Constructing The Green Connection with and without the use of an intermediate support

By

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Preface

This thesis is the final work for my master program Structural Engineering at the Delft University of Technology. The thesis is the subsequent report of the Preliminary Study. The research was conducted at Royal HaskoningDHV, in agreement with the TU Delft. It concerns several aspects in the design phases of a new to be constructed deck structure crossing an already existing motorway. The deck structure is called *"The Green Connection"*, and it will cross the motorway with a span of about 75 meters. The feasibility of the design will be investigated.

First of all, I would like to thank the members of my graduation committee. I would like to thank Professor Dick Hordijk and associate Professor Cor van der Veen of the TU Delft for their guidance during the thesis. Furthermore, words of thanks to my company supervisor Rob Vergoossen. I appreciated the inspiring conversations and constructive feedback you gave. Your structural speciality was very useful during dimensioning the deck structure. Additionally, I would like to thank all other colleagues at Royal HaskoningDHV for their help and discussions. I would like to express my gratitude to the company Royal HaskoningDHV for providing me the opportunity to conduct this thesis and to graduate as a civil engineer. I look back to a pleasant period.

Moreover, I'd like to thank Professor Aad van der Horst for his inspiring talks about the execution aspects. Your way of thinking describes the multidisciplinary approach of realizing complex projects. It was essential and meant a lot to the outcome of the thesis. I am also grateful to associate Professor Mandy Korff. Your geotechnical information plays a crucial role throughout this thesis.

To end, I would like to thank my friends and family for their support during my study period at the TU Delft.

Niels van Bergenhenegouwen

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Summary

In the Netherlands, a lot of traffic congestion occurs on motorways. This problem is most severe nearby larger cities. Utrecht is one of these cities. To improve traffic flow in this area, a huge masterplan is designed by Rijkswaterstaat called *"A27/A12 Ring Utrecht"*. One part of this masterplan consists of the motorway A27 at Amelisweerd. Here, the A27 is situated in a U-shaped concrete structure and must be expanded at both sides. Across the motorway a deck structure is going to be constructed with on top a public garden. This structure spans the total width of the A27 for 249 meters and is called The Green Connection.

According to the original design of Rijkswaterstaat, this deck structure should be realized with an intermediate support. It will be advantageous to omit this structure, since it has a complex execution method. Therefore, it's investigated if The Green Connection can be realized without the use of an intermediate support.

Then, the execution aspects of the original design will be discussed more thoroughly. The first challenge is constructing the extended parts which is explicated according to 11 main tasks. These tasks seem to be relatively straightforward to execute. After realizing the extended parts, an intermediate support must be constructed. It is found that the existing foundation lacks bearing capacity by far. A new strengthened strip foundation with extra foundation piles must be realised in the middle of the motorway. Due to the boundary conditions (such as the water pressure beneath the structure and permanent drainage is prohibited), the only possibility left is to construct small building pits, compartments, within the existing structure. Such a compartment has a rough length of about 20 meters, will be about 6.5 meters wide and must be constructed 13 times.

Thereafter, the deck structure should be assembled. Three alternatives in methods of assembly are outlined and discussed with the help of the same key-words. All three methods could be realized. But, it's important to indicate that with some extra investments, the remaining space for traffic could be maximized during assembly.

After discussing the execution aspects of the original design, the technical feasibility of the single span deck structure was investigated more thoroughly. In the Preliminary Study was deduced that two structural designs seem to be a possible solution in constructing The Green Connection. It turned out that the box beam design seems to be advantageous. Although this judgement is substantiated with preliminary calculations and an overall execution plan, it still required more research. Therefore, a reliable structural design is performed for a 75-meter span beam which can be used as a single span deck structure. It does exceed the boundary condition of 280 tons which was posed initially with 8%. However, no optimizations have been applied to this design. If the enumerated optimizations are performed, a beam can be designed according the boundary condition and possibly even less.

When the original design of Rijkswaterstaat is compared to the single span design, the differences are quite straightforward. In essence, the question arises whether the extra money of constructing a single span deck structure outweigh the money which can be saved by leaving out the intermediate support and constructing the extended parts 25% narrower.

Rijkswaterstaat has performed a design with an intermediate support. In this thesis, the feasibility of this design is investigated, and in particular this support. The only possible method of execution in realizing the support is upon condition that a temporary applied drainage system will be feasible and approved by the authority. When the authority states that draining ground water is prohibited, the original design isn't feasible anymore. In that case, the single span design isn't just an alternative to the original design, but is the only feasible solution.

Concludingly, providing the applied principles of this thesis, it is strongly recommended against constructing an intermediate support within the existing U-shaped concrete structure. Since the structural reliability of a 75-meter span beam is proven, an intermediate support wall is redundant. Therefore, the single span design is less risky, less time consuming and less expensive compared to the original design.

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1. Introduction

This report is the subsequent report of the Preliminary Study. Since the previous report forms the basis of this Master Thesis, the Preliminary Study will be rephrased briefly. It starts with an introduction, so that the subject of this research is clear again. For a more detailed introduction of the congestion problems around the city Utrecht, the masterplan *"A27/A12 Ring Utrecht"* and the historical background of the U-shaped concrete structure at Amelisweerd is referred to the Preliminary Study.

Another paragraph in this chapter is dedicated to the approach of this Master Thesis, emphasizing the core of the research. The research objective is slightly adapted compared to the one in the Preliminary Study. The master question is kept equal. But, in order to answer this question properly, some sub questions are formulated. Furthermore, a paragraph will express the report structure.

1.1 Preliminary Study

Around the city of Utrecht, traffic jams occur on a daily base on the local roads and motorways. It results in a low traffic flow which goes at the expense of the mobility of people who make use of this infrastructure. To improve the mobility in this area, a masterplan is designed by Rijkswaterstaat. This organization in the Netherlands is responsible for the design, construction, management and maintenance of the motorway network. One part of the masterplan *"A27/A12 Ring Utrecht"* consists of the A27 at Amelisweerd. Here, a concrete U-shaped structure must be expanded with maximum 15 meters in width at both sides to accommodate the 4 extra driving lanes.

To realize the extended parts of the motorway and accommodate space for the extra driving lanes, some severe construction tasks are required. To counterbalance these construction activities, a decision is made to realize a public garden on top of this motorway to return a certain area to nature. This park spans the A27 over a width of 249 meters. The reasons for this park are diverse. The important ones are adding value to the neighbouring area since it improves the accessibility to the estate of Amelisweerd. It is also beneficial for the ecological environment, since it forms a missing link in crossing the A27 for wildlife living in the adjacent forest. It's also a compensating measure to the chopped off trees which are required to realize the extended parts of motorway. But, to realize a public garden across the motorway a deck structure must be constructed first.

Figure 1.1 shows the current situation at the left side. The right side shows the intended new situation. As can be seen, the motorway A27 is expanded, and the deck structure with the public garden on top is visualized. This deck structure on top of the U-shaped concrete structure is called The Green Connection. This expression is used throughout the report.

Figure 1.1: Current situation (left) and new intend situation (right) [1].

1.2 Master Thesis

In the Master Thesis a certain approach will be followed. After clarifying the research objective and the research questions, the subsequent chapters are formatted to answer these questions.

1.2.1 Research objective

Since traffic jams occur on a daily base, the capacity of the surrounding motorways around the city Utrecht is not sufficient anymore. To increase the capacity of these motorways, a masterplan is designed by Rijkswaterstaat. This masterplan contains more than hundred engineering structures, but the only structure considered in this thesis is the U-shaped concrete structure. It is situated below ground surface level near the forest of Amelisweerd. To improve the traffic flow, this structure must be extended at both sides to provide space for the extra driving lanes. More important, after realization of the extended parts, the motorway must be covered up for over 249 meters, spanning the total width of the motorway. This opted structure is emphasized in the beginning of this chapter and visualized in Figure 1.1.

The construction of this deck structure is quite an ambitious task. According to the original design of Rijkswaterstaat, this deck structure should be realized with the help of an intermediate support. The construction of this intermediate support must be executed within one of the busiest motorways of the Netherlands. It will be a sequence of complex, expensive and risky tasks. Therefore, in this thesis it's investigated if realizing The Green Connection would be possible without the use of an intermediate support. In fact, The Green Connection will be realized, but it's just the question if it will be constructed with an intermediate support according the original design of Rijkswaterstaat, or without such a support. It should be investigated more thoroughly, because the construction activities have a high impact on the traffic dynamics at the A27.

1.2.2 Research question

From the Preliminary Study and the previous paragraphs the following main research question can be formulated:

> *How to construct The Green Connection as a concrete structure with and without the use of an intermediate support?*

This master question is going to be answered throughout the thesis. In order to provide a proper answer, some sub questions arose while attempting to answer the research question. The following sub research questions will help to answer the above stated master question:

- What could be an obvious way of constructing the extended parts?
- How to construct an intermediate support?
- How to realize the deck structure in an obvious way?
- Which of the two global designs will be the most suitable solution for constructing The Green Connection as a single span design?
- What will be a sufficient structural design for the single span deck structure of The Green Connection?
- What are the main findings while comparing both designs?

Answers to these sub questions will be gathered throughout this thesis in order to contribute to the main objective.

1.3 Thesis outline

This subchapter describes the outlines of the report. Starting in chapter 1, in which the problem is introduced and the main objective of this thesis is outlined.

The next chapter covers a summary of the boundary conditions which were performed in the Preliminary Study. First, some general considerations are provided which is important background information while designing. Thereafter, further limitations are described, since this project cover many structural aspects.

After that, one should know what kind of activities are required to realize each design. Therefore, the execution aspects of the original design are outlined in chapter 3. The chapter starts clarifying both road layouts of the designs and discusses the term availability. Then, an obvious way of constructing the extended parts is outlined, followed by a discussion in paragraph 3.4.2. The same chapter 3 consists of a plausible way of constructing the intermediate support. After explaining a viable method of execution, this topic will be discussed in paragraph 3.5.7. The last topic of the chapter considers three alternative method of assembly in constructing the deck structure of the original design. To end, it will be discussed too.

Chapter 4 covers two preliminary designs. These two designs have already been outlined in the Preliminary Study and could possibly become the new deck structure if a single span deck structure is considered. A basic structural design will be conducted for both ideas. The components bending moment resistance and shear capacity will be considered with the help of some preliminary calculations. Furthermore, the possibilities in transport of these elements of the deck structure will be discussed, the possible method of construction and thus the feasibility of the particular design at first glance.

Chapter 5 covers a calculation of the deck structure in more detail. With the help of some preliminary calculations outlined in chapter 4 is deduced that one of the two global designs will be most advantageous. Some more thorough calculations about this design will be performed. If it turns out that this design will not be satisfactory, a solution will be sought to increase the capacity of this beam to achieve a reliable design.

In chapter 6 is discussed about the comparisons between both designs, the original design of Rijkswaterstaat and the single span design. These two layouts are briefly rephrased, followed by discussing the three main construction tasks; the extended parts, the intermediate support and the deck structure. Thereafter, some words are mentioned about the term 'availability'. The chapter ends with discussion which answers the sub research question.

The obtained results of the research are discussed in chapter 7. Since constructing The Green Connection is such a major project, some other points of attention will be emphasized as well.

In chapter 8 the conclusion will be given, followed by the recommendations in chapter 9.

2. Boundary conditions and basic assumptions

In this chapter the main starting points of the previous report are emphasized. It's not just a summary of the first report, but more a numeration of several important findings and basic assumptions. These points are again stipulated in order to create a confined environment to perform the two global structural designs. The two global designs are described in chapter 4.

2.1 Overview of preliminary study

In order to create a confined design environment for the two designs in the next chapter, the basic assumptions, the boundary conditions and other required information like starting points are briefly rephrased in this chapter. When such an environment is created, a more optimal design can be performed and implemented in the surrounding area. These points are either from the previous report or are an extra assumption with respect to the designs.

The following points are outlined and can be used as a reference throughout this report. These are subdivided in categories as 'general', 'further limitations' and 'structural design'. More reference information can be found in the Preliminary Study. In that report the background information is discussed of the next paragraphs.

2.1.1 General

1. Beneath the concrete structure a water pressure of 67 kPa is present.
2. Traffic hindrance is judged to be very important during construction.
3. Transport of prefab elements from the only feasible prefabrication yard at Haitsma beton is considered as not possible due to their weight of 280 tons
4. Construction site number one is considered as the construction site which can be used for prefabrication of the elements of the deck structure. It is large enough to accommodate a prefabrication factory as well as a significant stock yard. The next figure is shown to visualize the surrounding area of The Green Connection and the appointed construction site one.

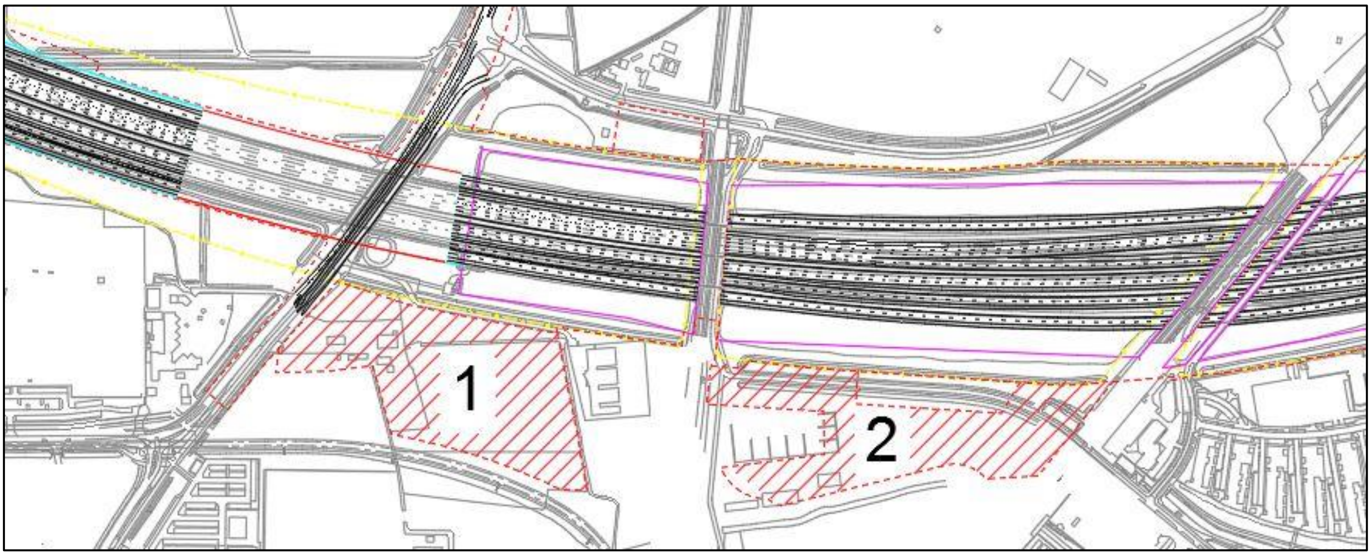

Figure 2.1: A digital 2D overview of the surrounding area with the two appointed construction sites [2].

2.1.2 Further limitations

5. The membrane structure in front of the structure at Amelisweerd is not considered in this research. The thesis is limited to the U-shaped concrete structure and the deck structure itself. However, choices in execution aspects could depend upon this membrane structure.

6. The extended parts of the motorway at both sides. All the tasks required to fulfil this widening are considered as feasible. If some special requirements at particular parts of the structure are necessary, then these requirements will be discussed. The same holds for the following subjects:

    a. Construction of the tension piles and the (underwater concrete) floor.

    b. Drainage of the building pit.

    c. The gravel layer on top of the underwater concrete floor and the structural concrete floor

    d. The supports, which will probably be structural walls.

7. The viaduct of the Koningsweg is incorporated in the design of The Green Connection. All temporary measures which are required in the design of this viaduct during construction are not considered in this thesis.

2.1.3 Structural design

8. The structural span without an intermediate support is 75 meters including the support distance.

9. The total width of The Green Connection will be 249 meters.

10. The load model on top of the structure consist of the governing load model resulting from the public garden. This load model is about twice as high as the normal traffic load according to the Eurocode. The load model resulting from the load of the public garden is determined via the ROK 1.4 . A soil layer of 1 meter is assumed with a permeable soil layer underneath of 0.2 meters will be required for the drainage system. A variable load resulting from the crowd load must be added. In short:

    a. 27 kN/m^2 of a permanent load resulting from the self-weight of the soil layers.

    b. 3.2 kN/m^2 of a variable load resulting from a crowd load.

11. Since no specific height requirements are stipulated, the height of the structure is not a boundary condition. However, the slender the deck structure, the better the structure can be incorporated in the surrounding area. Therefore, the height of the deck structure is still an important parameter.

12. An in-situ construction method is not applicable since it will cause too much traffic hindrance. Therefore, a custom-made prefab deck structure is required.

13. The maximum weight of a pre-tensioned precast (box) beam element is 280 tons, since lifting of these elements is a dominating factor.

The above-mentioned topics are numbered from 1 to 13. These topics are considered as important boundary conditions of the two global structural designs. Further conditions will be specified later if it seems to be necessary to confine the project even more. But first, the execution phases of the original design will be investigated.

3. Execution phases original design of Rijkswaterstaat

The execution aspects of constructing The Green Connection will be discussed if the design of Rijkswaterstaat is going to be realized. This design contains a deck structure with an intermediate support. The construction of The Green Connection can be roughly subdivided in 3 structural parts. It is most likely that first the extension of the U-shaped structure will take place, followed by the construction of the intermediate support and the deck structure. This sequence will be followed here as well. The execution aspects will be chronologically described for each phase. Then, it's judged whether the tasks are most likely executable, or the feasibility of the specific topic is quite in danger. In the last case, it could result in the infeasibility of the total project.

So, in this chapter the execution aspects will be discussed of constructing the reference project. It will outline some aspects to a preliminary design level, and tasks won't be dealt on a detailed scale. The aim of this chapter will therefore be to discuss several design aspects on a broad level to indicate if a 'showstopper' could occur. A showstopper is a task or an event which is enormously complex, and/or it has too much impact on the availability at the A27. Due to such a showstopper, the feasibility of the project is in danger or worse, it will be impossible to realize the project. Paragraph 3.1 emphasizes what is meant with the availability of the A27.

The execution aspects will be dealt on a broad level in order to scan the tasks and judging that a showstopper won't occur. If only one aspect will be considered in detail, it could be the case that a showstopper will occur within another aspect without noticing. It results in the fact that the other aspect which is dealt in detail won't be valid anymore, and the total project could suffer enormous financial losses and other damages during construction. This must be prevented at all times.

As said before, the current structure itself consists of an underwater concrete floor in combination with tension piles. At both sides of the motorway a concrete wall is present which retain soil and ground water. For more detailed information about the executional aspects during construction in the early days and the characteristics of the structural components, is referred to the Preliminary Study in chapter one. A digital version of the new opted lay-out of the design of Rijkswaterstaat is shown in Figure 3.1.

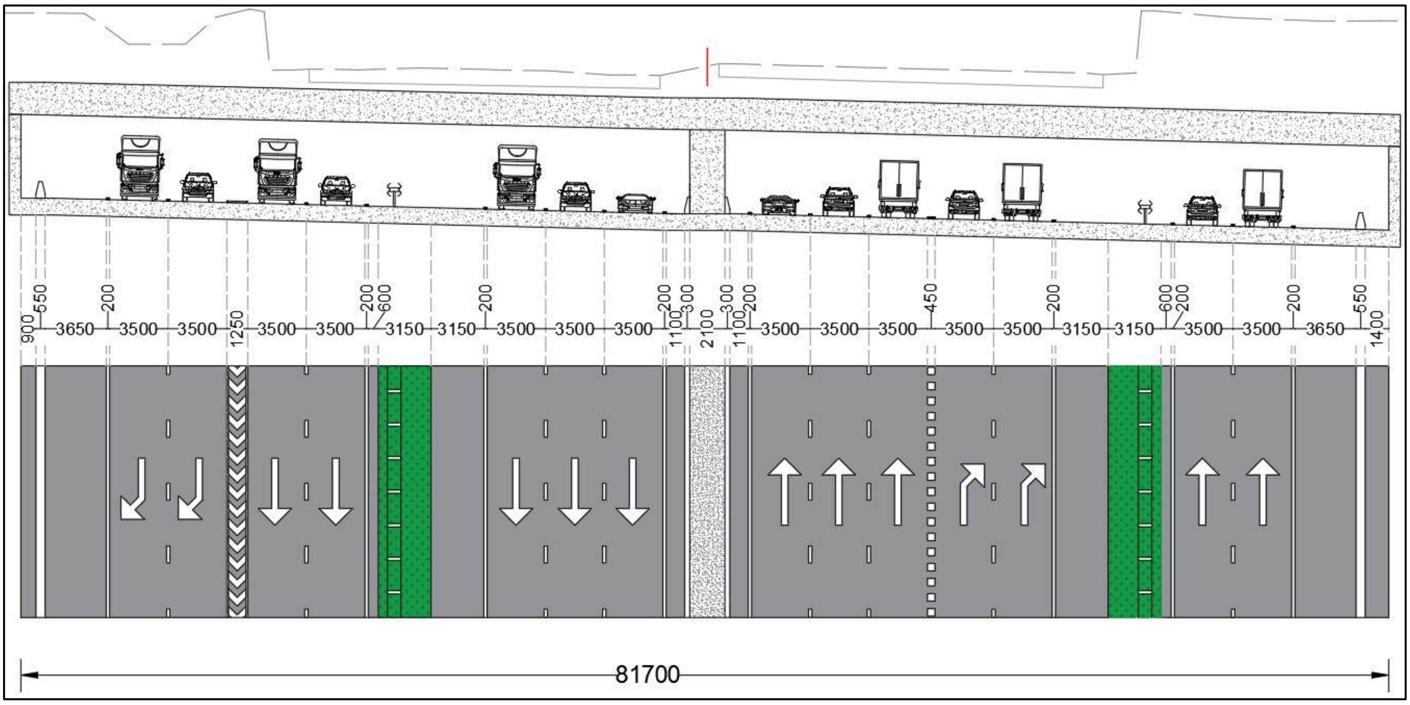

Figure 3.1: Digital version of the original design of Rijkswaterstaat

3.1 Availability of the A27

In the introduction of this chapter the expression showstopper is discussed. Several executional or structural aspects could be a showstopper, a task or event which results that the project can't be realized anymore. But what characterizes a showstopper? An example on structural level could be the lack of bearing capacity of the intermediate support even when the current foundation is strengthened. In that case no reliable structure can designed, and the project couldn't be realized.

However, from structural point of view, a reliable structure can be designed. It's just the question to what extend the execution aspects are considered, especially when a project takes place near a motorway. Several tasks could be a showstopper, if it causes too much hindrance to the motorway. But the main criteria will be to what extend the tasks are influencing the availability of the A27. In fact, every aspect which directly influences this availability could be a showstopper. But first, what is meant with the 'availability of the A27'?

The term 'availability of the A27' is quite arbitrary. This expression is broad. Moreover, it's difficult to judge the events which are incorporated with a decision. Therefore, it should be normalized in order to take in to account or to judge the impact related to the A27. At first, what is meant with 'availability'? Does it imply that the motorway can't be closed? Or does it mean that the availability of this motorway should be 100% during construction? Should the mobility of the motorists remain intact? And on the other hand, if some adaptions to the 'availability' will be approved, to what extend will these adaptions reach? Will it be approved by the authorities to narrow the original driving lane width and impose lower speed limits? The same holds for temporary closures, such as closures at nights or during weekends. It'll be like designing from no restrictions at all to designing in a more restricted environment, which implies that the amount of plausible solutions will reduce drastically.

After all, the main question of every executional task will be to what extend the task will influence the availability of the A27.

3.2 Components of the new to be designed structure

The construction tasks of the original design of Rijkswaterstaat can be roughly subdivided in three phases in order to realize The Green Connection. These phases are constructing the extended parts (floor and walls/side supports), constructing an intermediate support and constructing the deck structure. The executional aspects are mentioned followed by some words about the phasing in which the tasks should be executed.

3.3 General aspects

Before starting the construction phase, some general aspects must be installed to make sure that the building process will go by smoothly. In this project, it will be even more important since all the processes might influence the traffic flow at the A27. Therefore, it's of great importance to indicate the required facilities and all other kind of auxiliary structures and equipment. One must think of general facilities like a construction sheds. During breaks it's important that the employees can rest and have shelter from the environment. Sanitary facilities are unavoidable as well. Where to store these kinds of facilities and what amount of each is required? It all starts with an indication of the amount of workmanship.

Furthermore, it's important to appoint a location to the required equipment as well as the construction materials. It would be sensible to indicate such aspects, so they can be located before starting the construction phase. This will be beneficial and results in less hindrance to the surroundings.

Besides, during every phase in execution, the most important value is to pay attention to safety. Safety is the most important value and must be assured at all times. This holds for the motorists at the A27 and at the local roads, for all the employees and the local residents.

3.4 Constructing the extended parts

In this paragraph, the phasing of constructing the extended parts will be discussed. After this paragraph is obtained, the next sub question should be answered:

What could be an obvious way of constructing the extended parts?

In order to provide a proper answer to this question, the construction of the extended parts will be subdivided in multiple main tasks. These tasks are numbered, and the accompanying subtasks are briefly mentioned. Several ways of execution are possible and will be outlined. The aim of this paragraph is to demonstrate the feasibility of the mentioned tasks and not to explicate executional optimisations or variations. It will also indicate some points of attention which should be considered during the execution phase. The question rises if the applied method of execution implies a showstopper. Furthermore, some design considerations are mentioned.

The method of execution will be explained in an ordinary way of working, but at every task the main question arises: What kind of subtasks and measures are necessary to realize this project in a safe and economic way with as less hinder as possible? The point of departure is the current situation with the existing U-shaped concrete structure and an adjacent forest, which is schematically shown in the left part of Figure 3.2 on the next page. The right part of this figure will be obtained after a few tasks are executed, mentioned later on in this paragraph. Furthermore, the coming paragraphs will explain the method of execution in a chronological way. The last paragraph will explicate a brief risk analysis and some countermeasures of how to prevent unwanted events.

3.4.1 Phasing of the execution aspects

1. **Trees to chop down**

The starting point is an area next to the motorway full of trees, visualized in the left part of Figure 3.2. The blue line schematizes the ground water table and the dashed line shows the reference height of N.A.P. Half of the structure is drawn. According to the original plan of Rijkswaterstaat, about 15 meters at both sides of the motor way are necessary for the extended parts. At both sides a forest is present, so in general the tasks at both sides are the same. Therefore, only one half of the motorway is schematized.

First these trees have to be chopped down. Questions arise like how many trees have to be chopped down? What kind of equipment is necessary and how many? Where to store the equipment? And what is the capacity and the productivity of the equipment? Furthermore, when the trees are chopped down, what will be the purpose of these tree trunks. Will they be shredded or transported in one piece? In the last case one could think of access and exit roads, but where should these roads temporarily be realized?

Such questions are important to discuss beforehand in order to make sure the executional aspects run in a smooth way. The above stated questions and subtasks do not seem to be contravene with the feasibility, and from this point on, the starting point is a plain ground level without any trees next to the A27.

2. Sheet piling

Since a plain ground level is acquired, it's now time to start constructing a building pit. A common way to do that is with the help of sheet piles. A sheet pile wall has to be constructed for over about 500 meters, the length of the U-shaped concrete structure When constructing the existing U-shaped concrete structure in the early days, compartments of 40 meters in length were realized. These compartments were realized with sheet pile walls in cross direction, so segments were created with dimensions of 40 meters in length and 54 meters in width (the outer width of the existing U-shaped concrete structure). These compartments were used to construct the whole structure in a subsequent way of working. In the end, in total 11 of such segments were constructed to realize the U-shaped concrete structure.

When a sheet pile wall of these dimensions has to be realized, the sheet piles itself as well as the required equipment to install and supply the sheet piles are important aspects. Which sheet pile profile will be necessary and especially what length these sheet piles should have are important questions since the kind of equipment depends on the kind of sheet pile. The same holds for the way of applying them. Will they be driven in the soil or just vibrated? All these factors are important to judge what kind of equipment is required. When it's known, what are the dimensions of this equipment? How to position these enormous machines and what will be the capacity or productivity? These questions are only a few related ones.

However, these questions are always important if sheet piles are going to be installed. Therefore, it seems that constructing sheet pile walls in longitudinal as well as in cross direction won't result in problems related to feasibility. In Figure 3.2 the left part schematizes the starting point, while the right part shows the situation after chopping down the trees and sheet piling.

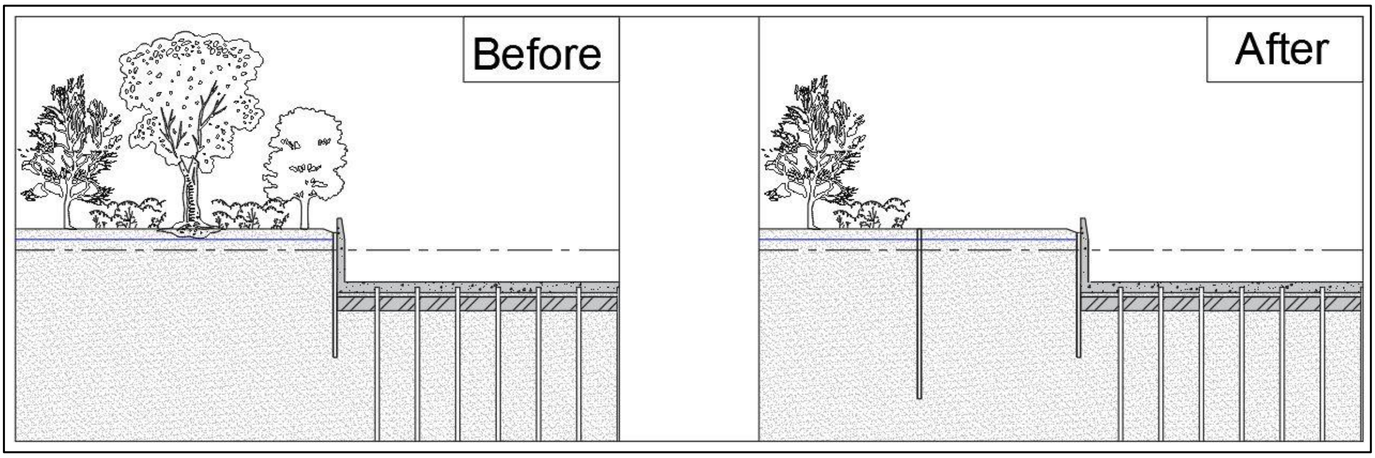

Figure 3.2: Extended part; from initial phase to phase 2.

3. Grouted anchors vs. struts and wales

Since the sheet pile wall is applied, the excavation of the pit can start. But, before the excavation, one must consider the stability of the sheet pile walls, the new one as well as the existing wall. In the early days anchors were applied to ensure horizontal equilibrium. It is expected that it will be necessary as well because it was also required in the early days. Normally, if the sheet pile wall itself doesn't have enough capacity to ensure horizontal equilibrium, two measures could be taken. Besides anchorage, also struts and wales could be applied.

First it must be proven that the sheet pile doesn't have enough capacity to resist the horizontal forces. If it lacks capacity, it must be strengthened with either anchorage or a strut and wale combination. Another possibility is to increase the length of the sheet pile wall in such a way that it will be just a stable solution. In some cases, it won't be sufficient because it results in too much displacement at the top.

In general, applying anchorage is riskier and more complex compared to applying struts and wales due to the uncertainty of soil parameters. Besides, the anchors are going to be situated beneath the forest outside the TB zone[1]. This reason itself could be a cause to choose for struts, especially when the building pit has a width of 15 meters which is a suitable measure for a strut and wale combination. The executional aspects should be considered when struts and wales are going to be applied. The building pit should be excavated a few meters, followed by the application of the wales. When this task is done, the struts can be assembled.

From this point on, a sheet pile wall in combination with struts and wales are applied. The next phase can be started, the excavation of the building pit.

4. Excavation

When the equilibrium of the building pit is ensured, the excavation can be continued. The excavation can be roughly done in three ways. Excavating from outside the building pit, from inside the pit and from above with a kind of engineering traverse [3]. Such an engineering traverse is a bridge which is constructed upon the sheet pile walls above the building pit. During the construction phase, it can move across these two walls. Upon this traverse, all kind of materials and equipment could be stored and used.

One must consider the fact that struts are assembled, so a dredging vessel which was applied in the early days won't be suitable. The struts however, won't result in big problems in combination with the engineering traverse.

The equipment which will be used is of great importance. What kind of equipment is suitable will be the question. Where to position it and what will be the effect on the A27? Are these machines stored at the to be designed building pit, at the A27 or at the appointed construction site outlined in the previous report? Does the surrounding ground level have enough capacity to bear the large forces resulting from these machines? And what to do with the excavated dirt? It'll be most likely that access roads are necessary for trucks to haul the excavated dirt. And if access roads are prohibited all these transport movements must go via the main motorway, the A27. Another possibility could be to use the engineering traverse to haul the excavated dirt, if applicable.

At the end of this phase, the building pit is excavated to a depth of about -5.7 N.A.P., which is the height of the bottom side of the existing underwater concrete floor at its deepest point. The struts and wales are applied as well. The width of the building pit should be about 15 meters confirm the original plan of Rijkswaterstaat. This is visualized in the next figure.

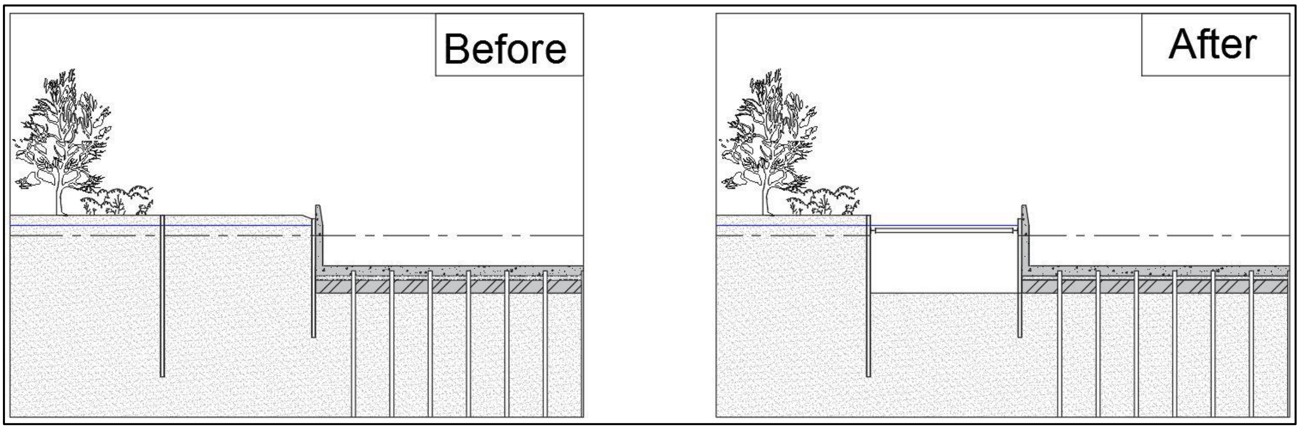

Figure 3.3: Extended part; From phase 2 to phase 4.

[1] In the Netherlands, no construction activities may take place beyond this border.

5. Foundation piles

The piles must be constructed, otherwise the underwater concrete will float. It has to counteract floating during construction, and it must also bear large vertical forces during use-phase. In the early days squared 400 mm prefab piles were applied with lengths varying from 20 to 21.5 meters. The lengths depend on the soil conditions. 20 meters of length will be the starting point here.

These piles can be applied via multiple ways, for instance the piles can be driven into the soil. This process can be done in dry as well as in wet conditions. When it's chosen to apply the piles in dry conditions, the piles must be driven into the soil before excavation. If it's chosen to install the piles in wet conditions, the driving equipment must be used upon pontoons or at the engineering traverse above the building pit. Important aspects of the equipment are weight, range, dimensions and capacity.

Which method will be executed is not considered here. The fact is that it is possible to install these piles. Further research is necessary to choose which method will be more beneficial. This procedure will be outside the scope of this thesis.

When the design of these piles is realized, it must be considered that supports are going to be constructed at the sides. In the original design the function of the walls was only to retain the soil pressure. The new situation consists walls again, but these walls must also function as supports. It implies large bearing forces which results in a severe support reaction. The support must resist this force. Therefore, a more though foundation has to be constructed beneath these walls compared to the existing foundation of the U-shaped concrete structure. This could be realized with tougher concrete piles in combination with a kind of strip foundation upon these piles. However, both options will be possible to construct.

However, if a strip foundation must be constructed, it could possibly result in the necessity to widen the building pit if this support wall has to be centrally constructed upon this strip foundation. This topic is dealt in paragraph 3.4.2. Emphasizing this topic is meant to realize that it's an important and differs from the existing structure, since the new to be realized wall should also function as a support.

6. Underwater concrete floor

After installation of the piles and excavation of the building pit, a dry working space must be acquired. A watertight layer is necessary to ensure a dry building pit. This watertight layer will be an underwater concrete floor. This layer will have a thickness of at least 1 meter. Points of attention will be the mixture itself, but mostly the equipment required and its logistics. Again, this task can be executed via the engineering traverse but also via a concrete pump/truck. In the early days it was done via a Hobdobber, which is a kind of frame across the building pit which could smoothly apply the underwater concrete to its location.

Concludingly, pouring this floor will be definitely suitable. A point of attention is the water tightness of this layer since it will also be poured against an existing sheet pile wall.

7. Dewatering/drainage of the building pit

After the underwater concrete floor is hardened, dewatering of the building pit can start. The discharge of the equipment is important in order to estimate the required time and the amount of water. Furthermore, where to keep the discharged water? Could it be drained to an existing sewer nearby or could it be repumped to the adjacent forest? Both options seem to be feasible.

After a dry building pit is acquired, one must check it on leakage points. If these points occur, they must be repaired because a dry workspace is required for further construction activities.

The next figure shows a schematization after excavating the building pit and after the subsequent tasks; piling, the underwater concrete floor and dewatering.

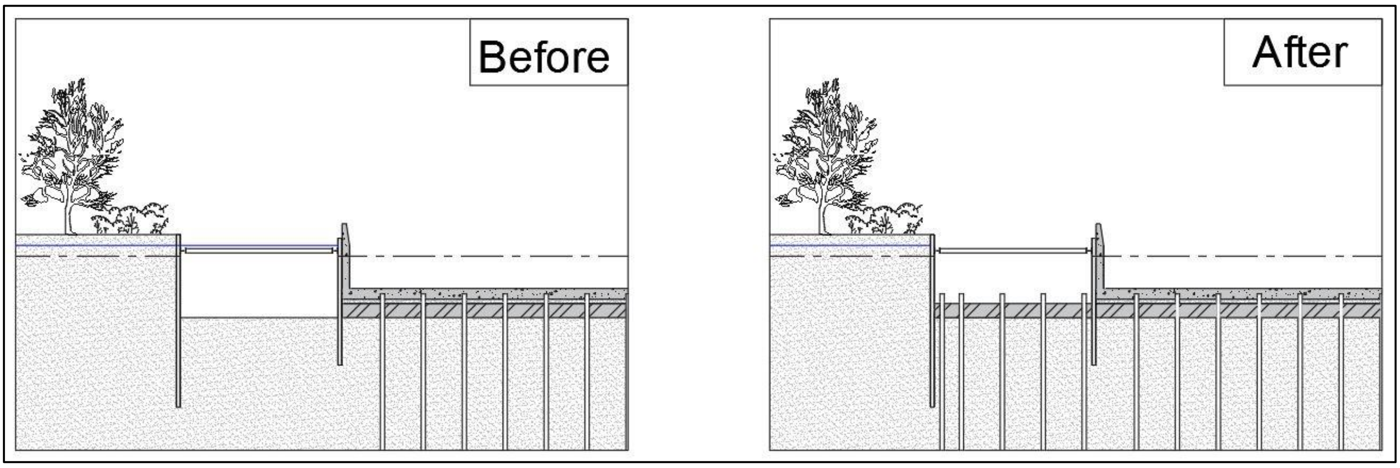

Figure 3.4: Extended part; From phase 5 to phase 7.

8. Gravel layer and structural concrete floor

After achieving a dry working space, a gravel layer must be constructed followed by the structural concrete floor. In the early days, an average layer of 0.3 meters of gravel was applied followed by a meter of reinforced concrete. This concrete layer was constructed under an angle due to water run-off issues. It's assumed that the same tasks are necessary to execute.

One important aspect is the supply of materials in the building pit. One must think of a substantial amount of gravel, reinforcements and concrete. This could be done either via the engineering traverse or via the A27. If the last possibility is chosen, the materials must be hoisted across the existing wall in the building pit. First questions are related to the quantity of materials as well as the quality. Furthermore, important aspects are again the required amount of equipment and where to position it.

Besides the mentioned tasks, it seems to be feasible to construct the two layers. Important aspects are related to executional aspects, such as where to put the cranes if no engineering traverse is used and what is the impact of positioning the cranes to the surrounding area.

9. Structural walls/supports

After constructing the gravel layer with on top the reinforced structural concrete floor, the next step is to construct the walls. In the existing design the primary function of the walls was to retain the soil. However, in the new design it must also function as a support for the deck structure. It is estimated that walls should have a width of about a meter. The height of the walls must be at least 5 meters, because of the clearance of the motorway.

In the early days the structural concrete walls were poured in three casts to obtain a height of about 6 meters. This kind of execution method can be used here as well with the help of climbing formwork or something like that. What must be kept in mind is the presence of the struts. During construction of the existing walls anchors were used to ensure the overall stability of the sheet piles. The walls were constructed to a height just below the attach point of the anchors at the sheet piles. Then, the anchors were removed, and construction of the walls was completed afterwards.

The same method of execution can be used when constructing the new structural walls because of the presence of struts. Therefore, the supports can be built to almost the height of the struts. The struts can be removed, and construction of the wall can be continued and finished.

Points of attention are again related to materials and the equipment. When constructing the walls, equipment can use the new designed structural floor as access ways or to store materials. Before this new auxiliary road can be used, the sheet piles in cross directions has to be removed. The next figure schematized the situation after constructing the structural floor and a part of the side wall.

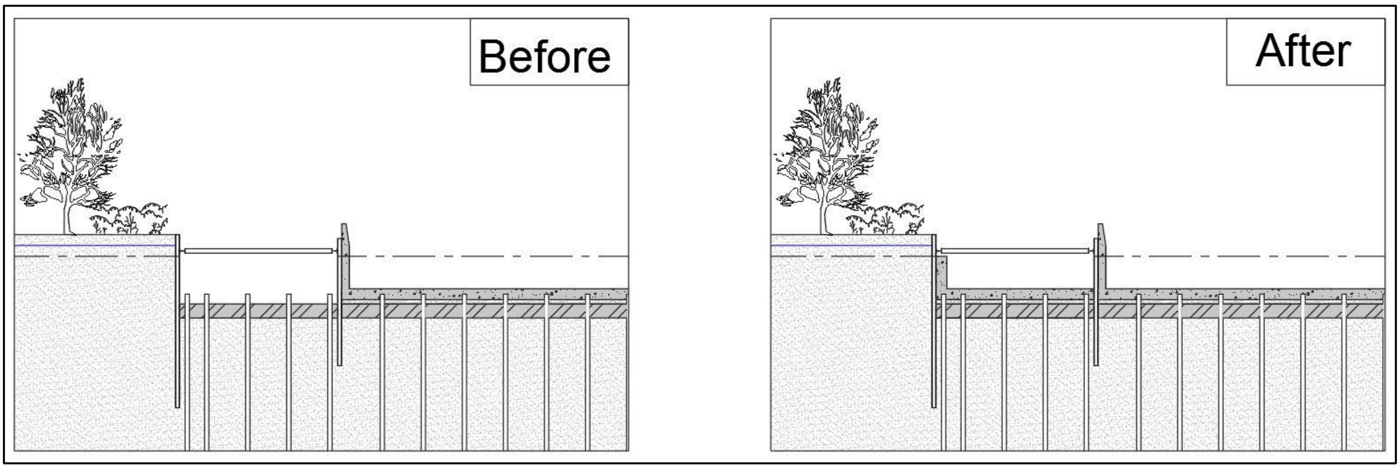

Figure 3.5: Extended part; From phase 8 to phase 9.

10. Struts and wales

The situation visualized in the right part of Figure 3.5 is the starting point. The supports must be built to their final height. To realize this final height, first the struts and wales must be removed which should be done with cranes. Important aspects are the positioning of the cranes to hoist the struts and the wales as well as the trucks which are necessary for transport. This equipment can be positioned at the new extended part of the motorway, because it seems to be a good place with less hindrance to the surroundings. It is assumed that the engineering traverse can't be used anymore, since the struts ensure the horizontal equilibrium.

After removing the struts and wales, the support wall can be constructed to the final height.

11. Existing sheet pile wall and concrete wall.

When the struts and wales are removed, and the construction of the supports has finished, the last part to complete the project must take place. This task entails removing the existing wall of the original U-shaped concrete structure. The existing wall consists of a sheet pile wall and a structural concrete wall. Both walls vary in lengths.

To spare the A27, it seems to be a good solution to execute the tasks from the inside of the building pit upon the new constructed road. The sheet piles can't be pulled out, since the underwater concrete floor is casted against it with some welded shear studs for extra friction. Therefore, the sheet piles have to be welded and removed. A crane is required to keep the sheet piles stable during welding so that these piles won't fall. After welding these sheet piles, these pieces can be temporary stored at the extending part as well. A truck trailer combination can easily transport them. These tasks are quite commonly executed, and it's therefore expected that it won't result in feasibility problems.

Removing a structural concrete wall of about 0.7 meter in thickness is slightly more complex. This tough wall could be crushed with special equipment. Some disadvantages of this method are that it's quite time consuming and a lot of noise as well as dust pollution occur to the surroundings. However, several measures exist in order to mitigate these negative aspects. Again, the amount of equipment, capacity and productivity are important aspects to discuss. Besides, it'll be required to install a safety wall/net at the inside of the existing wall. If concrete pieces will fall off during crushing of the wall, it won't fall on the A27 itself.

After removing this wall, the last executional tasks are fulfilled to realize the extending of the U-shaped concrete structure. The final result is schematized in the right part of Figure 3.6.

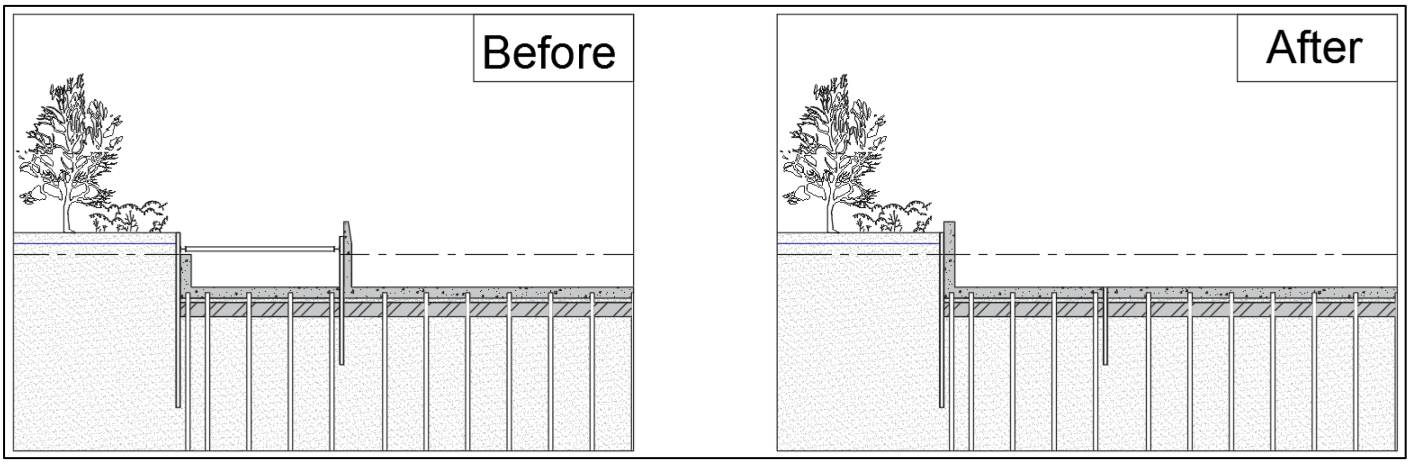

Figure 3.6: Extended part; From phase 9 to final phase.

3.4.2 Discussion extended part

The previous paragraph outlined a possible method of execution in constructing the extended part. Realizing a building pit with sheet piles, tension piles and an underwater concrete floor is an ordinary way of working. Therefore, it's expected that these tasks won't result in big problems. However, some assumptions need further attention, especially some design considerations and detailing. Although the aim of the paragraph is not to provide a final design regarding detailing, some of them are emphasized since these issues could have big impact upon the construction time/execution aspects.

