Master Thesis Report

The tunnel design connecting A15 and A12 crossing the Pannerdens canal

by

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This thesis is confidential and cannot be made public until October 30, 2020.

Preface

This thesis is my last product of the Master of Science in Geotechnial Engineering at the Faculty of Civil Engineering and Geosciences of the Delft University of Technology. It is in cooperation with Arcadis and Delft University of Technology. It is completed with assistance of my committee and friends.

First of all, I would like to express my gratitude to all my committees. Thank Wout Broere for guiding me throughout the whole thesis and always keeping smile. He gave me lots of advice to progress my project. Thank Wilfred Molenaar for guiding me on hydraulic knowledge and giving me very good ideas on meetings.

Secondly, many thanks to Sallo Van der Woude, my supervisor in Arcadis. He gave me good ideas from aspect of project and he sent me feedbacks on my thesis with good comments in order to make my report nicer. Also, many thanks to Arcadis for providing me good working environment with lots of skillful engineers. Many thanks to Ronald Heijmans for cost estimation of the project and background knowledge of Pannerdensch railway tunnel.

Last but not least, thank my friends for helping me and giving feedbacks on my report and my family for supporting me. Sincere thanks to my girlfriend for inspiring, encouraging and accompanying me.

Jiantuan Qin
Delft, October 2020

Contents

1 Introduction

This master thesis aims to do a detailed conceptual design of a tunnel to connect the road A15 and A12, which will cross the Pannerdens canal (represented by red color in figure 1.1) and a deep comparison with the bridge plan solution which has already been adopted by the government, which was named ViA15 project.

This chapter starts with a background introduction of this project (ViA15 project), including how the the bridge solution was formed. Thereafter, the research motivation and approach used in master thesis will be described. Finally, the report structure will be detailed.

Figure 1.1: *Overview of Pannerdens canal (Ministry of Infrastructure and Water Management)*

1.1. Background of the project

With an increase in vehicle flow, traffic jams often occur on the A50, A12, A325 and the Pleijroute (N325) (see figure 1.2). The traffic problems will become even greater in the future despite a series of updating measures have been made, like widening of A50 and A12, upgrading the N18 and construction of a second bridge at Nijmegen. The traffic conditions of the region and the reliability of the national and regional main road networks cannot reach expected quality.

The accessibility problems have a negative effect on the international attractiveness of the Randstad . The Arnhem region-Nijmegen itself can no longer spatially and economically develop well. Due to the overload, it has caused problems with the quality of life (shortage of traffic capacity, air pollution) in the region. In addition, crowded traffic bring greater risk of accidents. To solve these traffic problems, the central government, the province of Gelderland and the Arnhem Nijmegen city region have put their hands together to extend A15 to A12.

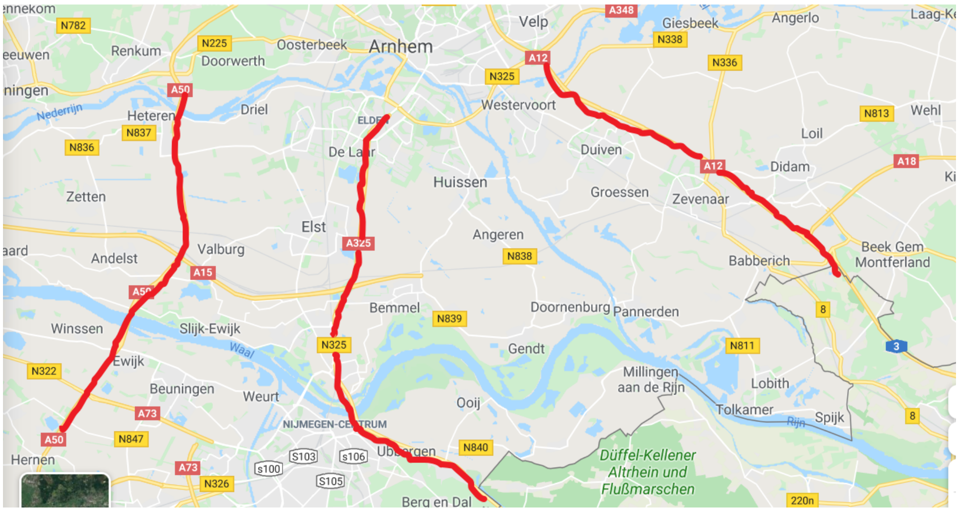


Figure 1.2: *Overview of A50, A12 and N325(Google Maps)*

1.1.1. Introduction to the nearby Betuweroute railway tunnel

In the vivinity, a railway tunnel has been built and is in service since 1990s, namely the Betuweroute railway tunnel. The decision-making process of this railway tunnel project may serve as a reference case for the conceptual design in this master thesis.

Decision making between bridge and tunnel

In 1990s, the Betuweroute railway tunnel was built to cross the Pannerdens canal. The main reason for building a tunnel at this location is the natural value of the nature reserve Rijnstrangengebied which is situated in the eastern floodplain of the Pannerdensch canal. Rare species living there were dorsal stripe toads, crested newts, toades and salamanders (see figure 1.3). Ponds had to be designed for these creatures to live to compensate for the habitat lost when constructing the eastern approach. At an earlier stage of the project, a bridge was also considered. However, there was a strong political lobby in favour of the tunnel to limit noise hindrance for the people living in Boerenhoek. Because of the fierce opposition to the proposed bridge, it was ultimately decided to make the river crossing by means of a tunnel.

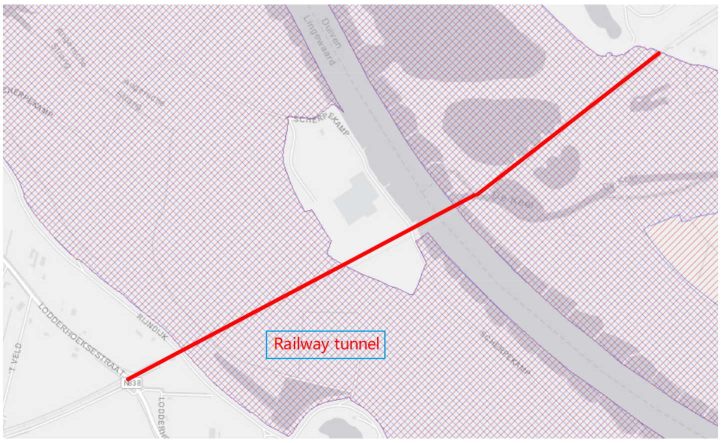


Figure 1.3: *Nature reserve in Pannerdens canal (Nature 2000 Network Viewer)*

Decision making between bored and immersed tunnel

When it was decided to build a tunnel, several designs were considered,including a concrete immersed tube tunnel,a steel-shell immersed tube tunnel and a bored tunnel. For the Pannerdens canal tunnel, despite the greater depth of a bored tunnel, the length equals that of an immersed tube tunnel due to the local condition. As there is no existing construction dock available for concrete immersed tunnel in the eastern part of the Netherlands, due to the shallowness of the river, it is not possible to use the existing construction dock in the western part of the Netherlands. Alternatively, a construction dock could be created at the site. However, the wide variation in water levels in the river made this solution relatively expensive. Also, for an immersed tunnel, the location found under the brickfactory is contaminated with pyrite which needs to be re-mediated. Whereas, it does not need to be re-mediated for bored tunnel because it is situated in a greater depth.

A steel-shell immersed tunnel allows the construction dock to be remote from the actual construction site because the draft is much less than that of concrete elements. The cost, however, was higher than that of a concrete immersed tunnel.

Since the preliminary design showed the concrete immersed tunnel and bored tunnel turned out to be the closest in construction cost, considering environmental aspects and hindrance during construction as well, it was further decided to build a bored tunnel.(see figure 1.4)

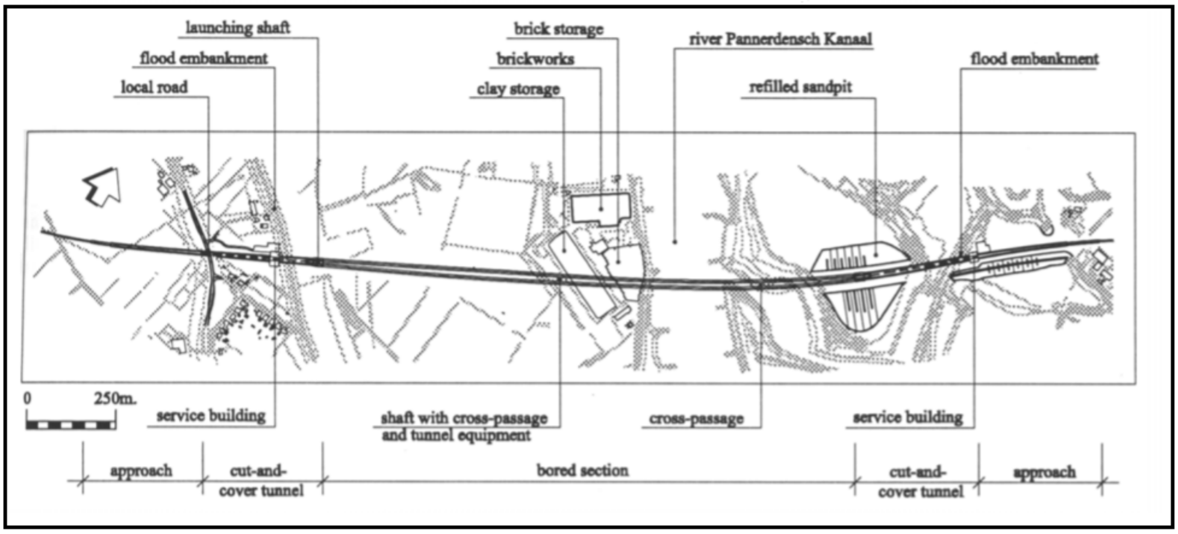


Figure 1.4: *Betuweroute railway tunnel alignment(Tunnelling and Underground Space Technology)*

Construction of Betuweroute railway tunnel

The railway line of the Betuweroute, runs through the former sand extraction pit Kandia. A sand dam has been constructed specifically for drilling the tunnel.(see figure 1.5)

When constructing the launching shaft, the contractor took measures to guarantee the required safety level of the dyke sections. A temporary ring dike and flood defenses were installed and provided the shaft with a dense block of low-strength mortar to guarantee high water-tightness between the shaft and the existing ground. This is necessary because the ground level is relatively high here, at NAP+11.0 meters, while strongly fluctuating water levels occur in the Pannerdensch Canal, varying from NAP+6.25 meters to NAP+16.32 meters.

At the receiving shaft, in order to enable drilling through the Kandiaplas, a total of $700000m^3$ of sand with a thickness of two meters has been filled. Large vibrating needles were used to obtain the correct oedometer stiffness.

The bored tunnel is located just 2.5 meters below the existing ground level at the nearby location of the receiving shaft. This buried depth was only 0.28 times of the tunnel diameter. To handle potential risks of ground blow-out, extra top loading was applied by a layer of magnetite, an iron-containing ore with a specific mass of 34 kN / m^3.

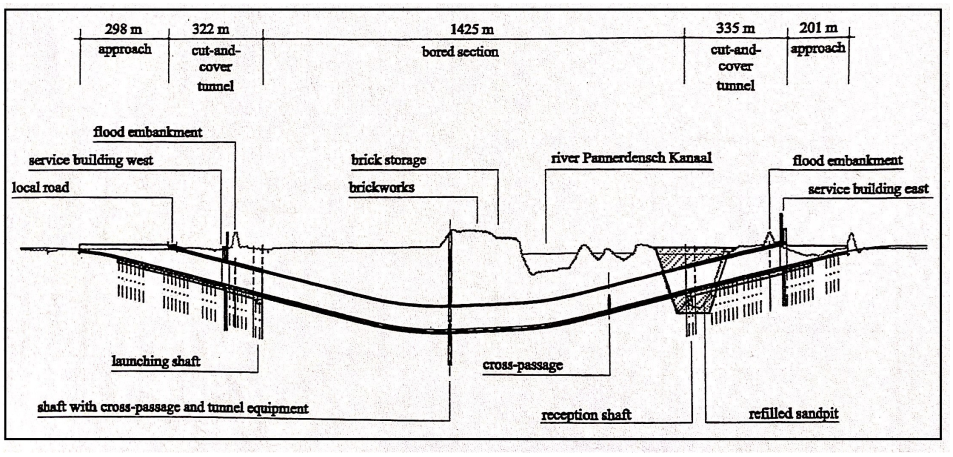


Figure 1.5: *Longitudinal section of railway tunnel(Tunnelling and Underground Space Technology)*

1.1.2. The current three alternatives

In the preliminary conceptual design phase of ViA15 project, there are three alternatives:

Alternative 1— Break-through

This alternative is also called North/South Transit Alternative, the A15 is extended from the Ressen junction to the A12 between Duiven and Zevenaar crossing the Pannerdensch canal. These alternatives also widen the existing A15 between Valburg and Ressen and the A12 between Duiven and the Oud-Dijk junction.(see figure 1.6)

Figure 1.6: Alternative 1(Trajectnota/MER samenvatting)

Alternative 2—Region combi (structure) alternatives

In this alternative, the capacity of the A12, A50 and Pleijroute (N325) is increased, in combination with an optimal use of public transport in A50, A325, A12 and Betuweroute. In this alternative, the A15 is not extended (see figure 1.7).

Figure 1.7: *Alternative 2(Trajectnota/MER samenvatting)*

Alternative 3—Bundling alternative A15

This alternative is a continuation of the A15 from the Ressen junction to the A12 east of Zevenaar crossing the Pannerdensch canal. The extension of the A15 follows the route of the Betuwe Route longer in this elaboration. In this alternative, the existing A15 between Valburg and Ressen and the A12 between Duiven and Oud-Dijk junction will also be widened (see figure 1.8).

Figure 1.8: *Alternative 3(Trajectnota/MER samenvatting)*

Preferred solution

In July 2012, the Minister of Infrastructure and the Environment made the choice for the A15 North Transit Alternative with a wider bend around Groessen as a preferred solution. In their draft route report (Decision, 2012), the cost of the South Transit Alternative is above the available budget(€ 804 million). This is due to the following reasons: this solution has a deepened location near Zevenaar and crosses the Betuweroute and the Arnhem - Emmerich Railway. As a result, this alternative is therefore deepened between the two tracks.

Regiocombi (structure) with only adjustments on the A12 shows the smallest improvement. A survey shows that companies find the A15 North Transit Alternative the best alternative, followed by closely the Bundling Alternative because this option leads to fewer driven kilometers on local and provincial roads. This ensures a better score on road safety for the North Transit alternative and the Bundling alternative than for the Region Combi (structure) alternatives.

Bundling alternative has an adverse effect on the quality of life in Zevenaar, especially when it comes to noise and fine dust. There is also a greater number of homes that must disappear in this alternative. There is a negative effect on social cohesion, because residential areas of Zevenaar are separated from each other.

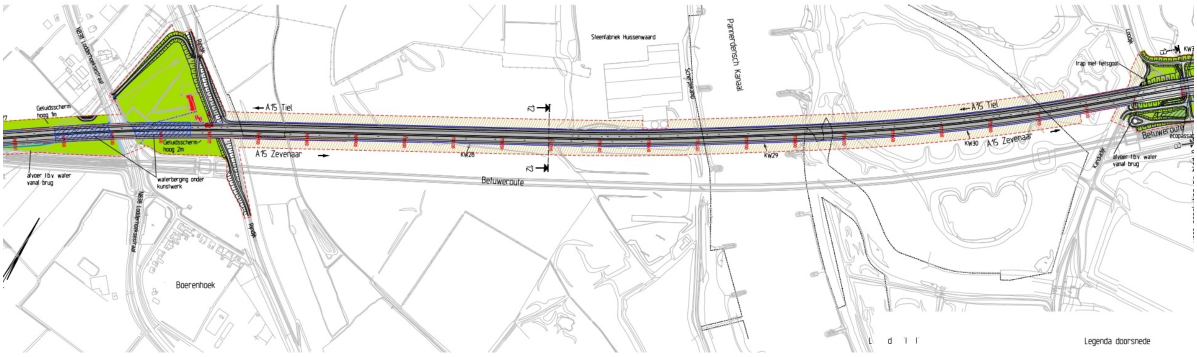


Figure 1.9: *Detailed alignment of bridge across canal (Ministery of Infrastructure and Water management)*

Comparison between tunnel and bridge

The government has chosen the bridge (see figure 1.10 and figure 1.9) as the crossing over Pannerdens canal instead of a tunnel. Stakeholders in favor of a tunnel mentioned that there is less burden on nature and landscape, less disturbance of the living environment of local residents and less use of space. Also, Betuweroute railway tunnel has already been built for many years.

In the views of the government (Views (2011)), as arguments in favor of a bridge, the participants mentioned that it offers the possibility of creating a bicycle connection across the Pannerdensch Canal, that a bridge is beautiful and can form a landmark in the landscape and that the costs are lower.

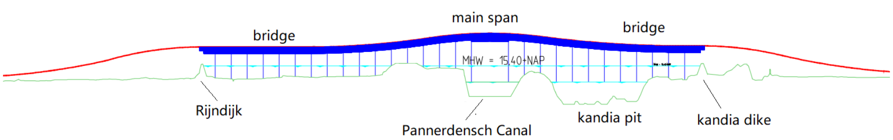


Figure 1.10: *The profile of the bridge solution(Ministery of Infrastructure and Water management)*

In the consideration of the minister of Infrastructure and Environment, opting for a tunnel instead of the bridge under the Pannerdensch Canal is € 210 million more expensive in construction and also costs € 5 million more annually in management and maintenance. No budget is available for these additional costs as the available budget is in accordance with the MIRT project € 804 million (price level 2011). In the 1990s, a railway tunnel was chosen for the Betuweroute. The minister thought this was not legally necessary at the time and the choice was made in other economic times.

The Commission EIA considers the reason why the bridge option is cheaper is that the effects of the move through bridge on nature in the Gelderse Poort have not been sufficiently investigated. The Commission recommends further elaboration of these effects. While, the minister did not agree with that and thought a plan-level assessment has been developed in accordance with the Guidelines.(Decision, 2012)

The comparison among the aspects and criteria is shown in table 1.1. From the table,the bridge solution has more negative impacts than tunnel on spatial structure, noise, nature reserve, social aspect and landscape, cultural history and archeology. But based on the plan-level assessment on nature, the analysis made minister confident that a bridge over the Pannerdensch Canal can be realized within the requirements of the Nature Conservation Act by taking some mitigating measures.(Report, 2011)

Finally, the minister insisted the Northern Transit Alternative (with bridge) has the highest social benefit / cost ratio of all alternatives. This ratio is 2.4, making this alternative a very cost-effective investment.(Decision, 2012)

Table 1.1: Comparison between the bridge and tunnel (Ministery of Infrastructure and Water management)

Aspect and criterion	North transit alternative	
	Bridge	Tunnel with dike
High water safety		
Robust network for evacuation at high tide	++	+
Spatial structure		
Bundling with existing infrastructure	86%	91%
Sound		
Change noise-affected surface	0/-	0
Change noise-affected surface to quiet areas	0	0
Nature		
Influence Nature2000 - Gelderse Poort	–	-
Influence on EHS - River area	–	-
Landscape,cultural history and archeology		
Influencing landscape values	–	-
Influence on amenities	–	-
Social aspect		
Visual hinder (houses)	100-110	90-100
Cost (in mln €)		
MIRT Investment costs	750	960
Maintenance costs for road infrastructure (per year)	10	15
Note: ’++’ means the criterion has high positive impact, ’+’ means slight positive impact, ’0’ means impact is neutral, ’-’ means slight negative impact, ’–’ means high negative impact		

The A12 / A15 Ressen - Oudbroeken (ViA15) project consists of three parts:

- extending the A15 from the Ressen junction to the A12 between Duiven and Zevenaar
- widening the A12 between Westervoort and the Oud-Dijk junction
- widening the A15 between the Valburg junction and the Ressen junction

They have made some changes to draft route decision which states following: (see figure 1.11 on page 10)

- A15 Valburg - Ressen: extension to 3 lanes;
- 15 Ressen - A12: new motorway with 2x2 lanes with connection to N839 and N810 and a new junction on A12 / A15 between Duiven and Zevenaar;
- The new A15 will have a (semi) deepened location from Groessen to the A12;

- A12 Westervoort - Oud Dijk: extension to 3 or 4 lanes with new Zevenaar-Oost connection and closure of the existing Zevenaar connection;

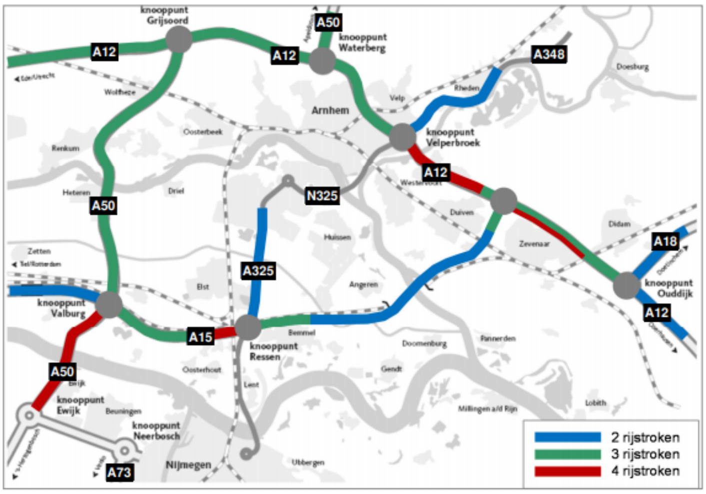


Figure 1.11: *New distribution of the route (Ontwerp Tracébesluit A12/A15 Ressen - Oudbroeken (ViA15) Deelrapport verkeer)*

At the last stage of conceptual design, the project adopted some amendments, which includes the nature, noise control, bridge size and soil wall Bemmel. The ViA15 project involves extending and connecting the A15 to the existing A12. This creates a direct connection between the port of Rotterdam and Germany. This expansion of the road network and the widening of the A12 and A15 also improves regional traffic flow. As a result, road users travel faster between Nijmegen / Arnhem, the Liemers and the Achterhoek. Therefore, it benefits the regional economy and employment.

Until now, the whole ViA15 project is still in design stage and has not broken ground yet. Even if they have chosen the North Transit Alternative with a bridge as the route decision and the alignment and road are fixed, there are still some risks which make the design not straightforward. The environmental impact is an important issue because the area between Bemmel and Zevenaar is a nature reserve. There are limitations about the noise level, release of nitrogen oxides, landscape and habitat of animals living nearby. And insufficient investigations about the impacts of the bridge on the nature make the cost lower than tunnel.

1.2. Motivation of the study

The background of the project is shown above, to tackle the traffic jam on A50,A12,A325 and N325, the government has finally chosen the North Transit Alternative with bridge across the Pannerdens canal. But the Commission EIA considers that the effects of the bridge on nature in the Gelderse Poort have not been sufficiently investigated. This could be the possible reason that the cost of bridge is € 210 million cheaper than the tunnel solution. The Commission recommends further elaboration of these effects. These impacts include the nature reserve, nitrogen oxides and noise. What's more, as a highly potential solution, a tunnel crossing the Pannerdens canal should be fully considered. This tunnel solution is going to be designed, elaborated, optimised and finally compared with the bridge solution on these aspects.

1.3. Aim

In the ViA15 project, bridge solution has brought the environmental impacts and there is already a railway tunnel through the canal and it is proved to be economical, so it is necessary to take tunnel option into consideration. A tunnel is going to be designed to go through the Pannerdens canal. A bored or an immersed tunnel are the two options and they will be evaluated on time, cost, the available capacity for vehicles, river discharge capacity and environmental effect of construction method. So the aims of this master thesis can be formulated as follows: (1) Design a tunnel connecting A15 and A12 going through the Pannerdens canal and; (2) Make comparison with bridge option on final cost.

1.4. Approach

This section mainly talks about the design method which is used in this MSc thesis to progress the tunnel design. The principles of the basic design loop by Roozenburg and Eekels (Roozenburg and Eekels, 1995) is going to be used. The figure of the design cycle is shown in figure 1.12

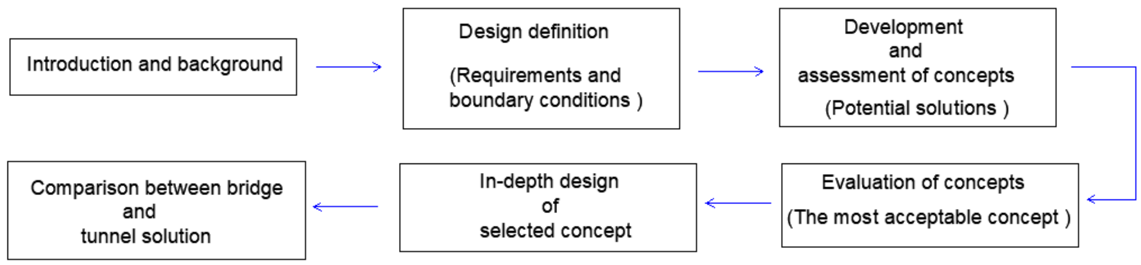


Figure 1.12: *Design cycle (Roozenburg Eekels, 1995)*

The first part is the problem analysis in which the problem is going to be stated. The problem is analysed from the aspects of bridge and railway tunnel decision-making process. After exposing the problems, a design should be defined in terms of requirements and boundary conditions. Based on the design definition, the possible concepts can be developed.

After developing concepts, they need to be verified from the points of alignment, brick factory, shipping and influence on environment. Then the verified design concepts need to be evaluated from a Multi Criteria Analysis (MCA) and cost estimation from which the best design concept is selected. Next step is the in-depth

design of the best design concept. Finally, it is about the optimisation of the best design concept.
Figure 1.12 is an outline of this study, which should be made into details to make the design much clearer.

Initiative: The first step is to collect data of the area between A15 and A12. The data includes geological conditions via DINOloket, the distribution of residential areas on Google map, water depth in the Pannerdens canal and width of canal. Besides, the website trace besluit ViA15 should be fully used as the government is planning to build a bridge across the canal instead of the tunnel because only the bridge is within the budget. The government has put a lot of documents on websites which include the decision making process, requirements, alignment and boundary conditions. All these documents and references are good sources for design.

Design definition: In this phase, the requirements and boundary conditions are taken from the bridge and tunnel design. Besides, it combines with guideline NOA 2007. Requirements are about the number of lanes, width of lanes and speed. Boundary conditions mainly focus on high fluctuating water level and geological conditions.

Development and assessment of concepts: Based on the requirements and boundary conditions, some potential concepts can be developed and they are a bored tunnel and an immersed tunnel. Then they are assessed from aspects of alignment, realistic existing and influence on shipping.

Evaluation: Based on the concepts which have been verified, this step is to evaluate the verified concepts using a Multi criteria analysis. The criteria are impact of construction method on the environment, available capacity for vehicle, river discharge capacity, time and cost. Finally the best design concept is selected from the MCA score.

In-depth design: If the concepts satisfy the evaluations, then this phase is in-depth design. It includes the calculation of uplift, tunnel lining, face stability, normal force, bending moment, thrust force, settlement trough and cross passage. Then it is the in-depth design and optimisation of the shaft to make it as small as possible to save cost. Finally the cost on tunnel design is going to be compared with bridge on cost.

1.5. Thesis Structure

The structure of the thesis is consistent with the above-mentioned approach . Chapter 1 is mainly about the decision making process about bridge and railway tunnel. In chapter 2, the report will focus on requirements and boundary conditions. In chapter 3, based on the chapter 2, there will be a rough design of the cross section of the tunnel and assessment of developed concepts. Chapter 4 is an evaluation of the verified concepts and then the best concept is selected based on a sensitivity analysis. In chapter 5, the preliminary design of the best concept will be elaborated and then in chapter 6, the in-depth design and optimisation of the shaft will be made. In chapter 7, comparison between bridge and tunnel solution is carried out and finally conclusion is drawn after comparison.

2

Design definition

In this chapter, the design will be defined from the point of requirements and boundary conditions. In order to be comparable with the bridge solution on the same level, firstly, it illustrates the requirements from bridge and tunnel design in the design report (Designnote, 2011), and then combines with the requirements from NOA 2007. Thereafter, the boundary conditions from the Betuweroute railway tunnel on aspects of water level and geological condition are taken into account.

2.1. Requirements

Bridge design

As we could see from the profile of the bridge(see figure 7.1), the pavement width per direction of travel is set at 12.50m. The bridge has no provisions for cyclists or slow traffic. The number of lanes on the highway is 2×2 with emergency lanes.

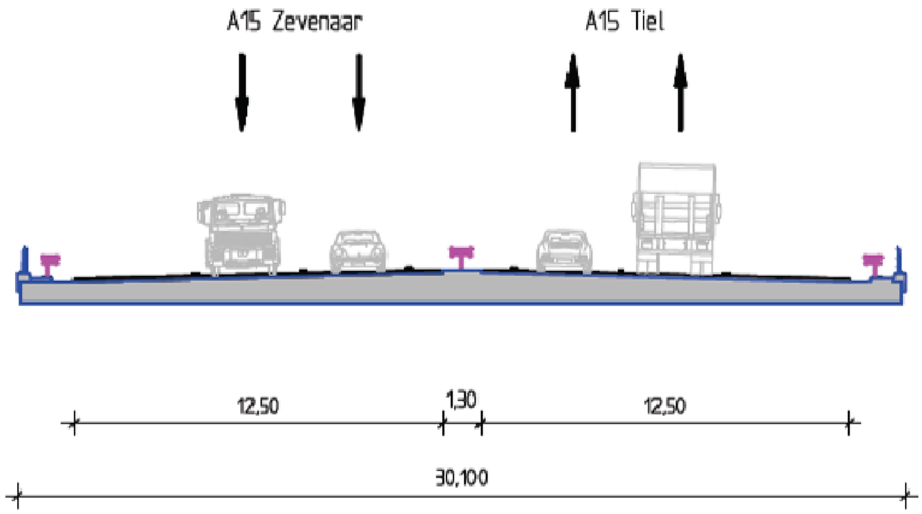


Figure 2.1: *New distribution of the route (Ontwerp Tracébesluit A12/A15 Ressen - Oudbroeken (ViA15) Deelrapport verkeer)*

Tunnel design

For a bored tunnel, the minimum clearance height in the tunnel has been set at 4.70 m, which is higher than prescribed by the guidelines (4.60 m). This is chosen because height clearance does not require the use of height detection, which makes the tunnel less susceptible to malfunction and increases safety. Tunnel installations (TTIs) will be built in the space above 4.70 m. The rainwater drainage takes place in the empty space under the road construction. The pump cellar can also be included in this.
The bored tunnel from the design report is shown in figure 2.2 and figure 2.3 (Designnote, 2011). It is 2 × 2 lanes, there is no emergency lane. Due to the high costs, no hard shoulder is laid in the tunnel. For safety reasons, a 1 m redress strip has been used to provide vehicles with relief space.

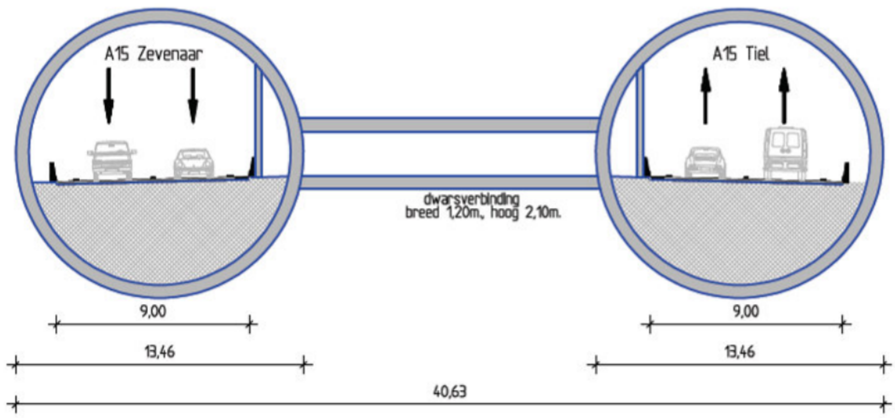


Figure 2.2: *Cross section of bored tunnel(Platform of particioate ViA15)*

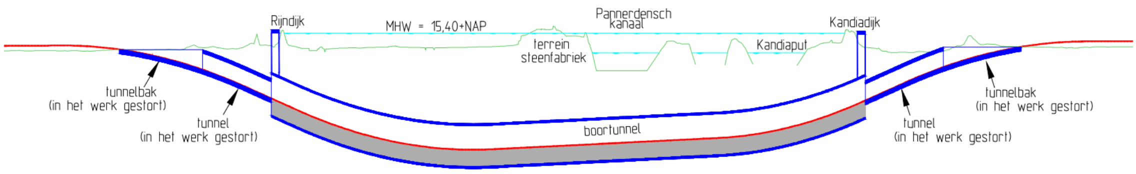


Figure 2.3: *Profile of bored tunnel(Platform of particioate ViA15)*

For the immersed tunnel,the construction of an immersed tunnel has a major impact on the flood plains and shipping, because it is being constructed from ground level. This means that large excavations are taking place in the flood plains, after which the tunnel elements enter and are sunk (see figure 2.4 and figure 2.5). From the cross section, there is no emergency lane as well, but there is a redress strip to provide vehicles with relief space.

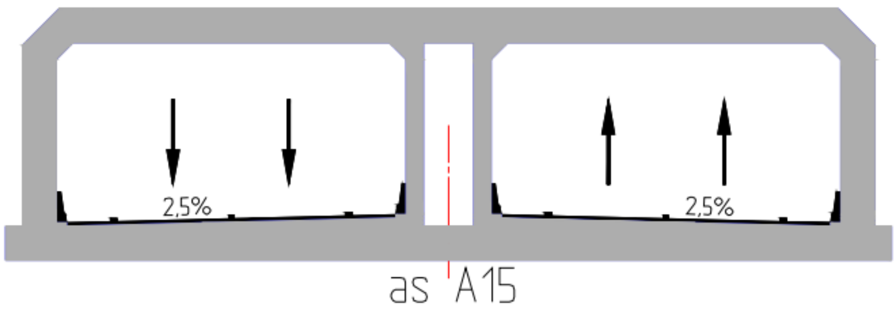


Figure 2.4: *Cross section of immersed tunnel(Platform of particioate ViA15)*

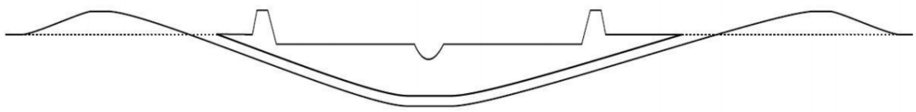


Figure 2.5: *Profile of immersed tunnel(Platform of particioate ViA15)*

NOA 2007

The width of the lanes are determined in accordance with NOA 2007 in figure 2.6 and the design speed is 100km/h.

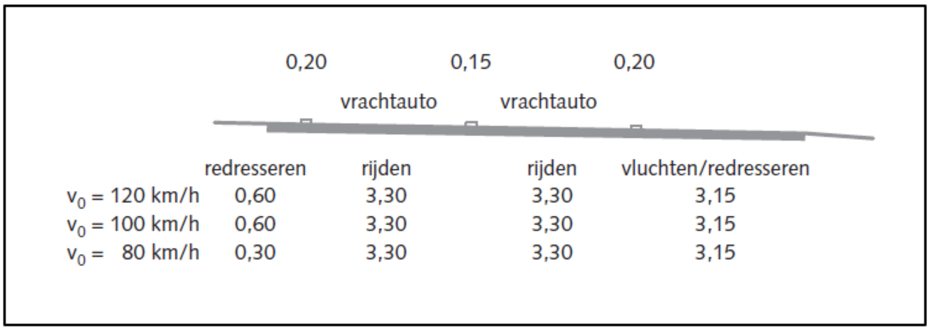


Figure 2.6: *NOA 2007*

The design of the cross section is determined by the clearance envelop of the road, required capacity (number of lanes, need for emergency stopping lane), sidewalks/curbs, tunnel drainage (rain water and waste water), space for ventilation and lighting, space for emergency access and escape and so on. (K.Reinders, 2018)

Lateral clearance is necessary between the tunnel walls and lanes. The aims of the lateral clearance are to avoid shoulder fear and create walking space for tunnel personel or drivers. If the wall is located close to the lane, cars will deviate from the wall, especially at high speed, thus decreasing the capacity of that lane. The higher the design speed, the wider the lateral clearance.

A relatively wide lateral clearance is preferred. However in order to keep the costs low, the width is limited, a guiding profile is used (see figure 2.7). This type of barrier has the ability to absorb the impact of a moving vehicle, guide the vehicle back into its original line of travel while the operator retains control depending on impacting angle of vehicle.

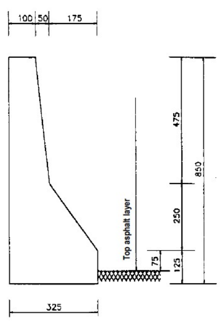


Figure 2.7: *Jersey Barrier (measures in mm)*

2.2. Boundary conditions

Water level

The water level is always changing with time, in winter time it will be relatively higher. The river bed in Pannerdens canal fluctuates from NAP+3.0 to NAP+4.7m. Water level in the period 1991-95 is shown in figure 2.8. The river has a high fluctuation from +NAP 6.25 to +NAP 15.4m. Because of the permeability of the soil, these fluctuations have an instant effect at a great distance from the river. The piezometric level can be higher even than the ground level. In winter, when the discharge of the river Rhine increases, the whole stretch of land between the flood embankments is flooded.(R.W.M.G.Heijmans and J.A.G.Jansen, 1999)

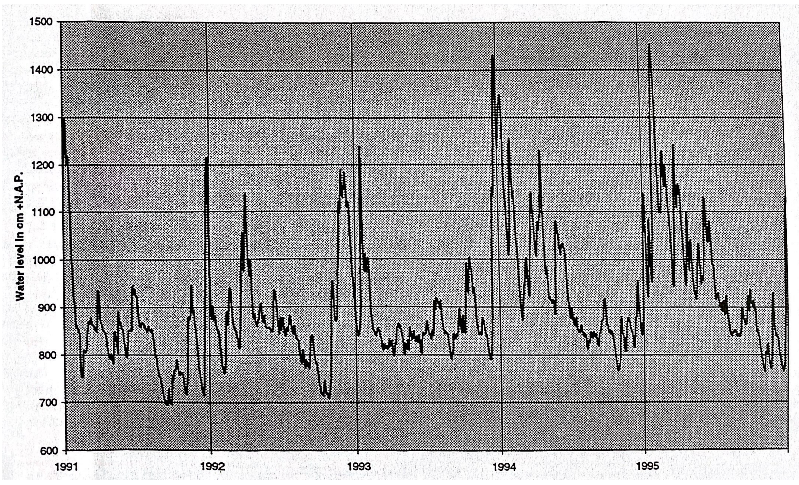


Figure 2.8: *The water level in Pannerdens canal during period 1991-95(Tunneling and Underground Space Technology)*

Geological conditions

As for the geological conditions of North Transit Alternative which is parallel with Betuweroute from Boerenhoek to the receiving shaft of the railway, the geological data is from DINOloket and then is presented as a geological map in figure 2.9. The soil types in this area consist of mainly medium sand to dense gravely sand.

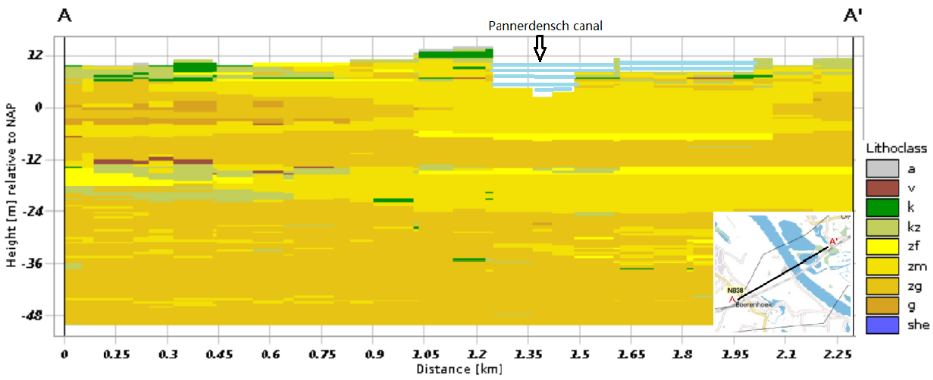


Figure 2.9: *Geological condition (DINOloket)*

During the construction of the railway tunnel, at some locations on the west bank of the river there is a 2-3 meter thick layer of stiff to very stiff silty clay. At two locations, a thick layer of stiff to very stiff organic clay was encountered. This layer was about 12m thick, completely covering the tunnel cross section.(see figure 2.10). So, it is possible to encounter stiff to very stiff clay in the North Transit Alternative around that depth.

