater level there will be two cases elaborated.

- Goes worst situation groundwater level +0.80 m NAP
- Goes realistic situation groundwater level -0.38 m NAP

To reach this goal there is made longitudinal design for both cases. All the other aspects, such as loads, lining design etc. are designed based on “Goes realistic situation”. In chapter six the difference in between the two cases is elaborated.

**Goes worst situation**
The maximum ballast layer that in the Netherlands is required is 1.05 meter, see paragraph 3.2. This ballast layer must be applied in Goes when the water level is lifted to the top of the road, +0.8 meter NAP. This results in a total construction height of 4.5 meter for the underpass, see drawing P3745.001, annex G “drawing P3745.001 Goes worst situation ”. For this preliminary design only a longitudinal design is made. All the other design aspects like connections and reinforcement are assumed equal as in “Goes realistic situation”. Because the loads in “Goes worst situation” are larger then in “Goes realistic situation”, this assumption should be verified in a later stage.

**Goes realistic situation**
In the “Goes realistic situation”, the circumstances are as described in chapter 3. The groundwater level is -0.38 meter NAP. The URUP underpass is constructed with a TBM that exactly fits for the profile of free space, there is no ballast layer required to prevent buoyancy, see annex E ‘Report determination buoyancy Goes”. This results in a total construction height of 3.45 meter. The design is shown in drawing, annex F “drawing P3745.002 Goes realistic situation”. For this preliminary design a longitudinal design, structural design for the elements, and connections are worked out.

Further are in paragraph 5.2 the material, loads and safety factors presented for “Goes realistic situation”. In paragraph 5.3 is the longitudinal design for both cases made. In paragraph 5.3 the structural design for the elements is made, based on the loads in “Goes realistic situation”. In chapter 5.5, 5.6 and 5.7 additional design aspects for “Goes realistic situation” are shown.

## *5.2 Materials, loads, and safety factors*

### 5.2.1 Material properties

The elements are built out of reinforced concrete elements. The concrete will be of the quality C53/B65. The properties can be found in the table 12 and are conform NEN 6720 [20].

| | $f'_{ck}$ (N/mm$^2$) | $f'_b$ (N/mm$^2$) | $f_b$ (N/mm$^2$) | $f_{bm}$ (N/mm$^2$) | $E'_b$ (N/mm$^2$) |
|---|---|---|---|---|---|
| C53/B65 | 65 | 39 | 2.15 | 4.3 | 38,500 |

*Table 12; overview concrete properties*

Where:
$f'_{ck}$= compressive strength of concrete
$f'_b$= design compressive strength of concrete
$f_b$= design tensile strength
$f_{bm}$ representative tensile strength
$f'_b$= Modules of Elasticity

The rebar will be of the type FeB 500, HKN, with the following properties as presented in table 13.

| $f_{s;\ rep}$ ($N/mm^2$) | $f_s$ ($N/mm^2$) | $f'_s$ ($N/mm^2$) | Yield strength ($N/mm^2$) |
|---|---|---|---|
| 500 | 435 | 435 | 500 |

*Table 13; overview steel properties*

Where:
$f_{s;\ rep}$ = representative value tensile strength
$f_s$ = design value tensile strength
$f'_s$ = design value compressive strength
Yield strength of rebar

The bolts are have quality 4.6 and 8.8. The material properties are presented in table 14 and 15.
Bolt 4.6

| | | |
|---|---|---|
| $f_{t;\ rep}$ | 400 | $N/mm^2$ |
| $t_{0.2\%\ rep}$ | 240 | $N/mm^2$ |

*Table 14; overview bolt 8.8 properties*

Bolt 8.8

| | | |
|---|---|---|
| $f_{t;\ rep}$ | 800 | $N/mm^2$ |
| $t_{0.2\%\ rep}$ | 640 | $N/mm^2$ |

*Table 15; overview bolt 8.8 properties*

## 5.2.2 Loads

The loads are based on “Goes realistic situation”. The description of the types of loads that act on the underpass can be found in paragraph 3.4. The normative section is there where the underpass crosses the road. A detailed quantifying of the loads at the normative section can be found in annex D1 “loads on top slab definitive phase”. The results are presented in table 16.

**Maximal expected loads in the definitive phase on the underpass**

| Item | Load S.L.S. ($kN/m^2$) | Load U.L.S ($kN/m^2$) |
|---|---|---|
| Load on the top of the top slab | 71 | 92 |
| Load on the top side of the wall | 185 | 236 |
| Load at the toe of the wall | 256 | 349 |
| Load at the underside of the bottom slab | 16.2 | 18 |

*Table 16; maximal expected loads in the definitive phase on the underpass*

Maximal expected thrust forces during start of the TBM process are shown in table 17, the calculation is elaborated in annex D2 “loads during push of process and during tunnel boring process”.

| # | | Load (kN) |
|---|---|---|
| W1 | friction shield – surrounding soil | 1139 |
| W2 | friction shield – tunnel lining | 1025 |
| W3 | Pressure force on cutting wheel | 1250 |
| W4 | resistance of the cutting wheel | 0 |
| W5 | support pressure TBM | 266 |
| Wtot | Total load | 3681 |

*Table 17; forces during the push of process*

The maximal expected thrust forces during the advance of the TBM are shown in table 18, the calculation is elaborated in annex D2 “loads during push of process and during tunnel boring process”.

| # | | Load (kN) |
|---|---|---|
| W1 | friction shield – surrounding soil | 2278 |
| W2 | friction shield – tunnel lining | 1170 |
| W3 | pressure force on cutting wheel | 2500 |
| W4 | resistance of the cutting wheel | 0 |
| W5 | support pressure TBM | 2396 |
| Wtot | Total load | 8345 |

*Table 18; push forces at the deepest point during the tunnel boring process*

### 5.2.3 Safety factors

The soil loads are determined conform table 1, NEN 6740 [21]. They are assumed very conservatively and therefore a safety factor of 1.0 can be taken in account conform the NEN 6744 [22]. Other dead loads, like asphalt etc have to be calculated with a safety factor of 1.35 in U.L.S. For life loads a safety factor of 1.5 has to taken in account, conform NEN 6706 [19]. A summary of the applied safety factors are shown in table 19.

| Force | Safety factor U.L.S. | Safety factor S.L.S. |
|---|---|---|
| Soil load | 1.0 | 1.0 |
| Other dead loads | 1.35 | 1.0 |
| Traffic | 1.35 | 1.0 |
| Life loads | 1.5 | 1.0 |

*Table 19; safety factors*

## *5.3 Longitudinal design*

### 5.3.1 General

The presence of a ballast layer influences the longitudinal design much. If there is a ballast layer the construction height increases, and therefore the bottom of the construction will lie deeper below the surface which results in longer ramps. Due to the difference in the thickness of the ballast layer between “Goes worst situation” and “Goes realistic situation”, there is a large difference in the longitudinal design, this is shown in the next paragraphs.

### 5.3.3 Longitudinal design “Goes worst situation”

Below the longitudinal design for “Goes worst situation” is presented with a ballast layer that fit for all situations in the Netherlands. The total construction height in “Goes worst situation” is 4.50 meter due to the ballast layer of 1.05 meter. The same lining as in “Goes realistic situation” is assumed. In table 20 an overview of the longitudinal design is presented, the drawing of the longitudinal design can be found in annex G, “drawing P3745.001 Goes worst situation”.

| Part | Distance from start location (m) | Length (m) | Full face | Remarks |
|---|---|---|---|---|
| In situ part | 0-15 | 15 | No | in situ placed concrete to complete the underpass, this will be constructed at the end in the time. |
| Bend downwards | 15-37 | 22 | No | The TBM will start with a downwards direction of 5.94 %. The alignment for cyclist will be 4% in the definitive phase |
| Bend downwards | 37-44 | 7 | Yes | Straight line with a down percentage of 5.94% |
| Curve downwards | 44-82 | 38 | Yes | Curve to a horizontal line with R=750 m |
| Curve upwards | 82-120 | 38 | Yes | Curve upwards to a bend of 5.94% with R=750 meter |
| Bend upwards | 120-127 | 7 | Yes | Straight line with a percentage up of 5.94% |
| Bend upwards | 127-149 | 22 | no | The TBM ends wit upwards direction of 5.94 % |
| In situ part | 149-164 | 15 | No | In situ placed concrete to complete the underpass, this will be constructed at the end in the time. |

*Table 20; overview longitudinal design P3745.001*

**In situ parts**
The in situ parts will be constructed after the tunnel boring process is finished. This part is too small to excavate with the TBM, therefore it is economical more interesting to make these parts in situ.

**Bend downwards and bend upwards**
The TBM starts with a bend downwards. Here the machine is not full face yet. The TBM has to push of on the previous elements. Therefore a number of dummies will be applied, when the machine is not full face. These dummies are placed on the ring, and then the TBM can push of on the dummies. After 5 rings the dummies are removed and placed at front again. In the definitive design there has to be checked if 5 rings of dummies are enough to take up the forces.

**Curve downwards and Curve upwards**
In the curved parts the TBM is full phase. The lining will be built conform the plan.

**Completing the Ramp**
When the ramp will be completed the following situation will occur. When the dummy will be removed the soil collapses and falls in the ramp. This has to be prevented. To prevent this solution will be applied. Element types A and C will be divided into two parts. One of the parts can be easily removed. This can be a possible solution. Some additional reinforcement is necessary, but it saves time and money.

As can be seen in the drawing the ballast layer is 1.05 meter thick in the normative cross section. In this normative section heavy ballast concrete is applied. In other non normative cross sections normal concrete or sand can be applied.  The total volume of the ballast layer is as follows assumed.