The first interesting point starts with applying the sheet piles at step 2. The old existing sheet pile wall was combined with grouted anchors. As said before, these anchors should have been removed, but from practise it's learned that in some cases this task has been forgotten. It results in major problems when constructing the new sheet pile walls. Therefore, it could be sensible to excavate a small trench behind the existing wall to indicate if these anchor rods are removed. The grouted bodies of the anchors are normally constructed to larger depths, but further research is recommended to determine this topic. Nevertheless, it's expected that constructing a sheet pile wall is feasible.

These sheets are going to be constructed exactly 15 meters next to the existing old sheet piles. However, it could be the case that an extra strip foundation is required, since one pile row might have too less baring capacity. A large support reaction will be present which is in the order of magnitude of 1700 kN per running meter, calculated via the same procedure as explicated in paragraph 3.5.2. If the distance between the new to be constructed piles is 2 meters, it means that 2 piles are required instead of one, since the capacity of the piles are about 2500 kN. It implies that a kind of strip foundation is required which means that extra space in width is necessary. In fact, this measure results in almost 2 meters of extra width of the building pit. Figure 3.7 schematizes this thought. The left part is a normal piled foundation, while the right figure schematizes the wall support with a kind of strip foundation. Extra piles are necessary to be realized. It is also shown that if such a foundation is required, the building pit should be enlarged. This will contravene the original plan of Rijkswaterstaat since the total width is a very important parameter. The figure is drawn to scale.

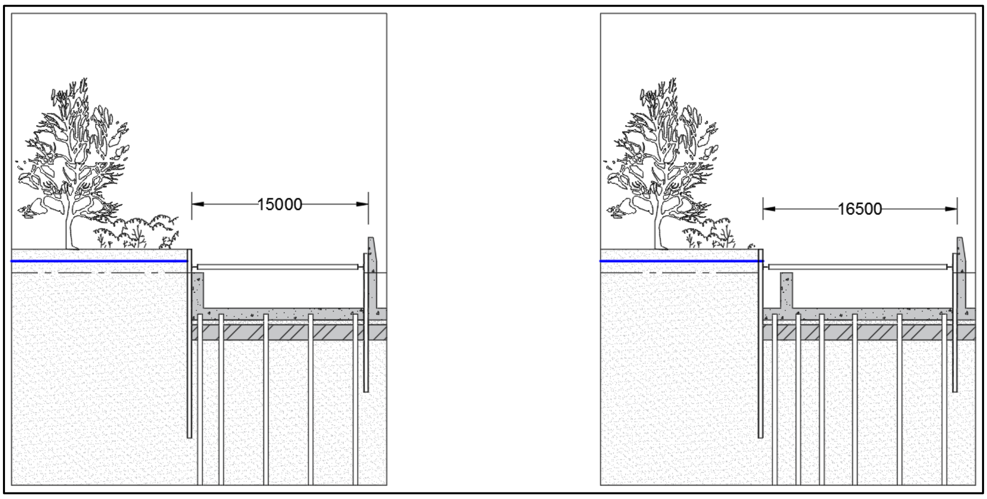

Figure 3.7: Difference between a normal piled foundation and a strip foundation with extra piles.

It might be considerable to use another design in constructing this kind of a building pit, because a large wall support must be realized as well. One could choose to use a combi-wall or diaphragm wall instead of an ordinary sheet pile wall. These other types of walls can function as a vertical wall to obtain a building pit during execution as well as a support wall in final stage.

Furthermore, a lot of variations exist in further execution of the extending part. As said before, it's not the aim to provide an optimal execution method, but just a method to judge the feasibility. The main tasks are quite common. But some points of attention do exist. It's just the question of these point can change the ordinary way of working and could lead to feasibility problems of the total project. One of these points are the way of attaching the new extended part to the existing structure and assure water tightness. Another important point is difference in compressive and tensile forces, especially at the supports. The main part of floor is exposed to floating forces while the supports are just subjected to high compressive forces resulting from the load of the deck structure. The settlements of the piles will be in opposite direction which causes cracks in the floor structure. This will lead to water tightness problems. Proper detailing these areas could avoid such problems. More attention should be paid to this topic, but it lays outside the scope of this thesis.

Concludingly, it can be stated that this paragraph discusses and visualizes an obvious way of working. The question to be answered after this paragraph implies:

What could be an obvious way of constructing the extended parts?

With the help of the 11 tasks a proper answer is provided to the sub question. The next paragraph sketches a brief risk analysis to indicate some unwanted events and how to mitigate these aspects.

3.4.3 Risk analysis

The previous paragraph outlined in total eleven tasks. Some of them are more complex or 'risky' than others. This paragraph is meant to provide an overview of the risks related to the execution and structural aspects. A risk analysis is provided in order to indicate which relevant treats or unwanted events could happen and what kind of risks are accompanied with these relevant treats or events. Such an analysis will outline the treats. Also, some measures which can be taken to mitigate these risks are provided. It could be a kind of tool to oversee the consequences.

The risks are categorized per construction task. These tasks are numbered with 1-11 just like in the previous paragraph. Then, the risks will get two marks. One for the probability of occurrence and one for the impact if the treat will happen. This last number will indicate the impact of the consequence. A "risk" can be best described according to the following "formula":

$$Risk = Probability * Consequence$$

The next figure shows a matrix for this project.

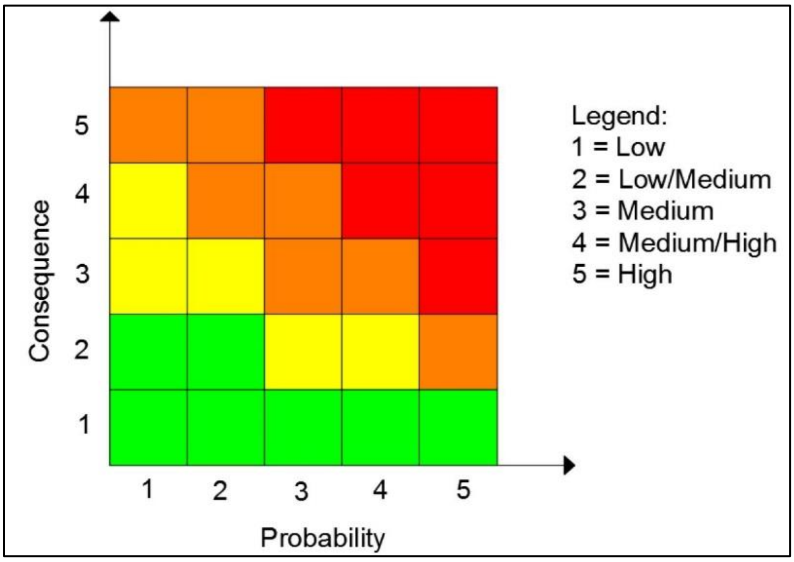

Figure 3.8: Risk matrix [4].

The following two pages show a table with the listed risks. These risks are judged quantitively and aimed to indicate the most severe treats related to the executional aspects. Although this process is quite subjective, the result provides a first indication of the unwanted events.

Nr.	Category	Cause	Effect	Conse.	Prob.	Risk	Countermeasure
1.1	Trees to chop down	Large tough to handle trees	Trees are falling upon the A27	5	2	10	Make a safety structure/net along the A27
2.1	Sheet piling	Large pieces of garbage in the soil	Sheet piles can't be driven into the soil	3	2	6	Extensive soil investigation
2.2		Soft ground surface	Subsiding equipment	2	2	4	Use auxiliary structures such as rubbing plates and dragline beams
3.1	Struts and wales	Insufficient cross section/large loads	Buckling of the struts	4	1	4	Thorough calculation
3.2		Strut and wale combination doesn't fit	Time delay and reorder the materials	2	1	2	Better modelling beforehand of the building pit
4.1	Excavation	Collapse of engineering traverse	Traverse falls into the building pit	5	2	10	Capacity check of sheet pile wall and better modelling beforehand
5.1	Foundation piles	Too dense soil	Piles can't be driven into the soil	3	3	9	Better soil investigation
5.2		Driving equipment too heavy/large for engineering traverse	Execution method must be adapted	5	2	10	Investigate capacity engineering traverse and equipment. Otherwise drive piles before excavation
6.1	Underwater concrete floor	Cracks in UWC-floor	Leakage, building pit can't be (fully) drained	3	3	9	Proper concrete mixture, adequate dimensions compartment
6.2		Insufficient attachment of concrete to piles/sheet piles	Leakage, building pit can't be (fully) drained	4	2	8	Apply fixation points at sheet piles and piles and/or use divers as quality control during pouring
6.3		Floating and compression forces	Cracks	3	2	6	Apply a membrane layer or longitudinal joint between strip foundation and structural floor

Nr.	Category	Cause	Effect	Conse.	Prob.	Risk	Countermeasure
7.1	Dewatering/drainage of the building pit	Low capacity equipment	Can't achieve a dry building pit and/or time consuming	2	1	2	Extra drainage equipment
7.2		No sewer in neighbouring area	Can't discharge the water via the existing sewer	3	2	6	Pump water to the adjacent forest to the A27 or transport by truck with water tanks
8.1	Gravel layer and structural concrete floor	Traffic jams	Not enough supply of materials (concrete)	2	2	4	Consider a stock of materials/ concrete trucks
9.1	Structural walls/support	Lack of capacity	Insufficient design	5	1	5	Design calculations and consider alternatives of support
9.2		Inaccurate way of working	Interference with struts	2	3	6	Proper modelling before construction will prevent such events
10.1	Struts and walls	Too tight due to small deformations of the sheet piles	Struts and wales can't be easily removed	2	2	4	Use heavy equipment or hydraulic devices to jack it
11.1	Existing sheet pile wall and concrete wall	Lifting cable breaks into pieces	Sheet piles will fall	2	1	2	Use an extra cable or apply cable inspection before lifting
11.2		Crushing the existing concrete wall	Dust and noise pollution	2	3	6	Use water during crushing and work only during the day

Table 3.1: Overview risks.

The risks from Table 3.1 are numbered and judged about the impact of the consequence and the probability of occurrence. The result is implemented and visualized in the risk matrix of Figure 3.9.

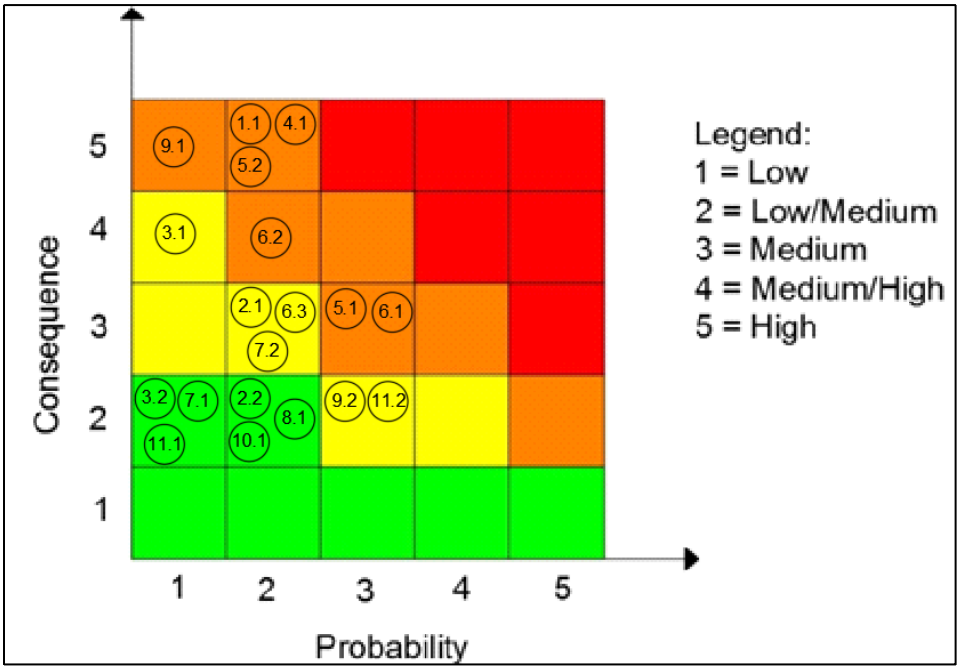

Figure 3.9: Risks implemented in the risk matrix.

It appears that the most important risks are the ones with the highest scores. The weakest treats are situated in the green area, while the heaviest ones are in the red area. In between are two colours, yellow and orange. Each colour in this matrix indicates the risk in a certain way. The risks are assigned to a colour judged on impact and possibility of occurrence. The highest treats/risks are 1.1; 4.1 and 5.2 of Table 3.1. Risk 5.1 and 6.1 are important too. That can be judged from the colour at which each risk is located in the risk matrix in Figure 3.9.

Managing the risk is a study in itself and won't be considered here thoroughly. What must be clear is that risks located in the red area are a big treat to the project and something must be done with it. Risks in the red area could imply casualties or enormous financial consequences. These risks must be eliminated. However, such risks do not occur in this execution method, but risks in the orange area do occur. It's important to consider these risks. One can insure these risks at a certain insurance company, so that if such a risk happens, the financial losses are paid by this company.

These risks can be greatly reduced by first being aware of the possibility of occurrence and unwanted consequences. Besides, one can take the countermeasures or at least consider these measures, also explicated in last column of Table 3.1. Via these measures the impact can be reduced or even totally prevented.

As said before, this manner of analysing risks is quite subjective with rating probabilities and consequences. The outcomes or scores are discussable. However, the approach outlined in this paragraph is an interesting way to obtain a first impression of the important risks and the ones who should be mitigated.

3.5 Design considerations in constructing the intermediate support

At first sight, it would be sensible to start construction of this middle support after extending the U-shaped concrete. It results in extra space at both sides of about 10-15 meters. In that case more space can be used to retain the amount of space for driving lanes because a building site must be realized in the middle of the motorway. This site requires a substantial amount of space which goes at the expense of the amount of space meant for driving lanes. It's inevitable to create such a site since a support wall must be constructed. Therefore, this paragraph is obtained to answer the following sub question:

How to construct an intermediate support?

Just as emphasized in the beginning of this chapter, every task will be considered roughly. Furthermore, it's important to investigate to what extend the construction activities will influence the availability of the A27.

3.5.1 Design approach

If this intermediate support is going to be constructed, the tasks can be roughly subdivided in constructing the superstructure and substructure. The superstructure consists of a concrete wall support. No further attention will be paid to this part since it's quite straightforward to construct. However, the dimensions of the substructure depend on the height of the support reaction which will be acting or could be acting on it in final stage.

In general, three possibilities could occur in order to adapt the current foundation properly. The first one implies that the current foundation have enough capacity to resist the design forces. No further adaptions should be applied to the foundation. The second possibility could be the case that the foundation must be locally improved, such as applying injections or constructing an extra strip foundation to mobilize extra foundation piles. The third option will be that the current foundation is not capable of resisting the support reaction. Some extra compression piles must be constructed to increase the capacity of the foundation. It implies that a kind of gap or trench must be constructed in the structural floor and the underwater concrete floor for more than 250 meters and several meters in width in order drive compression piles. This complex task is unfavourable, but it might be unavoidable.

This topic must be dealt to judge which of the three solutions will be the reality. Solution one is most advantageous since no adaptions to the current foundation are required. The capacity of the current foundation must be determined to judge whether it will be possible. Furthermore, the phenomenon 'redistribution of forces' will be dealt with, and in what manner it could be applied. If the first and second option are not a suitable solution, the third option must be outlined with all the accompanying tasks and design criterion. Concludingly, the design approach can be summarized in the following flow chart, visualised in the next figure.

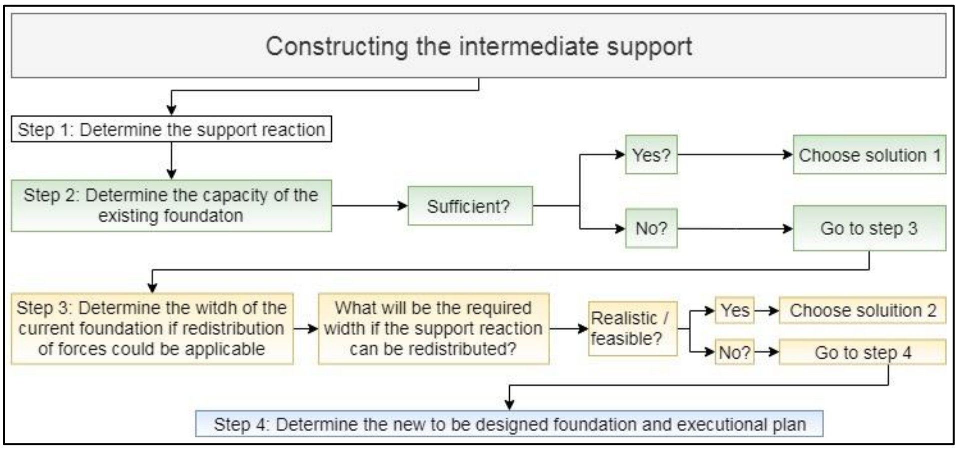

Figure 3.10: Design process of the intermediate support.

3.5.2 Support reaction

In order to judge whether the existing foundation will be safe in resisting the load, the design support reaction in ultimate limit state must be calculated. In the preliminary study is found that the design external load upon The Green Connection will be 41.6 kN/m2. The deck structure itself is still unknown, but one must estimate the weight per square meter to obtain the support reaction. The preliminary design of a custom-made box beam of 75 meters had a slenderness of about 25. Therefore, it's assumed that the deck structure shall have a comparable slenderness, which implies a custom-made beam comparable to an existing beam with a height of 1.7 meters. Therefore, the design value of the deck structure is estimated to be 30 kN/m2, which seems to be reasonable.

Normally, this kind of deck structure is simply supported, which implies the following mechanical scheme is applicable schematically visualized in Figure 3.11. The distributed load is also shown and consist of permanent load as well as the external permanent load and variable load.

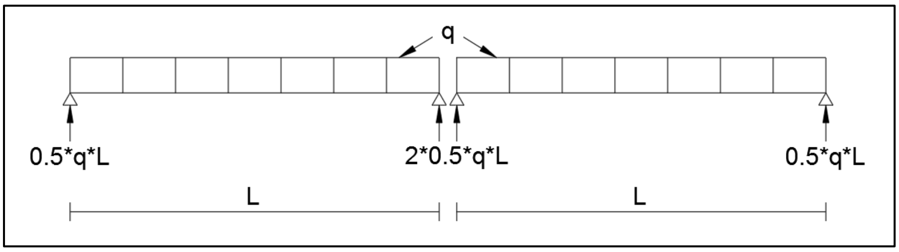

Figure 3.11: Mechanical scheme.

Besides the external load and the load resulting from the permanent deck structure, also the self-weight of the wall must be taken into account. A wall of 1 meter wide and at least 5 meters high is assumed to be a reasonable support. In short, the following components are taken into account in determining the support load exerting upon the foundation:

$Support\ reaction\ due\ to\ external\ load$:	$2 * 0.5 * 41.6\ kN/m^2 * 42\ m \cong 1750\ kN/m$
$Support\ reaction\ due\ to\ permanent\ load$:	$2 * 0.5 * 30\ kN/m^2 * 42\ m \cong 1250\ kN/m$
$Self\ weight\ support\ wall\ of\ at\ least\ 5\ m$:	$5\ m * 1\ m * 26\ kN/m^3 * 1.4\ [-] \cong 200\ kN/m$
$The\ design\ support\ reaction$:	$3200\ kN/m$

REMARK: Normally, the resulting upward pressure below the foundation must be taken into account. It is resulting pressure of the water pressure and the self-weight of the foundation itself. A basic assumption in the Preliminary Study is a water pressure below the underwater concrete layer of 67 kPa. The foundation is composed of at least 1 meter of this underwater concrete, a layer of gravel/sand depending on top of the concrete due to water runoff issues in cross direction, and a layer of a meter structural concrete. If the weight of these layers is calculated, it has almost no resulting pressure if the water pressure is taken into account. However, the composition of the foundation is known, but the exact thickness of the layers isn't. Besides, it's quite uncertain if the assumed thicknesses are accurate, since the reference documents show minimum values. Nevertheless, the resulting pressure will be in the order of magnitude of 10 kPa pressure which is the same as 10 kN/m2. In this global design some values are assumed and estimated to provide an estimation of the load. As seen above, the ´uncertain´ pressure of 10 kN/m2 is neglectable if it's compared to the support reaction of 3200 kN/m and therefore this pressure will be left behind.

In the middle of the existing U-shaped concrete section a design line load of 3200 kN/m must be resisted. It seems quite unlikely that the current foundation can resist this relatively high load, otherwise this foundation would be enormously over dimensioned in the early days. However, this assumption can't be taken for granted and must be proved.

To prove this statement, the capacity of the existing foundation will be calculated, the second step depicted in the flow chart in Figure 3.10. Then, the step to take is to determine the pile capacity at the location of the support. Therefore, first some detailed information is outlined about the existing structure, such as centre to centre distances of the piles, soil conditions and so on.

3.5.3 Detailed information of the existing foundation

The capacity of the foundation can be determined if extra detailed information is available. In Figure 3.12 a snap shot of a drawing of the existing foundation is visualized [5]. Several important aspects are shown. The blue lines are sheet pile walls. These walls are located at the perimeter of the U-shaped concrete structure as well as in cross direction. The function of these screens in cross direction was to divide the structure in compartments which was beneficial during casting of the underwater concrete floor. And, the configuration of the pile grid differs within each compartment. The red line shows the centre line of the U-shaped concrete structure. At this location, the middle support has to be realized for The Green Connection. Also, two cone penetration tests, CPTs, are found at the location of this support and indicated in the figure. These two CPTs are necessary to determine the soil conditions and thus the pile capacity. Furthermore, the green shaded area will be the surface of The Green Connection which will cover the U-shaped structure. This area is only drawn to show which part of the existing structure will be covered up, so that the length of the support wall will be visible.

A lot of piles are drawn in the figure. These piles have different grids and different capacity due to length differences and variations in soil conditions. The piles itself are prefabricated pre-tensioned squared 400 mm piles which were driven into the soil. The length varies from 20 to 21.5 meters.

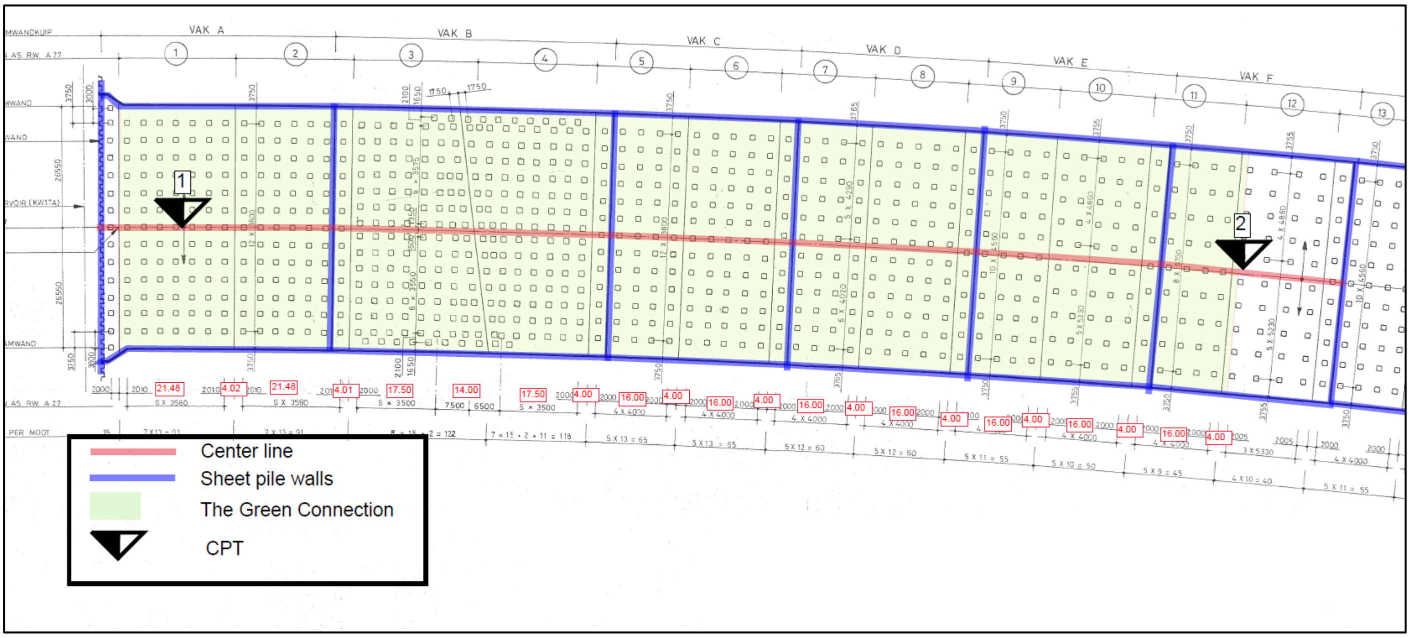

Figure 3.12: Overview of the existing foundation of the U-shaped concrete structure [5].

Pile capacity

As seen in Figure 3.12, two CPTs are found at the location of the to be constructed middle support. The location of these CTPs is indicated in this figure as well. With the information of the piles and the estimated height of the pile head, the capacity is determined. This process is done with the help of a software program D-foundations. A report of the pile capacity nearby CPT 1 and 2 is attached in Appendix D. Some differences in length and capacity are present, since the CPTs show some variations in soil conditions. However, the minimum arithmetic value of the piles at CPT 1 was 2838 kN and at CPT 2 2155 kN. The main question still rest, does the foundation have enough capacity to resist the 3200 kN/m support reaction?

It must be stated that the foundation and especially the pile grid differ at every compartment. In some cases, the grid even differs in the compartment itself. The pile grid at CPT 1 is 3.58 to 3.80 meters and the grid at CPT 2 is 4.00 to 5.70 meters. The centre to centre distances 3.80 and 5.7 meters are in cross direction of the structure. Therefore, to get insight in the capacity of the foundation, two calculations are performed at both CPTs.

The piles are 3.58 meters centre to centre in the direction of the support, the driving direction of the motorists at the A27. This implies that if no redistribution of the piles takes place one pile must carry about $F_{Ed1} = 3.58\,m * 3200\,kN/m = 11500\,kN$, while the capacity of one pile at the location of CPT 1 is 2835 kN. This situation is schematically visualized in Figure 3.13. This figure shows a part of the structure around the CPT number 1. A part of the support wall is also drawn for imaging. Nevertheless, the capacity of the piles isn't sufficient.

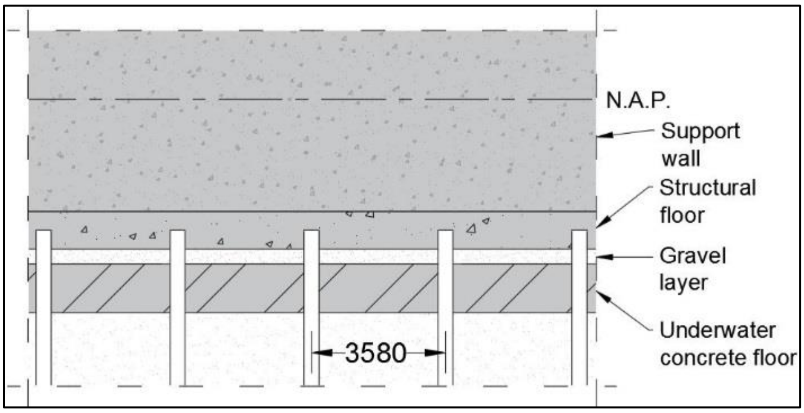

Figure 3.13: Schematic view of the existing foundation.

The same approach is followed with the piles at CPT 2. These are centre to centre 4 meters instead of the 3.58 meters. The design support reaction will be $F_{Ed2} = 4\,m * 3200\,kN/m = 12800\,kN$, which is even higher. It implies that at both locations the piles don't have enough capacity.

Then, step three of the flow chart in Figure 3.10 is going to be considered. What if the force in the support can be redistributed? If this phenomenon is taken into account with the existing pile grid, could the foundation resist the force? To illustrate this thought, Figure 3.14 is drawn. This figure shows a cross section in cross direction of the A27 at the CPT 1. If 11 500 kN must be resisted by one existing pile row which is 3.8 meters centre to centre in cross direction, at least 5 piles should be mobilized. It implies a width of the foundation of 4 centre to centre distances, which will be a kind of strip foundation of a width of 3.8 * 4 = 15.2 m that should function together as one block. In Figure 3.14 a red dashed line is drawn under an angle of 45 degrees. As visualized, this part could probably be mobilised if some injections are implemented in the sand layer. In that case the three layers will interact. After sufficient reinforcement of this intermediate layer, 3 piles might be mobilized when the part of the foundation could function as a block. It seems that mobilizing 5 piles together is impossible.

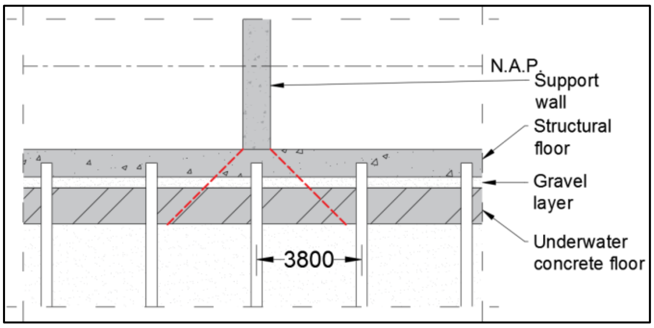

Figure 3.14: Schematic view of the existing foundation at CPT 1 in cross direction.

Besides, at the location of CPT 1, the pile grid is the closest grid of the total structure. If the phenomenon 'redistribution of forces' is investigated at the location of CPT 2, the width of the pile row which must be mobilized is even larger. The centre to centre distance in cross direction is 5.7 meters instead of 3.8 meters as depicted in Figure 3.14. Besides, the pile capacity is less due to the soil conditions and the centre to centre distance in the driving direction of the motorists at the A27 is 4 meters instead of 3.58. It implies that 12 800 kN should be resisted by 6 piles with a bearing capacity of 2155 kN, which means a width of about 5.7 meters * 6 = 34.2 m should be mobilized and interact as one block. Mobilizing almost 35 meters of existing foundation isn't feasible and doesn't make sense at all!

Concludingly, it can be stated that the existing foundation and pile grid isn't able to resist the high support load. Even if the redistribution of forces is applied, it still cannot resist the force of 3200 kN/m. In short, the pile capacity is insufficient, and redistribution of forces is not an option if almost 35 meters must be mobilised. Therefore, a new reliable foundation must be realized if an intermediate support is going to a part of The Green Connection. It implies that extra piles must be constructed in the existing foundation, step 4 of the flow chart in Figure 3.10.

3.5.4 The new to be designed foundation

As said before, the existing foundation isn't capable to resist the support reaction of the intermediate support. Therefore, the existing foundation should be adapted sufficiently. It implies another conclusion: Piles must be installed. But how many piles are required in order to achieve a reliable structure? And related to the execution aspects, what should be the width of the new to be designed foundation block? Besides, is it even possible to install the piles in the middle of the motorway?

In general, two approaches could be considered. The first one will take the structural point of view as starting point. It implies that first the required foundation is considered. The amount of piles to be installed is an important aspect. Via this approach the width of trench in the existing foundation could be determined. From there on, the execution aspects are considered, and the feasibility will be judged.

The other approach consists of taking the method of execution as a point of view as. The starting point should be to consider the restrictions related to the availability of the A27. Considering this information, a feasible execution plan must be thought and from that point of view the structural aspects should be implemented.

Both the approaches seem to be reasonable. In the end, the final solution should be the same, because with both approaches the same topics are dealt. Since no further information is specified about the availability of the A27 or other aspects related to hindrance to the surrounding area, the first approach is followed. It implies that first the structural parts are going to be emphasized, followed by the execution aspects.

Strengthening the existing foundation from structural point of view implies that extra piles have to be installed. From the point of view of the execution method, it's very important to determine the maximum width of the to be constructed trench in the U-shaped concrete structure. This width is directly related to the amount of piles which could be constructed. How many piles must be constructed is just the question. But more important, the required trench width which will be required to construct this foundation goes directly at the expense of the total available width of the motorway. It implies that it will influence the availability of the A27, and it's just the question in what order of magnitude this will be.

3.5.5 Structural components of the reinforced foundation

As said before, several pile grids are present. These grids vary and must be calculated individually. One objective of this paragraph is to estimate the width of the trench so that the executional aspects could be indicated in more detail. Since the pile grids differ as well as the capacity of the piles, it wouldn't be sufficient to only calculate one pile grid. One must search for the governing values related to the amount of piles to be constructed and the accompanying trench width. Therefore, an engineering judgement is made that two calculations will be sufficient to provide a realistic indication of the boundaries of these values. One calculation is performed at the closest pile grid at CPT 1 and one calculation is conducted at the largest pile grid, which is at CPT 2. The first calculation is based upon the pile grid at CPT 1. At both locations, the load of 3200 kN/m is the same.

Pile grid at CPT 1

The main point is to determine the required amount of piles and the trench width. Starting point of the calculation is the current pile grid, which is shown in Figure 3.15. In this figure, a part of the pile grid at CPT 1 is shown with the pile distances. Also, one area is hatched. This area is referred as the effective width of one pile row.

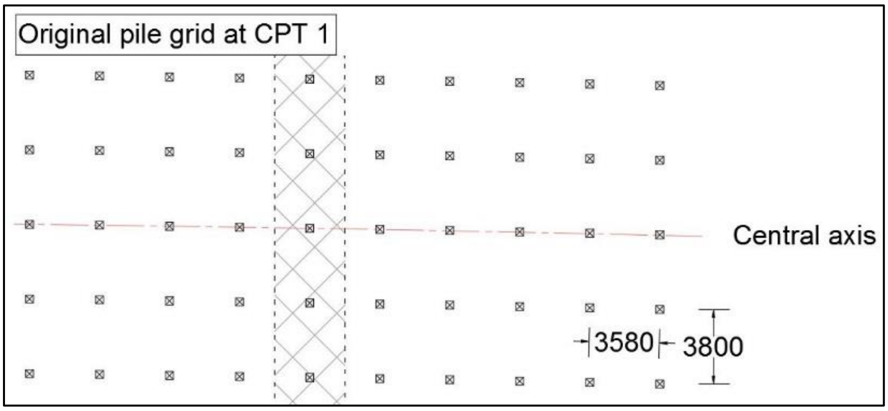

Figure 3.15: Existing pile grid at CPT 1.

The design resistance of one existing pile is about 2835 kN. It is assumed that the same piles are used for the new piles so that the 2835 kN can be used as a measure for the capacity of one pile at CPT 1. The design load exerted upon the effective width of one pile row is 11 500 kN. It implies that at least 4 piles need to be constructed in such an effective width area. However, 5 piles in total means that some eccentricity is present. Therefore, it's assumed that in total 5 piles are necessary to be constructed. It results in total to 6 piles per effective width area if the existing pile is reused. An option could be the following pile configuration visualized in the next figure.

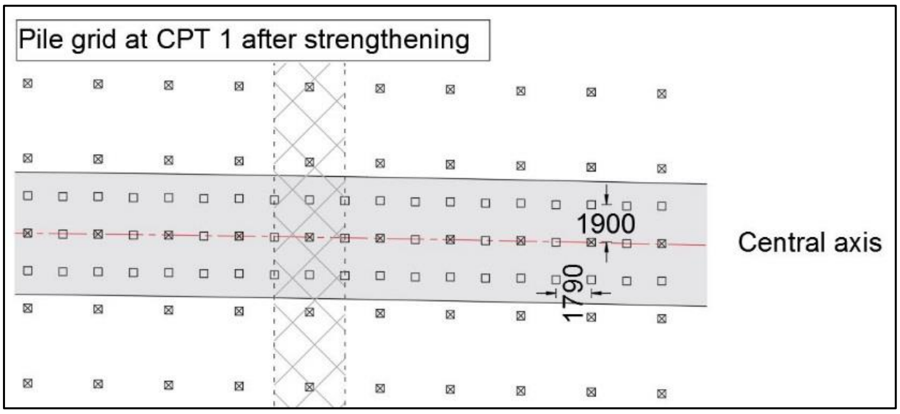

Figure 3.16: Strengthened pile grid with strip foundation.

The figure also visualizes a hatched area and some extra piles. Locally, the pile grid will be 1.79 to 1.90 meters. The hatched area schematizes the trench area. The width of this area is based upon the one original centre to centre distance plus one pile dimension. Furthermore, another meter at both sides is assumed to assure enough space for bending the reinforcement and placing formwork to construct the strip foundation. This kind of foundation must be constructed upon these piles so that the piles can interact together. The width of the trench will be around 6.5 meters at the location of CPT 1.

REMARK: *The aim of this paragraph is to estimate the width of the trench in order to discuss about the executional aspects and relate it to what extend it will influence the availability of the A27. As an assumption, the same piles and pile capacity is used as the existing foundation. However, it could be that other (bigger) piles are going to be constructed. Another possibility could be to construct larger piles to a deeper level, in such a way that the capacity can be increased and less piles are necessary. But it doesn't vanish the fact that a trench has to be made through the two concrete foundation layers. It also means that at both sides of the existing pile row piles must be constructed since only one pile row will not have enough capacity, and only one new pile row with the existing piles leads to eccentricity. Thus, the width of the trench will be in the same order of magnitude after optimizations as in the example above.*

Pile grid at CPT 2

The same approach is applied at the second CPT location. Despite the larger grid and lower pile capacity, comparable measures must be taken. The design resistance of a pile is 2155 kN. The design load per effective width of one pile row is 12 800 kN. It implies that one pile row with the hatched effective area should contain at least 6 piles, which will be just sufficient. If more capacity is preferred, longer piles could be applied, or other measures as stipulated in the small paragraph REMARK above. In that case the trench width could be comparable to the trench at CPT 1. Figure 3.17 shows two schematizations. The left schematization is the original pile grid with a hatched area. This area is the effective width of one pile row. Furthermore, the pile grid is 4.00 to 5.70 meters.

The right schematization is the same pile grid at CPT 2, only after installing the extra piles. The differences between the two figures are the amount of piles per effective width and the hatched area which will become a strip foundation. After installing the extra piles, the effective area per pile row will contain 6 piles instead of 1.

Furthermore, another area is hatched in the right figure above. This will be the area of the to be constructed trench. The width of this area will be comparable to the width of the trench at the location of CPT 1, about 6.5 meters.

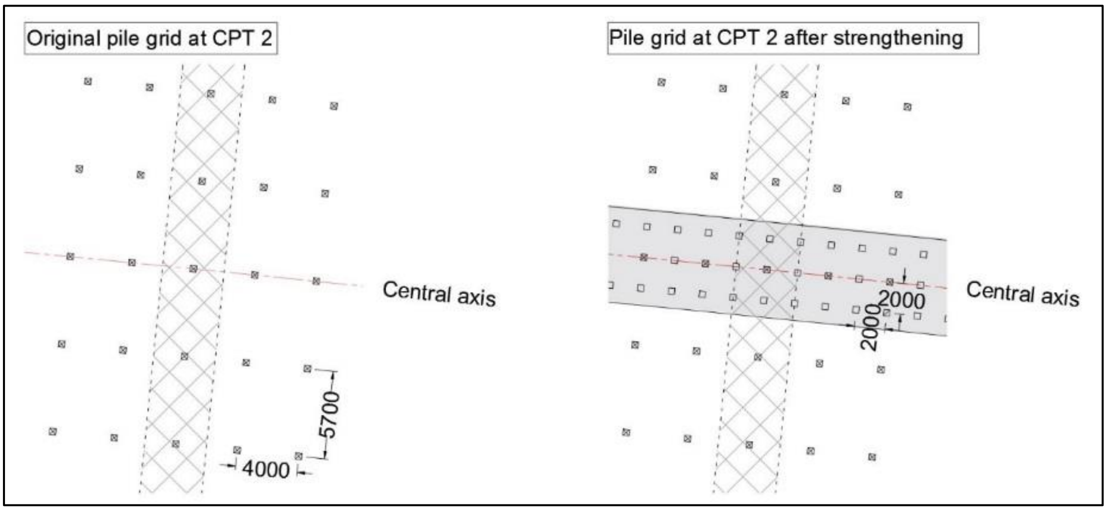

Figure 3.17: Two schematizations of the pile grid at CPT 2; the existing foundation (left) and after strengthening (right).

3.5.6 Execution aspects

One must realise that a dry working space must be created in order to install the piles or to pour the structural concrete floor/strip foundation. This 'creating a dry working space' is quite a challenging task of the execution aspects. Therefore, one part is related to the task creating a dry working space within the motorway. The other part consists of 'common' execution aspects in constructing a piled foundation within the existing foundation. First the key-issue of creating a dry working space is discussed, followed by the other of the method of execution.

Creating a dry workspace[2]

The required structural components are already determined, so it's quite clear what kind of measures must be taken to realise a reliable foundation. For now, the challenge will be how to install these components, especially the piles. A dry workspace must be created. First, it's investigated in what way it could be possible to realize such a space without any kind of drainage, because drainage affects the surroundings in a negative way. As said before, drainage wasn't allowed when the existing structure was built in the past. But, a trench must be constructed through the three layers of the existing foundation. The structural concrete floor must be removed followed by the intermediate layer of sand. Removing these two layers won't result in big problems. After removing these layers, the underwater concrete floor is still present. It could be the case that cracks are located in this layer and water tightness problems do arise. This will be a point of attention, because water leakage could lead to large problems.

Nevertheless, this last layer of unreinforced underwater concrete must be removed at the position of the trench. One must understand that once a gap is made in this underwater concrete layer, the water will splash out to the surface, since the pore water pressure beneath this layer is 67 kPa. Therefore, just making a gab in this layer can't be done without taking countermeasures.

Normally, two alternative measures could be applied if such a problem exists. This will be either applying a grouted screen or try to freeze the pore water of the subsoil in such a way that the ground water can't flow. Via both manners the water outside the building pit can't penetrate inwards. But, when the subsoil is frozen, it's impossible to drive piles through this layer. The same holds for a grouted sublayer. Other kind of foundation piles

[2] *This paragraph/sentence is obtained after a discussion with a geotechnical specialist dr.ir. M. Korff of the TU Delft.*

could be used instead, such as bored piles. Even if these two possibilities could assure water tightness of the pit and drivability of the piles, these two measures couldn't be applied. First a gap must be created in the underwater concrete floor to put the kind of lance through the underwater concrete floor to a certain depth. The chemical or grouted compound can be injected. When creating this gap in the floor, water will directly flow into the pit due to the overpressure of 67 kPa. Therefore, these two manners can't be applied. However, some contractors might have some ingenious system or measure to overcome the problem. It might be a kind of lance which can penetrate through the underwater concrete floor. This lance will inflate after it reaches the confined aquifer, so (almost) no leakage water will rise to the surface. Or maybe some other inventive method of execution could be made up. Until now none of such methods are known. Even though such new and risky methods could be applied, it will be quite expensive. Besides, it contains a high-risk level. Imaging that such an operation won't go by as planned, and some water leakage occurs resulting in a flooded A27. Hindrance to the surroundings will rise to a next level. It'll result in ground water flows which leads to all other kinds of effects such as settlements to buildings nearby leading to damages. Besides, one of the important effect will be that the motorway A27 must be closed for a certain period of time. It's just the question if a contractor is willing to take this risk, since such a closure must be excluded at all times.

In short, it seems to be that creating a dry working space can't be obtained without a sufficient drainage system[2].

It means that a drainage system is required. Two possible options could be applied. The first one consists of a permanent drainage system. It will permanently lower the ground water table to the required height for a certain period of time. It depends on the time necessary to construct the trench, install all the piles and the (strip) foundation. The time is estimated to be at least a year. In general, the longer the drainage period, the higher the influence will be for the surrounding area. This surrounding area will have the size of several kilometres in radius. Besides the adjacent forest will suffer and within the vicinity some districts of houses and other engineering structures are present. Due to the lowering of the ground water table, the vegetation doesn't have enough water to flourish and grow. Nevertheless, the buildings and structures in the vicinity could be subjected to differential settlements with high risk on damages. Concludingly, all these effects results in the fact that a permanent drainage system won't be approved by the (local) authorities.

The only possibility left is a combination of the two described options. It consists of a locally applied drainage system. The main difference with a permanently applied drainage system will be that it's only applied for a small period of time, e.g. a week. This will minimize the settlements in the surrounding area and it will also minimize the influence on the adjacent forest since the time span is a very important parameter. The ground water table can be lowered to just beneath the underwater concrete level. When this is accomplished one can penetrate the underwater concrete layer in a safe way. Then, several lances can inject the compounds and penetrate safely through this layer without water will flow into the building pit. A vertical 'grouted shield' can be made and after that a horizontal layer can be injected which ensures a total dry building pit.

In essence, a building pit is realized within an old existing building pit, the U-shaped concrete structure. The following paragraphs show a rough description of the tasks to be executed, and the chronological phasing of creating such a building pit in the existing structure. The same holds for constructing the new foundation for the intermediate support. It's just an assumed way of working and it's explained for one compartment with the pile grid at the location of CPT 1, visualised in Figure 3.12.

Realizing such compartments with temporary drainage seems to be the only solution if permanent drainage is prohibited. Therefore, this method of execution will be briefly discussed.

REMARK: *Until now, it's undetermined what kind of drainage system could be applied in order to obtain a dry working space. The same applies for the duration and the impact of the temporary drainage. It's uncertain yet, and feasibility of an appropriate temporary drainage system should be proved.*

Dimensions compartment, the top two layers of the foundation and the drainage system

A possible way of working could be to construct compartments in a consecutive way. It implies that the trenches are going to be constructed in compartments of about 20 meters in length. The dimensions of the compartment must be accurately chosen with respect to the existing pile grid, otherwise not enough space will be available for the to be constructed piles. Furthermore, the dimensions of the grouted bodies must be taken into account, which will be emphasized later on.

One must consider that these tasks can't be executed only by hand. Large equipment will be required in order to remove the concrete layer. The question arises like what kind of equipment will be necessary and how to get it into position, considering the logistics of this equipment. Besides, what kind of drainage system will be required, and will it be positioned at the motorway nearby the trench, or will it be situated outside the extended U-shaped concrete structure? These aspects are important ones and must be considered in detail.

When the kind of drainage system is determined and applied, the idea consists of starting the dewatering only for a small period of time. Lowering of the ground water table must not be done for the whole 250 meters length of the U-shaped concrete structure at the same time. The intermediate support and thus the foundation can't be executed at the same time since it'll be too time consuming. The assumption still rests that only temporary drainage will be approved.

After determining the outer dimensions, first the structural concrete floor and the sand layer in-between must be removed for over the length of the compartment. As determined in paragraph 3.5.5, the trench width will be about 6.5 meters. The same width must be persisted. When this task is executed, the drainage system is already installed and applied. This situation is shown in Figure 3.18. The left part schematizes the top view of the existing foundation at CPT1. The dimensions of one compartment are hatched. The right part shows a kind of front view/cross section in longitudinal direction with the blue line as ground water level. After the ground water table is lowered to a level just beneath the underwater concrete floor, the next phase can start.

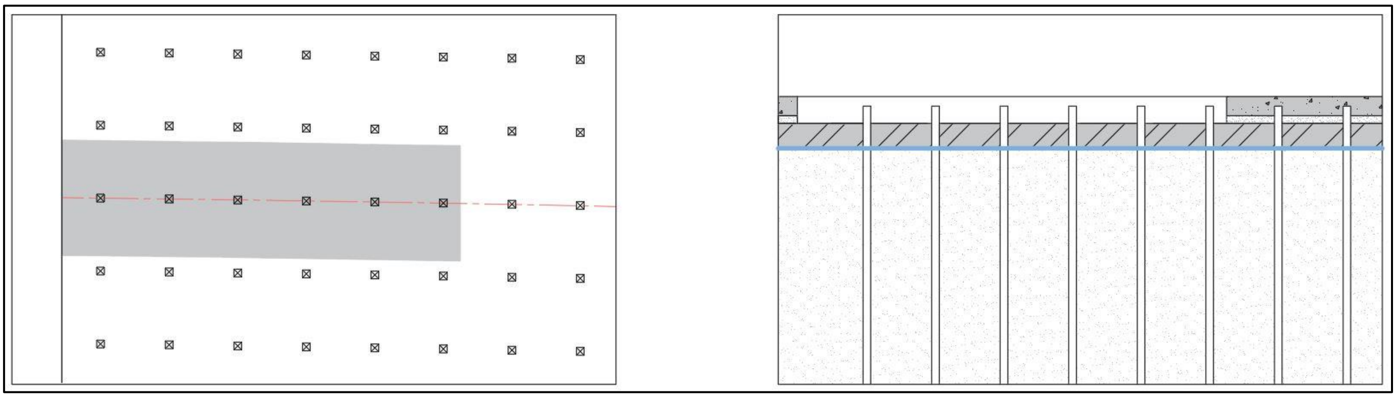

Figure 3.18: The first phase of one compartment.

Ground water table, underwater concrete floor, vertical grouted screens and horizontal membrane layer[3]

The drainage system is applied and in use. Before penetrating the underwater concrete layer for over the whole compartment length, one can assure the water head by applying a well pipe which can monitor the water head. This is a kind of quality assurance for determining the water level/pressure beneath the underwater concrete layer. This measure diminishes the risk on water inflow on large scale.