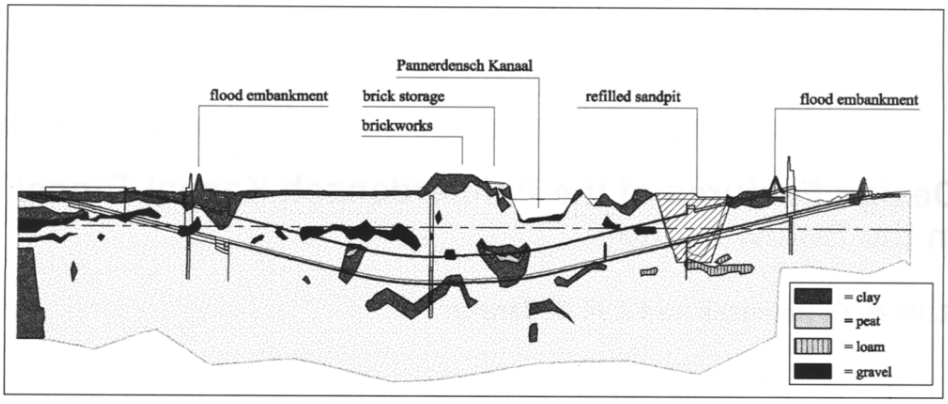


Figure 2.10: *Geological condition of railway tunnel(Tunneling and Underground Space Technology)*

2.3. Conclusion

It can be concluded that as for requirements, the number of lanes should be 2×2 with emergency lane or 2×2 with 1m redress lane. The width of the lanes is determined according to NOA 2007, which is different from both bridge and tunnel design. As for boundary conditions, the Pannerdensch canal has a high fluctuating water level which could flood the whole stretch of land between the flood embankments. It is possible to encounter very stiff clay on the west bank of canal in the North Transit Alternative.

3

Developments and assessment of concepts

3.1. Introduction

In this chapter, based on the requirements and boundary conditions, some concepts will be developed. Two types of tunnel are taken into consideration: bored tunnels and immersed tunnels. This section will first develop the cross section of the design concepts and then assess them from aspects of shipping, influence on railway tunnel and brick factory on west bank of river. Finally concepts that are deemed feasible are confirmed.

3.2. Design Concepts

In this section, circular bored tunnels and rectangular immersed tunnels are developed. Note that the tunnel cross-section shape depends on construction methods and ground condition. For example, circular tunnels are usually constructed by using tunnel boring machine(TBM) or by drill or blast in rock. Rectangular tunnels are often constructed by cut-and-cover method or immersed tube method.

3.2.1. Concept A: Double tube bored tunnel (with emergency lane)

For a double tube bored tunnel, each tube will have a diameter of 13m. The cross section is shown in figure 3.1. In this concept, 2×2 lanes and emergency lane are fitted in case of diversion of failed vehicles and the emergency lane is 3.15m including lateral clearance. An emergency lane can provide a wider space for dealing with emergency cases.

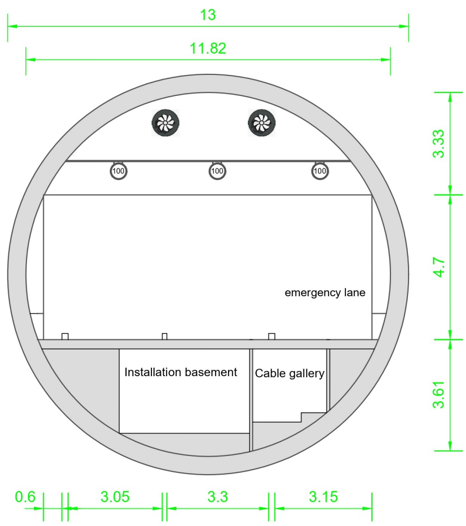


Figure 3.1: *Cross section of the double tube bored tunnel with emergency lane*

3.2.2. Concept B: Double tube bored tunnel (without emergency lane)

For this concept, there is no emergency lane and the diameter of each tube is reduced to 11.3m. The number of lanes is the same as in original report. For an "average" Dutch tunnel with both 2x2 or 2x3 lanes, constructing an emergency lane increases the construction costs with about 15%. If no emergency lanes are built in the tunnel, additional measures are needed for safety. A possibility is the "crossing off" of a lane, in which a car has failed, by lighting up red crosses. As there is no emergency lane, additional lateral clearance should also be added to deal with the emergency case. A 0.6m narrow strip is set between the tunnel wall and lane according to NOA 2007 which is different from 1m redress lane in original report. And the height should be the same as the road surface. The width of each lane is 3.5m and the cross section of this concept is shown in figure 3.2. For this concept, the cost is lower but the capacity to deal with traffic problems is lower as well, as will be evaluated using MCA method in Chapter 4.

3.2.3. Concept C: Single tube bored tunnel

The diameter of this concept is larger than the above two concepts. It takes less space in the alignment but more space in starting and reception shaft. The diameter of this concept is 17.3m which is a little smaller

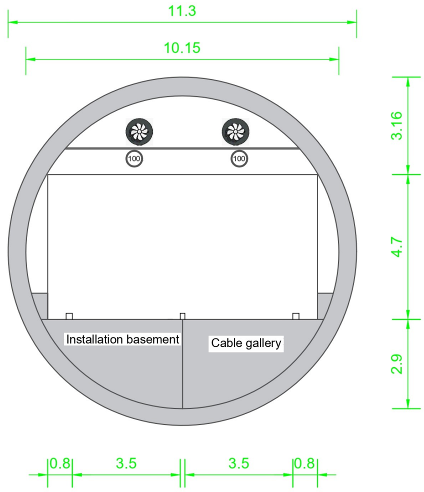


Figure 3.2: *Cross section of the double tube bored tunnel without emergency lane*

than the ever largest highway tunnel in Hongkong(17.6m). It has 2 × 2 lanes with emergency lane and the cross section is shown in figure 3.3.

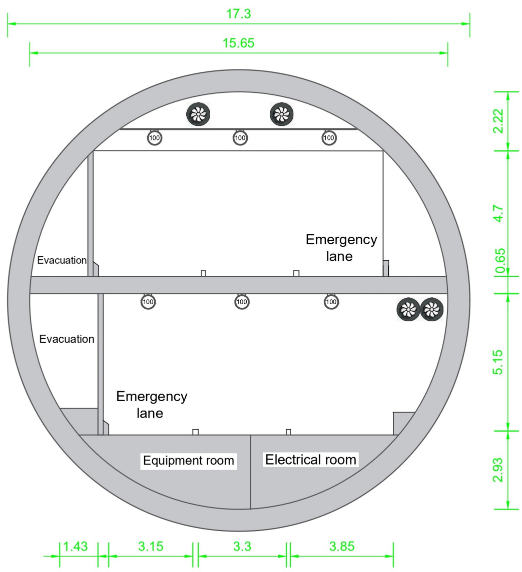


Figure 3.3: *Cross section of single tube bored tunnel*

3.2.4. Concept D: Immersed tunnel

As the immersed tunnel is relatively shallower than the bored tunnel. An immersed tunnel only needs a few meters cover below the waterway which prevent damage to the tunnel by falling anchors or sinking ships.

Concept D is a 2 × 2 lanes immersed tunnel which provides an emergency lane. The design of the cross section is based on buoyancy during floating, immersion and operation phase. And a 0.15m freeboard is also taken into account. (see figure 3.4)

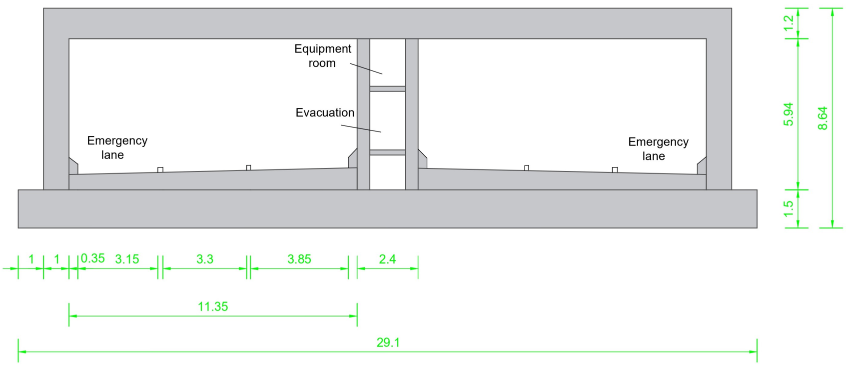


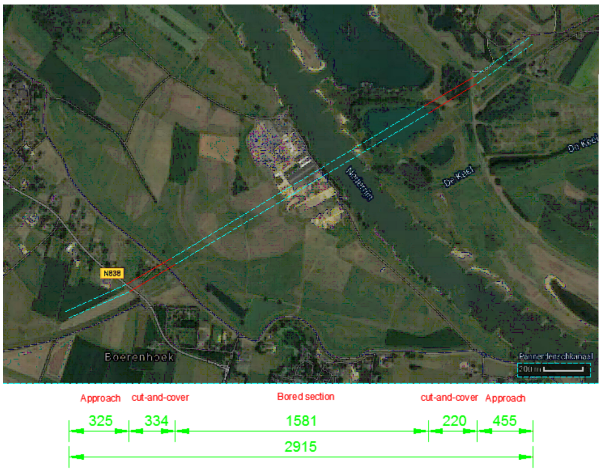
Figure 3.4: *Cross section of single tube bored tunnel*

3.3. Assessment of the design concepts and their alignments

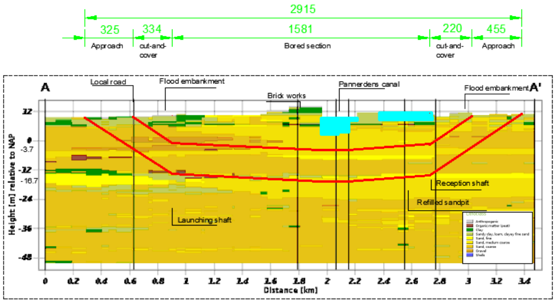
In this section, the concepts illustrated in section 3.2 would be assessed on the aspects of alignment, realistic existings and influence on shipping and railway tunnel. And the maximum slope angle in all tunnel concepts is 4% (Hansen, 2003). The depth of the tunnel is determined based on preventing uplift. The detailed uplift calculation is shown in Appendix A. There are two roads on the surface ground, in the railway tunnel design which was constructed in 1990s, residents nearby were against to block the roads. Also, for a narrow railway tunnel a deep open approach is more expensive than a cut-and-cover tunnel at the same depth. This is because no permanent struts or steel anchors were allowed and retaining walls would be extremely heavily loaded. Finally, the designers used the cut-and- cover tunnel in this length. Here in this project, both cut-and-cover method and open approach method are used, so there are two alignments in each concept to be assessed.

3.3.1. Concept A: Double tube bored tunnel with emergency lane

There are two alignments for concept A which are shown in figure 3.5 and figure 3.6. Both of two alignments are feasible options. The difference between them is if there is cut-and-cover section or not. It is different construction way. The alignment with cut-and-cover section is shorter but the volume of excavation is larger than without cut-and-cover section. Both alignments do not influence railway tunnel nearby, shipping and brick factory on the west bank of the canal.

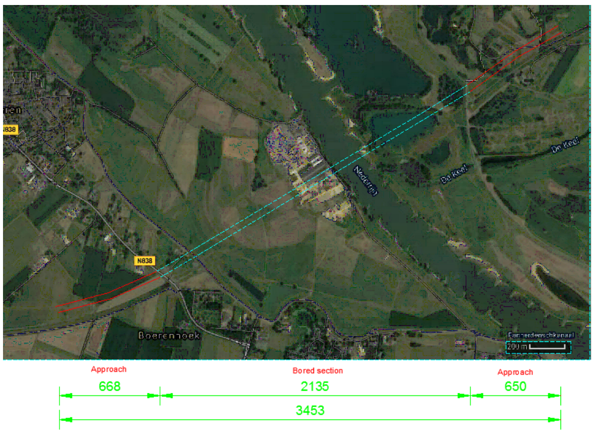


(a) *Alignment of double tube bored tunnel with emergency lane with cut-and-cover section (Google map)*

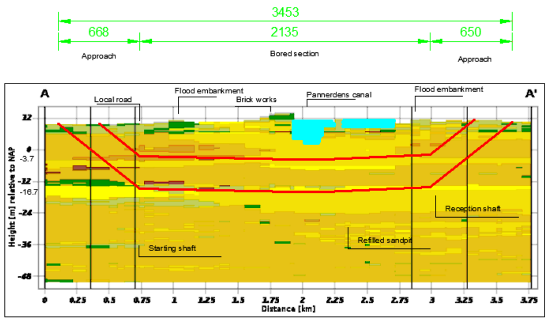


(b) *Profile of double tube bored tunnel with emergency lane with cut-and-cover section*

Figure 3.5: Double tube bored tunnel with emergency lane with cut-and-cover section



(a) *Alignment of double tube bored tunnel with emergency lane without cut-and-cover section (Google map)*

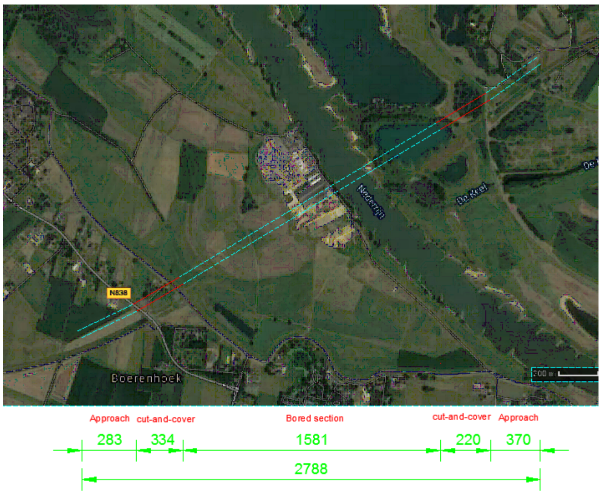


(b) *Alignment of double tube bored tunnel with emergency lane without cut-and-cover section (Google map)*

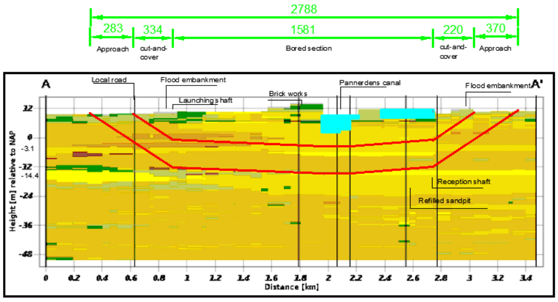
Figure 3.6: Double tube bored tunnel with emergency lane without cut-and-cover section

3.3.2. Concept B: Double tube bored tunnel without emergency lane

A double tube bored tunnel without emergency lane is a feasible solution. There are two construction ways in this concept. One is with cut-and-cover section the other is without it. The diameter is 11.3m which is a bit smaller than that with emergency lane. In this concept, there is no emergency lane. As it is a bored tunnel, both alignments do not influence the railway tunnel nearby, shipping or the brick factory on the west bank of the canal.

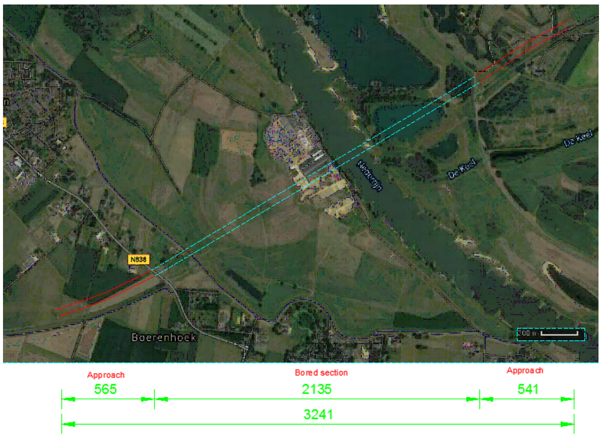


(a) *Alignment of double tube bored tunnel without emergency lane with cut-and-cover section (Google map)*

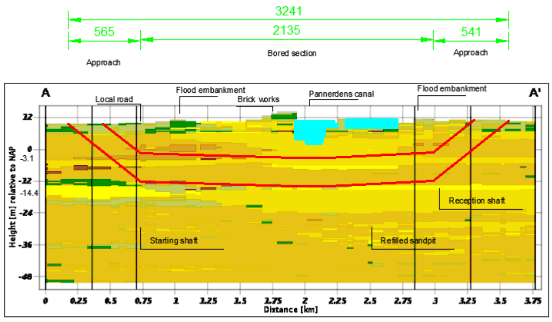


(b) *Profile of double tube bored tunnel without emergency lane with cut-and-cover section*

Figure 3.7: Double tube bored tunnel without emergency lane with cut-and-cover section



(a) *Alignment of double tube bored tunnel without emergency lane without cut-and-cover section (Google map)*

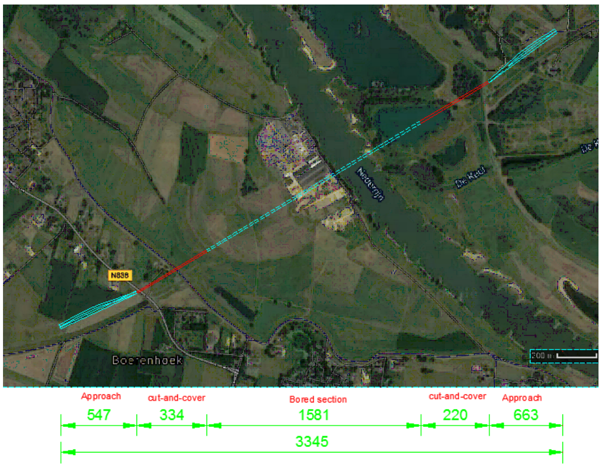


(b) *Profile of double tube bored tunnel without emergency lane without cut-and-cover section (Google map)*

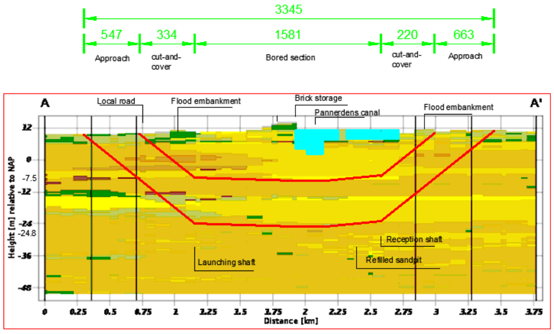
Figure 3.8: Double tube bored tunnel without emergency lane without cut-and-cover section

3.3.3. Concept C: Single tube bored tunnel

The single tube bored tunnel concept is also a feasible solution. The diameter is 17.3m which is smaller than the largest highway bored tunnel(17.6m) in the world now. There are two alignments in this concept as well. Both of the alignments do not affect the brick factory or shipping. As for influence on the railway tunnel, it would affect it a bit but the effect can be reduced by extending the distance between the single tube tunnel and the railway tunnel.

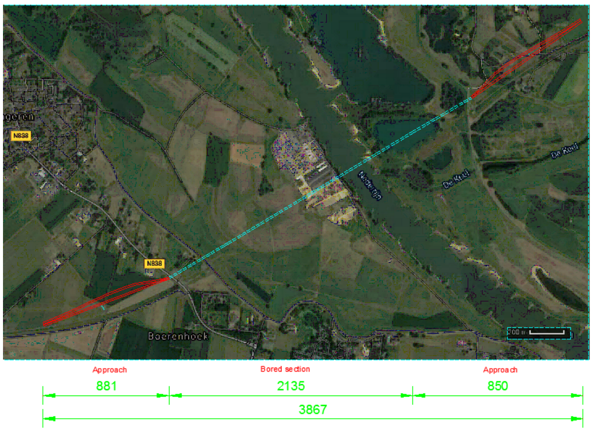


(a) *Alignment of single tube bored tunnel with emergency lane with cut-and-cover section (Google map)*

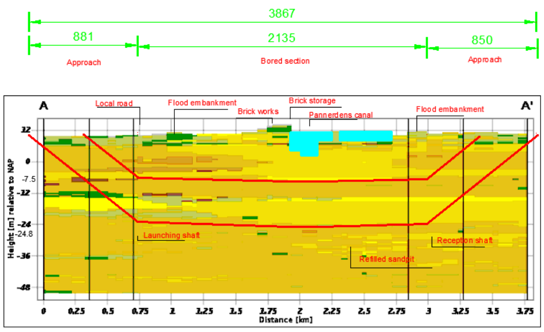


(b) *Profile of single tube bored tunnel with emergency lane with cut-and-cover section*

Figure 3.9: Single tube bored tunnel with emergency lane with cut-and-cover section



(a) *Alignment of single tube bored tunnel with emergency lane without cut-and-cover section (Google map)*



(b) *Alignment of double tube bored tunnel with emergency lane without cut-and-cover section (Google map)*

Figure 3.10: Single tube bored tunnel with emergency lane without cut-and-cover section

3.3.4. Concept D: Immersed tunnel

There are several ways to construct the immersed tunnel. If the tunnel elements are not able to be made at the construction site, they can be made in the casting factory and transported to the construction site. As it can be seen from figure 3.11, all the rivers or canals connecting with Pannerdensch canal have a water depth of around or lower than 8.64m which is not deep enough for tugging the concrete elements. It means there would be a large amount of dredging. This is unrealistic because it is not economical.

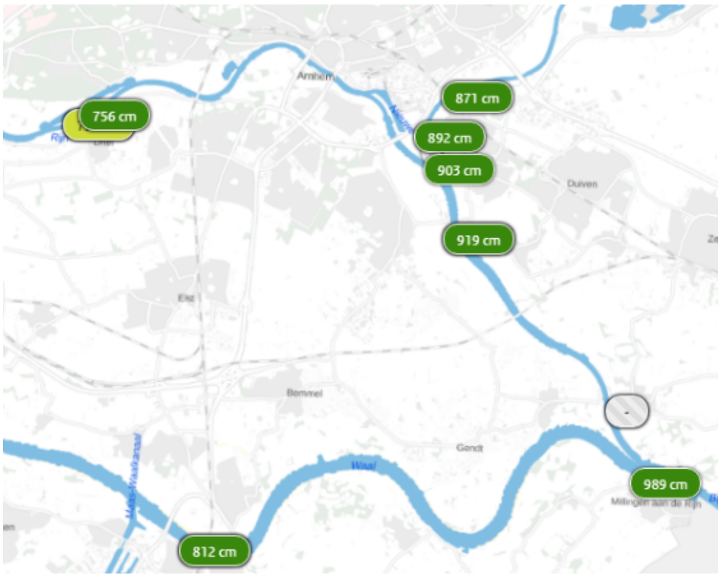
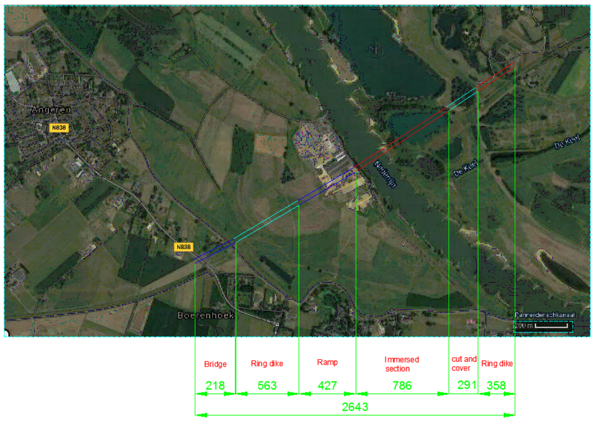


Figure 3.11: *Water level around the Pannerdensch canal (Ministry of Infrastructure and Water Management)*

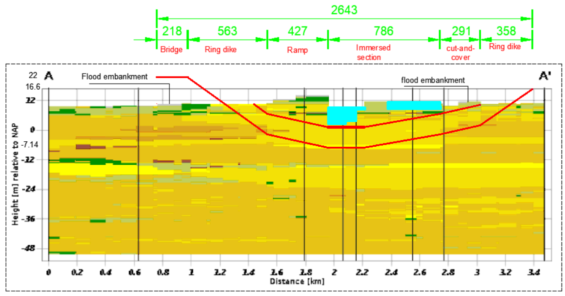
If the tunnel elements are made at site which means a construction dock near alignment needs to be built. There will be a large area of excavation. This would cause damage to the nature reserves temporarily as it takes a large space to build the construction dock. As the entry and exit of immersed tunnel would be located in the flood plain area, there will be water flowing into the tunnel during tunnel operation. Another problem is the brick factory is near the river and it is a barrier for the immersed tunnel. So extra measurements need to be taken to combat any significant disturbance. There are 4 alternatives to construct immersed tunnel:

The first alternative

The first and also the shortest immersed section is shown in figure 3.12. This alternative is not a feasible solution.The high fluctuating water level is from NAP+6.25 to NAP +16.32m. In this alignment, a ring dike is used permanently in order to prevent flooding into the tunnel. This construction way can cause a serious river discharge problem. The length of ring dike in the floodplain is 563m because it needs to reach the height of the primary dike. So it would narrow the flood plain and increase the water level and flow rate. As a result, there may be flood on the residential area due to increased water level.



(a) *Alignment of first alternative of immersed tunnel (Google map)*

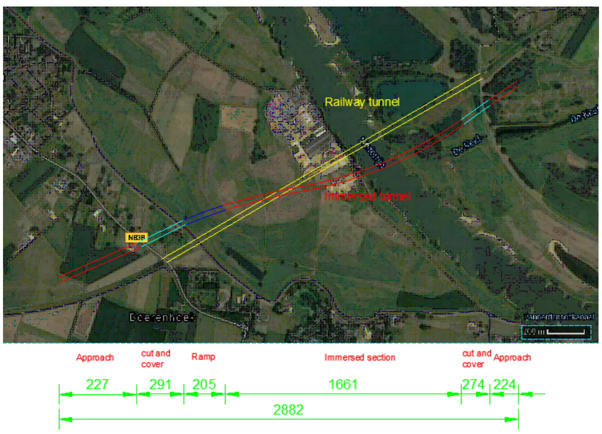


(b) *Profile of first alternative of immersed tunnel*

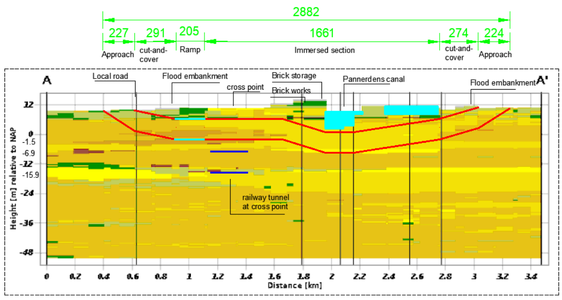
Figure 3.12: The first alternative of immersed tunnel

The second alternative

The second alternative is shown in figure 3.13. It is not a feasible solution. In this alignment, in order to prevent flood into tunnel and avoid the brick factory, the alignment is a curve horizontally and the radius is 2000m. Also the dock is constructed near the primary dike temporarily. As there is a railway tunnel nearby, the immersed tunnel needs to dredge above the railway tunnel. As it can be seen in figure 3.13, there is only 5.4m soil cover on the railway tunnel while the diameter of railway tunnel is 9m, so the soil cover is around 0.5 time of diameter of railway tunnel. This is very dangerous as it will influence the stability of the railway tunnel. Besides, on the east bank of canal, it has to go back to the north of the railway tunnel. It is not realistic to realise this alternative because it is expensive.



(a) *Alignment of second alternative of immersed tunnel (Google map)*



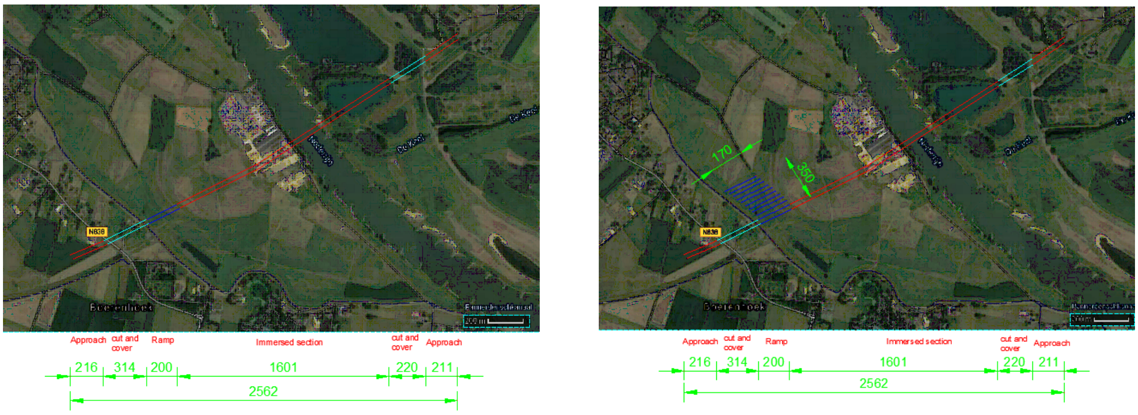
(b) *Profile of immersed tunnel and railway tunnel at cross point*

Figure 3.13: The second alternative of immersed tunnel

The third and fourth alternatives

The third and fourth alternatives are in figure 3.14. The only difference is the area of construction dock. They are feasible solutions. Figure 3.14a shows the alignment that only one element can be made at one time even if it is very time consuming. The canal has high currents and there will be sediments on the previous element before transporting next element, which means dredging the sedimented materials needs to be done before transporting every element. This increases an overall budget by adding extra cost. But this way has less influence on the environment compared with building the construction dock in figure 3.14b.

As the ramp and construction dock are located in floodplain, so it will have impact on the river discharge during constructing elements. But the length of the ramp and dock is around 200m,the impact is not that much. The dock would be refilled and covered by greenfield after construction, so the impact of both of the two alternatives on the environment are temporary. Both alternatives affect the shipping during immersing elements. Both of them will go through the brick factory in the same way as the bridge solution.



(a) *The third alternative of immersed tunnel (Google map)*

(b) *The fourth alternative of immersed tunnel (Google map)*

Figure 3.14: The fourth alternative of immersed tunnel

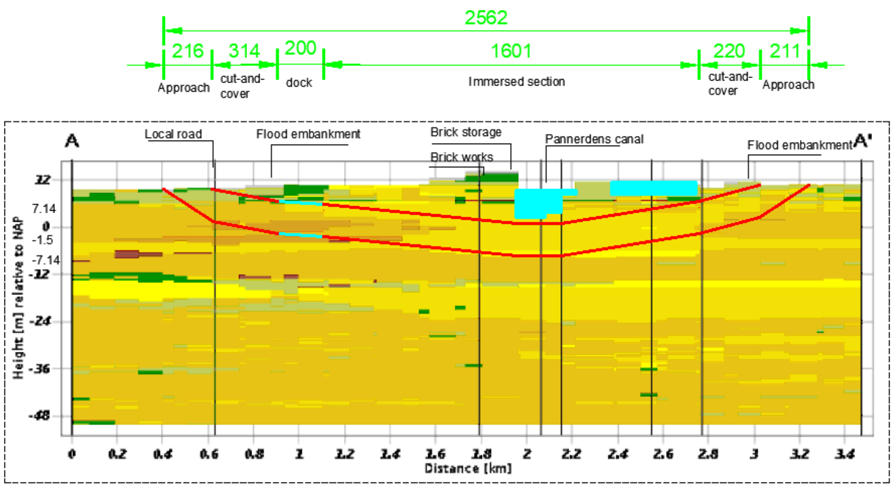


Figure 3.15: *Profile of the third and fourth alternative of immersed tunnel*

3.4. Conclusion

From the results of the assessment, it turns out that not all concepts are feasible, there are only a few concepts which can satisfy the requirements of alignment,river discharge capacity and influence on the railway tunnel and shipping. These assessed concepts are listed in the table 3.1. Construction method with cut-and-cover means there is cut-and-cover section to connect bored section and open ramp. Construction method with open approach means there is only bored section and open ramp along the alignment.

Table 3.1: Verified concepts after assessment

Concept	Concept A		Concept B		Concept C		Concept D	
Construction method	cut-and-cover	open approach	cut-and-cover	open approach	cut-and-cover	open approach	cut-and-cover	open approach
Length (m)	2915	3453	2788	3241	3345	3867	2562	2562

4

Evaluation of verified design concepts

4.1. Introduction

After verifying the potential concepts, the remaining concepts need to be evaluated. In this chapter, they are going to be evaluated based on a Multi Criteria Analysis (MCA). Finally, evaluated results of each concept are described and an optimal concept is determined by a sensitivity analysis.

4.2. Multi Criteria Analysis

The Multi Criteria Analysis is a methodology by which the relative merit of alternatives can be compared by using a range of quantitative and qualitative criteria. Steps are very clear within MCA to evaluate the alternatives (Toorn, 2015). There would be criteria to be set for comparing the alternatives. Each criterion has a weighting factor as in this report the importance of different criteria is different. A matrix is used to compare the criterion one by one, and each criterion is given a score in the comparison. According to this scoring rules, the most important criteria is scored as 1 and the least important one is 0. Finally, the weighting factor of each criterion is the sum of scores divided by the total score 6.

Then a score is given to each alternative concept against each criterion. For every criterion, the score of the least important criterion is set as the lowest value, and the score of the most important criterion with the highest value. Finally, the score of all criteria are summed up and there would be a total score for each concept (Van der Toorn, 2015).

In the last, the total score is multiplied by the weighting factor and then the final score is achieved.

4.2.1. Criterion Description

In this section, criteria are used to evaluate the design variants from the point of environment, traffic capacity, river discharge capacity, time and cost. The basic information of verified concepts is shown in table 4.1. Construction method with cut-and-cover section means there is cut-and-cover section to connect bored section with open ramp. Construction method with open approach means there are only bored section and open ramp along the alignment. The figures for these two methods have been shown in chapter 3.3

Table 4.1: Basic information of verified concepts

	Construction method	**Length (m)**	**Volume of excavation (m3)**	**Time (month)**	**Cost(€)**
Concept A (Double-tube bored tunnel with emergency lane)	cut-and-cover	2915	376541.4	57	370,076,249
	open approach	3453	364847.7	49	442,739,323
Concept B (Double-tube bored tunnel without emergency lane)	cut-and-cover	2788	243740	56	367,565,038
	open approach	3241	218820	47	405,863,163
Concept C (Single-tube bored tunnel with emergency lane)	cut-and-cover	3345	427403.2	50	402,320,308
	open approach	3867	550724	48	442,758,834
Concept D (Immersed tunnel)	with dock	2562	1538176	39	332,097,443
	without dock	2562	942775.98	46	330,652,688

Criterion 1: Impact of construction on the environment

The impact on the environment is a big concern that should be taken into consideration. Chapter 1 mentions the reason why the tunnel option is considered —— the great impact of building the bridge on the environment. This is also the main reason that the national and local government chose the tunnel instead of the bridge for the railway across the Pannerdens canal in 1990s. The environmental impacts include noise, nitrogen oxides, visual hindrance, cultural history and archaeology. Here only nitrogen oxides and noise are analyzed and the rest aspects are on the website Platform Participation ViA15.

The emission of nitrogen oxides is a serious issue as it would have a great impact on people's health and the environment, meanwhile, vulnerable plant species are disappearing. The National Institute for Public Health has indicated that too much nitrogen in the soil is an important cause for the decline of rare species in the ecosystem. The amount of nitrogen in the soil increases due to nitrogen deposition from the air. Ammonia is the dominant chemical that keeps two thirds of the nitrogen within the soil, and it comes mainly from agriculture. The remaining deposition comes from the traffic and industries. Vulnerable plant species disappear when the nitrogen deposition exceeds the critical deposition level. The higher the exceeding and the longer it lasts, the greater the effects. Food-poor ecosystems are particularly sensitive to environmental pressure from nitrogen emissions.

The current environmental pressure due to nitrogen deposition is still too high in many onshore ecosystems. In particular in the forest, open dune and health ecosystem types, the conditions in these places due to nitrogen deposition over almost the entire area are moderate or poor. Trend data (see figure 4.1) shows that the nitrogen availability of the soil in open dunes and semi-natural grasslands has increased.

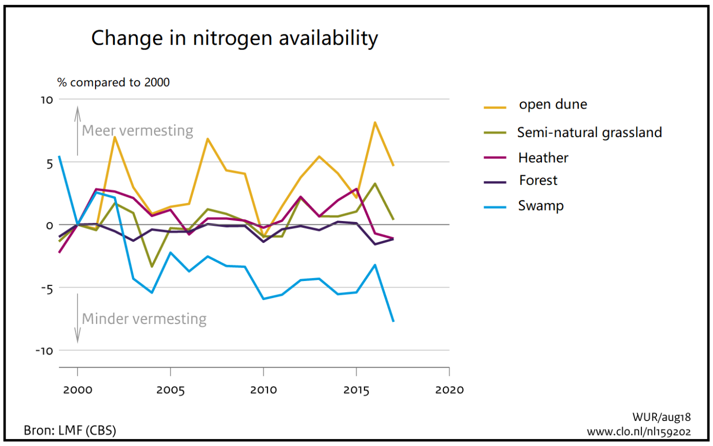


Figure 4.1: *Change in nitrogen availability (Nitrogen RIVM)*

The area of nature conservation where the critical deposition values were exceeded decreased from approximately 80% to approximately 70% between 1995 and 2016 (see figure 4.2). The difference between deposition and critical deposition level is a measure of the risk of deterioration in nature quality. It can be seen there is still a large percent of area in nature reserves where critical deposition value is exceeded. So it is necessary to take some measures to reduce the nitrogen emission to protect the nature reserves.

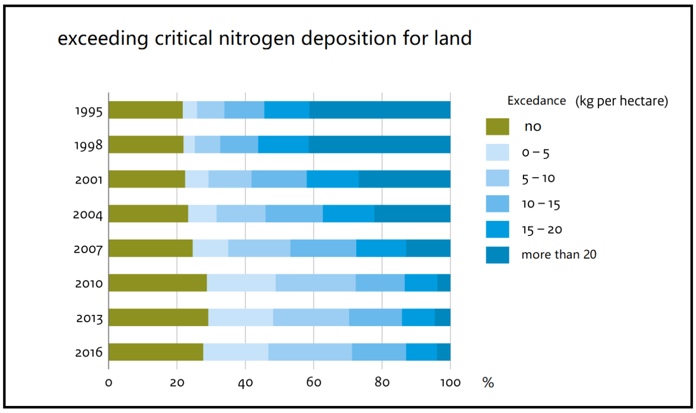


Figure 4.2: *Exceeding critical nitrogen deposition for land nature (Nitrogen RIVM)*

The noise coming from the construction process and transportation tools can cause disturbance to the animals or creatures in the lakes or canal. Noise hindrance was one of the main reasons why residents opposed the proposed bridge in the decision-making process of the railway in 1990s. The most important effects are the land use within the North and the South influencing main ecological structure and meadow bird and geese conservation area. In addition, as for the bridge option, barrier effect occurs due to intersection of geese and meadow bird area.

Criterion 2: River discharge capacity

The design concept should influence the river discharge capacity as little as possible. The canal is located in floodplain, there is strict requirement on the discharge of flood area and it is not allowed to narrow the flood area. if the embankments are narrowed, during winter period, water level will increase thus it will affect the road functions and cause a threat to public safety. Rising water level could affect the floras and faunas living nearby, which means it may have an adverse impact on their habitat.

Criterion 3: Available capacity for vehicles

Traffic capacity is a basic but very important criterion in the tunnel design. The reason why A15 is extended is the excess traffic load on A50, A12 and A325. So the design concept should be able to relief the traffic load as much as possible. There should be enough space for the running vehicles. Meanwhile, in emergency situations, it can provide enough capacity to deal with them to prevent the traffic jam.