- 60% Sand
- 30% normal concrete
- 10% heavy concrete a 2800 kg/$m^3$

### 5.3.2 Longitudinal design "Goes realistic situation"

An overview of the longitudinal design for "Goes realistic situation" is presented in table 21. The total construction height in this case is 3.45 meter.

| Part | Distance from start location (m) | Length (m) | Full face | Remarks |
|---|---|---|---|---|
| In situ part | 0-15 | 15 | No | in situ placed concrete to complete the underpass, this will be constructed at the end in the time. |
| Bend downwards | 15-33 | 18 | No | The TBM will start with a downwards direction of 5.94 %. The alignment for cyclist will be 4% in the definitive phase |
| Bend downwards | 33-41 | 8 | Yes | Straight line with a down percentage of 5.94% |
| Curve downwards | 41-71 | 30 | Yes | Curve to a horizontal line with R=750 m |
| Curve upwards | 71-101 | 30 | Yes | Curve upwards to a bend of 5.94% with R=750 meter |
| Bend upwards | 101-109 | 8 | Yes | Straight line with a percentage up of 5.94% |
| Bend upwards | 109-127 | 18 | No | The TBM ends wit upwards direction of 5.94 % |
| In situ part | 127-142 | 15 | No | In situ placed concrete to complete the underpass, this will be constructed at the end in the time. |

*Table 21; overview different sections in longitudinal section, drawing P3745.002*

## *5.4 Structural design cross section "Goes realistic situation"*

### 5.4.1 General

Due to the absence of a ballast layer there must be a new element configuration designed. The new element configuration is based on the considerations and concepts that are presented in chapter 4. In the new configuration the element in the wall element is left out, in comparison to the selected element configuration in the chapter 4. The selected element configuration for "Goes realistic situation" is presented in figure 44a and figure 44b.

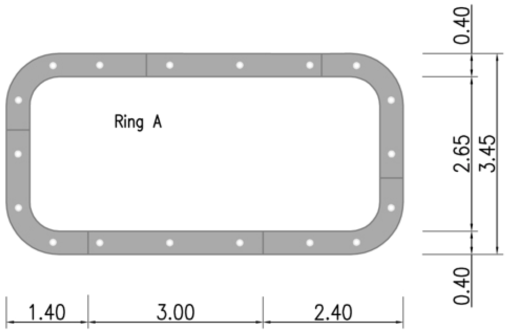



*Figure 44a; overview ring distributions*

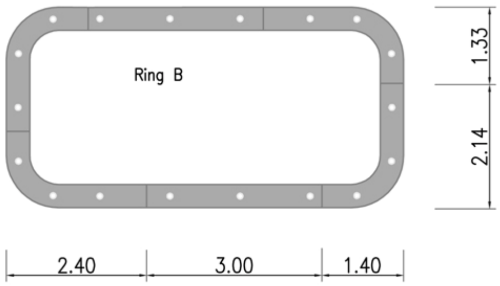



*Figure 44b; overview ring distributions*

The elements have got the following volume as presented in table 22.

| Element | Volume ($m^3$) |
|---|---|
| A | 1.20 |
| B | 1.13 |
| C | 1.13 |

*Table 22; volumes of elements*

The monolith ring has geometric properties as presented in table 21 and 22. The span of the top and floor slab is 6.4 meter because this is the centre to centre distance between the two walls. The length of the wall is 3.05 meter because this is the centre to centre distance from bottom slab to top slab.

**Geometric properties top slab and floor slab**

| | | | |
|---|---|---|---|
| length | l | 6400 | mm |
| width | b | 1000 | mm |
| height | h | 400 | mm |
| thickness | d | 345 | mm |
| cover | c | 35 | mm |
| main reinforcement | D25 | 25 | mm |
| reinforcement cage | k | 10 | mm |
| moment of Inertia | I | 3.42E+09 | $mm^4$ |

*table 21; Geometric properties top slab*

**Geometric properties wall**

| | | | |
|---|---|---|---|
| Length | l | 3050 | mm |
| Width | b | 1000 | mm |
| Height wall | h | 400 | mm |
| Thickness | d | 345 | mm |
| Cover | c | 35 | mm |
| Main reinforcement | D25 | 25 | mm |
| reinforcement cage | k | 10 | mm |
| Moment of Inertia | I | 3.42E+09 | $mm^4$ |

*table 22; Geometric properties wall*

## 5.4.2 Determination reinforcement

**Moment distribution**
The loads in the definitive phase in combination with the lining properties as presented in table 21 and 22 lead to the moment distribution for the top slab as presented in figure 45 and for the wall as presented in figure 46.

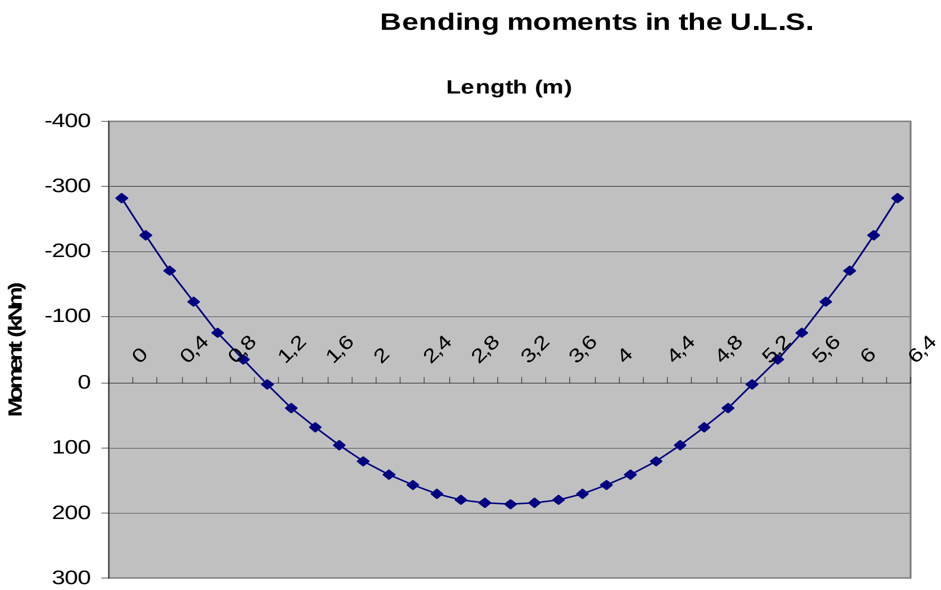



*Figure 45; bending moment in the top slab of the underpass*

$$M = \frac{Q*l}{2} * \left( x - \frac{x^2}{l} - \frac{l}{6} \right)$$ With Q= 92 kN/m and l= 6.4 m

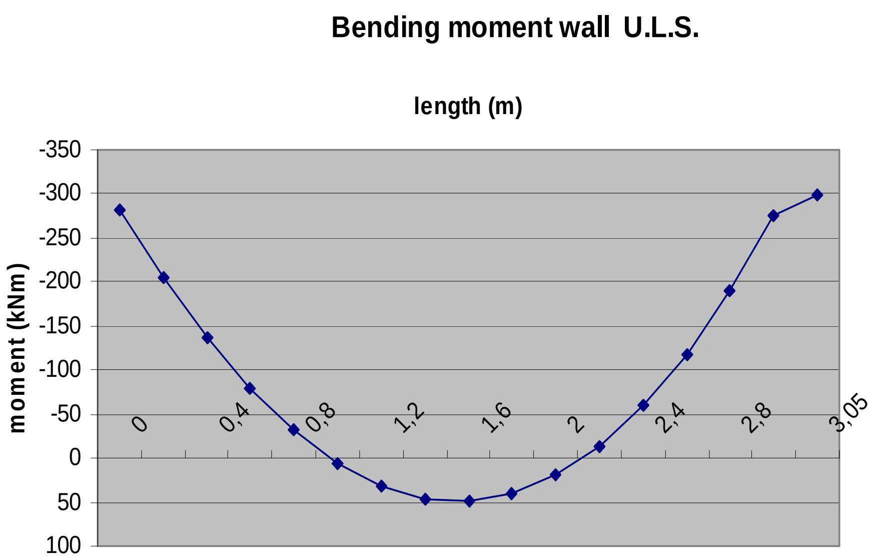



*Figure 46; bending moment in wall of the underpass*

$$M = \frac{Q_{rectangular} * l}{2} * \left( x - \frac{x^2}{l} - \frac{l}{6} \right) + \frac{Q_{triangular} * l}{2} \left( \frac{3x}{10} - \frac{l}{15} - \frac{x^3}{3l^2} \right)$$

With
$Q_{rectangular}$ is 236 kN/m
$Q_{triangular}$ is 349 kN/m
L=3.05 meter.