[3] In this case a membrane layer is a horizontal water tight layer from sodium silicate.

When the water head is assured, the underwater concrete layer can be removed over the dimensions of the compartment. This task must be executed carefully, because it's planned to reuse the existing piles. The following task is the application of vertical screens. The kind of screen must be determined later on. It could be a sheet pile wall or a grouted wall which can be applied via grout lances. Since the sheet piles can't be retrieved afterwards, it seems that a grouted vertical wall is less expensive. This kind of grout lances can construct vertical cylinders of soil mixed with grout. Every cylinder has a diameter of about half a meter, otherwise problems related to space and pile driving could occur. In theory, several cylinders next to each other result in vertical watertight screens. In practise, it prevents or minimize horizontal water inflow in such a way that a minimum amount of well drainage will be required.

However, if the temporary drainage stops, the water level rises to original height if no horizontal impermeable layer is constructed. Therefore, such a layer must be realized to provide a watertight building pit. As said before, if a horizontal grouted body is injected, no piles can be driven through this layer. It won't be an option. The same holds if the soil will be frozen.

An alternative of a grouted layer consists of injecting a horizontal membrane layer of sodium silicate. This compound is a kind of gel, soluble in water and pretty well applicable. The idea consists of injecting a fluid at sufficient dept. This fluid results in a horizontal impermeable gel layer after hardening of 48-72 hours. The technique is quite rare because of its expensiveness but is used in projects. According to a research study [6], several reference projects are known where building pits are kept dry via injecting this kind of a horizontal layer. In most of these projects piles were installed and afterwards the horizontal layer was injected. However, four projects are known with a different methodology of applying such a layer. In these projects the injections took place first, and after the chemical reaction the fluid turns into a gel and the prefabricated piles were installed. The results were quite positive with less water leakage in the constructed building pit. A small drainage system was sufficient. The next figure schematizes the injected gel bodies at a certain dept. In this case sheet piles are visualized as vertical screens.

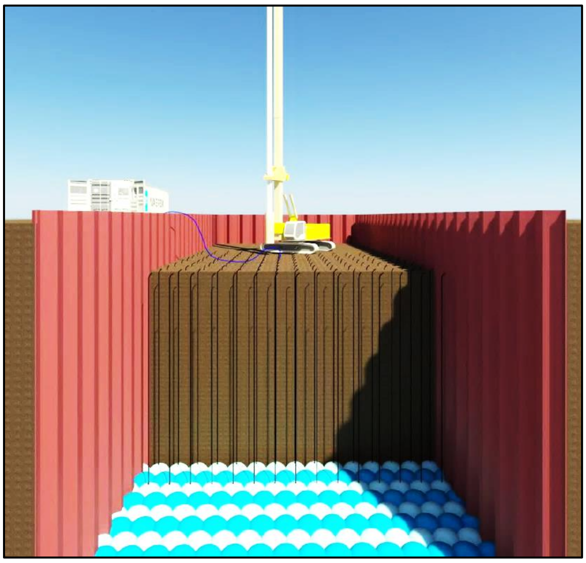

Figure 3.19: 3D representation of injecting spheres of sodium silicate to realize a watertight horizontal layer [6].

The depth of this layer can be determined easily via Archimedes law. With the basic assumption of 67 kPa of pressure below the existing structure, and the self-weight of saturated sand of 20 kN/m^3, the gel layer should be applied at about minus 14 meters N.A.P. It implies that the weight of the saturated soil above the gel layer is able to resist the upwards water pressure beneath this layer. After hardening of this gel layer, a building pit with minimum leakage is realized. One strong advantage of this method is that piles can be driven through this layer. If grouted bodies and freezing bodies were applied, no piles could be driven through the layer.

Another important aspect is to consider the amount of equipment, especially the dimensions, the weight and the position. The used equipment is quite enormous. To indicate the rough dimensions, Figure 3.20 shows two pictures when this equipment applies the injection points at the pre-determined locations.

Figure 3.20: Equipment used for applying horizontal watertight layers of sodium silicate [7].

However, one must indicate that these enormous machines must be positioned within the compartment to install the injection points. The centre to centre distances of these injection points are 0.9 to 0.9 meters in a triangular grid. The diameter of the spheres are about 1.2 meters, because overlapping must occur to achieve a watertight horizontal layer. These machines can't be located within this compartment, due to their dimensions and the presence of the existing piles. Therefore, a kind of auxiliary structure must be engineered and assembled across the trench so that the injection points can be installed at all the required, pre-determined points.

At the end of this phase, the underwater concrete floor is removed, vertical grouted screens and the horizontal water tight layer are applied. When these tasks are executed, drainage for this compartment shouldn't be applied anymore. Figure 3.21 shows the situation after the tasks are executed, again for one compartment at CPT 1.

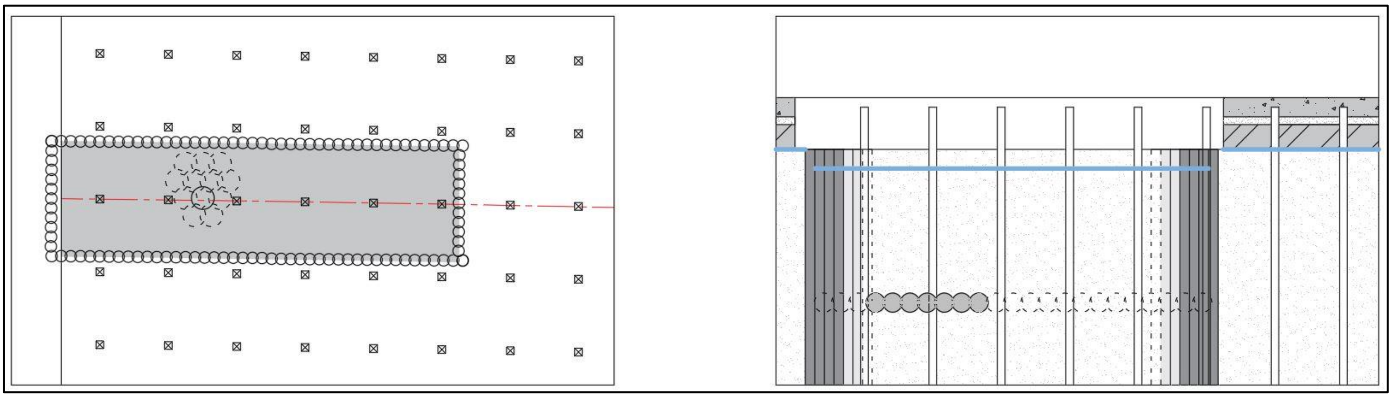

Figure 3.21: Second phase; removing the underwater concrete floor and achieve a watertight building pit.

In this figure, the vertical grouted screens are visualized at both sides of the compartment and not over the full compartment length, otherwise the other parts won't be visible. The same holds for the horizontal layer which is a layer consisting of the injected spheres of sodium silicate. A few of them are visible, the other ones are dashed. Furthermore, it can be seen that the water level within the compartment is still beneath the local surface, while the water level outside this compartment is restored to its normal level.

Pile driving

After realizing the vertical grouted bodies and injected gel spheres which form the horizontal layer, a watertight building pit is constructed within the existing U-shaped structure. The consecutive task is to realize a reliable foundation, which starts with increasing the bearing capacity. Therefore, piles have to be driven in the new building pit compartment conform a pile grid as outlined in paragraph 3.5.5. The centre to centre distance will be 1.79 to 1.90 meters. It implies that at every existing pile row, 5 piles have to be added, as visualized in Figure 3.16.

Installing the piles results in almost the same considerations as with applying the injection points regarding the required equipment, because the dimensions and weight are comparable. After installing the piles, the situation will be as schematized in Figure 3.22. This picture just shows what structural aspects are required to achieve a reliable structure. The multiple topics related to execution aspects are considered subsequently. A lot of variations are possible, but the next paragraphs are meant to explicate the considerations if a certain execution method is chosen.

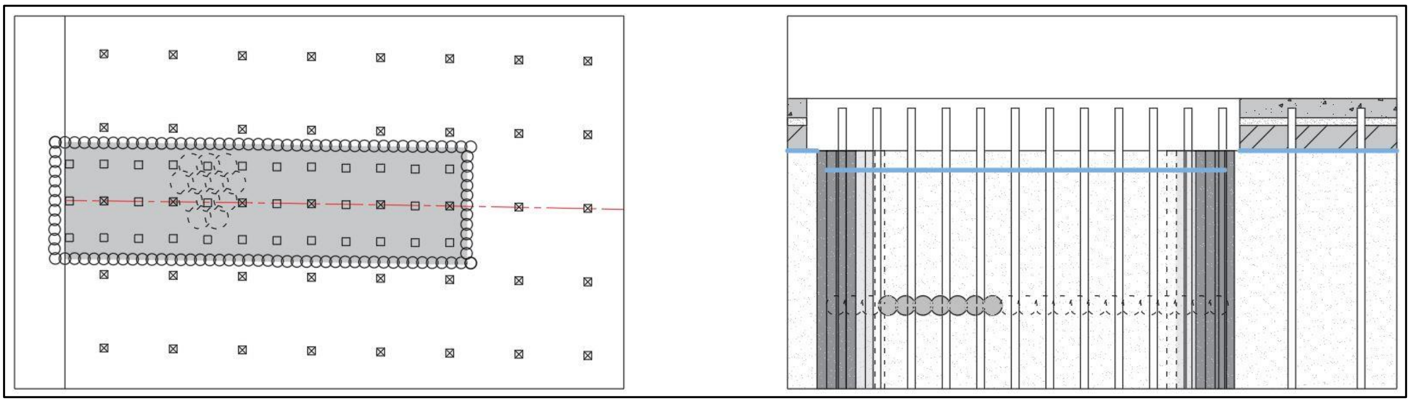

Figure 3.22: Installing the extra piles required

As seen in the figure above, in total three pile rows must be added. If these piles are installed in an ordinary way, the piling equipment will be situated perpendicular to the compartment. It implies that at both sides of the compartment equipment should be positioned, which requires a substantial amount of space. In Appendix E an example of a tough driving installation is outlined with its dimensions. It seems that machines like this require about 13 meters of space in width without taken into account any safety margins. It's just the dimension of the equipment itself. These piling machines should be supplied with the prefabricated piles. In other words, access roads for the supply of these piles are required. A temporary driving lane is assumed to be about 5 meters wide, since tough engineering equipment must be delivered to this site. These dimensions combining with the original trench width, barriers and other safety measures results in quite a large width. The construction site in the middle of the motorway will become about 40 to 45 meters in width. It implies that only 30-35 meters in width will be available for traffic lanes in both directions, while the current road arrangement consists of about 50 meters.

Besides, if the piles will be installed when the equipment is situated perpendicular to the compartment, the pile row in-between can't be installed. The distance between the location of the to be installed piles and the position of the driving equipment will be too large to install the piles. Therefore, this row of piles must be applied in a different way. As said in the previous paragraph, the equipment of installing the injection points is quite large and couldn't reach all the pre-determined points if the equipment will be situated at the sides of the compartment. Therefore, a kind of auxiliary structure will be required. The same structure could be used when the piles are driven into the soil, because the intermediate pile row is out of range if the driving equipment positioned perpendicular to the compartment.

But, when the pile row in the middle will be constructed from this auxiliary structure, the other piles could be installed as well via this manner. It'll spare a lot of space in width of the building site if no equipment should be positioned perpendicular to the compartment. It seems to be advantageous that every pile should be constructed via the help of this auxiliary structure. The structure is also required when installing the injection points. Figure 3.23 indicates this structure with a large piece of equipment drawn to scale. This machine is about 13 meters long and has almost the same width as the to be constructed trench.

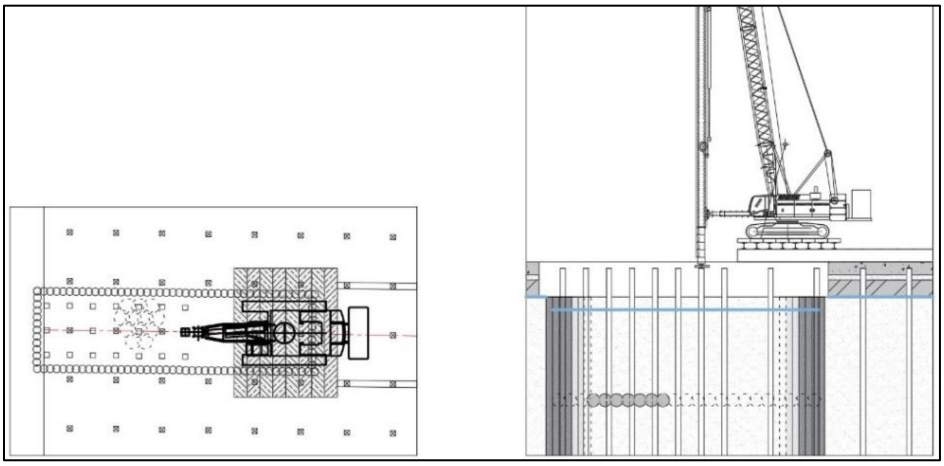

Figure 3.23: Plausible solution of an auxiliary structure

This kind of pile driving includes a complex execution method, but something like this might be inevitable. If only equipment is situated at the sides, the middle pile row can't be realized. It will take too much space in cross direction which will go at the expense of the availability/capacity of the A27. Besides, such a structure will also be necessary when the injection points are applied with comparable dimensions of the (heavy) equipment. Installing these injection points and pile driving are subsequent activities, so these tasks aren't executed simultaneously. It's a perfect possibility to use such an auxiliary structure twice. It will also result in less hindrance to the A27, because the building site can be made smaller.

However, this kind of equipment must be transported to the building site. It must have the possibility to steer, turn and unloaded from a trailer. The supply of materials is also required and general aspects such as construction sheds should be located as well. Figure 3.24 shows a possible configuration of the building site in the middle of the A27 which considers these aspects.

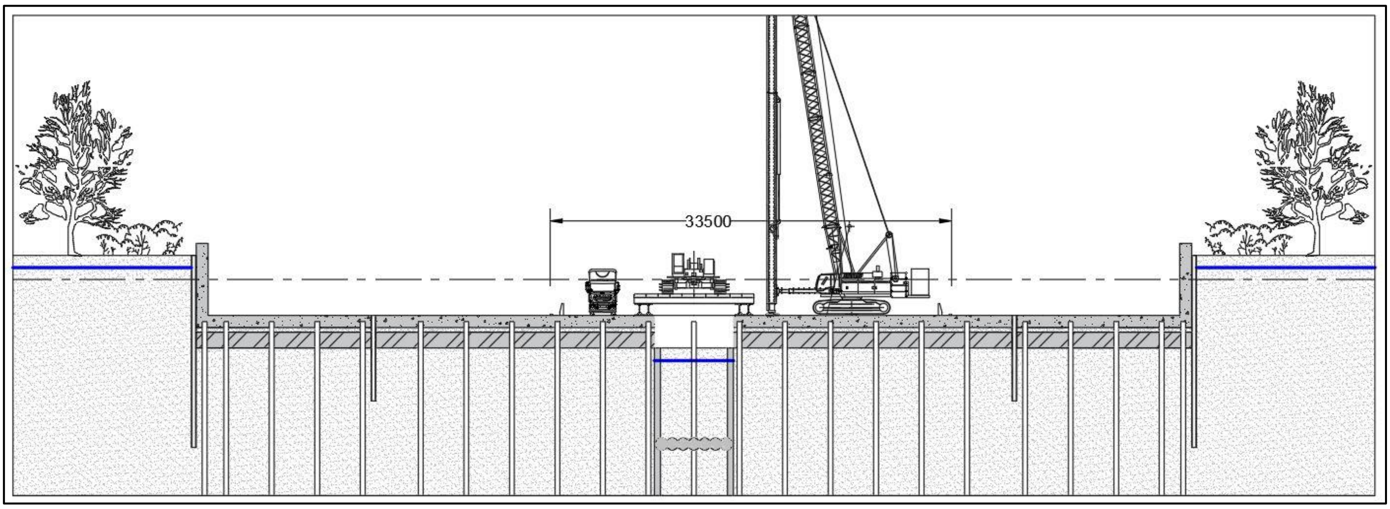

Figure 3.24: Possible layout of the equipment, units in [mm].

REMARK: *This figure is meant to provide a plausible solution, and to indicate the required width of the building site which goes at the expense of the available space of the A27.*

The driving equipment must be unloaded from a trailer and should have the possibility to stir and to manoeuvre. This is schematically shown at one side of the compartment. At the other side, an access road with a truck is drawn. A width of 5 meters is assumed to be sufficient for this temporary road. 5 meters seems to be a large width, since the normal width of a driving lane is 3.5 meters. But in this case all kind of exceptional transport should also have access to the building site, otherwise all the different kind of equipment can't be delivered to the site. Furthermore, the figure is drawn to scale, and the width of the building site will be in the order of magnitude of 30-35 meters if such an auxiliary structure is applied. It implies that the building site width can be diminished with at least 10 meters compared to a building site without an auxiliary structure.

Furthermore, an important topic is related the prefabricated pile itself. The location, position and the dimensions of the equipment has already been outlined. The access road is mentioned as a required part of the building site and is taken into account in the layout of this site, so a trailer with prefabricates piles can drive along the compartment. The driving equipment is situated upon the auxiliary structure across the compartment. The piles have to be hoisted to the driving equipment, but these piles are at least 20 meters long and weigh up to 7 tons. The equipment on this auxiliary structure is confined in its movements, since the structure is kept as small as possible to reduce the width. Therefore, it seems that the piling machine can't hoist the piles in position to drive them into the soil. It will be likely that an auxiliary crane is required to hoist the piles from the trailer to the driving equipment situated on the structure across the compartment. The construction site will look comparable as described before, visualized in Figure 3.23. Such an auxiliary crane is normally used as well to support and supply the driving machine of prefabricated piles.

To summarize the above stated paragraph, to install the piles it's required to have a truck and trailer combination which can deliver the prefabricated piles, an auxiliary crane to lift the piles and the driving equipment situated on the auxiliary structure across the compartment to install the piles. The driving machine is almost statically positioned to the structure, but the truck and the auxiliary crane should have to drive to another position or to manoeuvre in general. These two machines should also have the freedom to move and be replaced if something goes wrong. The layout of the building site will be comparable to the layout depicted in Figure 3.24, only the auxiliary crane will be a bit smaller than the piling machine sketched.

Furthermore, two important parts of the executional tasks should be discussed even more thoroughly. The logistics of the equipment plus materials, and the hoisting of objects at the building site in general. One must consider that every kind of equipment, all the workmanship, the amount of cubic meters concrete and prefabricated piles have to be delivered at the building site situated in the middle of one of the busiest motorways of the Netherlands. The same holds for all kind of construction waste, such as the crushed concrete of the to be removed existing structural concrete floor for instance. Everything must be transported.

So, every part has to be transported to the building site. It implies that the heavy-duty trucks with tough equipment should drive at the fast lane at the left side of the driving direction at the A27. These trucks should pre-sort while driving. A substantial distance will be required in front of the building site. Imaging the event that vehicles like the convoys exceptional and mobile cranes drive at this lane at the left side, while the width of the motorway had already been reduced. It will result in large disturbances in the existing traffic dynamics and a high amount of hindrance. This will not only be the case when these vehicles will enter the building site, but also when these vehicles will leave the site.

To provide a solution to the logistics if vehicles like this should enter and depart the building site, it seems to be advantageous to apply a kind of drive-through for all kind of transport. This drive-through will be a one-way driving lane only meant for construction related traffic. The advantage will be that one driving lane will be sufficient to provide an access and depart to this site. Otherwise two driving lanes should be applied, which goes at the expense of the available space meant for driving lanes of the A27. However, the disadvantage is that access to this site is only possible via one side of the A27. If this results in problems related to efficiency and accessibility, one can still apply an extra access road at the other driving direction of the motorway up ahead of the building site. Via this manner it could minimize traffic hindrance and it won't go directly at the expense of the available width of the A27.

Figure 3.25 visualizes an example of a possible solution of the building site when the first compartment is under construction. The access road for construction related traffic is hatched and labelled as a one-way traffic road. In front of the compartment, a kind of load/unload area is drawn for heavy transport. Furthermore, next to the compartment an auxiliary crane is drawn which can hoist the prefabricated piles from the truck and trailer combination at the other side of the compartment to the driving equipment. This truck is situated on the one-way traffic lane. If it turns out that it'll be more advantageous to position this truck at the same side of the auxiliary crane, this position should be chosen. The green line in the figure represents the perimeter of The Green Connection. In fact, this figure schematizes the considerations discussed before in a nutshell. Such a schematic view of the layout of the A27 provides a proper estimation of the remaining available space which for traffic. Or, in other words, what available space will go at the expense of the space meant for traffic at the motorway.

The dimensions of the building site are comparable with the ones visualized in Figure 3.24. But, one must consider the fact that the heaviest trucks loaded with equipment or materials must have access to this site and need some approach distance to be able to manoeuvre through the traffic which is present at that moment at the A27. Nevertheless, this construction site will be a bottleneck within the A27 and will definitely result in hindrance. It will be just the question to diminish this hindrance as much as possible without losing safety related aspects of all kind.

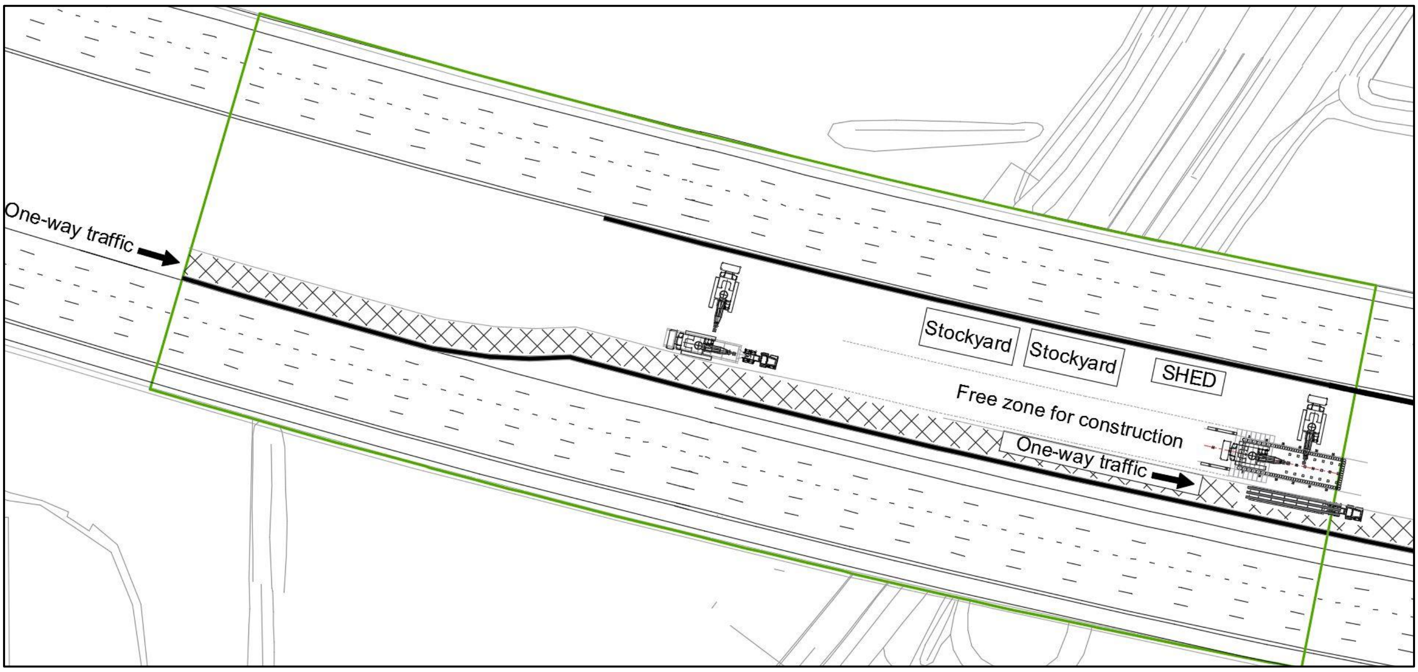

Figure 3.25: A plausible plan regarding the logistics around the building site.

Figure 3.25 visualizes a proper estimation of all the dimensions of the building on a larger scale. The topic logistics is quite complex, since the construction site is situated within the A27. Traffic dynamics is an important parameter related to the traffic flow at the motorway. Some aspects are briefly discussed, such as the remaining space for the motorway and the phenomenon that heavy transport should drive at the left driving lane in order to have access to the site. In what way these aspects will result in traffic jams and other negative effects will be just the question. These topics can be dealt very extensively, but it's not the aim of this thesis. Further research will be required to estimate the impact these aspects or to provide a more optimal solution to the traffic dynamics around the building site. For now, it can be estimated that a building site shall have a width of at least 30 meters to accommodate enough space to construct the intermediate support.

Besides the logistic aspects, the hoisting of objects at the building site will be an important topic to discuss. As said before, it'll be quite likely that an auxiliary crane will support the piling machine to hoist the prefabricated piles from the trucks. It means that every foundation pile will be hoisted and with every lifting task some extra safety measure should be taken into account. These foundation piles are at least 20 meters long and weigh up to 7 tons. In what way the piles are hoisted will be important. As a rule of thumb, nobody can stand beneath an object that is being hoisted. If something goes wrong, for instance the lifting cable of the crane breaks or slips off the lifted object, it won't result in casualties.

As depicted in Figure 3.24, the construction site will be about 30 meters wide. Therefore, it doesn't matter in what way the piles are hoisted, as long as it doesn't exceed the boundaries of the construction site. It won't be allowed to hoist the prefab piles and the piles are hanging above the motorway while hoisting. If in that case something goes wrong, such a pile could fall on the motorway and the consequences will be catastrophically. Such a risk can't be taken by the contractor.

In short, every time an object will be hoisted, no employee or other kind of workmanship can be situated beneath the hoisted object. This holds for unloading materials from the transport vehicles or with supplying the driving equipment with piles. The construction site will be wide enough to hoist one of the largest objects, the foundation piles, so it'll not be necessary to exceed the boundaries of the building site. With proper communication, an efficient planning and a well-though layout of the building site it will be possible to hoist the required objects at within the boundaries of the building site.

Concrete foundation

From here on, the starting point will be that the required foundation piles are driven and realized. The consecutive task is the realization of the strip foundation. First a kind of gravel layer might be poured to overcome the height difference of the underwater concrete layer. An underwater concrete layer wouldn't be necessary because a dry working space has already been created. Applying a gravel layer will be much cheaper compared to a thicker concrete floor. When this layer is applied, the required reinforcement must be placed with protruding bars at the location of the to be constructed wall support. It results in a firm connection between the foundation and the support wall. Then, the concrete can be poured. After hardening, the result is schematized in the next figure.

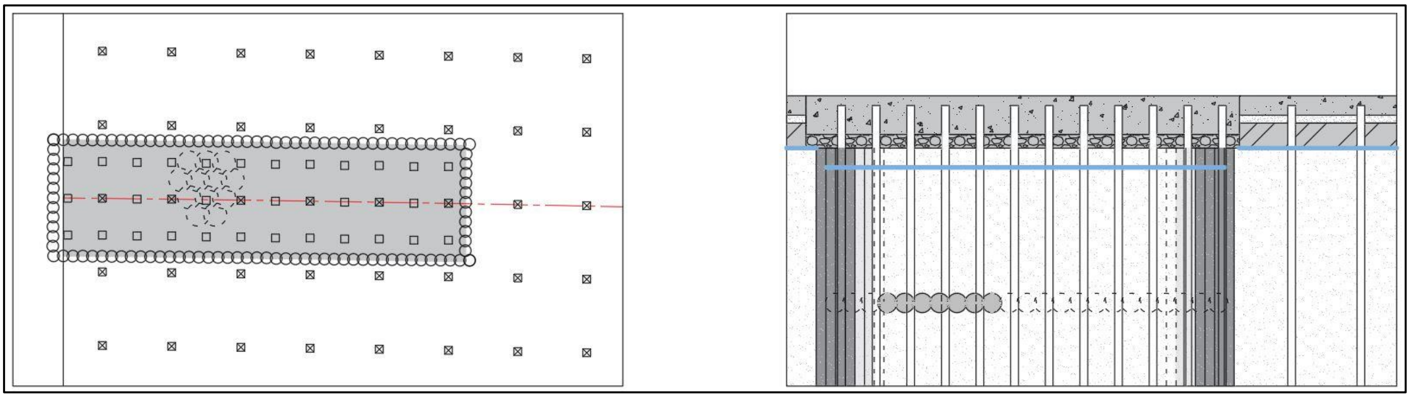

Figure 3.26: Last phase of constructing the foundation. The protruding bars are not drawn for the reasons of clarity.

When the concrete is hardened, the support wall can be constructed over the compartment length. No heavy equipment is required to pour the concrete. Just some formwork must be installed after the required reinforcement is applied. Trucks with concrete must arrive at the building site the same time. The surface is estimated to have the perimeter of one compartment (6.5x20 meters), and a 1.5 meters thick strip foundation. It'll result in about 200 m^3 concrete. This amount can be poured in one shift, if a constant supply of concrete is available. It requires about 15 trucks with concrete which must arrive subsequently in a small period of time. Normally, this won't result in problems, because the trucks can drive by one by one. But, when other construction activities take place at other compartments, it could result in logistic problems at the construction site. Therefore, it'll be recommended to plan such large concrete pours beforehand.

Nevertheless, this task won't result in large problems or threats. Attention should be paid to the logistic part of pouring the concrete in one shift. While this task is executed, other tasks can be executed for the subsequent compartment as specified in these paragraphs.

Time estimation one compartment

The first task is to determine the exact location of each compartment considering the existing pile grid. Then, the concrete and sand layer must be removed which must be carefully executed due to the existing piles while simultaneously the drainage equipment must be installed. It is estimated that these tasks require a period of one week.

Furthermore, it is estimated that one week of temporary drainage is necessary. Within this week, the underwater concrete layer must be removed carefully, due to the presence of the existing piles. The vertical grouted walls can be constructed as well as the application of the injection points. After injecting the chemical compound, the chemical reaction takes place and the drainage can be stopped. The next week will necessary to transport and locate the driving equipment and install the piles followed by another week to apply the gravel layer, place the reinforcement and pour the concrete of the particular compartment.

It's estimated that constructing one compartment of roughly 20 meters in length will take at least one month. It implies that approximately 13 compartments are required to realize this intermediate support, which results in more than one year of construction time without taken into account all the general tasks related to constructing a building site in the middle of the motorway A27. In the end, it's estimated that the total construction time will be at least one and a half year. It is a very rough estimation which depends on several aspects. However, performing a time estimation for the construction activities related to the intermediate support is a study in its own. Therefore, only a rough estimation is provided. Further research will be required to obtain a more accurate estimation. For now, it's assumed that the construction activities related to this support requires one and a half year of construction time.

3.5.7 Discussion of the intermediate support

As emphasized in the Preliminary Study, the construction of the intermediate support will be a quite extensive and complex task. After indicating the required tasks to be executed, a better insight is obtained about the complexity of constructing this intermediate support. It is important to distinguish the tasks which directly influences the availability of the A27 and to what level hindrance will occur. In general, the availability of the A27 during constructing will be the key-point criteria by the authority Rijkswaterstaat.

Furthermore, a design approach is performed to realize a reliable support wall in a durable and efficient way. One criterion was a proper estimation of the support reaction. It is found that the design support reaction foundation level will be in the order of magnitude of 3200 kN/m. This value is used to judge whether the existing structure will be safe. One interesting finding is that the current foundation lacks capacity. The existing design capacity is related to the capacity of a pile and the grid in which the pile is situated. At its weakest point, one pile could resist a design load of 2155 kN. Since the piles are spaced 4 meters, the capacity is about 540 kN/m. It implies that the capacity criterion has not been met by far and extra piles needs to be constructed. After determining the required pile grid, in general 5 piles are required to be realized at every existing pile beneath the support wall.

Constructing 5 extra piles per effective width of one pile row will be sufficient at the location of CPT 1, but isn't at CPT2 if exactly the same piles are used. It would be more efficient to drive 5 longer piles with higher capacity than constructing extra piles, for instance 7 piles per effective width of one pile row. In that case, the width of the trench should become even bigger, which will go at the expense of the available width for driving lanes at the A27 during constructing this support.

Nevertheless, driving piles requires a dry working space because a water pressure of 67 kPa is present below the underwater concrete floor. The level of installing the piles is situated below local ground water head. It implies that without any kind of drainage, the building pit will flood if the underwater concrete floor is removed. Therefore, a drainage system will be required. From point of view of the execution method, a drainage system which can permanently lower the ground water table will be most advantageous. However, in the early days during construction of the existing U-shaped concrete structure, any kind of drainage was prohibited. Therefore, it'll be most favourable to construct this intermediate support without any kind of drainage, but this won't be realistic. Permanent draining is assumed to be prohibited, due to the negative effects to the surrounding area. The only possibility left will be that only short periods of time drainage will be approved and this variant is discussed in this chapter. During these short periods of time, small building pits should be realized and the construction of the foundation can follow. However, the opted method of construction could be different because contractors might have some inventive method of execution. Another possibility might be that Rijkswaterstaat do approve a permanent drainage system, since drainage tests near the U-shaped structure are fully applied nowadays to indicate the side effects [8]. Nevertheless, it doesn't vanish the fact that applying a drainage system has a high-risk potential and will be still questionable.

However, if drainage in general won't be approved by the authorities, draining small periods of time might not be approved as well. In that case, it's most likely that constructing this new foundation for the intermediate support will be impossible to realize.

Realizing a building pit within the existing structure is a sequence of risky tasks. First of all, the dimensions of each compartment should be chosen very accurately because the vertical grouted screens must be constructed exactly between the existing piles. Extra piles are going to be driven in-between and may not interfere these screens.

Besides, sheet pile walls are present in cross direction. These screens could be advantageous when creating the building pit as compartments, but are quite inconvenient related to the new to be installed piles. These sheet piles are situated in-between the existing piles, exactly at the position of the new to be installed piles. It means that at the location of the existing sheet pile walls in cross direction, piles should be driven into the soil. But this is impossible because of the presence of the sheet piles. At these locations visualized with blue lines in Figure 3.12, some extensive solutions need to be invented. Retrieving the existing sheet piles will be a difficult job since these sheet piles are situated for decades in the soil, poured into the concrete with shear stud connectors and interconnected with the other sheet piles. It's expected that the solution will be complex. An extra trench widening might be required to overcome the problem to achieve a reliable structure. Such measures result in a more complex and costly solution.

Furthermore, one of the biggest challenges is to obtain a dry working space. The first challenge is to lower the water head beneath the existing structure. This could only be done if the drainage system should have worked properly. After the water head is assured, the underwater concrete layer can be removed, and via jet grouting the vertical columns of about 10 meters in length can be realized. This task itself is quite susceptible to water leakage. The same holds for the horizontal watertight layer of injected sodium silicate. Especially if the sodium silicate compound is injected under high pressure nearby the grouted vertical screen. In fact, this technique of injection points with sodium silicate is never applied in combination with grouted vertical walls. Normally, it's applied with sheet pile walls which function as vertical screens.

Even if these grouted and injected screens are applied in a perfect way, pile driving could affect these screens. According to the geotechnical specialist (2), the relative amount of soil displacement due to pile driving shouldn't exceed the 8% otherwise drivability issues might play a role. If the centre to centre distance of the piles are 1.79 meters by 1.90 meters, the relative amount of soil displacement will be about 5%. If only the 2D plain is considered, it won't result in drivability issues. However, the soil displacement in 3D plain, along the pile shaft could be locally higher than the maximum of 8%. In that case drivability problems might occur. If such problems occur, a solution could be to relocate the piles differentiating from the initial pile grid. It implies that the amount of piles will be equal to the original pile grid, but the exact location of the installed piles will differ. This countermeasure will have some impact in the distribution of forces to the structural floor on top of the piles, as well as the pile head itself. With an adequate reinforcement configuration, a reliable foundation strip can be designed.

However, the new to be driven piles could result in problems nearby the vertical grouted screens. Independent of the pile grid, driving the piles results in soil displacements which results in an increase in effective stress between the particles. These stresses could reach such a level that it will result in cracks in the vertical grouted screens. Besides, the relative amount of soil displacement is inaccurate, since differentiations occur in the diameter of the grouted cylindrical bodies. It could be the case that the grouted cylindrical body will be locally larger, so that the relative soil displacement is higher than expected when the piles are going to be driven in the soil. This will increase the stresses even more which increases the risks on cracks in the grouted soil bodies. In the worst-case scenario, the piles are hammered to a grouted body. In that case cracks will definitely occur in the grouted bodies resulting in leakage to a substantial amount. In the end, it's highly recommended to apply an extra drainage system within the compartment. If water leakage occurs, it could be drained immediately.

Another question is in what way the existing piles could be reused. In the early days, the pile heads were removed after obtaining the dry building pit. The pile heads were equipped with extra reinforcement. This is a common way to achieve a firm connection between the piles and the structural concrete floor. It's unknown to what extend these piles are poured into the structural floor and how these piles can be reused.

Besides, if new piles are driven nearby the existing ones, the phenomenon of 'redriving' could occur. This implies that while driving the new piles into the soil, the existing piles move in opposite direction of the soil. It will result in a kind of damage to the existing piles since the capacity diminishes. In what way this phenomenon will occur will be just the question.

The equipment which will be used has quite some impact on several aspects, since it has interfaces with other disciplines. A kind of auxiliary structure will be required to realize all the injection points and piles. One must consider the weight and dimensions of the equipment and how to spread the load to the existing foundation. But these machines should be supplied with materials and supported by other equipment. In the end, it will result in a building site of at least 30 meters in width. These thirty meters correspond to at least 6 driving lanes if some free zone is taken into account as well as the space used for the support wall itself. This is quite a substantial amount of space which goes at the expense of the A27. It will be just the question if such a reduction of available width of the motorway will be approved by the authorities.

However, the dimensions of the building site just sketch the static plain view of the space arrangement of the A27. Moreover, it's important to discuss the dynamic aspects, such as the hoisting of objects in general. For instance, the foundation piles are hoisted from the truck and trailers to either the local stock yard or directly to the driving machine itself. The building pit is wide enough to accommodate a free zone while lifting, so that no one will be beneath the lifting object. But, what if something goes wrong during lifting? Normally a kind of rope or chain is assembled around the pile. In some rare cases, this rope will slip off the pile or it breaks. Then, the pile will fall in the building pit. If this happens, it won't be just too bad. Imagine that the pile will fall and turns backwards to the motorway and hits a vehicle. Such a risk can't be taken if it could result in casualties. The downside however, will be that when each pile is hoisted a small traffic stop should take place to assure the safety of the motorists at the A27. It implies that at every compartment about 30 of such stops should be carried out which will have great impact on the traffic dynamics at the motorway. It will definitely result in traffic hindrance such as traffic jams.

The logistic aspects itself, and especially related to the construction site in the middle of the motorway are important aspects to consider. Every part, equipment, material and workmanship must be delivered to this site in the middle of the A27. Long, slow exceptional equipment must have access to this site and must leave it as well. These logistic movements will have quite some impact on the traffic dynamics, since these trucks should insert the driving lanes from the left side instead of the right side. In fact, the logistic aspects of this total construction site are quite an issue and very important. It's recommended to design an extensive logistic plan to indicate unwanted events such as delayal of transport.

Concludingly, it can be stated that this paragraph outlined all kinds of considerations about constructing an intermediate support in order to provide a proper answer to the following sub research question:

How to construct an intermediate support?

The paragraph outlined a plausible way of constructing an intermediate support. One should keep in mind that the tasks to be executed are an accumulation of risky, expensive, complex and time-consuming tasks with a high impact on the availability of the A27. It seems to be possible to construct such a support, but only when some basic assumptions are made, such as an approval of temporary drainage. If some of these starting points will differ, it could lead to infeasibility of the project. Therefore, no unambiguous answer can be posed to this sub question. The next paragraph emphasizes the uncertainty in feasibility even more when a brief risk analysis is performed.

3.5.8 Risk analysis

The previous paragraphs emphasized the number of required tasks in order to construct the intermediate support wall. These tasks have to be executed within the A27 and every task itself could go wrong. To indicate the risks related to the construction of the intermediate support the same procedure is followed as described for the extended part in paragraph 3.4.3. More information about the colours and procedure is described in that paragraph as well.

A brief risk analysis is also performed for the construction of the intermediate support. First the risks are indicated and qualitatively judged to the probability of occurrence and the impact of the consequence if such an unwanted event occurs. The impact is assessed by considering safety and financial aspects.

The risks are outlined in a table on the next page. These risks are numbered and schematically visualized in the risk matrix in Figure 3.27. Some other risks are already mentioned in the previous paragraphs. The more severe risks are outlined in Table 3.2.

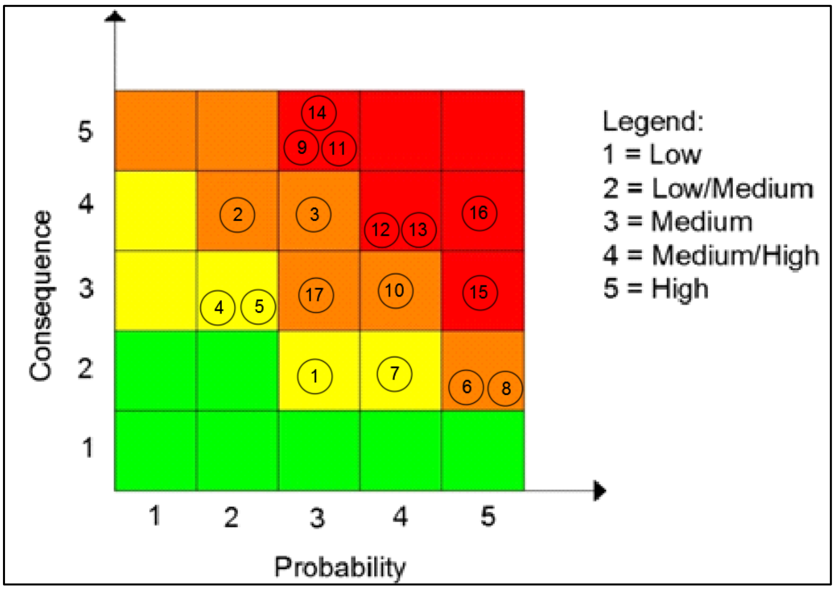

Figure 3.27: Risk inventory matrix.

As can be seen in the risk matrix above, some severe threats are present. Some of them can be prevented or countermeasures could be taken to mitigate the risk. Several risks are related to the water tightness of the compartment. If during constructing of this compartment a lot of water flows into the building pit, extra drainage systems are required. If the amount of water is too high, it could be catastrophic since the A27 is then going to be flooded. This event may never take place.

Other risks which could result in substantial threats are the numbered ones visualized in the red area. These 6 risks are indicated as the biggest threats and the feasibility of constructing the intermediate support is quite in danger. After all, it's just the question if a contractor is willing to take all these risks, what amount of financial reservations must be taken into account and to what extend the contractor is liable for these risks as well as other unforeseen risks.

Nr.	Category	Cause	Effect	Conse.	Prob.	Risk	Countermeasure
1	Existing foundation	Structural concrete floor can't be removed gently	The existing piles can't be reused to its full design capacity	2	3	6	Work cautiously
2		Inaccurately realizing compartments	Piles can't be driven due to too less space	4	2	8	Considering the compartments at every pile grid very accurately
3		Cracks in the underwater concrete floor	Water flows into the pit	4	3	12	Start removing the concrete layers after the drainage system is installed
4	Drainage system	Lack of capacity	Can't lower the ground water table sufficient	3	2	6	Apply extra drainage equipment
5		Drainage system or part breaks down	Can't lower the ground water table sufficiently	3	2	6	Extra drainage equipment in stock
6		Building pit isn't water tight	Water flows into the building pit	2	5	10	Apply extra drainage equipment
7	Realizing dry working space	Large pieces of debris into the soil	No water tight vertical screens can't be realized → water leakage	2	4	8	Extra drainage equipment in stock
8		Horizontal layer of sodium silicate isn't water tight	Water leakage	2	5	10	Extra drainage equipment in stock
9		Too much water must be drained at the A27	Capacity can't discharge all the pumped water and the A27 floods	5	3	15	Extra drainage system on land to drain the water
10		Compartment isn't finished in time	The drainage period must be extended	3	4	12	-
11	Foundation piles	Pile rope/chain slips off the pile during hoisting	Pile falls within the building pit with a risk on turning over to the motorway	5	3	15	Extra quality assurance on the manner of assembling the rope/chain
12		Drivability issues	Piles lack capacity and unsafe structure	4	4	16	Pile optimization: Use fundex or tubex piles
13		The relative amount of soil displacement will be too large due to the presence of the vertical grouted bodies	The piles can't be driven into the soil conform the design pile grid and the foundation will lack capacity	4	4	16	Relocate the piles differentiating from the initial pile grid.

Nr.	Category	Cause	Effect	Conse.	Prob.	Risk	Countermeasure
14		“Redriving of piles”	Piles lack capacity/large settlements after loading → cracks, water tightness issues and unsafe structure	5	3	15	Possibly to pre-load the piles, time and space consuming, and risky. Doubtful to succeed
15		Existing sheet pile wall can’t be removed	No piles can’t be driven into the soil at these locations	3	5	15	An extensive and a probably cumbersome solution must be thought
16		Existing sheet pile wall	Piles can’t be driven into the soil due to the existing sheet pile wall in cross direction → lack of capacity foundation	4	5	20	Trench should be locally widened. Kind of underground ‘bridge’ should be constructed. Costly and risky task
17	Strip foundation	Lack of concrete supply due to traffic jams for instance	The strip foundation can’t be poured within one shift. Time delay and structural disadvantages of the foundation itself	3	3	9	Proper planning of the concrete carrying trucks / assure a small stock yard

Table 3.2: Brief risk overview in constructing the intermediate support.

3.6 Constructing the deck structure

In this paragraph, some relevant aspects will be discussed related to constructing the deck structure. After the construction of the intermediate support, one can start realizing the deck structure of The Green connection itself. The sub research question to be answered is:

How to realize the deck structure in an obvious way?

The point of departure will be discussed first which starts with some considerations about the design of the deck structure followed with considerations with respect to the original design and the transport possibilities.

Furthermore, some differences do occur in relation to the Preliminary Study, because the starting point of this study entails a beam of 75 meters instead of the 42 meters used in the reference design of Rijkswaterstaat. This has some impact on the transport possibilities. Thereafter, the starting point of assembling the deck structure is discussed and three alternatives are considered. The aim of this paragraph will be to outline some execution aspects related to the assembly of the deck structure and judge whether it'll be feasible in order answer the sub research question.

Box beams

According to the reference document of Rijkswaterstaat [9], the most optimal solution will be a prefabricated deck structure. The plan consists of prefabricated box beams which are suitable for a span of 41 meters. According to the design graphs of the supplier Haitsma beton [10], a box beam with a height of 1800 mm will be sufficient to span 41 meters and an external load of 30 kN/m^2. The box beams of Spanbeton, another supplier of pre-tensioned precasted box beams, weigh 27.6 kN/m while the beams are 1.2 meters in width. It implies that the beams which span 42 meters weigh about 116 tons.

In the Preliminary Study, it's also emphasized that unique projects such as constructing The Green Connection ask for unique solutions and therefore a custom-made prefabricated box beam seems to be most advantageous. In that case, a boundary condition will be the weight of the beams. In the Preliminary Study reference projects are discussed in chapter seven. One of them is the project Bleizo. In this projects beams were transported by truck and trailer with an individual weight of 172 tons. So, to keep both the transport possibilities open, 172 tons of weight per beam is used as a boundary condition and can't be exceeded.

Furthermore, the maximum width can be estimated. If a 1.2 meters wide box beam weighs about 116 tons, the estimated maximum width without exceeding the 172 tons boundary condition will be 1.77 meters.

$$\frac{172\ tons}{116\ tons} \cong 1.48$$

$$1.48 * 1.2\ meters = 1.77\ meters$$

If it's chosen to use a 1.66 meters wide beam, in total 300 beams are required to be produced. However, if the distance over which the beam must be hoisted becomes too large, the beams should be reduced in weigh, otherwise the execution aspects could become in danger. Nevertheless, the weight of such a beam will be in the order of magnitude of 160 tons.

Transport

This topic is extensively dealt in the Preliminary Study in chapter 4. For detailed information about the possibilities it's referred to this chapter. However, in chapter 4 of the Preliminary Study, the transportation possibilities are discussed for beams with a length of 75 meters which weigh up to 280 tons. In that case the only possibility left is to construct these beams on a construction site nearby. But, this statement changes a bit if smaller beams are used. Then, transport by truck and trailer seems to be a solution as well. But, one must consider that in total 2 times 249 meters must be covered with these beams. It implies that 416 beams should be produced and transported individually. If a custom-made solution is chosen, this amount can be reduced drastically to 300 elements, as outlined in the previous paragraph.