Criterion 4: Cost estimation

Cost estimation could be a good criterion to define an optimal design concept as it plays a very important role for contractor to make decision. The cost estimation in this report includes the main components of project construction. For a bored tunnel, it covers the machinery (TBM), materials, labor cost, monitoring, cut-and-cover tunnel and open ramp. For an immersed tunnel, it covers the open ramp, transition structure, cut-and-cover, casting dock, dredging, transportation, immersion and cost related to mechanics and equipment. The cost estimation is based on the design experience and literature study. The detailed cost estimation process is shown in Appendix B. From table 4.1, open approach is more expensive than cut-and-cover, it is the same as in the railway tunnel nearby because the shaft is deeper and retaining walls would be heavily loaded and as a result it would cost more. An immersed tunnel is tens of millions euros cheaper than a bored tunnel and this conclusion is consistent with that in the original report of ViA15 project.

Criterion 5: Time

Time is also a criterion to evaluate the verified design concepts as more time to some extent means more cost. For an immersed tunnel, if it only constructs one element at one time then it would take more time and also cost more to dredge the currents which cover on the previous element. For a bored tunnel, more time means the machine would consume more energy and more measures need to be taken to keep it running. The detailed time distribution of each concept is shown in Appendix C.

4.2.2. Weight factor

In this section, each criterion mentioned above is going to be compared with other criteria. The aim is to get the weight factor of each criterion. The result is shown in table 4.2.

Table 4.2: Matrix of calculation of weight factor

	Impact of construction on the environment	River discharge capacity	Available capacity for vehicles	Time	Score	Weight factor
Impact of construction on the environment	-	1	1	1	3	0.5
River discharge capacity	0	-	0	1	1	0.17
Available capacity for vehicles	0	1	-	1	2	0.33
Time	0	0	0	-	0	0.05
Total score					6	1.05

In determination of weight factors, the impact on the environment is the most important criteria because this project is located in nature reserves. One of the reasons why the government decides to consider the tunnel option is exactly the environmental concern. The second most important criteria is the available capacity for vehicles. As the tunnel must provide enough space to satisfy the requirements of the lanes and tackle the traffic jam effectively. Meanwhile, the tunnel should have enough space for dealing with the emergency cases. The third most important factor is the impact on river discharge capacity. If the flood embankments are narrowed, river discharge capacity is reduced. As a result, the water level will rise, which will cause a threat to the residential area nearby. There might be influence on the habitat of protected animals living nearby as well.

4.2.3. MCA scores

In this section, each design variant is given a score and the total score of all design concepts in each criteria is 11 because the criterion ‘time’ is given a weight factor of 0.05. The score obtained by each design concept is determined by proportion, namely how much each design variant contributes to the criterion. For example, the impact of construction on the environment is determined by the volume of excavation divided by total excavation of all design concepts. It is the same for other criterion. For the impact on the environment and river discharge capacity, the more score the design concept gets, the less impact it will cause. The more score the concept achieves, the less time it takes to construct the tunnel. The score is shown in table 4.3

Table 4.3: Results of Multi Criteria Analysis

Criterion	Weight factor	Concept A with cut-and-cover (A1)	Concept A with open approach (A2)	Concept B with cut-and-cover(B1)	Concept B with open approach (B2)	Concept C with cut-and-cover(C1)	Concept C with open approach (C2)	Concept D with casting dock (D1)	Concept D without casting dock (D2)
Impact of construction on the environment	0.5	1.01	1.3	2.22	3.63	0.89	0.86	0.52	0.57
River discharge capacity	0.17	1.83	1.83	1.83	1.83	1.83	1.83	0	0
Available capacity for vehicles	0.33	1.46	1.46	1.2	1.2	1.46	1.46	1.38	1.38
Time	0.05	1.02	1.39	1.13	1.41	1.3	1.37	1.7	1.65
Score excl. factor		5.32	5.98	6.38	8.07	5.48	5.52	3.60	3.60
Score incl. factor		1.35	1.51	1.87	2.59	1.30	1.29	0.80	0.82

4.2.4. Explanation of MCA scores

As calculated in the section 4.2.2, the weight factor of ‘ time ’ is 0 in theory. Even if it is the least important factor among these criteria, a weight factor of 0.05 should be given to it because it plays a role in the determination of the design concepts.

Criterion 1: Impact of construction on the environment

The concept B with open approach scores the highest because the volume of excavation of approach is the smallest (see table 4.1). The concept D with casting dock scores the lowest because it has the largest volume of excavation which means the largest impact on the environment. Concept D would excavate a lot of soil to construct the dock which is located in the nature reserve, which means a serious impact on the environment.

Criterion 2: River discharge capacity

The river discharge capacity should be influenced as little as possible. From Eq (4.1) , it can be seen when river discharge Q keeps the same, if there is block on the flood plain which reduces the area of cross section of flood plain, then water velocity would increase and water level would increase as well. So the impact on the river discharge capacity depends on the area of block. Concept A, B, C have the least effect on the river discharge because they are constructed below the river bed with trenchless methods. The block area is zero for a bored tunnel. While for an immersed tunnel, casting dock and the ramp need to build a dike to prevent flooding into the dock or ramp. The block area of is 1140 m^2 which is the largest among all concepts. The percentage of block area of an immersed tunnel is 50% of total block area of all tunnel concepts. As a result, immersed tunnel concept scores the lowest because it influences the river discharge the most.

$$Q = A_1 * u_1 + A_2 * u_2 \tag{4.1}$$

Where:

Q discharge of river (m^3/s)

A_1 area of cross section of river (m^2)

A_2 area of cross section of flood plain (m^2)

u_1 velocity of water in river (m/s)

u_2 velocity of water on flood plain (m/s)

Criterion 3: Available capacity for vehicles

For the available capacity for vehicles, the wider the driving space, the more effective it can tackle the traffic jam. The score of this criterion is obtained as follows score = width/total width * 11. Concept A and C get the highest score because they have widest space which is 10.65m. While concept B scores the lowest as it has only two lanes without emergency lane so it is less effective to solve the problem when the emergency cases happen.

Criterion 4: Time

Time is also an important role in the whole project as it relates to project cost. For a bored tunnel, time includes approach access, cut-and-cover section, shaft, TBM installation, boring process. To estimate the time for each concept, the following assumptions are made according to the data collected in Netherlands: (1). 10m/d is used as the speed of TBM boring process. (2). Two tunnel boring machines are used for excavation for double tube tunnels. (3). 10 months are set for constructing the shaft and 3 month for TBM installation and demobilization. (4). For single approach access, 25 weeks is needed for the first compartment of 100m length and then 20m/week.

For an immersed tunnel, time includes trench dredging, constructing casting dock and tunnel elements, the cut-and-cover section and open approach. Concept D with casting dock scores the highest as it takes the least time (39 months) to complete the project. While concept A with cut-and-cover section scores the lowest because it takes the longest time (57 months) to construct the tunnels.

Criterion 5: Cost estimation

In the cost estimation, the highest cost is treated as the basic value and other costs are divided by the basic value and therefore they have their cost parameters. The smaller the cost parameter, the less it costs. The formulation is written as: cost parameter = cost / maximum cost, final score = score (incl.factor) / cost parameter. The cost of each concept is in table 4.4, concept B without cut-and-cover scores the highest and an immersed tunnel with casting dock scores the lowest.

Table 4.4: Cost estimation

Concepts	Concept A with cut-and-cover	Concept A without cut-and-cover	Concept B with cut-and-cover	Concept B without cut-and-cover	Concept C with cut-and-cover	Concept C without cut-and-cover	Concept D with casting dock	Concept D without casting dock
Cost(€)	370,076,249	442,739,323	367,565,038	405,863,163	402,320,308	442,758,834	332,097,443	330,652,688
Cost parameter	0.84	1.00	0.83	0.92	0.91	1.00	0.75	0.75
Final score	1.61	1.51	2.26	2.83	1.43	1.29	1.07	1.10

4.3. Sensitivity analysis

In the process of determining the best design concept, how to weigh all criteria is very important as different weight factor can change the result. In the Multi Criteria Analysis, weigh factor of different criteria affect the score results and then final results vary largely when the criteria are weighed differently. So it is necessary to do a sensitivity analysis of all criteria. In this section, all weight factors are re-arranged and totally there are 24 data sets for the sensitivity analysis. The score of each criterion is given in the order of magnitude (from the hignest to the lowest). From table 4.5 and table 4.6, there are three concepts which score the highest in the Multi Criteria Analysis and they are A1, B1 and B2. The profile of these three concepts are in figure 3.5, 3.7 and figure 3.8.

Table 4.5: Sensitivity analysis 1

Criterion	Weight factor												Score
Impact of construction on the environment	0.5	0.5	0.5	0.5	0.5	0.5	0.33	0.33	0.33	0.33	0.33	0.33	4.98
River discharge capacity	0.33	0.33	0.17	0.17	0.05	0.05	0.5	0.5	0.17	0.17	0.05	0.05	2.54
Available capacity for vehicles	0.17	0.05	0.33	0.05	0.33	0.17	0.17	0.05	0.5	0.05	0.5	0.17	2.54
Time	0.05	0.17	0.05	0.33	0.17	0.33	0.05	0.17	0.05	0.5	0.17	0.5	2.54
Final score	3.00	3.02	2.88	2.95	2.83	2.87	2.66	2.68	2.43	2.53	2.37	2.45	32.67
Design concept of highest score	B2	B2	B2	B2	B2	B2	B2	B2	B2	B2	B2	B2	-

Table 4.6: Sensitivity analysis 2

Criterion	Weight factor												Score
Impact of construction on the environment	0.17	0.17	0.17	0.17	0.17	0.17	0.05	0.05	0.05	0.05	0.05	0.05	1.32
River discharge capacity	0.5	0.5	0.33	0.33	0.05	0.05	0.5	0.5	0.33	0.33	0.17	0.17	3.76
Available capacity for vehicles	0.33	0.05	0.5	0.05	0.5	0.33	0.33	0.17	0.5	0.17	0.5	0.33	3.76
Time	0.05	0.33	0.05	0.5	0.33	0.5	0.17	0.33	0.17	0.5	0.33	0.5	3.76
Final score	2.22	2.29	2.1	2.21	1.97	2.01	1.96	1.96	1.87	1.89	1.73	1.77	23.98
Design concept of highest score	B2	B2	B2	B2	B2	B2	B1	B2	A1	B2	B2	B2	-

In order to better compare the abovementioned three concepts, they should be evaluated on a deeper plan level. These three concepts can be compared after optimising the soil cover at shafts and canal because it can largely reduce the cost,time and the impact on the environment. The whole optimisation process is in Appendix D. After optimisation, the new data of each concept is shown in table 4.7.

Table 4.7: The basic information of the concepts after optimisation

Concept type	Concept	Construction method	Length (m)	Volume of excavation(m^3)	Time (month)	Cost (€)
A1	double-tube bored tunnel with emergency lane	cut-and-cover	2785	352378	64	363,738,749
B1	double-tube bored tunnel without emergency lane	cut-and-cover	2720	261663	88	356,330,355
B2	double-tube bored tunnel without emergency lane	open approach	3006	393118	53	386,052,875

Multi criteria analysis and sensitivity analysis are conducted again based on the optimised concepts and the results are in table 4.8 and table 4.9.

Table 4.8: Sensitivity analysis after optimisation 1

Criterion	Weight factor												Score
Impact of construction on the environment	0.5	0.5	0.5	0.5	0.5	0.5	0.33	0.33	0.33	0.33	0.33	0.33	4.98
River discharge capacity	0.33	0.33	0.17	0.17	0.05	0.05	0.5	0.5	0.17	0.17	0.05	0.05	2.54
Available capacity for vehicles	0.17	0.05	0.33	0.05	0.33	0.17	0.17	0.05	0.5	0.05	0.5	0.17	2.54
Time	0.05	0.17	0.05	0.33	0.17	0.33	0.05	0.17	0.05	0.5	0.17	0.5	2.54
Final score of selected concept	4.41	4.5	4.37	4.57	4.42	4.53	4.22	4.3	4.13	4.44	4.18	4.41	52.48
Design concept of highest score	B2	B2	B2	B2	B2	B2	B2	B2	B2	B2	B2	B2	-

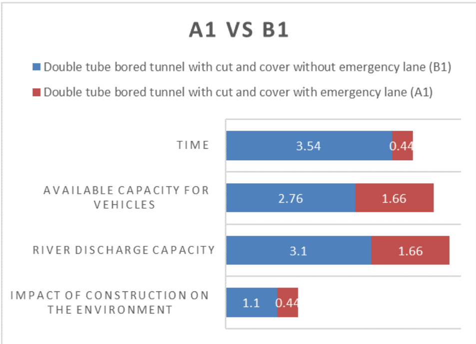
Table 4.9: Sensitivity analysis after optimisation 2

Criterion	Weight factor												Score
Impact of construction on the environment	0.17	0.17	0.17	0.17	0.17	0.17	0.05	0.05	0.05	0.05	0.05	0.05	1.32
River discharge capacity	0.5	0.5	0.33	0.33	0.05	0.05	0.5	0.5	0.33	0.33	0.17	0.17	3.76
Available capacity for vehicles	0.33	0.05	0.5	0.05	0.5	0.33	0.33	0.17	0.5	0.17	0.5	0.33	3.76
Time	0.05	0.33	0.05	0.5	0.33	0.5	0.17	0.33	0.17	0.5	0.33	0.5	3.76
Final score of selected concept	4.02	4.2	4.12	4.26	4.07	4.18	4.07	4.17	4.16	4.23	4.08	4.18	49.74
Design concept of highest score	A1/B1	B1	A1	B1	B1	B1/B2	A1/B1	B1	A1	B1	B1	B1	-

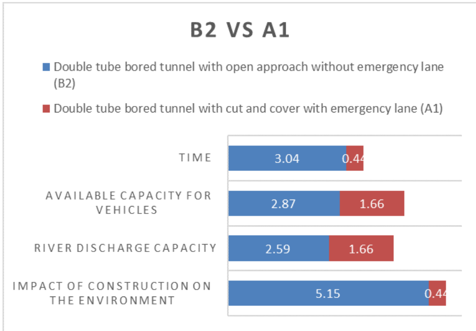
From sensitivity analysis, when weight factors are applied differently on each criterion, the results show significant change. For example, in the figure 4.3a, when time is considered more important and set a higher weight factor, then concept B with cut-and-cover is optimal and selected. When the available capacity for vehicle is attached with more importance, then concept A with cut-and-cover is selected. Since the main problem is to tackle the traffic problem, so the criterion "capacity for vehicle" is reasonably more important than criterion "time", therefore concept A with cut-and-cover is better than concept B with cut-and-cover.

In the figure 4.3b, it is the comparison between concept A with cut-and-cover and concept B with open approach, it can be seen when criterion "impact of construction on environment" is considered more, concept

B with open approach will be better. When available capacity of vehicles is considered more, then concept A will be better. As the impact of construction on the environment depends on the volume of excavation, whereas the excavation mainly occurs outside the nature reserve and will not cause a serious impact on the environment, therefore, the criterion "impact of construction on environment" can be considered less important. In this case, concept A can be potentially changed to 3*2 lanes in the future and it can tackle the traffic jam more efficiently. Hence concept A with cut-and-cover is the best option.



(a) Sensitivity analysis between A1 and B1



(b) Sensitivity analysis between A1 and B2

Figure 4.3: Sensitivity analysis

4.4. Conclusion

From the Multi Criteria Analysis, all proposed concepts are evaluated from the aspects of impact on the environment, river discharge capacity, available capacity for vehicles, time and cost estimation. Each concept has its score after evaluation. In the process of sensitivity analysis, all possible distributions of weight factors are analyzed and there are three best concepts. After optimising the three concepts, MCA and sensitivity analysis is conducted again. Finally, Concept A with cut-and-cover section is selected as the best concept.

5

Preliminary design of final concept

5.1. Introduction

In previous chapters, the best concept has been selected. The cross section and longitudinal profile are in figure 5.1 and figure 5.2. This chapter focuses on tunnel design which includes lining thickness, joints, internal force of tunnel ring, thrust force. Then it is calculation and control of settlement at critical positions. Finally it is pre-design of the cross passages.

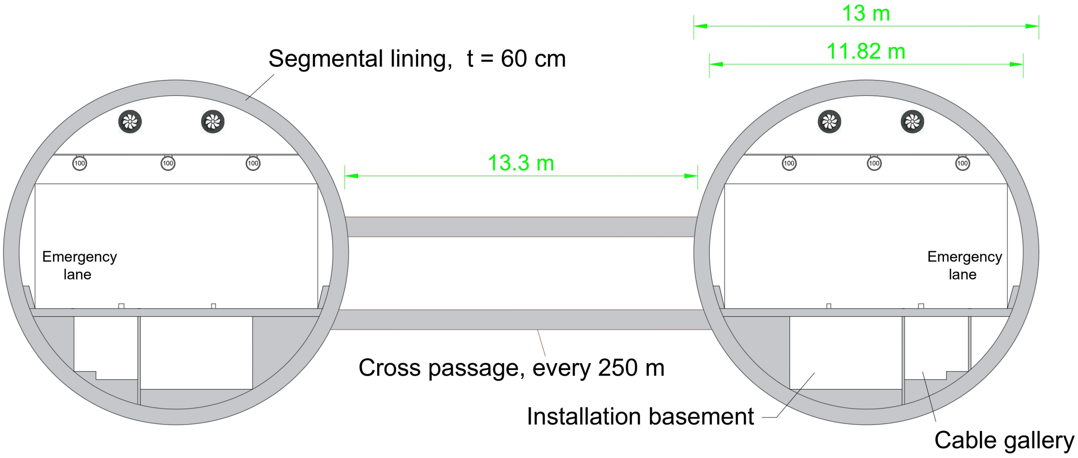


Figure 5.1: *Cross section of double-tube bored tunnels with emergency lane*

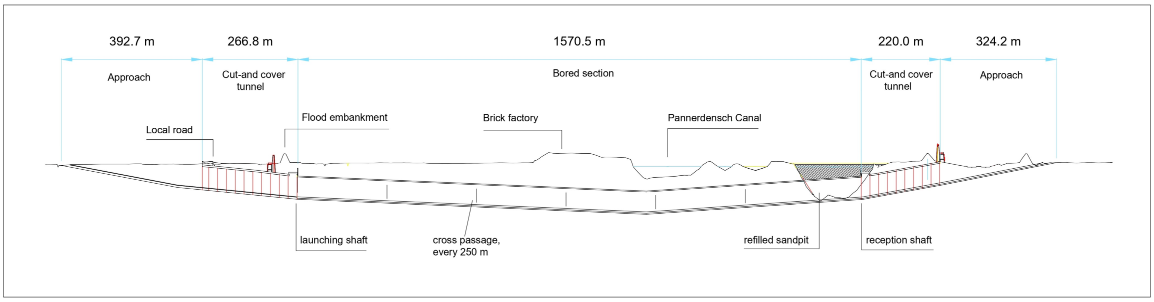


Figure 5.2: *Longitudinal profile of double-tube bored tunnels with emergency lane*

5.2. Tunnel lining design

The tunnel lining has to guarantee the structural safety and serviceability during the entire use of the tunnel. To most bored tunnels in the Nederlands, a single layer segment lining is used in the tunnel construction. Here we only consider the design of a segment single lining.

5.2.1. Segment lining

Segments are precast elements, installed next to each other in a ring, and to form the tunnel lining longitudinally. Segments can be made with different material such as steel, (steel) reinforced concrete and fiber-reinforcement concrete, et al. Reinforced concrete is the most widely used in the bored tunnelling lining. This project will choose precast reinforcement concrete segment.

According to ITA WG2 (Bakhsh and Nasri, 2019), a review of more than 100 projects published in ACI544.7R (2016), AFTES (2005), Groeneweg (2007) and Blom (2002) indicates that the internal tunnel diameter (ID) to the tunnel lining thickness falls in the range of 18-25 for the tunnel with an ID of more than 5.5m. JSCE (2007) recommends that the ring thickness should be less than 4% of the outer diameter of the segmental ring, which translates into an ID to thickness ratio of 23. In this design report, the lining thickness is around 1/20 of the tunnel outer diameter and the ID to the lining thickness ratio is 19.7. Since the tunnel diameter is 13m, so lining thickness is around 0.6m.

For tunnel diameters between 11m to 14m, a 9 segment ring is not prefered. Special solutions are required, such as dividing the ring into 8 segments (each covering 45 degree) and dividing one of the ordinary segments into key and counter-key segment (which cover 15 degree and 30 degree). By utilizing such a configuration, excessively large key segments can be avoided, while at the same time the configuration is compatible with the TBM thrust jacking pattern for an 8-segment ring. For each ring in this project, it consists of total 9 segments and a wedge-shaped keystone included, which is shown in figure 5.3. Seven of the 9 segments all have the same dimensions. These segments are called A stones (45 degree). The segment B (30 degree) completes

the ring together with the keystone (15 degree)- which is placed between the B and A stone.
In the ITA WG2 (Bakhsh and Nasri, 2019), typically, a ring length of 1.5m would be used for tunnel diameters of 6m to 7m, increasing to a ring length of 2m for tunnels larger than 9m in diameter. In this project the ring length is 2 meters wide. So each segment weighs around 14.4 tons. As the length of the bored section is 1581m, the amount of tunnel rings is 791 so the total amount of segments is 7115. These segments can be transported by the railway nearby.

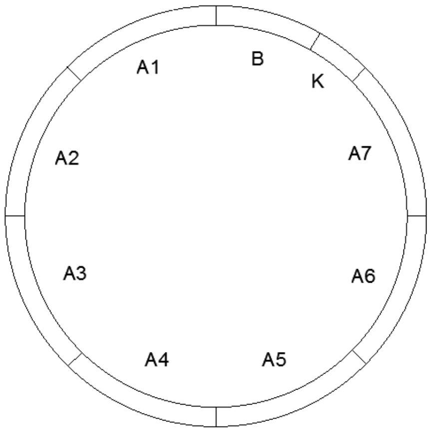


Figure 5.3: *Diagrammatic representation of a tunnel ring*

The front face of a complete ring is not parallel to its back face, but cone-shaped to accommodate curve radius of up to around 150 meters which is shown in figure 5.4

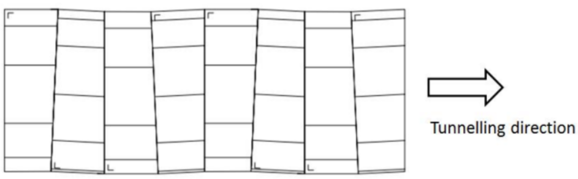


Figure 5.4: *Diagrammatic representation of conical rings*

Seal

A single rubber sealing ring is installed on the circumference of the tunnel segments, and must be able to ensure water tightness at pore pressure of 3.64 bar throughout the boring process. Since the material of the seal will suspend over time-throughout the replacement, assuming a 47% reduction in total relaxation, it is designed to withstand a water pressure of 6.9 bar.(figure 5.5)

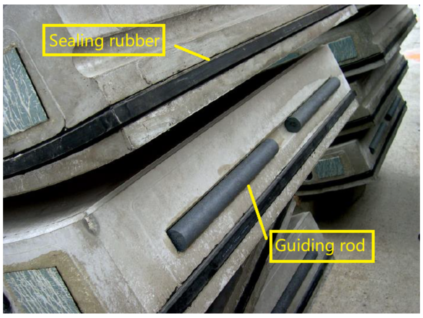


Figure 5.5: *Plastic guiding rod*

Ring joints

The flat joint is the simplest joint. The coupling is completed by friction, which does not help to assemble the ring. As an auxiliary tool for centering during the ring assembly process, plastic dowel can be used (See figure 5.6). This plastic dowel and groove connection improves the rigidity of the structure and reduces the differential displacement between the tunnel rings by absorbing radial forces.

Figure 5.6: *Plastic dowel*

Longitudinal joints

The longitudinal joints are usually also flat joints, with guiding rods for easier assembling (See figure 5.5). The longitudinal joints form the connections between the segments within a ring. These joints are designed as a concrete hinge with a certain rotational capacity.

Load concentration pads or fibreboard plates are used for force transfer in the ring joints. (See figure 5.7)

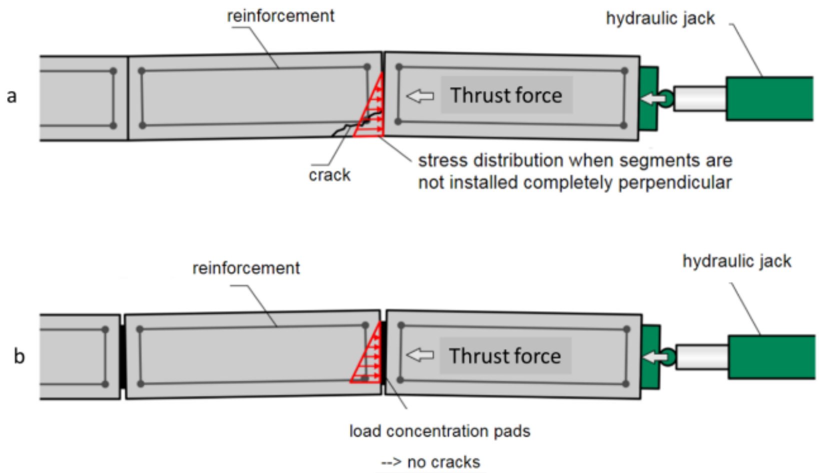


Figure 5.7: *Behaviour of segment (picture from Babendererde) a.without load concentration pads.b.with load concentration pads*

5.3. Thrust force

Having confirmed the lining thickness, ring length and joints, it turns to the assembling of tunnel segments. During construction, the tunnel boring machine advances by pushing itself off with hydraulic jacks (also called thrust cylinders) against the already installed concrete segments. The thrust is the drive force to make the machine advance (see figure 5.8). So it is necessary to calculate the thrust force to make sure the TBM can move forward after the previous tunnel ring is assembled.

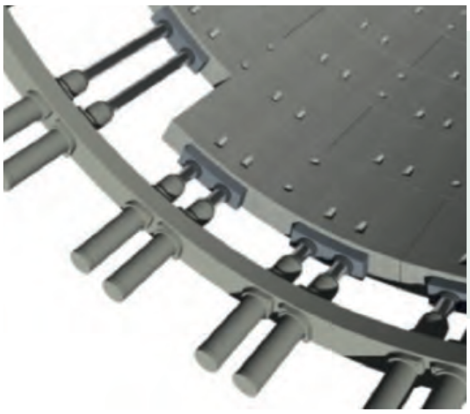


Figure 5.8: *Thrust force pushing on circumferential joints (from ITA Working Group 2)*

Calculation of the thrust force

The thrust cylinders should be designed to overcome the sum of all resistances.

$$P_v = \Sigma W + safetymargin \tag{5.1}$$

Where

P_v max. thrust cylinder force [kN]

W all resistance forces [kN]

The most important resistance forces for slurry shield are:

1. The friction force on the shield skin W_M.

2. Resistance from face support W_{FS}.

Besides, there are other resistance forces such as:

- Resistance to advance at the face due to excavation tools.
- Friction force between the lining and the tail seal of the shield.
- Increased skin friction through grouting, or swelling grounds, or curve driving and steering.
- Tractive force of the back-up system.
- Resistance to advance at the shield blade. This can be reduced (or even be zero) by intentional over-cut or by the cutting wheel being located at the front with a larger boring diameter.

The friction force on the shield skin is calculated with:

$$W_M = \mu \cdot \left[2\pi \cdot r \cdot L \cdot \frac{\sigma'_v + \sigma'_h}{2} + G_s \right] \tag{5.2}$$

Where

W_M friction force on the shield skin [kN]

μ friction coefficient

r radius of the shield [m]

L length of shield $L = 13m$

σ'_v effective vertical soil stress [kN/m^2]

σ'_h effective horizontal soil stress= $K_0 \cdot \sigma'_v [kN/m^2]$

K_0 coefficient of earth pressure at rest [-] $K_0 = 0.5$

G_s self-weight of the shield [kN]

In sandy and gravelly soils the shield skin can be lubricated with a bentonite or clay suspension, which can lower the friction coefficient to 0.1 to 0.2, here $\mu = 0.2$ is achieved.

$$W_{FS} = E_{max,ci} + E_{W,ci} \tag{5.3}$$

Where

W_{FS} resistance from face support [kN]

$E_{max,ci}$ resistance from earth pressure [kN]

$E_{W,ci}$ resistance from water pressure [kN]

To resist the thrust forces and prevent cracks, splitting reinforcement needs to be installed in the concrete lining. There are 9 segments in one tunnel ring, and for each normal segment, there are four jacks, for a key stone, there are two jacks. Totally, there are 34 jacks for one tunnel ring. The shield weight is related to the tunnel diameter. From figure 5.9, the weight of the slurry shield is around 2000 ton. (Blom, 2009)

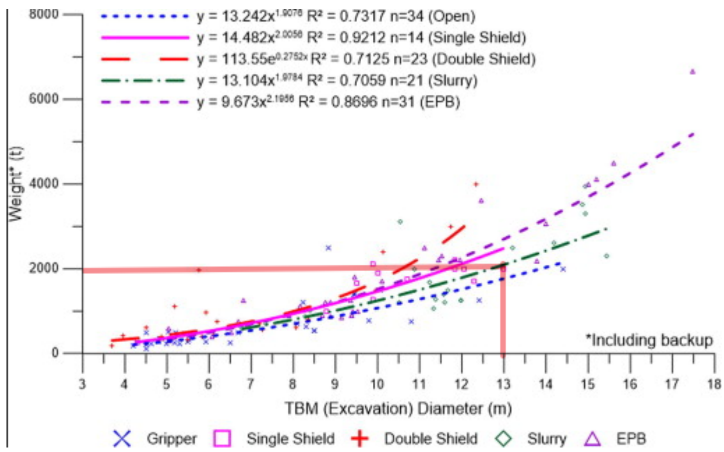


Figure 5.9: *Weight of TBM shield in relation to tunnel diameter*

Thrust force on lining is shown in figure 5.10.

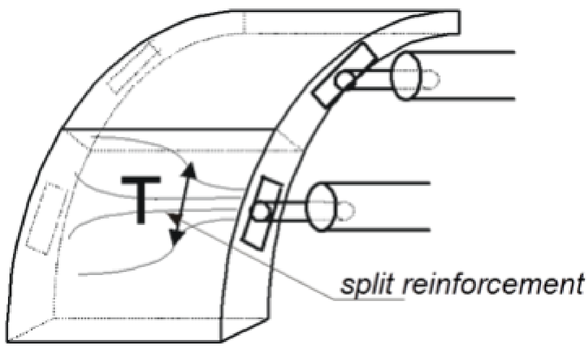


Figure 5.10: *Thrust force on lining (Nieuwe dictaat Tunnels 2018)*

Starting shaft

$G_s = 20000kN$, $L = 13m$, $r = 6.5m$, $\mu = 0.2$, $E_{max,ci} = 2854.7kN$, $E_{W,ci} = 15264.2kN$, $\sigma'_v = 176.5kN/m^2$, $\sigma'_h = 88.25kN/m^2$, $W_M = 18056.4kN$, $W_{FS} = 18118.9kN$, $P_v = W_M + W_{FS} = 36175.25kN$, Jack force for each jack is: $P_v/34 = 1064kN$

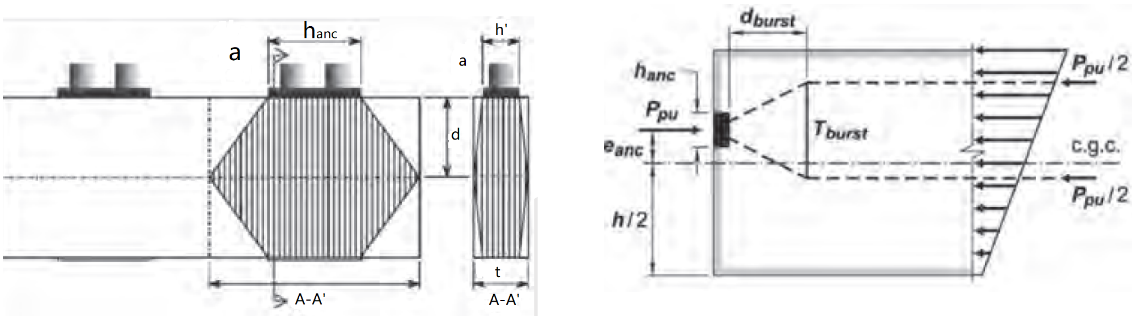
Canal

$G_s = 20000kN$, $L = 13m$, $r = 6.5m$, $\mu = 0.2$, $E_{max,ci} = 2919.2kN$, $E_{W,ci} = 39687kN$, $\sigma'_v = 267kN/m^2$, $\sigma'_h = 133.5kN/m^2$, $W_M = 25263.7kN$, $W_{FS} = 42606.2kN$, $P_v = W_M + W_{FS} = 67869.9kN$, Jack force for each jack is: $P_v/34 = 1996.2kN$

Reception shaft

$G_s = 20000kN$, $L = 13m$, $r = 6.5m$, $\mu = 0.2$, $E_{max,ci} = 3212kN$, $E_{W,ci} = 15264.2kN$, $\sigma'_v = 186kN/m^2$, $\sigma'_h = 93kN/m^2$, $W_M = 18812.9kN$, $W_{FS} = 18476.2kN$, $P_v = W_M + W_{FS} = 37289.1kN$, Jack force for each jack is: $P_v/34 = 1096.7kN$

High compression stresses develop under the jacking pads, which result in the formation of significant bursting tensile stresses deep within the segment (see figure 5.11). Furthermore, spalling tensile forces are generated between adjacent jack pads along circumferential joint. (Bakhsh and Nasri, 2019)



(a) disturbance area for strut stress in transverse direction

(b) Bursting tensile forces and corresponding parameters recommended by ACI 318-14; and DAUB (2013)

Figure 5.11: Bursting tensile stress calculation model (from WG2-ITA)

Structural concrete codes such as ACI 318-14 permit the use of simplified equation (5.4) to determine bursting tensile stress T_{burst}.

$$T_{burst} = 0.25 \times P_u \cdot (1 - \frac{h_{anc}}{h}) \tag{5.4}$$

Where

T_{burst}	bursting tensile force [kN]
P_u	thrust force [kN]
h_{anc}	length of jack pad [m]

The maximum thrust force (1996.2kN) achieved above is used to calculate bursting tensile force. Assume length of two jack pads h_{anc} is half of segment length. Then the bursting tensile force is calculated as:
$T_{burst} = 0.25 \times 1996.2 \times (1 - \frac{1}{2}) = 250kN$

Reinforcing rebar is designed to accommodate the significant bursting tensile stress developed by jacking force. Equation (5.5) is used to determine the required area A_s of reinforcing bars with a yield stress of f_y for a reinforced concrete segment.

$$T_{burst} = f_y \times A_s \tag{5.5}$$

Where

T_{burst}	bursting tensile force [kN]
A_s	area of reinforcing bars [mm^2]
f_y	yield stress of reinforcing bars [N/mm^2]

In Appendix F , it is about calculation of rebar area of reinforced concrete segment. $6800mm^2$ is the minimum required rebar area to guarantee the stability and safety of tunnel construction. Here it is used to check the bursting tensile stress:
$T_{burst} = 30 \times 6800 = 204kN \leq 250kN$

The minimum rebar area ($6800mm^2$) can not satisfy the requirement. In order to prevent the damage caused by significant bursting tensile stress, the required area A_s of reinforcing bars will increase to $8334mm^2$. Calculation is as follows:
$A_s = \frac{T_{burst}}{f_y} = \frac{250}{30} \times 10^3 = 8334mm^2$

5.3.1. Structural lining design

Having reinforced the segments and completed the assembling of the tunnel segments, it turns to check the lining force of tunnel rings. In this section, the parameters of tunnel segments, like lining thickness and ring length, are used to calculate the tunnel internal forces by adopting analytical method. Many models have been proposed and developed for structural lining design. The most commonly used model for shallow tunnels (depth $H \leq 2.5D$) is bedded-beam model (See figure 5.12), proposed by Duddeck and Erdmann (Duddeck and Erdmann, 1985).

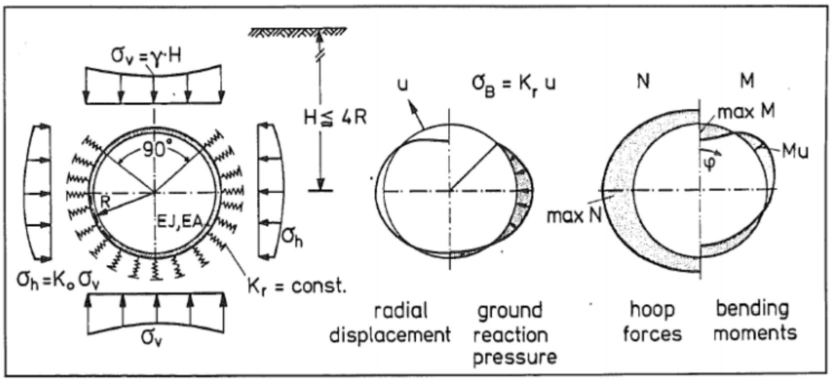


Figure 5.12: *Bedded Beam Model for Shallow Tunnels (picture by Minh V.)*

This model automatically includes the interaction between the soil and the structure. In the bedded-beam model, the interaction between the soil and the structure is captured by bedding springs. For shallow tunnels, Duddeck proposed that no stress relaxations occur at the crown of the tunnels, and therefore full primary vertical stress on top of the tunnel needs to be applied. For this case, soil springs are only applied where the deflection of tunnels is oriented outwards. At the crown of the tunnel, where the lining deforms inwards, there would be tension in the soil and no bedding is applied over a lining arc length of 90° to 120°.

For this model, some assumptions are applied:

- The stress-strain deformations of a cross-section are in plane strain conditions for both the tunnel lining and the ground.
- The soil stress in the tunnel lining is equal to the primary stress in the undisturbed condition.
- The ground will return to the conditions before tunneling at the final stage of tunnelling or in the long term.
- The interaction between ground and tunnel is only limited to radial springs.
- Ground and tunnel lining are elastic materials.