The bending moments in the corner of the underpass are not equal, however they should be. Therefore redistribution of bending moments is applied. The difference in moment is proportional divided over the length of the beams, and inversely with the stiffness of the beam. The following relation is applied.

$$\varphi = \frac{M * L}{3EI}$$

This leads to a correction factor for the bending moments which is already applied the figures that are showed above. The magnitude of this correction is:

Top slab: + 31 KNM, this reduces the moment in the top slab
Wall : - 64 KNM, this increases the moment in the wall

**Reinforcement against bending**
This calculation is made in the U.L.S, the results can be seen in table 23. The following formulas are applied to determine the reinforcement. In this calculation the internal arm (z) is determined on 0.9 times the thickness (d) conform NEN 6720 [20].

$$N_{steel} = \frac{M}{z}$$

$$A_{steel} = \frac{N_{steel}}{\sigma}$$

**Top slab**

| | | | |
|---|---|---|---|
| Maximal moment | $M_{max}$ | 282 | kNm |
| Internal arm | z | 311 | mm |
| Minimum area steel | $A_{ss}$ | 2088 | $mm^2$ |
| Diameter main bar | $d_b$ | 25 | mm |
| number of bars per element | n | 5 | (-) |
| center to center distance | t | 200 | mm |
| reinforcement | $\omega_0$ | 0.71 | % |
| minimum reinforcement | $\omega_{min}$ | 0.27 | % |

*table 23; results reinforcement against bending in the top slap*

**Wall**

| | | | |
|---|---|---|---|
| Maximal moment | $M_{max}$ | 282 | KNM |
| Internal arm | z | 311 | mm |
| Minimum area steel | $A_{ss}$ | 2217 | $mm^2$ |
| Diameter main bar | $d_b$ | 25 | mm |
| number of bars per element | n | 5 | - |
| center to center distance | t | 200 | Mm |
| reinforcement | $\omega_0$ | 0.71 | % |
| minimum reinforcement | $\omega_{min}$ | 0.27 | % |

*table 24; results reinforcement against bending in the wall*

For the top slab this leads to 5 bars of 25 mm diameter and a reinforcement percentage of 0.71%. This is above the minimum required reinforcement percentage, which is 0.27%. In the elements is also dividing reinforcement situated. The amount dividing reinforcement is estimated on 20% of the main reinforcement. For the wall this leads to 5 bars of 25 mm diameter and a reinforcement percentage of 0,71%. This is above the minimum required reinforcement percentage. The minimum required reinforcement percentage is 0.27%.

**Shear force reinforcement**

This calculation is made in the U.L.S. The maximal shear forces that act on the construction are presented in table 25.

| | | | |
|---|---|---|---|
| Top slab | $V_t$ | 293 | kN |
| Wall | $V_t$ | 480 | kN |

*Table 25: acting shear forces*

The following formulas are applied to determine the shear stress capacity of the concrete. The results of the calculation are shown in table 26 and table 27.

$$\tau_1 = 0.4 * f_b * k_y * m * \sqrt[3]{\omega_0} \geq 0.4 * f_b$$

$$V = \iota_1 * b * d$$

**Top slab**

| | | | |
|---|---|---|---|
| maximal shear force | $\tau_1$ | 0.92 | $n/mm^2$ |
| design tensile strength concrete | $f_b$ | 2.15 | $n/mm^2$ |
| Support factor | $k_y$ | 1.00 | (-) |
| Scale factor | m | 1.20 | (-) |
| Reinforcement percentage | $\omega_0$ | 0.72 | % |
| Shear force capacity by concrete | $V_b$ | 317 | kN |

*Table 26; calculating maximum shear stress in top slab*

The shear force capacity of the top slab is larger then the acting shear force (317 kN >293 kN). Therefore no shear force reinforcement in the top slab has to be applied. Conform this calculation there is no shear force required in element A.

**Wall**

| | | | |
|---|---|---|---|
| Maximal shear force | $\tau_1$ | 0.92 | n/mm$^2$ |
| fb | $f_b$ | 2.15 | n/mm$^2$ |
| Ky | $k_y$ | 1.00 | (-) |
| Scale factor | m | 1.20 | (-) |
| Reinforcement percentage | $\Omega_0$ | 0.72 | % |
| Shear force capacity by concrete | $V_b$ | 317 | kN |

*Table 27; calculating shear force capacity wall by concrete.*

The acting shear force is 480 kN and the shear force capacity is 317 kN. Therefore additional shear force reinforcement has to be situated in elements B and C. The shear force reinforcement is determined with the following equation. The calculation results are shown in table 28.

$$\frac{A_{ss}}{t} = \frac{V_s}{z * \cot(\varphi) * f_{yd}}$$

| | | | |
|---|---|---|---|
| Shear force that must be taken up by reinforcement | Vs | 163 | kN |
| Area of reinforcement | $A_{ss}$ | 2*50 | mm$^2$ |
| Distance between reinforcement bars | t | 83 | mm |
| Angle of cracks | φ | 45 | degree |
| Strength of steel | $f_{yd}$ | 435 | N/mm$^2$ |

*Table 28; shear force reinforcement*

This results in 12 bars with a diameter of 8 mm  a distance of 1 meter in the ends of element B and C shear force reinforcement should be applied.

**Transport and installation reinforcement**

There is assumed that the elements during installation are lifted at one point in the middle of the element. Therefore the maximal bending moment during the lifting and transport operation is equal to:

$$M = 0.5 * ql^2$$

| | | | |
|---|---|---|---|
| Bending moment | M | 52 | kNm |
| Own weight | q | 11.5 | kN/m |
| Span | l | 3 | m |

*Table 29; loads due to transportation and installation*

In table 29 can be seen that the maximal bending moment due to transport is 52 kNm. The minimum reinforcement percentage is 0.27%, with this percentage the following bending moment can be taken up by the elements.

$$M = \frac{\omega_{\min} * b * d^2 * \sigma * 0.9}{100}$$

| | | | |
|---|---|---|---|
| Mending moment | M | 126 | kNm |
| Cover | d | 345 | mm |
| Width | b | 1000 | mm |
| Maximal stress | σ | 435 | N/mm$^2$ |

*Table 30; installation reinforcement*

This results in a bending moment of 126 kNm, see table 30. This is higher then the moment due to lifting of the element. Therefore the minimum reinforcement percentage can applied on the outside of the elements. The span for the elements B and C is smaller then the span of element A. Therefore for this elements also the minimum reinforcement percentage can be applied.

**Split reinforcement**
The split reinforcement calculation is made conform lecture notes CT3150 [12]. The amount of spilt reinforcement is based on the maximum load that acts on a jack, see paragraph 5.2. There is assumed that the total jack load is proportional divided over the jacks. An overview of the loads and the number of jacks is shown in table 31.

| | | | |
|---|---|---|---|
| Total force | $F_{tot}$ | 8345 | kN |
| Number of jacks | n | 18 | (-) |
| Force per Jack representative | F | 463 | kN |
| Force per jack design | Fd | 695 | kN |

*Table 31; loads on jacks*

The thrust forces are assumed as a life load. Therefore a safety factor of 1.5 is taken in account, see paragraph 5.2. so the design value is 695 kN. There is assumed that the loads spread under an angel of 45 degrees, and that all the tensile is taken up by the steel rebars. So a tensile force of 695 kN must be taken up by the split reinforcement.

| | | | |
|---|---|---|---|
| Force taken up by reinforcement | $F_s$ | 695 | kN |
| maximum stress steel | σs | 435 | N/mm$^2$ |
| required area reinforcement | $A_s$ | 1597 | mm$^2$ |
| Diameter reinforcement | d | 16 | mm |
| number of bars | n | 8 | (-) |

Table 32; calculation reinforcement

$$A_s = \frac{F_s}{\sigma_s}$$

This results in a minimum area of 1600 mm$^2$, as shown in table 32. There will be 8 bars placed with a diameter of 16 mm. There is chosen for a diameter of 20 mm to have more space to pour the concrete between the reinforcement. These 4 bars will be placed between 0.3 and 0.8 times the thickness. They will be placed centre to centre 100 mm.

There must also be checked if the maximum pressure can be taken at the jackshoe. The results are shown in table 33. The size of the jack shoe is 300*300 mm. This will lead to a pressure of 7.7 N/mm$^2$. Which is lower then the maximum allowable stress of 39 N/mm$^2$ that yields on the concrete.

$$\sigma_c = \frac{F_t}{A_j}$$

| | | | |
|---|---|---|---|
| Area Jackshoe | $A^j$ | 90000 | $mm^2$ |
| Total force per jack | Ft | 695 | kN |
| Stress in Concrete | σc | 7.7 | $N/mm^2$ |
| Maximum stress in concrete | F´b | 39 | $N/mm^2$ |

*Table 33; stress at jack shoe area*

## 5.4.3 Remaining aspects

**Normal force**
This calculation is made in the U.L.S. situation. The following normal forces act on the underpass, see table 34.

| | | | |
|---|---|---|---|
| Top/ floor slab | N | 480 | kN |
| Wall | N | 326 | kN |

*table 34; acting normal force on the underpass*

The capacity of the construction elements will be determined with the following formula, the results are shown in table 35.

$$N_c = f_b * b * d$$

| | | | |
|---|---|---|---|
| Pressure capacity concrete | f´b | 39 | $N/mm^2$ |
| Width | b | 1000 | mm |
| Thickness | d | 365 | mm |
| Normal capacity | $N_c$ | 14235 | kN |

*Table 35; normal force capacity of wall*

The normal force capacity is much bigger then the acting normal force. Therefore the concrete will not collapse on pressure.

**Water tightness elements**
This calculation is made in the S.L.S. Te secure the water tightness of each element the pressure zone must be high enough. The minimum pressure zone is determined on 100 mm to prevent leakage. The pressure zone is with the following formula determined:

$$\frac{x_u}{d} = -n\omega + \sqrt{(n\omega)^2 + 2n\omega}$$

The results are presented in table 36.

| | | | |
|---|---|---|---|
| Thickness | d | 345 | mm |
| E module steel/ E module concrete | n | 15.58 | (-) |
| Reinforcement percentage | $\omega_0$ | 0.72 | % |
| Pressure zone | $x_u$ | 169 | mm |

*Table 36; overview height pressure zone.*

The pressure zone is high enough to prevent the water tightness of the each individual element. So no additional measures are required. In a later phase the water tightness of the joints will be verified.