Besides, as discussed in the Preliminary Study as well, it's just the question if one of these companies have the capacity to produce these beams, since it's about one and a half year of full production. Taken into account the duration of production, the transport of the beams and the available construction site, it's just the question which will be more beneficial/feasible; construction on the building site nearby or construction at the prefabrication hall at Spanbeton or Haitsma Beton.

From this point on it's assumed that 300 custom made box beams are required and should be produced. It's just the question if these beams are going to be manufactured at the construction site nearby or at a prefabrication company Haitsma beton/Spanbeton. In contrast to the Preliminary Study, it was assumed that only Haitsma Beton was a suitable company, since this company has a pre-stress facility which can produce pre-tensioned precast beams up to 90 meters. However, in this case only beams of 42 meters are required, which could also be produced at Spanbeton. But the transport route from Haitsma Beton to Amelisweerd is 170 km, while the route from Spanbeton is just 50 km. Therefore, if a prefabrication yard is chosen, Spanbeton is quite advantageous due to the reduced transport route.

In fact, two possibilities are present and it's just a consideration which one will be the most advantageous. The first one will be to produce the beams at Spanbeton and transport them individually to the location of The Green Connection by truck and trailer. The second option will be to produce the beams at the construction site just next to the final location. Only a few hundreds of meters must be overcome. Which possibility will be chosen depends on several aspects, such as the amount of hindrance, availability of the A27, costs and construction speed for instance.

3.6.1 Prefabrication at Spanbeton or at construction site

If the box beams are going to be prefabricated at Spanbeton, several variations in the method of the execution will be possible. In general, it consists of four main stages which can be subdivided in several other tasks. The prefabrication of the box beams is one of them.

After prefabrication, the large beams could be immediately transported to the project location, but normally these beams should be stored at the stock yard of the prefabrication company. When a certain stock is realized, these beams will be hoisted with a kind of overhead travelling crane to a truck and trailer combination and transported to the project location.

The reference project Bleizo has shown this way of transportation in an efficient way. A stock yard at project location minimizes the risk on transport delay. In other words, it was very beneficial to use a stock yard nearby the project location. After realizing a sufficient stock of several beams, these beams could be efficiently assembled to their final position.

The prefabrication of such beams is assumed as feasible and won't be dealt anymore. The transport of each beam from the company's stock yard to the temporary stock yard at Amelisweerd won't result in large problems as well, since the height and weight of the beams is lower than the boundary conditions. One can apply for a permit at RDW for the exceptional transport. Another point of attention is the realization of the temporary stock yard at Amelisweerd. From the Preliminary Study can be deduced that construction site number 1 could be used. As a recap, the location of construction site 1 can be seen in Figure 2.1. One prerequisite is that this stock at the construction site should also have a lifting facility such as an overhead travelling crane or at least two (mobile) cranes with sufficient capacity. At the temporary stock yard at the reference project Bleizo, two large overhead travelling cranes were present with sufficient capacity to load and unload the trucks. These cranes could also distribute the beams equally at this site.

However, if it's chosen to produce these beams at Spanbeton, a stock yard won't be required but it is highly advantageous to use a part of the construction site as a stock yard. At this site, a lifting facility will be required. Besides, custom-made moulds should be realized to construct these beams in an effective way. These three aspects should also be realized if a prefabrication yard at the construction site will be constructed. Until this point, no differences are present between prefabricating the beams at the company elsewhere or at the construction site with the temporary facility.

The differences do occur when considering the transport possibilities. If the beams are produced elsewhere, it implies 300 times an exceptional transport. In comparison with construction on site, the 300 exceptional transports with safety guidance and the returns to the factory could be omitted. This will have financial benefits, and will also be more sustainable in relation to exhaust gases.

When the beams are prefabricated at Spanbeton, the availability of the facility at Spanbeton could be quite an issue with this kind of tendency of the economic market. Besides, it´s just the question if this prefabrication yard can be adapted sufficiently.

Concludingly, it is important to determine the method of transportation of the beams to The Green Connection, because the method of assembly depends on it. The following statement is of importance. It doesn't matter if the beams are prefabricated at the construction site or at Spanbeton. When the beams are produced at the construction site, a certain stock yard will be allocated at this site before the beams are transported to the final location. If the beams are produced at Spanbeton, the beams will be transported to the stock yard at construction site 1. After a sufficient stock yard, the beams can be transported to the final location.

In other words, before the assembly of the beams, a certain stock yard is realized at the construction site number 1, independent of the location of production. Therefore, the starting point will be that the beams are stored at construction site number 1 and will be transported to the location of The Green Connection.

3.6.2 Starting point

Before discussing three alternative methods of assembly, a firm design environment should be created in order to compare these alternatives. At first, a clear layout will be provided. Although constructing the intermediate support is quite a complex and risky task, it is assumed that realizing a safe and reliable support wall in the middle of the A27 has been succeeded. The extended parts were already finished before constructing the intermediate support. Figure 3.28 shows the layout which is the starting point of constructing the deck structure.

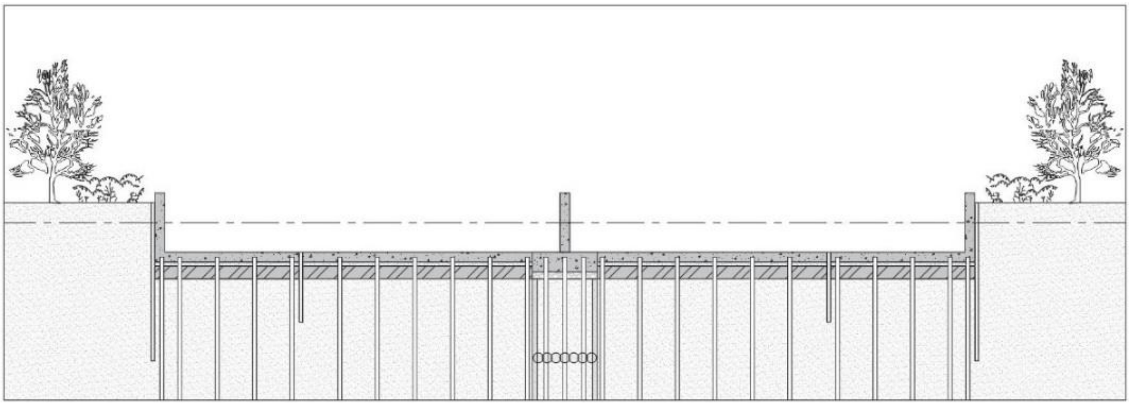

Figure 3.28: Starting layout when constructing the deck structure.

The deck structure will consist of box beams, as referred in the reference document of Rijkswaterstaat [9]. As stated in the beginning of paragraph 3.6 the custom-made box beams will be a more efficient solution for the deck structure. In chapter three of the Preliminary Study is discussed that a prefab solution is most advantageous as well. This is mainly due to a higher slenderness and less construction time on site compared to an in-situ constructed deck. A prefab solution would also result in less hindrance to the A27. For more detailed information, it's referred to this chapter in the Preliminary Study.

The estimated width will be 1.66 meters and in total 300 of such beams are required. The estimated weight will be around 160 tons. Furthermore, it's assumed that the prefabricated box beams are stored at construction site 1, independent of the location of prefabrication. Figure 3.29 shows the starting situation as discussed in this paragraph. The grey lines at the background show the contour lines of the existing surrounding area. The green hatched area will be the location of The Green Connection. The red dashed area will be the construction site one and within this area, the outer dimensions of the prefabrication facility are schematized. For sake of simplicity only one prefabricated beam is visualized, with the above-mentioned dimensions. Between the construction site and the A27 a temporary road is drawn. This road is meant to provide access to the A27 so that the beams can be transported via a short transport route. The location of this road is indicative, as well as the dimensions. Furthermore, Figure 3.29 is drawn to scale.

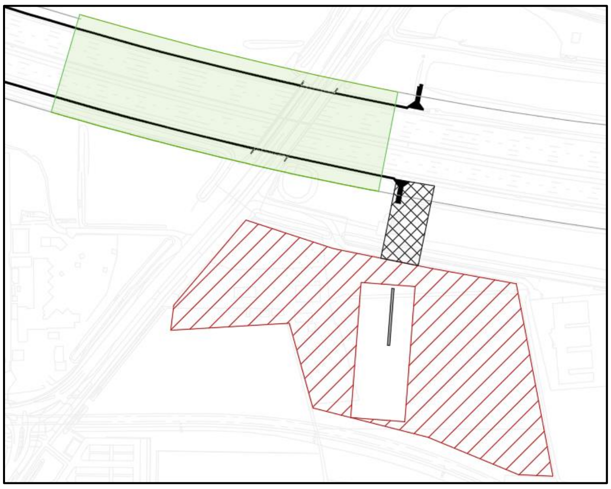

Figure 3.29: Starting situation.

In fact, the easiest way of assembling the beams is to close the motorway and hoist the beams into position one by one. However, the availability of this motorway is a key-issue of the total project which is discussed in paragraph 3.1 . Some measures or execution methods should be designed in order to minimise the duration of a closure or the number of closures of the A27. It could be helpful to design some alternatives. The variables in which these alternatives differ are the position and the number of the cranes, the manner of transportation, and the transport route for instance. In other projects, such as the Bleizo reference project, the beams were delivered in front of the structure perpendicular to the supports. In total three cranes were used. Two of them were positioned behind one abutment while the other one was positioned in front of the other abutment. Via this manner the beams could be delivered perpendicularly to the abutments, which resulted in a lever arm as small as possible.

However, if this same method is applied to The Green Connection, the lever arm will become too large since the width of this structure is almost 250 meters. It implies that after placing a few beams, the crane should be repositioned. When a few beams are already positioned, the distance between the point of attachment of the beams, and the centre of gravity of the crane becomes too large. In that case, the beams should be transported underneath the already placed beams, as schematized in the right part (b) of Figure 3.30. One other implication follows. The distance between the intermediate support and the support at the sides will be about 41 meters. The support distance at both sides is half a meter. It implies that the beam length will be 42 meters, as said before. The beams can't be transported perpendicularly in-between the supports, which has some impact to the executional aspects. Furthermore, one should consider some distance at both sides which will function as a buffer due to differentiations during hoisting. A meter at both sides is assumed to be sufficient. If this 'buffer distance' is taken into account, the fictitious length of the beam will be 44 meters. Therefore, the beams should be delivered skew between the supports. Theoretically, it implies that the lever arm will be increased with $\sqrt{44^2 - 41^2} =$ 16 meters. If the cranes are positioned ingeniously, this distance will be reduced. Figure 3.30 schematizes this thought. The bottom right side of The Green Connection is visualised in Figure 3.30. This figure is a close-up of Figure 3.29. The green line schematizes a part of the perimeter of The Green Connection. Also, a part of the temporary access road is visualized in the bottom right corner. Furthermore, it's chosen to start assembling the beams at the right side of the structure. It could also be to start at the other side of The Green Connection. Further research is required to prove whether it will be more advantageous to start at the left or right side of the structure.

It seems to be advantageous to start at this driving direction, at the south side of the intermediate support. In this case the beams can be transported to this side, while the other part of the motorway shouldn't be closed, and could theoretically be open. This topic will be discussed later.

In the left figure, the beam is delivered on two transport vehicles. This could be SPMTs or a truck and trailer combination. In this case it's chosen to use 2 SPMTs due to their manoeuvrability. Further research will be required which method of transportation is most advantageous. The cranes are positioned at just one driving direction at the motorway. The right figure shows that the beam is delivered a bit skew to overcome the difference in length. A few beams are already placed.

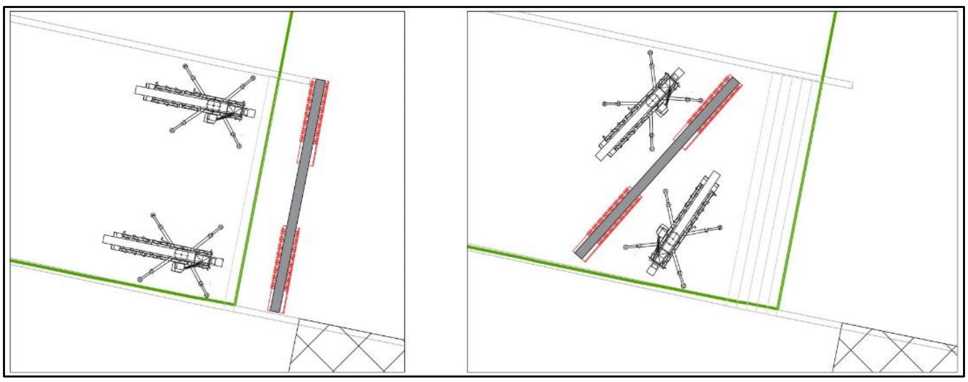

Figure 3.30: (a) Perpendicular delivery of the beams, smaller lever arm. (b) Skew delivery of the beams. Increased lever arm.

To overcome this problem of an increased lever arm, it is possible to position a crane at the other part of the motorway. But, this has some other implications related to the availability of the motorway. It can be stated though, that the first two alternatives are mentioned. The first option will have cranes only situated at one part of the A27, the other alternative will have cranes positioned at both sides of the A27.

In the previous paragraphs, the term 'lever arm' is mentioned a few times. It is a crucial aspect when large elements should be hoisted, especially the ones which are heavy. In general, a crane has a certain moment capacity. This capacity depends on the weight of the hoisted object and the length over which it should be hoisted, the lever arm. If these two values are multiplied, the occurring 'moment' can be determined. The higher this value, the tougher the crane will be which has its implications with respect to costs, time, manoeuvrability and weight for instance.

The weight of one beam is already determined to be 160 tons. It implies that at least 80 tons per beam side should be hoisted. The distance of the lever arm will be undetermined. But, if the toughest mobile crane is considered, one can make a proper estimation of this distance. According to an arbitrary company for mobile crane rental, Wagenborg, the toughest mobile crane in their equipment catalogue can hoist 80 tons over a distance of 22 meters [11]. It will be a so-called 700 tons crane. It seems that this will be a boundary condition, otherwise some extensive lifting inventions should be made or the beams should become less heavy. Some optimizations are present within this designing environment. But, the aim of this paragraph is to explicate plausible/feasible alternatives and not optimizations. Therefore, it won't be considered. Further research will be required to design an optimal solution between the beam weight, lever arm and crane capacity.

The following paragraphs will discuss the three methods of assembly of the beams for one span of 41 meter. The comparison will be made with the help of five characteristics, which are 'lever arm', 'crane', 'required space during execution', 'availability A27' and 'temporary access road'. These characteristics are chosen due to their relevance. Since a high load must be lifted, the lever arm is important. It's directly related to what kind of crane will be required, and if it will be a realistic method. With the kind of crane, a lot of accompanying considerations are present such as their self-weight, capacity and surface area. Especially this last one is of importance related to the required space during execution, since this area is quite large with tough mobile cranes. This amount of space can be related to the remaining area which can be used as available space for driving lanes at the A27, the availability of the A27. The temporary access road is the last characteristic, since some important considerations should be taken into account.

3.6.3 Alternative one

The first alternative consists of the idea depicted in Figure 3.30. This figure is drawn to scale, so everything can be easily compared/measured. The cranes are positioned only at one side of the motorway. The left part of this figure shows the starting position. If this position is kept the same, the lever arm become too large. Therefore, the right part shows the arrangement of the cranes after some beams are installed.

- **Lever arm**: The weight to be hoisted will be 80 tons at both sides. The arm can't exceed the 22 meters, otherwise a non-common hoisting measure should be thought with accompanying exceptional equipment. If the lever arm will substantially lower, a less tough crane can be rented, which is cheaper. For this alternative, the lever arm will be in the order of magnitude of 20 meters. So, with accurate positioning it will be just feasible. But, the first few beams could be hoisted without repositioning the crane, until the crane pole almost reach the deck structure. Then, the cranes should be positioned as in the right site of the figure, otherwise the lever arm will become too large and the beam can't be placed anymore.

- **Crane**: In total two cranes are required. The lever arm will be in the order of magnitude of the boundary condition. It implies that the toughest mobile cranes available must be used, a 700 tons mobile crane as specified before. Even in this case only one or maximum two beams can be hoisted into position. After placing this beam, the cranes should be relocated a few meters backwards. This relocating of the cranes could be time consuming and should be taken into account if a tight schedule is applied.

- **Required space during execution**: As can be seen in Figure 3.30, at least one half of the motorway is required during assembly, because the mobile cranes are situated over there.

- **Availability A27**: The beams are hoisted at one side of the A27. From the execution point of view, only this part of this motorway should be closed. But, when the beams are hoisted as schematized in the right part of Figure 3.30, the beams should be manoeuvred along the pole of the crane. This crane is positioned near the side support. Manoeuvring along this crane pole implies that the beam will hang above the other driving direction during hoisting. So, from the execution point of view the other side of the intermediate support could remain open but, as a safety measure nobody or no vehicle can be situated beneath a hoisted object. Therefore, taken into account safety issues, (a part of) the other driving direction should be closed as well. The assumed width which will go at the expense of the available driving space will be in the order of magnitude of 10 meters including some margins, because the hoisted beam must be manoeuvred along the pole of the crane. It implies that about 30 meters will be left for traffic in both directions. If such a partly closure of a driving direction won't be approved by the authorities, one last thing rests which is a total closure of the second half.

- **Temporary access road**: This method of assembly asks for one access road. This road should overcome a height difference of about 4 meters. Besides this height difference, another important aspect is present. At the location of the access road, the membrane structure is situated. It will be the question if a reliable road can be realized, since the load upon this road will be quite large. Further research will be required.

These five subjects will be emphasized for the other alternatives as well. After that, these three alternatives will be discussed briefly.

REMARK: *This alternative is schematized in Figure 3.30. At first sight, the cranes could be positioned visa versa compared to the cranes visualized in this figure. It implies that the hoisted beam shouldn't hang above the other part of the A27, but above the roadside. In that case, no safety area should be preserved and more available space will be left for traffic during hoisting of the beams. However, the forest of Amelisweerd is present at the roadside. Trees are situated as close as possible to the motorway. It will be the question if it's feasible to lift the beams via this manner and don't interfere with the trees, since the distance to manoeuvre is estimated to be 10 meters. For now, it's discussed that it won't be possible. After extending the motorway, crane specialist should judge whether this will be a better way of lifting the beams.*

3.6.4 Alternative two

The second alternative consists of a layout depicted in Figure 3.31.

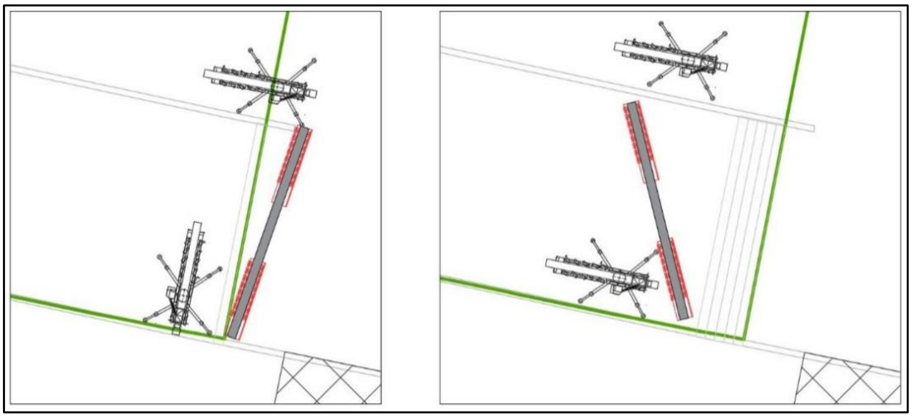

Figure 3.31: Alternative 2; One crane is positioned at the other side of the intermediate support.

This figure is comparable to the previous one, the schematization of alternative one. The left part shows the starting position. The beam will be delivered a bit skew, in such a way that the lever arm will be minimized. The right part of the figure schematized the layout when a few beams are already hoisted, and the lever arm becomes too large if the starting layout will be applied. Furthermore, the same characteristics will be discussed.

- **Lever arm**: The positions of the mobile cranes are drawn for two situations in Figure 3.31 which are present when alternative 2 is applied. The left part of the figure is at the start of assembly. The beams will be delivered at the front of the structure. In this case the lever arm will be about 10 meters. When a few beams are assembled, the situation changes as depicted in the right part of the figure. In that case the lever arm will become about 14 meters as a starting point. So, the lever arm will lie below the boundary distance.

- **Crane**: Again, in total two cranes are required. As said above, the lever arm will be about 10 and 14 meters as a starting point. But, it will depend on how many beams will be lifted without repositioning the cranes, because the lever arm will increase when more beams are hoisted without adapting the position of the cranes. If a 700 tons crane is used and the beams have a width of 1.66 meters, about 5 beams can be lifted without relocating the crane. On the other hand, a less tough crane could be used, because it has some (financial) benefits compared to the 700 tons crane. The 500 tons crane of Wagenborg can lift the 80 tons over a maximum distance of 18 meters [12]. It implies that this crane can be used as well. But in that case only three beams can be lifted without repositioning the crane. Which crane will be used is just the question. It implies that it will be feasible to lift the beams via this method.

- **Required space during execution**: With this alternative, a crane is positioned at each driving direction. It implies that one driving direction must be closed at all, because the beams will be delivered via this part. At the other side of the intermediate support, a crane is situated as well. The distance between the struts is about 12 meters for the toughest mobile crane. Also, some distance should be taken into account to place the bearing pads beneath the struts. With the help of these pads, the point load resulting from the struts of the crane can be spread gently to the substructure.

- **Availability A27**: In this case the beams are hoisted at both sides of the intermediate support. It implies that one part between the side support and the intermediate support must be closed at all. The other part only one crane is situated at the side of the intermediate support. It's prohibited to hoist objects which will hang above the driving lanes. Therefore, the distance between the struts of the crane and some margins should be subtracted from the available part for traffic. Furthermore, some margins and other safety measures should be taken into account as well as the dimensions of the bearing pads underneath the struts. Therefore, it's assumed at least these 15 meters should be subtracted from this side. It is estimated that the remaining part available for traffic will lie in the order of magnitude of 25 meters. Compared to alternative one, the same considerations are applicable related to the available driving space. Thus, it will be the question if the authorities will approve this way of working. If this is not the case, the second half must be closed resulting in a total closure of the motorway.

- **Temporary access road**: This method of assembly asks for one access road, the same number as the previous alternative. The same considerations are applicable as well.

3.6.5 Alternative three

The third alternative consists of the following layout as depicted in Figure 3.32.

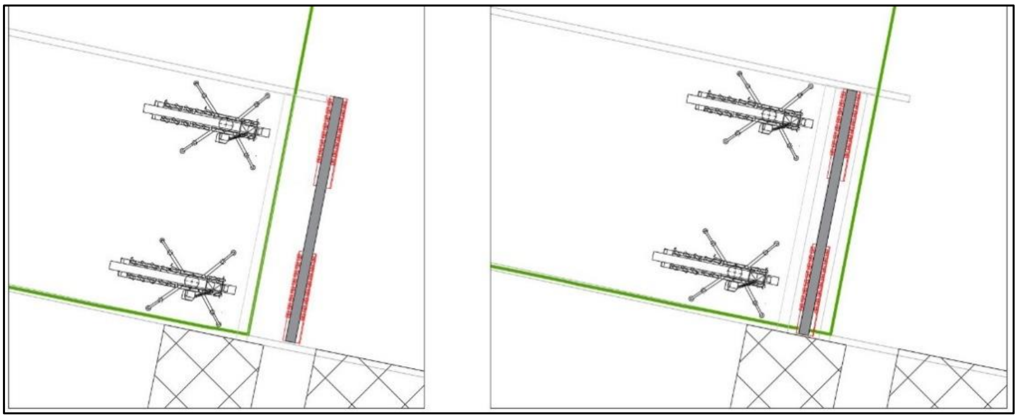

Figure 3.32: Alternative 3; both cranes at one side of the motorway, and transport of the beams via deck structure.

This figure has also two schematizations. The left figure can be compared to alternative one; two cranes are positioned in-between the support wall at the side, and the intermediate support. The right part of Figure 3.32 can be compared to alternative one as well, but the difference will be the transport of the beams. In this case the beams are transported via the deck structure. As outlined in the Preliminary study, the deck structure will be designed on a representative load of 30.2 kN/m^2. The self-weight of the beams is lower than this value. According to the a supplier, a 1800 mm high box beam weigh about 23 kN/m^2 [10]. Therefore, the beams weigh less than the representative load of the public garden on top, even when the weight of the SPMTs are considered. Besides, these transport vehicles are 2.5 meters wide [13]. So, it mobilizes at least two beams of the deck structure while it drives across it. It means that transporting the beams via the just placed deck structure could be an interesting feature.

Another point of attention will be the stability of the just placed beams which function as a deck structure. Normally, these beams are post-tensioned in transverse direction, or an in-situ casted top layer is applied. If one of these options is applied, the structure has enough stability. But, these tasks require time and time is scarce when the motorway is closed. In other words, strength will be no issue. Attention should be paid to acquire enough stability of the deck structure directly after installing the beams. It could be the case that a kind of auxiliary structure must be placed on top of the deck structure that the placed beams are immediately stable. Furthermore, the next characteristics are again emphasized for this alternative.

- **Lever arm**: The mobile cranes are schematized for two positions in Figure 3.32. In the left part of this figure, the minimum lever arm will be about 20 meters, the same as with alternative one. It implies that it will be just feasible to hoist the beams. In the right part of this figure however, the minimum lever arm will be about 14 meters. Besides, this distance could probably become even smaller, because the beams can be delivered more to the direction of the crane.

- **Crane**: As in the other alternatives, two cranes will be required. The first beams should be lifted with the toughest crane, since the lever arm is quite large. When a few beams are hoisted into position, the lever arms become smaller. In theory, less tough cranes can be used. But, the tough cranes are already positioned to hoist the first beams, so it may be advantageous to use them and reduce the assembling time.

- **Required space during execution**: This alternative three shows that 2 cranes can be positioned at one driving direction, at one side of the intermediate support. When the right part of the figure is discussed, it results in no changes with respect to the number of cranes and the required space. The difference will be that the beams will be delivered via the just constructed deck structure, the already placed beams.

- **Availability A27**: At this alternative one half of the motorway must be closed at all, because two cranes are situated here. When the beam is hoisted as schematized in the left part of Figure 3.32, no vehicles will be situated beneath the hoisted beam. The same holds for the layout as depicted in the right side of this figure. No manoeuvre to avoid the crane pole should be made, since it doesn't interfere with the hoisted object. In theory, it wouldn't be necessary to close or partly close the second half. But, in case something goes wrong, for instance a hand tool falls off and no measures are considered, it could be the situation that it falls on a vehicle which drives by. The contractor can't take such a risk, because if a certain event happens, is could result in an accident with high risk on casualties. Therefore, some buffer area should be considered with mitigating measures, such as safety net for instance or enlarged safety barriers. It is assumed that it will require at least one driving lane with some margins. In the end, if this method of assembly will be executed according this alternative, one half of the motorway or at least 35 meters will be left for traffic during construction.

- **Temporary access road**: This method of assembly asks for a second access road, in contrast to the previous alternatives. The same considerations are applicable. At the location of the second temporary access road, no membrane structure is present. It will be just normal soil conditions, but some height difference must be taken into account. This height can be estimated. According to existing drawings [2], the height of the surrounding area will be one meter above N.A.P., and the topside of the A27 will be 3 meters below N.A.P. The beam height including the bearings will be about 2 meters. The clearance at the motorway will be at least 5 meters. Taken all these considerations into account, the height difference will lay in the order of magnitude of 3 meters with respect to the existing surrounding area. This height difference must be overcome with a smooth slope, otherwise the SPMTs are not able to transport the beams from the construction site to the deck structure due to their large weight.

3.6.6 Discussing the deck structure and comparing alternatives

Before discussing the alternatives, a firm designing environment must be created. As the reference document of Rijkswaterstaat stated, a deck structure of box beams will most likely to be applied. Some important variables are the dimensions, the weight and the number of beams. These variables are interconnected. A standard beam design with accompanying height of 1800 mm will be sufficient. But, when more than 400 of such standard beams should be produced, it will be beneficial to design a more optimal beam. Since the aim of this paragraph is to discuss some obvious methods of assembly, a structural design of such a beam is not performed but estimated. The boundary condition of a beam was 172 tons, the heaviest beams transported via the existing infrastructure. In the end, the variables in considering methods of assembly were beams of 160 tons which are going to be produced 300 times and are 1.66 meters wide.

If the beams are going to be produced at the construction site one, the beams can become even heavier. But, to keep the transport possibilities open, it's chosen to determine the weight at 160 tons. This variable is an important parameter in the execution method. It is stated that the beams will be either prefabricated at construction site one or delivered to this construction site, because a certain stock yard at location reduces the risk on time delay during execution.

The topics as dimensions, weight, transport issues and stock yard at location are determined before designing an execution plan for the deck structure, because further trade-offs in the method of assembly will depend on these values. Furthermore, it's assumed that the intermediate support is realized, although this topic is quite a complex and risky task as emphasized in paragraph 3.5 . One must keep in mind that this starting point is based on some assumptions. Although these assumptions are substantiated and seems to be reasonable, a lot of variations and implications are possible which can crucially influence the final method of assembly.

The easiest way to assemble the deck structure is when no traffic is present at the motorway. But, the key-issue of the execution aspects will be to what extend hindrance will occur to the A27. It will be unacceptable to close the A27 for long period of time, especially when it's not completely necessary. For that reason, three alternatives were outlined in the previous paragraphs to consider possible methods of assembly. A solution is sought in the method of assembly where traffic should suffer as less as possible from the hindrance of constructing the deck structure. At every alternative, the most important value is the availability of the A27.

The position of the cranes is discussed at the start of assembly followed with the situation after placing some beams. The lever arm will become too large then, if the position of the cranes is kept the same. With a normal execution method like this, no suitable crane will be available. Therefore, the cranes must be repositioned.

In fact, the type of crane will have quite some impact. With the alternatives discussed, it is assumed that the only suitable type of crane will be a mobile crane. Such cranes are also used in reference projects, as outlined in the Preliminary Study. However, if a crane specialist is involved within the design method of assembling the beams, it could result in a more optimal choice of crane. Variations in crane type could occur, such as using ring cranes or crawler cranes. This will be another topic or optimization than what is aimed for in this chapter.

Considering these three alternatives, five characteristics are discussed based on the starting criteria, e.g. dimensions and weight. One important finding is that the beams of 160 tons must be lifted with two mobile cranes of at least the so-called "500 tons crane", depending on the alternative. In alternative one the only possibility will be to use two cranes of 700 tons, otherwise the capacity of the cranes won't be sufficient. However, these large and tough cranes are required if the beam width is 1.66 meters and weigh about 160 tons. If the original beam width will be used, the beams weigh 40 tons less. It implies that another less tough crane will be suitable as well. If the same tough cranes will be used, more beams can be hoisted while the cranes don't have to be repositioned. It's important to indicate that if a variable changes, it has some effect on other design considerations, such as the type of crane for instance. In other words, a relation can be found between the weight of the beam, the lever arm and the toughness of the crane.

One of the five characteristics discussed is the lever arm. It's an important parameter, especially because of the weight of the hoisted beam. Which crane should be used depends on the length of this arm. It will be just the question in what way the beams will be hoisted, because the arm differentiates. If this arm is kept as small as possible, less tough cranes can be used which has some financial benefits.

Furthermore, the differences between the alternatives with respect to the positions of the cranes are visualized in Figure 3.30 to Figure 3.32. The struts of the cranes are completely positioned in front of the structure, while in reality these struts could be (partly) positioned below the deck structure. It results in a reduction of the lever arm. But, one must consider the pole of the crane. This pole has some deflections while hoisting and the rotations around its axle. These movements require some space. Furthermore, the figures are meant to indicate/estimate the lever arms. To apply optimizations in positioning the cranes, it's recommended to consult a crane specialist. In that case, the lever arm could probably be reduced even more.

The positioning of the cranes and the transport of the beams require a considerable amount of space. This space could be interpreted differently. Alternative one needs at least one driving direction to be closed, while the second alternative also needs a reasonable amount of space of the second half. The third alternative only needs one half of the motorway. It implies that the alternatives differ in the minimal amount of space required for construction purposes.

If alternative one and two are compared, it can be stated that minimizing the required space results in the use of a tougher crane, because of the increasing lever arm. It has a positive effect on the remaining space for traffic at the A27. However, if the difference in availability is discussed, a buffer for safety should be applied, because of the hoisted beam should manoeuvred along the crane pole. Due to this safety buffer, the available space between alternative one and two won't differ that much. The difference will be about 5-6 meters. It will be just the question if the client is willing to pay the contractor extra in order to use tougher equipment and provide an extra 5-6 meters of available driving space. On the other hand, 6 meters corresponds with two driving lanes if a temporary road arrangement is applied with reduced driving lane width. These two extra lanes provide some added value during the execution phase.

The biggest difference between the first two alternatives and the third one is the way of transporting the beams After placing the first few beams, the remaining ones can be delivered via the deck structure. It will be beneficial to several topics. Less tough cranes could be used since the lever arm becomes smaller. Furthermore, it won't be required to manoeuvred along the crane pole, and no trees will be an obstacle with respect to lifting. However, the contractor must first design and realize a secondary transport road to assure access to the deck structure. A certain height difference should be overcome in a relative short distance. Attention should be paid to this topic.

The second temporary access road provides access to the deck structure. Via this way, it will give the possibility to transport the beams via the deck. Since the design load on the beams is significantly higher than the self-weight of the beams, transport of these beams will be possible across the deck structure from structural point of view. A point of attention will be the stability of the just hoisted beams. If the beams are transported via the deck structure, the amount of hindrance to the A27 can be influenced quite a lot. No beams have to be transported via the A27, and with some precautionary measures it could be possible that only one half of the A27 should be closed. It has also advantages regarding the position of the crane and the toughness of it.

Paragraph 3.6 only discusses some methods of assembly for hoisting the beams for one span. A lot of alternatives and other considerations are possible for the second span as well, probably the same as for the first span. It is decided to emphasize only one span, since the design considerations are comparable. However, it's recommended to pay attention to the method of assembly of the second span. An interesting question concerns position the cranes? Will it be possible to position these mobile cranes on top of the deck structure? A possibility could be to transport the beams for the second span via the deck structure if the first span is already covered with beams. Is it allowed to transport beams across this structure, while motorists are driving through this part of the deck structure? Questions regarding the design considerations can be performed, but it's not the aim of the chapter.

Concludingly, this chapter indicated some design considerations related to the lever arm, crane capacity and remaining available space. In fact, a lot of other alternatives and variations can be thought as well. Other starting variables will result in different results. It implies that a lot of other alternatives are still present. In other words, the topic "method of assembly" can be a study of variations in itself. It will be recommended to elaborate on this topic, since only three alternatives are discussed. If a cost engineer is included within the design of the final method of assembly, it probably will result in the most optimized solution. However, the aim of this paragraph is to discuss some obvious methods of assembling the deck structure, and provide an answer to the following question:

How to realize the deck structure in an obvious way?

This question is answered with the help of discussing three alternatives. It seems that the three alternatives are feasible to execute. It's important to indicate that with some extra investments, the remaining space for traffic could be maximized. It will be the consideration by the authority Rijkswaterstaat, what kind of financial compensations it offers to reduce the hindrance at the A27 during construction. The question arises if this organization is willing to pay this money.

4. The two global designs discussed in the Preliminary Study

From the Preliminary Study can be deduced that two types of structural designs need further research. The question to answer after this chapter will be:

> *Which of the two global designs will be the most suitable solution for constructing The Green Connection as a single span design?*

In order to provide a proper answer, several aspects are going to be discussed concerning both designs. First, the information about the particular designs of the previous report is outlined. After this introduction, a more global design is presented with the help of some basic calculations which relates to the rough dimensions, the amount of cubic meters concrete, the number of pre-stressing tendons and/or reinforcement and the accompanying weight.

Furthermore, a global execution plan will be performed which consists of several topics. The execution aspects of the structural design will be emphasized, followed by the possible methods of transportation.

4.1 Custom-made box beams

These custom-made box beams are rectangular beams filled with polystyrene blocks in order to reduce weight. The longest box beams ever made were 68 meters long and weigh up to 240 tons [14]. If box beams like this are the solution to construct The Green Connection, the length must be increased to 75 meters. Besides, what was the idea discussed in the previous report?

The current idea will be briefly discussed. More detailed information can be found in the Preliminary Study. For now, it can be described via the following three steps and an accompanying figure:

1. Design an optimal cross section of a pre-tensioned precast box beam which can resist the self-weight of the beam and at least the in-situ casted top layer.

2. On top of these beams, cast an in-situ layer to provide extra structural height and increase the compression zone capacity.

3. Apply post-tensioning in the hollow cores of the beams. Main idea is that the amount of post tensioning will balance the remaining load resulting from the public garden.

These three steps are visualized in Figure 4.1:

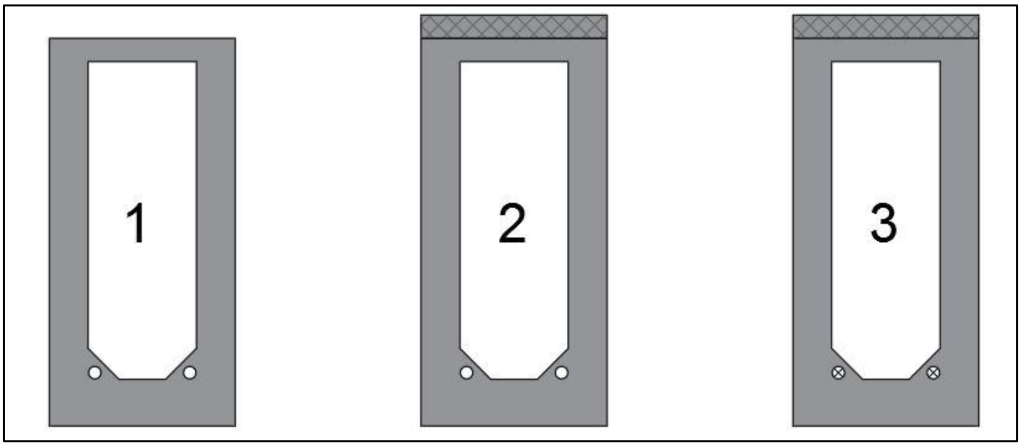

Figure 4.1: Global design phases of the custom-made box beam.

4.1.1 Design process

Some remarks have to be made with the above-mentioned 3 steps. The box beam which could be applied here will be larger than the biggest one ever produced. Besides, the load on top of the structure is almost twice as high (specified in 10).

Keeping this information in mind, it's expected that the box beam will be heavier than the 280 tons if the conventional design approach is used. It's the question if the required total amount of pre-stressing force can be applied since the load is about twice as high than the load on the 68-meter-long beams.

Considering the expectation that the weight criterion will be exceeded, a weight reduction must be applied. But how to achieve such a reduction without diminishing the capacity of the beam? In general, the larger the span, the higher the contribution of the self-weight of the structure will be. Therefore, the amount of concrete should be reduced to achieve a weight reduction. But, the bottom flange must be thick enough to accommodate all the pre-stressing tendons. The webs need sufficient thickness to prevent buckling and ensure the stability of the beam. It must also consist stirrups to ensure enough shear capacity. The only part left is the upper flange. This flange is in compression and might be reduced in height. Normally the thickness of the upper flange in the prefab beam is thick enough to resist the compression forces resulting from the own weight and the live loads.

However, in this case the idea is to minimize the thickness of the upper flange with such an amount that the flange has just enough thickness to resist the self-weight of the structure and the in-situ casted layer on top. This layer will be necessary to have enough resistance against the load of the public garden. Until this point one, the following remarks can be made:

- It is assumed that if the conventional design approach is used, the beams are too heavy and exceed the criterion of 280 tons weight. Therefore, this weight reduction is applied.
- If a calculation proves that the beams are not exceeding this criterion, the conventional design method is used which consists of just pre-tensioned tendons and a full prefabricated concrete cross section.

Due to the own weight of the prefabricated beam and the higher live load resulting from the public garden on top, a relatively high pre-stressing force is expected to be necessary to apply in the prefab beams. If it turns out that this force will be too high as well, or not enough space is available to accommodate all the pre-stressing tendons, an adaption must be made. In that case the idea is to pre-stress the beam as much as possible/allowable. The amount of pre-stressing force will be high enough to create a beam which can resist its own weight and the in-situ casted concrete top layer (if this layer is applied). Depending on this value, it might be carrying a part of self-weight of the public garden. The remaining part of the load resulting can't be resisted yet by the prefabricated beam. After hoisting the beams into position and after sufficient hardening of the in-situ casted concrete layer, the structure still lacks resistance to resist this remaining part load. The resistance can be increased by applying sufficient post tensioned steel. Then, the beam has reached the required resistance, and the soil layer on top of the structure can be applied to realize the public garden on top.

However, the design process outlined in the paragraphs above assumes that a design according to the conventional approach is not feasible. A design according to a conventional approach only consists of pre-tensioning cables and concrete constructed in a prefabrication yard. The load in combination with the relatively large span seem too vast in order to construct such a design. But, this can't be taken for granted and it must be proven. Therefore, a global design is performed according to this conventional approach to prove the infeasibility.

In short, the first global design can be performed according the following flowchart, visualized in Figure 4.2. This flowchart is a very schematic way of the design approach discussed in this paragraph.

First, a global design is performed according to the conventional approach. If this design isn't feasible, the option of the precasted beam with an in-situ concrete layer in combination with post tensioning will be outlined. However, if such a design will be feasible, it implies that a fully precasted beam solution is possible from structural point of view. Therefore, the more optimal solution with an in-situ casted concrete top layer and applying post tensioning will be definitely feasible and won't be considered anymore.

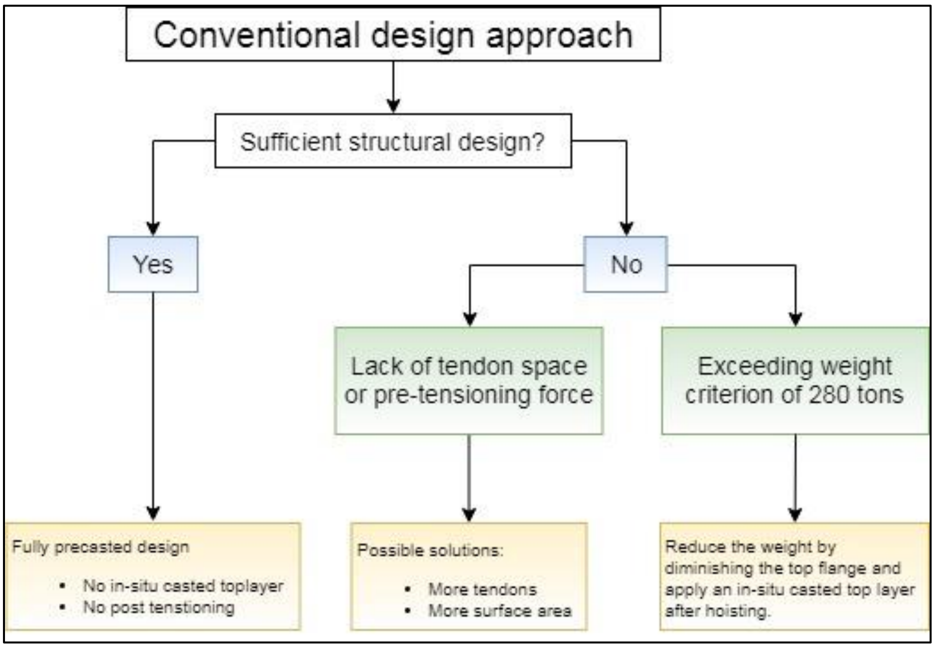

Figure 4.2: Flowchart design procedure global design 1.

4.1.2 Characteristics

Considering the high load and large span, an initial high concrete strength of C90/105 is chosen to be the point of departure. The following values are extracted from the Eurocode 2, NEN-EN 1992-1-1:

Density	ρ	26 kN/m^3
Characteristic compression strength	f_{ck}	90 N/mm^2
Characteristic tensile strength (fifth percentile)	$f_{ctk,0.05}$	3.5 N/mm^2
Mean modulus of elasticity	E_{cm}	44 000 N/mm^2

Table 4.1: Overview properties of concrete C90/105.

From Table 4.1 the following design values can be calculated:

The design compressive strength of concrete C90/105:

$$f_{cd} = \frac{\alpha_{cc} * f_{ck}}{\gamma_c} = \frac{1.0 * 90\ N/mm^2}{1.5} = 60\ N/mm^2$$

With $\alpha_{cc} = 1$ and $\gamma_c = 1.5$

The design tensile strength of concrete C90/105:

$$f_{ctd} = \frac{\alpha_{ct} * f_{ctk,0,05}}{\gamma_c} = \frac{1.0 * 3.5\ N/mm^2}{1.5} = 2\frac{1}{3}\ N/mm^2$$

With $\alpha_{ct} = 1$ and $\gamma_c = 1.5$

4.1.3 Design calculations

The design calculations of the box beam consist of roughly two checks. The first one is to determine the minimum pre-stress force to resist the bending moment. The second one consists of a shear force check.

Pre-stress force

One important criterion is to determine the height of the pre-stress force to get an estimation of the amount of pre-stressing tendons. These tendons will be pre-stressed and have to be applied in the bottom flange (at mid-span). It implies that the dimensions of the bottom flange of the beam are highly dependent of this number. This force can be calculated according to formulas. The following formulae are necessary to determine the height of the pre-stressing force. The procedure consists of two formulae after cutting the tendons (t=0) and another two after a certain period of time when the full load is applied (t=∞). Initially, no tension forces may occur at the top fibre and no higher compression stresses than $0.6 * f_{ck}$ may occur. The same holds for the stresses in the deck structure at final stage. In this stage 20% of pre-stress losses is assumed. It results in the following formulae:

Top fibre at t = 0:

$$+\frac{F_p * e_p}{W_{top}} - \frac{F_p}{A_{tot}} - \frac{M_g}{W_{top}} \leq 0 \tag{1}$$

Bottom fibre at t = 0:

$$-\frac{F_p * e_p}{W_{bot}} - \frac{F_p}{A_{tot}} + \frac{M_g}{W_{bot}} \geq 0.6 * f_{ck} \tag{2}$$

Top fibre at t = ∞:

$$+\frac{0.8 * F_p * e_p}{W_{top}} - \frac{0.8 * F_p}{A_{tot}} + \frac{M_{gq}}{W_{top}} \geq 0.6 * f_{ck} \tag{3}$$

Bottom fibre at t = ∞

$$-\frac{0.8 * F_p * e_p}{W_{bot}} - \frac{0.8 * F_p}{A_{tot}} + \frac{M_{gq}}{W_{bot}} \leq 0 \tag{4}$$

Shear force

The second check is to determine the maximum shear force capacity of the prefabricated beam. If this shear resistance is higher than the maximum shear force acting in the structure resulting from the load, the capacity of the structure is sufficient with a suitable shear reinforcement configuration. Therefore, also the maximum shear force resistance is calculated according to the next formula:

$$V_{Rd,max} = \frac{\alpha_{cw} * b_w * z * v_1 * f_{cd}}{\cot(\theta) + \tan(\theta)} \tag{5}$$

4.1.4 Calculation method

If the formulae in the previous paragraphs are going to be solved, some other parameters are required which are related to the deck structure and thus the box beams. However, especially this deck structure is just going to be determined and the dimensions of the box beam are still unknown. If parameters such as height and flange thicknesses change, the moment of inertia and the section modulus will change too. In other words, to come up with a sufficient cross section of the box beams an iterative calculation procedure is necessary.

Such a procedure is very time consuming and sensitive to mistakes if these calculations are provided by hand. Therefore, the calculation is done with the help of an Excel sheet in combination with Maple, an extensive mathematical tool. This Excel sheet and the design calculations are attached in Appendix A. The first page consists of an Excel sheet and the next pages explicate the calculations performed via Maple.

First, an initial calculation is conducted via some estimated dimension of the cross section of the box beam. As seen in the previous paragraphs, formulae (1) to (4) consist of values such as section moduli and moment distribution due to self-weight. If one value changes, it incorporates a change in the other value. Therefore, these values are programmed via an Excel sheet. The values can be found in Appendix A.

These values are inserted in the second program, Maple. This mathematical tool can easily calculate with unknown characters and is easily programmable. Via this program, the above stated formulae (1) to (4) are determined and solved.

As seen in the appendix, other parameters are determined as well, such as the spacing of the tendons for instance. In essence, every tendon can be stressed with a certain force. The higher the required pre-stressing force, the greater the number of tendons. But the bottom flange must have enough space to accommodate this number. The calculation and the spacing of the tendons is also done via the Maple sheet.

In this maple sheet an assumption is made about the strand diameter and the cover of the beam. With these values, the centre to centre distance of the tendons could be determined according to the Eurocode. A picture retrieved from the Eurocode which shows the minimum tendon spacing can be found in this appendix as well.

The maximum shear force resistance is also determined via the same Maple sheet. Again, some values from the Excel sheet are inserted in the Maple program. Via this way the maximum shear force resistance is determined.