This model assumes a uniform tunnel lining, while in reality, it includes longitudinal joints which affect the deformation, however this is not included in the bedded beam model.
The interaction between soil and the tunnel lining can be presented via a spring stiffness in this model. The stiffness of radial spring k_r is given by:

$$k_r = \frac{E_s}{r} \tag{5.6}$$

E_s is estimated as:

$$E_s = E \cdot \frac{1-\nu}{(1+\nu)(1-2\nu)} = 1.5E \tag{5.7}$$

Where

k_r	radial spring stiffness [kN/m^3]
E_s	Stiffness modulus of the soil [kN/m^2]
K_0	coefficient of neutral horizontal soil stress [-], $K_0 = 0.5$
ν	Poisson's ratio [-],$\nu = 1/3$
E	Young's modulus of the soil [kN/m^2]

The relative stiffness between the ground and the tunnel support is incorporated into the solution using two dimensionless parameters: the flexibility ratio α and the compressibility ratio β.

$$\alpha = \frac{E_s \cdot r^3}{EtI_t} \tag{5.8}$$

$$\beta = \frac{E_s \cdot r}{EtA_t} \tag{5.9}$$

Where

α	flexibility ratio [-]
β	compressibility ratio [-]
E_t	elasticity modulus of the concrete of the tunnel [kN/m^2]
I_t	moment of inertia of the lining per unit length of tunnel (i.e. $I_t = \frac{1}{12}d^3$) [m$^4/m$]
A_t	cross sectional area of the lining per unit length of tunnel (i.e. $A_t = d$) [m$^2/m$]
r	radius of the tunnel lining [m]
E_s	constraint modulus of the soil [kN/m^2]

Reduction factors can be used to calculate the normal force and the bending moment, including soil-structure interaction.

$$N(\theta) = -C_{N_0}\frac{\sigma_v^{'} + \sigma_h^{'}}{2} \cdot r - u \cdot r + C_{N_2}\frac{\sigma_v^{'} - \sigma_h^{'}}{2} \cdot r \cdot \cos(2\theta) \tag{5.10}$$

$$M(\theta) = -C_m \frac{\sigma_v^{'} - \sigma_h^{'}}{4} \cdot r^2 \cdot \cos(2\theta) \tag{5.11}$$

These reduction factors also depend on the bonding condition between the lining and the ground. In this project, the bonding condition is assumed: A perfectly rough condition (full bond,= 0.3), resulting in complete compatibility of radial and circumferential displacement and full transmission of normal stresses and shear stresses across the interface.

$$C_M \approx \frac{4}{4+0.342\cdot\alpha} \tag{5.12}$$

$$C_{N_0} \approx \frac{2}{2+1.54\cdot\beta} \tag{5.13}$$

$$C_{N_2} \approx \frac{2\cdot(1+0.064\cdot\alpha)}{2+0.171\cdot\alpha} \tag{5.14}$$

Where

$N(\theta)$	normal force, including interaction [kN/m]
$M(\theta)$	bending moment, including interaction [kNm/m]
C_M	reduction factor that accounts for the bending moment [-]
C_{N_0}	reduction factor for the homogeneous part of the circumferential normal force [-]
C_{N_2}	reduction factor for the part of the normal force that depends on the position in the lining [-]
α	flexibility ratio [-]
β	compressibility ratio [-]

Starting shaft

At the tunnel axis, $H = 14m, NAP - 3.5m$, concrete 45/55 is applied:
$E_{100} = 45MPa,\ \nu = 0.2,\ \sigma'_v = 176.5kpa,\ \sigma'_h = 88.25kpa,\ E = E_{100}\left(\frac{\sigma'_v}{100}\right)^{0.5} = 59784kpa,\ k_r = 10219,\ E_t = 36283Mpa,\ I_t = 0.018m^4/m,\ \alpha = 27.9, \beta = 0.02$, when $\theta = \frac{\pi}{2}, N_{max} = -1830KN$, when $\theta = 0, M_{max} = -275KN\cdot m$

Canal

At the tunnel axis, $H = 24.9m, NAP - 21.9$, concrete 45/55 is applied:
$E_{100} = 45MPa, \nu = 0.2, \sigma'_v = 267kpa, \sigma'_h = 133.5kpa, E = E_{100}\left(\frac{\sigma'_v}{100}\right)^{0.5} = 73530kpa, k_r = 12569, E_t = 36283Mpa,$
$I_t = 0.018m^4/m,\ \alpha = 34.4, \beta = 0.024$, when $\theta = \frac{\pi}{2}, N_{max} = -2813KN$, when $\theta = 0, M_{max} = -358KN\cdot m$

Reception shaft

At the tunnel axis, $H = 14.5m, NAP - 3.5$, concrete 45/55 is applied:
$E_{100} = 45MPa, \nu = 0.2, \sigma'_v = 186kpa, \sigma'_h = 93kpa, E = E_{100}\left(\frac{\sigma'_v}{100}\right)^{0.5} = 61371kpa, k_r = 10490, E_t = 36283Mpa,$
$I_t = 0.018m^4/m,\ \alpha = 28.7, \beta = 0.02$, when $\theta = \frac{\pi}{2}, N_{max} = -1888KN$, when $\theta = 0, M_{max} = -284KN\cdot m$

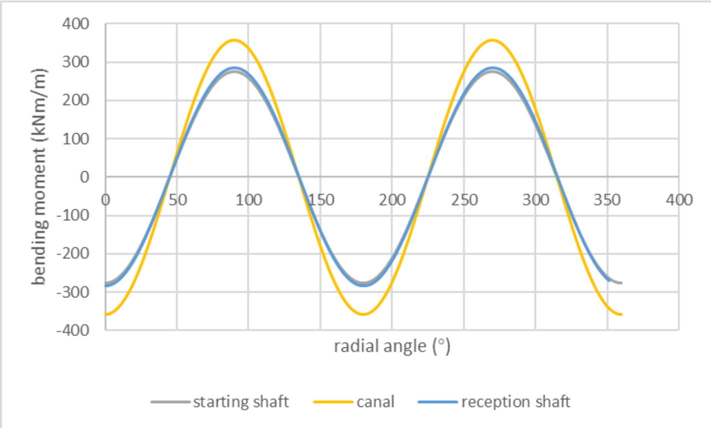


Figure 5.13: *Bending moment calculated by analytical method*

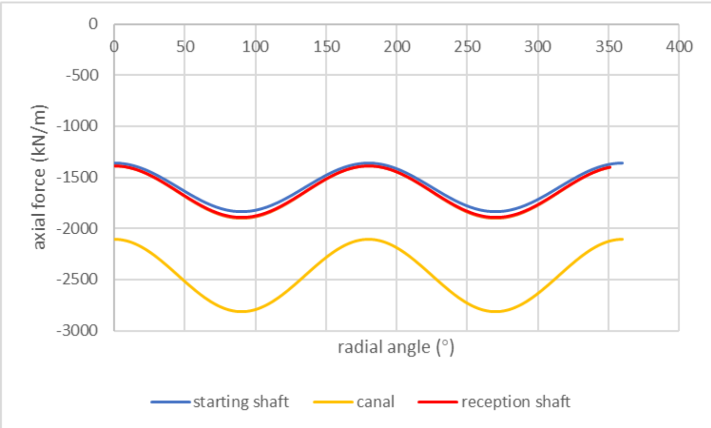


Figure 5.14: *Axial force calculated by analytical method*

Three critical points are picked to calculate the normal force and bending moment (see figure 5.13, 5.14). It can be seen the bending moment and axial force at starting and reception shaft are almost the same and lower than those at canal. The internal forces at canal are always the largest because the soil cover is 13.4m which is thicker than that at shafts (around 7.5m). So a heavier overburden causes larger internal forces. Figure 5.13 also shows that the maximum bending moment occurs at the top and waist of the tunnel. And the maximum axial force happens at the waist of the tunnel.

5.4. Settlement

In last section, the internal force has been calculated. In this section, the external deformation will be checked. It is important to calculate the settlement during and after tunnel construction. In a bored tunnel construction, total settlement is summed by short term and long term settlement. Figure 5.15 shows the settlement in the longitudinal tunnelling direction. In general, surface settlement of the ground during and after bored tunnelling can be caused by: 1). Excessive soil removal at the excavation face; 2). Insufficient support of the excavation face; 3). Deformation of the tunnel lining itself; 4). Redistribution of the stresses and strains around the tunnel; 5). Adjustment of the water pressure distribution around the tunnel.

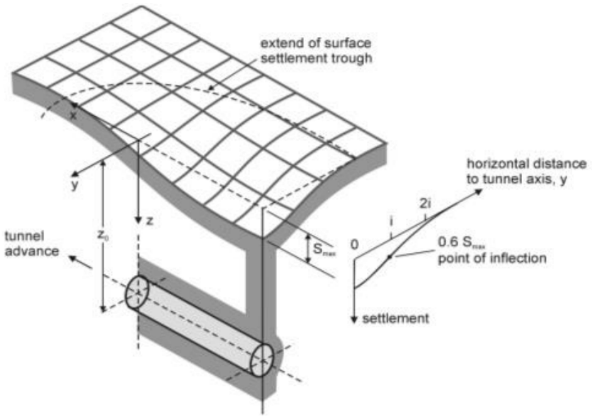


Figure 5.15: *Surface settlement due to tunneling (Nieuwe dictaat Tunnels 2018)*

In this report, the short term settlement is calculated with Peck's formula (Peck, 1969). This empirical formula is based on observations and analyses of a very large amount of monitoring data from tunnels and is most commonly used for settlement calculations in 2D plane.

According to Peck's formula, the settlement trough that occurs in a cross section perpendicular to the tunnel axis has the same shape as a Gaussian curve. The major assumption is that the volume of the settlement trough equals the volume of soil loss around the tunnel (see figure 5.16).

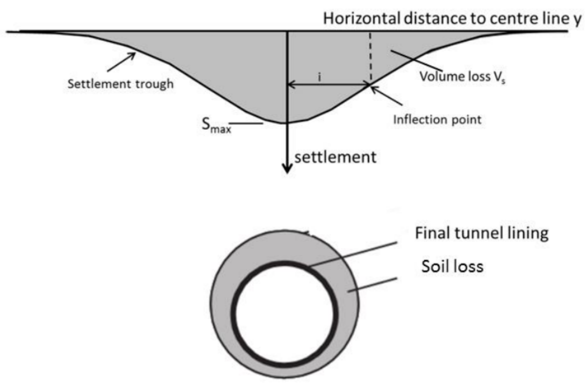


Figure 5.16: *Settlement trough (Gaussian curve) (Nieuwe dictaat Tunnels 2018)*

There are some equations to describe the Gaussian curve of the settlement:

$$Q(y) = S_{max} \cdot e^{-\left(\frac{y^2}{2 \cdot i^2}\right)} \tag{5.15}$$

The maximum settlement is calculated with:

$$S_{max} = \frac{V_s}{i \cdot \sqrt{2\pi}} \quad (5.16)$$

Where

$S(y)$	settlement at ground level [m]
S_{max}	maximum settlement at ground level [m]
y	horizontal distance from the tunnel axis [m]
i	settlement trough factor [m]
Vs	volume of the settlement trough [m$^3/m$]

The settlement trough is determined by the distance between the tunnel axis and the inflection point of Gaussian point. The inflection point is also called settlement trough factor. New and O'Reilly (New.B and O'Reilly.M, 1982) determined the following settlement trough factors:

Fine grained soils (e.g.clay):i=0.43$\cdot z + 1.1$

Coarse grained soils (e.g.sand): i=0.28$\cdot z - 0.1$

Where

i	settlement trough factor [m]
z	depth of tunnel axis [m]

The volume of the settlement trough V_s includes settlements due to insufficient face support, tunnelling machine passage and the annular gap grouting. This factor is calculated as a percentage of the volume of the tunnel V_{tunnel}.

$$V_s = V_L \cdot V_{tunnel} \quad (5.17)$$

Where

V_s	volume of the settlement trough per m [m$^3/m$]
V_L	volume loss influence factor [%]. Typical empirical values lie between 0.5 % (slurry machine) and 1 % (EPB machine). In recent projects volumes losses of 0.1 to 0.3% were even achieved
V_{tunnel}	volume of the tunnel per m [m$^3/m$], $V_{tunnel} = \frac{\pi \cdot D_0^2}{4}$
D_0	outer diameter tunnel [m]

From the above equation, in this project 4 locations are picked to calculate the settlement :

Starting shaft:

z=14m, i=0.28$\times z - 0.1 = 3.82m$,V$_{tunnel} = \frac{\pi \times 13^2}{4} = 132.7m^3/m$, $V_L = 0.001$, $V_s = V_L \times V_{tunnel} = 0.23m^3/m$,

$S_{max} = \frac{V_s}{i \cdot \sqrt{2\pi}} = 23.7mm$

If the allowable maximum settlement is 25mm in greenfield, then the volume loss is 0.18%. Settlement trough at starting is shown in figure 5.17.

Brick factory:

z=23.2m, i=$0.28 \times z - 0.1 = 6.4m$,$V_{tunnel} = \frac{\pi \times 13^2}{4} = 132.7m^3/m$, $V_L = 0.001$, $V_s = V_L \times V_{tunnel} = 0.23m^3/m$,

$S_{max} = \frac{V_s}{i \cdot \sqrt{2\pi}} = 14.1mm$

If the allowable maximum settlement is 25mm in greenfield, then the volume loss is 0.17%. Settlement trough at brick factory is shown in figure 5.17.

Canal:

z=19.9m, i=$0.28 \times z - 0.1 = 5.5m$,$V_{tunnel} = \frac{\pi \times 13^2}{4} = 132.7m^3/m$, $V_L = 0.001$, $V_s = V_L \times V_{tunnel} = 0.13m^3/m$,

$S_{max} = \frac{V_s}{i \cdot \sqrt{2\pi}} = 16.5mm.$

If the allowable maximum settlement is 25mm in greenfield, then the volume loss is 0.26%. Settlement trough at canal is shown in figure 5.17.

Reception shaft:

z=14.5m, i=$0.28 \times z - 0.1 = 3.96m$,$V_{tunnel} = \frac{\pi \times 13^2}{4} = 132.7m^3/m$, $V_L = 0.001$, $V_s = V_L \times V_{tunnel} = 0.13m^3/m$,

$S_{max} = \frac{V_s}{i \cdot \sqrt{2\pi}} = 22.8mm$

If the allowable maximum settlement is 25mm in greenfield, then the volume loss is 0.19%. Settlement trough at reception shaft is shown in figure 5.17.

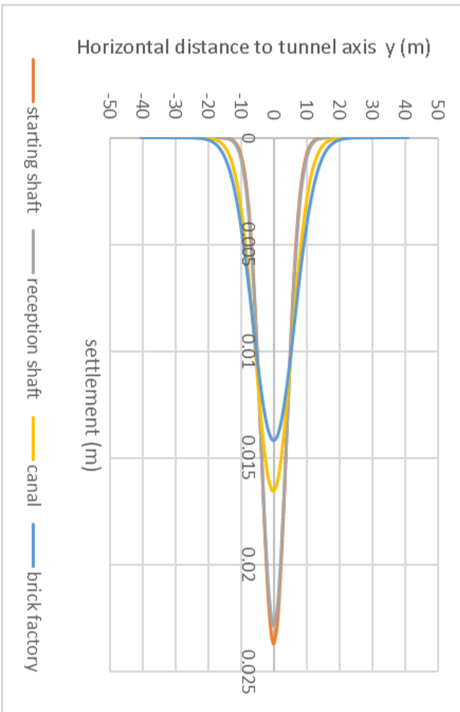


Figure 5.17: *settlement at reception shaft*

It is concluded that the maximum volume loss occurs at the center of the tunnel where the maximum settlement happens. When the soil cover decreases, the maximum settlement increases and the influenced area on the top decreases.

5.5. Cross passage

The previous sections are about the tunnel lining design, internal and external behaviors. This section will focus on the cross passage design. Cross passages are required to connect the twin tunnels in this project. They provide emergency egress between tunnel tubes and house installations and electrical equipment that support tunnel operations. In case of disaster, people should be able to proceed to the other safe tunnel tube via a cross passage. Simultaneously, the emergency services can safely reach the location of the accident or fire via the cross passage.

5.5.1. Location of cross passage

According to Dutch safety standard, the maximum spacing between cross passage is 250m. The distribution of cross passages in the longitudinal direction is shown in figure 5.18. There are totally 5 cross passages along the alignment. They are all located in bored section. The geology of all cross passages are mainly from gravelly sand to fine sand. Only there are clay and peat in cross passage 2. Water pressure to be restrained varies from 1.34 to 2.79 bar.

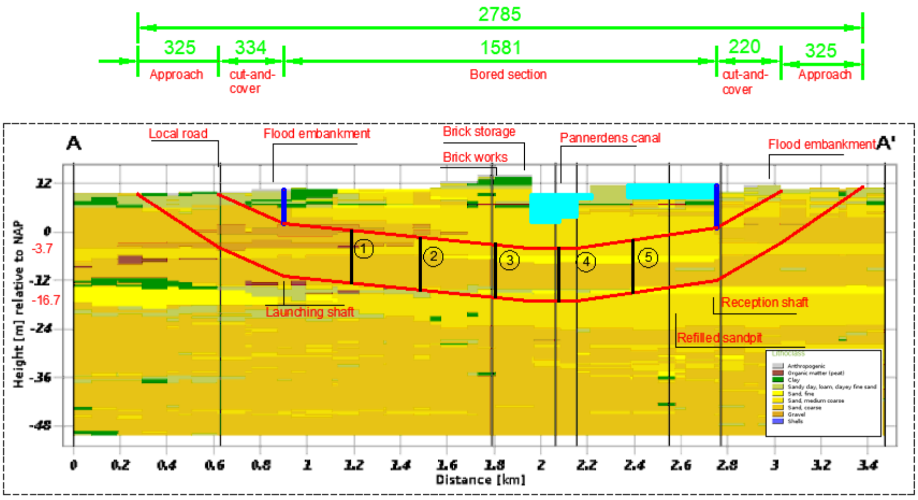


Figure 5.18: *Distribution of cross passages*

5.5.2. Construction of cross passages

The construction of the cross passages has to be carried out entirely underground. The deepest cross passage is situated at a depth of NAP -16.7m and about 19.8m under the river bed. Excavation of soil is necessary for the construction of cross passage. In order to excavate the ground, the surrounding soil which mainly consists of medium sand and coarse sand, must be made adequately load-bearing, stable and watertight. In theory, there are three construction methods for building cross passages (J.Heijboer and de Linde, 2004) :

- Working under increased air pressure
- The realisation of a stable soil mass by means of soil grouting
- The realisation of a stable soil mass by freezing the water present in the soil around the cross passage

For the third and the fourth cross passages which are located in deeper positions, they are built in medium to coarse sand. The water pressure varies from 1.72 to 2.79 bar. The realisation of an absolutely watertight grout-body and under increased air pressure is considered as risky. Therefore, freezing technology is used to construct cross passages.
The cross passage will also house electrical equipment and ducts to allow for cables to connect the running tubes. These measurements result in an internal diameter of 3.6m which can provide a space of 2.5 meters wide and 2.75m high. This reference design provides for an access to the cross passage with a width of 1.5m which makes it possible to situate opening within a single tunnel ring. Cross passage will be made by using shotcrete and cast in-situ reinforced concrete. Steel segment will be used around the opening in the lining of running tubes to divert the normal forces in the lining around the opening (Catsman, 2018). (See figure 5.19)

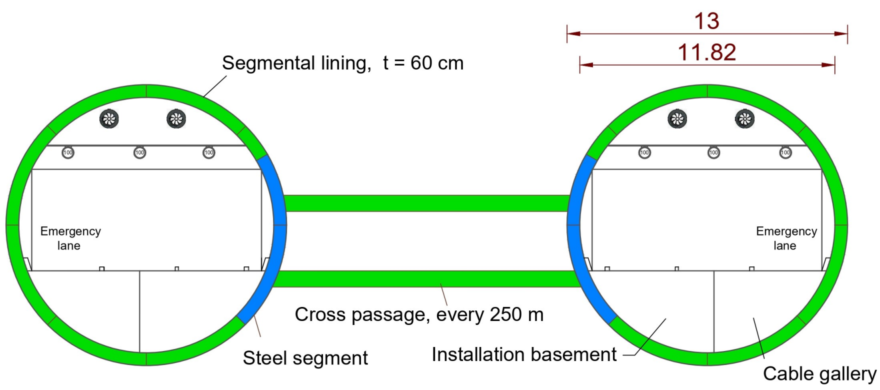


Figure 5.19: *The profile of cross passage in bored section*

For the first, second and fifth cross passages, they are located in medium to coarse sand mixed with some

anthropogenics. Water pressure to be restrained varies from 1.34 to 2.62 bar which is not too high. The cost of freezing technique is relatively high, so the jet grouting technique is suggested to construct these three cross passages.
The technique of jet grouting utilizes high-pressure water-cement jet streams (sheathed with air pressure) to cut, replace and mix with native soils. Due to the lack of soil data, as a rough design of cross passage, it is hard to give a size of jet grouting zone. But the plan view of the cross passage is shown in figure 5.20.

For both the first and second cross passages, shafts will be built in the future cross passage and there are struts inside the shaft to support the earth pressure. For instance in the first cross passage, it is constructed at a depth from 5.4m -NAP to 15.1m -NAP including a 4.4 m-deep drainage sump. The profile is shown in figure 5.21.

For the fifth cross passage which is below the lake, it is relatively hard to construct the shaft in the lake, so the excavation will start from one tunnel tube to the other. In order to avoid the stress concentration and divert the normal force around the opening in the main tunnel lining, steel segment will be used to replace the concrete segment and a system of pre-stressed steel rings inside the tunnel tubes will be placed. Figure 5.22 shows the configuration of this technique.(Mortier and Leon L.T.C., 2004)

It is necessary to use the water-leak test on the jet-grouted body to check the improvement of the soil. If a significant amount of water-leak is measured in the tunnel, additional chemical grouting will be conducted. Even the compressed-air method will also be used to repress the inflow of groundwater at the face of excavation, and to increase the safety of construction.(Fang et al., 2013)

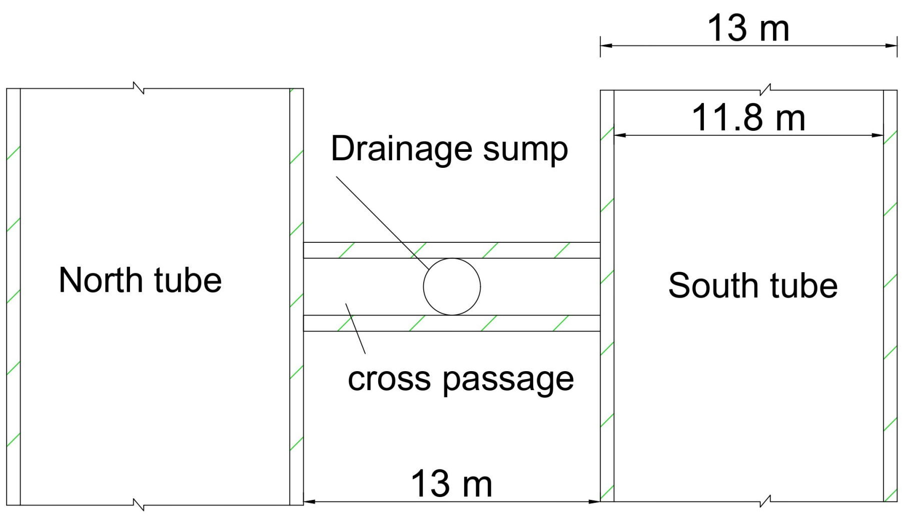


Figure 5.20: *Plan of cross passage*

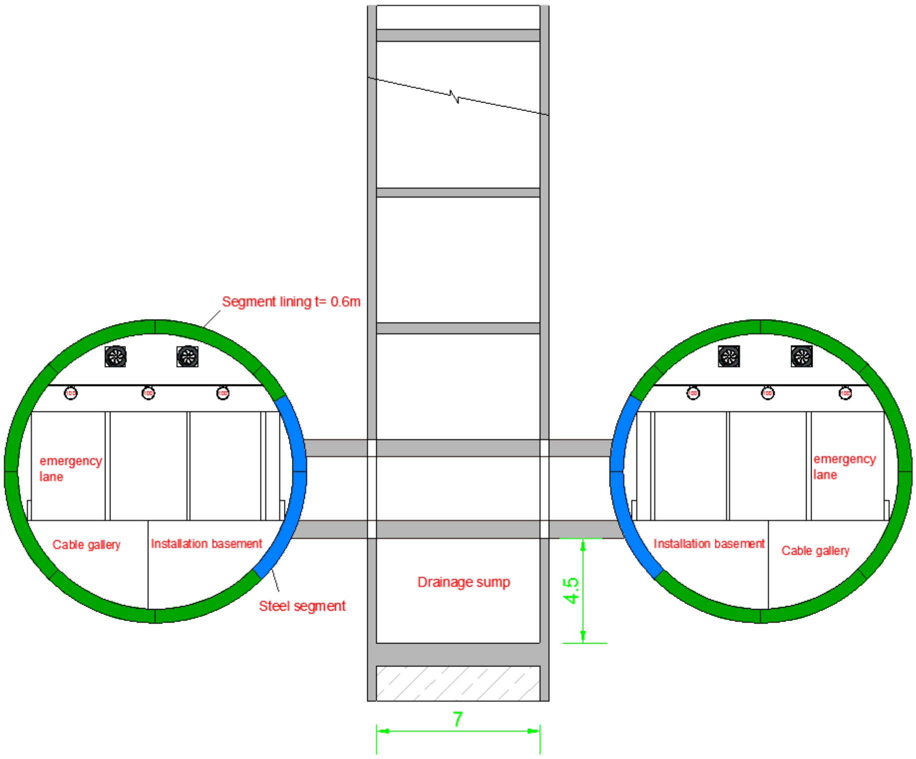


Figure 5.21: *profile of the third and fourth cross passages*

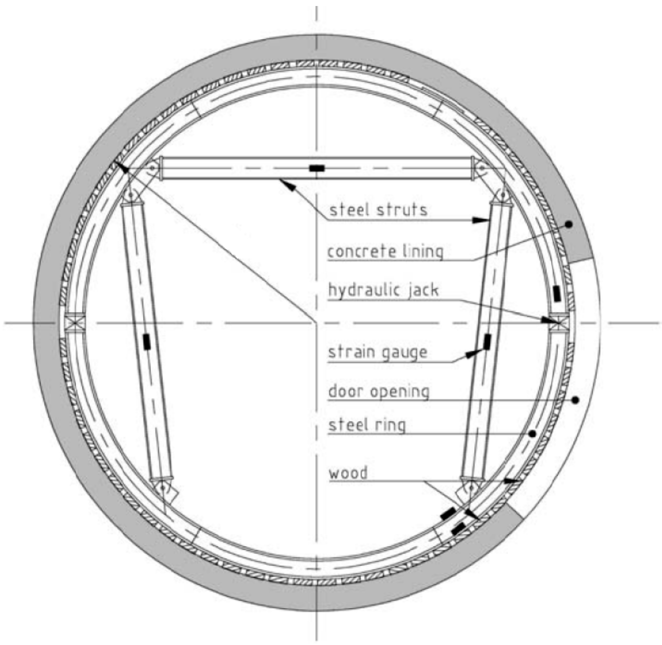


Figure 5.22: *Pre-stressed steel ring inside tunnel (from Mortier 2004)*

5.6. Conclusion

This chapter gives a preliminary design of tunnel. The design determines lining thickness, joints, calculates internal force of tunnel ring, thrust force, surface ground settlement due to tunnelling at critical positions and describes the construction of cross passages.

Lining thickness is determined as 0.6m. In the 2D plane design, the analytical solution does not consider the effect of the construction of the second tunnel on the first tunnel, so the internal force is symmetrical in the right and left half ring. By controlling volume loss, surface ground settlement can be controlled within 25mm at shafts, canal and brick factory. Cross passages are constructed by applying freezing and jet grouting due to different geological conditions.

6

In-depth design and optimisation of the chosen concept

6.1. Introduction

This chapter mainly focuses on the in-depth design of reception shaft of the chosen concept. The cost of constructing the shaft is an important aspect for the contractor to consider. The construction of shaft can cost a lot if the distance between tunnel tubes is too large. However, if the distance is too small, then constructing such narrow-spaced tubes will be difficult, as the second tunnel tube would influence the structural deformation of the first tunnel. Since the spacing between two tubes affects the project execution a lot, an in-depth design of the tunnel spacing is necessary. In this chapter, numerical simulation is used to model the tunnel construction and the structure behavior, so as to assist the optimization.

A short description regarding the numerical software PLAXIS 2D and related soil model and input parameters are presented in Section 6.2. The impact of spacing on tunnel internal force and soil settlement is investigated by using PLAXIS 2D in Section 6.3. Section 6.4 shows the process of determining spacing based on stability and safety of both tunnels. Optimisation of tunnels spacing is elaborated by adopting some soil improvement techniques in section 6.5 . These soil improvement techniques are evaluated in section 6.6. Section 6.7 checks the stability of tunnels after removing magnetite on surface ground. Section 6.8 checks soil settlement at the brick factory and the dike.

6.2. PLAXIS 2D

PLAXIS 2D is a two-dimensional finite element program used to perform deformation, stability and flow analysis for various types of geotechnical applications (R.B.J.Brinkgreve, 2020a). The mechanical behavior of soils depends on the stress-strain relationship that is called constitutive model or material model. Hence, the models form the theoretical framework and the qualitative mechanical behavior of the soils (Brinkgreve, Lecture notes CIE4361 Behavior of Soils and Rocks). To represent the stress-strain behavior of the soils and rocks, different constitutive models are developed and these models need different input parameters to quantify the soil behaviors. In this section, a few constitutive models are explained.

6.2.1. Constitutive models

Mohr-Coulomb model and Hardening Soil model are explained from aspects of their characteristics and relevant input parameters for analysis.

Mohr-Coulomb model

The Linear-Elastic Perfectly-Plastic (LEPP) Mohr-Coulomb model consists of both elastic and plastic parts. The linear elastic part of the Mohr-Coulomb model is based on the Hooke's law of isotropic elasticity. The perfectly plastic part is based on the Mohr-Coulomb failure criterion, formulated in a non-associated plasticity framework (PLAXIS 2D, Material Models, 2019). Plasticity involves the development of irreversible strains. During unloading and reloading, the behavior becomes elastic again, leaving some residual strain. As for the time that plasticity occurs and how much the plastic strain is, the theoretical formulation of a plasticity model involves a yield function and plastic potential (Brinkgreve, Lecture notes CIE4361 Behavior of Soils and Roks). The stress-strain path of this model is in figure 6.1

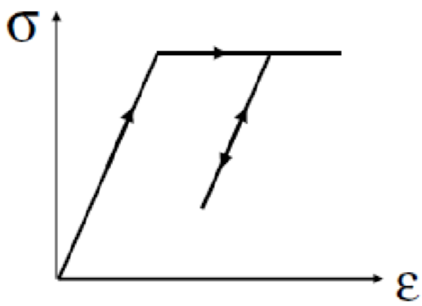


Figure 6.1: *The linear-Elastic Perfectly-plastic (LEPP) stress-strain behavior (lecture notes CIE4361)*

The input parameters for this model:

- Two elastic parameters: Young's modulus (E), Poisson's ratio (ν)
- Three plastic and strength parameters: Cohesion (c), Friction angle (ϕ) and Dilatancy angle (ψ)

Hardening Soil model

The Hardening Soil model is different from elastic perfectly-plastic model, the yield surface of a hardening plasticity model is not fixed in principal stress space. A basic feature of this model is the stress dependency of soil stiffness and thus stiffness increases with pressure (R.B.J.Brinkgreve, 2020b). When subjected to primary deviatoric loading, soil shows a decreasing stiffness and simultaneously irreversible plastic strains develop. During unloading and reloading, the soil becomes stiffer and may involve hysteresis as shown in figure below.

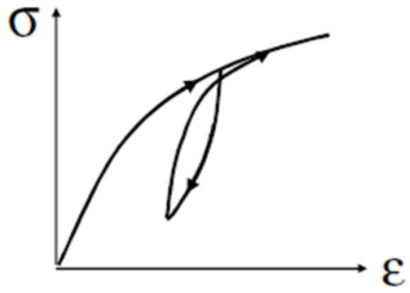


Figure 6.2: *The stress-strain relationship of Hardening Soil model (lecture notes CIE4361)*

The Hardening Soil model is an advanced model for simulating the behavior of different types of soil, both soft and stiff soils (R.B.J.Brinkgreve, 2020b). The input parameters for this model:

- Four stiffness parameters: the triaxial loading stiffness (E_{50}), the triaxial unloading stiffness (E_{ur}), the oedometer loading stiffness (E_{oed}) and the rate of stress-dependency (m)
- One elastic parameter: the Poisson's ratio (μ)
- Three strength paraments: cohesion (c), friction angle (ϕ), dilatancy angle (ψ)
- Reference pressure: P^{ref}
- Failure ratio R_f
- Stress ratio in 1D primary compression: K_0^{nc}

6.2.2. Soil parameters determination

The determination of parameters should be based on the site investigations and lab tests. In the bridge solution, soil data is not available until now. Luckily, in the Betuweroute railway project, Fugro Ingenieursbureau

BV carried out a soil investigation consisting of 2 CPTs (Appendix E). They have interpreted the soil investigation and estimated the soil parameters for the construction site on the eastern ramp. The input soil data is shown in Appendix E.

As the geology is mainly medium dense sand and there is enough soil data, it is more accurate to choose the Hardening Soil model for the tunnel simulation and it is the same soil model in the Betuweroute railway tunnel. The tunnel alignment is close to the railway tunnel, so these soil data can be assumed as input parameters in the selected tunnel concept and the final input parameters are shown in table 6.1 and table 6.2.

Table 6.1: Input soil data 1

Soil layer	Material	Constitutive model	Type	N.A.P (Top)	N.A.P (Bottom)	γ_{unsat} $[kN/m^2]$	γ_{sat} $[kN/m^2]$	k_x $[m/d]$	k_y $[m/d]$	E_{50}^{ref} $[kN/m^2]$	E_{oed}^{ref} $[kN/m^2]$	E_{ur}^{ref} $[kN/m^2]$	c_{ref} $[kN/m^2]$
1	sand	Hardening Soil	Drained	10	9.5	18	20	1	1	50000	50000	2.0E5	0.1
2	clay	Hardening Soil	Undrained	9.5	8	18.9	18.9	2.68E-3	2.68E-3	2500	2500	10000	4
3	medium sand	Hardening Soil	Drained	8	-7	18.9	18.9	1	1	45000	45000	1.8E5	0.1
4	coarse sand	Hardening Soil	Drained	-7	-14	19.5	19.5	1	1	1.0E5	1.0E5	4.0E5	0.1
5	deep sand	Hardening Soil	Drained	-14	-23	18.4	19.4	1	1	55000	55000	2.2E5	0.1

Table 6.2: Input soil data 2

Soil layer	ϕ $\circ$	ψ $\circ$	v	P_{ref} $[kN/m^2]$	Power	k_0^{nc}	c_{incr} $[kN/m^3]$	$y_{ref} m > m$	R_f	T-strength $[kN/m^2]$	R_{inter}	ζ_{inter}
1	35	5	0.2	100	0.5	0.426	0	0	0.9	0	0.7	0
2	27.5	0	0.2	110	0.9	0.538	0	0	0.9	0	0.7	0
3	36.5	6.5	0.2	180	0.5	0.405	0	0	0.9	0	0.7	0
4	37.9	7.9	0.2	280	0.5	0.386	0	0	0.9	0	0.7	0
5	36.5	6.5	0.2	400	0.5	0.405	0	0	0.9	0	0.7	0

For the tunnel lining, the input parameters are shown in table 6.3.

Table 6.3: Properties of the tunnel lining

Parameter	Name	Lining	Unit
Material type	Type	Elastic, Isotropic	-
Normal stiffness	EA	7.98E+5	kN/m
Flexural rigidity	EI	1.12E+7	kNm^2/m
Weight	w	13.4	$kN/m/m$
Poisson's ratio	v	0.15	-

6.3. Different spacings between tunnel tubes

In this section, finite element analysis is conducted to simulate the spacing between the tunnel tubes by using PLAXIS 2D. The mesh is shown in figure 6.3. It contains 2402 triangular elements, 20021 nodes. The depth of soil layer is H = 2.5D = 32.5m. The horizontal extension of soil mass is 170m (about 13 times of the tunnel diameter). This extension defines no lateral boundary effect on the numerical modelling of the tunnel construction. There are totally 3 spacings and they are L = D = 13m, L = 0.5D = 6.5m and L = 0.25D = 3.2m, here D means the diameter of the tunnel and L means the distance between the outside edge of tunnel tubes. The right tunnel is the first tunnel to be constructed (FT) and the left tunnel is the second constructed tunnel (ST).

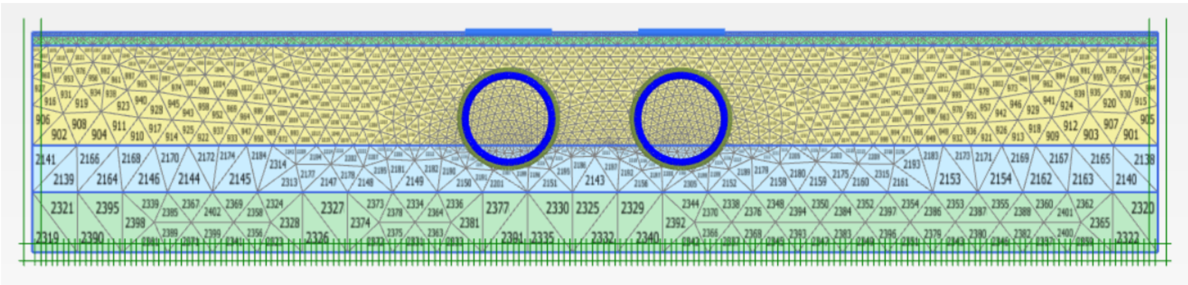
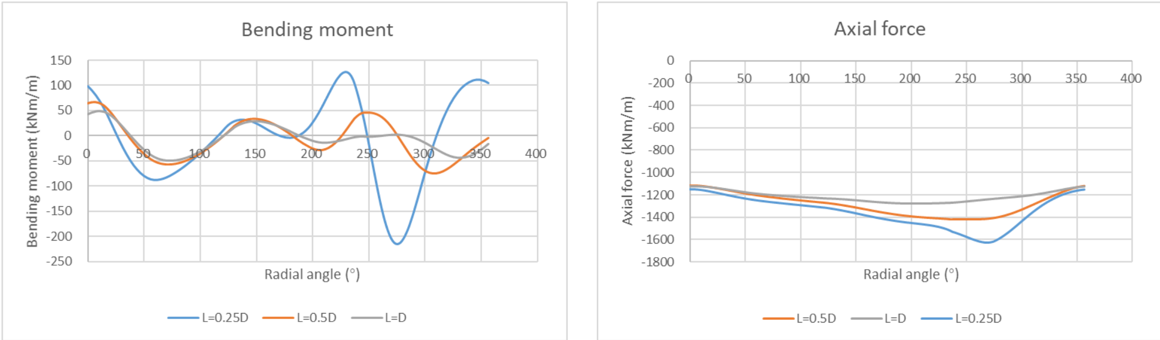


Figure 6.3: *Mesh generated by Plaxis 2D*

6.3.1. Effects of spacing on tunnel lining internal force

3 different tunnel spacing ratios (L/D = 1.0, 0.5, 0.25) are analysed. Figure 6.4 shows the influence on bending moment and axial force. The bending moment and axial force have increased a lot with decreasing the tunnel spacing. On the right side of the first tube (FT), the bending moment keeps almost the same path when decreasing the spacing from L=D to L=0.5D. Only on the left side of FT (the side close to the second tube), the bending moment shows an increase by 200%. When decreasing the spacing from L=D to L=0.25D, bending moment shows a big jump on the left side of the FT. The maximum change increases by 600%. As for axial force, it shows the similar trend. The change mainly occurs on the left side of the first tunnel. The maximum change increases by 17% and 14% when decreasing the spacing from L=D to L=0.5D and from L=0.5D to L=0.25D, respectively. The reason why small spacing increases the bending moment and axial force is that the second tunnel (ST) construction disturbs the soil between the tunnel tubes and stress release occurs, which increases the internal force (Channabasavaraj and Visvanath, 2013).
The internal forces generated by PLAXIS are smaller than those calculated by analytical method. Although the bedded-beam model proposed by Duddeck also considers the interaction between the soil and the structure, the possible reason of this result is that PLAXIS could simulate the soil-structure interaction more accurately thus the internal forces generated by PLAXIS are smaller.



(a) Influence on bending moment of the FT

(b) Influence on axial force of the FT

Figure 6.4: Internal force under different spacing

6.3.2. Effect of spacing on soil settlement

Figure 6.5 shows the soil settlement under 3 different tunnel spacings before constructing tunnels. The settlement is around 62mm which is caused by the magnetite on the surface ground because the unit weight of magnetite is quite heavy. In order to compensate the settlement caused by the magnetite before tunnel construction, extra soil is placed on the top to deal with it. The result of compensation is indicated in figure 6.6. The settlement decreases a lot from 62mm to around 29mm. It shows the settlement increases with decreasing the tunnel spacing. It grows up from 22mm (L=D) to 29mm (L=0.25D). This is because interaction and soil stress release is more obvious when decreasing the tunnel spacing (S.M.F. Hossaini and Talebinejad, 2012). For the 3 tunnel spacings, the maximum settlement occurs at the left side of the right tunnel (closest part of both tunnels). This is because the right tunnel is the first constructed tunnel and soil stress at left side can be affected more during construction of the second tunnel. When decreasing the tunnel spacing, the overlapping area of volume loss increases and settlement of the soil between tunnels increases as well. This is due to the effect of superposition of settlement.