**Displacement**
This calculation is made in the S.L.S. First the maximal allowable displacement of the top slab and wall are presented and then there is checked if this requirement is fulfilled. The maximal allowable settlements for the top slab and wall are determined conform code 6702 [18] and are 0.004 of the span, as presented in table 37.

| | | |
|---|---|---|
| Maximal allowable settlement top slab | 25.6 | mm |
| Maximal allowable settlement wall | 12.2 | mm |

*Table 37; maximum allowable displacement for top slab*

To calculate the displacement of the top slab the stiffness EI and the pressure zone must be determined. For the stiffness the stiffness in the cracked situation is taken. The pressure zone is in a previous paragraph determined and is 169 mm. With this value the stiffness in the cracked situation is calculated with the following formula. The variables are shown in table 38.

$$EI_{gescheurd} = E_c * b * x_u{}^3 + E_c * (b * x_u) * (0.5 * x_u) + (d_s - x_u)^2 * A_s * n * E_c$$

| | | | |
|---|---|---|---|
| Stiffness cracked long term | $EI_{cracked}$ | $2.77*10^{14}$ | $Nmm^2$ |
| E module concrete | $E_c$ | 38500 | $N/mm^2$ |
| Width | B | 1000 | mm |
| Height pressure zone | $X_u$ | 169 | mm |
| Outside concrete to centre reinforcement | $d_s$ | 345 | mm |
| Area steel | As | 2454 | $mm^2$ |
| Esteel / E concrete long term | N | 15.58 | (-) |

*Table 38; calculation of stiffness*

**Top slab**
With this value the displacement on the long term can be calculated with the following formula and variables, table 39. The results are presented in figure 47.

$$f = \frac{Q * x^2}{24EI} * \left(l^3 - 2lx + x^3\right)$$

| | | | |
|---|---|---|---|
| Load per meter | Q | 79 | kN/m |
| Span | L | 6.1 | m |
| Stiffness, cracked long term | EI | 3.42E9 | $mm^4$ |
| Position on the span | x | 0<x<6.1 | m |

*Table 39, calculation displacement of the top slab*

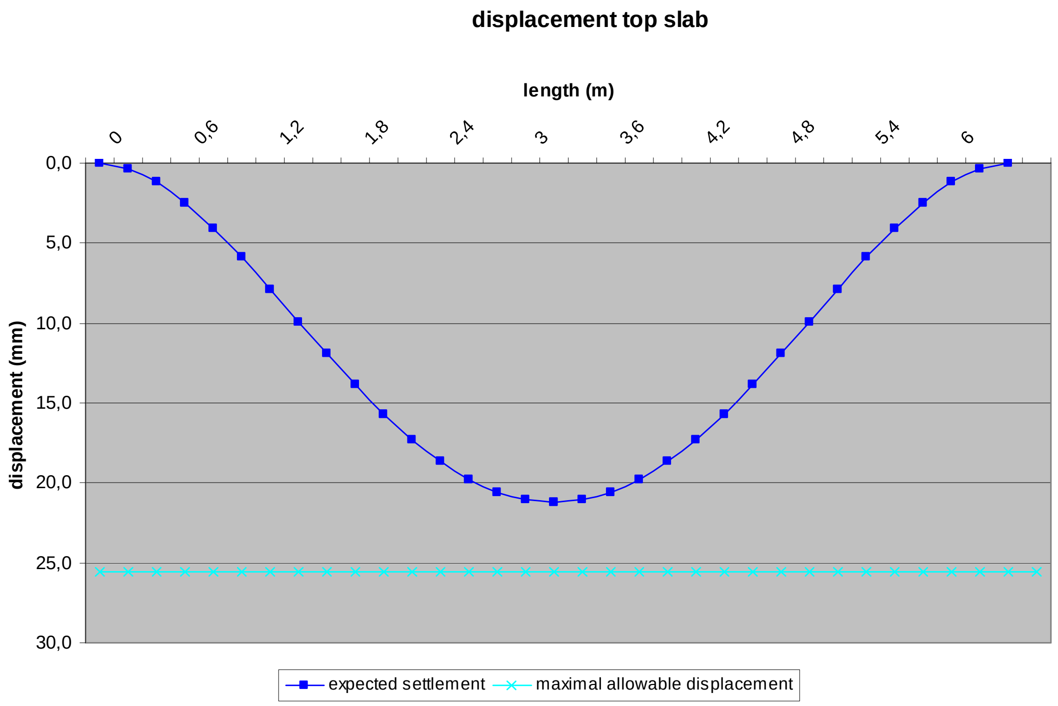



*figure 47; settlement of the top slab of the underpass*

The maximal expected displacements are lower then the maximal allowable displacements.

**Wall**

The displacement of the wall is determined with the following formula and variables in table 40. The results are presented in figure 48.

$$f = \frac{Q_{rectangular} * x^2}{24EI} * \left(l^3 - 2lx + x^3\right) + \frac{Q_{triangular} * l}{30EI} * \left(3x^3 - 2lx - \frac{x^5}{l^2}\right)$$

with

| | | | |
|---|---|---|---|
| Load per meter | Q triangular | 71 | kN/m |
| Load per meter | Q rectangular | 185 | kN/m |
| Span | L | 3.05 | m |
| Stiffness, cracked long term | EI | 3.42E9 | $mm^4$ |
| Position on the span | x | 0<x<3.05 | m |

*Table 40; calculation displacement of the wall.*

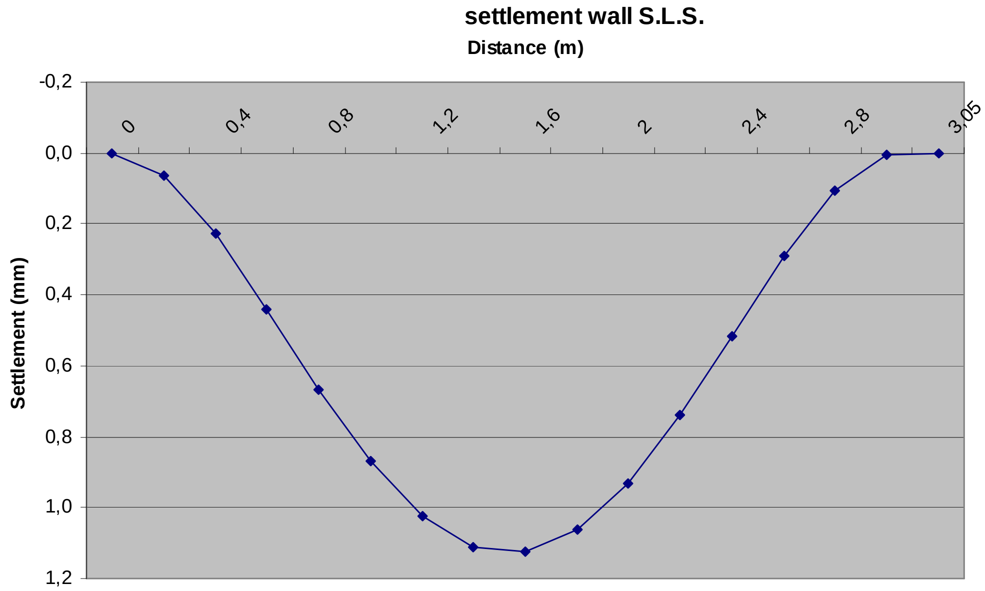



*Figure 48; settlement of the wall in the underpass*

The maximal settlement in the wall is lower then the maximal allowable settlement which is 12.2 mm, see table 37.

### 5.4.4. Summery structural design lining

The calculations lead to the following reinforcements in the elements, see table 41. After evaluation it seemed that the main bar has a relative large diameter. In a definitive design it is possible to reduce the diameter of the main bar. This results in more bars and therefore a better stability of the reinforcement cage.

| | Element A | Element B | Element C |
|---|---|---|---|
| Bending reinforcement | 5* 25 mm | 5* 25 mm | 5* 25 mm |
| Installing reinforcement | 2*25 mm | 2*25 mm | 2*25 mm |
| Dividing reinforcement | 8*16 mm | 8*16 mm | 8*16 mm |
| Split reinforcement | 4*20 mm per item | 4*20 mm per item | 4*20 mm per item |
| Shear force reinforcement | Not present | 12* 8 mm | 12*8 mm |

*Table 41; overview available reinforcement*

## 5.5 Structural design connection detail

### 5.5.1 General

The structural design of the connection method is elaborated in this paragraph, first the required normal force for water tightness between the joints is worked out. After that the strength and stiffness of the longitudinal connection are elaborated. The technical drawings of the applied connection details in longitudinal and lateral direction can be found in annex H1 “longitudinal connection” and annex I2 “lateral connection”. The connection method as selected in chapter 4 is applied. An example of the box method is shown in figure 49.

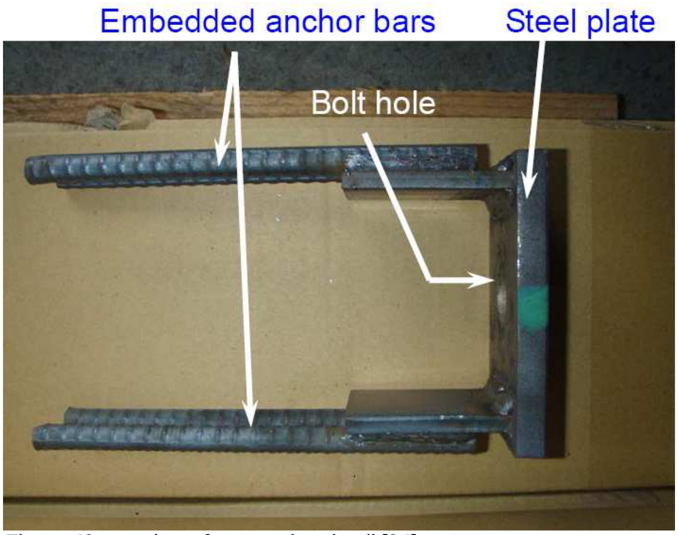



*Figure 49; top view of connection detail [24]*

Per element side two boxes are poured into the concrete. The side view of the element has the shape as can be seen in figure 50. The connection exists of two steel boxes. In the each box two bolts will be placed. The bolts are of type M30.