4.1.5 Result global design

In the same Appendix A some results can be found. The values can be summarized per box beam. These values are outlined in the Table 4.2 and Table 4.3.

	abbreviation	amount [mm]
height	h	3000
width	w	1000
top flange thickness	h_t	220
bottom flange thickness	h_b	370
web thickness	b_w	150

Table 4.2: Overview dimensions of a box beam.

weight	280 tons
number of pre-stressing tendons Ø15.7	136 [-]
amount of concrete	108 m^3

Table 4.3: Overview key-figures of one box beam.

In Figure 4.3 a sketch is shown to visualize the dimensions of the cross section of the beam. This global design consists of 136 pre-stressing tendons. The beam width is one meter and since The Green Connection has a width of 249 meters, in total 249 of these beams have to be produced at the temporary prefabrication yard on site.

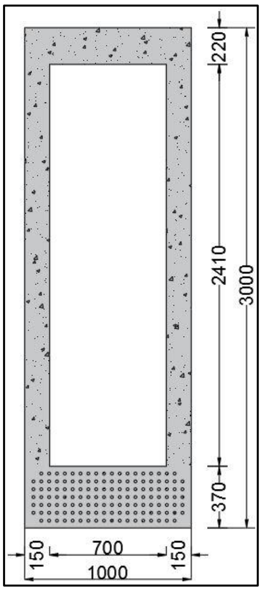

Figure 4.3: Global design one, units [mm].

The construction of these 249 prefab box beams result in some key-figures. These values are emphasized since it can be easily compared with the second variant. The key-figures are a rough estimate and can be seen at the last page of the Maple file in Appendix A. The following values indicate the amount of materials which are required for constructing the deck structure with these box beams.

- About 26 800 cubic meters of concrete
- About 33 900 pre-stressing tendons of Ø15.7

4.1.6 Execution method

In paragraph 2.1.1 (specified in 4) it is discussed that the production of the deck structure will be realized at construction site number one as visualized in Figure 2.1. In this paragraph, it's assumed that the production of the deck structure consists of the prefabrication of 249 precast pre-tensioned concrete box beams.

Furthermore, the method of execution will only be discussed briefly, since the executional aspects are a study on its own. A prefabrication yard is going to be realized at the construction site. The dimensions are 85 meters long and 40 meters in width roughly.

In line with this 85-meter length of the prefabrication yard the stock yard is situated. This stock yard must have a length of at least the length of a box beam because it must store these beams. It results in a total rectangular area of 160 meters in length and 40 meters in width. The gantry crane must have access to this entire area in order to lift the beams form the production hall to the stock yard. After a sufficient stock the beams can be lifted upon a heavy-duty truck. Depending on the weight and axle load, the method of transportation could be done with the help of SPMTs[4], but further research will be required to judge which transport method is better to apply.

[4] SPMTs: Self-Propelled Modular Transporter, used to transport heavy loads.

In chapter 7 of the previous report the reference project Bleizo is discussed. In this project in total 31 precasted beams stored at a stock yard nearby. These beams were hoisted into position within one weekend closure. This project shows the feasibility of the criterion of 31 beams per weekend closure. The dimensions of these beams were a bit smaller than the ones used in this project, but in essence the same procedure is necessary. Therefore, a comparable criterion will be used in this project. In total 249 beams must be produced in order to construct The Green Connection. If this number of beams is taken into account, about 8 weekend closures should be taken place. It implies about 31/32 beams per weekend should be transported from the stock yard via the construction site and the temporary road to the location of The Green Connection. At this location the beams are going to be hoisted into position by tough mobile cranes which have the capacity to hoist about 140 tons each over a certain distance.

Figure 4.4 shows a close-up of the left part of Figure 2.1 and is drawn to scale. The green hatched area is the predetermined location of The Green Connection. The walls of the current U-shaped concrete structure are drawn in black as well. These walls are demolished after constructing the extended parts. The widening is clearly visible in comparison with the old walls. The dimensions of the hatched green area are 249 meters by 75.

The striped red area is construction site number 1 of Figure 2.1. Within this area, a rectangle with dimensions of 160 by 40 meters is drawn. It schematizes the prefabrication hall and the stock yard in front of this hall. The stock yard is positioned in line with the hall, otherwise the stock yard lies out of range of the gantry crane. To emphasize the scale of the figure even more, a black line is drawn at the location of the stock. It schematizes one box beam with the dimensions specified in the beginning of this chapter (75x1 meter). Between the construction site and the A27 a temporary construction road is designed. This road is necessary to transport the beams to the location of The Green Connection. The position of the prefabrication hall and the stock yard is designed in such a way that the transport of the elements can be easily made, taken into account the curvature from the construction site to the A27.

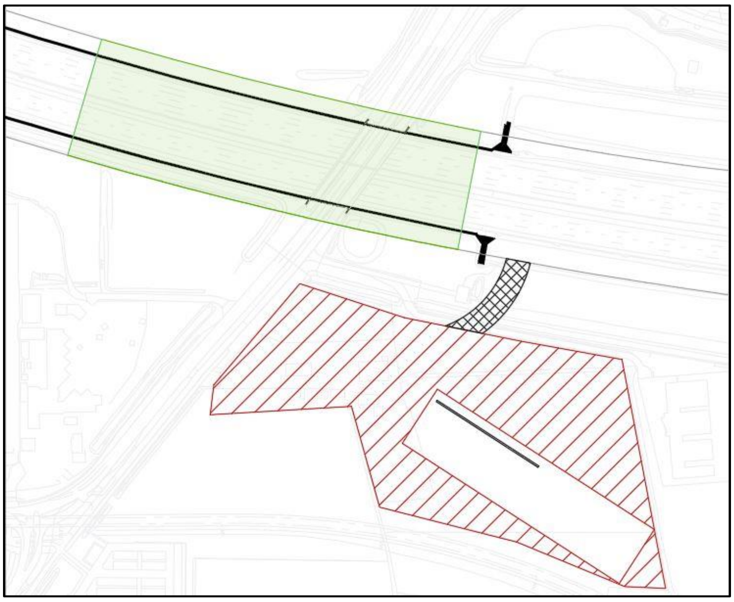

Figure 4.4: Schematic close-up of The Green Connection and construction site 1 for design two [2].

As said before, the assembly of the beams can only take place when the widening of the A27 at both sides has already taken place as well as the construction of the abutments. When these tasks are executed, one must start the construction of the building site. The idea implies that when the motorway is extended at both sides and the new abutments are finished, also the first shift of the 31 precasted elements are finished. Then, the first weekend closure can take place and the first shift of beams can be assembled.

The mobile cranes can be situated at the extended parts of the highway. Due to the high weight of beams, large pressures are assumed to be present at the position of the cranes. In paragraph 2.1.2 (specified in 6) it is stated that the underwater concrete floor of the extended part is not considered. It is assumed that this extended part will have a comparable capacity than the current one. However, if it turns out that the floor lacks capacity, an idea could be to design the extended floor part of the U-shaped concrete structure in such a way that it could resist the high strut forces from the mobile cranes.

One remark must be made according to the executional plan. In front of the U-shaped concrete structure a membrane structure is present [15]. This is also emphasized in paragraph 2.6 of the previous report. In that report as well as in paragraph 2.1.2 (specified in 5) of this report it's outlined that this structure won't be considered. It lies outside this scope of the research. However, it must be kept in mind when the temporary road is constructed from the building site to the A27. This membrane structure is of importance and must be kept undamaged, otherwise the stability of the motorway in front of the U-shaped concrete structure could become in danger. Although the quality of this membrane structure is unknown, the possibility on cracks in this layer must be avoided. The risk of cracks in this structure must be reduced as much as possible. Cracks in this layer can occur according two principles. The first one is too much or (locally) too heavy loading. This could occur during transport of the prefabricated elements. The second one is the opposite. If the load upon this membrane structure is just too low the layer could burst up due to the water pressure underneath this structure. It can lead to cracks and stability problems. This could also occur when the temporary road is applied. In other words, this membrane structure must be kept in mind when transport of the deck structure takes place from the building site to the A27.

Besides the attention paid to membrane structure, some other aspect must be considered. According to the existing drawings a difference in height of about 4 meters can be deduced between the A27 and the surrounding area [16]. To overcome this difference in height a very smooth temporary construction road must be created. This road must be designed in such a way that it won't tear the membrane structure. Nevertheless, the structure must stay intact during loading as well. Transport of the elements results in a heavy load acting on the ground surface and must be checked whether the structure can resist it. Nevertheless, it can be roughly said that the risk on cracks in the membrane structure increases when the load upon the temporary road heavily increases or decreases.

4.1.7 Conclusion preliminary design one

A design according to the conventional approach seems to be feasible. It consists of the following key-points:

- Constructing a prefabrication yard with a stock yard and a gantry crane.
- Constructing 31-32 elements 8 times
- Constructing a temporary road from the building site to the A27.
- In total 8 weekend closures are necessary to construct The Green Connection.
- The box beams are heavy elements (280 tons), although they can still be hoisted.

The structural design of the precast pre-tensioned box beams consists among other things of 136 tendons, 108 m3 concrete and individual weight of 280 tons.

4.2 Segmental bridge construction; The single cell box girder bridge

In chapter 6.3 of the Preliminary Study this idea is described in more detail. In this paragraph, a briefer version is described again to introduce this version. For more information can be referred to the paragraph 6.3 of the Preliminary Study.

The idea ‘segmental bridge construction; the single cell box girder bridge’ consists of in total three versions. Version one is a prefabricated bridge structure of about 12 meters in width and 75 meters long. This structure is a single cell box girder and about 20 of such deck structures must be constructed in order to realize The Green Connection. Transportation of this structure will be done via SPMT’s from the adjacent building site to the final location. The elements will be vertically jacked to their final height, because lifting by a mobile crane will be unfeasible since the elements are too heavy.

The dimensions of the deck structure of the second and third idea are the same, only some differences occur in the method of execution. In this case not a structure of 12*75 meters is constructed, but three elements of about 25 meters long and 12 meters in width. Transport of these elements will be less tough. It means that instead of 20 elements of 75 meters long, 60 elements of about 25 meters long have to be produced. The difference in version two and three is visualized in Figure 4.5. In second option the first elements are positioned at the sides, while the third option the first elements are positioned in the middle of the high way. In both cases the elements will rest on temporary supports.

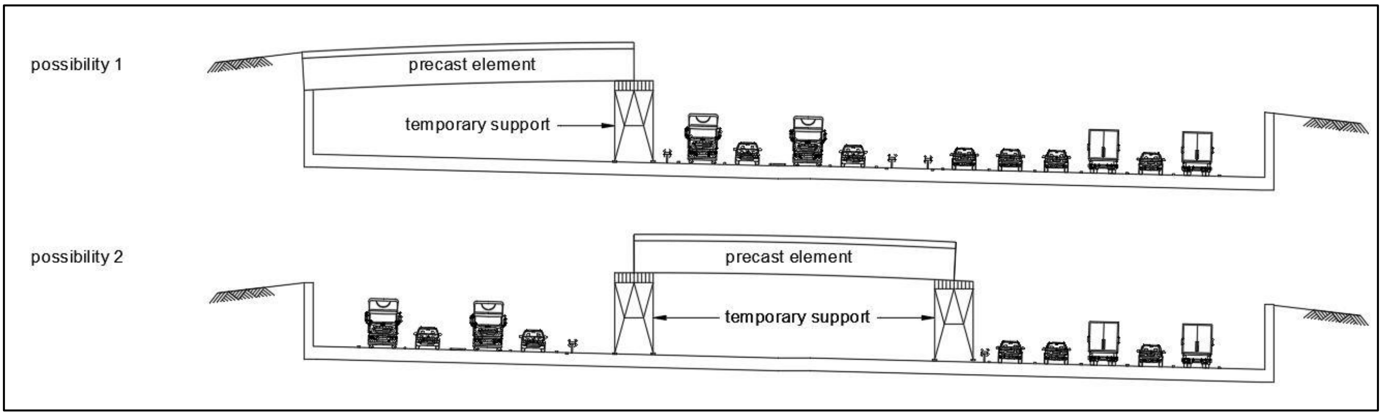

Figure 4.5: Option two and three of the segmental bridge construction idea.

After the positioning of one element as shown in Figure 4.5, the subsequent element can be positioned and post tensioned to the previous element. However, applying post tensioning is mostly done in a subsequent manner, and not in one direction and followed by post tensioning in contra direction. This post tensioning in contra direction is necessary if the third version is considered. Relatively high pre-stress losses occur when the elements are post tensioned in the third version in comparison to the second one. Nevertheless, an engineering judgement is made and it seems that alternative two and three are too cumbersome, extensive and risky.

Therefore, it’s chosen to only emphasize the first idea of the three versions. This alternative consists of one part of 75 meters length and 12 meters wide. The following preliminary design procedure, calculations and execution method are performed for this variant. The two other variants won’t be considered anymore.

4.2.1 Design process

As can be concluded from the Preliminary Study, a single cell box girder seems to be a feasible deck structure in order to realize The Green Connection. Since the load model resulting from the public garden and the variable crowd loading is about twice as high compared to the normal traffic load, the slenderness ratio can be roughly divided by the square root of two, as explained in appendix D of the previous report.

The structure will be a prefab deck structure because it will be constructed at the building site and transported to the location of The Green Connection. But, the method of construction here will be a kind of in-between a prefab and an in-situ casted method of construction. At the building site, the single cell box girder will be constructed via an in-situ method, and will be temporary supported during casting if required. A common way of constructing bridges like this is via the so-called 'span-by-span' method. For more information about this method is referred to the Preliminary study. The rules of thumb of an in-situ casted method are applied here. After constructing one element at its total, it will be transported as a prefab structure to The Green Connection.

Before using these rules of thumb, some remarks must be made about the design. As stated in the Preliminary study, this kind of structure is realized with the help of post tensioned steel, which will also be applied in this design. However, post tensioning is mostly applied after installing the bridge in its final position or to strengthen a bridge which is already in use. This is not the case with the preliminary design. It's going to be constructed at the building site and after hardening the post tensioning will be applied. Although it will be quite arbitrary, the same design procedure is applied here as with the pre-tensioned box beams. The awareness is present while dimensioning this bridge deck, but it provides a rough estimation about the cross-section, the self-weight and the amount of cubic meters concrete and the number of post tensioning cables. If a more detailed calculation is going to be performed, the design procedure should be reconsidered. A more detailed method of calculation is recommended.

Continuing the design process, a slenderness ratio of an in-situ casted single cell box girder bridge is about 25. This value should be divided by the square root of two. It results in a slenderness ratio of 18 for this structure. The dimensions of a single cell box girder are designed according to the rules of thumb for a box girder bridge design. These rules are explicated in Table 4.4, and the abbreviations are visualized in Figure 4.6.

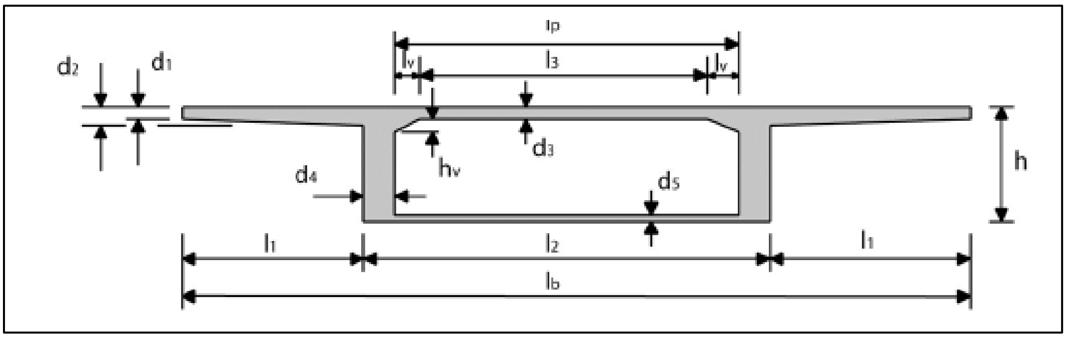

Figure 4.6: Abbreviations dimensions box girder [17].

Part	Rule of thumb	Length/thickness		Part	Rule of thumb	Length/thickness
l_b	$l_b = 12$ m	12 m		d_2	$d_1/d_2 =$ ca. 0.75	0.5 m
h	$l_{span} / 18 = 4$ m	5 m		l_p	$12 - 2{*}l_1 - 2{*}d_4$	5.5 m
l_1	2 to 3.5 m	2.75 m		d_3	$d_3 > lp/30 = 0.25$	0.30 m
l_2	$12 - 2{*}l_1$ and 5-7 m	6.5 m		l_v	$l_v/l_p < 0.2 = 1.1$ m	1 m
d_4	$d_4 > 0.35$ m	0.5 m		h_v	$h_v = d_3$	0.3 m
d_1	$d_1 > 0.25$ m	0.3 m		d_5	$d_5 > 0.15$	0.3 m

Table 4.4: Rules of thumb belonging to Figure 4.6 [17].

REMARK: *After performing the first calculation, the values according to the rules of thump where not sufficient. Therefore, other values are chosen after some iterations. This will be discussed in the following paragraphs.*

4.2.2 Characteristics

To perform a fair comparison, the same characteristics are valid for this design as for the previous design. These values are explicated in 4.1.2.

4.2.3 Design calculations

In this paragraph some design calculations are performed per deck structure element with rough dimensions of 12 meters in width and 75 meters length. Parts of the calculations are the topics self-weight, bending moment resistance and shear resistance. These topics were also dealt in design number one. The same approach will be followed are as well.

Self-weight

The cross section showed in Figure 4.7 is acquired in the program AutoCAD. This program can calculate the surface area. This area adds up to 11.9 m^2. So, per running meter 11.9 m^3 concrete will be necessary. One single cell box girder bridge of 75 meters long will consist of 893 m^3 concrete. The self-weight of the concrete is taken as 26 kN/m^3, because of the expected high reinforcement ratio and the high number of post tensioning cables. It implies that one part of the deck structure will weigh about 2300 tons.

Post-tensioning force

As described in paragraph 4.2.1 the same formulae are applied for determining the amount of post-tensioning steel and the number of cables. For this preliminary design, it's assumed that 25% of the tension forces will carried by the ordinary reinforcement and 75% will be carried by the post-tensioned steel. The formulae can be found in paragraph 4.1.3.

Post-tensioning cables

The previous design was conducted with pre-tensioned tendons, while in this case post-tensioned cables are used. Which kind of cables and how many will depend on the post-tensioning force. All kinds of variations are present. The number of cables must fit in the bottom flange of the single cell box beam, or at least the biggest part of them.

Shear force

The shear force capacity will also be determined in the same way as for the box beams. The $V_{Rd,max}$ will be determined. If this shear force is larger than the external design shear force, the structure will be reliable on shear force capacity with adequate reinforcement.

4.2.4 Calculation method

The calculation is going to be performed via the same way as for the box beams. It implies that an iterative procedure will be followed via the help of two programs. An Excel sheet is programmed and the data in this sheet is used in the programmed Maple sheet. The Excel sheet contains a cross-section of the single cell box girder. With this particular cross-section, values like the section modulus and the moment of inertia are determined. These values are required for the subsequent calculation in Maple. When the calculation proved that the design was insufficient, the cross-sections is adapted in the Excel sheet. Only with adapting the dimensions the sheet automatically calculated the new values for the section moduli are calculated. These values are again loaded in Maple and the calculation and executed another time. This iterative procedure is executed multiple times. In what way the section modulus and moment of inertia is acquired, is explicated in Appendix B. The final calculation of the single cell box girder is provided in Appendix C. In this appendix the Excel sheet is provided first, followed by the calculation in Maple.

Furthermore, within the iterative design procedure, a choice in configuration of the post-tensioning cables should be made. After the required post-tension force was determined, the surface area of the amount of steel could be determined. With this number an accurate configuration must be chosen. This will be discussed in the next paragraph.

4.2.5 Result global design

The dimensions, mechanical properties and other values can be found in the calculation attached in Appendix C. The rough dimensions are already outlined and summarized in Table 4.4. Furthermore, the amount of required surface area of post-tension steel results from the post-tension force. It's chosen to use a 7-wired strand with a surface area of 150 mm², nominal diameter of 15.7 mm and tensile strength of 1860 MPa. One cable consists of 24 of these strands. In total 24 of such cables were required. Information of the post-tension cables is acquired via a document of BBR [18]. The values which are used for this design are summarized in the next table.

Gross weight	± 2300 tons		**Cable:**	24*7-wired strands
			surface area	3600 mm²
Strand:	7-wired strand		**maximal nominal diameter**	340 mm
surface area	150 mm²			
nominal diameter	15.7 mm		**Amount of concrete**	± 900 m³
Tensile strength	1860 MPa			

Table 4.5: Overview.

In Figure 4.7 the result of the calculation is shown. The cross-section of a single cell box girder is visualized and the circles in the bottom flange schematize the nominal diameter of the anchor plates of the post-tensioned cables. In total at 20 of such structures should be constructed.

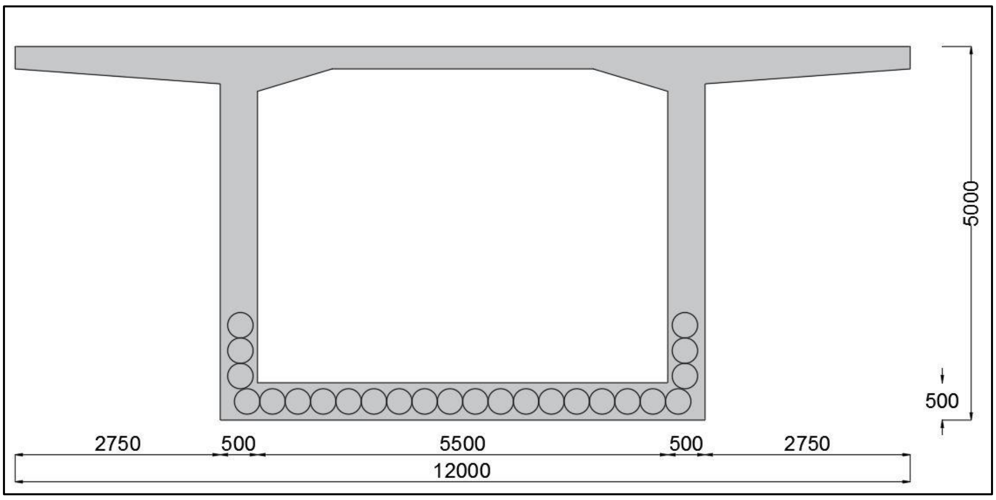

Figure 4.7: Cross section global design two.

Furthermore, the calculations These values are emphasized since it can be easily compared with the first alternative. The key-figures are a rough estimate and can be seen at the last page of the Maple file in Appendix C. The following values indicate the amount of materials which are required for constructing the deck structure with these single cell box girders.

- About 18 000 cubic meters of concrete
- About 480 cables which corresponds with 11 500 post-tensioned tendons of Ø15.7

However, the bottom side of the deck structure has open areas. This is prohibited due to probability of a collision. Therefore, these 20 box beams should be connected with some in-situ concrete for instance. This amount is not added in the amount above. Nevertheless, these values are a rough estimation if a design like this will be realized. Further research will be required to obtain more accurate numbers.

4.2.6 Execution method

The deck structure will be realized at construction site number one. In this paragraph it's briefly outlined in what way the 20 single cell box girders of 75 meters length and 12 meters wide could be produced. A prefabrication yard is going to be realized in order to accommodate protected circumstances during constructing of the element. The yard will have the same order of magnitude as with the box beams, estimated to be 85 meters long and 40 meters wide. This extra space is required to prefabricate the reinforcements, the supply of concrete trucks and other facilities.

In contrast to the production of the box beams of the first design, no stock might be created, because it's expected that the transport of one element requires one weekend. Therefore, the total yard will be smaller compared to the first design. After the production of one single cell box girder, the structure is ready for transport. It could be jacked with hydraulic presses to launch it on SPMTs. Otherwise the structure should have already been constructed upon these transport vehicles. But, further research will be required if such an execution method is applied.

In chapter 7 of the preliminary study the reference project "constructing a surpass at Ypenburg" is discussed. In this project a whole bridge structure is constructed at an adjacent building site. When the construction of the bridge structure was finished, it was driven to its position by SPMTs. The total weight of this deck was about 1900 tons. Transport of this deck structure was executed within one weekend break. In essence, the same procedure will be required with this design.

In Figure 4.8 the same figure is shown as visualized in Figure 4.4, only applied to the second design. Within the red striped area, a rectangle is visualized which schematizes the construction yard. Within this yard, one single cell box girder bridge is schematized at its entirety, while the other one is under construction. The location of this construction yard is positioned in such a way that the beams can be easily transported via the temporary road. When the structure is transported to the A27, it should be pressed/jacked to a certain height, because it can't be transported perpendicularly between the abutments of the U-shaped concrete structure. The deck structure should be transported at least 5 meters above local ground level to rise above the abutments. The first element should be positioned as far as possible, because no other element can be transported beneath an already placed element, because the height of the elements is 5 meters, while the clearance at the motorway is lower. The height of the SPMT should be added to these 5 meters as well. It won't fit.

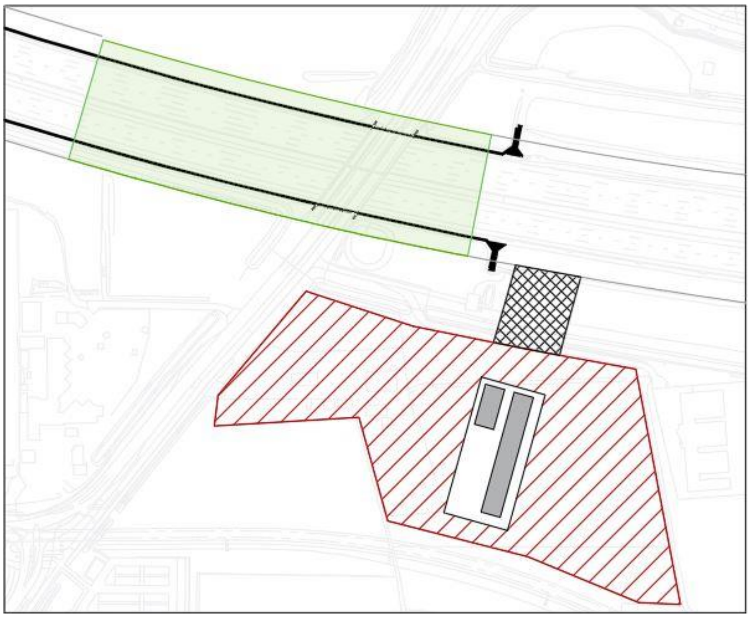

Figure 4.8: Schematic close-up of The Green Connection and construction site 1 for design two [2].

Furthermore, the same considerations are present as discussed in the end of paragraph 4.1.6. These considerations are related to the temporary access road from the construction site to the A27, since the risk on cracks is present in the membrane structure beneath which is situated beneath this temporary road. Also, the height difference is still present between the level on which the building site is constructed and the A27. Besides, the high pressure resulting from the weight of on top and the SPMT itself on the motorway. These topics need some attention, because it could result in unwanted events.

4.2.7 Conclusion preliminary design two

A design according to this design method seems to be feasible. It consists of the following key-points:

- Constructing a construction yard with sufficient space in front of the temporary access road.
- Constructing elements of 75 meters long and 12 meters wide 20 times.
- Constructing a temporary access road from the building site to the A27 crossing the membrane structure.
- In total 20 weekend closures will be required to realize the deck structure of The Green Connection.
- The elements are heavy bridge parts of about 2300 tons and can't be hoisted.

4.3 Discussion

Although both preliminary designs are already discussed in this chapter, this paragraph will briefly outline some differences between the designs in order to answer the next sub question:

Which of the two global designs will be the most suitable solution for constructing The Green Connection as a single span design?

The preliminary design of a custom-made box beam showed that a pre-tensioned beam seems to be feasible according to conventional approach. At first sight, this approach was used to prove that no feasible solution could be performed, and a reliable beam solution should be sought in a partly pre-tensioned beam in combination with an in-situ casted top layer. However, such a design procedure is not required, since a solution is found according to the conventional approach. This approach implies a commonly used method of just pre-tensioned tendons. This technique is well known and used for decades nowadays, in contrast to the method of the single cell box girder design. This kind of bridge structure is not common, especially not for such long span bridges. Normally, cross-sections like this are used as a balanced cantilever bride, and the post-tensioning is situated in the top flange instead of the bottom flange. Furthermore, the same calculations are used as for the pre-tensioned box beams which is questionable. Besides, the box beams are commonly used next to each other. It differs with the single cell box girder design. If the width increases, the cross section becomes a twin-cell box girder for instance. It will be just the question if this structure can be used next to each other.

Furthermore, if the parts of 75 to 12 meters are assembled next to each other, the bottom side of the deck structure is not a smooth surface. It implies that such a deck structure is susceptible to damages if collision forces are considered. It will result that the elements should connected to each other which results in extra weight and extra load. This is not the case by the ordinary box beams and is a big advantage.

When the method of execution is considered, some differences do occur. As stated before, this topic is a study in its own and is just briefly outlined. Realizing a prefabrication facility for pre-tensioned precasted beams is quite a disadvantage, since large pre-tensioning units should be realized. Also, tough gantry cranes are required. On the other hand, the second design implies a smaller prefabrication yard, but these elements cannot be hoisted at all. Their manoeuvrability is quite low compared to the box beams. This is mainly due to their dimensions and weight.

Moreover, it’s estimated that design one requires 8 weekend closures to assemble the deck structure, while the second design requires 20 weekend closures. This will be a rough estimation for both designs, but these values are based on reference projects and seems to be a reasonable estimate.

Nevertheless, the structure must be incorporated in the surrounded area. The structure of design one is 3 meters, while the second design has a height of 5 meters. It results that the first design can be better implemented in the surrounded area. This is quite important, since The Green Connection also functioned as a viaduct, and should be incorporated to the adjacent road alignment.

Concludingly, the custom-made box beam design is a less risky design at first sight compared to the second design. The technique of pre-tensioning is well known and commonly applied. It is expected that the difference in weekend closures will be a governing issue in determining which design seems to be most advantageous. To answer the sub question at the beginning of this paragraph, the custom-made box beam design seems to be most suitable solution in constructing The Green Connection as a single span design. Therefore, this design is investigated more thoroughly to prove the structural reliability. It is outlined in the next chapter.

5. Structural design check

This chapter covers a design check of the single span deck structure. In chapter 4 two global structural designs were performed which require more research as emphasised in the Preliminary Study. But, in chapter 4 only some primary calculations were performed. According to these calculations and aspects such as the method of execution, it could be deduced that the 75-meter span box beam seems to be the suitable solution for the deck structure. However, the primary calculations are not sufficient evidence. Therefore, the preliminary design discussed in paragraph 4.1 is investigated more thoroughly with extra calculations in order to answer the following sub research question:

What will be a sufficient structural design for the single span deck structure of The Green Connection?

This question is going to be answered with the help of this chapter. If it turns out that the outcome of the more newly performed calculations of the preliminary design will be sufficient, the above stated sub question is answered. If the result of this calculation is not satisfactory, it will be investigated what kind of adaptions/measures will be required to provide a sufficient structural design. But first, the starting point of the preliminary design of chapter 4 will be discussed.

5.1 Overview characteristics of global design one; Custom-made box beam

The following values are extracted from the Eurocode 2, NEN-EN 1992-1-1, or calculated:

Density	ρ	26 kN/m³
Characteristic compressive strength	f_{ck}	90 N/mm²
Design compressive strength	f_{cd}	60 N/mm²
Design tensile strength	f_{ctd}	$2\,{}^{1}/_{3}$ N/mm²
Characteristic tensile strength (fifth percentile)	$f_{ctk,0.05}$	3.5 N/mm²
Mean modulus of elasticity	E_{cm}	44 000 N/mm²

Table 5.1: Overview properties of concrete C90/105.

The results of chapter 4 is schematized in Figure 5.1. The values are summarized in Table 5.2.

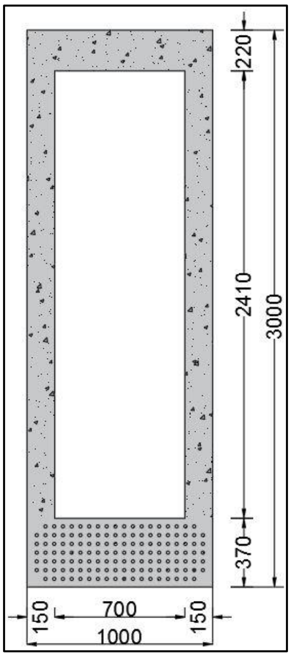

Figure 5.1: Global design custom-made box beam.

Height	h	3000 mm
Width	b	1000 mm
Thickness top flange	h_t	220 mm
Thickness bottom flange	h_b	370 mm
Web thickness	b_w	150 mm
Weight		280 tons
Number of pre-stressing tendons		136 [-]

Table 5.2: Overview properties global design.

5.2 Design approach

The preliminary calculations consist of 2 checks. One check will be a calculation in which the stresses in the structure should lie below the stress range when applying the pre-stress force and during loading phase. Via this way the pre-stress force could be determined. The other check consists of determining the maximum shear resistance. It implied that both checks were sufficient, as can be deduced from the calculation provided in Appendix A.

However, more calculations are required to demonstrate the structural reliability of the beam. These calculations will be provided next. First, one must determine which calculations should be performed. One important check is the bending moment resistance of the beam. The load on the beam results in an external bending moment which must be resisted by the internal moment equilibrium of the beam. This bending moment resistance should be checked in the serviceability limit state (SLS) for durability issues and the ultimate limit state (ULS) for structural reliability. The other important reliability check is the shear resistance of the beam which should be investigated more thoroughly at the supports, since the shear force is at its largest near the supports.

In order to provide the above-mentioned calculations, one should determine more properties of the beam than only the cross-section at mid-span. Therefore, these dimensions will be provided. Some remarks should be made regarding the design.

5.2.1 Remarks design

Due to the relatively large pre-stress force, tensile stresses could occur in the top fibre. This may not happen. The phenomenon is already checked for the cross-section at mid span, but not for the beam at the supports. The location at which this phenomenon is most likely to occur is at the supports, because the externally applied load doesn't result in a bending moment at the supports. It's important, because this bending moment results in opposite stresses at the outer fibre of the cross-section which can equalize the stresses due to the pre-stress force. This check implies that the average position of the tendons must lie in a core area of the cross-section. It means that the resultant eccentricity of the force cannot exceed $^1/_6$ of the beam height of the neutral axis at the top and bottom side for rectangular cross-sections [19]. Therefore, the resultant pre-stress force should lie within or at the boundary of the core area. But, several tendons are present. These tendons should be divided over the cross-section in order to achieve the average tendon position to lie in the core area. Since the tendons are applied symmetrically in width, the check doesn't have to be performed for this direction.

Furthermore, to gently initiate the pre-stress force, the beam should have a solid cross-section at the support instead of a hollow rectangular one. The length over which the solid part should continue will lie in the order of magnitude of the height of the beam. Therefore, this measure is taken as an initial starting point.

The cross-section at mid-span is already determined, but the cross-section at the supports isn't. Therefore, this should be done first, in order to perform the calculations about the average height of the tendons. This height of the tendon should lie within the core area.

5.2.2 Average tendon height in core area of the cross section

The core area can be calculated via 4 core points according to the manner specified in the previous paragraph. First step is to determine the neutral axis of the cross section. This central point is exactly in the middle of the cross section, since it's a solid symmetrical beam.

If the 136 tendons are positioned at the bottom flange, the average height of the tendon won't go through the core area. If the tendons are easily located within the core area, the capacity of the beam will diminish which may not happen. Two measures could be taken to let the average tendon height be situated in the core area at the supports without reducing the capacity at mid-span.

The first option is to apply a kinked tendon profile. The tendons are situated in the bottom flange at mid-span. But, after 10-15% to both sides from mid-span, this capacity isn't required anymore, because the height of the bending moment diminishes with this simply supported structure. From there on, a kink can be applied in the tendons which let the tendon run linearly to the top part of the cross-section at the support. One important aspect should be considered. Every kink means a pre-stress loss. This lost part of the force will work in vertical, in horizontal or in both directions, depending on the direction of the kink. The bigger the kink, the larger the pre-stress force loss will be. However, only the tendons which can applied under a kink are the ones near the outer parts below the webs, otherwise the tendons will be unbonded. Although the force is relatively low due to the small inclination, a summation of a substantial number of tendons will result in a considerable load. Therefore, the design should be performed cautiously.

The other measure implies that some tendons should be unbonded at the ends. This number of tendons must be calculated. However, if this measure is applied, it means that these pre-tensioned tendons are not used for its purpose. Therefore, this solution could be an option, but less efficiently.

5.2.3 The normal centre of the beam

In the preliminary design, the eccentricity of the pre-stress tendons was determined by a simplified calculation. This calculation entailed a homogeneous cross-section of concrete. Taken into account the difference in flange thickness, it resulted in the position of the neutral axis of 1596 mm towards the top fibre. The eccentricity became 1.219 meters (it can be deduced from the calculation in Appendix A). The capacity of the beam depends on this measure, because it's multiplied with the pre-stress force.

However, the calculation of the position of the neutral axis is performed again, but more accurate. In the second calculation, the difference in modulus of elasticity between concrete and pre-stressing steel is taken into account. It resulted in a higher distance between the neutral axis and the top fibre of the beam, and thus a lower eccentricity between the pre-stress force. In fact, the difference is only 62 mm. Due to this lower eccentricity, the calculation in Appendix A won't suffice anymore. However, if 2 extra tendons are applied, the pre-stress force is large enough again and the calculation will be sufficient. Therefore, these two tendons are added in the design.

Eventually, a combination of both measures is applied to realize the position of the average tendon height in the bottom core point of the core area. The 26 tendons situated below the webs are applied with a kink. From that point on it is calculated how many tendons must be unbonded at the ends near the support. It implies that 60 tendons should be unbonded for a certain length at the supports. The calculation is attached in Appendix F. This calculation holds for the solid part.

However, it must be verified for the hollow part too. At 3 meters from the support the hollow core initiates and should be checked. But, the bottom core point at the position where the hollow core initiates is situated lower than at the position of the solid part. In fact, this core point is calculated in the Excel sheet attached in Appendix

A. In the design, the value of the solid part of the beam is used, but in reality the bottom core point is situated lower. Therefore, the average tendon height lies in the core area and suffice the criterion.

5.3 The design to be checked

After determining the tendon profile, this preliminary design can be schematized with the next two figures. The first figure schematizes the longitudinal cross-section of the initial beam. Only one half of this beam is schematized, since it's a symmetrical beam. A rule of thumb is used for the position of the kink in the tendons. It implies that the measure of 10-15% of the span at both sides of mid-span will be sufficient. Therefore, it's chosen to locate the kink at 10 meters from both sides of mid-span.

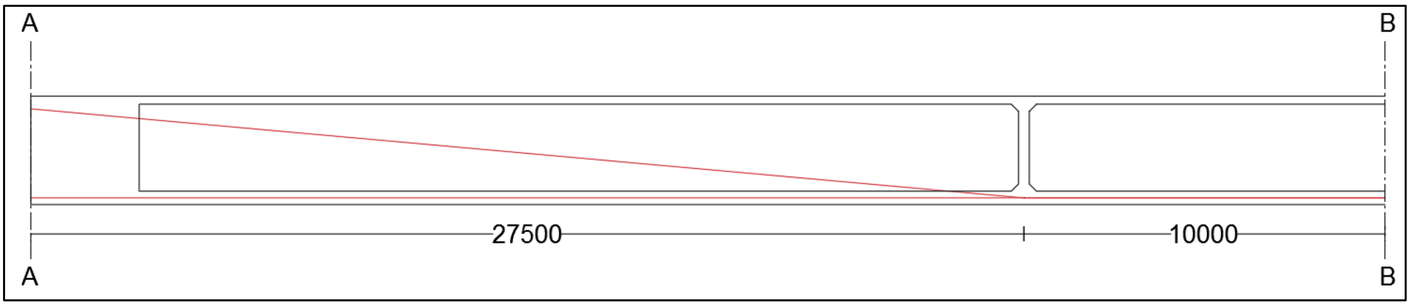

Figure 5.2: Longitudinal cross-section of the beam; tendons are sketched in red.

Furthermore, Figure 5.2 shows two indicated locations; A-A and B-B. At these locations, two cross-sections are drawn and visualized in Figure 5.3. The left cross-section A-A is at the supports. The core area is indicated and in the bottom flange only 52 tendons are hatched. It means that in total 86 tendons are not hatched compared to cross-section B-B at mid-span. 60 of them are unbonded. The other 26 tendons are situated in the top of the webs. The average tendon position is located just above the bottom core point of the core area conforming the calculation in Appendix F. It's indicated with a red dot. Some measurements are added as well.

The right part of the figure schematizes the cross-section at mid-span. It's important to note that the total number of 138 pre-tensioned tendons are situated in the bottom flange.

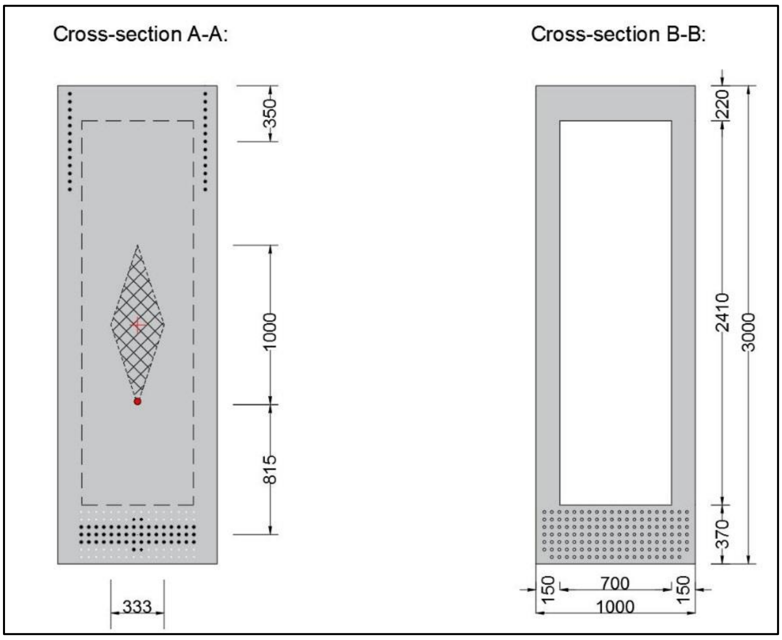

Figure 5.3: Two cross-sections; the left one at A-A and the right one at B-B of Figure 5.2.

5.4 Bending moment resistance

The bending moment resistance must be proven in order to design a reliable beam to span the 75 meters. Some words are mentioned about the bending moment resistance in serviceability limit state (SLS). The resistance of the beam in ultimate limit state (ULS) is substantiated with some calculations.

5.4.1 Serviceability limit state (SLS)

The serviceability limit state of a structure implies among others a verification of durability and comfort. Only some words about the bending moment resistance will be stated. Comfort (deformations) won't be checked. It lies out of the scope of this structural check. Durability of a structure implies that it will be a reliable structure for a long period of time. If relatively large cracks occur in the concrete structure, the quality of the reinforcement/pre-stress cables could deteriorate and the reliability is in danger. Cracks in the concrete occur when the stresses in the concrete exceed the tensile stresses. Therefore, verifying the bending moment of the beam in SLS implies a check of concrete tensile stresses. However, this beam is dimensioned with preliminary design calculations with the boundary condition that stresses in the outer fibre must be equal to zero or in compression. Therefore, no tensile stresses occur in the outer fibre and theoretically no cracks will occur too. It implies that the beam design is sufficient in SLS regarding cracks in the tensile zone.

5.4.2 Ultimate limit state (ULS)

The check in which the reliability/safety of the beam should be determined consists of a bending moment resistance calculation. This calculation goes in depth of the preliminary calculation. The calculation of the bending moment resistance is attached in Appendix G. The following paragraphs will explain and emphasize the approach of this calculation.

At first, some parameters are used as input. The effective height is the average distance of tendons to the top side of the beam at mid-span. Furthermore, the working pre-stress force is assumed to be 80% of the initial pre-stress force, which is a common value to take into account all the pre-stress losses. This value as well as the total surface area of the pre-stress steel was already determined in the preliminary calculation and is used here as well. Further properties of the concrete C90/105 are provided in Table 5.3.

Effective height	dp	2815 mm
Top flange height	h_fl	220 mm
Design compressive strength	f_cd	60 N/mm²
Elastic strain of concrete C90/105	ε_c3	0.0023[-]
E-modulus pre-stressing steel Y1860S7	Ep	195 000 N/mm²
E-modulus concrete bi-linear diagram	Ec=f_cd/ ε_c3	26 087 N/mm²
Working pre-stress force	Pm_inf	23 040 000 N
Total surface area pre-stress steel	Ap	20 700 mm²

Table 5.3: Input parameters calculation.

After the input of previous values and the determination of the concrete properties, the calculation started. At first, the strains were determined throughout the cross-section, because the internal forces can be determined with the help of these strains. But, these strains are unknown. Therefore, each strain component is written as one unknown strain component. The thought is to determine this one unknown strain via horizontal force equilibrium and after this known one, every strain and force can be calculated.

In order to stick to this approach, the strains which should be calculated must be determined. It depends on the cross-section. Since the design is a rectangular hollow core box beam, it becomes quite extensive. Normally, the outer strain of the flange in compression is used to determine the ratio between the other strains. The beam itself is 3 meters high. It implies a quite large concrete compression zone will be present. The top flange is only 220 mm. If this approach is used, a criterion isn't met. According to the NEN-EN 1992-1-1 chapter 6.1 sub (5), the average strain in flanges of box beams should be limited to the elastic limit, ε_c3. Therefore, it's chosen to use this value half way the flange and interpolate/extrapolate the other strains. The strains and the internal forces are visualized in Figure 5.4.

The strain diagram is a drawn as an ordinary strain diagram. The values however, are unknown yet. The height of the compression zone is drawn indicatively.

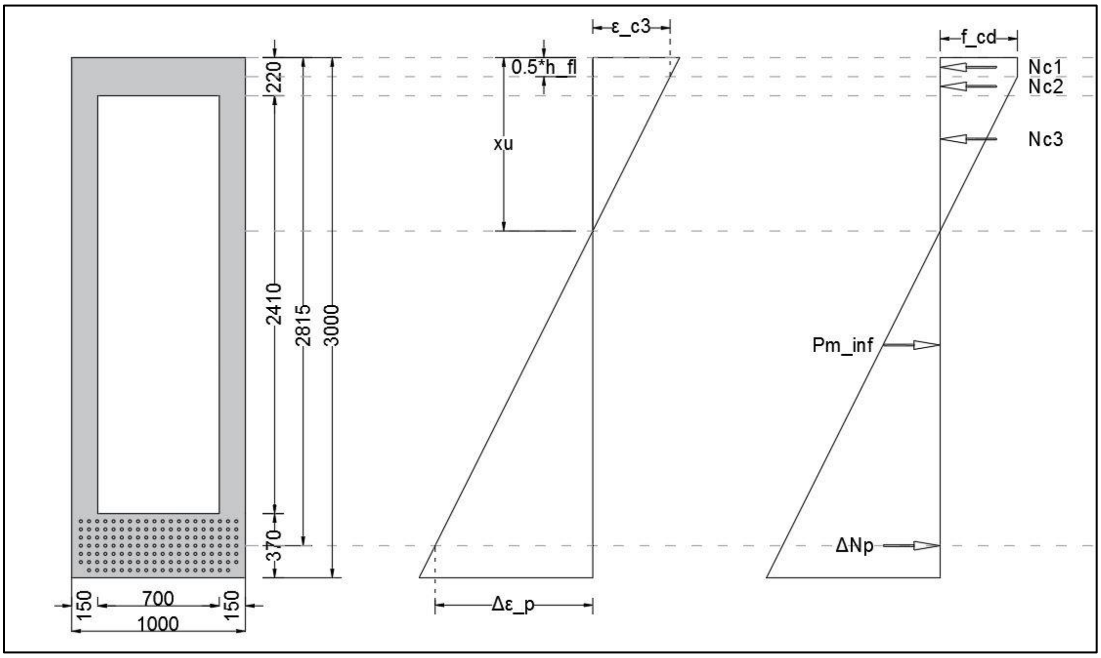

Figure 5.4: Strains and internal forces of the initial beam with an indicatively drawn xu.