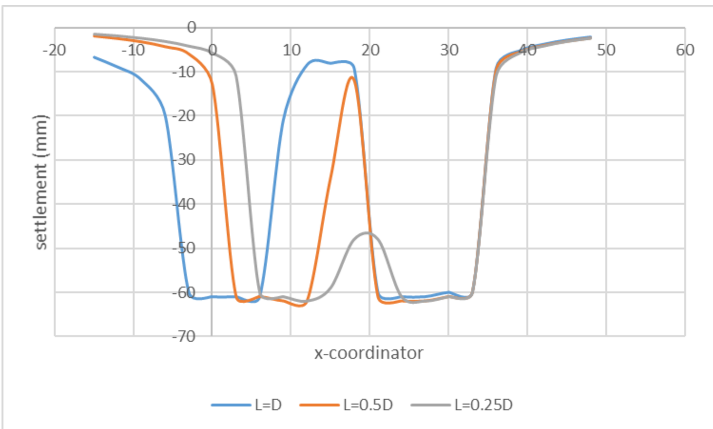


Figure 6.5: *Soil settlement only caused by magnetite*

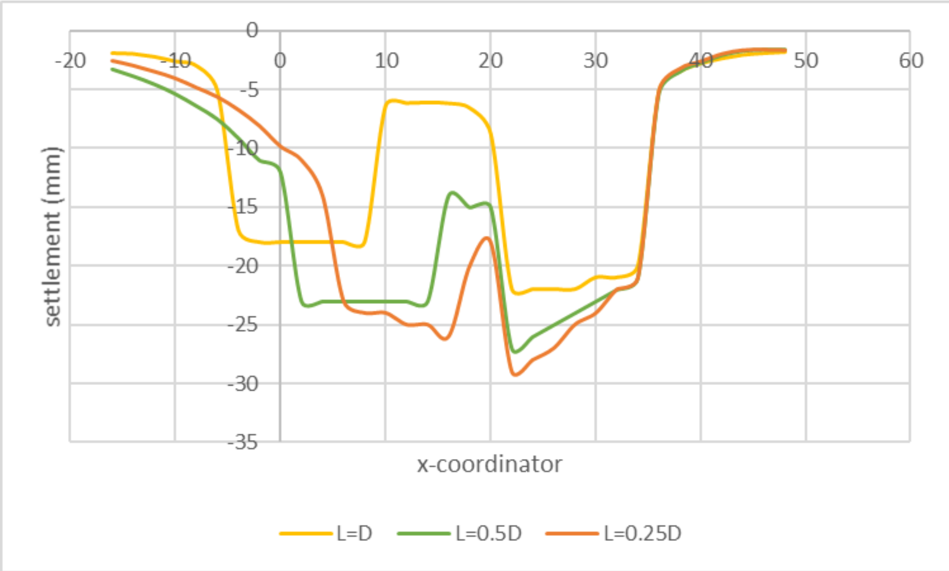


Figure 6.6: *Soil settlement after compensation by placing extra soil on the top*

6.4. Determination of tunnel spacing

As the internal force of tunnel lining has a wide range when decreasing the spacing to L=0.25D, it is necessary to find a proper spacing to guarantee the stability and safety of both tunnels. The tunnel would be not safe when the internal force exceeds the design resistance. Also, there is leakage when the crack width of segment is beyond the allowable crack width. So the internal force and the crack width are regarded as two criteria to assess the stability and safety of tunnels. As long as the internal force generated by PLAXIS 2D is below the ultimate limit state (ULS) of lining and the crack width is within 0.2mm, then the tunnel tubes stay stable and safe. In this case, it is necessary to firstly calculate the maximum allowable internal force and crack width in order to determine the tunnel spacing.

6.4.1. Ultimate Limit State consideration

In order to find the ultimate limit state, it is important to calculate the maximum allowable internal forces and then compare with the internal forces generated by PlAXIS 2D, if the generated internal forces are below the allowable internal force, it indicates the tunnel stays stable.

To calculate the maximum allowable internal force, a preliminary design of reinforcement concrete is carried out. The minimum reinforcement ratio should be able to guarantee the stability of tunnel lining during construction. The stability of tunnel includes two requirements which are listed below:

- The internal force generated by PLAXIS should be lower than the maximum allowable internal force which is determined by reinforced tunnel ring.

- The maximum crack width should be lower than 0.2mm (DAUB, 2013)

The detailed calculation is in Appendix F. From the calculation, it is concluded that the reinforcement ratio is determined by the maximum crack width rather than the allowable internal force because the internal forces generated in different tunnel spacing are all below the allowable internal force. After guaranteeing the stability of the first tunnel (FT), the next step is to check the stability of the second tunnel (ST) because the FT has disturbed the soil and influenced the internal force of the ST.

The method to ensure the safety of the ST is to ensure the crack width within 0.2mm by changing the tunnel spacing. Different tunnel spacing (L/D = 1.25, 1.0, 0.75, 0.5, 0.25) are carried out and the crack width of the second constructed tunnel is shown in figure 6.7. It can be seen crack width increases with decreasing the tunnel spacing because the soil stress will be more influenced by the first constructed tunnel, thus the internal force of the second constructed tunnel is strongly affected as well.

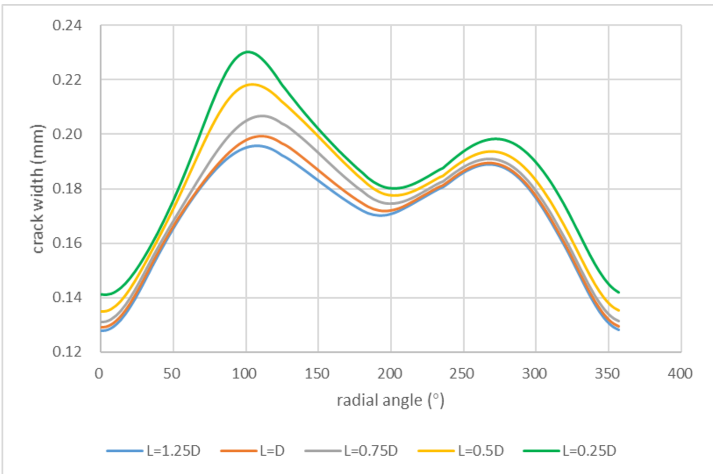


Figure 6.7: *Width of crack under different tunnel spacing*

From the figure, it is clear that only $L \geq D$, the crack width is lower than 0.2mm and thus the tunnel is safe. So L = D = 13m can be determined as the tunnel spacing .

6.5. Optimisation of tunnel tubes spacing

Although the tubes spacing (L=D) is determined in previous section, this spacing is still a bit large. In order to save the cost on constructing the shaft, the spacing can be optimised by applying some ground improvement techniques. This section will optimise the tunnel spacing from the aspects of soil strength, stiffness, reducing the soil disturbance and strengthening the tunnel segments.

6.5.1. More reinforcement in tunnel lining

As is mentioned in last section, the rebar area in segments is one of the factors that influence the designed internal force and crack width, which indicates increasing the rebar area can increase the designed internal force and decrease the cracking width. In this section, the impact on crack width by increasing rebar area is investigated.

Figure 6.8 shows when the rebar percentage (rebar area / area of cross section) in tension zone is increased from 0.57% to 0.66%, which is higher than minimum rebar percentage (0.26%) but lower than maximum rebar percentage (3.11%) (Shen, 2012), the maximum crack width decreases from 0.23mm to 0.2mm when the tunnel spacing (L) equals to 3.2m (0.25D). In order to find an optimal spacing, different rebar percentages have been tried. Table 6.4 shows the relation between rebar percentage and optimal tunnel spacing when the maximum crack is reached. When tunnel spacing is 0.25D (3.2m), rebar percentage is 0.66% when crack reaches the maximum value. So the tunnel spacing can be optimised to 0.25D (3.2m) when increasing rebar percentage.

The horizontal alignment is in figure 6.9, the curvature is 3000m according to the designed speed. In last section, the spacing is controlled at D without any soil improvement, herein this value is the critical value, so the length of tunnel to be more reinforced is 109m.

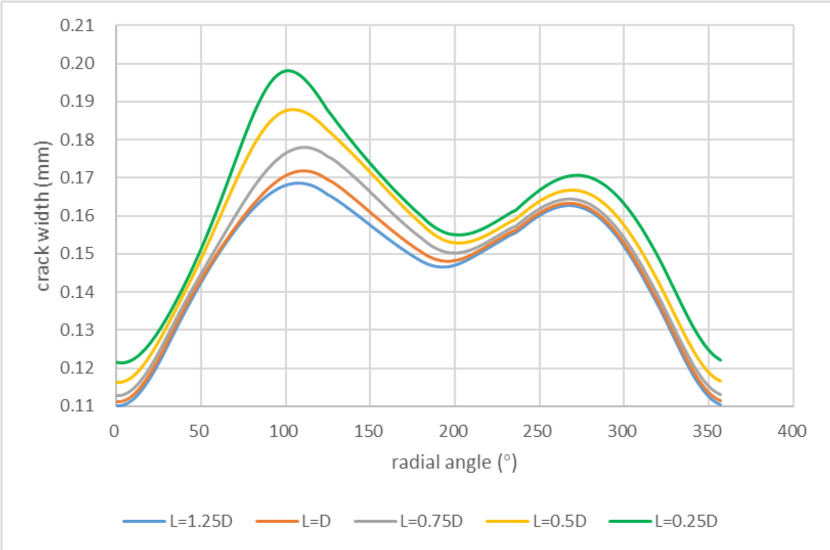


Figure 6.8: *Crack width by applying more rebar*

Table 6.4: Relation between rebar percentage and optimal tunnel spacing

L/D	1.0	0.75	0.5	0.25
max.crack width	0.2	0.2	0.2	0.2
rebar percentage	0.57%	0.58%	0.62%	0.66%

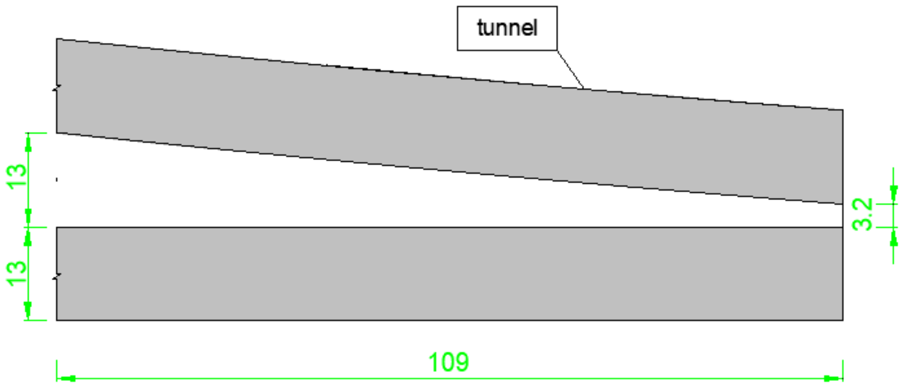


Figure 6.9: *Horizontal alignment of applying more rebar*

6.5.2. Deep soil mixing

The Deep Soil Mixing (DSM) is an in situ soil treatment technology whereby the soil is blended with cementitious and/or other materials. They are injected through hollow, rotated mixing shafts tipped with some type of cutting tool. The cemented soil material that is produced generally has a higher strength, lower permeability, and lower compressibility than the natural soil.(Bruce, 2000)

In this section, the DSM is analyzed by FEM model with support of Plaxis software to estimate the internal force and crack width when diameter of mixed soil column is 0.5 m; depth of column is 20m and column spacing is 1m (see figure 6.11). In the Plaxis 2D model, the mixed soil column is realized by changing the input soil parameters. The input mixed soil parameters is in table 6.5.

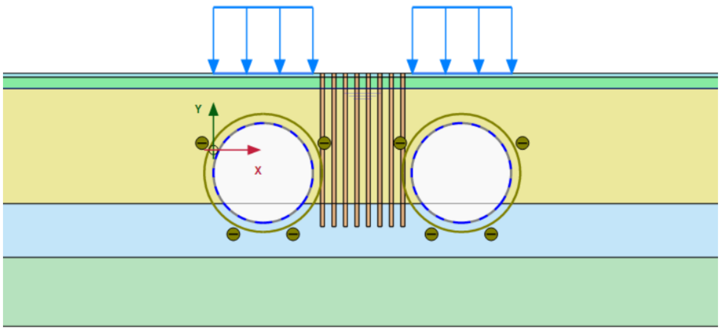
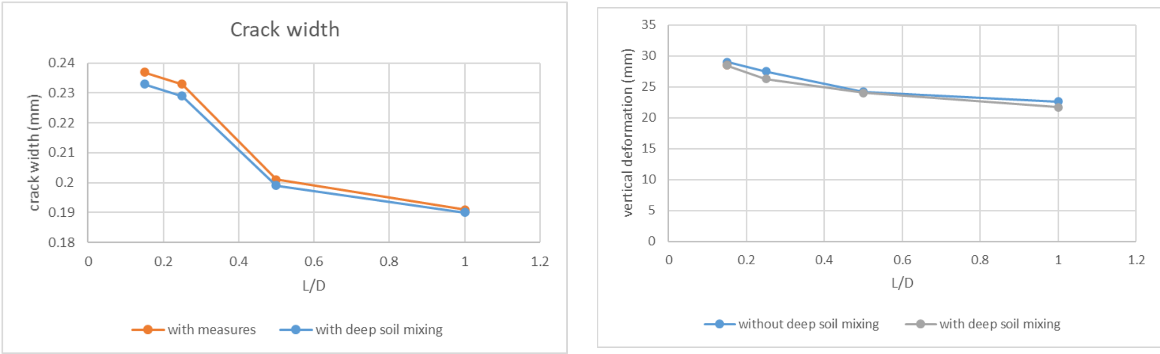


Figure 6.10: *Deep soil mixing technique*

Table 6.5: Input parameters of deep soil mixing

Name	Type	γ_{unsat} $[kN/m^3]$	γ_{sat} $[kN/m^3]$	K_x $[m/day]$	K_y $[m/day]$	E' $[kN/m^2]$	ν' $[-]$	ϕ [°]	ψ [°]	R_{inter} $[-]$	C'_{ref} $[kN/m^2]$
column	Mohr-Coulomb	18	21	2.68E-3	2.68E-3	1.0E+6	0.25	35	0	0.67	0

Figure 6.11a compares the DSM with the normal construction way. From L/D=1.0 to L/D=0.5, the maximum crack width shows a smooth increase. After that the curve shows a big jump to 0.233mm when L=0.15D. By applying the deep soil mixing technique, the maximum crack width has been reduced by 0.5%, 1.0% when L/D is 1.0 and 0.5 respectively. As for the vertical deformation, it decreases a little and it is not that obvious when L/D ranges from 1.0 to 0.15. The maximum vertical deformation decreases from 29mm to 28.5mm. From the DSM technique, it can be concluded the spacing can be optimised to 0.5D.



(a) Crack width with deep soil mixing

(b) Deformation with DSM

Figure 6.11: a) Crack width with DSM. b) Vertical deformation with DSM

Without any soil improvement, the optimal spacing is one tunnel diameter based on the tunnel safety and stability. This value (13m) is seen as the critical point along the tunnel alignment, so the length of the deep soil mixing along tunnel longitudinal axis is 76m. The area of deep soil mixing is 750m^2. (see figure 6.12)

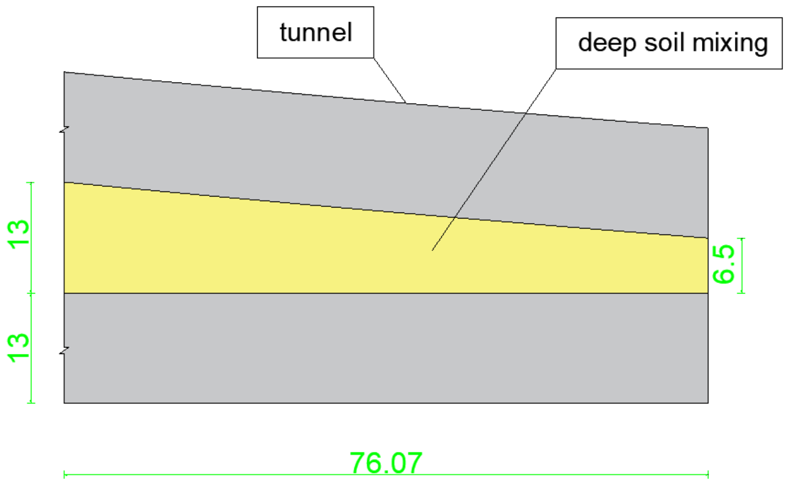


Figure 6.12: *Horizontal alignment of deep soil mixing*

6.5.3. Diaphram wall (D-wall)

A diaphragm wall is a structural concrete wall constructed in a deep trench excavation, either cast in situ or using precast concrete components. Diaphragm walls are often used on congested sites, where there is restricted headroom. Diaphragm walls are suitable for most subsoils and their installation generates only a small amount of vibration and noise, which increases their suitability for works carried out close to existing structures. (DBW, 2019)

The walls generally range in thickness from 500 - 1,500 mm and can be excavated to depths of over 50 m. Excavation is typically carried out using rope-suspended mechanical or hydraulically-operated grabs. Figure 6.13 shows the general construction of diaphragm wall. The excavation stability is maintained by the use of a drilling fluid, usually a bentonite slurry. This is a controlled mixture that has thixotropic properties, meaning that it exerts a pressure in excess of the earth and hydrostatic pressures on the sides of the excavation. The walls are constructed, using reinforced or unreinforced concrete, in discrete panel lengths generally ranging between 2.5 - 7 m.

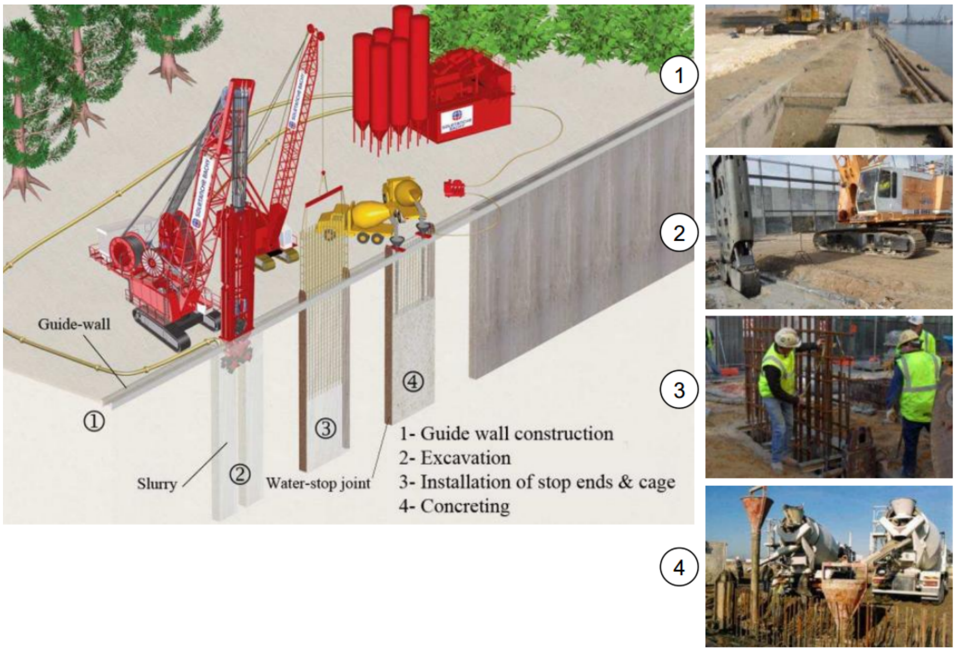


Figure 6.13: *Construction of diaphragm wall (from railsystem.net)*

In this project, diaphragm wall can not improve the soil strength and stiffness but it can prevent the soil interaction. It can reduce the first tunnel disturbance induced by the second tunnel construction, as a result the internal force of tunnel and deformation can be reduced as well. The configuration is shown in figure 6.14

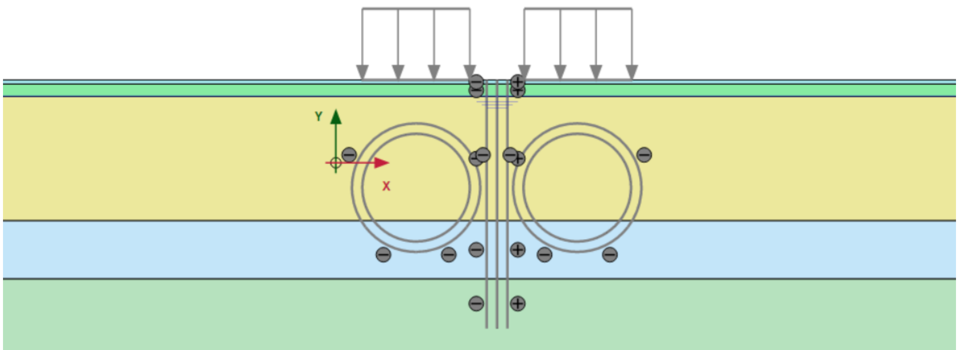


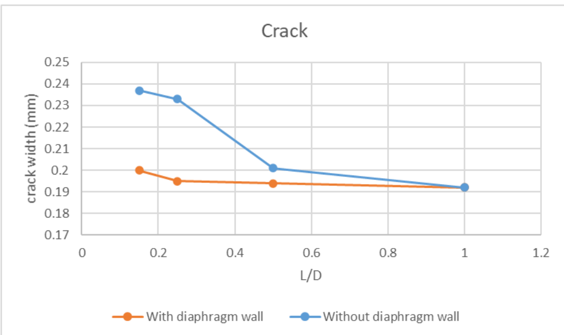
Figure 6.14: *Application of the diaphragm wall*

The diaphragm wall is installed first before constructing the two tunnels. The depth of the D-wall is 30m and the input parameters are shown in table 6.6. PLAXIS 2D simulates the whole construction process. Four different tunnel spacings are analysed (L/D = 1.0, 0.5, 0.25, 0.15).

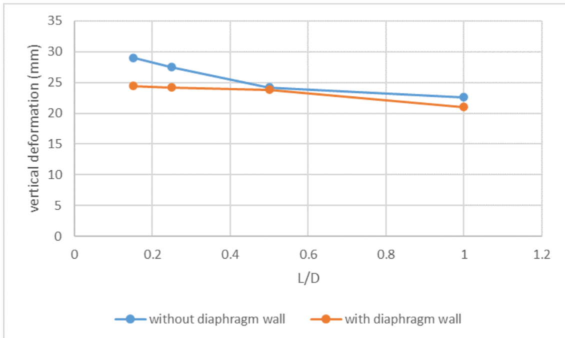
Table 6.6: Input parameters of diaphragm wall

Name	E (GPa)	b (m)	h (m)	EA (kN/m)	EI kNm^2/m	d (m)	w (kN/m/m)	ν (−)
Diaphragm wall	35	1.0	0.6	2.1E+7	6.3E+5	0.6	14.4	0.15

The result of analysis is shown in figure 6.15. With applying the diaphragm wall, the crack width increases smoothly with decreasing spacing. The effectiveness is very obvious when spacing is reduced to 0.25D and 0.15D and the reduction of crack width is 16.3% and 15.6% respectively compared with the normal construction method. Compared with the normal construction way (without diaphragm wall), the vertical deformation is almost the same when L/D decreases from 1.0 to 0.5. When L/D is lower than 0.5, the vertical deformation shows an obvious decrease from 29mm to 24.4mm. It indicates the diaphragm wall has a stronger effect on reducing soil disturbance when tunnel tubes are closer. In terms of the maximum crack width, the spacing could be optimised to 0.25D with applying the diaphragm wall.



(a) Crack width with D-wall



(b) Deformation with D-wall

Figure 6.15: a) Crack width with D-wall. b) Vertical deformation with D-wall

The length of the diaphragm wall can be determined from figure 6.7. When the spacing is D, the crack width is 0.2mm. From the curvature of horizontal alignment of the diaphragm wall in figure 6.16, the radius of curvature is 3100 which is in accordance with designed speed, the length of the diaphragm wall longitudinally is around 109m.

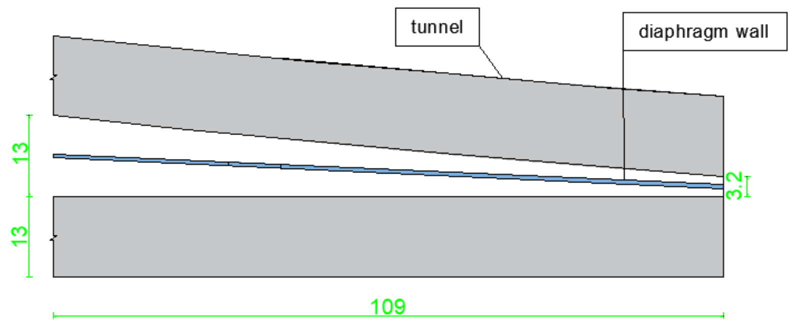


Figure 6.16: *Horizontal alignment of diaphragm wall*

6.5.4. Ground freezing technique

The principle of ground freezing is the artificial cooling of the soil to below freezing point. The pore water in the soil will act as the "binding agent". The low temperature required for freezing the soil is achieved by installing freezing pipes. The freezing agent or cold medium in the freezing pipes would circulate and extract the heat from the soil. As a result, the soil is gradually frozen with time increasing. When achieving the required thickness of the frozen soil, the excavation can start within the frozen soil.(J.Heijboer and de Linde, 2004)

In this project, FEM model with support of PLAXIS is used to simulate the frozen and thawing process during the tunnel construction. Fully coupled flow-deformation is selected as the calculation type because it is time-dependent analysis of deformation and pore water pressure. By first installing the freeze pipes in the soil, the soil freezes and becomes watertight so that tunnel construction can take place. This method of construction requires a lot of energy for cooling of the soil while groundwater flow is present, so both groundwater flow and thermal flow are used to simulate the water flow and temperature.
A groundwater flow from right to left is present, influencing the thermal behavior of the soil. First there are totally 17 freezing pipes which are installed in the middle of tunnel tubes and the length of pipes is 0.3m. Then the soil will be subjected to the low temperature (250.0k) of the freeze pipes, and once the soil has frozen for 3 days, tunnel construction can take place. During tunnel construction, the soil temperature always keeps the same at 250.3k and the thickness of frozen soil is 1.3m (see figure 6.17). After tunnel construction, the frozen soil becomes thawing and it lasts for 30 days.

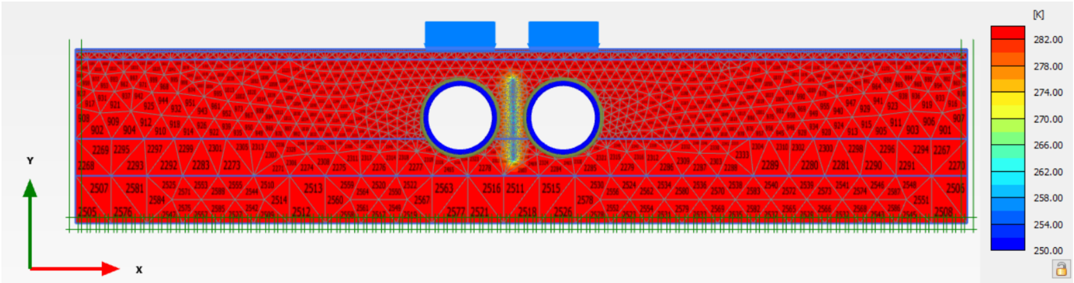
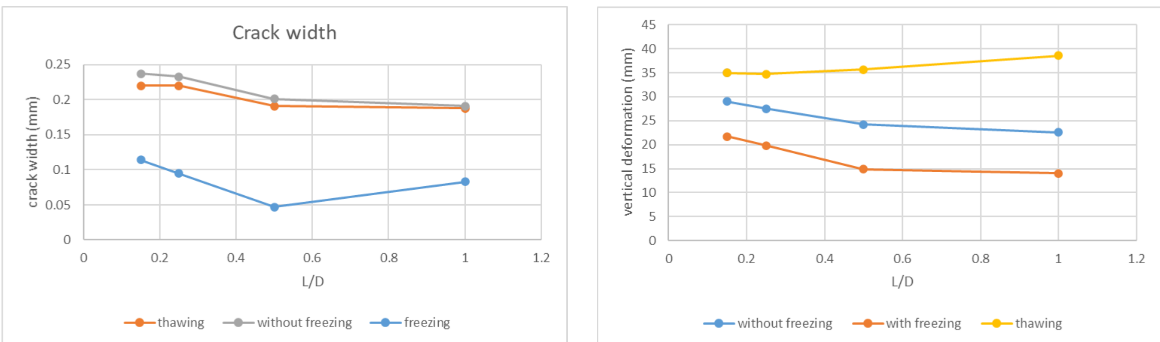


Figure 6.17: *Temperature distribution after freezing*

4 different spacing values (L/D = 1.0, 0.5, 0.25, 0.15) are simulated in order to analyse the internal force and concrete crack width. The result is shown in figure 6.18. The outcome by ground freezing technique is very obvious during construction, the maximum crack width is reduced from 0.237mm to 0.114mm. The reduction of crack width is 76.6%, which is the highest when spacing equals to 0.5D. The maximum vertical deformation has reduced from 29mm to 21.7mm. With decreasing the spacing, the vertical deformation still increases. Generally, applying freezing technique achieves a smaller deformation because freezing can strengthen the soil and lessen the soil disturbance induced by the second tunnel construction.
Theoretically, the spacing can be optimised to 0.15D (2m) by applying freezing technique because the crack width and vertical deformation are below the allowable limit. Indeed, volume decreases when the soil thaws (figure 6.18) . As a result, the vertical deformation increases to 38.6mm and the crack width increases with decreasing spacing. When the spacing is below 0.5D, crack width is higher than 0.2mm. So the spacing can be optimised to 0.5D.
According to the guideline, the horizontal alignment is shown in figure 6.19, the radius of horizontal alignment is 3100m, the length of frozen soil longitudinally is 76m.



(a) Crack width with freezing

(b) Deformation with freezing

Figure 6.18: a) Crack width with freezing. b) Vertical deformation with freezing

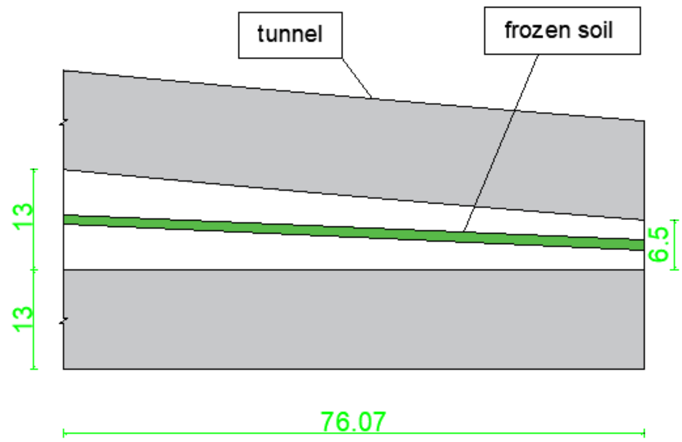


Figure 6.19: *Horizontal alignment of frozen soil*

6.5.5. Post-tension

The post-tensioned strands in cylinder shape concrete structures, such as tunnels, shafts, water tanks, concrete pipes, cylinder shape towers or any other similar structures, can be an option to improve the tunnel lining resistance to cracking. These structures adopt pre-stress in the circumference and improve their maximum resistance by increasing strength and reducing cracks in the concrete. (KHORSHIDI and BEHZAD, 2015)

The method involves inserting strand through the preliminary embedded duct/sheath in the concrete and then tensioning the strand by hydraulic jack and fastening it.

Post-tension by strands of lining reduces the steel reinforcement or fibre volume or thickness of the concrete structure which would result in reducing the manufacturing cost of them. It would reduce cracking and improve water tightness and provide smooth intrados due to eliminated bolts and joints in the tunnel segmental lining. Strands can be used in spiral, circumferential, longitudinal or horizontal directions or their combinations. But in this design project, strands are only used in circumferential direction for 2D plane analysis (See figure 6.20).

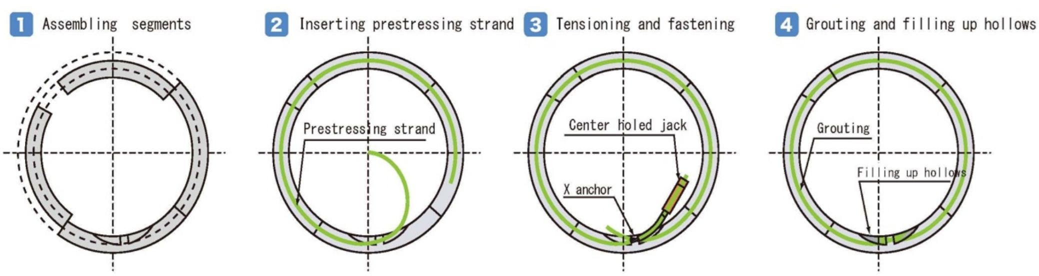


Figure 6.20: *Construction sequence of post-tensioned segmental lining*

In this model, strands are used to provide pre-stress by inserting them through the preliminary embedded duct in the concrete (precast). The area of the strand is $193mm^2$, $f_{ptk} = 1960N/mm^2$, the pre-stress that strand can provide is calculated:

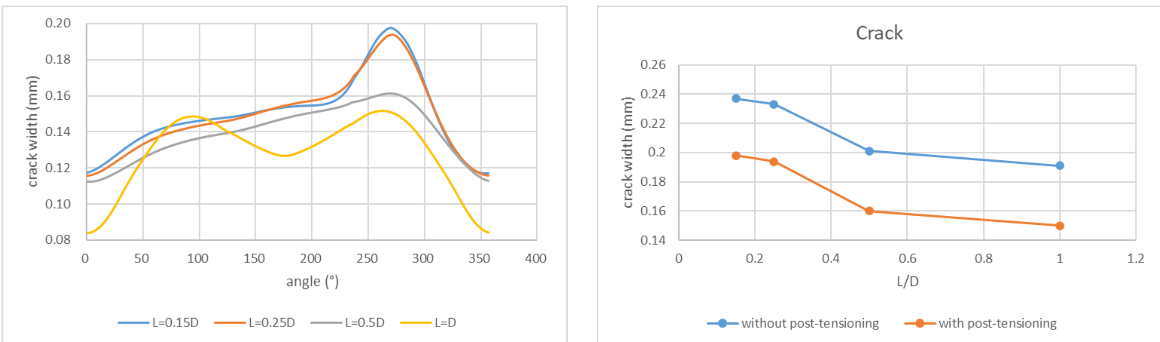
$$\sigma_{con} \leq 0.75 \cdot f_{ptk} = 0.75 \times 1960 = 1470N/mm^2 \tag{6.1}$$

$$N_p = \sigma_{con} \cdot A_p = 1470 \times 193 \times 10^{-3} = 283kN \tag{6.2}$$

Where

σ_{con}	tensioning stress of strand $[N/mm^2]$
f_{ptk}	standard value of ultimate strength of strand $[N/mm^2]$
A_p	area of the strand mm^2
N_p	tensioning force that strand can provide $[kN]$

Since it is not available to input pre-stress in the concrete model in PLAXIS 2D, for simplicity, the internal forces are first calculated without considering pre-stress, then these internal forces plus pre-stress are used to generate the final internal forces. After that the maximum cracking width can be obtained. The detailed calculation is in Appendix G. The result is shown in figure 6.21.



(a) Crack width with post-tension

(b) Crack width comparison with and without post-tension

Figure 6.21: Crack width after applying post-tension

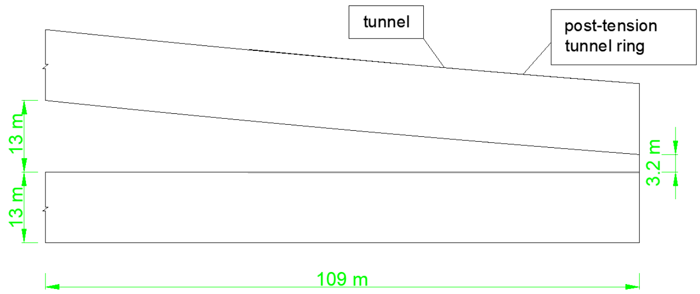


Figure 6.22: *Horizontal alignment of post-tension*

From the result, it can be seen after applying post-tension, the crack reduces by 21% compared without applying post-tension. Also, crack increases with decreasing tunnel spacing from D to 0.15D. When spacing is 0.15D, crack is almost equal to 0.2mm. For safety consideration, the spacing should be optimised to 0.25D (3.2m) . The horizontal alignment is in figure 6.22.

6.6. Evaluation

In the last section, different measures have been taken to optimize the spacing. And each measure has achieved a optimum spacing. In this section, these measures are going to be evaluated from aspects of effectiveness, risk and cost.

6.6.1. Effectiveness

Effectiveness is expressed by crack reduction compared with the crack before taking any measures. It is an important criteria to evaluate these techniques. Among the five techniques, the most effective measure is post-tension which can reduce the crack width by 17%. The second and the third effective techniques are installing diaphragm wall and adding more reinforcement. They can decrease the crack width by 16.3% and 14%, respectively. The fourth and the least effective techniques are soil freezing and deep soil mixing, the crack width is reduced by 5% and 1%, respectively. The detailed effectiveness is show in figure 6.23.

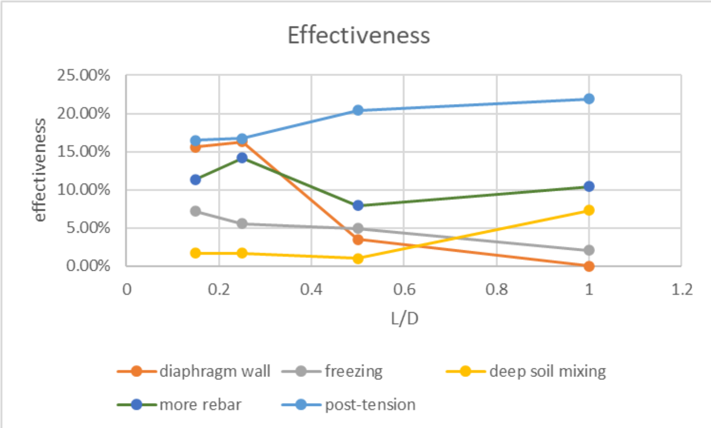


Figure 6.23: *Effectiveness of reducing crack width*

6.6.2. Risk

The most risky technique is soil freezing because the volume would expand with freezing the soil. One of the reasons is the volume of water expands when transforming to ice. The expansion of soil could influence the first tunnel at internal force and deformation. Furthermore, a volume decrease can be seen when the soil thaws again. This may lead to (non- uniform) settlement on the second tunnel. From freezing to thawing, the crack width increases by 75.3%, from 0.047mm to 0.191mm. The vertical deformation increases by 58.2%, from 14.9 to 35.7mm. (see figure 6.18)

The second risky technique is diaphragm wall because the wall can deflect in the horizontal direction. During the construction of tunnels, the soil close to the wall would be disturbed and thus soil stress changes. As a result the wall deforms 5.1mm horizontally and causes the deflection. The smaller the spacing, the larger the deflection. The deflection of the wall is shown in figure 6.24. Besides, the installation of a diaphragm wall will cause volume loss as well. Since the diaphragm wall is installed before tunnel construction, this risk can be reduced by putting an extra soil on the top to compensate.

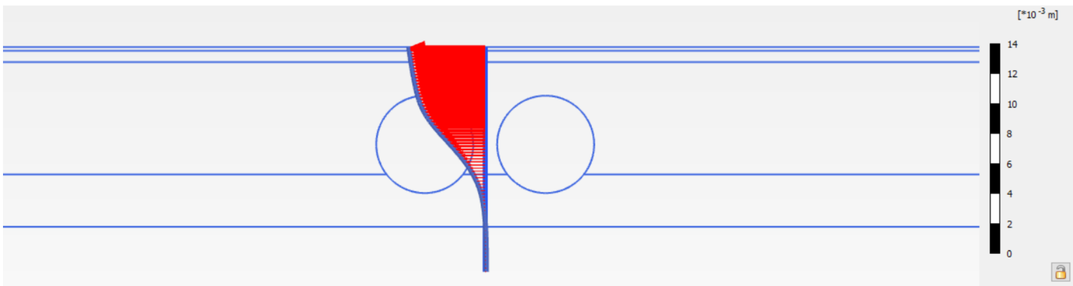


Figure 6.24: *Deflection of diaphragm wall*

For the deep soil mixing, it is implemented before constructing tunnels. And it aims at improving the soil stiffness and strength so it does not have an adverse effect on both tunnels. The risky point is whether the soil can be improved equally. This depends on the implementation accuracy.

The least risky measures are adding more reinforcement and the post-tension, they do not change the soil conditions and only change the segments. The risky point for applying more rebar is the logistics of assembling segments because it is easy to put the more reinforced segments in the wrong positions. Another risk may be the non-uniform deformation of the tunnel ring due to non-uniform designed internal force. For applying post-tension, there is pre-stress loss because of friction force in the duct and shrinkage of strands, which will reduce the pre-stress and the stiffness of the segments. As a result, the crack and deformation will increase.