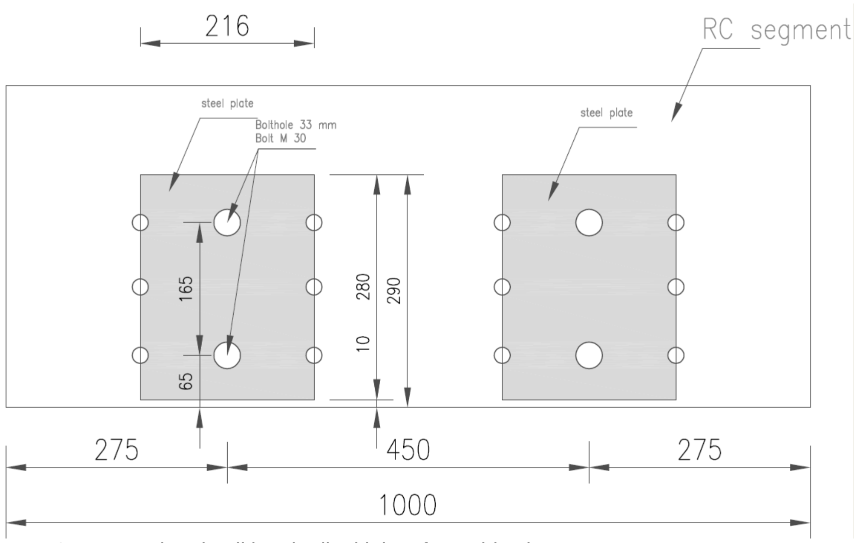



*Figure 50; connection detail longitudinal joint, front side view..*

The bolts have the following material properties, table 42. Bolt A is the upper bolt, bolt B is the lower bolt.

**Bolt A and Bolt B type M30.**

| | | |
|---|---|---|
| Diameter | 30 | mm |
| steel quality | 4.6 | (-) |
| Area | 707 | $mm^2$ |
| $f_{t;\ rep}$ | 400 | $N/mm^2$ |
| $t_{0.2\%\ rep}$ | 240 | $N/mm^2$ |
| Material factor | 1.25 | (-) |

*Table 42; material properties*

## 5.5.2. Water tightness joint

To secure the water tightness gaskets will place in the joints. Due to the pressure the gaskets are compressed. If they are enough compressed, water tightness is secured. In this preliminary design the gasket in figure 51a is chosen quite arbitrary. The gasket fulfilled the requirement, but it is possible that another gasket is cheaper or smaller etc. Therefore it might be possible that in the definitive design an optimization is possible. The technical properties of this gasket can be found in annex I "properties gaskets"

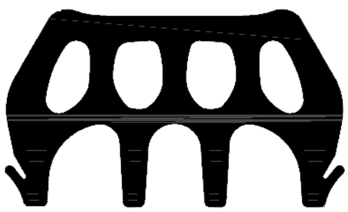


*Figure 51a: gasket 86-259, 26 mm*

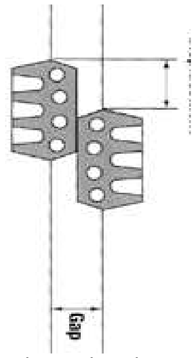



*figure 51b; gaskets in touch*

It is important that the gaskets stay in touch to secure the water tightness, see figure 51b. Therefore the maximum settlement between the gaskets will be elaborated. The water tightness is depending of the following items.

- Expected waterlevel.
- Contact points between rubber and concrete
- Stiffness rubber after relaxation

The following safety factors are applied for determining the design waterlevel, see table 43.

| Cause | Safety factor |
|---|---|
| Expected waterlevel [18] | 1.2 |
| Remaining stress rubber | 1.5 |
| Total (multiplied) | 1.8 |

*Table 43; safety factors for water tightness joint.*

**Justification safety factors**
The demanded lifetime is 100 years. After 100 years the remaining stress in the rubber is decreased to approximately 50%. Therefore a safety factor of 1.5 is applied [3]. The normative design location is below the object that must be crossed, in the floor slab, where the hydrostatic pressure is the most high. An overview of the representative and design waterlevel can be found in table 44.

| | Floor |
|---|---|
| Representative waterlevel (m) | 4.95 |
| Design waterlevel (m) | 9.0 |

*Table 44; design water level for water tightness.*

**Required Normal force**
With the chosen gasket a gap of 11.5 mm is allowed, and a pressure force of 9.5 kN per meter. To prevent water tightness, this requirement must be fulfilled in the longitudinal and lateral direction.

### 5.5.3 Calculation failure mechanisms

The failure mechanisms are only evaluated for the longitudinal joints, because in these direction the loads are much higher. If the loads in the longitudinal direction can be taken up, the loads in lateral direction can also be taken up.

**Bending moment capacity**
The maximum bending moment that can be taken up depends on the maximal tensile force that can be taken up by the bolts in the box. The moment will be taken around the neutral line.
Due to the bending moment there will be a pressure zone in the top of the detail, and a tensile zone below the neutral line. First the maximum tensile strength of the bolts will be determined. From there the maximum allowable moment will be calculated. The maximum tensile force in the bolts is in the following way calculated, the results are presented in table 45.

$$F_{t;u;d} = \frac{0,9 * \alpha_{red2} * f_{t;rep} * A_{bs}}{\gamma_m} \quad [14]$$

| Description | Symbol | Quantity | |
|---|---|---|---|
| Maximum allowable tensile force | $F_{v;u;d}$ | 203 | kN |
| If the bolt is rolled or not | $A_{red;2}$ | 1 | (-) |
| Steel quality | $F_{t;rep}$ | 400 | N/mm$^2$ |
| Bolt area | $A_{abs}$ | 706 | mm$^2$ |
| Material factor | $Y_m$ | 1.25 | (-) |

*Table 45; results tensile capacity*

The tensile force on the steel plate is in the following way determined and results in table 46.

$$F_{t;u;d} = \frac{0.6 * \pi * d_m * t_p * f_{t;rep}}{\gamma m} \quad [14]$$

| Description | Symbol | Quantity | |
|---|---|---|---|
| Maximum allowable tensile force plate | $F_{v;u;d}$ | 295 | kN |
| Stuikfactor | dm | 30 | mm |
| Thickness plate | tp | 15 | mm |
| Quality plates | ftp | 435 | N/mm$^2$ |
| Material factor | $Y_m$ | 1.25 | (-) |

*Table 46; tensile capacity of steel plate*

This shows that the normative part are the bolts and not the steel plate. The maximum allowable tensile force per bolt is 203 kN. The moment will be calculated around the neutral line. The height of the neutral line is 29 mm in the U.L.S.. This is determined with the following formula's and values in table 47.

$$M = N * z$$

$$z = d - 0,389 x_u$$

$$N = \alpha * b * f'b * x_u$$

| | | | |
|---|---|---|---|
| Moment | M | 282 | kNm |
| Cover | d | 345 | mm |
| Factor | α | 0.75 | (-) |
| Width | b | 1000 | mm |
| Strength concrete | $f'_b$ | 39 | mm |

*Table 47; calculation of neutral line*

This will results in the following bending moment that can taken up, see table 48.

| | | |
|---|---|---|
| Length pressure zone | 29 | mm |
| Bolt A below top | 335 | mm |
| Bolt B below top | 170 | mm |
| Middle of pressure zone | 85 | mm |
| Moment per detail | 97 | kNm |
| Moment per connection | 194 | kNm |

*Table 48; Bending moment capacity*

This is the moment with the applying of normal bolts. If the quality of the bolts will be increased to 8.8., the bending moment capacity doubles.

**Shear force capacity**
The shear force of a bolt will be determined with the following formula, and result in the capacity shown in table 49.

$$F_{v;u;d} = \frac{0.6 * \alpha_{red2} * ft_{rep} * A_{bs}}{\gamma_m} \quad [14]$$

| | | |
|---|---|---|
| Bolt A | 136 | kN |
| Bolt B | 136 | kN |
| Total per element | 542 | kN |

*Table 49; shear force capacity*

**Punching of the steel house**
The connection should not collapse on shear. Therefore the shear capacity of the connection detail will be checked. The thickness of the steel house in calculated with the following formula and the results are presented in table 50.

$$F = \frac{L * W * f_s}{y_m}$$

| | | | |
|---|---|---|---|
| Maximal shear force | F | 156 | kN |
| Length of the house | L | 15 | Mm |
| Width of the bolt | W | 30 | Mm |
| Steel quality | $F_s$ | 435 | $N/mm^2$ |
| Material factor | $Y_m$ | 1.25 | (-) |

*Table 50; required thickness steel house.*

The shear force capacity of the connection is 156 kN. The weakest point in the connection detail is the bolt with 136 kN per bolt. Therefore the shear force capacity is 136 kN per bolt and in total 542 kN per joint.

**Normal force capacity**
Due to the water tightness there have to be a normal force between the elements. The normal force capacity of the detail will be determined below. There will be a pre stressing assumed, with a loss of 20%. In a later phase this has to be verified. The results of the calculation are shown in table 51.

$$F_n = f_s * A_{bs} * y_p$$

| | | | |
|---|---|---|---|
| Normal force per bolt | $F_n$ | 226 | kN |
| Steel quality | $F_s$ | 400 | $N/mm^2$ |
| Area Bolt | $A_{bs}$ | 707 | $mm^2$ |
| Effective used prestressing | $Y_p$ | 0.8 | (-) |
| Number of bolts per element | | 4 | (-) |
| Number of bolts in total | | 2150 | kN |

*Table 51: normal force*

This is much more then the required normal force of 9.5 kN per meter as determined earlier in this paragraph.