As said before, the first step is to determine the strains and write them as one unknown.

$$\frac{\Delta\varepsilon_p}{d_p - x_u} = \frac{\varepsilon_{c3}}{x_u - 0.5 * h_{fl}} \tag{6}$$

$$\frac{\Delta\varepsilon_p}{d_p - x_u} = \frac{\varepsilon_{flange}}{x_u - h_{fl}} \tag{7}$$

$$\varepsilon_p = \Delta\varepsilon_p + \frac{\sigma_{p\_inf}}{Ep} \tag{8}$$

$$\Delta\sigma_p = 1522\frac{N}{mm^2} + \frac{\varepsilon_p - 0.0078}{0.0035 - 0.0078} * (1691\frac{N}{mm^2} - 1522\frac{N}{mm^2}) \tag{9}$$

With these 4 equations, the concrete compressive zone height x_u, the increase of pre-stressing steel stress $\Delta\sigma_p$, and the strain at the bottom side of the top flange ε_{flange} are written as one unknown, the $\Delta\varepsilon_p$. This is one important step, because the set of equations can be solved and every unknown can be expressed in a function of the one unknown $\Delta\varepsilon_p$. The answers are shown in Appendix G. First, the width of the beam and the web width must be used as input. Then, the forces can be determined according to the following formulae:

$$N_{c1} = b * f_{cd} * 0.5 * h_{fl} \tag{10}$$

$$N_{c2} = b * 0.5 * h_{fl} * \varepsilon_{flange} * E_c + 0.5 * b * 0.5 * h_{fl} * (f_{cd} - \varepsilon_{flange} * E_c) \tag{11}$$

$$N_{c3} = 2 * 0.5 * b_{web} * (x_u - h_{fl}) * \varepsilon_{flange} * E_c \tag{12}$$

These three forces above are cross-sectional concrete forces which acts in the compression zone. In fact, the only unknown in these formulae is still the $\Delta\varepsilon_p$. The concrete compressive forces should be in equilibrium with the pre-stress forces. In other words, horizontal force equilibrium holds and $\Delta\varepsilon_p$ can be determined. Therefore:

$$\sum F_H = 0: \qquad N_{c1} + N_{c2} + N_{c3} - P_{m\_inf} - \Delta N_p = 0 \tag{13}$$

Via this formula, $\Delta\varepsilon_p$ is determined. It implies that the other forces are determined as well, since these were a function of this $\Delta\varepsilon_p$. It resulted that the concrete compression zone height x_u become 2353 mm, which seems to be too large since the beam is 3 meters high. Therefore, the first check is to determine the rotational capacity of the beam instead of the bending moment capacity.

According to NEN-EN 1992-1-1+C2 chapter 5.5 sub (4) and the accompanying National Annex chapter 6.1 sub (9), the following criterion should be met for a beam in C90/105:

$$\frac{x_u}{d_p} \leq \frac{\varepsilon_{cu3} 10^6}{\varepsilon_{cu3} 10^6 + 7f} \tag{14}$$

$$f = \frac{\left(\frac{f_{pk}}{\gamma_s} - \sigma_{pm\infty}\right) A_p + f_{yd} A_s}{A_p + A_s} \tag{15}$$

It resulted in a concrete compression zone boundary of 1101 mm. This boundary condition isn't met by far, since the calculated one is 2353 mm. Therefore, this beam isn't a sufficient design and the bending moment capacity won't be checked anymore.

5.5 Reconsideration of the preliminary design

The previous paragraphs and calculations indicated that the initial preliminary design isn't sufficient. It lacks capacity in the concrete compression zone. The calculation proved that the concrete compression zone height is too large. Due to this large height, the rotational capacity check wasn't met by far. To overcome this problem, capacity should be added in the compression zone. A solution could be to increase the top flange with a certain thickness, which is going to be investigated in the coming paragraphs. But, increasing the top flange means and increase of the amount of concrete and thus increasing the self-weight. The initial boundary condition of a prefab beam should weigh less than 280 tons will be exceeded since the initial beam weighs already 280 tons. It will be the question in what order of magnitude the amount self-weight of the beam increases to achieve a reliable structural design. The same calculation procedure is considered as in the previous paragraphs. The first key criterion consists of finding a compression zone which lies below the maximum compression zone height criterion stated in the previous paragraphs. With the help of the same calculation sheet as attached in Appendix G, a new top flange height is found after an iterative calculation procedure. The flange thickness should become 540 mm instead of 220 mm initially. However, two important remarks should be made regarding this renewed design.

Remark one

The first remark relates to the method of calculation. As explained in paragraph 5.4.2, the strain diagram is based on ε_c3 halfway flange height. Via this way, the average concrete strain in the top flange won't exceed this strain which is normalized in the Eurocode the NEN-EN 1992-1-1 chapter 6.1 sub (5). However, due to the increase of the flange height by 320 mm and a compression zone height of about 1100 mm, the compressive strain in the outer fibre exceeds ε_{cu3}. This problem is schematically visualized in Figure 5.5.

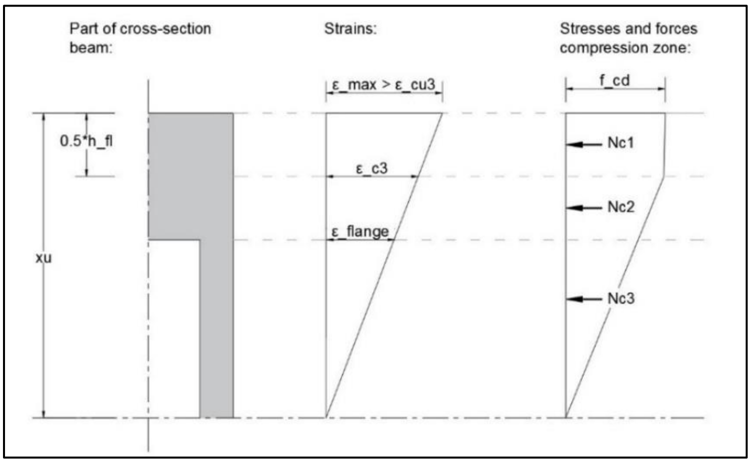

Figure 5.5: A part of the cross-section of the beam with accompanying strain and stress diagram.

Therefore, if the top flange is increased with 320 mm, the formulae in the calculation sheet should be adapted. The design approach will differ compared to the first calculation. Instead of the strain ε_{c3} halfway flange height, the ultimate limit strain ε_{cu3} is used at the outer fibre as a starting point. From there on, the strain ε_{c3} can be determined at a certain height. This 'certain height' is indicated as $h_{epsilon\_c3}$ and depends on several aspects, such as the compression height x_u and the flange thickness h_{fl}. It's important to locate the ε_{c3}, because from this point on the stresses will be equal to f_{cd}. To clarify this difference related to Figure 5.5, the next figure is drawn.

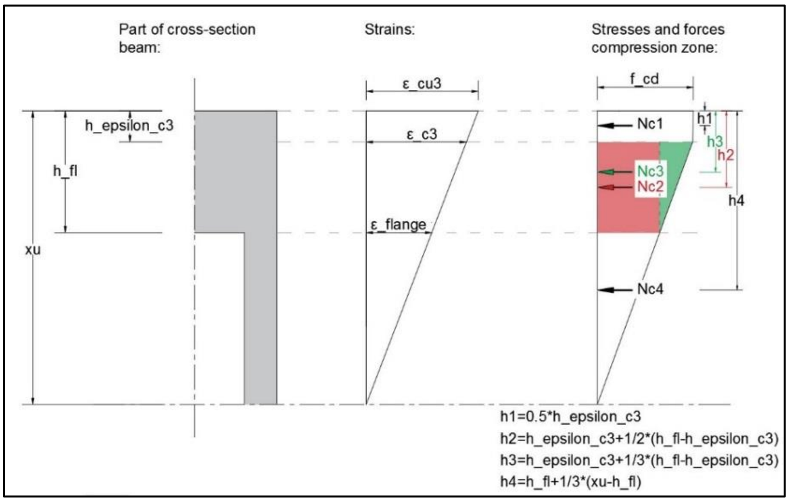

Figure 5.6: Correct strain diagram compared to Figure 5.5.

Figure 5.6 shows one extra unknown which is $h_{epsilon\_c3}$. The figure indicates the location of the strain ε_{c3} in the top flange. A function of this unknown related to $\Delta\varepsilon_p$ is added to the first set of equations, formulae (6) to (9). This function holds:

$$\frac{\Delta\varepsilon_p}{d_p - x_u} = \frac{\varepsilon_{c3}}{x_u - h_{epsilon\_c3}} \tag{16}$$

Via this function, the $h_{epsilon\_c3}$ can be written as a function of $\Delta\varepsilon_p$ as well. Thereafter, the compression forces should be determined. However, due to a difference in strain diagram, the formulae to determine the compression forces differ. In fact, the formulae don't hold anymore and should be determined again. Compared to the previous calculation, in total 4 concrete compression components are present instead of 3. These 4 forces have already been visualized in Figure 5.6.

$$N_{c1} = b * f_{cd} * h_{epsilon\_c3} \tag{17}$$

$$N_{c2} = b * \varepsilon_{flange} * E_c * (h_{fl} - h_{epsilon_{c3}}) \tag{18}$$

$$N_{c3} = 0.5 * b * \left(f_{cd} - \varepsilon_{flange} * E_c\right) * (h_{fl} - h_{epsilon_{c3}}) \tag{19}$$

$$N_{c4} = 2 * 0.5 * b_{web} * (x_u - h_{fl}) * \varepsilon_{flange} * E_c \tag{20}$$

These 4 forces above are cross-sectional concrete forces which acts in the compression zone. In fact, the only unknown in these formulae is still the $\Delta\varepsilon_p$. The concrete compressive forces should be in equilibrium with the pre-stress forces. In other words, horizontal force equilibrium holds and $\Delta\varepsilon_p$ can be determined. Therefore:

$$\sum F_H = 0: \qquad N_{c1} + N_{c2} + N_{c3} + N_{c4} - P_{m\_inf} - \Delta N_p = 0 \tag{21}$$

Via this horizontal equilibrium, the compression zone can be calculated and checked whether it's according the rotational capacity criterion explicated in (14). It is the same procedure as explained in the previous calculation.

After some iterations, a sufficient compression zone height is found. With this information, the forces in formula (21) are determined and the bending moment capacity of the beam can be calculated. The lever arm of each force should be calculated first before calculating the capacity. The sum is taken around the force ΔN_p, which is denoted with a red dot in Figure 5.7. The neutral axis is determined via a programmed Excel sheet (this sheet will be will be attached with the final calculation, Appendix H). The distances from the top fibre to the forces $N_{c1}$to N_{c4} is denoted with h_1 to h_4. A close-up of these distances is provided in Figure 5.6. In the same figure, these distances are outlined with the same notation as in the calculation. The bending moment capacity can be written as follows:

$$\sum T_{|\Delta Np} = 0 \qquad \begin{aligned} &N_{c1} * \left(d_p - h_1\right) + N_{c2} * \left(d_p - h_2\right) + N_{c3} * \left(d_p - h_3\right) + N_{c4} * \left(d_p - h_4\right) \\ &- P_{m\_inf} * \left(d_p - z_{top}\right) - M_{Rd} = 0 \end{aligned} \tag{22}$$

From this equation, the bending moment resistance M_{Rd} can be determined. This moment must be equal or higher than the design bending moment resulting from the load. The load consists of the permanent loads self-weight of the beam and the load of the park. Furthermore, a variable load resulting from a crowd load of the park is present. The loads resulting from the park is outlined in detail in the Preliminary Study Appendix B. The background information is presented here. It turns out that the external design bending moment can be calculated via (23):

$$M_{Ed} = 1.4 * \left(\frac{1}{8} * q_{Eg,beam} * l^2\right) + 1.4 * \left(\frac{1}{8} * q_{Eg,park} * l^2\right) + 0.8 * 1.5 * \left(\frac{1}{8} * q_{var} * l^2\right) \qquad (23)$$

To end, a unity check is provided. The design resistance of the beam should be at least equal or higher than the design bending moment resulting from the total external load. In other words, the unity check entails:

$$Unity\ check\ bending\ moment\text{:} \quad \frac{M_{Ed}}{M_{Rd}} \leq 1.00 \qquad (24)$$

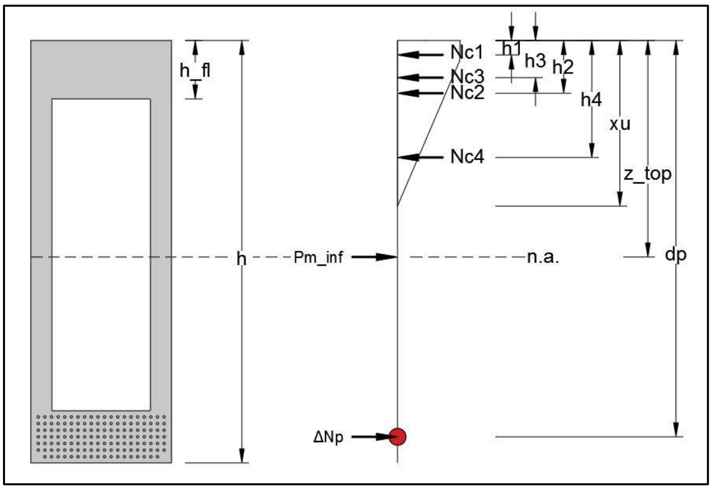

Figure 5.7: Bending moment sum around the position of the delta P force.

After an iterative process, a solution is found. This solution resulted in a top flange of 615 mm. These calculations are provided in Appendix H. However, the second important remark isn't considered yet in this calculation. It will be first explained.

Remark two

The second remark relates to the Eurocode, in particular to NEN-EN 1992-1-1+C2 chapter 6.1 and the accompanying National Annex chapter 6.1 sub (9). This section covers that it's allowed to apply a reduction to the amount of reinforcement or pre-stressing steel which is required to resist the predefined load. In Appendix H the unity check for bending moment is hatched in yellow and has the value of 0.799. It implies that the beam has more capacity than required. From this point of view, it implies that the total applied amount of surface area pre-stress steel is not required. The part that must be required can be deduced by reducing the pre-stress force and the amount of surface area until the unity check on bending moment capacity reaches 1.00.

Taken this "remark two" into account, the design calculated in Appendix H should be reconsidered. However, this design is not wrong, but less beneficial/optimal. In fact, exactly the same procedure can be applied to this calculation. The only difference will be a reduction to the pre-stress force and amount of surface area of the steel. After performing some iterations, the calculation of the bending moment capacity with this reduction is attached in Appendix I. It results in a design with a top flange thickness of only 380 mm, which is just 160 mm more than the preliminary design.

5.6 Shear resistance

The second structural design check consists of a shear resistance validation of the beam. First, one must determine at which position the shear force is at its largest. For a simply supported beam with distributed loads on top, this location is near the supports. At the supports, the first three meters of the beam consists of a solid beam. After these three meters, the rectangular hollow core initiates. This change in cross-section implies a significant drop of the concrete area and thus a drop of the shear resistance of only the concrete part. Therefore, two checks are performed. One shear check is performed at the solid part of the beam, while the other check is situated just in front of the solid part, in the rectangular hollow cross-section.

The calculations of both shear checks are performed and attached in Appendix J. In the coming paragraph, the procedure of the calculation is outlined for both checks.

5.6.1 Shear check at solid part beam

The first task is to determine the design shear force at the support. It is a summation of formula (25) in which γ_i is the load factor, q_i the distributed load and l the span. Since the structure is simply supported, it can be just calculated with the next formula:

$$V_{Ed} = \gamma_i * \frac{1}{2} * q_i * l \tag{25}$$

The background information of the load is discussed in more detail in the Preliminary Study in Appendix B and used in the calculation. The first check considers the solid part of the beam. The shear resistance can be calculated as follows:

$$V_{Rd} = \left(\vartheta_{min} + k_1 * \sigma_{cp}\right) * b * d_p \tag{26}$$

in which

$$\vartheta_{min} = 0.6 * \left(1 - \frac{f_{ck}}{250}\right);\ k_1 = 0.15;\ \sigma_{cp} = 0.2 * f_{cd} \tag{27}$$

The unity check of the shear resistance turns out to be in the order of magnitude of 0.3. Therefore, practical stirrup reinforcement will be sufficient in the solid part of the beam.

5.6.2 Shear check at hollow core of the beam

Next task is to determine the shear resistance of the rectangular hollow part of the beam. This part starts after 3 meters of the supports. Therefore, first step is to determine the shear force at this location. The gradient should be calculated. The maximum shear force determined in (25) is divided by the length of half the span, because at this point the shear force is zero. Multiplied with the distance at which the hollow core started, the actual shear force is calculated. In resulted in a unity check of 1.75, which is way too large.

Therefore, it implies that the concrete part doesn't have enough resistance by itself and a proper stirrup configuration is required. NEN-EN 1992-1-1+C2:2011 equation (6.1) states that if shear reinforcement is applied, the total force should be resisted by the stirrups itself. The concrete part cannot be added. The configuration is calculated next.

In the beginning of the calculation sheet, the internal lever arm z is determined. The distance from every N_{ci} of formulae (17) up to (20) to top fibre is calculated. Via this way the average distance from all N_{ci} can be calculated. Subtracting this value from d_p results in the internal lever arm z.

Furthermore, theta is chosen to be 45 degrees, which is a conservative value. The following formula holds for determining the amount of surface area required for stirrups:

$$V_{Rd,s} = \frac{A_{sw}}{s} * z * f_{yd} * \cot(\theta) \tag{28}$$

The A_{sw} is the total amount of surface area of the stirrups, s is the spacing of the stirrups in longitudinal direction, z is the internal lever arm which had already been calculated and f_{yd} is the yield strength of the stirrups. The total cross-section at three meters from support has two webs. Usually, one web has one stirrup which has two cross-sections. Therefore, at every distance s, 4 stirrup cross-sections are present. Furthermore, the $V_{Rd,s}$ should be at least equal or larger than V_{Ed} at the predetermined location.

After performing some iterations, it is found that if s is chosen to be 150 mm, stirrups of diameter 12 mm will be sufficient. This configuration can easily be applied to a web width of 150 mm. The stirrup configuration can be gradually diminished in longitudinal direction of the beam, because the shear force reduces until the mid-span position of the beam has been reached.

REMARK: *During the shear check calculations, the 3 meters of solid core of the beam at the supports isn't added to the total support reaction due to simplicity reasons. It just adds a marginal part of the total shear force. After determining the shear resistance of the beam, it turns out that the unity check is about 0.3 at this section. Due to this low unity check, the beam has enough overcapacity and this solid part can be neglected. The same holds for the shear check at three meters before beam end, just after the solid part. In that case, the solid part of the beam should be subtracted as well, so the solid part doesn't have any influence at all. In all other calculations such as the bending moment capacity and weight, this solid part is added.*

5.6.3 Tensile tie check

Besides the two shear checks, it is recommended to verify the strength of the tensile tie. Until now, the strength of the concrete compressive strut is calculated as well as the number of stirrups which should be applied. The strength of these two components had already been verified.

However, the shear force also results an increase of the stress in the pre-tensioned tendons. These tendons are already stressed to a certain height and function as the tensile tie. After a while, the stresses in the tendons reach $\sigma_{pm\infty}$. It must be proven that the level of pre-stress plus the increase of stress due to the shear force doesn't exceed the so-called yield stress of the pre-stressing tendons. This calculation is also attached and showed at the last page of Appendix J. Text in red is also added to clarify some values.

The tensile check is performed at 3 meters from the support. At beam end, in total 60 tendons are unbonded and are gradually initiated. It is assumed that 20 of these tendons are bonded at 3 meters and 40 are still unbonded. 26 tendons are applied with a kink in the top of the webs. Therefore, it means that 138-40-26=72 tendons are bonded in the bottom flange. The working pre-stress force after cutting the tendons and after a certain period of time, $\sigma_{pm\infty}$ plus the increase of tension should not exceed the limit stress. The calculation proves that the unity check is 0.729. Therefore, the tensile tie has sufficient strength.

REMARK: *The tensile tie is verified by exerting the total external shear force,* V_{Ed}*, at 3 meters from support. However, 26 tendons are applied with a kink. At the location of the kink of each tendon, a vertical component is present and pointing in upwards direction. These components may be subtracted from the shear force* V_{Ed}*. But, the tensile tie check does already suffice while these vertical components aren't taken into account. If these vertical components of the kinked tendons are subtracted from the* V_{Ed}*, the unity check of the tensile tie becomes even lower.*

5.7 Result calculations

Although some further structural checks should be performed and attention should be paid to detailing, one can state with more certainty that the next beam will be a sufficient design. An overview of the properties is summarized in Table 5.4

Height	h	3000 mm
Width	b	1000 mm
Thickness top flange	h_t	380 mm
Thickness bottom flange	h_b	370 mm
Web thickness	b_w	150 mm
Length solid part near support	l_{solid}	3000 mm
Weight		302 tons
Number of pre-stressing tendons		138 [-]
Stirrup spacing at webs	s	150 mm
Required amount of stirrup reinforcement	A_{sw}	436 mm²
Applied amount of stirrup reinforcement	$A_{sw,applied}$	452 mm²

Table 5.4: Overview properties of a sufficient design.

The results in the above stated table are visualized in Figure 5.8.

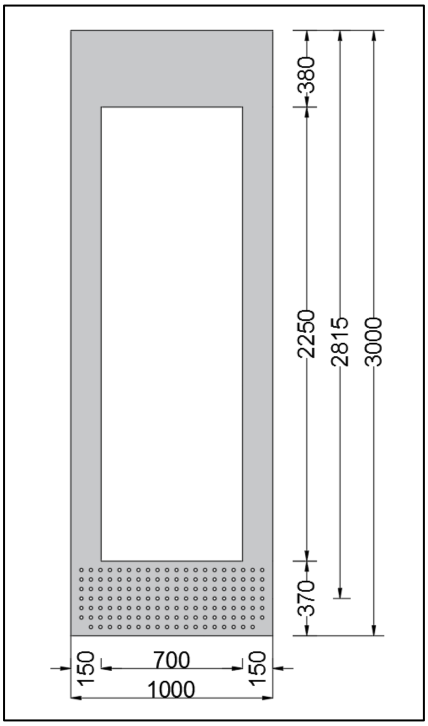

Figure 5.8: The results of the calculation visualized in a cross-section.

5.8 Optimizations

This chapter was intended to prove the structural reliability of the initial design. However, after some calculations, it turned out that this design is insufficient. The problem of the original design was a too large compression zone height which can be interpreted as a lack of the compression force. This force must establish the internal moment equilibrium. Therefore, capacity must be added in the compression zone and a new reliable design was considered in the same way as the original design. It was done by increasing the top flange inwardly. The outer dimensions of the beam were kept equal. Via this method, a reliable design was performed.

However, due to this increase of the thickness of the top flange in the prefab design, the beam weight exceeds the boundary conditions of 280 tons which was posed initially. In fact, the weight become 302 tons. Although it is just an increase of 7.8%, this value is larger than the boundary condition. Therefore, it might be interesting to emphasize optimizations in order to reduce the weight of a single beam. Since optimizing the beam is a study in itself, it won't be performed in this thesis. But, a list of (possible) optimizations is provided to prove weight reducing possibilities without diminishing the structural reliability. Each opportunity requires more research.

Increasing the height of the beam

Due to the lack of capacity in the compression zone of the preliminary design, it might be advantageous to increase the height of the prefab design. Via this method the internal lever arm increases, which might result in a lower pre-stress force and less pre-stressing tendons in the bottom flange. Less tendons could then result in thinner flanges.

The other possibility is to apply an in-situ top layer instead of increasing the prefab top flange inwardly. A layer on top will result in an increase of the initial internal lever arm, thus an increase of the bending moment capacity. It could result in less pre-stressing cables and a thinner bottom flange as well. Besides, the rectangular box beams are post tensioned in transverse direction if no in-situ layer is applied. However, if such a layer is applied sufficiently, no post tensioning has to be applied.

Besides, it could lead to a different design approach. Increasing the thickness of the flange with an in-situ casted topping partly corresponds with the preliminary design. A solution could be to lower the initial beam weight by reducing the prefab top flange. The design has just enough strength to carry its own weight. After placing the beams into position, an in-situ layer will be applied. This method will result in less self-weight during assembling and vanishes the required post-tensioning in transversal direction.

Web optimizations

The web width of this structural design is estimated to be 150 mm. It is based upon some reference projects and used throughout this report. However, it might be possible to minimize the width of the webs. Alternatives should be performed with single section stirrups instead of double section stirrups which is used in the design. Via this method it reduces the web width and the total weight of the beam.

Bundling/ repositioning of the tendons

The idea of both measures is reducing the bottom flange thickness. This design has 7 layers of tendons in the bottom flange. It might be a possibility to bundle the tendons in this flange. It results in more tendons in one horizontal tendon layer. Via this way it might be possible to reduce one horizontal layer of tendons in the bottom flange. The downside of this measure is that kinks occur which results in force components. These forces should be equalized with adequate reinforcements.

The other possibility is to apply some extra tendons in the bottom part of the webs. Via this manner the effective height does reduce slightly, but it might result in one horizontal layer of tendons less below the hollow core.

Apply compression reinforcement in top flange

It was found that the original design lacks capacity in the compression zone of the beam. Therefore, the flange thickness was increased by 160 mm inwardly to obtain a reliable structural design. However, instead of increasing the height of the beam with concrete, one could consider applying compression reinforcement. The surface area of the compression reinforcement can resist higher stresses resulting in higher compression forces compared to concrete C90/105. However, one must consider that the aim of the optimizations is to reduce the overall weight of one beam. If concrete C90/105 is compared to ordinary reinforcement steel, it can be concluded that steel is about 3 times as heavy as concrete (78.5 kN/m^3 vs. 26 kN/m^3), but the strength is about 7 times as high (f_{yd}=435 MPa vs. f_{cd}=60 MPa). Therefore, taken into account the self-weight, the surface area of compression steel has only an advantage of a factor 2.33 compared to the relatively high concrete strength C90/105. Nevertheless, it's still an advantage though.

5.9 Discussion

This chapter covers the structural reliability check of the preliminary design. The eccentricity of the average tendon height in the bottom flange at mid-span was determined by considering the flange thicknesses and homogeneous material. When the difference in modulus of elasticity of the pre-stressing steel and concrete was considered, it was found that the position of the neutral axis became lower. It does make sense, since the modulus of elasticity of the pre-stressing steel is almost 5 times as high as the modulus of concrete C90/105. Adding two more tendons vanishes this problem of the decreased eccentricity. Therefore, the design to be checked consists of two more tendons.

In total 26 tendons are applied with a kink. These tendons are situated below the web. In the web, sufficient space is available to apply even more tendons with a kink. However, if more tendons are applied, the tendons don't only make a kink in the vertical plane, but also in the horizontal plane. It means that the solution will be more extensive, due to the resultant components in vertical and horizontal direction. The amount of pre-stress losses increases with more kinks. Therefore, it is chosen to bend only the tendons which are situated below the webs.

Some remarks can be made by the method of calculating the bending moment capacity of the beam. The beam is dimensioned upon no tensile stresses in the outer fibre. Therefore, theoretically no cracks will occur. But, the comfort of the beam isn't verified yet. More research will be required to determine the deflection of the beam.

Furthermore, the first 'reliable design' had a top flange thickness of 540 mm. But, the strain in the outer fibre exceeded the maximum strain and therefore the design doesn't comply with a sufficient design. With an adequate strain diagram, it resulted in an increase of the top width. It does make sense, because the height of the strain area which was equal or higher than ε_{c3} decreases with the reconsidered design. Therefore, the compression force was lower and the compression zone should increase.

The shear force capacity of the beam is calculated somewhat conservative. In fact, the kinked tendons have a vertical component which may be subtracted from the total shear force resulting from the external load and the self-weight of the beam. Nevertheless, without applying this reduction (which is in fact an optimization), a reliable structural design is found with respect to shear capacity. But, if this reduction is taken into account, it has a positive effect on the amount of shear reinforcement. One can state that this beam design is still conservative with respect to the configuration of the stirrups. But, the aim of the chapter is to prove the feasibility of the beam and not optimizing a structural design.

Due to the design process is on the boundary of structural feasibility, this beam is optimized with respect to bending moment resistance, since the effective pre-stress force and surface area is taken into account (Remark two). But it doesn't mean that this beam is optimized in its entirety. In fact, a lot of optimizations are left.

To end, the sub research question to be answered after this chapter is:

What will be a sufficient structural design for the single span deck structure of The Green Connection?

The preliminary design outlined in chapter 4 and in the beginning of this chapter is not a sufficient structural design. The height of the construction zone is more than twice as large than maximum allowed, and therefore no further calculations were performed. However, with only increasing the top flange with 160 mm, a sufficient beam design is achieved. One should keep in mind that the initial boundary condition of 280 tons is exceeded, because the reconsidered beam design weighs 302 tons. It's just an increase of weight of less than 8%, but it has impact on several other aspects (method of assembly, crane capacity, transport, etcetera). However, this reconsidered beam design hasn't been optimized yet in its entirety. The gross dimensions are just kept equal as in the preliminary design, only the flange thickness is increased inwardly. In fact, several optimizations are present in the design. Furthermore, the self-weight of the concrete is taken as 26 kN/m^3 due to the relatively high amount of reinforcement which is a conservative measure for concrete. Concludingly, the structural reliability of a box beam is proved which can be used in constructing The Green Connection as a single span design. After performing an optimum design, the beams will individually weigh 280 tons or even less.

6. Original design Rijkswaterstaat vs. single span design

This chapter is meant to compare both designs and to indicate the biggest differences. After this chapter, the following sub question should be answered:

What are the main findings while comparing both designs?

One layout of the A27 in the original design of Rijkswaterstaat consists of 2 spans of 41 meters. The other road arrangement has a single span of almost 75 meters. First, the opted final result is discussed briefly, followed by discussing the three main aspects, the extended part, the intermediate support and the deck structure. Thereafter, some words are mentioned about the term 'availability'. The chapter ends with discussion which answers the sub research question.

6.1 Road arrangement

In the Preliminary Study, the current situation, the U-shaped concrete structure and the new intended situation opted by Rijkswaterstaat is discussed. These designs were outlined respectively in paragraph 1.2 and 2.1 of the previous report.

Furthermore, the two designs will be compared and discussed in the following paragraphs. For clarity reasons, both designs are visualized in Figure 6.1. The original design of Rijkswaterstaat is schematized in the top, while the single span design is shown at the bottom side of the figure. The dimensions of the structural parts are drawn indicatively.

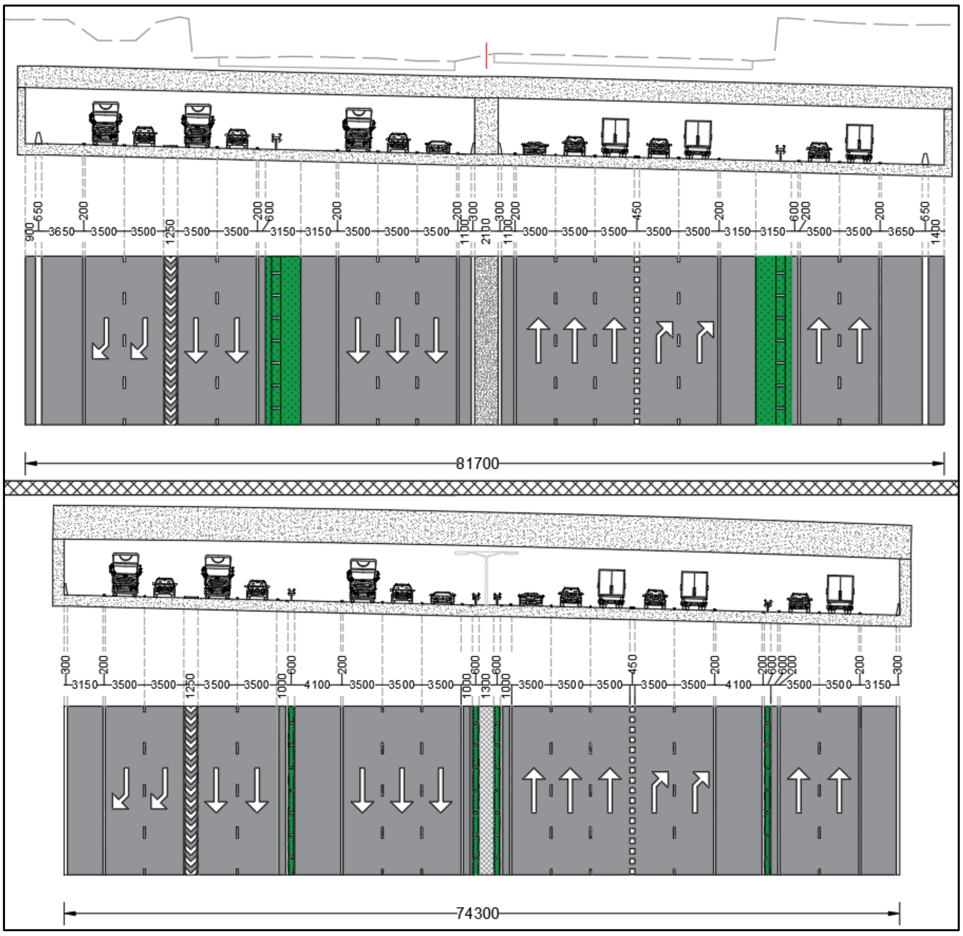

Figure 6.1: Top schematization; original design of Rijkswaterstaat. Bottom schematization; single span design.

The differences are directly visible between the two layouts. A key figure of the designs is the internal width which is measured between the side wall supports. This distance is opted to be 81.7 meters which is exactly 30 meters wider than the internal width of the existing structure nowadays. It implies an extension of the motorway of 15 meters at both sides. However, the total internal width of the single span design will be 74.3 meters. It means that the extended parts are 11.4 meters instead of 15. This reduction of 25% has some impact on the arrangement of the driving lanes at the motorway, because the number of driving lanes can't be reduced.

The U-shaped structure is only a small part of the motorway A27. When the layout of the motorway is realized conform 'the single span' design and the other parts of the A27 as the original design of Rijkswaterstaat, the road arrangement of both parts of the motorway won't connect smoothly. In that case the U-shaped structure will function as a kind of bottleneck within this motorway, because the internal width in the structure is smaller than the approaching parts of the motorway. This must be avoided, because it could cause hindrance to the motorists. Therefore, the layout within the U-shaped concrete structure should fit in the whole A27, and should correspond to the other parts of the motorway. When the layout of the A27 won´t be the same as the layout within the U-shaped structure, it should have at least a smooth transition to the U-shaped structure. Via this manner it could provide a smooth transition within the A27 and result in less hindrance to the motorists.

6.2 The extended parts

The existing structure must be extended at both sides to provide space for the extra driving lanes. As can be seen in Figure 6.1, both designs schematize an extending of the existing U-shaped structure. One difference is the quantity of widening at both sides. Paragraph 3.4 discusses a feasible manner of realizing the extended part. The method of execution is subdivided in 11 tasks to be completed and based on a widening of 15 meters. In that paragraph, the tasks are outlined and discussed afterwards. For more detailed information, it's referred to this section.

More important will be the difference between the two designs, the one of Rijkswaterstaat and the single span design. What effect does a reduction of four meters have at both sides compared to the original design? A narrower extending doesn't mean that some tasks can be left behind. But the amount of work at every task could diminish. First of all, the number of trees which has to be chopped down reduces substantially. According to a research study [20], more than 800 trees must be chopped off from the highly valued forest of Amelisweerd to provide enough space for the extra driving lanes in the U-shaped structure. If the width of the extended part is reduced with 25%, it will result in a reduction of trees to chopped off. Theoretically, it implies that more than 200 trees will be preserved, which will strengthen the social awareness of the single span design by the local residents.

No changes occur in the length of the sheet pile walls in longitudinal direction, because the length will be still the same. But, changes do occur with the sheet pile walls in cross direction. These walls are meant to realize the compartments, and will diminishes with 3.7 meters. Another large difference will occur in the amount of excavated dirt. 25% less cubic meters of soil have to be excavated and transported. This will save a lot of effort, time and money.

Further savings could be realized regarding the amount of material. The number of foundation piles used to counteract floating will reduce too. The amount of concrete to pour the underwater concrete floor will be reduced by 25% as well, and dewatering the building pit will be less time consuming, since the total amount of water is reduced. The same amount of savings related to the gravel layer and the structural floor will be realized as well.

Altogether, a lot of materials can be saved. This doesn't only cut the expenses of the materials itself, but also the transport, equipment and manpower. Nevertheless, using less material causes less transport movements which will also be a more sustainable solution.

However, one important aspect is the support wall. In the existing design, the primary function is to retain soil. In both new designs, it will still have this function, but the main function will be the support for the deck structure. If an intermediate support is used, the support reaction will be in the order of magnitude of 1700 kN/m, while support reaction in the single span design will be in the order of magnitude of 3500 kN/m. The difference implies more than a factor 2. It could require a different type of foundation for both types of design as well as the structural design of the support walls. What kind of foundation will be the question. It could be for instance a diaphragm wall or a kind of strip foundation on multiple piles. Another alternative could be a Combi-wall. Further research must prove which kind of foundation type will be most advantageous.

6.3 The intermediate support

If both the designs are compared, one can clearly see the biggest difference. The original design of Rijkswaterstaat has an intermediate support, while the single span design doesn't have such a support. The construction of the intermediate support is outlined extensively in paragraph 3.5 . In a nutshell, this paragraph entails:

> In the section is discussed that if no drainage will be applied, the structure can't be realized. Permanent drainage is prohibited by the authorities. The only possibility left, will be temporary drainage in short periods of time. Within such a period, a compartment must be created. The main aspects are removing the existing foundation, realizing a watertight substructure with vertical screens and horizontal injected layer, installing a large amount of piles, followed by constructing the strip foundation and the structural wall of at least 5 meters high. Such a compartment must be created about 13 times consecutively.

Besides the structural aspects, a lot of attention is paid to the execution aspects. Especially related to how to construct the intermediate support, what kind of equipment will be necessary and the logistic aspects. These aspects related to mobilizing all the materials, equipment and workmanship in the middle of the motorway. All these aspects and tasks are considered in relation to the availability of the A27.

If this only possible method of execution will be realized, it will be still a complex method with a lot of risks. Even though every task to be executed will go by as planned and this support is realized, it's still doubtful how this intermediate support will behave over time, especially after applying the deck structure. Imagine the following situation: Forces in upwards direction are present beneath the motorway resulting from the water pressure, and large vertical forces in opposite direction are present due to the load of the deck structure. It implies that the intermediate support will settle, while the other part of the structure at motorway will be either fixed or wants to move in the opposite direction. It will result in deformations of the intermediate support and high shear forces near the connection between the existing structure and the new compartment. This phenomenon is visualized in Figure 6.2. A part of the intermediate support is shown, and the red piles schematize the forces.

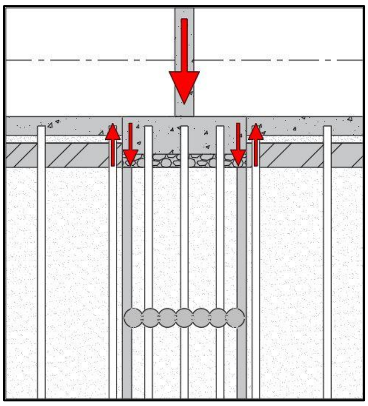

Figure 6.2: Shear forces between the connection of the existing structure and new one after loading.

Cracks could occur between the connection of the old and new foundation, resulting in water leakage in the middle of the motorway. The safety of the motorists is quite in danger if this amount of water leakage is large. Depending on the discharge, the A27 should probably be closed which would be catastrophic.

Paragraph 3.5.6 also provides a rough time estimation of constructing this intermediate support. It is stated that the construction time will lie in the order of magnitude of one and a half year. Again, it is a rough estimation, based on the construction time of one compartment. A better estimation requires more research. Therefore, more research is recommended about this topic in order to provide a detailed planning including a work schedule. But it doesn't vanish the fact that one and a half year a construction site will be situated in the middle of the motorway. In the same paragraph it is discussed that the estimated width of this site will be in the order of magnitude of 35 meters. It implies that one and a half year 35 meters of motorway can't be used for its purpose and just causes hindrance to the traffic at the A27.

The above-mentioned information and the information outlined in paragraph 3.5 concerns the intermediate support. As emphasized, constructing this support is a complex, risky and costly task. It's a major challenge to realize such a support which is a part of the original design of Rijkswaterstaat. The single span design doesn't have an intermediate support. It implies that all the tasks mentioned don't have to be executed which means major savings in relation to the original design of Rijkswaterstaat. Besides all the advantages in costs, effort and risks, it'll directly reduce the construction time with one and a half year, because the intermediate support should be realized in order to assemble the deck structure. It will be a major advantage compared to the original design of Rijkswaterstaat.

6.4 The deck structure

In both designs the U-shaped concrete structure should be covered up either with or without an intermediate support. The main difference is the number of spans. The design of Rijkswaterstaat has two spans of 41 meters and the other design has just one span of 74.3 meters. The deck structure will consist in both designs of box beams, but the dimensions of both beams will differ. The differences will be discussed from structural point of view and from the point of view of the method of execution.

6.4.1 Structural aspects

Until this point on, an estimation is only provided for the dimensions of the box beams for the double span. A lot of variations are possible, but as a starting point is stated that 300 beams of 160 tons have to be used which are 1.8 meters high and 1.66 meters wide. In contrast to the box beams for 41 meters span, a global structural design is designed for the box beams or the single span in chapter 4. This global design is a 3 meters high box beam and one meter wide. It weighs 280 tons and in total 249 of such beams must be realized. The differences are very obvious if the preliminary structural design of the beams is discussed. This is mainly due to the difference in span of both designs. Furthermore, the variables dimension, weight and number of beams are important issues if the execution aspects are considered.

The dimensions are already outlined. One of them is emphasized even more, because it's an important aesthetic aspect. The height of the beam is an important parameter when the deck structure is fitted in the surrounding area. Figure 6.3 visualizes both deck structures in the same layout. The figure is a close-up of the cross section of the motorway at the support wall, and is meant to visualize both the deck structures in the surrounding area. The scale on which the figure is drawn is in both schematizations the same. This visual shows the 1.2 meters of height difference. On top of this structure, almost a meter of soil will be situated in order to realize the public garden on top. This layer isn't visualized, because it's in both schematizations the same. A height difference of about 4 meters should be overcome to fit this park gently in the surrounding area. The public garden could be connected to the surrounding area with a kind of stairs which fit in the area. Another option could be a smooth hill for instance. Further esthetical aspects won't be considered anymore, since it isn't the purpose of this thesis.

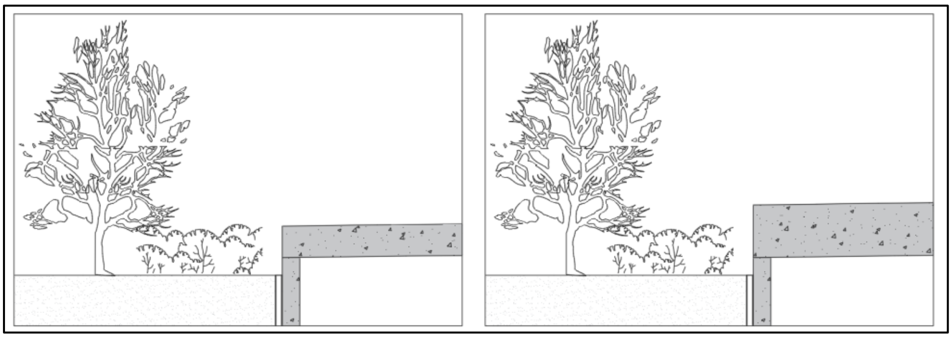

Figure 6.3: Difference in deck structure. Double span (left) and single span (right).

6.4.2 Execution aspects

In paragraph 3.6 some execution aspects are discussed about the original design of Rijkswaterstaat. The information can be found in this section. The dimensions regarding the deck structure itself are already emphasized in the previous paragraph.

One difference is the opportunity of the location of prefabricating the box beams. The prefabricated box beams for the double span design could be constructed elsewhere or at the construction site. However, the beams for the single span are too heavy to transport via the road infrastructure by a truck and trailer combination. The beams should be constructed on site at the temporary prefabrication facility discussed in chapter 4 and 5 of the Preliminary Study and in chapter 4 of this Master Thesis. The starting point will be still the same. The beams must be transported from construction site one to the location of The Green Connection. This situation is discussed in chapter 4 and visualized in Figure 4.4. It is expected that transportation could only be feasible with the help of SPMTs, since the weight is too large to transport the beams with convoy exceptional like truck and trailer.

A cross section of the starting point for the single span is schematized in Figure 6.4. In contrast to the starting point of the original design (Figure 3.28), the extended parts are less wide and no intermediate support is visible with the accompanying complex foundation.

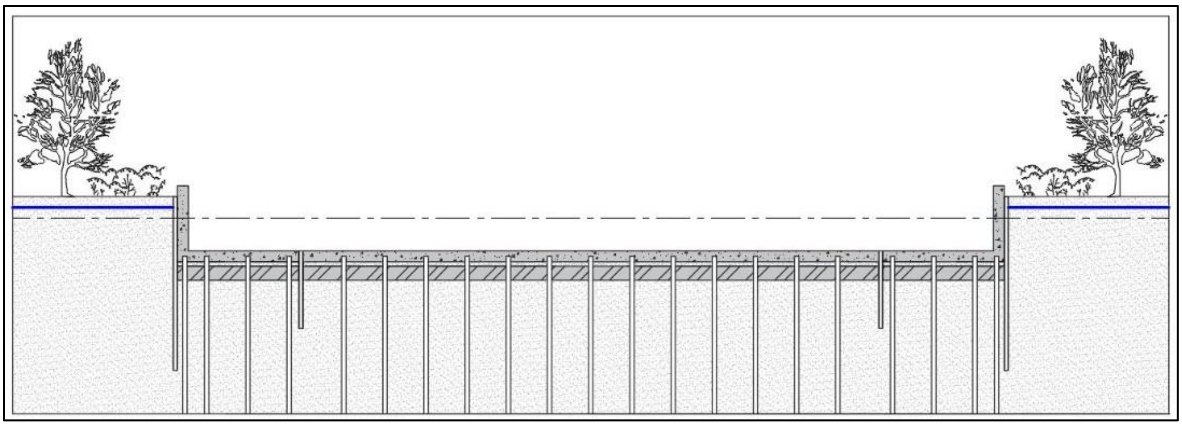

Figure 6.4: Starting point cross section single span.

The starting point for the single span design is determined. In paragraph 3.6 the alternatives are discussed in hoisting the deck structure for the original design. The same considerations are presented here, but for the single span, otherwise the execution method of both designs can't be compared. In contrast to the original design, no alternatives are compared regarding the single span design. However, the same subjects related to the execution aspects will be discussed here for the single span design.

The weight of these beams is estimated to be 280 tons. It implies that 140 tons should be lifted at each side of the beam. This number is substantially larger compared to the 80 tons of the original design. Therefore, the lever arm becomes even more important. If the same 700 tons crane is considered, the maximum distance over which 140 tons can be lifted will be 14 meters at its maximum [11]. Considering the beam length of 75 meters and 1 meter margin at both sides, the beams are delivered skew. This has some impact on the lever arm. Theoretically, it implies that the lever arm will be increased with $\sqrt{77^2 - 74.3^2} = 20$ meters. This distance is too large if the crane capacity is considered, but if the cranes are positioned ingeniously, this distance could be reduced. Another solution will be to use an extra crane with the same capacity. This third crane can support the other crane. Via this way the lever arm could be almost doubled. One should also consider the position of this third crane, so that it won't interfere with the transport of the beam. Besides, the use of a third crane was also done at the reference project "Construction at Zuidhorn", discussed in paragraph 7.1.2 of the Preliminary Study.

The next figure schematizes a plausible execution plan of hoisting the first beams into position. The length of the schematized beam is 75 meters and one meter wide transported by SPMTs. In total three cranes are schematized. One near the support at the side of the temporary transport road and two at the other side. This arrangement of the cranes could also be visa versa. Furthermore, a part of the perimeter of The Green Connection is drawn as a green line. A temporary road should be realized to provide access to the A27, which is hatched in the figure. A red pile is drawn indicatively in this temporary road to point the used path.

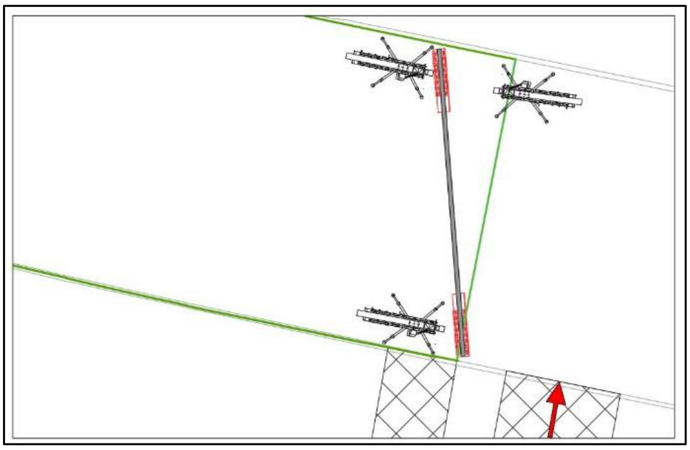

Figure 6.5: Layout of the cranes in hoisting the 280 tons beam

But, after a few beams are installed, the same problem arises as when the beams are hoisted into position conform the original design. The lever arm increases and becomes too large. The same considerations are present. If a third crane is used again, it will become quite extensive to hoist the beam, since one should manoeuvre along the pole of this crane. It will not be impossible, but quite extensive.