6.6.3. Cost

It is not easy to give a detailed cost of these five techniques. Herein, a cost estimation is made based on the experience. The construction costs for a launch shaft are approximately Euro 150000 per running meter. Table 6.7 shows the cost of shaft construction, length of applying techniques and optimised spacing under different techniques. The cost does not include applying techniques.

The table 6.7 shows that adding more rebar, the diaphragm wall and the post-tension are the cheapest measures to construct the tunnel because they can optimise the spacing to 0.25D (3.2m) which is the shortest spacing among these techniques. As for the cost of implementing these techniques, post-tension and adding more rebar are cheaper than diaphragm wall.

Table 6.7: Cost under different techniques

	More reinforcement	Deep soil mixing	Diaphragm wall	Freezing	Post-tension
Tunnel spacing (m)	6.5	6.5	3.2	6.5	3.2
Applied length (m)	109	76	109	76	109
Cost (€) excl. techniques	4980000	5475000	4980000	5475000	4980000

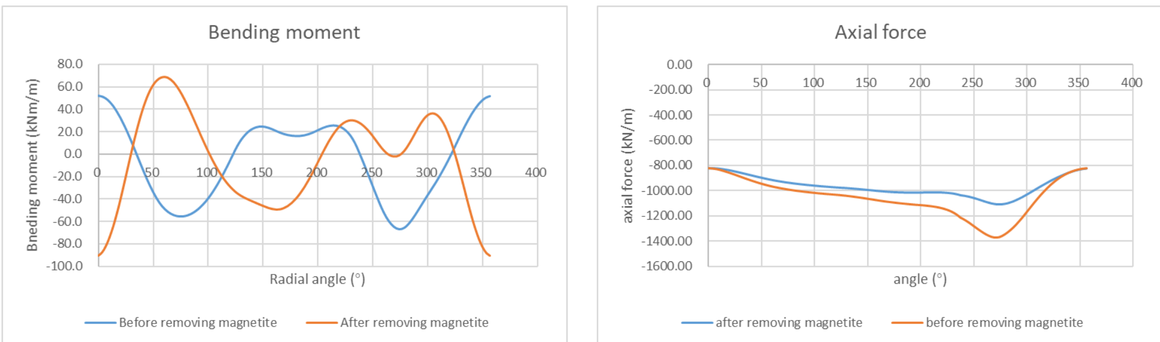
6.6.4. Conclusion

From the analysis above, post-tension is the most effective measure to optimise the spacing, at the same time it is the cheapest measure to construct the exit shaft. Although the length of applying post-tension is around 30m longer, the cost is still relatively cheaper than the cost of applying other techniques. Finally post-tension is chosen as the optimum measure and optimised spacing is 0.25D (3.2m).

6.7. Removing magnetite

The magnetite is mined from the nature (Kiruna, Northern Sweden) which has been proved it is harmless to the environment. It is put on the surface ground to prevent blow-out. While it is better to remove it after finishing the tunnel construction considering the reuse of material and environmental effect in the long term. Due to the decrease of the overburden, it is necessary to check the stability of the tunnels because it may influence internal force and crack of the tunnels.

Figure 6.25 shows the change of the bending moment and axial force. It can be seen the bending moment goes the opposite direction comparing with the original one, which the compression zone becomes the tensile zone. The absolute value does not show a big change at the same radial angle. The axial force decreases by 10% from 1200kN to 1100kN. From the calculation of the allowable bending moment, it is 1501kNm/m which is far higher than bending moment (90.7kNm/m) generated from Plaxis. From the change of axial force, the crack width can be calculated and the result is shown in figure 6.26. It can be seen the crack width decreases from 0.192mm to 0.16mm after removing the magnetite. It is still within the allowable value (0.2mm). So after removing the magnetite on the ground surface, the tunnel is safe and stable.



(a) Bending moment of right tunnel after removing magnetite

(b) Axial force of right tunnel after removing magnetite

Figure 6.25: Internal force after removing magnetite

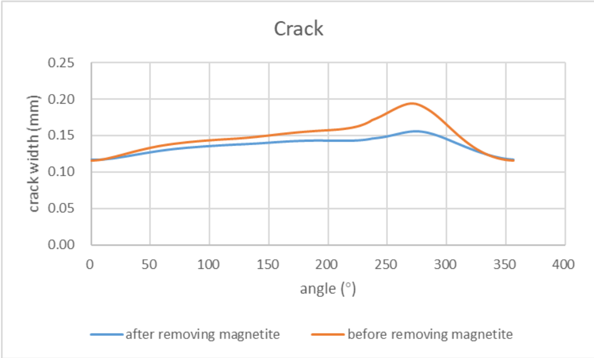


Figure 6.26: *Crack after removing magnetite on the surface ground*

6.8. Settlement

The vertical deformation at eastern ramp has been considered in previous section. Meanwhile the settlement at brick factory and dike need to be checked as well because there are buildings and brick storage rooms at brick factory (see figure 6.27). Also the dike should be protected from volume loss due to tunnel construction. So both locations have a stricter tolerance for the contractor / TBM operator to adhere to in terms of boring quality control. The vertical deformation should be controlled within 25mm.

6.8.1. Settlement at brick factory and dike

The brick factory and dike are located at western bank of Pannerdensch canal. The soil covers at these two positions are 16.7m (C/D=1.28) and 21.4m (C/D=1.65), respectively. The soil type ranges from clay to medium sand and coarse sand. The spacing between tunnel tubes is 13m (L=D). PLAXIS 2D is used to simulate the settlement at both locations. In order to avoid the damage on the buildings and dike due to the tunnel construction, the settlement at the two positions has a strict limit which should be lower than 25mm.

When the soil loss is 0.5%, the settlement is 31.1mm and 31.5mm at brick factory and dike, respectively. The values are higher than 25mm, which are not allowed at these two positions. In order to lower it, volume loss can be reduced to a certain value by taking some measures, like improving the strength of grouting materials or selecting the grouting material which can reach strength requirement in a shorter period. (Vu et al., 2016) shows some case studies in which different cover-to-diameter ratios (C/D) have different volume losses in sand projects. In the Second Heinenoord Tunnel , the volume loss is 0.21% when C/D is 1.25. In the Sophia Railway Tunnel, the volume loss can reach 0.25% when C/D is 1.6. In this project, by changing the input parameter of volume loss in PLAXIS 2D, it is found when the volume loss is reduced from 0.5% to 0.38% and 0.35% at brick factory and dike, the settlement is reduced to 24.7mm and 24mm, respectively (see figure 6.28

and 6.29).

Figure 6.27: *Overview of brick factory and dike*

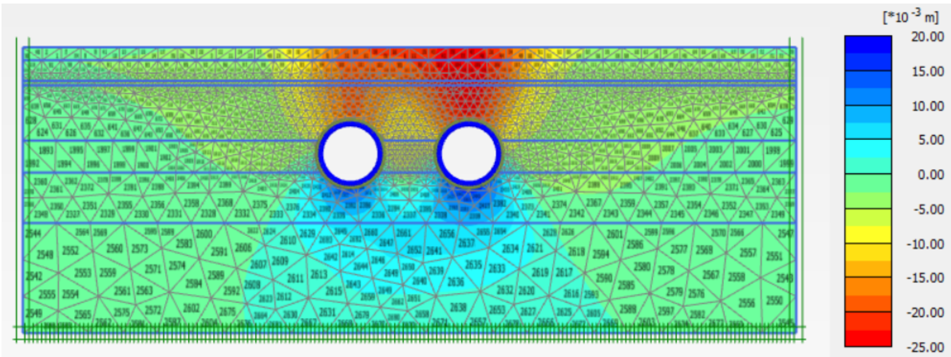


Figure 6.28: *Vertical deformation at brick factory*

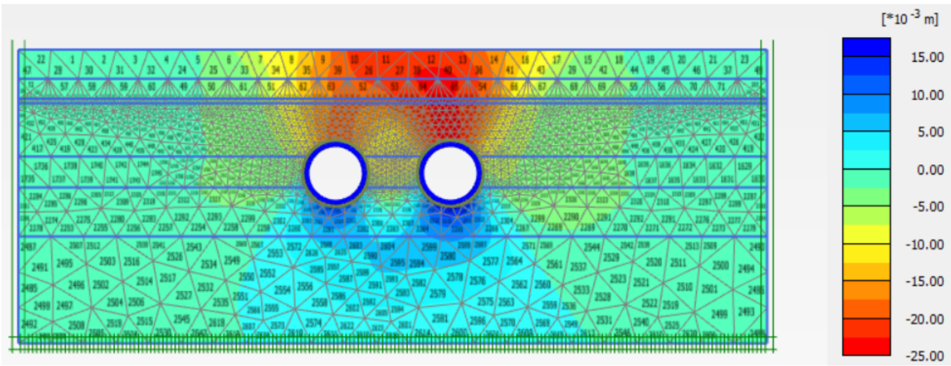


Figure 6.29: *Vertical deformation at eastern dike*

7

Comparison and discussion between tunnel and bridge

In the past few years, there are debates on whether a bridge or a tunnel is adopted to cross the Pannerdensch canal. Finally the government has decided to adopt the bridge because it is € 210 million cheaper than the tunnels. The opponents think the effects of the move through bridge on nature in the Gelderse Poort have not been sufficiently investigated. This master thesis gives a detailed design of the tunnel in the above chapters. This chapter focuses on the comparison between a tunnel and a bridge. First it gives an introduction of selected tunnel and bridge. Then both tunnel and bridge are compared from aspect of nature, landscape, social aspect, sound and cost. Finally some potential concepts will be discussed by adopting social cost-benefit analysis if technical problems can be solved.

7.1. Brief introduction of tunnel and bridge

Having carried out a relatively detailed design of the tunnel, figures 7.2, 7.3, 7.4 below depict the cross section, horizontal and vertical alignments of the tunnel and the bridge. In this section, an introduction of the tunnel and the bridge is firstly described. Then they are compared from aspects of spatial structure, sound, nature, social aspect, landscape, cultural history and archeology and cost.

In the tunnel option, it is 2×2 with emergency lane and the width of the road for passing vehicles is 10.7m, width of the lane is 3.3m. The length of the tunnel is around 2772m of which bored section is 1570m. The burial depth at shafts is 6.5m which is half diameter of tunnel. With putting a layer of magnetite on the river bed, the burial depth is reduced to 13.4m at the canal. The western shaft is located 20m away from Rijndijk. The eastern shaft is located in the sandpit where the pit is filled up with sand in order to have sufficient overburden. There are 5 cross passages (every 250m) along the bored section.

In the bridge option, a construction period is 4 years. The year 2014 is used as the start of construction. The opening up of the infrastructure will then take place on 01-01-2018. It is 2 × 2 with emergency lane as well. The pavement width is 12.5m and width of each lane is 3.5m. The length of the bridge is around 3000m. The height of the main span is based on a design highest water level of + 15.40m NAP and a minimum clear passage height of 9.10m. A span length of 60m has also been used for the bridge. A greater span length is created at the location of the channel.

7.2. Comparable aspects

Nature

A distinction is made for the criteria influencing the Natura 2000 (Gelderse Poort) and influencing National Ecological Network (river area). As for the tunnel, there is limited space requirement in the Natura 2000 area of the Gelderse Poort and the river area, and the noise load has also been considerably reduced. As for the bridge which is in the open air, the construction process will affect the nature reserve and the abutments of the bridge have impacts on the river area. Also, noise load is still a nuisance for the residents living nearby.

Landscape, cultural history and archeology

There is influence on the landscape value. Through the use of a tunnel, no intersections take place in the national landscape "Gelderse Poort" and the provincially valuable landscape "Ooipolder and Rijnstrangen". The overall effect is thereby reduced. A tunnel largely keeps the value of the landscape at the location of the Pannerdensch Canal intact. The bridge has serious effects on the land use within the North and South influencing main ecological structure and meadow bird and geese conservation area. In addition, barrier effect occurs due to intersection of the geese and meadow bird area.

Social aspect

Social aspect considers the visual nuisance. A tunnel reduces the number of severely hindered houses. The number of houses hindered by tunnel is 90-100. While it is lower than that by bridge which is 100-110

Sound

Sound mainly considers the noise-affected surface. As the tunnel goes below the ground level along the alignment, there is less noise disturbance to the environment and the residential area near the open ramp. While, bridge goes above the ground level and the impact of noise on the environment is still there. Also, it influences the residents living on the either side of the bridge.

Cost

In the previous chapters, cost of tunnel, which includes cost of different structural elements, is roughly estimated. The total cost of the tunnel is around €375,056,000. With regard to the bridge, there are three methods to estimate the cost of bridge construction. The first method shows the detailed implementation of road cover (asphalt) (Michael et al., 2015). The unit price is around € $16.5/m^2$. The construction price of bridge can be roughly estimated by subtracting the cost of road cover from the total project cost: (total cost of road cover + bridge) - (cost of road cover) = (cost of bridge construction). This method ignores the local road crossings with small viaducts and road embankments for the road in other parts of the project. The cost of the bridge construction is €744,040,200. Detailed calculation is shown below. The cost of a bridge is €369 million more expensive than a tunnel. The first method overestimates the cost of bridge construction because it excludes the cost of embankments, viaduct and deepened locations in other parts of the project.

Length of project: $12km$
Width of road: $30.1m$
Total area of road cover: $12000 \times 30.1 = 361200m^2$
Unit price of road cover (asphalt): €$16.5/m^2$
Total cost of road cover (€): $361200 \times 16.5 = 5,959,800$
Total cost of project (€): £$750,000,000$
Total cost of bridge construction (€): $750,000,000 - 5,959,800 = 744,040,200$

The second method to calculate the cost is the proportion of the length of bridge to the total length of the project. The length of the bridge is 3km and the total length of the project is 12km. Then the cost of bridge construction is around 187.5 million euros. This method underestimates the bridge construction because the unit price of constructing a bridge is far more expensive than constructing road.

The third method can be formulated as: cost of bridge construction = total project cost - road embankment - road cover - viaducts - diver - tunnelbak. Let's assume the height of embankment is 3m, the width of road surface is 30.1m, slope ratio is 1:1.5. The total volume of material used for the embankment is 635,000m^3. Assume delivering the sand on site costs €$10/m^3$, the unit price of constructing viaduct and diver is €26,200/m, noise barrier is €765/m. The unit price of implementing these structures is referred from Bouwkostenkompas (Bouwkostenkompas, 2017).

Embankment: €$10/m^3$, volume 635,000m^3, cost € 6.35mln
Road cover: €$16.5/m^2$, area 361,200m^2, cost € 6.0mln
Viaduct+diver: €26,136/m, length 1357m, cost € 35.5mln
Tunnelbak: two tunnelbaks, one part is 1530m, the other is 565m, the cost is estimated according to the unit price of cut-and-cover tunnel € 4125/m2, area 63,060m^2, cost € 260mln
Noise barrier: €765/m, length 1357m, cost € 1.0 mln
Cost of bridge construction = 750 - 6.35 - 6.0 - 35.5 - 260 - 1.0 = € 441.15mln

The cost estimation of three methods ranges a lot from 187.5mln to 744mln. The third method is more fair and reliable because it includes the cost of road embankment, tunnelbak, viaduct and diver. But the cost estimation of bridge construction of the third method is still a bit higher because it does not include all structural elements such as foot bridge, bicycle tunnel.

7.2.1. Conclusion

In the comparison with bridge, the double tube bored tunnel with emergency lane with cut-and-cover has obvious advantages on aspects of nature protection, landscape, cultural history and archaeology, social aspect and noise-affected surface. It can not be concluded if tunnel is cheaper because the cost of bridge has a wide range from 187.5 to 744 million euros and the cost of bored tunnel is 375 million euros. Even if there is a more reliable cost estimation €441.15mln of bridge construction but it is still an upper estimation.

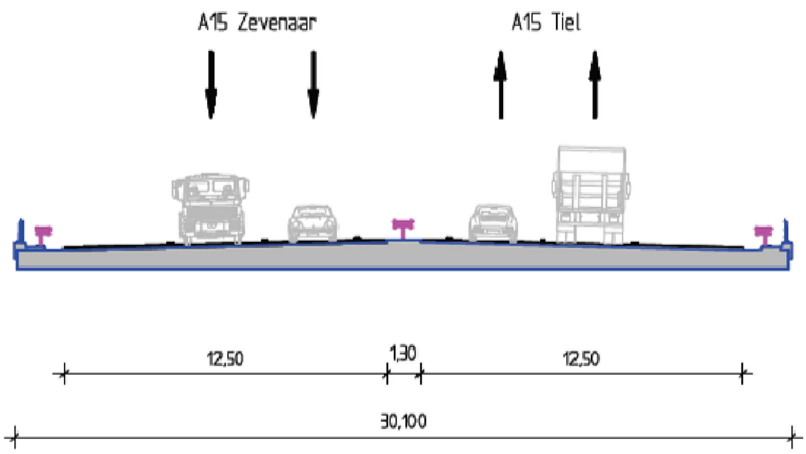


Figure 7.1: *New distribution of the route (Ontwerp Tracébesluit A12/A15 Ressen - Oudbroeken (ViA15) Deelrapport verkeer)*

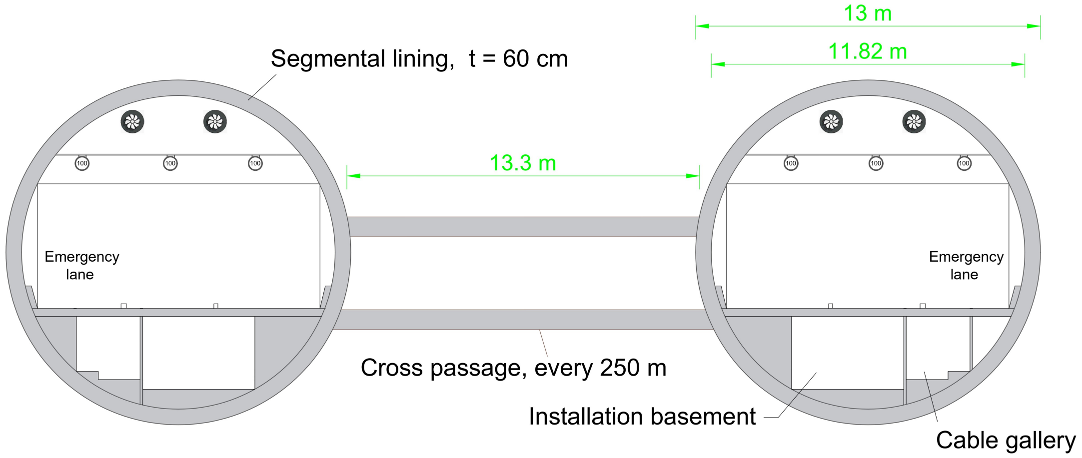
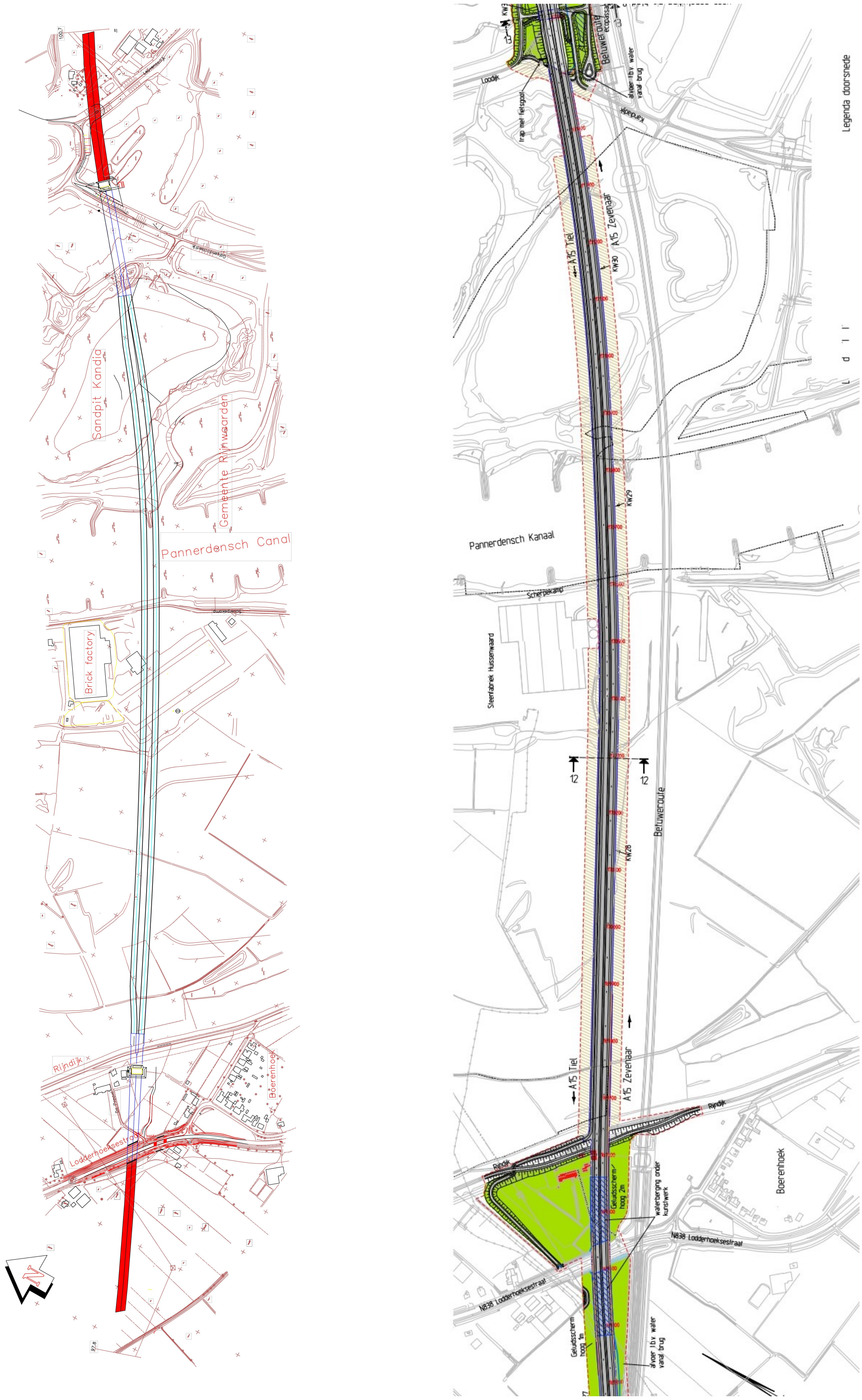
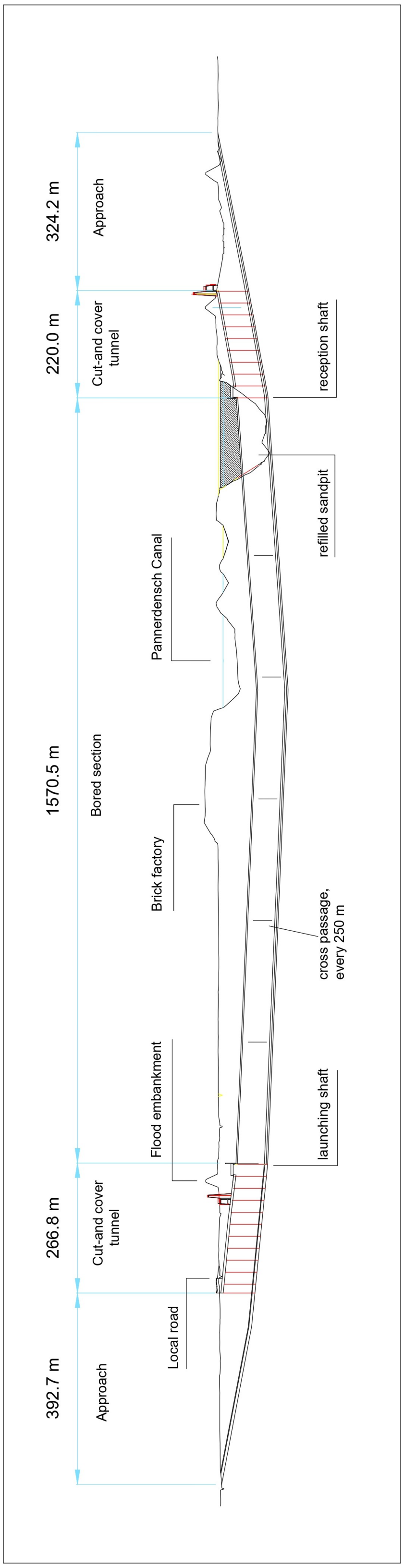


Figure 7.2: *Cross section of double-tube bored tunnels with emergency lane*

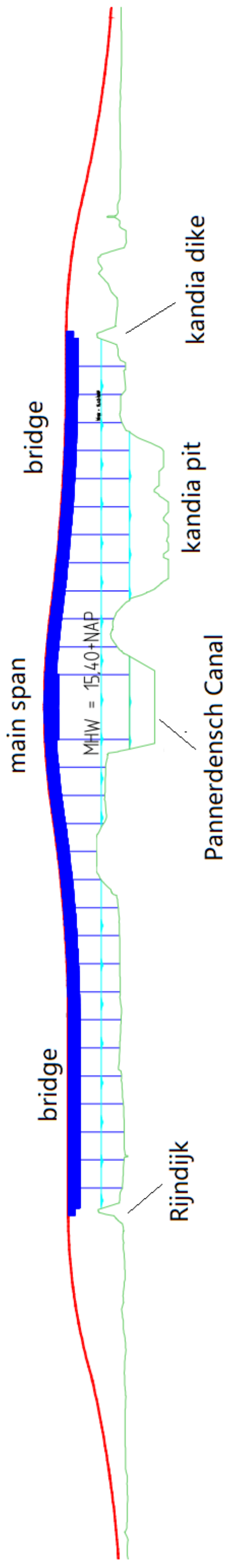


(a) (b)

Figure 7.3: a) Horizontal alignment of double-tube bored tunnels b) Horizontal alignment of bridge



(a)



(b)

Figure 7.4: a) Longitudinal alignment of double-tube bored tunnels. b) Longitudinal alignment of bridge

7.3. Discussion

Chapter 4 evaluates those concepts by using Multi Criteria Analysis (MCA) and finally selects the best concept - double tube bored tunnel with cut-and-cover section with emergency lane. All criteria can be quantified by MCA but they are not monetised. If some technical problems can be solved, it is meaningful to evaluate these alternatives by adopting social cost-benefit analysis.

This section will estimate the cost of uncertainties of all concepts and compare the selected bored tunnel with other tunnel concepts. Some critical technical problems need to be solved and the cost should be estimated in the discussion .

For example, the first problem of constructing an immersed tunnel is de-watering, the water level in the dock needs to be lowered from NAP+8.0m to NAP -2.5m in an area of $59500m^2$ ($170m \times 350m$). It would be difficult to de-water the dock because the dock is located in a floodplain and it has a high fluctuating water level. So deep well drainage is applied every 10 m around the excavation. Wells will be 20 m deep and the purchase costs Euro 5000/well. Installation is Euro 500/well, demobilization is Euro 500/well. Since the license to access the Plaxis is expired, it is hard to calculate the amount of water to be extracted, Now it is assumed 100 m^3/minute. Provincial tax is assumed Euro 0,25/m^3. This means for a year the taxes are $365 \times 100 \times 60 \times 24 \times 0.25 = 13$ Million Euro.

The number of drainage wells: $17 \times 35 = 595$
The cost of purchasing wells: $595 \times 5000 = 2,975,000$ euros
The cost of installation and demobilization is: $595 \times 1000 = 595000$ euros
The total cost of de-watering is: 16.6 million euros

The second problem is that the immersed tunnel will go through the contamination area under the brick factory, which means the contaminated area must be re-mediated before immersion. It is assumed the whole area is contaminated, then the volume of contamination is $72080m^3$. 100 euro/m^3 is used for re-mediating the contamination. The cost is 7,208,000 euros. The third problem is part of brick factory will be removed. It is assumed Euro 500/m^2, the area is $200 \times 50 = 10000m^2$, then the cost of buying land is Euro 5,000,000. So the total cost of solving these three technical problem is Euro 28.8 million.

However, the construction time of an immersed tunnel is 1.5 years shorter than the selected bored tunnel. This saving period could create direct and indirect monetary value. A social cost-benefit analysis (SCBA) is adopted in order to express this monetary value clearly.

The social cost-benefit analysis (SCBA) is one of the tools available for policymakers to make substantial proposals for tackling societal challenges. A SCBA provides insight into the positive and negative effects of policy options on society. It assists policymakers to improve, compare and choose between different solution directions.

A cost-benefit analysis is an analysis of various policy measures or alternatives in which the relevant social effects of these alternatives are being brought. These effects are quantified and monetized as much as possible so that they can be added together and are easily compared. Of all effects that can be expressed in monetary value, a balance of the costs and benefits is determined.

The social effects of alternatives bored tunnel and immersed tunnel in this project relates to accessibility, safety, living environment and indirect impacts. The detailed calculation of monetary values are referred from Ministry of Infrastructure and Water Management (ViA15, 2011). Since the other tunnel concepts can be completed earlier than the double tube bored tunnel with emergency lane with cut-and-cover, which means this highway will be open earlier. This saving period has direct and indirect monetary values which are shown in table 7.1 and table 7.2. The alternative double tube bored tunnel with emergency lane with cut-and-cover is the reference case because it needs the longest construction period. All the other tunnel concepts have different costs and benefits based on a shorter construction period. Both tables indicate that constructing an immersed tunnel would bring the highest monetary values around 30.97 million euros.

Table 7.1: Net present values in millions of euros as of January 1, 2011

Alternatives	Construction method	Construction period (month)	Travel time	Travel time reliability	Ride cost	Nuisance during construction	Road safety	Air quality
double tube bored tunnel with emergency lane	cut and cover	57	0	0	0	0	0	0
	open approach	49	10.85	2.71	0.76	0	0	-0.3
double tube bored tunnel without emergency lane	cut and cover	56	1.3	0.32	0.09	0	0/-	-0.036
	open approach	47	13.4	3.36	0.95	0	0/-	-0.37
single tube bored tunnel with emergency lane	cut and cover	50	9.4	2.35	0.66	0	0	-0.26
	open approach	48	9.4	2.35	0.66	0	0	-0.26
immersed tunnel	without dock	46	14.9	3.73	1.05	-	0	-0.41
immersed tunnel	with dock	39	24.3	6.1	1.7	–	0	-0.67
bridge	-	48	12.15	3.04	0.86	–	0	-0.338

Table 7.2: Net present values in millions of euros as of January 1, 2011

Alternatives	Construction method	Noise pollution	Nature	Landscape	Archaeological value	Indirect value	Benefit	Cost
double tube bored tunnel with emergency lane	cut and cover	0	0	0	0	0	0	370,076,249
	open approach	-0.07	-0.13	0	0	++	13.82	428,919,323
double tube bored tunnel without emergency lane	cut and cover	-0.008	-0.015	0	0	++	1.65	365,915,038
	open approach	-0.09	-0.163	0	0	++	17.08	388,783,163
single tube bored tunnel with emergency lane	cut and cover	-0.064	-0.114	0	0	++	11.97	390,350,308
	open approach	-0.064	-0.114	0	0	++	11.97	430,788,834
immersed tunnel	without dock	-0.1	-0.18	0	0	++	19	323,852,443
immersed tunnel	with dock	-0.17	-0.29	-/0	-/0	++	30.97	329,927,688
bridge	-	-0.083	-0.147	-/0	0	++	16.16	171,340,000 - 727,840,000

Table 7.3: Cost range from low to high

Alternatives	Cost (€)	Rank
immersed tunnel without dock	323,852,443	1
immersed tunnel with dock	329,927,688	2
double tube bored tunnel without emergency lane with cut-and-cover	365,915,038	3
double tube bored tunnel with emergency lane with cut-and-cover	370,076,249	4
double tube bored tunnel without emergency lane with open approach	388,783,163	5
single tube bored tunnel with emergency lane with cut-and-cover	390,350,308	6
double tube bored tunnel with emergency lane with open approach	428,919,323	7
single tube bored tunnel with emergency lane with open approach	430,788,834	8
bridge	171,340,000 - 727,840,000	9

Travel time benefits for commuting, business traffic, freight traffic and other traffic. Freight traffic to and from the Germany has great benefits because of the shorter route. Also the congestion in Arnhem-Nijmegen has decreased a lot. In the study area (see figure 7.5), traffic model has been used to simulate the traffic performance in the year 2025. Zero alternative means it keeps the original route without any extension and it is the reference case. Bridge or tunnel means there is extension from A15 to A12. Table 7.4 shows the total loss time (delay on the road network) will decrease by 18% from 52000 hours to 42000 hours for 1 year period. Weighted loss time (per km travelled) will decrease from 0.0043 hour to 0.0032 hour. If an immersed tunnel is constructed, this saving travel time would save 24.3 million euros.

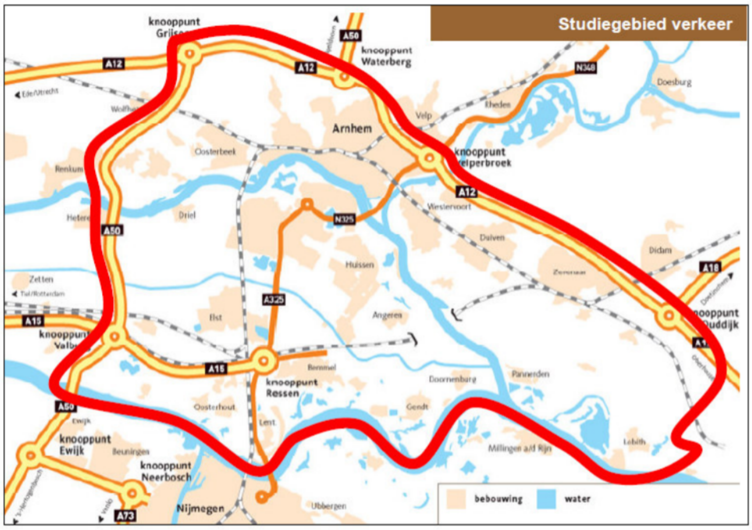


Figure 7.5: *Study area in the traffic model*

Table 7.4: Traffic performance calculated for year 2025

Main road network	zero alternative (reference case)	tunnel/bridge	increase (+)/decrease (-) percentage (%)
Vehicle kilometers	8,729,000	9,863,000	13
Loss time (hours)	27,000	19,000	-29
Weighted loss time (hours per km travelled)	0.0031	0.0020	-37
Underlying road network			
Vehicle kilometers	3,394,000	3,292,180	-3
Loss time (hours)	25,000	23,000	-7
Weighted loss time (hours per km travelled)	0.0075	0.0062	-18
Total road network			
Vehicle kilometers	12,123,000	13,214,000	9
Loss time (hours)	52,000	42,000	-18
Weighted loss time (hours per km travelled)	0.0043	0.0032	-25

Travel time reliability concerns the relative certainty of the travel time (or the probability of arrive on time). After all, certainty has a value and unreliability costs. Travel time reliability would save 6.1 million euros.

Ride costs are variable costs that motorists incur for driving distance. The distances (in kilometers per year) are then multiplied by the variable journey costs per kilometer. Ride cost would save 1.7 million euros because of a shorter route and earlier opening period.

Nuisance during the immersed tunnel construction has a negative impact compared with the bored tunnel because immersed tunnel concerns large excavations and dredging. It would remove part of the brick factory and block the canal during immersion phase. Road safety has a neutral impact for both immersed tunnel and bored tunnel.

Living environment includes air quality, noise pollution, nature, landscape and archaeological value. All of these effects related to living environment have negative monetary value because an earlier period of the highway tunnel would allow vehicles to emit more CO_2, NO_x and PM10. Thus it will bring more noise pollution. The influence on the nature is the change of biodiversity.

Table 7.5: Emisssions compared to zero alternative

	zero alternative (reference case)	tunnel/bridge	cost (millions of euros)
Change in CO2 emissions (tons/year)	0	410	-0.34
Change in NOx emissions (kg/year)	0	600	-0.08
Change in PM10 (kg/year)	0	60	-0.03
Total	0	410.66	-0.45

Table 7.6: Change in the number of people who are annoyed by noise per noise category compared to zero alternative

Category	tunnel/bridge	cost (millions of euros)
49-53dB	569	-0.005
54-58dB	-154	0.005
59-63dB	315	-0.015
64-69dB	1355	-0.09
>69dB	82	-0.006
Total	2167	-0.11

Indirect effects are effects that do not occur directly as a result of the project. They are effects that arise as a result of direct effects. Indirect value includes the corridor function, job market, business location and logistics position. The alternative immersed tunnel with dock has a strong positive impact among all tunnel concepts because it saves the longest time period (1.5 years). It means the earlier the project can be completed, the more indirect monetary value it will get.

7.4. Conclusion

From the Multi Criteria Analysis (MCA) in chapter 4, double tube bored tunnel with emergency lane with cut-and-cover is the best tunnel concept. However, in the social cost-benefit analysis (SCBA), double tube bored tunnel with emergency lane with cut-and-cover is not the best tunnel concept because it takes the longest time to complete the project. This saving period could create monetary value for other tunnel concepts especially the immersed tunnel without dock. Having taken the cost of solving technical problems into account, like de-watering, re-mediation of contamination and removing part of brick factory, the immersed tunnel without dock is 323.8 million euros and is cheaper than other tunnel concepts. But it still can not conclude whether the immersed tunnel without dock is cheaper than a bridge construction or not. Even if the cost of bridge construction is estimated €441.15mln but it is an upper estimation.

8

Conclusion and Recommendation

This chapter will first draw an conclusion of each design phase. Then it is the recommendation on each phase because this report is completed by means of some assumptions and simplifications.

8.1. Conclusions

This section gives conclusions on each design phase which basically follows the structure of the report.

Design definition

Design definition includes requirements and boundary conditions. The requirements combine the design requirements in the original report and the guideline NOA 2007. It considers both an immersed tunnel and a bored tunnel. It includes the number of lanes, width of each lane, design speed and some detailed requirements. The design speed is 100km/h, number of lanes is normally 2×2 including emergency lane. 2×2 without emergency lane is also taken into account but additional lateral clearance should be considered. Both of them are compared at the same level by a Multi Criteria Analysis in chapter 4.

The boundary conditions are also analysed from aspects of water level and geology. Since the water level fluctuates a lot and these fluctuations have an instant effect at a great distance from the river, so the tunnel goes across the floodplain area. As for geology, stiff to very stiff organic clay has been encountered in the investigation of railway tunnel, there is probably stiff clay along the bored tunnel because both tunnels are very close.

Design concepts

Based on the design definition and requirements, different concepts are developed. These concepts include a bored and an immersed tunnel. For a bored tunnel, there are two alignments: with or without cut-and-cover section. These concepts are assessed from aspects of alignment, realistic existings and influence on shipping and railway tunnel. From the assessment of design concepts, it turns out not all concepts are feasible. The remaining concepts are evaluated by a Multi Criteria Analysis (MCA) and finally double tube bored tunnel with emergency lane with cut-and-cover section is chosen as the best design concept.

In-depth design

After selecting the best design concept, the settlement and thrust force are calculated at shafts and canal positions. Tunnel lining and cross passages are designed as well. Different techniques, like freezing and soil grouting, are applied to construct the cross passages.

The in-depth design of the exit shaft is elaborated in details. After reinforcing the tunnel segments, tunnel crack is considered as a criteria to determine the tunnel spacing. It turns out 13m as tunnel spacing satisfies the requirement. But it is still too wide and expensive to build the shaft, thus different techniques, like more reinforcement, freezing soil, deep soil mixing, diaphragm wall, post-tension, are applied to optimise the spacing. Each technique has its optimum spacing and then these techniques are evaluated from aspects of effectiveness, risk and cost. Finally post-tension is chosen as the best measure and 3.2m is determined as the optimum tunnel spacing.

Since the magnetite is put on the surface ground to guarantee the support pressure and prevent blow-out, after tunnel construction it is necessary to remove it. It turns out the internal force and crack width satisfy the requirement after removing the magnetite.

The settlement at brick factory and dike is also checked because both locations have a strict limit of 25mm settlement. It turns out the settlements on both locations are higher than 25mm. In order to control it within the allowable settlement, decreasing the soil loss from 0.5% to 0.35% can make the settlement lower than 25mm.