### Unity checks connections

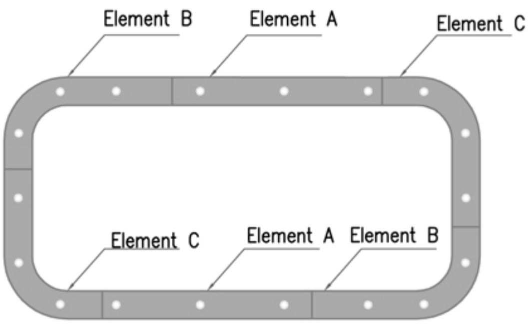



*Figure 52: Named cross sections*

The details are named A to F. The details A to D are normative. Therefore the unity checks of these connections are given. Below the evaluated unity checks are given, the abbreviations are shown in table 52. The values of the unity checks can be found in table 53a and table 53b.

Shear force
$$\frac{Fv;s;d}{F;v;u;d} \leq 1$$

Moment capacity
$$\frac{M;t;s;d}{M;t;u;d} \leq 1$$

Combination shear and moment
$$\frac{Fv;s;d}{F;v;u;d} + \frac{M;t;s;d}{1.4M;t;u;d} \leq 1$$

Normal force
$$\frac{Fn;s;d}{F;n;u;d} \leq 1$$

With

| Abbreviation | Explanation |
|---|---|
| F;v;s;d | Shear force in bolt due to load |
| F;v;u;d | Shear force in bolt due to capacity |
| M;t;s;d | Moment force in bolt due to load |
| M;t;u;d | Moment force in bolt due to capacity |
| F;c;s;d | Tensile force due to load |
| F;c;u;d | Tensile force due to capacity |
| Fn;s;d | Normal force due to load |
| F;n;u;d | Tensile force due to capacity |

*Table 52; abbreviations*

| | capacity joint per meter | Joint force A | A; unity check | Joint force B | B; unity check |
|---|---|---|---|---|---|
| shear force | 543 | 92 | 0.17 | 183 | 0.34 |
| maximum moment KNM | 194 | 141 | 0.73 | 4 | 0.02 |
| combinatin shear and bending moments (-) | | | 0.69 | | 0.35 |
| normal force KN | 2156 | 10 | 0.00 | 10 | 0.00 |

*Table 53a: unity checks*

| | capacity joint per meter | Joint force C | C; unity check | Joint force D | D; Unity check |
|---|---|---|---|---|---|
| shear force | 543 | 127 | 0.23 | 113 | 0.21 |
| maximum moment KNM | 194 | 16 | 0.08 | 21 | 0.11 |
| Combination shear and bending moments (-) | | | 0.29 | | 0.29 |
| normal force KN | 2156 | 10 | 0.00 | 10 | 0.00 |

Table 53b; unity checks

The calculations show that the capacity of the connection principle is large enough. Even while there are bolts with a quality of 4.6 applied. In further research bolts of quality 8.8 are suggested because the price of these bolts is almost equal to the 4.6 quality bolts.
The other optimization that can be made is to lengthen the elements from 1.0 meter to 1.5 meter. This results in fewer connections, and a reduction of the costs. In the chapter costs is this optimization already applied, however there are no detailed structural calculations made, but the previous calculations give enough base to apply this optimization.

Something that is not taken in account in the design and the calculations of the connections is the possibility of non working bolts. There must be paid attention to this in possible phenomena. There must also pay attention to vandalism. To prevent collapsing of the underpass there is suggested to seal the bolts.

## 5.6 Additional design aspects

**Rain water disposal**
To disposal the rain water out the underpass, there is a reservoir to store the water and a drainage system necessary. The normative precipitation quantities in the Netherlands are shown in " Handleiding Wegenbouw, Ontwerp Hemelwaterafvoer [6]". The normative precipitation is a rain shower of 10 minutes with in total 23.0 mm water. Conform the same source the minimal capacity of the pumping system must be 1.2 mm/min. The remaining water can be stored in the reservoir. The capacity is in the following way determined, see formula and table 54.

$$R = (P_t - D_t) * A$$

| | | | |
|---|---|---|---|
| R | Reservoir size | 1.56 | $m^3$ |
| $P_t$ | precipitation in 10 minutes | 0.021 | m/min |
| $D_t$ | Disposoal in 10 minutes | 0.012 | m/min |
| A | Area ramp | 174 | $m^2$ |

*Table 54; normative parameters for design storage.*

This results in minimum dimensions of the reservoir of 1.6 $M^3$ in each ramp. In the middle of the underpass a reservoir of 1 $m^3$ is located stored the remaining water that flows into the underpass. This can be water from cleaning etc. This leads to the reservoir sizes as shown in table 55.

| Reservoir | Location | Size |
|---|---|---|
| 1 | On the end of ramp 1 | 1.6$m^3$ |
| 2 | On the end of ramp 2 | 1$m^3$ |
| 3 | In the middle of the underpass at the deepest point | 1.6$m^3$ |

*Table 55, overview required water storage*

The reservoirs are situated below the asphalt layer and steel plates. The assumed thickness of the asphalt layer is 53 mm. The thickness of the steel plate is assumed on 7 mm. therefore the length of the reservoir is approximately two meter, so two rings. A sketch of the locating of the drainage system is shown in figure 53.

The drainage pipes must be large enough to prevent clogging, but on the other hand fit in the construction. Therefore drainage pipes of 0.10 meter are suggested.

In this preliminary design, without a ballast layer it is very hard to fit a drainage system into the underpass. Probably the method will be executed with a small ballast layer and then it is much easier to locate a drainage system.

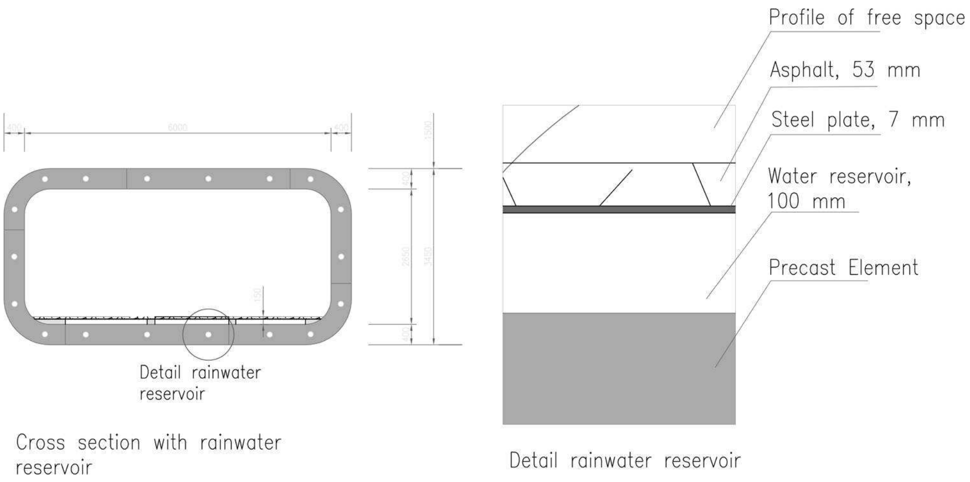



*Figure 53; overview cross section*

**Remaining facilities underpass**

**Asphalt**
There is approximately 60 mm of asphalt required. On the asphalt layer also a water resistant layer will be placed.

**Anti graffiti**
On the walls of the underpass anti graffiti will be applied.

**Light**
The lighting plan will be made in accordance with social safety requirements.

**Fence**
On top of the ramps a fence should be placed to prevent falling into the underpass.

**Cables and sewers**
In the shallow surface there will be several cables and sewers situated. Before the project starts there has to be paid attention where they are situated and if they are on the bore path. For example, if there is a sewer situated parallel to the road it might be possible that the cover must be enlarged.

**Element shape**
During the advance of the TBM process there will made curves in horizontal en vertical direction. To be able to make these curves the elements have the following schematic shape.

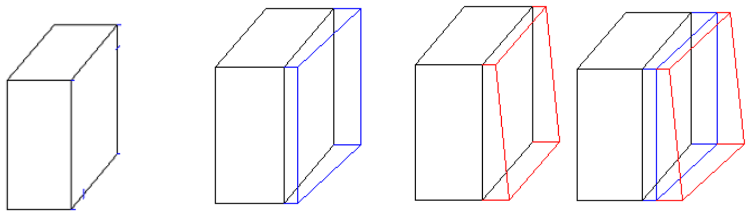


*Figure 60; standard element, and elements for curve to horizontal, vertical and both directions*

## *5.7 Equipment and construction plan*

### 5.7.1 TBM and settlement control

The longitudinal design has a very low cover. The maximal cover is 1.5 meter as described in paragraph 5.3. Therefore an EPB (Earth Pressure Balance) shield must be applied. The dimension of the TBM face is 3.75*7.10 meter. This is on all sides 0.15 m more larger then on the outside of the underpass. The length of the machine is 7 meters. The expected weight of the TBM is roughly 300 ton [24].

The soil will be excavated and disposed through the screw and via conveyor belts transported to the outside of the underpass. The soil will be made plastic by adding foam. Then the soil becomes better compressible which results in a better controllable pressure, and therefore a smaller settlement of the surface.

To prevent rerolling and keeping of the TBM during the tunnel boring process, there are two vertical arms situated on the outside of the TBM. The function of these arms is to gain stability against turning around the length axis. This improves the bore accuracy. With accurate boring a volume loss of between 0.5% should be possible. This may result in the order of settlements as presented in figure 61. conform the method Peck [11]. This is an indicative estimation of what the settlement could be. Detailed soil parameters and rectangular shape of the underpass are for example not taken in account. In the pilot project that is discussed in chapter 2 Obayashi measured settlements of 10.0 mm. During this tunnel boring process a intensive monitoring plan is applied. If unexpected high settlement were registered, immediately measures could have been taken.