Transport of the beams via the deck structure might be possible as well. At first sight, the beams are dimensioned on a design load resulting from the public garden of 41.6 kN/m^2, as emphasized in the Preliminary Study. The weight of the 75 meters long beam is 280 tons, transported by two SPMTs. It results in 140 tons load per SPMT when two of these units are considered during transport. But the mechanical scheme differs compared to a distributed load. A point load of 140 tons and 10 tons for self-weight of the SPMT is used which results in a maximum moment at mid span. Both values lie in the same order of magnitude, but the point load resulting from self-weight doesn't include a safety factor. Therefore, this design moment from the self-weight of the beam will be higher than the design moment resulting from the park if the load is considered on one beam.

$$Design\ load\ resulting\ from\ the\ park: \frac{1}{8} * 41.6\ kN/m^2\ * 75^2\ m\ = 29250\ kNm/m$$

$$Representative\ load\ resulting\ from\ self\ weight\ beam: \frac{1}{4} * 1500\ kN\ * 75\ m\ = 28125\ kNm$$

However, the width of a SPMT unit is about 2.5 meters. It will mobilize at least three beams while driving across the deck structure. In fact, one beam doesn't have enough capacity, but three beams together will definitely have sufficient capacity to function as a reliable deck structure. It implies that transport of the same beam as the deck structure itself is possible via SPMTs. It might be sensibly to use a kind of substructure between the deck structure and the SPMTs. This auxiliary structure could assure the stability of the beams and redistribute the force from the SPMT gently to the deck structure.

When the beams are transported via the deck structure, only two cranes will be sufficient. It has some advantages compared to use three cranes. Besides the costs, also the way of hoisting will be easier. No crane pole should be avoided for instance. The next figure shows the two possibilities in hoisting the beams into position after a few beams had already been installed.

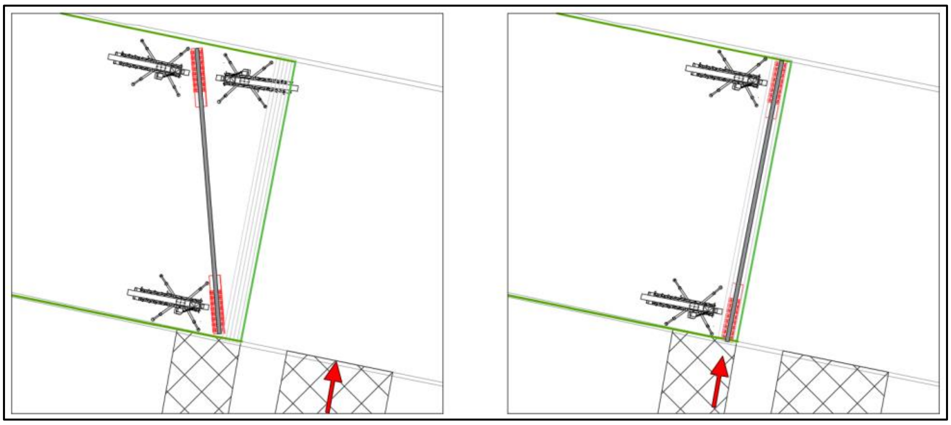

Figure 6.6: Two possible methods of assembly after placing the first few beams.

From this point on, it can be concluded that it is possible to hoist a 280 tons beam with mobile cranes. Although it will be tough cranes, it will still fit in the motorway without exceeding the maximum lever arm at the start of assembly. When the lever arm is exceeding this maximum, it is still possible to hoist the beam with extra auxiliary crane. After placing a few beams, the lever arm becomes too large. This problem can be overcome with the use of a third crane. Attention should be paid to the rotation of the crane around its axle, since large protruding counterweights are present. These weights may not interfere with the support walls or the deck structure.

The other possibility will be to make use of the deck structure. The beams can be transported one by one with SPMTs. In that case, two cranes will have sufficient capacity instead of three. When for instance 10 beams are placed, it might be possible to transport 2 beams simultaneously which is possible when multiple SPMTs are coupled. Attention should be paid to the large load which must be resisted by either the temporary road or the deck structure.

REMARK: *This method of assembly is meant to provide an estimation of the load, lever arm, type of crane and gross dimensions of the beam. However, more detailed information is required to indicate which method of assembly will be most advantageous. It's recommended to consult a crane specialist in order to determine the most optimal solution.*

6.5 Availability A27

When constructing The Green Connection, the availability of the A27 will be a key-issue during the whole execution phase. To what extend the construction activities will influence the availability of this motorway will be just the question. It is a difficult question which can't be answered yet. More research will be required with respect to mobility and traffic dynamics, which lay out of the scope of this Master Thesis. As said before, the term 'availability' is quite arbitrary. But, some preliminary findings can be provided in comparing both designs with respect this availability.

When the first execution phase 'constructing the extended parts' is discussed, some differences occur between the designs in hindrance to the A27. At first, the amount of work diminishes substantially. In total 25% of the extended parts can be reduced if the single span design is going to be realized. This 25% reduction doesn't imply that hindrance to the motorway will reduce with this same amount. But, when the extended parts are realized smaller than planned by Rijkswaterstaat, it does imply a major reduction of the amount of chopped off trees and excavated dirt. It also means a large reduction of construction materials to be supplied. All these components do mean a reduction of the number of transport movements, since everything is transported via the adjacent roads and motorways. More research will be required to provide detailed numbers with respect to time and costs. In the end, it will definitely result in less hindrance to the surrounding infrastructure.

The difference between the designs with respect to the intermediate support is quite obvious. The original design has a support, while the single span doesn't have one. When the single span design is considered, no tasks need to be executed after realizing the extended parts. It implies that no hindrance will occur too. One can directly start with the next phase, constructing the deck structure.

In contrast to the single span design, the original design of Rijkswaterstaat contains an intermediate support. This support requires a construction site of 35 meters of width which goes at the expense of available space of the A27. This building site is estimated to be situated within the motorway for about one and a half year. It implies that one and a half year the amount of driving lanes must be reduced. Nowadays, 10 driving lanes are present. After realizing the extended parts another 4 driving lanes are realized. But, the construction site requires 35 meters of width. The remaining space for driving lanes will be about 20 meters in each direction which is comparable to 4 driving lanes, a hard shoulder and some safety[5]. It implies that during this one and a half year of construction either 8 or 14 driving lanes could be used. In other words, it is stated here that constructing an intermediate support requires 6 driving lanes for one and a half year. It's up to the authorities whether this will be approved.

The hindrance resulting from the method of assembly in constructing the deck structure differs quite a lot between both the designs. Three alternatives are discussed for the original design. Independent of the alternative, when the first span is going to be covered with beams, the motorway doesn't have to be closed in its entirety. The remaining space depends on the alternative and differs between 25-35 meters during assembling of the beams. Only when the second span is going to be covered, the entire motorway might be closed in order to supply the first beams to the second span, crossing the adjacent driving lane. After these beams are placed, further transport of the beams could take place via the deck structure, as visualized in Figure 6.7. This figure schematizes that the first span is already covered with beams. In theory, vehicles can drive underneath this first part of The Green Connection while beams are transported across the just placed deck structure. Though, the authority must approve this method of assembly, otherwise a total closure of the motorway will be unavoidable. Another

[5] The 4 driving lanes, a hard shoulder and some safety will take about 20 meters. If a temporary road layout is considered, it might be possible to reduce the driving lane width. If that will happen, an extra driving lane could be achieved. However, this judgement is up to the authority.

possibility will be to transport the beams via the A27 crossing the other driving direction which will also result in a total closure.

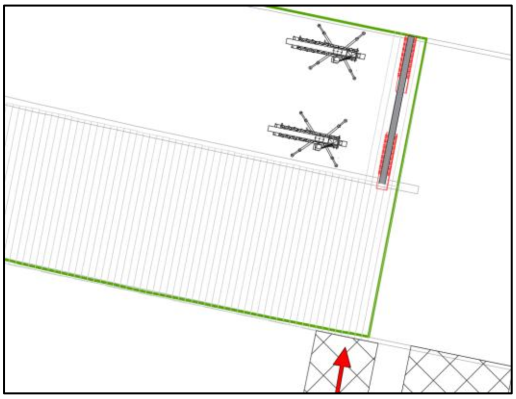

Figure 6.7: Method of assembly of the beams covering the second span.

When the single span design is discussed, multiple total closures of the entire motorway are inevitable. It doesn't matter if the transport of the beams takes place via the deck structure itself (after installing some beams) or via the A27. The amount of hindrance cannot be said easily, because it depends on what way the beams will be assembled. A possibility could be that the assembly of the beams takes place during weekend closures. As said before, at the reference project Bleizo 31 beams were assembled within one weekend closure. If this same measure is applied, in total 8 weekend closures are required. With the use of some extra equipment, and the fact that the task is repetitive, fewer closures of the motorway might be required.

Nevertheless, differences occur in the amount of hindrance to the A27 regarding the method of assembly of both designs. It can be stated that no total closures of the entire motorway are required in the original design. This judgement is highly dependent on the opinion of the authority. In contrast to the original design, the single span design requires a few total closures of the A27. In what way these closures will appear and how many of them are required will be just the question. A proper estimation seems to be the measure which is achieved at the reference project Bleizo.

6.6 Discussion

The difference between both designs is a reduction of the total width of 7.4 meters. The number of driving lanes is kept equal and the design is according to all safety considerations as outlined in the Preliminary Study.

The extended parts of the motorway could become 25% narrower compared to the original design. It is beneficial in almost every task during execution. Nevertheless, it's cutting the expenses substantially and the amount of hindrance will reduce as well. A point of attention will be the connection between the support wall and the floor due to large load differences. This problem does occur in both designs. Especially this topic needs further research.

During construction of the intermediate support, it's estimated that one and a half year will be required for realization. The building site will have a width which lies in the order of magnitude of 35 meters. Compared to the single span design, this whole task doesn't have to be executed, the building site don't have to be realized. It will again result in major savings in time, effort, hindrance and money.

The structural aspects of the deck structure differ in both designs, because the span is almost double as large. Besides, the moment distribution increases with an increasing span, since the span is quadratically in the formula. So, from structural point of view the differences do make sense.

A possible method of assembly is discussed for the single span design. The starting layout is visualized and the dimensions of the beams are mentioned. It implies that hoisting the 75-meter-long beams will be feasible with three large mobile cranes. After placing a few beams, transport via the deck structure is possible. A point of attention will be the temporary road across the membrane structure. This topic won't be considered in this master thesis and requires more research.

In both designs, the same tasks need to be executed in order to realize the extended parts. However, the difference between the designs lies in the quantity of broadening the motorway. 25% reduction could be achieved which result in less transport movements in the surrounding infrastructural network over a shorter period of time. What kind of impact these lower number of movements has will be the question. It does imply a substantial reduction.

If an intermediate support is going to be constructed, it'll result in a construction site of 35 meters width which goes directly at the expense of available driving space of the motorway. The new road layout consists of 14 driving lanes in total. Subtracting the distance of the construction site will result in a road arrangement of about 8 driving lanes. In other words, constructing this intermediate support result in a closure of 6 driving lanes over a period of one and a half year.

A disadvantage of the method of assembly for the single span design will be the necessity to close the A27 temporarily during hoisting. Cranes are situated at both sides of the motorway which is required to lift the beams. These beams will cover the whole motorway during hoisting. In contrast to the double span design, the A27 doesn't have to be closed in total. About 30 meters of width of the motorway will be left. It implies about 3 driving lanes in both direction and a hard shoulder. So, reducing the amount of driving lanes from 10 originally to 6 will definitely result in hindrance such as traffic jams. Besides, it's still depending on the judgement of the authority if traffic is approved during the assembly phase, otherwise a total closure will be still inevitable.

Concludingly, this chapter emphasizes the differences between the two designs, and indirectly answers the sub research question:

What are the main findings while comparing both designs?

A reliable motorway layout can be realized for the single span design cutting the internal width by 7.4 meters, and still corresponds to the safety regulations. Furthermore, it can be stated that the single span design is advantageous in constructing the extended parts, since these parts can be constructed less wide. No intermediate support must be realized which has major advantages as well. However, disadvantages are present when the deck structure of the single span is considered. The structural design of beams of this kind is never constructed before. It's designing on the boundary of feasibility from structural point of view. The unique deck structure of custom-made prefabricated pre-tensioned box beams will be more expensive compared to the deck structure of the original design. But, the deck structure of the original design will be unique too. In contrast to the single span beams, the 42-meter long beams could be constructed in a factory elsewhere, although it's highly favourable to construct it locally. In essence, the question arises whether the extra money of constructing a single span deck structure outweigh the money which can be saved by leaving out the intermediate support and constructing the extended parts 25% narrower.

7. Discussion

Throughout this thesis, some limitation, assumptions and simplifications have been made. Therefore, it is useful to discuss certain items. In some cases, decisions had to be made in order to continue to research, otherwise a specific topic couldn't be outlined. Several discussions have already been written at the end of the previous chapters. This chapter covers the main items and remaining considerations.

The construction of the extended parts is divided in eleven main tasks. These tasks seem to be relatively straightforward to execute. However, if these tasks are outlined in more detail, some issues arise which require more attention. With proper detailing and engineering, it won't result in feasibility problems. Further research will be required to optimize the method of execution, which lies outside the scope of this thesis.

Constructing the intermediate support implies realizing a new foundation within the existing U-shaped concrete structure. A key-criterion is the drainage system. In this thesis, it is stated that it's impossible to construct the opted foundation without applying any kind of drainage. Since applying a permanent drainage system is prohibited, the only possibility left is to implement a partly applied drainage system. Upon this fact, the only possible method of construction is explained which include constructing compartments 13 times. However, the method of construction is highly dependent on what amount of drainage is allowed. On the other hand, if permanent drainage is prohibited, does it make sense that a temporary applied drainage system will be approved? It is still draining ground water, only on smaller scale. However, if also temporary drainage will be prohibited, it implies that the intermediate support can't be constructed. And, if this support can't be realized, the whole design can't be completed. One must search for another alternative. This alternative will be not just an alternative, but the solution since constructing an intermediate support will be infeasible. Therefore, the applied drainage system is a key-issue in realizing the intermediate support.

The design considerations in assembling the deck structure are indicated for three obvious methods of execution. With the help of some key-words, the variants are discussed. In fact, every alternative is judged by the impact on the availability of the A27. It's important to indicate that with some extra investments, the remaining space for traffic could be maximized during the assembly of the deck structure. It will be the consideration by the authority Rijkswaterstaat what kind of financial compensations it offers to reduce the hindrance at the A27 during construction. The question rises if this organization is willing to pay this money.

Besides the execution aspects, two preliminary designs are discussed in more detail. In fact, only two alternatives are dealt and indicated as possible solutions. Just considering two alternatives as a plausible solution for constructing the deck structure seems to be too limited. For an improved synthesis, more alternatives should be considered. The most optimal solution could be determined via a kind of trade-off matrix. However, it is chosen to elaborate on only these two most obvious designs, otherwise it will pass the objective of this thesis.

A reliable structural design is performed for a 75-meter span beam which can be used as a deck structure in the single span design. It does exceed the boundary condition of 280 tons which was posed initially with 8%. However, no optimizations have been applied to this design. If the enumerated optimizations are performed, a beam can be designed according the boundary condition and possibly even less.

Furthermore, constructing the intermediate support implies many construction risks. Some of these risks are indicated as 'high' and are situated in the red area of the risk matrix. It implies that if some task doesn't go by as planned, the event could result in major financial consequences and/or a complete closure of the A27. In worst case scenario, it could also result in casualties.

Variations are present in the method of assembly. If the deck structure of the single span design is considered, multiple total closures of the A27 are inevitable. The duration of these closures depends on the number of beams which can be hoisted during one shift. Assembling the deck structure of the original design doesn't imply a total closure of the motorway. At least one driving direction should be blocked since the cranes are situated there. Depending on the judgement of the authority, the second one should be partly closed as well. It will result in remaining space of about 25 to 35 meters, depending on the alternative. On the other hand, reducing the available space at the motorway from 80 to 25 meters will definitely result in large traffic hindrance. It might be the consideration to close the motorway to avoid large traffic jams.

Nevertheless, discussing the execution aspects of each topic individually isn't a fair comparison. The total project should be considered. In other words, not only the extended parts, or the intermediate support, or the deck structure should be considered, but all execution aspects in realizing The Green Connection as a whole. Only in that case a proper decision can be made regarding the designs.

Another important issue is related to the forest of Amelisweerd. Since constructing The Green Connection goes at the expense of this forest, the social awareness of the master plan "A27/A12 Ring Utrecht" is at a high level. Nowadays, a few demonstrations by local residents have taken place to omit the construction activities at the forest of Amelisweerd. This is mainly due to the extending of the A27, which goes at the expense of this greatly appreciated forest adjacent to the structure. Considering this fact, the single span design reduces the width of the extended parts with 25%. It implies that more than 200 trees could be preserved which will increase the social awareness of the single span design.

To end the discussion, this research only considers the method of construction and the structural aspects of The Green Connection, but this project is still a part of the total masterplan. This masterplan is way larger than only this project, and therefore the surrounding subprojects should be considered as well. Project managing of the masterplan "A27/A12 Ring Utrecht" is an essential subject. In other words, the described aspects throughout this thesis should be implemented to the construction tasks of the surrounding projects. Consider for instance the two railway viaducts situated in front of the U-shaped concrete structure. If the road arrangement changes to the single span lay-out, it should also change near the viaducts since these are situated only a few hundred meters up ahead the motorway. It might imply that even these bridges should span the motorway without the use of an intermediate support too. The question rises whether it will be a viable solution for these railway viaducts. Therefore, realizing The Green Connection as a successful project doesn't automatically mean that the masterplan is successful. One should consider this plan as a whole, and each sub-project must be implemented appropriately.

8. Conclusion

This chapter will provide an overview of the research questions throughout the thesis. At the end of this chapter, a final conclusion will be outlined including a recommendation.

8.1 Sub questions

In total 6 sub research questions have been enumerated in the introduction. These questions will be answered respectively in order to substantiate the answer to the main research question.

What could be an obvious way of constructing the extended parts?

Constructing the extended parts is explicated according 11 main tasks. Each task is divided in other sub tasks, risks and considerations. Nevertheless, the first main task starts with chopping off the existing trees, followed by installing sheet pile walls and applying anchors and/or struts and wales. After that, one should consider in what way the piles are going to be installed, before or after excavating the building pit. Eventually, the building pit is excavated and the foundation piles are applied. Then, one can start pouring the underwater concrete floor. After draining the building pit, the layer of sand and a structural concrete floor can be realized. Thereafter, the side support wall is going to be constructed. When this wall reaches a certain level, it is safe to remove the struts and wales/anchors. The construction of the wall can be finished and the existing old concrete wall can be removed. After all, executing these 11 consecutive tasks shows an obvious way of constructing the extended parts.

How to construct an intermediate support?

The design load of 3200 kN/m resulting from The Green Connection must be resisted by the support. It is found that the existing foundation lacks bearing capacity by far. A new strengthened strip foundation with extra foundation piles must be realised in the middle of the motorway to resist this large load. Due to the boundary conditions (such as the water pressure beneath the structure and permanent drainage is prohibited), the only possibility left is to construct small building pits, compartments, within the existing structure. Such a compartment has a rough length of about 20 meters and will be about 6.5 meters wide. The sequence of tasks to be executed starts first with appointing the dimensions of the compartment and installing/applying the temporary drainage system.

Then, the top layers of the existing foundation can be removed. After assuring the water head to the appreciated level, the underwater concrete floor can be removed. Thereafter, the vertical grouted screens should be applied around the perimeter of the compartment followed by injecting the horizontal membrane layer. After hardening, the compartment is realized within the existing structure. The drainage can be stopped and pile driving will follow to realize the required bearing capacity. After installing the piles, the reinforcement of the strip foundation can be placed and the concrete can be poured. The last phase consists of constructing the intermediate support wall itself.

One should keep in mind that the tasks to be executed are an accumulation of risky, expensive, complex and time-consuming tasks with a high impact on the availability of the A27. It seems to be possible to construct such a support, but only when some basic assumptions are made, such as an approval of temporary drainage. If some of these starting points will differ, it could lead to infeasibility of realizing an intermediate support.

How to realize the deck structure in an obvious way?

The deck structure will consist of 42-meter span beams which can be transported individually via the motorway. Furthermore, three alternative methods of assembly are discussed and judged to be feasible. At both support sides, a crane will be required to hoist the beams into position. These cranes are at least the so-called 500-tons canes. One interesting finding implies that after placing some beams, this structure has enough strength to allow transport of these beams via the deck structure. It is advantageous to transport the beams via the deck structure, because it reduces the lever arm substantially.

Which of the two global designs will be the most suitable solution for constructing The Green Connection as a single span design?

The preliminary design of a custom-made box beam showed that a pre-tensioned beam seems to be a feasible design according to conventional approach. At first sight, this approach was used to prove that no feasible solution could be performed. A reliable beam solution should be sought in a partly pre-tensioned beam in combination with an in-situ casted top layer and applying post-tensioning in longitudinal direction. However, such a design procedure is not required, since a solution is found according to the conventional approach. Concludingly, to answer the sub question, the custom-made box beam design seems to be the most suitable solution in constructing The Green Connection as a single span design.

What will be a sufficient structural design for the single span deck structure of The Green Connection?

After performing some more thorough calculations, it turned out that the preliminary design couldn't be a reliable deck structure. The height of the compression zone is more than a factor 2 larger than maximum allowed, and therefore no further calculations were performed. However, with only increasing the top flange with 160 mm inwardly, a sufficient prefab beam design is achieved. One should keep in mind that the initial boundary condition of 280 tons is exceeded. The reconsidered beam weighs 302 tons. It's just an increase of weight of 8%, but has impact on several other aspects (method of assembly, crane capacity, transport, etcetera). But, this reconsidered beam design hasn't been optimized yet. The gross dimensions are just kept equal as in the preliminary design, only the thickness of the top flange is increased. Concludingly, if the top flange of preliminary design is increased with only 160 mm inwardly, the beam will be a reliable structural design for the single span deck structure.

What are the main findings while comparing both designs?

A reliable motorway layout can be realized for the single span design cutting the internal width by 7.4 meters and still corresponds to all safety regulations. It means that the extended parts can be constructed 25% narrower compared to the original design. It is beneficial in almost every task during execution. Nevertheless, it's cutting the expenses substantially and the amount of hindrance will reduce too.

No intermediate support must be realized at the single span design, which has several advantages too. It results in major savings in constructing a new foundation and a support wall for over 250 meters. It is also enormously beneficial if the hindrance to the motorway is considered. If such a support should be constructed as in the original design, a large building site should be realized in the motorway. Besides the required space of this site, this whole task 'constructing the intermediate support' is extensive, risky, costly and causes a lot of hindrance.

However, disadvantages are present when the deck structure of the single span is considered. The structural design of beams of this kind has never been constructed before. The single span beams must be constructed at the temporary construction yard because the beams are too heavy for ordinary transport via motorways. The unique deck will be more expensive compared to the deck structure of the original design.

In essence, the question arises whether the extra money of constructing a single span deck structure outweigh the money which can be saved by leaving out the intermediate support and constructing the extended parts 25% narrower.

8.2 Research question

After answering the sub research questions, a better insight is acquired about constructing The Green Connection. Throughout the report, each subject is discussed in more detail. The following main research question will be answered in this paragraph:

> *How to construct The Green Connection as a concrete structure with and without the use of an intermediate support?*

Constructing The Green Connection with an intermediate support can be realized by first extending the motorway at both sides with 15 meters according the described 11 phases. After that, the intermediate support should be realized in 13 compartments. Each compartment consists of roughly 4 construction phases. The last task implies realizing the deck structure. Beams should be individually hoisted to realize the deck structure, connecting the forest of Amelisweerd to the city of Utrecht.

Constructing The Green Connection as a single span design also starts with realizing the extending parts of the motorway, only 25% smaller at both sides. The same construction method will be applied as in the original design. In contrast to this design, no intermediate support should be realized. After finishing the extending parts, one can directly start assembling the single span deck structure. These beams should be individually hoisted too in order to realize the deck structure.

8.3 Final conclusion

Rijkswaterstaat has performed a design to realize The Green Connection with an intermediate support. In this thesis, the feasibility of this design is investigated, and in particular this support wall. The only possible method of execution in realizing the support is upon condition that a temporary applied drainage system will be feasible, and will be approved by the authority. When the authority states that draining ground water is prohibited, the original design isn't feasible anymore. In that case, the single span design isn't just an alternative to the original design, but is the only feasible solution.

Concludingly, providing the applied principles of this thesis, it is strongly recommended against constructing an intermediate support within the existing U-shaped concrete structure. Since the structural reliability of a 75-meter span beam is proven, the intermediate support is redundant. Therefore, the single span design is less risky, less time consuming and less expensive compared to the original design.

9. Recommendations

The main recommendations for further research are presented below.

9.1 Contract and statement of requirements

During this research, some assumptions are made. Until now, the contract of the project isn't published yet. One doesn't know which degrees of freedom are present. Therefore, the assumptions should be considered in more detail. One should establish the limitations in the contract and consider the impact of the assumptions made throughout this thesis. In other words, the statement of requirements should be verified.

9.2 Optimizing the single span box beam

In this thesis, the technical feasibility of the single span box beam is proved and substantiated by calculations. However, this beam isn't optimized yet. In paragraph 5.8 the optimizations are enumerated. Besides optimizing the dimensions to reduce the self-weight, one could also apply an in-situ casted concrete topping. If for instance the initial prefab design is dimensioned upon their self-weight, the beams become less heavy than initially. Then, after placing, a concrete topping is added on top. It will increase the capacity and no post-tensioning in transverse direction should have to be applied. In essence, the technical feasibility is proved, but there's still room for optimizations.

9.3 The alternative with an intermediate support outside the U-shaped structure

Even if the authority considers the single span design as too risky, another interesting alternative is present. In this case, a support is constructed exactly next to the existing side wall. This layout can only be feasible when the support wall is situated near the forest of Amelisweerd. If the support is going to be constructed at the other side of the motorway, the position of the support will be located in the middle of a driving lane which isn't possible. Besides the structural advantage of a reduction of about 12 meters in span, the biggest advantage is to omit constructing the intermediate support within the U-shaped concrete structure. The situation is visualized in Figure 9.1. In this figure, a dashed grey line is drawn in the top of this figure to visualize the current structure. The small vertical red line is the middle axis of the motorway. The 63-meter span beams can be optimized as well.

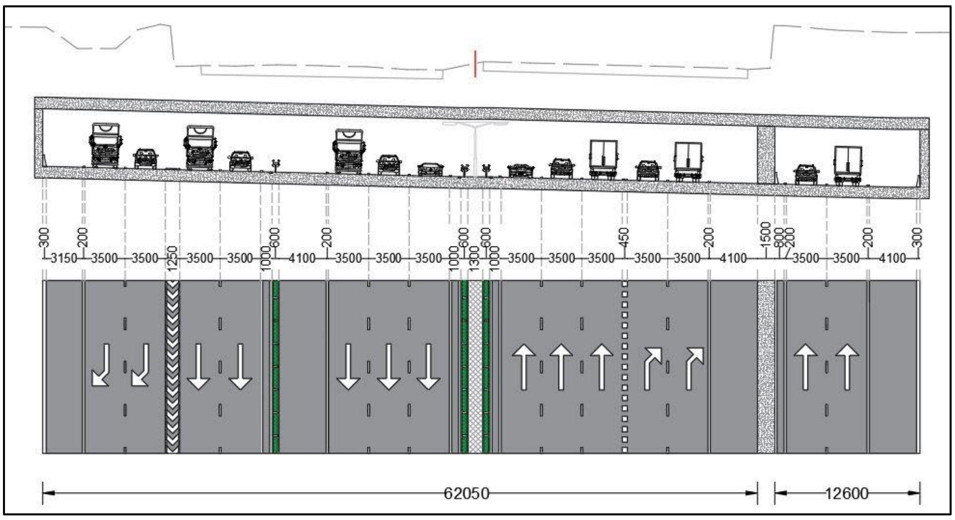

Figure 9.1: The alternative with an intermediate support outside the U-shaped concrete structure.

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Appendices

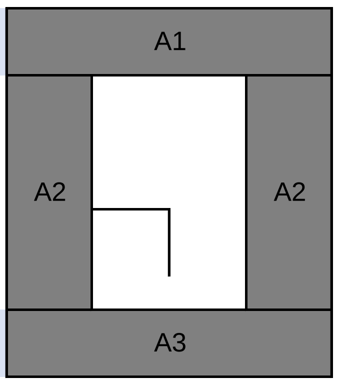
Appendix A Global calculation of preliminary design one

This appendix contains an Excel sheet of a custom-made box beam and a Maple sheet with the accompanying calculations.

Box beam design

SLS

h:	3000
ht:	220
hi:	2410
hb:	370



1659

1341

150	700	150
bw	bi	bw

b: 1000

A tendon	150	mm2
Number of tendons	138	[-]
A1	220000	mm2
A2	361500	mm2
A3	370000	mm2
z tov top	1659	mm

INPUT IN MAPLE:

Atot	1,313	m2
Izz	1,4335	m4
Wbot	1,0689	m3
Wtop	0,8641	m3
ep	1,1561	m
L	75	M
q_ligger_eg	34,138	kN/m
q_ligger_eq	30,2	kN/m

Econcrete	44000	MPa
Esteel	195000	MPa
Span L	75	m
rho C90/105	26	kN/m3
q eq	30,2	kN/m2
rho steel	78	kN/m3
G	2560,35	kN
Geinds	236,8548	kN
Gtot	2797,205	kN
Mg:=	24003	kNm
Mq:=	21234	kNm
Mgq:=	45238	kNm

core point to bottom flange 814 mm

z = (O2*K4*0,5*C4+2*O2*K5*(C4+0,5*C6)+O2*(K6-K2*K3)*(C4+C6+0,5*C13)+O3*K2*K3*(C4+C6+0,5*C13))/(O2*K4+2*O2*K5+O2*(K6-K2*K3)+O3*K2*K3)

SUM: EA*a/EA

z = 1659 mm

Homogeneous material:

z = 1596

Density of both material:

z = 1634

> #BOX BEAM CALCULATION WITH EXCEL SHEET

>

> #Sentence in yellow is input

>

> restart;

>

> #Data loaded from excel sheet and units in meters and kN:

> with(ExcelTools) :

> Info1 := ExcelTools:-Import("C:\\Users\\908277\\Documents\\Master Thesis Niels van Bergenhenegouwen\\Thesis bestanden\\Excel sheets\\Boxbeam.xlsx", "Sheet1", "K11:K18"); Info11 := convert(Info1, Vector) :

$$Info1 := \begin{bmatrix} 1.313 \\ 1.4334779416666668 \\ 1.0212599397215554 \\ 0.897964736779539 \\ 1.2186367098248285 \\ 75.0 \\ 34.138 \\ 30.2 \end{bmatrix} \quad (1)$$

> $Atot := Info11[1]; Izz := Info11[2]; Wbot := Info11[3]; Wtop := Info11[4]; ep := Info11[5]; L := Info11[6]; q\_ligger\_eg := Info11[7]; q\_ligger\_eq := Info11[8];$

$$Atot := 1.313$$
$$Izz := 1.4334779416666668$$
$$Wbot := 1.0212599397215554$$
$$Wtop := 0.897964736779539$$
$$ep := 1.2186367098248285$$
$$L := 75.0$$
$$q\_ligger\_eg := 34.138$$
$$q\_ligger\_eq := 30.2 \quad (2)$$

> $Mg := \frac{1}{8} \cdot q\_ligger\_eg \cdot L^2; Mq := \frac{1}{8} \cdot q\_ligger\_eq \cdot L^2; Mgq := Mg + Mq; V := 0.5 \cdot (q\_ligger\_eq + q\_ligger\_eg) \cdot L;$

$$Mg := 24003.28125$$
$$Mq := 21234.37500$$
$$Mgq := 45237.65625$$
$$V := 2412.67500 \quad (3)$$

>

> #Top fiber, t=0:

> $eq1 := +\frac{Fp \cdot ep}{Wtop} - \frac{Fp}{Atot} - \frac{Mg}{Wtop} \leq 0$

$$eq1 := 0.5954951390\,Fp \leq 26730.76154 \qquad (4)$$

>

> #Bottom fiber, t=0:

> $eq2 := -\frac{Fp \cdot ep}{Wbot} - \frac{Fp}{Atot} + \frac{Mg}{Wbot} \geq -0.6 \cdot 90000$

$$eq2 := 0. \leq -1.954882529\,Fp + 77503.59620 \qquad (5)$$

>

> #Top fiber, t=inf: Assume 20% prestress losses:

> $eq3 := +\frac{0.8 \cdot Fp \cdot ep}{Wtop} - \frac{0.8 \cdot Fp}{Atot} - \frac{Mgq}{Wtop} \geq -0.6 \cdot 90000$

$$eq3 := 0. \leq 0.4763961116\,Fp + 3622.01254 \qquad (6)$$

>

> #Bottom fiber, t=inf: Assume 20% prestress losses:

> $eq4 := -\frac{0.8 \cdot Fp \cdot ep}{Wbot} - \frac{0.8 \cdot Fp}{Atot} + \frac{Mgq}{Wbot} \leq 0$

$$eq4 := -1.563906023\,Fp \leq -44295.92749 \qquad (7)$$

>

> #OVERVIEW:

> $sol1 := solve(\{eq1\}, \{Fp\}); sol2 := solve(\{eq2\}, \{Fp\}); sol3 := solve(\{eq3\}, \{Fp\}); sol4 := solve(\{eq4\}, \{Fp\});$

$$sol1 := \{Fp \leq 44888.29512\}$$
$$sol2 := \{Fp \leq 39646.16546\}$$
$$sol3 := \{-7602.943122 \leq Fp\}$$
$$sol4 := \{28323.90619 \leq Fp\} \qquad (8)$$

>

> #input; Fp in [N]:

> $Fpp := 28324000\,N; Astrand := 150\,mm^2; sigma\_p0 := \frac{1395\,N}{mm^2};$

$$Fpp := 28324000\,N$$
$$Astrand := 150\,mm^2$$
$$sigma\_p0 := \frac{1395\,N}{mm^2} \qquad (9)$$

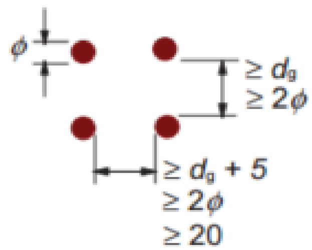
> #Number of strands:

> $Aptot := evalf\left(\frac{Fpp}{sigma\_p0}\right); Amount\_stands := \frac{Aptot}{Astrand};$

$$Aptot := 20303.94265\,mm^2$$
$$Amount\_stands := 135.3596177 \qquad (10)$$

>

> **#CONFIGURATION TENDONS in [mm]:**



>

> **#INPUT: cover and strand diameter**

> $cover := 35\ mm;\ d\_strand := 15.7\ mm;\ h\_distance := 2 \cdot d\_strand;\ v\_distance := 2 \cdot d\_strand;\ h\_hoh := h\_distance + d\_strand;\ v\_hoh := v\_distance + d\_strand;$

$$cover := 35\ mm$$
$$d\_strand := 15.7\ mm$$
$$h\_distance := 31.4\ mm$$
$$v\_distance := 31.4\ mm$$
$$h\_hoh := 47.1\ mm$$
$$v\_hoh := 47.1\ mm \tag{11}$$

>

> **#horizontal distance = vertical distance if dg mixture is 16 mm:**

> $hoh\_tendon := h\_hoh;$

$$hoh\_tendon := 47.1\ mm \tag{12}$$

> $Info2 := ExcelTools$:-$Import$("C:\\Users\\908277\\Documents\\Master Thesis Niels van Bergenhenegouwen\\Thesis bestanden\\Excel sheets\\Boxbeam.xlsx", "Sheet1", "D18:D18"); $Info22 := convert(Info2, Vector)$: $bf := Info22[1]mm;$

$$Info2 := \begin{bmatrix} 1000.0 \end{bmatrix}$$
$$bf := 1000.0\ mm \tag{13}$$

> $Remaining\_strand\_space := bf - 2 \cdot cover - d\_strand;$

$$Remaining\_strand\_space := 914.3\ mm \tag{14}$$

> $N\_of\_strands\_per\_layer := \frac{Remaining\_strand\_space}{hoh\_tendon} + 1;\ Number\_of\_strands\_per\_layer := \mathrm{trunc}(N\_of\_strands\_per\_layer);\ A\_of\_layers\_strands := \frac{Amount\_stands}{Number\_of\_strands\_per\_layer};\ Amount\_of\_layers\_strands := \mathrm{ceil}(A\_of\_layers\_strands);\ Amount\_strands;$

$$N\_of\_strands\_per\_layer := 20.41188960$$
$$Number\_of\_strands\_per\_layer := 20$$
$$A\_of\_layers\_strands := 6.767980885$$
$$Amount\_of\_layers\_strands := 7$$
$$Amount\_strands \tag{15}$$

> $t\_flange := 2 \cdot cover + hoh\_tendon \cdot (Amount\_of\_layers\_strands - 1) + d\_strand$

$$t\_flange := 368.3\ mm \tag{16}$$

>

> # #Shear force;

> $Vsls := V \cdot 1000\ N$

$$Vsls := 2.412675000\ 10^6\ N \tag{17}$$

> $Vsls;$
$Shear\_info := ExcelTools:-$
Import("C:\\Users\\908277\\Documents\\Master Thesis Niels van Bergenhenegouwen\\Thesis bestanden\\Excel sheets\\Boxbeam.xlsx", "Sheet1", "A4:A4"); $Shear\_info1 := convert(Shear\_info, Vector)$:
$Shear\_info2 := ExcelTools:-$
Import("C:\\Users\\908277\\Documents\\Master Thesis Niels van Bergenhenegouwen\\Thesis bestanden\\Excel sheets\\Boxbeam.xlsx", "Sheet1", "D16:D16"); $Shear\_info3 := convert(Shear\_info2, Vector)$:

$$2.412675000\ 10^6\ N$$
$$Shear\_info := \left[\ 3000.0\ \right]$$
$$Shear\_info2 := \left[\ 150.0\ \right] \tag{18}$$

>

> # #Height structure and thickness webs:

> $h := Shear\_info1[1]mm; b\_web := Shear\_info3[1]mm; bw := 2 \cdot b\_web; z := 0.8 \cdot h; \sigma\_cp := \frac{Fpp}{Atot \cdot 1000000\ mm^2}; f\_ck := \frac{90\ N}{mm^2}; f\_cd := \frac{f\_ck}{1.5};$

$$h := 3000.0\ mm$$
$$b\_web := 150.0\ mm$$
$$bw := 300.0\ mm$$
$$z := 2400.00\ mm$$
$$\sigma\_cp := \frac{21.57197258\ N}{mm^2}$$
$$f\_ck := \frac{90\ N}{mm^2}$$
$$f\_cd := \frac{60.00000000\ N}{mm^2} \tag{19}$$

>

> # #Determining a_cw and v1=0.9-f_ck/200 >= 0.5 and assumption theta = 45 degrees:

> $a\_cw := 1.25; v1 := 0.5; \theta := convert(45\ degrees, radians);$

$$a\_cw := 1.25$$
$$v1 := 0.5$$

(20)

$$\theta := \frac{\pi}{4} \tag{20}$$

>

> # #Vrd_max:

> $Vrd\_max := evalf\left(\frac{a\_cw \cdot bw \cdot z \cdot v1 \cdot f\_cd}{\cot(\text{theta}) + \tan(\text{theta})}\right); Vsls := Vsls$

$$Vrd\_max := 1.350000000\ 10^{7}\ N$$

$$Vsls := 2.412675000\ 10^{6}\ N \tag{21}$$

>

> # #key figures

> $m3\_concrete\_per\_beam := Atot \cdot Lm^{3} + \left(\frac{(bf - bw)}{1000\ mm} \cdot 2.41 \cdot 0.9 \cdot 3\ m^{3}\right) \cdot 2; Amount\_strands := \text{ceil}(Amount\_stands); m3\_total\_concrete := m3\_concrete\_per\_beam \cdot 249; Amount\_strands\_total := Amount\_strands \cdot 249;$

$$m3\_concrete\_per\_beam := 107.5848000\ m^{3}$$

$$Amount\_strands := 136$$

$$m3\_total\_concrete := 26788.61520\ m^{3}$$

$$Amount\_strands\_total := 33864 \tag{22}$$

>

>

>

>

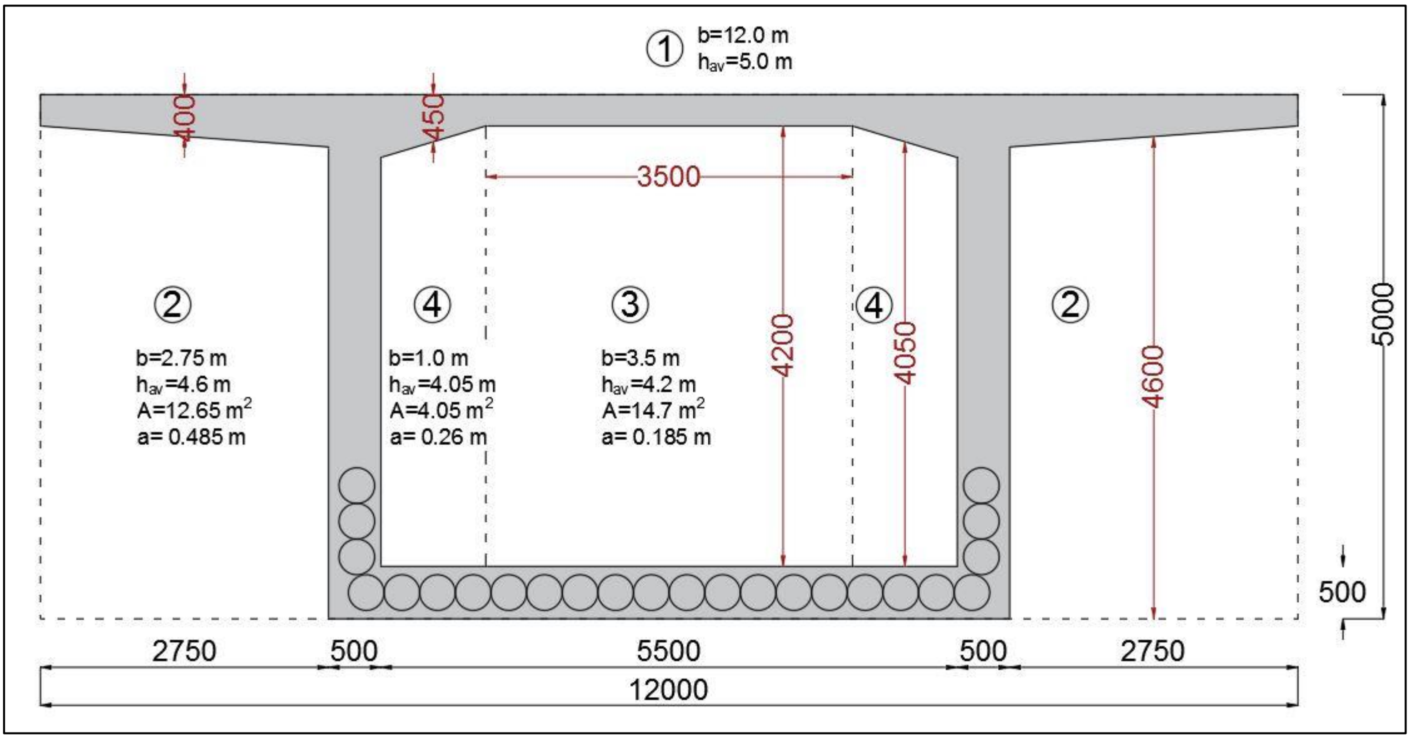
>

>

Appendix B Moment of inertia single cell box girder

To determine the working pre-stress force, the section modulus is required. One way to achieve this value is by calculating the moment of inertia. The moment of inertia of the box girder is determined via the figure in the Excel sheet in Appendix C. This appendix is meant to clarify the Excel sheet.

First, the neutral axis of the single cell box girder is calculated. Since the cross section is symmetrical around the vertical axis, the distance is calculated to top fibre. Thereafter, the moment of inertia "I" is determined of the total rectangular with its height of 5 m and width of 12 m. Then, the moment of inertia of the areas 2, 3 and 4 in are calculated and subtracted of the total one. These areas are visualized in Appendix Figure 1. The required values are represented in the figure as well as missing measurements. "A" is the total surface of each part and "a" is de distance from the centroidal axis of the particular area to the centroidal axis of the total rectangular. "h_{av}" is the average height of the area.



Appendix Figure 1 Overview single cell box girder cross-section with their values of the particular parts.

The estimated distance from the centroidal axis to top fibre can be calculated as follows:

$$z_{to\ top\ fibre} = \frac{\sum A_i * a_i}{\sum A_i} = 2215\ mm$$

With the value of $z_{to\ top\ fibre}$ every individual eccentricity of the areas 2, 3 and 4 can be calculated which are required to determine the moment of inertia of every single part. The section moduli result from the moments of inertia. These values are provided on the next page.

Moments of inertia and section moduli

To determine the working pre-stressing force, the section modulus is required. One way to achieve this value is by calculating the moment of inertia. The parts 1 to 4 can be retrieved in Appendix Figure 1.

1. $= \frac{1}{12} * b * h^3 = \frac{1}{12} * 12 * 5^3 = 125\, m^4$

2. $= \left(\frac{1}{12} * b * h^3 + a^2 * A\right) = \left(\frac{1}{12} * 2.75 * 4.6^3 + 0.485^2 * 12.65\right) = 25.28\, m^4$

3. $= \left(\frac{1}{12} * b * h^3 + a^2 * A\right) = \left(\frac{1}{12} * 3.5 * 4.2^3 + 0.185^2 * 14.7\right) = 22.11\, m^4$

4. $= \left(\frac{1}{12} * b * h^3 + a^2 * A\right) = \left(\frac{1}{12} * 1.5 * 4.806^3 + 0.097^2 * 1.5 * 4.806\right) = 13.94\, m^4$

The moment of inertia of the box girder:

$$1 - 2 * 2 - 3 - 2 * 4 = 40.7\, m^4$$

The section modulus W_{top} and W_{bot} are roughly:

$$W_{top} = \frac{I}{e_{top}} = \frac{40.7\, m^4}{2.2\, m} = 18.4\, m^3$$

$$W_{bot} = \frac{I}{e_{bot}} = \frac{40.7\, m^4}{2.8\, m} = 14.6\, m^3$$

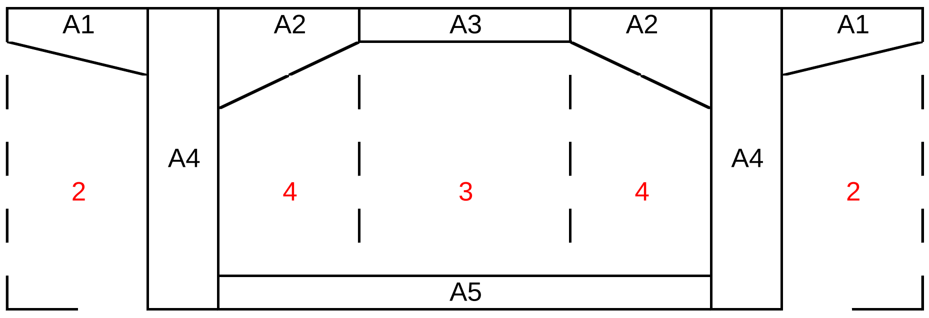
These values correspond with the values depicted in the Excel sheet in the next Appendix.

Appendix C Global calculation of preliminary design two

This appendix contains an Excel sheet of a single cell box girder and a Maple sheet with the accompanying calculations.