Comparison

Having compared the bored tunnel with the bridge, it turns out the double tube bored tunnel with emergency lane with cut-and-cover has advantages on aspects of nature protection, landscape, cultural history and archaeology, social aspect and noise-affected surface. Since the cost of bridge construction has a wide range from 187.5 to 744 million euros, the cost of the double tube bored tunnel with emergency lane with cut-and-cover is 375 million euros, it can not be concluded the double tube bored tunnel with emergency lane with cut-and-cover is cheaper than bridge construction or not.

Discussion

In the discussion section, instead of Multi Criteria Analysis (MCA), a social cost-benefit analysis (SCBA) is adopted to evaluate the criteria related to environmental impacts. All criteria are quantified and monetized. It is found the immersed tunnel without dock is cheaper than other tunnel alternatives when taking technical problems into account. The cost of the immersed tunnel without dock is 323.8 million euros. Since the cost of bridge ranges from 187.5 to 744 million euros, so I can not give a conclusion if the immersed tunnel without dock is cheaper than bridge only when there is a precise cost estimation of the bridge construction.

8.2. Recommendation

In some phases, different assumptions and simplifications are made to progress this thesis. So it is necessary to recommend further studies in these phases.

Design definition

At some locations on the west bank of the river, there is 2 to 3m thick layer of stiff to very stiff silty clay. At two locations, a thick layer of stiff to very stiff organic clay was encountered. This layer reaches a thickness of some 12m, completely covering the railway tunnel cross-section. So more in-situ investigations need to carry out along the tunnel alignment to check if there is stiff clay or organic clay because both two tunnels are close.

Evaluation of verified concepts

Time on the construction of each concept is determined from practical experience. The time on the shaft construction is the same for concept A (double-tube bored tunnel with emergency lane) and concept B (double-tube bored tunnel without emergency lane). This is not realistic because the dimension of the shaft is different, so the time can be estimated more accurately. The cost estimation is not precise as well because it does not include all structural components. It only considers the amount and price of materials and equipment, construction cost and the price of the main structural components. The cost can be elaborated into more details and thus the evaluation would be more comprehensive. In the evaluation of verified concepts, five criteria are evaluated by a Multi Criteria Analysis (MCA). Different or more criteria could be added to the MCA.

Preliminary design

In the pre-design of cross passages, freezing and soil grouting techniques are adopted based on the water pressure and the location of cross passages. The cross passages can be elaborated to check the risks and cost of each technique. More techniques, like increasing air pressure, could be added to construct the cross passages. It is necessary to make a comparison among different techniques and take measures to mitigate the risks.

In-depth design of eastern shaft

Since the soil data of this project is lack, but the railway tunnel is close to this project so the soil data in railway tunnel is used in the Plaxis 2D model. In order to make the simulation and FEM more accurate, in-situ tests need be carried out and the real soil data should be provided.

In the Ultimate Limit State consideration, for simplicity, it does not take the effect of joints into account and only considers the allowable internal force and crack width. The effect of joints can be added as an additional criteria to assess the optimum tunnel spacing.

As for the horizontal alignment of optimisation techniques, the position of 13m spacing is determined as the location where the tunnel crack is 0.2mm. This assumes the soil properties and buried depth are the same as in the eastern shaft. Actually the buried depth is 1.2m deeper than in the shaft. As for the soil properties, more soil investigations should be carried out in order to determine the length of applying these techniques.

Comparison and discussion

The cost of solving technical problems is roughy estimated because the amount of water to be drained from dry dock is assumed based on the experience, a detailed cost estimation is recommended. Also, a more precise cost of bridge construction should be estimated as well in order to be comparable with tunnel alternatives. From the social cost-benefit analysis, the immersed tunnel without dock is the cheapest among all tunnel concepts and a detailed design of this concept should be carried out. With regard to the detailed design of the immersed tunnel without dock, in order to save the cost of transporting and immersing tunnel elements, the already made elements could be stored temporarily in the lake nearby. After all tunnel elements are ready and then they can be transported and immersed together.

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A

Uplift calculation

In the calculation of uplift, the tunnel should stay stable along the alignment. Here only the critical position canal is picked to calculate to prevent uplift.The vertical equilibrium in the construction phase is checked with:

$$A \leq G_{1,1} + G_2 \tag{A.1}$$

Where

A	uplift force calculated [kN/m]
$G_{1,1}$	weight tunnel lining [kN/m]
G_2	effective ground weight [kN/m]

$$A = \frac{\pi}{4} D_o^2 \gamma_w \tag{A.2}$$

Where

D_o	external diameter tunnel [m]
γ_w	unit weight of water [kN/m^3]

$$G_{1,1} = \frac{\pi(D_o^2 - D_i^2)}{4}\gamma_c \tag{A.3}$$

Where

D_i	internal diameter tunnel [m]
γ_c	unit weight of concrete [kN/m^3]

$$G_2 = D_o \cdot h \cdot \gamma_g^{'} - \frac{\pi}{8} \cdot D_o^2 \cdot \gamma_g^{'} \tag{A.4}$$

Where

D_i internal diameter tunnel [m]

$\gamma_g^{'}$ effective soil unit weight [kN/m^3]

D_o external diameter tunnel [m]

h depth of tunnel axis [m]

Concept A

$D_o = 13m, D_i = 11.8m$, after calculation,the uplift and downward force at canal position is:

$A = \frac{\pi}{4} D_o^2 \gamma_w \times 1.06 = 1406kN$

When h=13.2m (denotes the soil cover to the center line of tunnel):

$G_{1,1} + G_2 = 6.7 \times 13 \times 10 + (6.5 \times 13 - \frac{\pi D_o^2}{8}) \times 10 + \frac{\pi(D_O^2 - D_i^2)}{4} \times 24 = 1612kN$

$G_{1,1} + G_2 \geq A$

Concept B

$D_o = 11.3m, D_i = 10.15m$, after calculation,the uplift and downward force at canal position is:

$A = \frac{\pi}{4} D_o^2 \gamma_w \times 1.06 = 1062kN$

When h=11.75m (denotes the soil cover to the center line of tunnel):

$G_{1,1} + G_2 = 6.1 \times 11.3 \times 10 + (5.65 \times 11.3 - \frac{\pi D_o^2}{8}) \times 9.5 + \frac{\pi(11.3^2 - 10.15^2)}{4} \times 24 = 1284kN$

$G_{1,1} + G_2 \geq A$

Concept C

$D_o = 17.3m, D_i = 15.65m$, after calculation,the uplift and downward force at canal position is:

$A = \frac{\pi}{4} D_o^2 \gamma_w \times 1.06 = 2490kN$

When h=19.15m (denotes the soil cover to the center line of tunnel):

$G_{1,1} + G_2 = 10.5 \times 17.3 \times 9.5 + (8.65 \times 17.3 - \frac{\pi D_o^2}{8}) \times 10 + \frac{\pi(17.3^2 - 15.65^2)}{4} \times 24 = 3070kN$

$G_{1,1} + G_2 \geq A$

B

Cost estimation

Double tube bored tunnel with cut and cover with emergency lane (A1)								
Non-bored section		quantity			quantity		unit rate	cost(€)
Cut&Cover tunnel					1108		2750	39,611,000
Length open approaches					1560		1250	25,350,000
							Non-bored	97,441,500
Bored section								
Buy TBM at around 13.0 m					2	st	31300000	62600000
Version with special detection front TBM					2	st	500000	1000000
(special provisions) / estimated								
Supply of drill equipment on TBM					2	st	705000	1410000
Mobilization					2	st	0.94265	1.8853
Demobilizaton					2	st	794300	1588600
Labor costs mob / demob					2	st	6345000	12690000
Residual value (approx. 10% purchase)	TBM				2	st	-3130000	-6260000
	TBM				2	st	-50000	-100000
	TBM				2	st	-70500	-141000
Buy separation plant					2	st	4935000	9870000
Mobilization					2	st	2068000	4136000
Demobilizaton					2	st	799000	1598000
Labor costs mob / demob					2	st	423000	846000
Residual value (approx. 30.00% purchase)					2	st	-1480500	-2961000
Production site costs fixed					0	pst	940000	0
Production site costs variable		1581	2		3162	m1	4250	13438500
Ground drain 152m3 / m1		1581	152	2	480624	m3	15	7209360
Waste bentonite slurry (approx. 20% soil to be removed)		1581	30	2	94860	m3	22.5	2134350
Concrete consumption (35 kg / m3 soil)		1581	152	2	480624	m3	7	3364368
Maintenance and replacement of chisels		1581	2		3162	m1	30	94860
Extra maintenance (due to expected quantities of rubble		1581	2		3162	m1	6	18972
from '40 -'45) / drilling shield / 20.00%								
K&L in drilling tunnel		1581	2		3162	m1	1500	4743000
Groutinjectie 0,15m > 6,5m3/m1		1581	6.5	2	20553	m3	95	1952535
Lining d=0,60m > 23.36m3/m1		1581	23.36	2	73864.32	m3	650	48011808
Mob / demob energy supply		1			1	pst	1128000	1128000
Consumption 7200 kW power> 30% continuous € 0.23 / kwh		176	2		351	day	11280	3963040
Labor costs (3 teams 24 man)								
(3 x24 x 611,5 x 1,1 x8x2)		158.1	2	633.6	200344.32	mu	65	13022380.8
Man-hour average € 65 / hour								
Based on an average of 10m1 drills per day								
Concrete intermediate floor d = 0.40 m		1581	2	4.4	13912.8	m3	425	5912940
Concrete partitions d = 0.40 m		1581	2	1.44	4553.28	m3	550	2504304
Concrete behind safety bumpers (0.9x0.20)		1581	2	0.18	569.16	m3	250	142290
Cable ducts, add HWA		1581	2		3162	m1	2000	6324000
Wall finish tiles bottom 3 m		1581	3	4	18972	m2	100	1897200
Fire prevention (sprayed concrete with reinforcement) 23.7 m2 / m1		1581	45.5	2	143871	m2	75	10790325
vulbeton ca 6m3 / m1		1581	2	6	18972	m3	120	2276640
ceiling		11067	2	10	221340	m2	200	44268000
monitoring special points		3	2		6	pst	500000	3000000
engineering distortions and settings		2				pst	250000	500000
cross connections (ie approx. 250m)		5				st	1500000	7500000
vertical access shafts (2x5) to ground level		5				st	2500000	12500000
leakage facilities		4				st	750000	3000000
B5 block (11x20x20 per tunnel tube)(soil improvement)		4400	2	2	17600	m3	350	6160000
Surcharge "spectacle walls"		2	2		4	pst	282000	1128000
Compensation grouting km 3.050 - km 6.400 (50%)		0	0.5	45	0	m3	1565	0
Extra facilities for passage through tunnels		1				st	4700000	4700000
							Bored cost	272,634,749
							Total cost	370,076,249

Table B.1: Cost estimation of A1

Double tube bored tunnel with open approach with emergency lane (A2)								
Non-bored section		quantity			quantity		unit rate	cost(€)
Length open approaches					2636		1250	51,402,000
							Non-bored	77,103,000
Bored section								
Buy TBM at around 13.0 m					2	st	31300000	62600000
Version with special detection front TBM					2	st	500000	1000000
(special provisions) / estimated								
Supply of drill equipment on TBM					2	st	705000	1410000
Mobilization					2	st	0.94265	1.8853
Demobilizaton					2	st	794300	1588600
Labor costs mob / demob					2	st	6345000	12690000
Residual value (approx. 10% purchase)	TBM				2	st	-3130000	-6260000
	TBM				2	st	-50000	-100000
	TBM				2	st	-70500	-141000
Buy separation plant					2	st	4935000	9870000
Mobilization					2	st	2068000	4136000
Demobilizaton					2	st	799000	1598000
Labor costs mob / demob					2	st	423000	846000
Residual value (approx. 30.00% purchase)					2	st	-1480500	-2961000
Production site costs fixed					2	pst	940000	1880000
Production site costs variable		2135	2		4270	m1	4250	18147500
Ground drain 152m3 / m1		2135	152	2	649040	m3	15	9735600
Waste bentonite slurry (approx. 20% soil to be removed)		2135	30	2	128100	m3	22.5	2882250
Concrete consumption (35 kg / m3 soil)		2135	152	2	649040	m3	7	4543280
Maintenance and replacement of chisels		2135	2		4270	m1	30	128100
Extra maintenance (due to expected quantities of rubble		2135	2		4270	m1	6	25620
from '40 -'45) / drilling shield / 20.00%								
K&L in drilling tunnel		2135	2		4270	m1	1500	6405000
Groutinjectie 0,15m > 6,5m3/m1		2135	6.5	2	27755	m3	95	2636725
Lining d=0,60m > 23.36m3/m1		2135	23.36	2	99747.2	m3	650	64835680
Mob / demob energy supply		1			1	pst	1128000	1128000
Consumption 7200 kW power> 30% continuous € 0.23 / kwh		237	2		474	dg	11280	5351733.333
Labor costs (3 teams 24 man)								
(3 x24 x 611,5 x 1,1 x8x2)		213.5	2	633.6	270547.2	mu	65	17585568
Man-hour average € 65 / hour								
Based on an average of 10m1 drills per day								
Concrete intermediate floor d = 0.40 m		2135	2	4.4	18788	m3	425	7984900
Concrete partitions d = 0.40 m		2135	2	1.44	6148.8	m3	550	3381840
Concrete behind safety bumpers (0.9x0.20)		2135	2	0.18	768.6	m3	250	192150
Cable ducts, add HWA		2135	2		4270	m1	2000	8540000
Wall finish tiles bottom 3 m		2135	3	4	25620	m2	100	2562000
Fire prevention (sprayed concrete with reinforcement) 23.7 m2 / m1		2135	45.5	2	194285	m2	75	14571375
vulbeton ca 6m3 / m1		2135	2	6	25620	m3	120	3074400
ceiling		14945	2	10	298900	m2	200	59780000
monitoring special points		3	2		6	pst	500000	3000000
engineering distortions and settings		2				pst	250000	500000
cross connections (ie approx. 250m)		7				st	1500000	10500000
vertical access shafts (2x5) to ground level		6				st	2500000	15000000
leakage facilities		4				st	750000	3000000
B5 block (11x20x20 per tunnel tube)(soil improvement)		4400	2	2	17600	m3	350	6160000
Surcharge "spectacle walls"		2	2		4	pst	282000	1128000
Compensation grouting km 3.050 - km 6.400 (50%)		0	0.5	45	0	m3	1565	0
Extra facilities for passage through tunnels		1				st	4700000	4700000
							Bored cos	365,636,323
							Total cos	442,739,323

Table B.2: Cost estimation of A2

Double tube bored tunnel with cut and cover without emergency lane (B1)								
Non-bored section		quantity			quantity		unit rate	cost(€)
Cut&Cover tunnel					1108		2750	34,431,100
Length open approaches					1306		1250	18,447,250
							Non-bored	79,317,525
Bored section								
Buy TBM at around 11.3m					2	st	27200000	54400000
Version with special detection front TBM					2	st	500000	1000000
(special provisions) / estimated								
Supply of drill equipment on TBM					2	st	600000	1200000
Mobilization					2	st	2120000	4240000
Demobilizaton					2	st	676000	1352000
Labor costs mob / demob					2	st	5400000	10800000
Residual value (approx. 10% purchase)	TBM				2	st	-2720000	-5440000
	TBM				2	st	-50000	-100000
	TBM				2	st	-60000	-120000
Buy separation plant					2	st	4200000	8400000
Mobilization					2	st	1760000	3520000
Demobilizaton					2	st	680000	1360000
Labor costs mob / demob					2	st	360000	720000
Residual value (approx. 30.00% purchase)					2	st	-1260000	-2520000
Production site costs fixed					2	pst	1200000	2400000
Production site costs variable		1581	2		3162	m1	4250	13438500
Ground drain 152m3 / m1		1581	152	2	480624	m3	15	7209360
Waste bentonite slurry (approx. 20% soil to be removed)		1581	30	2	94860	m3	22.5	2134350
Concrete consumption (35 kg / m3 soil)		1581	152	2	480624	m3	7	3364368
Maintenance and replacement of chisels		1581	2		3162	m1	30	94860
Extra maintenance (due to expected quantities of rubble		1581	2		3162	m1	6	18972
from '40 -'45) / drilling shield / 20.00%								
K&L in drilling tunnel		1581	2		3162	m1	1500	4743000
Groutinjectie 0,15m > 5.4m3/m1		1581	5.4	2	17074.8	m3	95	1622106
Lining d=0,58m > 19.36m3/m1		1581	19.36	2	61216.32	m3	650	39790608
Mob / demob energy supply		1			1	pst	960000	960000
Consumption 7200 kW power> 30% continuous € 0.23 / kwh		176	2		351	dg	9600	3372800
Labor costs (3 teams 24 man)								
(3 x24 x 611,5 x 1,1 x8x2)		158.1	2	633.6	200344.32	mu	65	13022380.8
Man-hour average € 65 / hour								
Based on an average of 10m1 drills per day								
Concrete intermediate floor d = 0.40 m		1581	2	3.66	11572.92	m3	425	4918491
Concrete partitions d = 0.40 m		1581	2	1.44	4553.28	m3	550	2504304
Concrete behind safety bumpers (0.9x0.20)		1581	4	0.18	1138.32	m3	250	284580
Cable ducts, add HWA		1581	2		3162	m1	2000	6324000
Wall finish tiles bottom 3 m		1581	4	3	18972	m2	100	1897200
Fire prevention (sprayed concrete with reinforcement) 23.7 m2 / m1		1581	40.9	2	129325.8	m2	75	9699435
vulbeton ca 6m3 / m1		1581	2	6	18972	m3	120	2276640
ceiling		11067	2	10	221340	m2	200	44268000
monitoring special points		3	2		6	pst	500000	3000000
engineering distortions and settings		2				pst	250000	500000
cross connections (ie approx. 250m)		5				st	1500000	7500000
vertical access shafts (2x5) to ground level		5				st	2500000	12500000
leakage facilities		4				st	750000	3000000
B5 block (11x17x20 per tunnel tube)(soil improvement)		3740	2	2	14960	m3	350	5236000
Surcharge "spectacle walls"		2	2		4	pst	240000	960000
Compensation grouting km 3.050 - km 6.400 (50%)		0	0.5	45	0	m3	1565	0
Extra facilities for passage through tunnels		1				st	4000000	4000000
							Bored cost	288,247,513
							Total cost	367,565,038

Table B.3: Cost estimation of B1

Double tube bored tunnel with open approach without emergency lane (B2)							
Non-bored section	quantity			quantity		unit rate	cost(€)
Length open approaches				2212		1250	37,493,400
						Non-bored	56,240,100
Bored section							
Buy TBM at around 11.3m				2	st	27200000	54400000
Version with special detection front TBM				2	st	500000	1000000
(special provisions) / estimated							
Supply of drill equipment on TBM				2	st	600000	1200000
Mobilization				2	st	2120000	4240000
Demobilizaton				2	st	676000	1352000
Labor costs mob / demob				2	st	5400000	10800000
Residual value (approx. 10% purchase) TBM				2	st	-2720000	-5440000
TBM				2	st	-50000	-100000
TBM				2	st	-60000	-120000
Buy separation plant				2	st	4200000	8400000
Mobilization				2	st	1760000	3520000
Demobilizaton				2	st	680000	1360000
Labor costs mob / demob				2	st	360000	720000
Residual value (approx. 30.00% purchase)				2	st	-1260000	-2520000
Production site costs fixed				2	pst	1200000	2400000
Production site costs variable	2135	2		4270	m1	4250	18147500
Ground drain 152m3 / m1	2135	152	2	649040	m3	15	9735600
Waste bentonite slurry (approx. 20% soil to be removed)	2135	30	2	128100	m3	22.5	2882250
Concrete consumption (35 kg / m3 soil)	2135	152	2	649040	m3	7	4543280
Maintenance and replacement of chisels	2135	2		4270	m1	30	128100
Extra maintenance (due to expected quantities of rubble	2135	2		4270	m1	6	25620
from '40 -'45) / drilling shield / 20.00%							
K&L in drilling tunnel	2135	2		4270	m1	1500	6405000
Groutinjectie 0,15m > 5.4m3/m1	2135	5.4	2	23058	m3	95	2190510
Lining d=0,58m > 19.36m3/m1	2135	19.36	2	82667.2	m3	650	53733680
Mob / demob energy supply	1			1	pst	960000	960000
Consumption 7200 kW power> 30% continuous € 0.23 / kwh	237	2		474	dg	9600	4554666.667
Labor costs (3 teams 24 man)							
(3 x24 x 611,5 x 1,1 x8x2)	213.5	2	633.6	270547.2	mu	65	17585568
Man-hour average € 65 / hour							
Based on an average of 10m1 drills per day							
Concrete intermediate floor d = 0.40 m	2135	2	3.66	15628.2	m3	425	6641985
Concrete partitions d = 0.40 m	2135	2	1.44	6148.8	m3	550	3381840
Concrete behind safety bumpers (0.9x0.20)	2135	4	0.18	1537.2	m3	250	384300
Cable ducts, add HWA	2135	2		4270	m1	2000	8540000
Wall finish tiles bottom 3 m	2135	4	3	25620	m2	100	2562000
Fire prevention (sprayed concrete with reinforcement) 23.7 m2 / m1	2135	40.9	2	174643	m2	75	13098225
vulbeton ca 6m3 / m1	2135	2	6	25620	m3	120	3074400
ceiling	14945	2	10	298900	m2	200	59780000
monitoring special points	3	2		6	pst	500000	3000000
engineering distortions and settings	2				pst	250000	500000
cross connections (ie approx. 250m)	7				st	1500000	10500000
vertical access shafts (2x5) to ground level	6				st	2500000	15000000
leakage facilities	4				st	750000	3000000
B5 block (11x17x20 per tunnel tube)(soil improvement)	3740	2	2	14960	m3	350	5236000
Surcharge "spectacle walls"	2	2		4	pst	240000	960000
Compensation grouting km 3.050 - km 6.400 (50%)	0	0.5	45	0	m3	1565	0
Extra facilities for passage through tunnels	1				st	4000000	4000000
						Bored cost	349,623,063
						Total cost	405,863,163

Table B.4: Cost estimation of B2

Single tube bored tunnel with cut and cover with emergency lane (C1)							
Non-bored section	quantity			quantity		unit rate	cost(€)
Cut&Cover tunnel				554		2750	51, 395, 273
Length open approaches				2420		1500	57, 565, 750
						Non-bored	163, 441, 534
Bored section							
Buy TBM at around 17.3m				1	st	40800000	40800000
Version with special detection front TBM				2	st	500000	1000000
(special provisions) / estimated							
Supply of drill equipment on TBM				1	st	915000	915000
Mobilization				1	st	3233000	3233000
Demobilizaton				1	st	1030900	1030900
Labor costs mob / demob				1	st	8235000	8235000
Residual value (approx. 10% purchase)TBM				1	st	-4080000	-4080000
TBM				1	st	-50000	-50000
TBM				1	st	-91500	-91500
Buy separation plant				1	st	6405000	6405000
Mobilization				1	st	2684000	2684000
Demobilizaton				1	st	1037000	1037000
Labor costs mob / demob				1	st	549000	549000
Residual value (approx. 30.00% purchase)				1	st	-1921500	-1921500
Production site costs fixed				1	pst	1830000	1830000
Production site costs variable	1581	1		1581	m1	4250	6719250
Ground drain 152m3 / m1	1581	152	1	240312	m3	15	3604680
Waste bentonite slurry (approx. 20% soil to be removed)	1581	30	1	47430	m3	22.5	1067175
Concrete consumption (35 kg / m3 soil)	1581	152	1	240312	m3	7	1682184
Maintenance and replacement of chisels	1581	1		1581	m1	30	47430
Extra maintenance (due to expected quantities of rubble	1581	1		1581	m1	6	9486
from '40 -'45) / drilling shield / 20.00%							
K&L in drilling tunnel	1581	1		1581	m1	1500	2371500
Groutinjectie 0,15m > 8.08m3/m1	1581	8.08	1	12774.48	m3	95	1213575.6
Lining d=0,8m > 33.37m3/m1	1581	33.37	1	52757.97	m3	650	34292680.5
Mob / demob energy supply	1			1	pst	1464000	1464000
Consumption 7200 kW power> 30% continuous € 0.23 / kwh	176	1		176	dg	14640	2571760
Labor costs (3 teams 24 man)							
(3 x24 x 611,5 x 1,1 x8x2)	158.1	2	633.6	200344.3	mu	65	13022380.8
Man-hour average € 65 / hour							
Based on an average of 10m1 drills per day							
Concrete intermediate floor d = 0.40 m	1581	2	15.8	49959.6	m3	425	21232830
Concrete partitions d = 0.40 m	1581	2	3.42	10814.04	m3	550	5947722
Concrete behind safety bumpers (0.9x0.20)	1581	4	0.18	1138.32	m3	250	284580
Cable ducts, add HWA	1581	2		3162	m1	2000	6324000
Wall finish tiles bottom 3 m	1581	4	3	18972	m2	100	1897200
Fire prevention (sprayed concrete with reinforcement) 23.7 m2 / m1	1581	84.07	1	132914.7	m2	75	9968600.25
vulbeton ca 6m3 / m1	1581	2	6	18972	m3	120	2276640
ceiling	11067	2	10	221340	m2	200	44268000
monitoring special points	3	2		6	pst	500000	3000000
engineering distortions and settings	2				pst	250000	500000
cross connections (ie approx. 250m)	0				st	1500000	0
vertical access shafts (2x5) to ground level	0				st	2500000	0
leakage facilities	2				st	750000	1500000
B5 block (11x26x26 per tunnel tube)(soil improvement)	7436	1	2	14872	m3	350	5205200
Surcharge "spectacle walls"	2	1		2	pst	366000	732000
Compensation grouting km 3.050 - km 6.400 (50%)	0	0.5	45	0	m3	1565	0
Extra facilities for passage through tunnels	1				st	6100000	6100000
						Bored cost	238, 878, 774
						Total cost	402, 320, 308

Table B.5: Cost estimation of C1

Single tube bored tunnel with open approach with emergency lane (C2)							
Non-bored section	quantity			quantity		unit rate	cost(€)
Length open approaches				3462		1250	98,822,790
						Non-bored	148,234,185
Bored section							
Buy TBM at around 17.3m				1	st	40800000	40800000
Version with special detection front TBM				2	st	500000	1000000
(special provisions) / estimated							
Supply of drill equipment on TBM				1	st	915000	915000
Mobilization				1	st	3233000	3233000
Demobilizaton				1	st	1030900	1030900
Labor costs mob / demob				1	st	8235000	8235000
Residual value (approx. 10% purchase) TBM				1	st	-4080000	-4080000
TBM				1	st	-50000	-50000
TBM				1	st	-91500	-91500
Buy separation plant				1	st	6405000	6405000
Mobilization				1	st	2684000	2684000
Demobilizaton				1	st	1037000	1037000
Labor costs mob / demob				1	st	549000	549000
Residual value (approx. 30.00% purchase)				1	st	-1921500	-1921500
Production site costs fixed				1	pst	1830000	1830000
Production site costs variable	2135	1		2135	m1	4250	9073750
Ground drain 152m3 / m1	2135	152	1	324520	m3	15	4867800
Waste bentonite slurry (approx. 20% soil to be removed)	2135	30	1	64050	m3	22.5	1441125
Concrete consumption (35 kg / m3 soil)	2135	152	1	324520	m3	7	2271640
Maintenance and replacement of chisels	2135	1		2135	m1	30	64050
Extra maintenance (due to expected quantities of rubble	2135	1		2135	m1	6	12810
from '40 -'45) / drilling shield / 20.00%							
K&L in drilling tunnel	2135	1		2135	m1	1500	3202500
Groutinjectie 0,15m > 8.08m3/m1	2135	8.08	1	17250.8	m3	95	1638826
Lining d=0,8m > 33.37m3/m1	2135	33.37	1	71244.95	m3	650	46309217.5
Mob / demob energy supply	1			1	pst	1464000	1464000
Consumption 7200 kW power> 30% continuous € 0.23 / kwh	237	1		237	dg	14640	3472933.333
Labor costs (3 teams 24 man)							
(3 x24 x 611,5 x 1,1 x8x2)	213.5	2	633.6	270547.2	mu	65	17585568
Man-hour average € 65 / hour							
Based on an average of 10m1 drills per day							
Concrete intermediate floor d = 0.40 m	2135	2	15.8	67466	m3	425	28673050
Concrete partitions d = 0.40 m	2135	2	3.42	14603.4	m3	550	8031870
Concrete behind safety bumpers (0.9x0.20)	2135	4	0.18	1537.2	m3	250	384300
Cable ducts, add HWA	2135	2		4270	m1	2000	8540000
Wall finish tiles bottom 3 m	2135	4	3	25620	m2	100	2562000
Fire prevention (sprayed concrete with reinforcement) 23.7 m2 / m1	2135	84.07	1	179489.5	m2	75	13461708.75
vulbeton ca 6m3 / m1	2135	2	6	25620	m3	120	3074400
ceiling	14945	2	10	298900	m2	200	59780000
monitoring special points	3	2		6	pst	500000	3000000
engineering distortions and settings	2				pst	250000	500000
cross connections (ie approx. 250m)	0				st	1500000	0
vertical access shafts (2x5) to ground level	0				st	2500000	0
leakage facilities	2				st	750000	1500000
B5 block (11x26x26 per tunnel tube)(soil improvement)	7436	1	2	14872	m3	350	5205200
Surcharge "spectacle walls"	2	1		2	pst	366000	732000
Compensation grouting km 3.050 - km 6.400 (50%)	0	0.5	45	0	m3	1565	0
Extra facilities for passage through tunnels	1				st	6100000	6100000
						Bored cost	294,524,649
						Total cost	442,758,834

Table B.6: Cost estimation of C2

	Pannerdensch canal without construction dock	Pannerdensch canal with construction dock	
Width of the tunnel (m)	29.1	29.1	
Height of the tunnel (m)	8.64	8.64	
Volume of dredged material (m3)	143172	101268	
Volume of excavated material land side (m3)	942775	1538176	
Length of Immersed tunnel m	1601	1601	
Number of TE	10	10	
Transition structures length	45	45	
Cut&Cover tunnel	489	489	
Covered tunnel length	2135	2135	
Length open approaches	427	427	
Number of tuber for MEP	2	2	
	Cost	**Cost**	
Open approaches (incl. earthwork and retaining walls)	15,532,125	15,532,125	
Transition structures	6,547,500	6,547,500	
Cut&Cover tunnels	39,132,225	39,132,225	
Immersed tunnel including casting yard	92,389,835	94,191,085	
Dredging	2,863,440	2,025,360	
Transport and Immersion	17,000,000	17,000,000	
MEP	46,970,000	46,970,000	
Total costs for construction	220,435,125	221,398,295	
Construction costs	330,652,688	332,097,443	
	unit rate(€)		
Immersion joint	600000	/Tunnel element	
Ballast tanks etc	600000	/Tunnel element	
Transportation	tbd	/Tunnel element	
Transportation	250000	/Tunnel element	
Immersion	850000	/Tunnel element	
Tunnel element	1850	/m2	
Casting yard/m	1250	/m	
Dredging	20	/m3	
Transition structures	5000	/m2	
Cut and cover tunnel	2750	/m2	
Open approach	1250	/m2	
MEP	11000	/m	
Roads/asphalt	125	/m2	
Open approach DEEP	1500	/m2	
- Transport costs> approx. 50K to 500K per zinc section			
- Sinking> approx. 700K to 1,000K per zinc chip			
- Dredging (incl. Replenishment, removal, etc.)> approx. 10 to 20 / m3 (dredging itself is between 6 and 8 / m3)			
- Buildings> 1,250 to 1,500 / m2			
- Roadwork (incl. Road equipment)> 100 to 125 / m2			
- TTI> 7,000 to 11,000 per m1 / tunnel tube (short tunnels give higher TTI costs)			
- Landscape adaptations> approx. 3% to 5% direct costs			
- Storage factor directly to investment costs> 2.0 to 2.25 excl. VAT			

Table B.7: Cost estimation of immersed tunnel

C

Time on construction of each concept

In this project, it takes 10 months to construct the shaft, 7 months to install and disassemble the TBM , 10m per day in bored section. In the open approach and cut and cover section, it takes 25 weeks in the first 100m compartment and after 20m per week. For immersed tunnel, it takes 12 weeks to construct one element and there are totally 10 elements and 2 production line. The detailed time is shown in table C.1 and table C.2.

Table C.1: Construction time of bored tunnel

Design concept	shaft (month)	TBM install and disassemble (month)	bored section (month)	open approach (month)	cut-and-cover section (month)	Total time (month)
Double tube bored tunnel with cut and cover with emergency lane (A1)	10	7	5	16	19	57
Double tube bored tunnel without cut and cover with emergency lane (A2)	10	7	7	25	0	49
Double tube bored tunnel with cut and cover without emergency lane (B1)	10	7	5	16	18	56
Double tube bored tunnel without cut and cover without emergency lane (B2)	10	7	7	23	0	47
Single tube bored tunnel with cut and cover with emergency lane (C1)	12	7	5	8	18	50
Single tube bored tunnel without cut and cover with emergency lane (C2)	12	7	7	24	0	50

Table C.2: construction time of immersed tunnel

Design concept	dredging (month)	tunnel element (month)	construction dock (month)	open approach (month)	cut and cover section (month)	Total time (month)
Immersed tunnel with dry dock	2	14	8	7	8	39
Immersed tunnel without dry dock	2	29	0	7	8	46

D

Optimisation of Concept A1, B1, B2

Before optimising the three concepts, they should first satisfy the requirements of face stability to prevent the blow-out and settlement in vertical direction, and then to minimize the soil cover at both shafts and canal position.

D.1. Face stability

D.1.1. Introduction

To keep the tunnel face stable, the key point is to make the face pressure be in equilibrium with the water and the ground pressure. In this section, 3D model is applied to calculate the minimum support pressure and a simple 2D model to calculate the maximum support pressure. For minimum and maximum support pressure, three critical points are picked to calculate which are starting shaft, reception shaft and canal.

D.1.2. 3D Model for Minimum Support Pressure

The Jancsecz Steiner (1994) (Jancsecz and Steiner.W, 1994) is used to calculated the minimum support pressure (see figure D.1). It includes the effect of vertical and horizontal soil arching. This model is more accurate than Horn's model as vertical soil arching means, that part of the soil load above the tunnel crown does not act as a load on it but is carried by the surrounding soil.

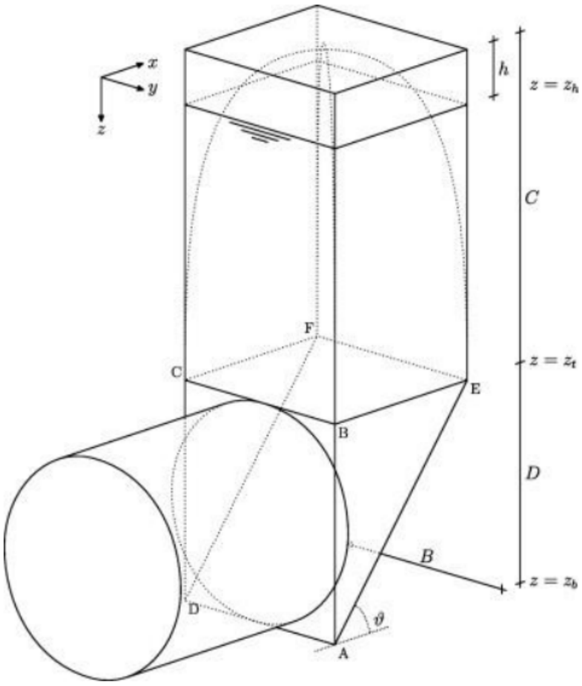

Figure D.1: *Jancsecz Steiner Failure Mechanism with Wedge*

The minimal required support force is calculated with:

$$S_{ci} = \eta_E \times E_{max,ci} + \eta_W \times E_{W,ci} \tag{D.1}$$

Where

S_{ci}	required minimum support force [kN]
η_E	safety factor for effective earth pressure [-]. In German codes this value is 1.5
η_W	safety factor for water pressure [-]. In German codes this value is 1.05
$E_{max,ci}$	resultant force of effective earth pressures for circulartunnel face [kN]
$E_{W,ci}$	resultant force of water pressures for circular tunnel face [kN]

The minimal required support pressure is then calculated with:

$$p_{min} = \frac{S_{ci}}{\frac{\pi \times D^2}{4}} + \Delta p_{op} \tag{D.2}$$

Where

S_{ci}	required minimum support force [kN]
p_{min}	minimum support pressure [kN/m^2]
D	diameter of the tunnel [m]
δp_{op}	Operation tolerance: for a slurry shield a variation of. +/- 10 kPa and for the EPB shield a variation of +/- 30 kPa should be included.

Resultant force of water pressures

$$E_{W,ci} = A_0 \times \sigma_w \tag{D.3}$$

Where

A_0	area of the tunnel face, $\frac{\pi \times D^2}{4}$ [m^2]
σ_w	water pressure at the axis of the tunnel [kN/m^2]
D	diameter of the tunnel [m]

Resultant force of earth pressures

The equilibrium depends on the soil weight, the friction angleϑ, cohesion and sliding angle of the wedge.See figure D.2 for the force in this model.

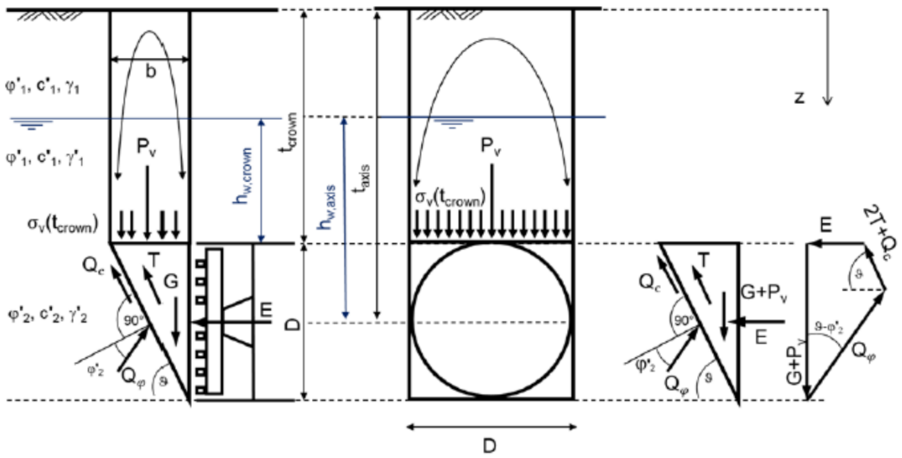

Figure D.2: *Forces acting on the wedge*

Note: Forces acting on the wedge with E_{re}=support force due to the earth pressure,G=own weight of wedge,P_v=vertical load from the soil prism, T=shear force on the vertical slip surface,ϑ=sliding angle, φ'=friction angle,c=cohesion of the soil,D=shield diameter and Q=shear force on inclined surface,γ=unit weight of soil

$$E_{max,ci} = E_{re(\vartheta)} \times \frac{\frac{\pi \times D^2}{4}}{D^2} \tag{D.4}$$

Based on the conditions of horizontal and vertical equilibrium in Eq(D.5)

$$E_{re(\vartheta)} = \frac{(G + P_v) \times [\sin(\vartheta) - \cos(\vartheta) \times \tan(\varphi_2^{'})] - 2 \times T - Q_c}{\sin(\vartheta) \times \tan(\varphi_2^{'}) + \cos(\vartheta)} \tag{D.5}$$

Where

$E_{max,ci}$	resultant force of earth pressures on the tunnel face[kN]
$E_{re(\vartheta)}$	resultant force of earth pressures on the area of $D^2[kN]$
ϑ	sliding angle of the wedge [°]. As a first estimate the following formula can be used $\vartheta = 45 + \frac{1}{2} \times \varphi_{av}^{'}$
D	diameter of the tunnel [m]
$\varphi_{av}^{'}$	angle of internal friction [°]
G	self-weight of the wedge [kN]
P_v	vertical force on the wedge [kN]
T	shear resistance on the triangular plane of the sliding wedge [kN]
Q_c	shear resistance on wedge

The self-weight of the wedge is calculated with:

$$G = \frac{1}{2} \cdot \frac{D^3}{\tan(\vartheta_{crit})} \cdot \gamma_{2,av}^{'} \tag{D.6}$$

Where

G	self-weight of the wedge [kN].
ϑ	critical sliding angle of the wedge [°]
D	diameter of the tunnel [m]
$\gamma_{2,av}^{'}$	average effective soil weight in the tunnel face area[kN/m^3]

The vertical force of the soil prism on the wedge is calculated with:

$$P_v = F \cdot \frac{\frac{F}{U}\gamma - c}{\lambda \tan(\varphi)} \{1 - e^{-t\frac{U}{F}\lambda \tan(\varphi)}\} \tag{D.7}$$

Where

γ	unit weight
φ	internal friction angle
G	self-weight of the wedge [kN]
λ	earth pressure coefficient.$\lambda = 0.8$
U	circumference of the horizontal plane from the soil wedge.$U = 2(D + \frac{D}{\tan\vartheta})$
F	area of the horizontal plane from the soil wedge. $F = \frac{D^2}{\tan\vartheta}$

The shear resistance on the triangular plane of the sliding wedge, is calculated with:

$$T = T_R + T_C \tag{D.8}$$

Where

T	shear resistance on the triangular plane of the sliding wedge [kN] (see figure D.3)
T_R	shear resistance on the triangular plane of the sliding wedge, due to friction [kN].
T_C	shear resistance on the triangular plane of the sliding wedge, due to cohesion [kN].