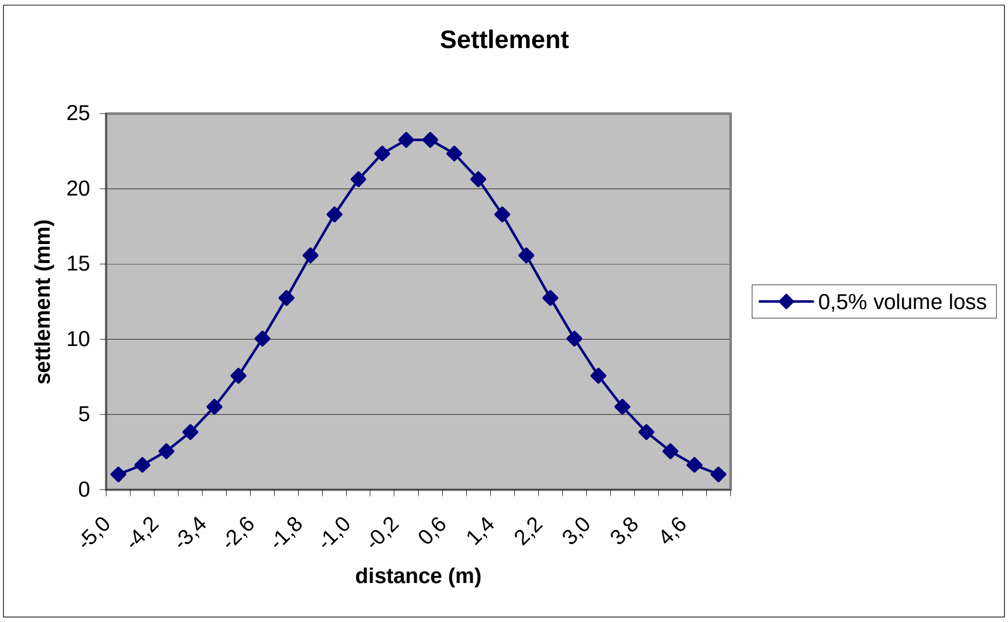



*Figure 61; 1ste estimation of settlement magnitude, sources “bored and immersed tunnel*

## 5.7.2. Launching construction

The launching cradle must be strong enough to take the push of forces and preventing settlement of the TBM before the tunnel boring process starts. Beside this two demands it is also important that the supply train can ride into the underpass. Therefore a profile of free space for the supply train is required. There is chosen for a launching cradle as sketched in figure 62. This is a first concept solution, no detailed calculations are made.

There launching cradle exists of steal beams. The jacks of the TBM will push of on this steal construction. On the locations where the jacks push on the frame, additional reinforcements on the frame will be made. The forces are lead into the soil by sheet pile walls. If in a later phase appears that the construction with sheet pile walls does not perform, there can be grout anchors applied [24].

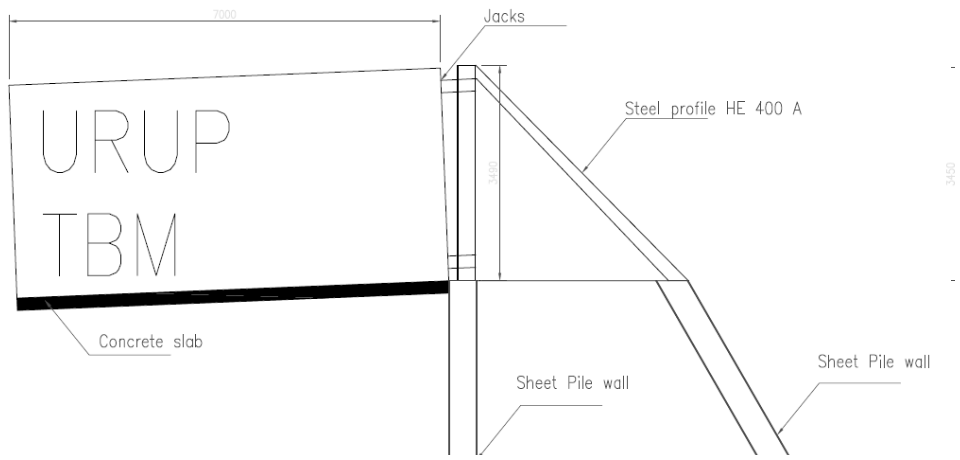



*Figure 62a; side view of the launching cradle*

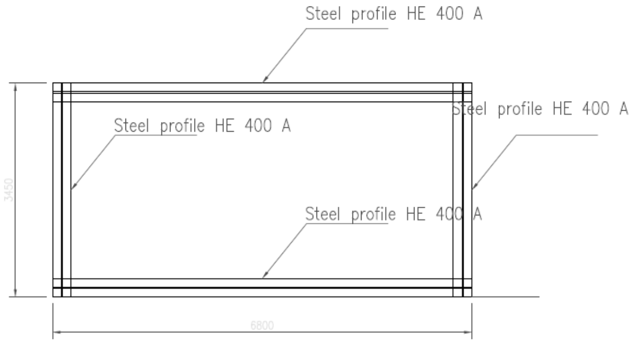



*Figure 62b; front view of launching cradle*

### 5.7.3 Execution sequence and construction plan

A construction plan for the first project is given in annex J "construction plan" [24]. The total duration is 17 months. In the first plan the building and preparation of the TBM is included and takes 10 months. In the next projects there is only maintenances of the TBM, which is several months, therefore the building time in next projects can be reduced to 13 months.

## *5.8 Evaluation preliminary design and points for further design*

Only in the preliminary design of Goes realistic situation a detailed structural design is made. This situation has lower loads then Goes worst situation. However there are not detailed structural calculations for Goes worst situation made, there is expected that in this situation also a lining can be designed that can resist the loads. These expectation is based on the calculations made for Goes realistic design.

The underpass in the preliminary designs is below road not below a railway, this causes a smaller loads as described in paragraph 3.4. On the actual preliminary designs there is expected that there can be a lining made that resists the loads due to a railway.

In the preliminary design some parts are not designed of can be optimized in the definitive design. Underneath an overview of possible points that can be optimized is given.

The chosen main reinforcement configuration fulfills the safety requirements but in respect to the cage stability it is suggested to apply a smaller main bar. This results in more bars and therefore a better stability of the cage.

The type of bolts that is applied is 4.6. In the definitive design a quality of 8.8 is suggested. The additional costs are not so high. Together with this optimization it is also suggested to lengthen the elements from 1.0 to 1.5 meter. Then the amount of connections reduces.

To be able to correct steering inaccuracies a small ballast layer is suggested. The rainwater disposal can also more easily integrate.

On the following items additional research is necessary.

- The type of gasket
- Launching cradle
- Improvement of social safety

# 6. Cost estimations

## *6.1 Introduction and set up of the cost estimation*

**Introduction**
In order to get an insight if the URUP method is economical of interest in the Netherlands, the costs of the URUP method are compared with the costs of the conventional methods that are applied nowadays in the Netherlands. The cost estimations for the URUP project are based on a preliminary design and the cost estimations for the conventional design are based on the definitive design. Therefore the estimations for the URUP method are less accurate then the estimations for the conventional design. The prices are based on the price level of May 2010. The estimations are set up by VHB, based on the detailed input that Obayashi provided. There will be three situations evaluated.

- Goes worst situation
- Goes realistic situation
- Goes railway situation

In “Goes worst situation” the preliminary design of the URUP method is compared with a conventional design supplemented with underwater concrete and sheet piles. From this design no detailed drawing is available, but a global drawing can be seen in, annex K “drawing conventional design". The underwater concrete and sheet pile walls are not shown on the drawing.

In “Goes realistic situation” the preliminary design as elaborated in chapter 5 is compared with the conventional design, based on a bid that VHB made on the tender for that location. The drawing, 80092041-T-954, is shown in annex K “drawing conventional design".

In “Goes railway situation” there has been assumed that the road is a railway, and that the groundwater level is equal to “Goes worst situation”. There are no detailed drawings available of this assumed situation. In this case the URUP method, with a maximum ballast layer is compared with the conventional method supplemented with costs for the railway works.

**Set up of the estimations**
To make the comparisons between the URUP method and the conventional method, as fair as possible the boundary conditions are set equal. Underneath a list of items that are kept the same in every estimation is presented.

- Labor costs: 40 Euro per man-hour
- Costs finalizing tunnel 50.000 Euro
- .General costs 8% of subtotal costs
- Profit and risk: 14% of subtotal costs

The following aspects cover the item “costs finalizing tunnel” in all variants.

- Water resistant layer for foot and bicycle path
- Anti graffiti
- Lights
- Pumping installation
- Sustainability layer for footpath
- Sustainability layer for bicycle path

**URUP estimations**

The URUP estimations are based on the preliminary designs that has been made and are described in chapter 4. Because the estimations are based on a preliminary design the margin of the cost accuracy is set on 20%. Some important assumptions are made in the URUP estimations of which an overview is given below.

- It has been assumed that the project will be executed ten times with the same TBM. This means that the fixed costs, for example the purchase and shipping of the TBM are divided over ten projects.
- In the preliminary design a ringlength of 1.0 meter is assumed. In the cost estimations a ringlength of 1.5 meter is assumed, this causes a reduction in the number of connections and therefore in the costs. This is a reliable assumption as described in paragraph 4.3.
- In the estimation for “Goes realistic situation” the TBM price for the “Goes worst situation” is taken in account. However the TBM for “Goes realistic situation” will be cheaper then the TBM for “Goes worst situation”. This is a conservative assumption.
- The price for the designed detail in the longitudinal joint is assumed on 50 Euro.
- The price for the designed detail in the lateral direction is assumed on 10 Euro
- The ballast layer in “Goes worst situation” is maximal 1.05 meter. The amount of heavy concrete, normal concrete and sand in the ballast volume are divided as mentioned in paragraph 5.3.
- The advantages of the method URUP as mentioned in paragraph 2.1 are not translated into money.