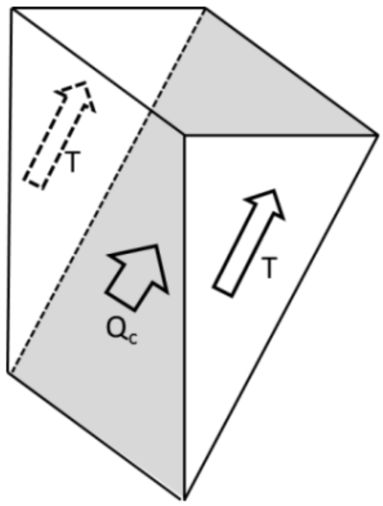

Figure D.3: *Shear forces on wedge*

$$T_R = K_2 \cdot \tan(\varphi'_{2,av}) \cdot \left[\frac{D^2 \cdot \sigma'_v}{2 \cdot \tan(\vartheta_{crit})} + \frac{D^3 \cdot \gamma'_{2,av}}{6 \cdot \tan(\vartheta_{crit})} \right] \tag{D.9}$$

Where

K_2	lateral earth coefficient [-]
T_R	shear resistance on the triangular plane of the sliding wedge, due to friction [kN].
ϑ_{crit}	critical sliding angle of the wedge [°]
D	diameter of the tunnel [m]
$\gamma'_{2,av}$	average effective soil unit weight in the tunnel face area[kN/m^3].
$\gamma'_{1,av}$	average effective soil unit weight on top of the tunnel crown [kN/m3]
t	distance tunnel crown to surface tunnel [m]
σ'_v	effective vertical load, calculated with (**??**)

The lateral earth coefficient is calculated based on Jancsecz-Steiner $K_2 = \frac{K_0+K_a}{2}$

$$T_c = c'_2 \cdot \frac{D^2}{2 \cdot \tan(\vartheta)} \tag{D.10}$$

Where

T_c	shear forces on the triangular plane of the sliding wedge, due to friction [kN]
ϑ_{crit}	critical sliding angle of the wedge [°]
D	diameter of the tunnel [m]
$c_2^{'}$	average cohesion in the tunnel face area [kN/m^2]

The shear resistance on the rectangular plane of the sliding wedge, is calculated with:

$$Q_c = c_2^{'} \cdot \frac{D^2}{2 \cdot \sin(\vartheta)} \tag{D.11}$$

Where

Q_c	shear forces on the wedge [kN]
ϑ_{crit}	critical sliding angle of the wedge [°]
D	diameter of the tunnel [m]
$c_2^{'}$	average cohesion in the tunnel face area [kN/m^2]

D.1.3. Simple 2D Calculation Method for the Maximum Support Pressure

In case of a shallow tunnel and high groundwater, it should be verified that the maximum support pressure does not cause a blowout in case of compressed air or a heave when using slurry machine. Usually safety against blowout and heave is guaranteed if the maximum support pressure is lower than the soil and water load. However, when the face is supported by air, and not completely sealed with a filter cake , air will leak out of the working chamber and flow into the soil.
Besides, during process of slurry machine, fluid pressure can push soil particles forward and make them apart so the cracks are formed in horizontal or vertical direction. Normally the pressure loss is negligible and the cracks would propogate along its path and this path is always fast. Cracks can not be found in time so there will be geo-technical hazard as a blow-out.
The maximum support pressure is calculated with:

$$P_{max} \leq \sigma_v \tag{D.12}$$

Where

P_{max}	maximum support pressure [kN/m^2]
σv	total soil stresses at top of tunnel [kN/m^2]

In this model, shear stresses between the moving soil body and its surroundings are also included and it takes into account friction forces along the vertical sides of the soil column. See figure D.4

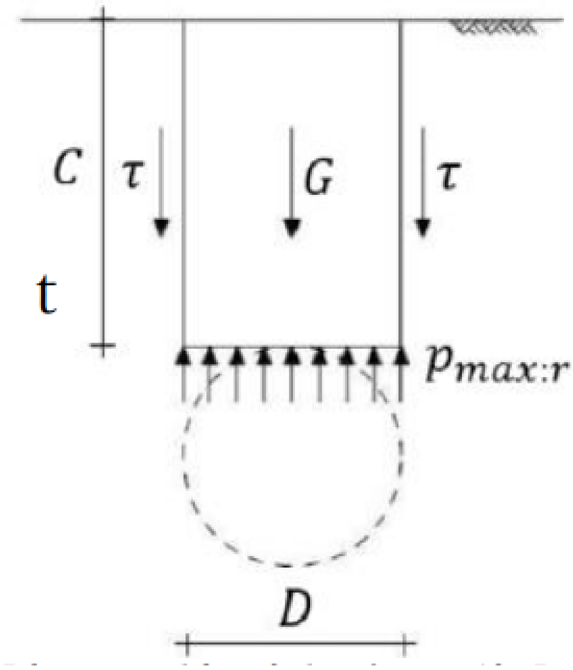

Figure D.4: *Blowout model, including friction (by Broere, 2001)*

$$P_{max} \leq \gamma_G \cdot \left(\sigma_v + \frac{2c^{'} + t \cdot K_0 \cdot \gamma^{'} \tan(\varphi)}{D} \right) \tag{D.13}$$

Where

P_{max}	maximum support pressure [kN/m^2]
σ_v	total soil stresses at top of tunnel [kN/m^2]
$c\prime$	cohesion of soil layer on top of the tunnel [kN/m^2]
t	distance tunnel crown to surface tunnel [m]
$\gamma^{'}$	average effective soil unit weight on top of the tunnel crown [kN/m^3]
$\varphi^{'}$	angle of internal friction of soil layer on top of the tunnel [°]
K_0	coefficient of earth pressure at rest [-]
γ_G	safety factor for permanent loads [-]. In German and Dutch codes this value is 0.9

D.2. Optimisation of A1

First of all, this concept should satisfy the face stability which means the difference between the maximum and minimum pressure should not be less than 50Kpa. The soil cover at shafts and canal is in the calculation below:

Starting shaft

D=13m

t=16.5m

$P_{max} = 298.3Kpa$

$P_{min} = 247.9Kpa$

$P_{max} - P_{min} = 50.4Kpa > 50Kpa$

Sliding angle: 63°

Canal

D=13m

t=21m=1.85D

$P_{max} = 427.1Kpa$

$P_{min} = 372.9Kpa$

$P_{max} - P_{min} = 54.2Kpa > 50Kpa$

Sliding angle: 64°

Reception shaft

D=13m

t=16m

$P_{max} = 289.0Kpa$

$P_{min} = 237.3Kpa$

$P_{max} - P_{min} = 51.7Kpa > 50Kpa$

Sliding angle: 64°

After satisfying the face stability, this concept is optimised by optimising some calculation parameters. In order to keep soil cover at shafts as shallow as possible to save cost, a slightly blow-out is allowed at both shafts, so a lower coefficient of earth pressure (1.3), pore pressure (1.0) and blow-out (1.0) is applied and only 20KPa margin for the maximum and minimum pressure difference. Also, a layer of magnetite, an iron-containing ore with a specific mass of 34KN/m^2 is applied on the ground surface. This special magnetite which is used as heavyweight ballast is harmless to the environment and non-toxic in all its forms. It is natural mineral magnetite and is mined from Kiruna, Northern Sweden (LKAB). While, in deeper locations like in canal and brick factory, those parameters keep the same.

Starting shaft

The calculation is shown below and see figure D.5.

D=13m

t=7.5m=0.66D

Thickness of iron layer: 1.2m

$P_{max} = 167.8Kpa$

$P_{min} = 142.9pa$

$P_{max} - P_{min} = 24.9Kpa > 20Kpa$

Sliding angle: 66°

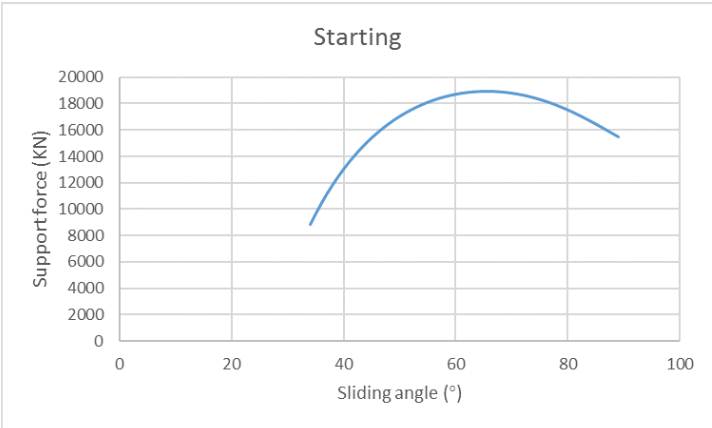

Figure D.5: *Support force in relation to sliding angle at starting shaft*

Canal

In order to control the maximum slope angle to be 4%, 5m soil in the canal is replaced by iron layer. This iron-containing layer is used to make face stable and will be removed after the TBM drills through the canal. The calculation is shown below and see figure D.6.

D=13m

t=13.4m=1.0D

Thickness of iron layer: 5.0m

$P_{max} = 350.5Kpa$

$P_{min} = 298Kpa$

$P_{max} - P_{min} = 52.5Kpa > 50Kpa$

Sliding angle: 64°

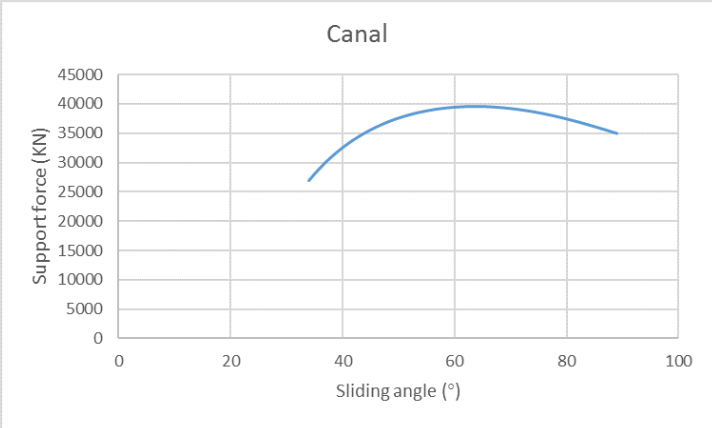

Figure D.6: *Support force in relation to sliding angle at canal*

Reception shaft

The calculation is shown below and see figure D.7.

D=13m

t=8.0m=0.7D

Thickness of iron layer: 1.0m

$P_{max} = 173.7Kpa$

$P_{min} = 146.5Kpa$

$P_{max} - P_{min} = 27.3Kpa > 20Kpa$

Sliding angle: 64°

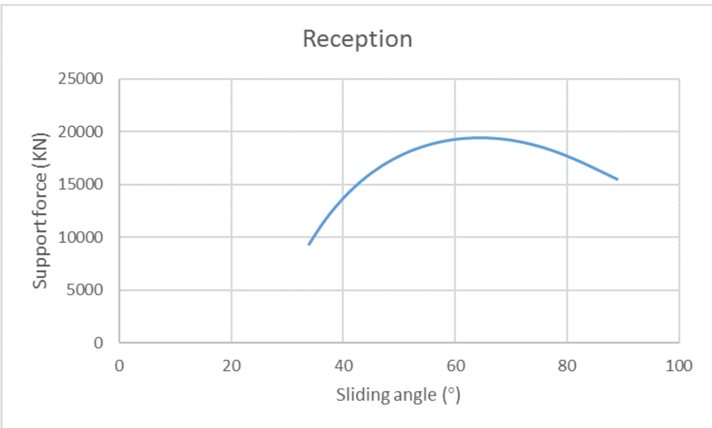

Figure D.7: *Support force in relation to sliding angle at reception shaft*

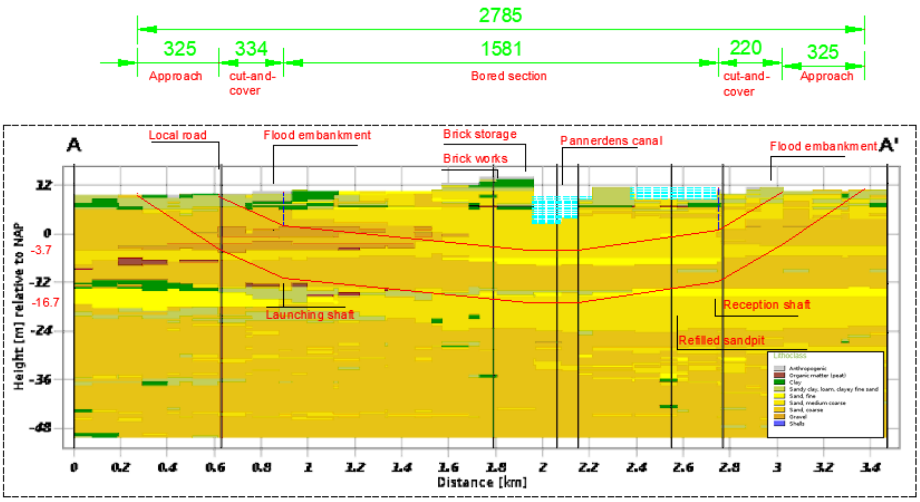

Figure D.8: *Optimised longitudinal profile of concept A with cut and cover section*

The optimized profile is shown in figure D.8.

D.3. Optimisation of B1

It is the same as optimisation of A1. This concept should first satisfy the face stability which means the difference between the maximum and minimum pressure should not be less than 50Kpa. The soil cover at shafts and canal is in the calculation below:

Starting shaft

D=11.3m

t=15.1m

$P_{max} = 273.3Kpa$

$P_{min} = 220.4Kpa$

$P_{max} - P_{min} = 52.9Kpa > 50Kpa$

Sliding angle: 63°

Canal

D=11.3m

t=19.4m=1.7D

$P_{max} = 395.9Kpa$

$P_{min} = 343.2Kpa$

$P_{max} - P_{min} = 52.6Kpa > 50Kpa$

Sliding angle: 64°

Reception shaft

D=11.3m

t=14.5m

$P_{max} = 262.3Kpa$

$P_{min} = 208.8Kpa$

$P_{max} - P_{min} = 54.5Kpa > 50Kpa$

Sliding angle: 64°

After satisfying the face stability, this concept is optimised by optimising some calculation parameters. It is the same method as the optimisation of A1 to optimize B1 and the calculation is below:

Starting shaft

The calculation is shown below and see figure D.9.

D=11.3m

t=6m=0.53D

Thickness of iron layer:1.5m

$P_{max} = 145.2Kpa$

$P_{min} = 117.4pa$

$P_{max} - P_{min} = 27.8Kpa > 20Kpa$

Sliding angle: 65°

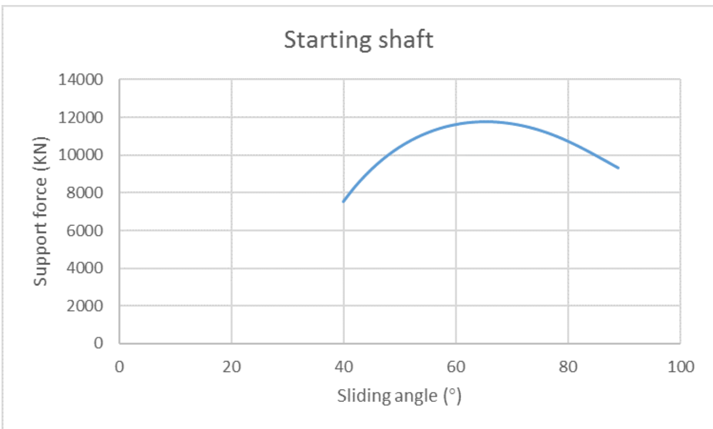

Figure D.9: *Support force in relation to sliding angle at starting shaft*

Canal

In order to control the maximum slope angle to be 4%, 5m soil in the canal is replaced by iron layer. This iron-containing layer is used to make face stable and will be removed after the TBM drills through the canal. The calculation is shown below and see figure D.10.

D=11.3m

t=12m=1.1D

Thickness of iron layer:5.0m

$P_{max} = 325.6Kpa$

$P_{min} = 269.6Kpa$

$P_{max} - P_{min} = 56Kpa > 50Kpa$

Sliding angle: 64°

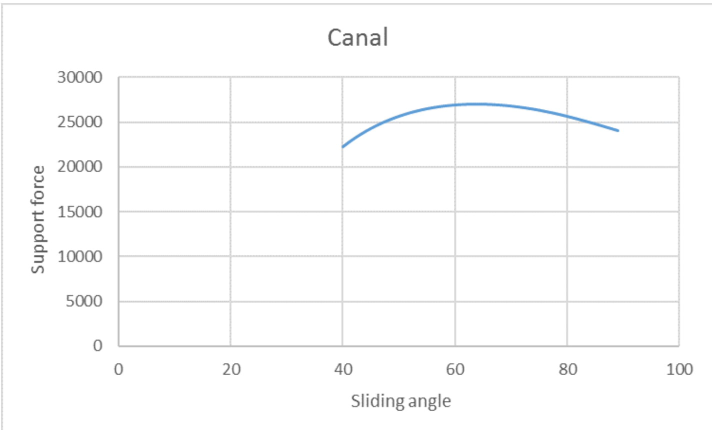

Figure D.10: *Support force in relation to sliding angle at canal*

Reception shaft

The calculation is shown below and see figure D.11.

D=11.3m

t=7.0m=0.6D

Thickness of iron layer:1.0m

$P_{max} = 153Kpa$

$P_{min} = 125.1Kpa$

$P_{max} - P_{min} = 27.9Kpa > 20Kpa$

Sliding angle: 65°

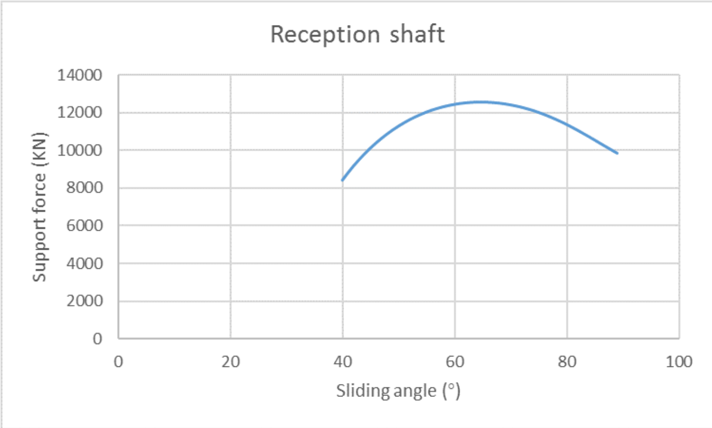

Figure D.11: *Support force in relation to sliding angle at reception shaft*

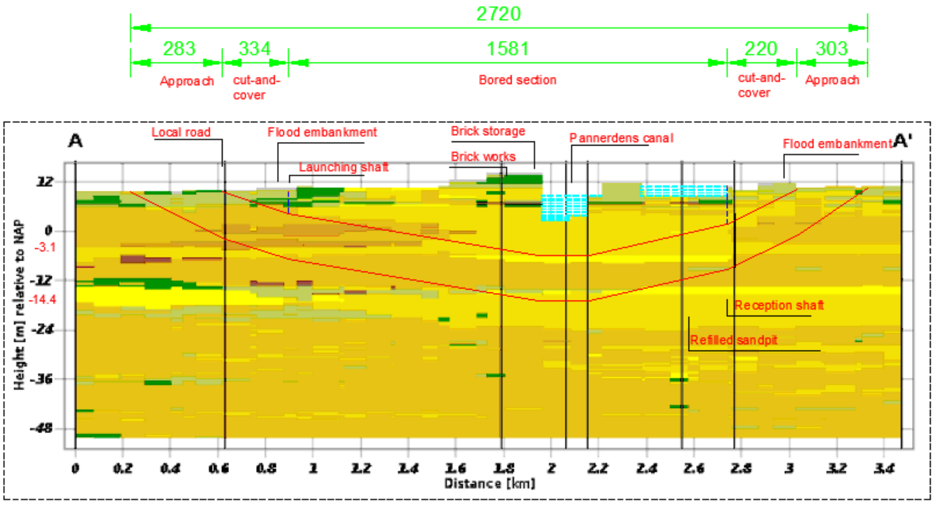

Figure D.12: *Optimised longitudinal profile of concept B with cut and cover section*

The optimized profile is shown in figure D.12.

D.4. Optimisation of B2

First of all, this concept should satisfy the face stability which means the difference between the maximum and minimum pressure should not be less than 50Kpa. The soil cover at shafts and canal is in the calculation below:

Starting shaft

D=11.3m

t=14.5m=1.3D

$P_{max} = 268.3Kpa$

$P_{min} = 215.9Kpa$

$P_{max} - P_{min} = 52.4Kpa > 50Kpa$

Sliding angle: 63°

Canal

D=11.3m

t=19m=1.7D

$P_{max} = 388.6Kpa$

$P_{min} = 338.8Kpa$

$P_{max} - P_{min} = 50Kpa = 50Kpa$

Sliding angle: 64°

Reception shaft

D=11.3m

t=13.5m=1.2D

$P_{max} = 253.8Kpa$

$P_{min} = 200Kpa$

$P_{max} - P_{min} = 53.8Kpa > 50Kpa$

Sliding angle: 64°

After satisfying the face stability, this concept is optimised by optimising some calculation parameters. It is the same method as the optimisation of A1 to optimize B1 and the calculation is below:
The calculation of optimisation is below and the optimized profile is shown in figure D.16.

Starting shaft

The calculation is shown below and see figure D.13.

D=11.3m

t=6.5m=0.6D

Thickness of iron layer:1.0m

$P_{max} = 146.9Kpa$

$P_{min} = 120.4pa$

$P_{max} - P_{min} = 26.5Kpa > 20Kpa$

Sliding angle: 64°

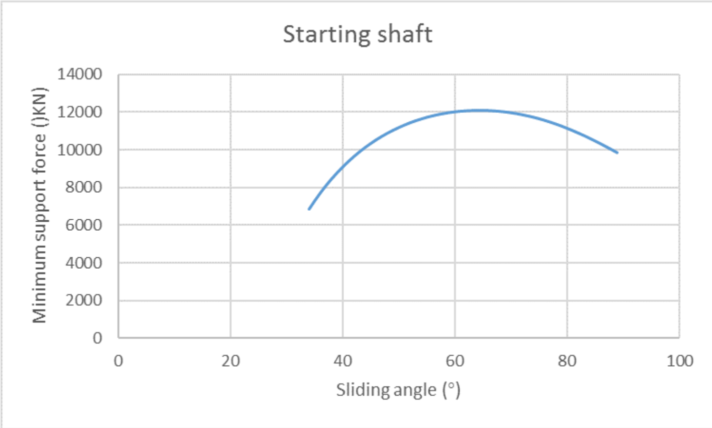

Figure D.13: *Support force in relation to sliding angle at starting shaft*

Canal

In order to control the maximum slope angle to be 4%, 5m soil in the canal is replaced by iron layer. This iron-containing layer is used to make face stable and will be removed after the TBM drills through the canal. The calculation is shown below and see figure D.14.

D=11.3m

t=12m=1.1D

$P_{max} = 325.7Kpa$

$P_{min} = 270.1Kpa$

$P_{max} - P_{min} = 55.6Kpa > 50Kpa$

Sliding angle: 64°

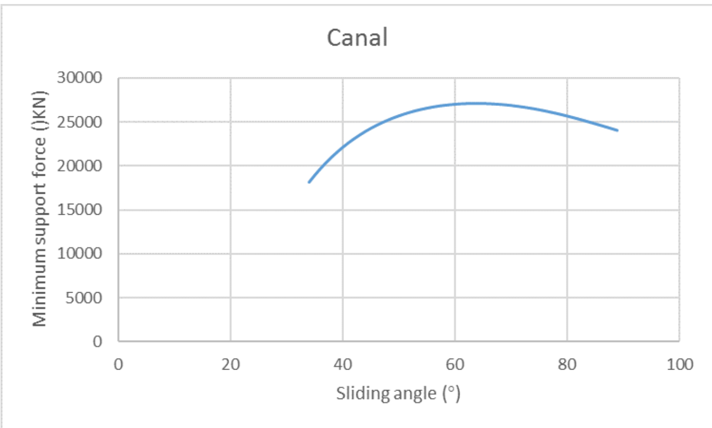

Figure D.14: *Support force in relation to sliding angle at canal*

Reception shaft

The calculation is shown below and see figure D.15.

D=11.3m

t=6.5m=0.6D

Thickness of iron layer:1.0m

$P_{max} = 149.9Kpa$

$P_{min} = 117.1Kpa$

$P_{max} - P_{min} = 32.8Kpa > 20Kpa$

Sliding angle: 65°

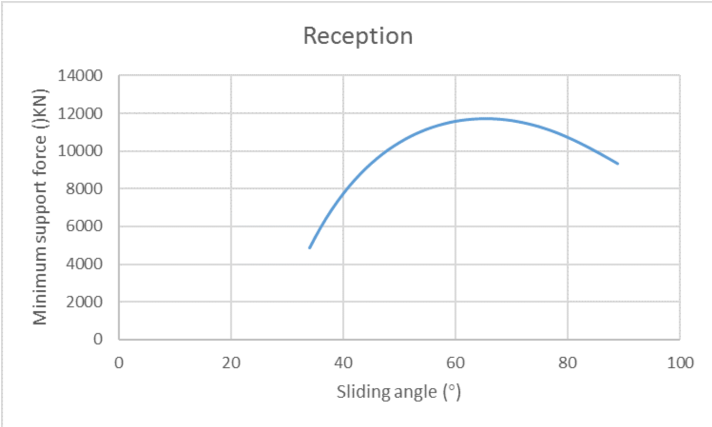

Figure D.15: *Support force in relation to sliding angle at reception shaft*

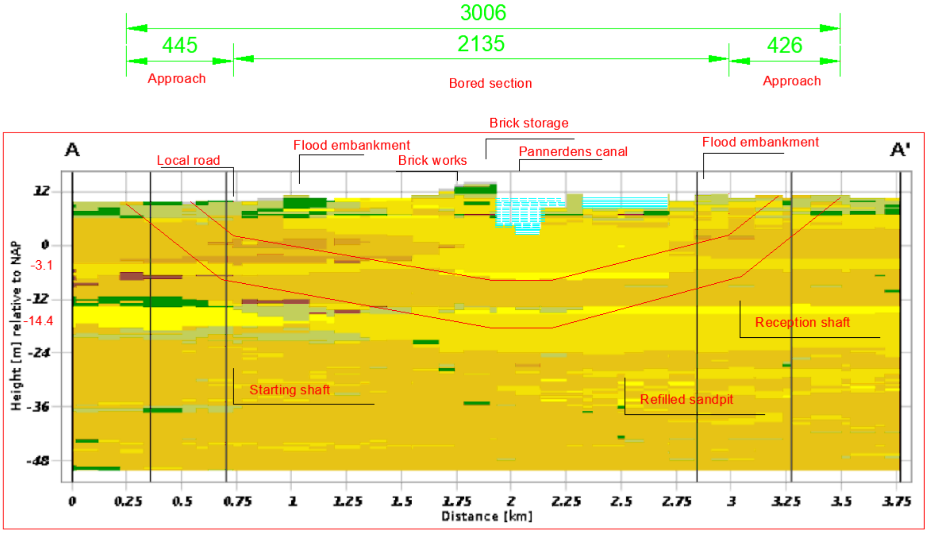

Figure D.16: *Optimised longitudinal profile of concept B without cut and cover section*

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E
Soil Investigation

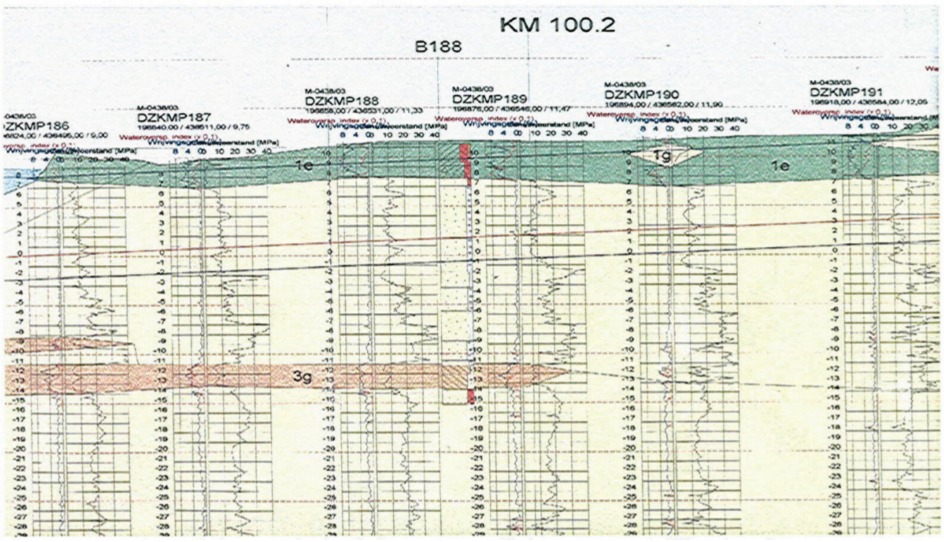

Figure E.1: *CPTs from two locations at eastern ramp in railway project(COB)*

PLAXIS - Finite Element Code for Soil and Rock Analyses

Project description	: Materiaal parameters	PLAXIS 8.x
User name	: FUGRO Ingenieursbureau b.v.	
Project name	: Pan kanaal_4_5	Date : 22-7-03
Output	: Soil and Interfaces Info - Hardening Soil	Step : 67 Page : 1

ID	Name	Type	γ_{unsat} [kN/m³]	γ_{sat} [kN/m³]	k_x [m/day]	k_y [m/day]	E_{50}^{ref} [kN/m²]	E_{oed}^{ref} [kN/m²]	E_{ur}^{ref} [kN/m²]	c_{ref} [kN/m²]
1	zand, vast toplaag	Drained	18.0	20.0	1.0000	1.0000	50000.0	50000.0	2E5	0.1
2	klei	Undrained	18.9	18.9	2.6800E-3	2.6800E-3	2500.0	2500.0	10000.0	4.0
3	zand matig vast	Drained	18.9	18.9	1.0000	1.0000	45000.0	45000.0	1.8E5	0.1
4	zand zeer vast	Drained	19.5	19.5	1.0000	1.0000	1E5	1E5	4E5	0.1
5	zand diep	Drained	18.4	19.4	1.0000	1.0000	55000.0	55000.0	2.2E5	0.1

Table E.1: Soil data 1

PLAXIS - Finite Element Code for Soil and Rock Analyses

Project description	: Materiaal parameters	PLAXIS 8.x
User name	: FUGRO Ingenieursbureau b.v.	
Project name	: Pan kanaal_4_5	Date : 22-7-03
Output	: Soil and Interfaces Info - Hardening Soil	Step : 67 Page : 2

ID	φ [°]	ψ [°]	ν_{ur} [-]	p_{ref} [kN/m²]	Power [-]	K_0^{nc} [-]	c_{incr} [kN/m³]	y_{ref} [m]	R_f [-]	T-Strength [kN/m²]	R_{inter} [-]	δ-inter [-]
1	35.0	5.0	0.20	100	0.500	0.426	0.0	0.0	0.9	0.0	0.70	0.000
2	27.5	0.0	0.20	110	0.900	0.538	0.0	0.0	0.9	0.0	0.70	0.000
3	36.5	6.5	0.20	180	0.500	0.405	0.0	0.0	0.9	0.0	0.70	0.000
4	37.9	7.9	0.20	280	0.500	0.386	0.0	0.0	0.9	0.0	0.70	0.000
5	36.5	6.5	0.20	400	0.500	0.405	0.0	0.0	0.9	0.0	0.70	0.000

Table E.2: Soil data 2

F

Reinforced concrete calculation

In order to know the maximum allowable internal force, the reinforced concrete calculation is carried out. The calculation is based on ensuring the stability and safety of the first tunnel construction.

F.1. Internal force

Among all construction stages, contraction stage has a maximum bending moment, it is considered as the critical stage. In this stage, the maximum bending moment appears at the top of the tunnel ring (0° of cross section), so the internal force at this point is chosen for the reinforced concrete calculation.

The calculation is based on the method $\eta - \xi$, as the longitudinal joints reduce the stiffness of tunnel ring, so the reduced ring stiffness is:
$\eta EI = 0.7 \times 653094 = 457166 kN \cdot m^2$
The internal force of the segment:
$M_s = (1+\xi)M = (1+0.3) \times 355 = 461.5 kNm$
$N_s = N = 656 kN$

$e_i = e_0 = \frac{M_s}{N_s} = 700 mm \geq= 0.3 h_0 = 165 mm$, the cross section area $A = 2000 \times 600 = 1200000 mm^2$
This indicates the segment is compressed with large eccentricity.

$k = 1.1, a = a^{'} = 50mm, e = ke_0 + \frac{h}{2} - a = 1020mm$

Taking the symmetrical reinforcement:

$N = b \cdot x \cdot R_w$

$N \cdot e = b \cdot x \cdot R_w(h_0 - \frac{x}{2}) + A_g^{'} \cdot R_g^{'}(h_0 - a^{'})$

Width of segment b=2000mm, design compression strength of concrete $R_w = 30N/mm^2$, design compression strength of rebar $R_g^{'} = 360N/mm^2$, thickness of concrete cover $a^{'} = 50mm$

Substituting the values into the above equations:

$656000 = 2000 \cdot x \cdot 30$

$656000 \times 1020 = 2000 \times 10.9 \times 30 \times (550 - \frac{10.9}{2}) + A_g^{'} \times 360 \times (550 - 50)$

So $x = 10.9mm$, the area of compressed rebar $A_g^{'} = 1739mm^2 \leq 0.2\%A = 2400mm^2$

The total area of rebar in the cross section should be larger than 0.55%A = 6600mm^2, so $A_g^{'} = 6800mm^2$

F.2. Verification of ultimate compression force

$N = 656kN, i = \sqrt{\frac{1}{12}} \cdot h = 173mm$, length of segment $l_0 = \frac{pi}{4} \cdot R = 5102mm$, $\frac{l_0}{i} = 29.5, \phi = 0.99$

The ultimate compression force $N_{cu} = 0.9\phi(A \cdot R_w + A_g^{'} R_g^{'}) = 0.90.99 \times (2000 \times 600 \times 30 + 6800 \times 360) = 34257kN \geq N = 656kN$

So the tunnel is stable at this point in this construction stage. In order to ensure the stability of the tunnel throughout the whole construction phases, it is necessary to verify the internal force at different points and different construction phases.

The method to do that is when it is given an axial force generated by Plaxis, the respective design bending moment (the maximum allowable design bending moment) can be decided and then it compares with the bending moment generated by Plaxis. The axial force generated by Plaxis should also need to compare with N_{cu}.

As a result, all bending moment and axial force generated by Plaxis in different points and different consruction phases are lower than the maximum allowable design bending moment and N_{cu}. This indicates the tunnel can always stay stable.

F.3. Verification of crack

The crack of concrete plays an important role in determining the safety of the tunnel as there would be leakage once the width of crack on the surface of concrete segment beyond the allowable width of crack. In order to ensure the safety of the tunnel, the width of crack in concrete should lower than the maximum allowable width of crack (0.2mm). The maximum bending moment point (0° of cross section) is chosen to verify.

$l_0/h = \frac{\frac{\pi}{4}6500}{600} = 8.5 < 14, \eta_s = 1.0$

$y_s = \frac{h}{2} - a_s = \frac{600}{2} - 50 = 250mm$

$e_0 = 700mm, e_s = \eta_s e_0 + y_s = 1 \times 700 + 250 = 950mm$

$z = [0.87 - 0.12(1 - \gamma_f^{'}) \times (\frac{h_0}{e_s})^2] \cdot h_0 = [0.87 - 0.12 \times (\frac{550}{950})^2] \times 550 = 456N/mm^2$

$\sigma_{ss} = \frac{N_s(e_s - z)}{A_s \cdot z} = \frac{656000 \times (950 - 456)}{6800 \times 456} = 104.5N/mm^2$

$W_{tk} = c_1 c_2 c_3 \frac{\sigma_{ss}}{E_s} \cdot \frac{30+d}{0.28+10\rho} = 1 \times 1.5 \times 0.9 \times \frac{104.5}{200000} \times \frac{30+14}{0.28+10\times0.55\%} = 0.09mm < 0.2mm$

The width of crack satisfy the requirement at this point and this construction stage. The same method is applied in verifying the safety of tunnel in all points and construction phases. It is found all width of cracks is lower than the maximum allowable width (0.2mm).

This indicates the area of rebar can guarantee the stability and safety of the first constructed tunnel.

G

Post tension calculation

Since it is not possible to input pre-stress in concrete in PLAXIS 2D, in order to simplify it, internal force of tunnel segments are firstly calculated without considering pre-stress. Then these internal forces generated by PLAXIS plus the pre-stress are equal to final internal force of segments. This assumption makes the axial force higher than real one , which makes design relatively conservative. Crack width of an arbitrary point in the tunnel is shown below:

When tunnel spacing L=0.15D, pick one point at angle=0°, axial force N generated by Plaxis equals -1114.3kN

$$\sigma_{con} \leq 0.75 \cdot f_{ptk} = 0.75 \times 1960 = 1470 N/mm^2 \tag{G.1}$$

$$N_p = \sigma_{con} \cdot A_p = 1470 \times 193 \times 10^{-3} = 283 kN \tag{G.2}$$

$N_t = N + N_p = -1114.3 + 283 = -831.29 kN$

$\sigma_{ss} = \frac{N_t \times (e_s - z)}{A_s \cdot z} = \frac{831.29 \times 10^3 \times (950-456)}{6800 \times 456} = 132.44 kN/m^2$

Crack width is calculated as follows:

$W_{tk} = c_1 c_2 c_3 \frac{\sigma_{ss}}{E_s} \cdot \frac{30+d}{0.28+10\rho} = 1 \times 1.5 \times 0.9 \times \frac{132.44}{200000} \times \frac{30+14}{0.28+10 \times 0.55\%} = 0.12 mm < 0.2 mm$

Where

σ_{con}	tensioning stress of strand $[N/mm^2]$
f_{ptk}	standard value of ultimate strength of strand $[N/mm^2]$
A_p	area of the strand $[mm^2]$
N_p	tensioning force that strand can provide $[kN]$
N_t	axial force of segment after post tension $[kN]$
W_{tk}	crack width $[mm]$
σ_{ss}	stress of reinforcement in tension state at crack position $[kN/mm^2]$
A_s	rebar area $[mm^2]$
ρ	rebar ratio in tension state $[-]$
c_1	parameter related to shape of rebar $[-]$
c_2	parameter considering long term effect $[-]$
c_1	parameter related to stress state of structural element $[-]$

The above calculation shows crack width of one point in the tunnel. The calculation process is the same for other points on the tunnel ring, which means crack width can be obtained once the axial force is generated by Plaxis.