Underneath is shown out of which details each cost items consist.

**Shield machine**
- Purchase of a URUP TBM, per 10 projects
- Transport and insurance by shipping to the Netherlands, per 10 projects
- Additional tools like conveyer belt etc.
- Transport and storage from site A to site B in the Netherlands.

**URUP Lining**
- Reinforced concrete segments
- Labor during excavation by machine
- Soil disposal
- Additives for excavation
- Backfill grout

**TBM preparation and driving costs**
- Cleaning of boring path
- Launching cradle
- Machine assembly and disassembly

**URUP Connections**
- Connection piece
- Connection bolts and nuts

**URUP other finishing works**
- Water resistant layer
- Anti graffiti
- Lights
- Pomp installation
- Anti wear layer for footpath bicycle path
- Design costs
- Ballast materials
- Railing

**Conventional estimations**
The estimations of the conventional design are based on the bid that VHB made on the tender for the location in Goes. The conventional design is in the following way phased: the underpass is divided into two equal sections. Each section is built as follows. First the road is redirected, then the soil is excavated to groundwater level. Then drainage takes place and there will be excavated to the designed depth. Then the concrete work of the underpass is executed. When the first half of the underpass is completed, the second part will be executed.

In “Goes worst situation” the phasing is different then in “Goes realistic situation”. In this project the underpass is also divided into two parts, and the underpass is built part by part. One part is built in the following sequence, there are sheet piles driven into the soil, then the soil is excavated and underwater concrete is poured. After that the building pit is pumped dry and the underpass will be built. Finally the sheet pile walls can be pulled out and possible reused.

In “Goes railway situation” additional costs for the railway works are taken in account. And therefore the price of the conventional method will be larger.

There are some road works added in these estimations, these were not included in the tender. The estimations for the conventional design are based on the definitive design and have a margin of 5% to 7.5%

## *6.2 Economical interest of URUP in the Netherlands*

### 6.3.1 Cost comparison

From the initial cost comparison as described in the previous chapter, it can be learned that the costs of the URUP method used under a railway are two times higher then the conventional methods. However, some remarks are to be made in favour of the URUP method. The time that the TBM is on site is very short, this means that in a given time period many more underpasses can be built.
The hindrance is minimal, because after the THBM has left the site all the finalizing works are inside the underpass.

### 6.3.2 Opportunities

To make the URUP method economical of interest the gap between the prices for the conventional method and the URUP method must be reduced. This can be done in the following manners.

- Motivate the client to pay a higher price due to the advances of the URUP method
- Strategic choice of height ballast layer and cover
- Reduce the costs of the URUP method.

By exploring these opportunities there must be focussed on the first two. Because it is uncertain that the costs of the URUP method can be reduced. There are some cost reducing items that are mentioned below, but there are also lot of uncertainties which can work cost increasing.

**Motivate the client to pay the additional costs**
As stated in chapter 2 the URUP method has advantages, and the main and most important is that it not necessary to cut of the road or rail during the construction process. The client should be motivated to pay an extra price to gain this advantage.

**Strategic choice height of ballast layer and cover**
In the investigated cases the maximum ballast layer or no ballast layer is applied. It is possible to choose a ballast layer between these values, for example 0.70 meter. With this ballast layer can maybe 80% of the projects executed, the other 20% cannot executed because the underpass will buoyancy in that cases. This reduction on the ballast layer reduces the cost significantly and therefore the URUP method is economical more of interest in comparison with conventional methods where ballast concrete and sheet piles are necessary. The optimal thickness of the ballast layer depends also on the location of the potential projects, this has to be determined in further research.

If the cover reduces the ramps will be shortened, as described in paragraph 5.3. A possible measure is to reduce the cover of the underpass from 1.5 meter to 1.0 meter. This reduces the length of the underpass with 25 meter, and therefore the costs.
When an underpass with a cover of 1.0 meter is realized it is necessary to close the road when the TBM underpasses this road, this takes probably several days.

**Reduce the costs of the URUP project**
Underneath some items are mentioned where the costs can reduced. The items are in order of the percentage of the total price and start at the highest percentage.

- TBM preparation and driving costs
- Lining costs
- Connection costs
- Executing time

The price for TBM related costs are based on an estimations that not has been verified. The TBM related costs are a large part of the total price, therefore it is suggested to investigate this assumption more in detail.
The thickness of the lining is determined on 0.40 meter in a preliminary design. If the thickness can be reduced the costs decrease significantly. This is suggested to investigate in further research. The prices of the connection details are based on the preliminary design in the previous chapter. It is possible to reduce the costs of the connections by using less steel. This has to be investigated in the definitive design.

The execution time will be reduced if several projects are executed and a repetition effect occurs. Therefore it might be possible that in the future smaller facility costs can be taken in account, which reduces the price of the URUP method.

# 7. Conclusion and Recommendations

## *7.1 Conclusion*

URUP is an innovative method, developed in Japan, where the TBM is launched from surface. At small underpasses the TBM has a rectangular face. The method is only executed in soils that can be characterized as soft and cohesive. The groundwater level was very low and not of any influence at these projects. The URUP method is not only applied for small underpasses but also for large diameter tunnels with a circular cross section such as the Ooi tunnel in Tokio.

In the Netherlands the length of the underpass should be shorter then 250 meter, because if this length is exceeded additional tunnel laws come into force. The underpass should be social safe, this results in a minimal required profile of free space of 6.00*2.50 meter. The maximal allowable slope for cycle and pedestrians underpasses is 4.0%.

Unlike the soil conditions in previous projects in Japan, the soils conditions in the Netherlands have in general a high groundwater table. This causes buoyancy of the underpass. In the worst case a ballast layer of one meter is required to prevent buoyancy.

The possibility to adapt the URUP method in the Netherlands is investigated through a selected case, which is an underpass in the town of Goes. Preliminary designs were made for "Goes worst situation" and "Goes realistic situation". In "Goes worst situation" the groundwater level is lifted to the top of the road, therefore the maximum ballast layer of 1.05 meter is required to prevent buoyancy in the Netherlands. In "Goes realistic situation" there is no ballast layer required. The absence of a ballast layer results in a shorter alignment for "Goes realistic situation".

The lining exists out of reinforced concrete elements because potential client ProRail refuses steel underpasses. The lining is built ring by ring with a length of 1.0 meter. Longer elements mean fewer connections, but it must also be possible to correct small bore deviations. There are two types of rings applied which are mirrored to each other. This is done to gain more building accuracy.
The longitudinal joints are situated on locations where the bending elements are relatively low, this is roughly at ¼ and ¾ of the span. The connections are part of the definitive solution. In the case study has been chosen for a "steel box solution" . The box is embedded in the prefab elements, later bolts connect the boxes together. In the lateral joint the same principle is applied, but these connections are executed in a lighter variant.

The costs of the URUP method are compared with the conventional method. In "Goes railway situation" there is assumed that the road is a railway and that the groundwater table is very high, so there is made an underpass below a railway. In this situation the costs of the URUP method are compared with the costs of the conventional method supplemented with additional works for the railway. The method URUP is 2 times more expensive.

Based on these research can be concluded that the costs of this new and innovative construction method exceeds the costs of the conventional method by 2 times. This method is only of economical interest when large number of underpasses need to be constructed in a short time and a minimal hindrance is of the essence. Further research may cause a reduction in the price difference between both methods.

## *7.2 Recommendations*

To get a better understanding and a more optimized design of the URUP method, the following aspects are recommended to adapt, or to investigate more in detail.

**Technical**
In case the method is industrialized executed a small ballast layer is recommended. In this layer additional design aspects such as a water disposal system can be situated. Bore deviations during the construction phase can also be corrected by changing the thickness of the ballast layer.

To increase the social safety in the underpass it is suggested to pay more attention to the design of the walls and light installations.

In the definitive design there are optimizations possible related to the structural design of the underpass. Below an overview of the topics is given:

- Reinforcement of the elements
- Length of the elements in longitudinal direction
- Design of the connection detail.

For the launch construction a global idea is developed, in a later phase a detailed design must be made.
A first order estimation of the expected settlements has been made. This must be verified with additional research. In order to reduce the magnitude of the settlements due to the construction process a detailed TBM design together with a monitoring plan must be developed.

**Economical understanding**
To reduce the price difference between the URUP method and the conventional method the following opportunities can be investigated. The focus should be on the first two opportunities, because these are more realistic.

- Motivate the client to pay a higher price for the URUP method due to the benefits
- Strategic choice of height ballast layer and cover
- Reduce the costs of the URUP method.

Further market investigation there can be made a comparison with the trenches technologies. Nowadays roads and railways will not be closed for making a sewer, altrough this is sometimes cheaper.

Beside this there can also be looked at other markets. Pipe owners such as “Gasunie”and “Waternet” may be potential clients when they join forces and construct utility tunnels for their cables and pipes. The square cross section has advantages for them.
The market for ecological passages, for a and fauna, is growing and have possibly the same dimensions.

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## Annex A: Buoyancy calculation of an underpass under typical Dutch conditions

## Annex B: Determining Boundary conditions, requirements and case selection

## Annex C: Soil Profile case Goes

## Annex D1: loads on top slab definitive phase

## Annex D2: loads during push of process and during tunnel boring process

## Annex E: Report determination buoyancy Goes

**Annex F: Drawing P3745.002 Goes realistic situation**

**And**

**Annex G: Drawing P3745.001 Goes worst situation**

**Annex H1: drawing longitudinal connection**

## Annex H2: drawing lateral connection

## Annex I: properties gasket

## Annex J: construction plan

**Annex K: drawing conventional design**