Girderbridge SLS

h:					
	d_av	0,4	d3	0,3	
			hv	0,3	
5	h_inw	4,1			
	d5	0,5			



	0,50	1	3,5	1	0,50	
	d4	lv	l_inw	lv	d4	
2,75			6,5			2,75
l1			l2			l1
			12			
			b			

span L	75	m
rho C90/105	26	kN/m3
q_eq	30,2	kN/m2
Mg:=	217546,9	kNm
Mq:=	254813	kNm
Mgq:=	472359	kNm
G=	23205	kN

A1	1,1	m2
A2	0,45	m2
A3	1,05	m2
A4	2,5	m2
A5	2,75	m2

Atot	11,9	m2
Izz	40,72	m4
Wbot	14,62	m3
Wtop	18,38	m3
ep	2,53	m2
L	75	m
q_ligger_eg	309,4	kN/m
q_ligger_eq	362,4	kN/m
l2	6,5	m

estimate distance z from neutral axis to top fiber

$$z = \frac{A1*0,5*d\_av*2+A2*(d3+0,5*hv)/2*2+A3*0,5*d3+A4*0,5*h*2+A5*(d\_av+h\_inw+0,5*d5)}{A2*A1+2*A2+A3+2*A4+A5}$$

z 2,215 m

lever arm (Steiner) and moments of inertia of the areas 2, 3 and 4:

1				I1	125	m4
2	a2	0,48	m	I2	25,28	m4
3	a3	0,18	m	I3	22,11	m4
4	a4	0,26	m	I4	5,81	m4

> #GIRDERBRIDGE CALCULATION WITH EXCEL SHEET

>

> #Sentence in yellow is input

>

> *restart*;

>

> #Data loaded from excel sheet and units in meters and kN:

> *with*(*ExcelTools*) :

> *Info1* := *ExcelTools*:-*Import*("C:\\Users\\908277\\Documents\\Master Thesis Niels van Bergenhenegouwen\\Thesis bestanden\\Excel sheets\\Girderbridge.xlsx", "Sheet1", "K21:K29"); *Info11* := *convert*(*Info1*, *Vector*) :

$$Info1 := \begin{bmatrix} 11.9 \\ 40.71660684126594 \\ 14.62173131380052 \\ 18.37942613223574 \\ 2.5346638655462184 \\ 75.0 \\ 309.40000000000003 \\ 362.4 \\ 6.5 \end{bmatrix} \quad \textbf{(1)}$$

> *Atot* := *Info11*[1]; *Izz* := *Info11*[2] ; *Wbot* := *Info11*[3]; *Wtop* := *Info11*[4] ; *ep* := *Info11*[5]; *L* := *Info11*[6] ; *q_ligger_eg* := *Info11*[7]; *q_ligger_eq* := *Info11*[8]; *l2* := *Info11*[9];

$$Atot := 11.9$$
$$Izz := 40.71660684126594$$
$$Wbot := 14.62173131380052$$
$$Wtop := 18.37942613223574$$
$$ep := 2.5346638655462184$$
$$L := 75.0$$
$$q\_ligger\_eg := 309.40000000000003$$
$$q\_ligger\_eq := 362.4$$
$$l2 := 6.5 \quad \textbf{(2)}$$

> $Mg := \frac{1}{8} \cdot q\_ligger\_eg \cdot L^2$; $Mq := \frac{1}{8} \cdot q\_ligger\_eq \cdot L^2$; $Mgq := Mg + Mq$; $V := 0.5 \cdot (q\_ligger\_eq + q\_ligger\_eg) \cdot L$;

$$Mg := 217546.8750$$
$$Mq := 254812.5000$$
$$Mgq := 472359.3750$$
$$V := 25192.50000 \quad \textbf{(3)}$$

>

> **#Top fiber, t=0:**

> $eq1 := + \frac{Fp \cdot ep}{Wtop} - \frac{Fp}{Atot} - \frac{Mg}{Wtop} \le 0$

$$eq1 := 0.05387405835\ Fp \le 11836.43458 \quad (4)$$

>

> **#Bottom fiber, t=0:**

> $eq2 := - \frac{Fp \cdot ep}{Wbot} - \frac{Fp}{Atot} + \frac{Mg}{Wbot} \ge -0.6 \cdot 90000$

$$eq2 := 0. \le -0.2573827068\ Fp + 68878.32531 \quad (5)$$

>

> **#Top fiber, t=inf: Assume 20% prestress losses:**

> $eq3 := + \frac{0.8 \cdot Fp \cdot ep}{Wtop} - \frac{0.8 \cdot Fp}{Atot} - \frac{Mgq}{Wtop} \ge -0.6 \cdot 90000$

$$eq3 := 0. \le 0.04309924664\ Fp + 28299.55801 \quad (6)$$

>

> **#Bottom fiber, t=inf: Assume 20% prestress losses:**

> $eq4 := - \frac{0.8 \cdot Fp \cdot ep}{Wbot} - \frac{0.8 \cdot Fp}{Atot} + \frac{Mgq}{Wbot} \le 0$

$$eq4 := -0.2059061655\ Fp \le -32305.29716 \quad (7)$$

>

> **#OVERVIEW:**

> $sol1 := solve(\{eq1\}, \{Fp\}); sol2 := solve(\{eq2\}, \{Fp\}); sol3 := solve(\{eq3\}, \{Fp\}); sol4 := solve(\{eq4\}, \{Fp\});$

$$sol1 := \{Fp \le 219705.6421\}$$
$$sol2 := \{Fp \le 267610.5406\}$$
$$sol3 := \{-656613.7512 \le Fp\}$$
$$sol4 := \{156893.2969 \le Fp\} \quad (8)$$

>

> **#input; Fp in [N]:About 75% prestress & 25% reinforecement:**

> $Fpp := 0.75 \cdot 157000000\ N; Astrand := 150\ mm^2; Acable := 24 \cdot Astrand; sigma\_p0 := \frac{1395\ N}{mm^2};$

$$Fpp := 1.177500000\ 10^8\ N$$
$$Astrand := 150\ mm^2$$
$$Acable := 3600\ mm^2$$
$$sigma\_p0 := \frac{1395\ N}{mm^2} \quad (9)$$

> **#Number of strands:Assume 3*2 cables in web:**

> $Aptot := evalf\left(\frac{Fpp}{sigma\_p0}\right)$; $Number\_cables\_flange := \mathrm{ceil}\left(\frac{Aptot}{Acable}\right)$;
$diameter\_anchor\_plate := 0.34\ m$; $required\_space := (Number\_cables\_flange - 6) \cdot (diameter\_anchor\_plate) \leq l2$;

$$Aptot := 84408.60215\ mm^2$$
$$Number\_cables\_flange := 24$$
$$diameter\_anchor\_plate := 0.34\ m$$
$$required\_space := 6.12\ m \leq 6.5 \tag{10}$$

>

> #Shear force;

> $Vsls := V \cdot 1000\ N$

$$Vsls := 2.519250000\ 10^7\ N \tag{11}$$

> $Vsls$;
$Height := ExcelTools$:-
$Import$("C:\\Users\\908277\\Documents\\Master Thesis Niels van Bergenhenegouwen\\Thesis bestanden\\Excel sheets\\Girderbridge.xlsx", "Sheet1", "B4:B4"); $Height1 := convert(Height, Vector)$:
$Web\_width := ExcelTools$:-
$Import$("C:\\Users\\908277\\Documents\\Master Thesis Niels van Bergenhenegouwen\\Thesis bestanden\\Excel sheets\\Girderbridge.xlsx", "Sheet1", "J14:J14"); $Web\_width1 := convert(Web\_width, Vector)$:

$$2.519250000\ 10^7\ N$$
$$Height := \left[\ 5.0\ \right]$$
$$Web\_width := \left[\ 0.5\ \right] \tag{12}$$

>

> #Height structure and thickness webs:

> $h := Height1[1] \cdot 1000\ mm$; $b\_web := Web\_width1[1] \cdot 1000\ mm$; $bw := 2 \cdot b\_web$; $z := 0.8 \cdot h$;
$\sigma\_cp := \frac{Fpp}{Atot \cdot 1000000\ mm^2}$; $f\_ck := \frac{90\ N}{mm^2}$; $f\_cd := \frac{f\_ck}{1.5}$;

$$h := 5000.0\ mm$$
$$b\_web := 500.0\ mm$$
$$bw := 1000.0\ mm$$
$$z := 4000.00\ mm$$
$$\sigma\_cp := \frac{9.894957984\ N}{mm^2}$$
$$f\_ck := \frac{90\ N}{mm^2}$$
$$f\_cd := \frac{60.00000000\ N}{mm^2} \tag{13}$$

>

> **#Determining a_cw and v1=0.9-f_ck/200 >= 0.5 and assumption theta = 45 degrees:**

> $a\_cw := 1 + \frac{\sigma\_cp}{f\_cd}$; $v1 := 0.5$; theta $:= evalf(convert(45\ degrees, radians))$;

$$a\_cw := 1.164915966$$
$$v1 := 0.5$$
$$\theta := 0.7853981635 \qquad (14)$$

>

> **#Vrd_max:More than enough capacity**

> $Vrd\_max := evalf\left(\frac{a\_cw \cdot bw \cdot z \cdot v1 \cdot f\_cd}{\cot(\text{theta}) + \tan(\text{theta})}\right)$; $Vuls := 1.5 \cdot Vsls$

$$Vrd\_max := 6.989495795\ 10^{7}\ N$$
$$Vuls := 3.778875000\ 10^{7}\ N \qquad (15)$$

>

>

> **#key figures**

> $m3\_concrete\_per\_beam := Atot \cdot Lm^{3}$; $Number\_cable := \text{ceil}\left(\frac{Aptot}{Acable}\right)$; $weight := \text{ceil}\left(\frac{m3\_concrete\_per\_beam \cdot 26}{10 m^{3}}\right) \cdot tons$; $m3\_total\_concrete := m3\_concrete\_per\_beam \cdot 20$; $Number\_post\_tensioning\_cables\_total := Number\_cable \cdot 20$; $Number\_strands := Number\_post\_tensioning\_cables\_total \cdot 24$

$$m3\_concrete\_per\_beam := 892.50\ m^{3}$$
$$Number\_cable := 24$$
$$weight := 2321\ tons$$
$$m3\_total\_concrete := 17850.00\ m^{3}$$
$$Number\_post\_tensioning\_cables\_total := 480$$
$$Number\_strands := 11520 \qquad (16)$$

>

>

>

Appendix D Pile capacity

On the next pages, the pile capacity is determined with the help of the software program D-foundations. The piles around CPT 1 have a capacity of 2838 kN, while the piles at CPT 2 have a capacity of 2155 kN.

Report for D-Foundations 17.1

Design and Verification according to Eurocode 7 of Bearing/Tension Piles and Shallow Foundations
Developed by Deltares

Company:	Royal HaskoningDHV
Date of report:	3-10-2018
Time of report:	13:22:23
Date of calculation:	3-10-2018
Time of calculation:	12:29:50
Filename:	C:\Users\907810\Box Sync\My Box (907810)\CPT digitaliseren\Niels\1
Project identification:	
	D-Foundations 1

Pile capacity at CPT 1

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2 Input Data

2.1 General Input Data

Model Bearing Piles (EC7-NL)

2.2 General Report Data

Geotechnical consultant :
Design engineer superstructure :
Principal :
Title 1 :
Title 2 :
Title 3 : D-Foundations 1
Number of project :
Location of project :

2.3 Application Area Model Bearing Piles

The verifications performed by the model BEARING PILES of D-FOUNDATIONS concern pile foundations on which axial static or quasi-static loads cause pressures in the piles. The calculations of pile forces and pile displacements are based on Cone Penetration Tests. Possible rise of (tension-)piles and horizontal displacements of piles and/or pile groups are not taken into account.

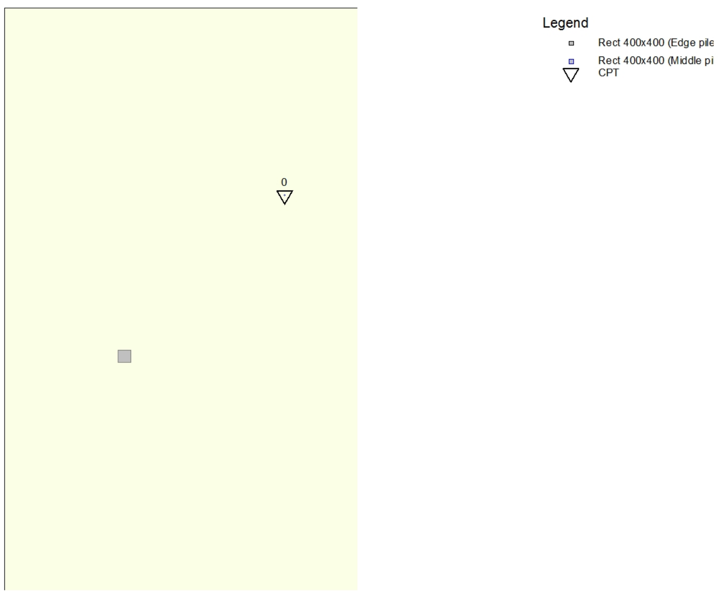
2.4 Superstructure

Rigidity of the superstructure : Non-Rigid

2.5 General CPT Data

Number of CPT's : 1
Timing of CPT's : CPT - Excavation - Install

2.5.1 View of CPT's in Foundation Plan



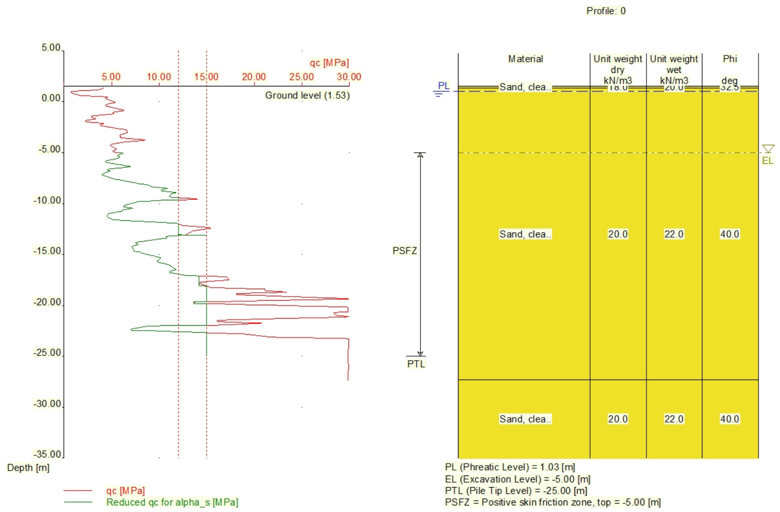
Number/Name CPT	Pile tip level [m R.L.]	Top of pos. friction zone [m R.L.]	Bottom of neg. friction zone [m R.L.]	X-coor-dinate [m]	Y-coor-dinate [m]
1: 0	-25.00	-5.00	1.53	0.00	0.00

2.6 Soil Data

Number of soil profiles (= number of CPT's) : 1

2.6.1 Soil Profile 0

Belonging to CPT	0
Surface level in [m. reference level] :	1.53
Phreatic level in [m. reference level] :	1.03
Pile tip level in [m. reference level] :	-25.00
Top of positive skin friction zone in [m. reference level] :	-5.00
Bottom of negative skin friction zone in [m. reference level] :	1.53
OCR-value foundation layer :	1.00
Expected groundlevel settlement in [m] :	0.11
Number of layers in profile :	3



Number layer	Top layer [m R.L.]	Gamma [kN/m3]	Gamma;sat [kN/m3]	Phi [deg]	Soil Type	Median (Sand/Gravel) [mm]
1	1.530	18.00	20.00	32.50	Sand	0.200
2	1.363	20.00	22.00	40.00	Sand	0.200
3	-27.315	20.00	22.00	40.00	Sand	0.200

2.7 Pile Types

2.7.1 Pile type : Rect 400x400

Pile type : Prefabricated concrete pile

Note: Factor alpha_p has the pre 2016 value.

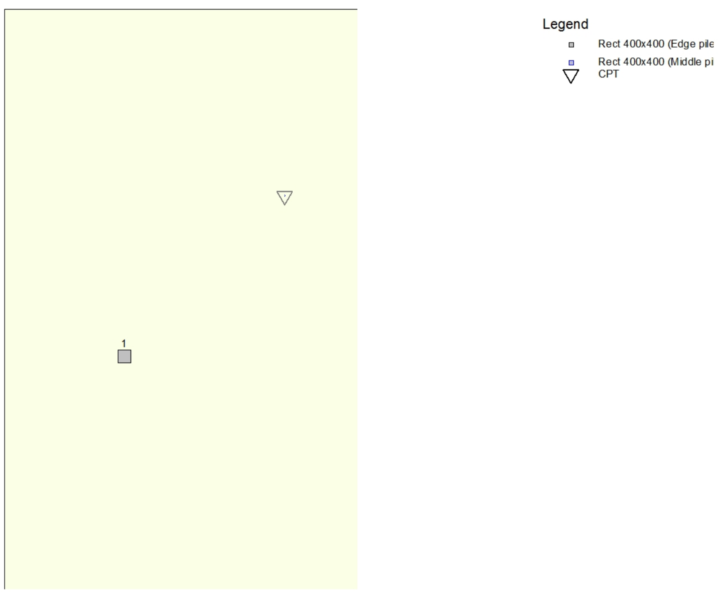
Materialtype for pile : Concrete
Slip layer : None
Pile shape : Rectangular pile
beta (Shape factor) according to figure 7.i, NEN 9997-1:2016.
s (factor for the influence of the shape of the crosssection of the pile base) according to NEN 9997-1:2016.

Pile dimensions :
Smallest side pile tip [m] : 0.400
Largest side pile tip [m] : 0.400

2.8 Foundation Plan

Number of piles : 1
Number of collaborating piles* : 1
* : 0 = not defined, 1 = non rigid superstructure, >1 = rigid superstructure

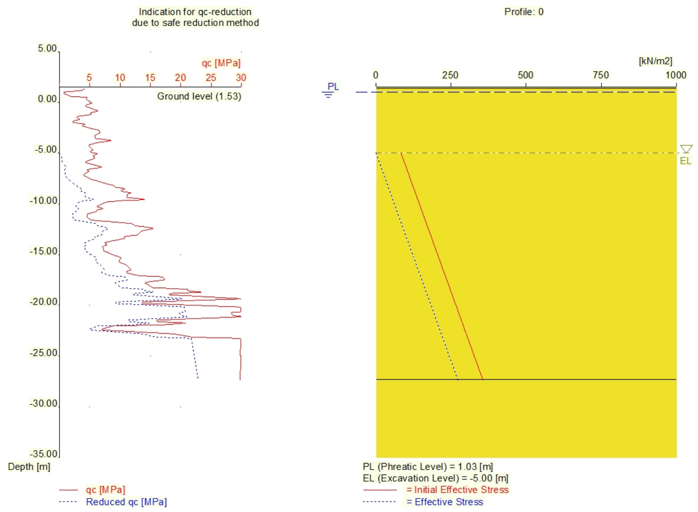
2.8.1 View of Foundation Plan



Pile nr/name	X-coor-dinate [m]	Y-coor-dinate [m]	Fc;d (STR/GEO) [kN]	Fc;d (SLS) [kN]	P0 [kN/m2]	Pile head level [m R.L.]
1: 1	-5.00	-5.00	0.00	0.00	0.00	-5.00

2.9 Excavation Data

Excavation level in [m. reference level] : -5.00
Reduction model : Safe (NEN)



2.10 Overruled Parameters

All parameters according to standard.

2.11 Model Options

Use pilegroup for negative skin friction (standard)
Do not create intermediate results file
Use reduction for continuous flight auger piles (standard)
Use the influence of excavations (standard).

2.12 Model Options

Selected pile types :
-Rect 400x400

Selected profiles :
-0

3 Bearing Piles (EC7-NL): Results of the option Preliminary Design, Bearing capacity at fi

3.1 Errors and Warnings

Warning : The depth of the CPT's does not meet the requirements as set by NEN 9997-1:2016 art. 3.2.3.

3.2 Remarks

When checking the survey and testing of soil according to NEN 9997-1:2016 art. 3.2.3 lid (e), the program uses the provided CPT test level. It does NOT take into account possible different pile tip levels. When different pile tip levels are used in this calculation, the user itself must check for possibly required additional survey and testing of soil.

Note : The calculations performed are based on a single pile for limit state STR/GEO (= ultimate limit state).
Due to the nature of preliminary design, a single pile is always assumed. A possible pileplan is disregarded when using the preliminary design option. Hence a non rigid superstructure is assumed and pile group effects are not considered.

3.3 Calculation Parameters

3.3.1 Pile Factors

gamma;b (NEN 9997-1:2016, table A.6 A.7 A.8, Limit State STR/GEO) :	1.20
gamma;b (NEN 9997-1:2016, table A.6 A.7 A.8, the Serviceability Limit State) :	1.00
gamma;s (NEN 9997-1:2016, table A.6 A.7 A.8, Limit State STR/GEO) :	1.20
gamma;s (NEN 9997-1:2016, table A.6 A.7 A.8, the Serviceability Limit State) :	1.00
xi3 (NEN 9997-1:2016, table A.10a, for N = 1) :	1.39
xi4 (NEN 9997-1:2016, table A.10a, for N = 1) :	1.39

3.3.2 Pile type : Rect 400x400

Pile type :	Prefabricated concrete pile
Note: Factor alpha_p has the pre 2016 value.	
Materialtype for pile :	Concrete
Slip layer :	None
Pile shape :	Rectangular pile
beta (Shape factor: figuur 7.i, NEN 9997-1:2016 art. 7.6.2.3(g) : Pile tip) :	1.00
s (NEN 9997-1:2016 art. 7.6.2.3(h) : factor for the influence of the shape of the crosssection of the pile base) :	1.00

Pile dimensions :

Smallest side pile tip [m] :	0.400
Largest side pile tip [m] :	0.400

CPT	Alpha_s Sand/ Gravel	Alpha_s Clay/Loam Peat	Alpha_p
0	0.0100	--	1.0000

3.4 Results for pile type : Rect 400x400

CPT name	Level [m R.L.]	Groundlevel [m R.L.]	Rb;cal;max [kN]	Rs;cal;max [kN]	Rc;cal;max [kN]	Rc;d [kN]	F;nsf;rep [kN]	Fnsf;d [kN]
0	-25.00	1.53	2400	2335	4735	2838	0	0

3.5 Summary Net Bearing Capacity in kN

CPT name	Groundlevel [m R.L.]	Level [m R.L.]	Rect 400x400 Rc;net;d [kN]
0	1.53	-25.00	2838.00

* Rc;net;d = Rc;d - Fnsf;d

End of Report

Report for D-Foundations 17.1

Design and Verification according to Eurocode 7 of Bearing/Tension Piles and Shallow Foundations
Developed by Deltares

Company:	Royal HaskoningDHV
Date of report:	3-10-2018
Time of report:	13:22:08
Date of calculation:	3-10-2018
Time of calculation:	13:04:45
Filename:	C:\Users\907810\Box Sync\My Box (907810)\CPT digitaliseren\Niels\2
Project identification:	
	D-Foundations 2

Pile capacity at CPT 2

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2 Input Data

2.1 General Input Data

Model Bearing Piles (EC7-NL)

2.2 General Report Data

Geotechnical consultant :
Design engineer superstructure :
Principal :
Title 1 :
Title 2 :
Title 3 : D-Foundations 2
Number of project :
Location of project :

2.3 Application Area Model Bearing Piles

The verifications performed by the model BEARING PILES of D-FOUNDATIONS concern pile foundations on which axial static or quasi-static loads cause pressures in the piles. The calculations of pile forces and pile displacements are based on Cone Penetration Tests. Possible rise of (tension-)piles and horizontal displacements of piles and/or pile groups are not taken into account.

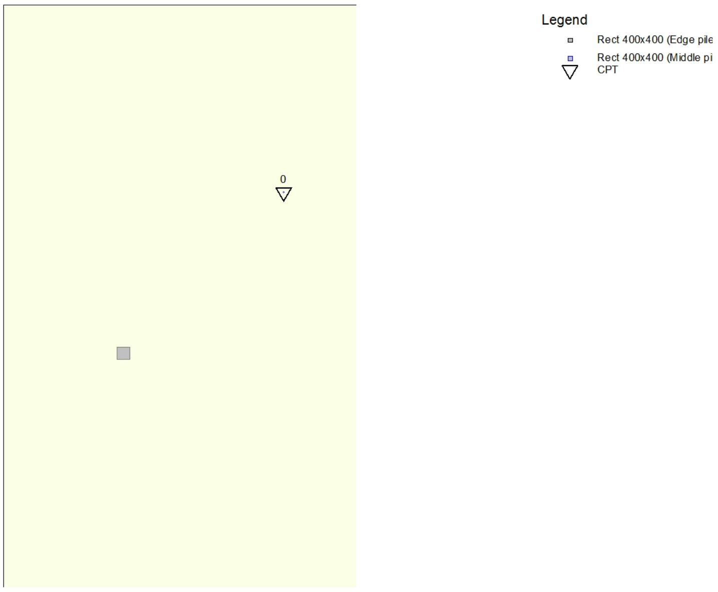
2.4 Superstructure

Rigidity of the superstructure : Non-Rigid

2.5 General CPT Data

Number of CPT's : 1
Timing of CPT's : CPT - Excavation - Install

2.5.1 View of CPT's in Foundation Plan



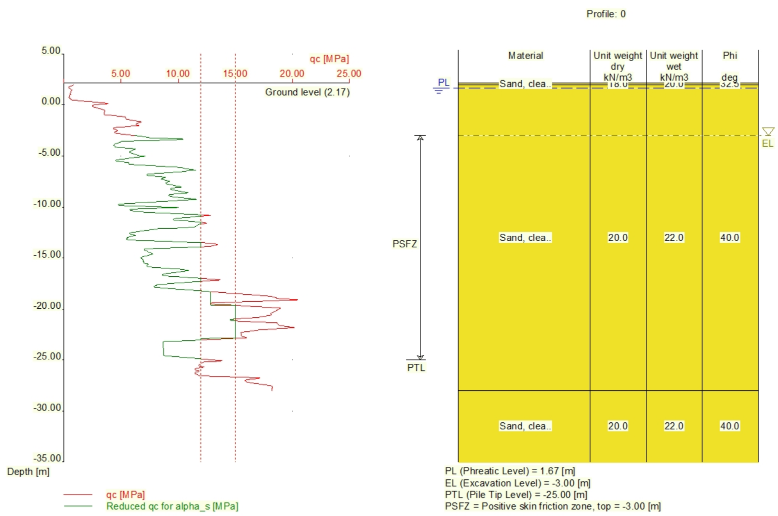
Number/Name CPT	Pile tip level [m R.L.]	Top of pos. friction zone [m R.L.]	Bottom of neg. friction zone [m R.L.]	X-coor-dinate [m]	Y-coor-dinate [m]
1: 0	-25.00	-3.00	2.17	0.00	0.00

2.6 Soil Data

Number of soil profiles (= number of CPT's) : 1

2.6.1 Soil Profile 0

Belonging to CPT	0
Surface level in [m. reference level] :	2.17
Phreatic level in [m. reference level] :	1.67
Pile tip level in [m. reference level] :	-25.00
Top of positive skin friction zone in [m. reference level] :	-3.00
Bottom of negative skin friction zone in [m. reference level] :	2.17
OCR-value foundation layer :	1.00
Expected groundlevel settlement in [m] :	0.11
Number of layers in profile :	3



Number layer	Top layer [m R.L.]	Gamma [kN/m3]	Gamma;sat [kN/m3]	Phi [deg]	Soil Type	Median (Sand/Gravel) [mm]
1	2.170	18.00	20.00	32.50	Sand	0.200
2	1.987	20.00	22.00	40.00	Sand	0.200
3	-27.995	20.00	22.00	40.00	Sand	0.200

2.7 Pile Types

2.7.1 Pile type : Rect 400x400

Pile type : Prefabricated concrete pile
Note: Factor alpha_p has the pre 2016 value.

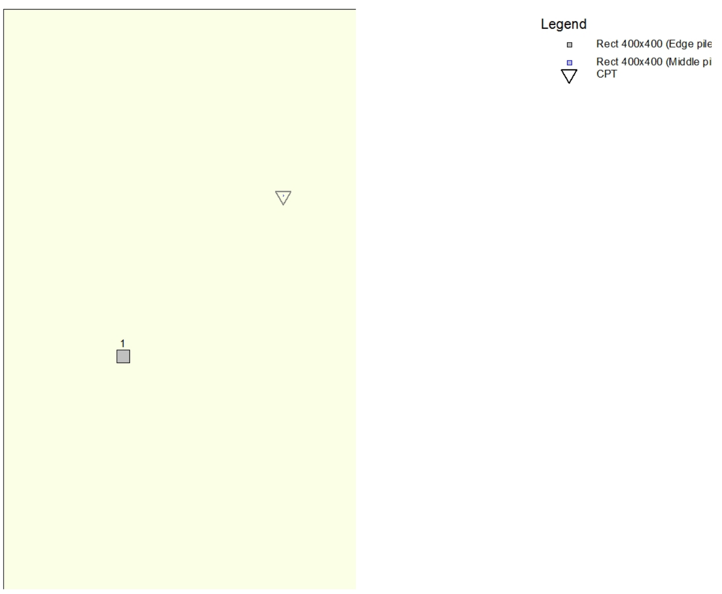
Materialtype for pile : Concrete
Slip layer : None
Pile shape : Rectangular pile
beta (Shape factor) according to figure 7.i, NEN 9997-1:2016.
s (factor for the influence of the shape of the crosssection of the pile base) according to NEN 9997-1:2016.

Pile dimensions :
Smallest side pile tip [m] : 0.400
Largest side pile tip [m] : 0.400

2.8 Foundation Plan

Number of piles : 1
Number of collaborating piles* : 1
* : 0 = not defined, 1 = non rigid superstructure, >1 = rigid superstructure

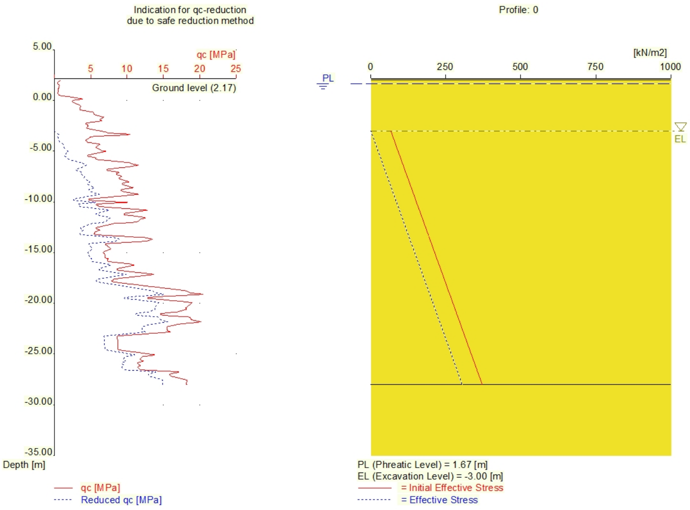
2.8.1 View of Foundation Plan



Pile nr/name	X-coor-dinate [m]	Y-coor-dinate [m]	Fc;d (STR/GEO) [kN]	Fc;d (SLS) [kN]	P0 [kN/m2]	Pile head level [m R.L.]
1: 1	-5.00	-5.00	0.00	0.00	0.00	-3.00

2.9 Excavation Data

Excavation level in [m. reference level] : -3.00
Reduction model : Safe (NEN)



2.10 Overruled Parameters

All parameters according to standard.

2.11 Model Options

Use pilegroup for negative skin friction (standard)
Do not create intermediate results file
Use reduction for continuous flight auger piles (standard)
Use the influence of excavations (standard).

2.12 Model Options

Selected pile types :
-Rect 400x400

Selected profiles :
-0

3 Bearing Piles (EC7-NL): Results of the option Preliminary Design, Bearing capacity at fix

3.1 Errors and Warnings

Warning : The depth of the CPT's does not meet the requirements as set by NEN 9997-1:2016 art. 3.2.3.

3.2 Remarks

When checking the survey and testing of soil according to NEN 9997-1:2016 art. 3.2.3 lid (e), the program uses the provided CPT test level. It does NOT take into account possible different pile tip levels. When different pile tip levels are used in this calculation, the user itself must check for possibly required additional survey and testing of soil.

Note : The calculations performed are based on a single pile for limit state STR/GEO (= ultimate limit state).
Due to the nature of preliminary design, a single pile is always assumed. A possible pileplan is disregarded when using the preliminary design option. Hence a non rigid superstructure is assumed and pile group effects are not considered.

3.3 Calculation Parameters

3.3.1 Pile Factors

gamma;b (NEN 9997-1:2016, table A.6 A.7 A.8, Limit State STR/GEO) :	1.20
gamma;b (NEN 9997-1:2016, table A.6 A.7 A.8, the Serviceability Limit State) :	1.00
gamma;s (NEN 9997-1:2016, table A.6 A.7 A.8, Limit State STR/GEO) :	1.20
gamma;s (NEN 9997-1:2016, table A.6 A.7 A.8, the Serviceability Limit State) :	1.00
xi3 (NEN 9997-1:2016, table A.10a, for N = 1) :	1.39
xi4 (NEN 9997-1:2016, table A.10a, for N = 1) :	1.39

3.3.2 Pile type : Rect 400x400

Pile type :	Prefabricated concrete pile
Note: Factor alpha_p has the pre 2016 value.	
Materialtype for pile :	Concrete
Slip layer :	None
Pile shape :	Rectangular pile
beta (Shape factor: figuur 7.i, NEN 9997-1:2016 art. 7.6.2.3(g) : Pile tip) :	1.00
s (NEN 9997-1:2016 art. 7.6.2.3(h) : factor for the influence of the shape of the crosssection of the pile base) :	1.00
Pile dimensions :	
Smallest side pile tip [m] :	0.400
Largest side pile tip [m] :	0.400

CPT	Alpha_s Sand/ Gravel	Alpha_s Clay/Loam Peat	Alpha_p
0	0.0100	--	1.0000

3.4 Results for pile type : Rect 400x400

CPT name	Level [m R.L.]	Groundlevel [m R.L.]	Rb;cal;max [kN]	Rs;cal;max [kN]	Rc;cal;max [kN]	Rc;d [kN]	F;nsf;rep [kN]	Fnsf;d [kN]
0	-25.00	2.17	1322	2272	3594	2155	0	0

3.5 Summary Net Bearing Capacity in kN

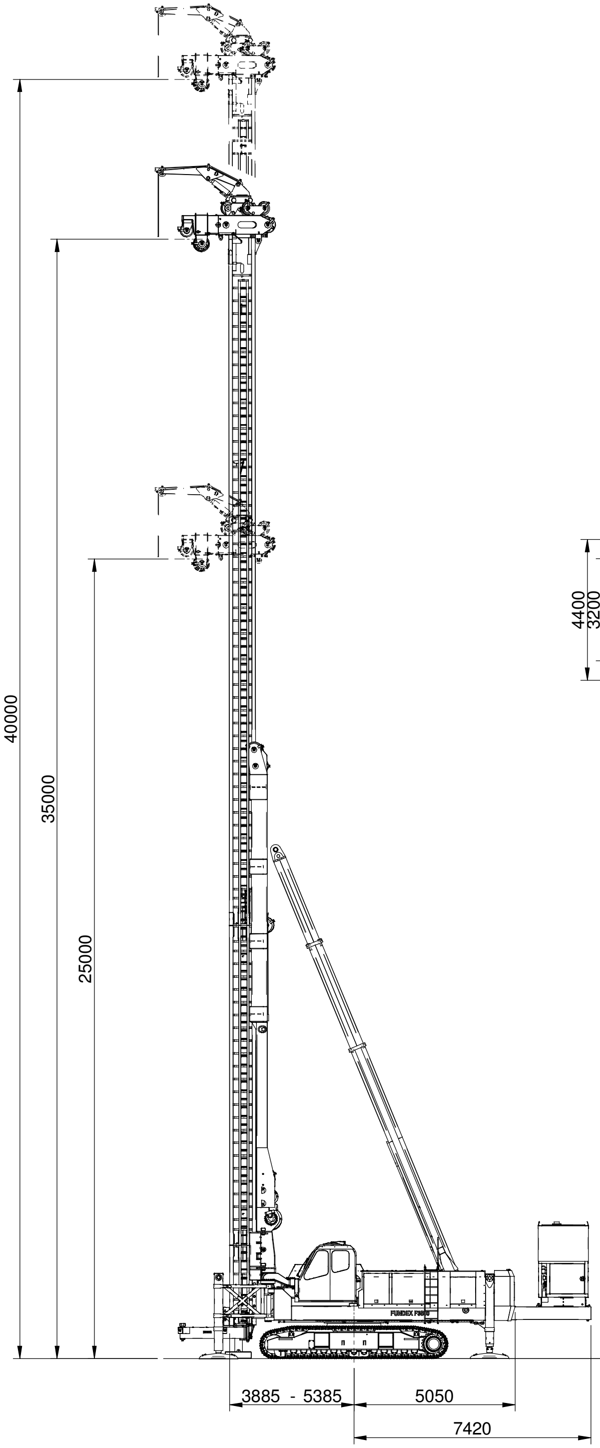
CPT name	Groundlevel [m R.L.]	Level [m R.L.]	Rect 400x400 Rc;net;d [kN]
0	2.17	-25.00	2155.00

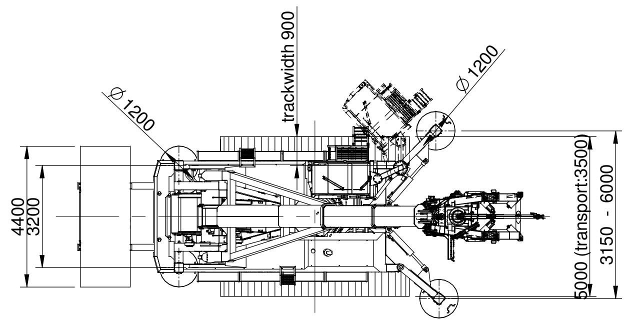
* Rc;net;d = Rc;d - Fnsf;d

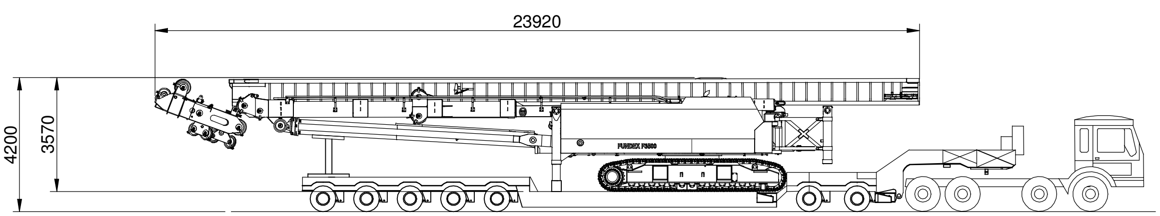
End of Report

Appendix E Piling equipment

The Terracon F3500 piling equipment [21].


40000
35000
25000
3885 - 5385
5050
7420


trackwidth 900
∅1200
∅1200
4400
3200
5000 (transport:3500)
3150 - 6000


23920
4200
3570

Appendix F Calculation average tendon height

The next pages show the calculation of the average tendon height. The calculation is performed with the program Maple, which is an advanced mathematical program useful to compute extensive calculations. The red text explicates the calculation briefly.

> **#Average tendon height:**

> **#x is the number of tendons above neutral axis.**
> **#y is the average distance to topfiber of the number of tendons x.**
> **#z is the number of tendons in bottom flange which should be unbonded.**

>

> $restart;$

> $eq6 := 2815(138 - x) + y \cdot x - 815 \cdot z = 138 \cdot 2000;$

$$eq6 := y\,x - 2815\,x - 815\,z + 388470 = 276000 \quad (1)$$

> $x := 26;\ y := 350;\ z11 := evalf(solve(eq6, z));$

$$x := 26$$
$$y := 350$$
$$z11 := 59.36196319 \quad (2)$$

> $number\_unbonded\_tendon := \mathrm{ceil}(z11);$

$$number\_unbonded\_tendon := 60 \quad (3)$$

>

> **#60 tendons should be unbonded. Then, the fictitious tendon lies in core area. This is proven with the next calculation:**

>

> $eq7 := 2815((138 - 60) - x2) + y2 \cdot x2 - 815 \cdot z2 = 2000 \cdot (138 - 60);$

$$eq7 := y2\,x2 - 2815\,x2 - 815\,z2 + 219570 = 156000 \quad (4)$$

> $x2 := 26;\ y2 := 350;\ z22 := evalf(solve(eq7, z2));$

$$x2 := 26$$
$$y2 := 350$$
$$z22 := -0.6380368098 \quad (5)$$

> **#z is almost zero, therefore, it's proven.**

>

> $Actual\_average\_tendon\_height\_towards\_topfibre :=$
$$\mathrm{trunc}\left(evalf\left(\frac{x \cdot y + (136 - x - number\_unbonded\_tendon) \cdot 2815}{x + (136 - x - number\_unbonded\_tendon)}\right)\right) \cdot mm;$$

$$Actual\_average\_tendon\_height\_towards\_topfibre := 1971\ mm \quad (6)$$

>

Appendix G Design check preliminary design

The following calculation proves that the preliminary box beam design isn't sufficient. It doesn't meet the rotational capacity criterion by far. Therefore, the calculation is stopped after this result.

> #Calculation xu of initial beam:

> restart;

> #input parameters C90/105, height flange=220 mm, dp=2815 mm on average:

> $dp := 2815\ mm;\ \varepsilon\_c3 := 0.0023;\ h\_fl := 220\ mm;\ f\_cd := \frac{60\ N}{mm^2};\ Ec := \frac{f\_cd}{\varepsilon\_c3};\ Ep := \frac{195000\ N}{mm^2};$

$$dp := 2815\ mm$$
$$\varepsilon\_c3 := 0.0023$$
$$h\_fl := 220\ mm$$
$$f\_cd := \frac{60\ N}{mm^2}$$
$$Ec := \frac{26086.95652\ N}{mm^2}$$
$$Ep := \frac{195000\ N}{mm^2} \quad \textbf{(1)}$$

>

> #Pm_inf is determined in initial calculation as well as Ap:

> $Pm\_inf := 0.8 \cdot 28800000\ N;\ Ap := 138 \cdot 150\ mm^2;\ \sigma\_pinf := \frac{Pm\_inf}{Ap};\ \varepsilon\_p := \Delta\varepsilon\_p + \frac{\sigma\_pinf}{Ep};$

$$Pm\_inf := 2.30400000\ 10^7\ N$$
$$Ap := 20700\ mm^2$$
$$\sigma\_pinf := \frac{1113.043478\ N}{mm^2}$$
$$\varepsilon\_p := \Delta\varepsilon\_p + 0.005707915272 \quad \textbf{(2)}$$

>

> #Delta epsilon p is the unknown. Strains are calculated as function of this unknown:

> $eq1 := \frac{\Delta\varepsilon\_p}{dp - xu} = \frac{\varepsilon\_c3}{xu - \frac{h\_fl}{2}};$

$eq2 := \frac{\varepsilon\_flange}{xu - h\_fl} = \frac{\Delta\varepsilon\_p}{dp - xu};$

$eq3 := \Delta\sigma\_p = \left(\frac{1522\ N}{mm^2} + \left(\frac{\left(\varepsilon\_p - 7.8 \cdot 10^{-3}\right)}{\left(35 \cdot 10^{-3} - 7.8 \cdot 10^{-3}\right)}\right) \cdot \left(\frac{1691\ N}{mm^2} - \frac{1522\ N}{mm^2}\right)\right) - \sigma\_pinf;$

$$eq1 := \frac{\Delta\varepsilon\_p}{2815\ mm - xu} = \frac{0.0023}{xu - 110\ mm}$$
$$eq2 := \frac{\varepsilon\_flange}{xu - 220\ mm} = \frac{\Delta\varepsilon\_p}{2815\ mm - xu}$$

$$eq3 := \Delta\sigma\_p = \frac{408.956522\ N}{mm^2} + \frac{6213.235294\ (\Delta\varepsilon\_p - 0.002092084728)\ N}{mm^2} \quad \textbf{(3)}$$

> $Answer := solve(\{eq1, eq2, eq3\}, \{\varepsilon\_flange, xu, \Delta\sigma\_p\});$

$$Answer := \left\{xu = \frac{5.\ mm\ (220000.\ \Delta\varepsilon\_p + 12949.)}{10000.\ \Delta\varepsilon\_p + 23.}, \Delta\sigma\_p = \frac{1.600000000\ 10^{-17}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p)}{mm^2}, \varepsilon\_flange = -0.04066543438\ \Delta\varepsilon\_p + 0.002206469501\right\} \quad \textbf{(4)}$$

> $xu := rhs(Answer[1]); \Delta\sigma\_p := rhs(Answer[2]); \varepsilon\_flange := rhs(Answer[3]);$

$$xu := \frac{5.\ mm\ (220000.\ \Delta\varepsilon\_p + 12949.)}{10000.\ \Delta\varepsilon\_p + 23.}$$

$$\Delta\sigma\_p := \frac{1.600000000\ 10^{-17}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p)}{mm^2}$$

$$\varepsilon\_flange := -0.04066543438\ \Delta\varepsilon\_p + 0.002206469501 \quad \textbf{(5)}$$

>

```
> #input: width beam 'b' and web width b_web, and equations of the
  forces:
```

> $b := 1000\ mm; b\_web := 150\ mm;$

$$b := 1000\ mm$$

$$b\_web := 150\ mm \quad \textbf{(6)}$$

> $Nc1 := b \cdot f\_cd \cdot 0.5 \cdot h\_fl;$
$Nc2 := b \cdot \varepsilon\_flange \cdot Ec \cdot 0.5 \cdot h\_fl + 0.5 \cdot b \cdot 0.5 \cdot h\_fl \cdot (f\_cd - \varepsilon\_flange \cdot Ec);$
$Nc3 := 2 \cdot 0.5 \cdot b\_web \cdot (xu - h\_fl) \cdot \varepsilon\_flange \cdot Ec;$
$Pm\_inf;$
$\Delta Np := Ap \cdot \Delta\sigma\_p;$

$$Nc1 := 6.6000000\ 10^{6}\ N$$

$$Nc2 := 2.869565217\ 10^{9}\ (-0.04066543438\ \Delta\varepsilon\_p + 0.002206469501)\ N + 55000.00\ mm^2 \left(-\frac{26086.95652\ N\ (-0.04066543438\ \Delta\varepsilon\_p + 0.002206469501)}{mm^2} + \frac{60\ N}{mm^2}\right)$$

$$Nc3 := \frac{1}{mm}\left(3.913043478\ 10^{6}\left(\frac{5.\ mm\ (220000.\ \Delta\varepsilon\_p + 12949.)}{10000.\ \Delta\varepsilon\_p + 23.} - 220\ mm\right)(-0.04066543438\ \Delta\varepsilon\_p + 0.002206469501)\ N\right)$$

$$2.30400000\ 10^{7}\ N$$

$$\Delta Np := 3.312000000\ 10^{-13}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p) \quad \textbf{(7)}$$

>

```
> #Horizontal equilibrium and determine the increase of strain in
```

pre-stressing steel:

> $eq4 := Nc1 + Nc2 + Nc3 = \Delta Np + Pm\_inf$

$$eq4 := 6.6000000\ 10^6\ N + 2.869565217\ 10^9\ (-0.04066543438\ \Delta\varepsilon\_p + 0.002206469501)\ N + 55000.00\ mm^2 \left(-\frac{26086.95652\ N\ (-0.04066543438\ \Delta\varepsilon\_p + 0.002206469501)}{mm^2} + \frac{60\ N}{mm^2}\right) + \frac{1}{mm}\left(3.913043478\ 10^6 \left(\frac{5.\ mm\ (220000.\ \Delta\varepsilon\_p + 12949.)}{10000.\ \Delta\varepsilon\_p + 23.} - 220\ mm\right)(-0.04066543438\ \Delta\varepsilon\_p + 0.002206469501)\ N\right) = 3.312000000\ 10^{-13}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p) + 2.30400000\ 10^7\ N \tag{8}$$

> $answer\_\Delta\varepsilon\_p := solve(eq4, \Delta\varepsilon\_p);$

$$answer\_\Delta\varepsilon\_p := 0.0004732647172, -0.1214485596 \tag{9}$$

> $\Delta\varepsilon\_p := answer\_\Delta\varepsilon\_p[1]; xu; Nc1; Nc2; Nc3; \Delta Np; Pm\_inf;$

$$\Delta\varepsilon\_p := 0.0004732647172$$

$$2353.384832\ mm$$

$$6.6000000\ 10^6\ N$$

$$6.438190936\ 10^6\ N$$

$$1.825900621\ 10^7\ N$$

$$8.257197138\ 10^6\ N$$

$$2.30400000\ 10^7\ N \tag{10}$$

>

> **#rotational capacity for C90/105:**

> $f := \frac{\left(\frac{1691\ N}{mm^2} - \sigma\_pinf\right) \cdot Ap}{Ap}\ \frac{mm^2}{N};\ \varepsilon\_cu3 := 0.0026;\ f2 := \frac{max\_height\_xu}{dp} \leq \frac{\varepsilon\_cu3 \cdot 10^6}{\varepsilon\_cu3 \cdot 10^6 + 7 \cdot f};$

$$f := 577.9565222$$

$$\varepsilon\_cu3 := 0.0026$$

$$f2 := \frac{max\_height\_xu}{2815\ mm} \leq 0.3912306755 \tag{11}$$

> $xu\_max := solve(f2, max\_height\_xu);$

$$xu\_max := \begin{cases} [\{1101.314352\ mm \leq max\_height\_xu\}] & mm < 0. \\ [\ ] & mm = 0. \\ [\{max\_height\_xu \leq 1101.314352\ mm\}] & 0. < mm \end{cases} \tag{12}$$

>

> **#xu must be equal or lower than xu_max:**

> $eq5 := xu \leq xu\_max;$

$$eq5 := 2353.384832\ mm \leq \begin{cases} [\{1101.314352\ mm \leq max\_height\_xu\}] & mm < 0. \\ [\ ] & mm = 0. \\ [\{max\_height\_xu \leq 1101.314352\ mm\}] & 0. < mm \end{cases} \tag{13}$$

```
>
> #xu is higher than maximal allowed. Therefore, design is not sufficient! No further calculations will be provided.
>
```

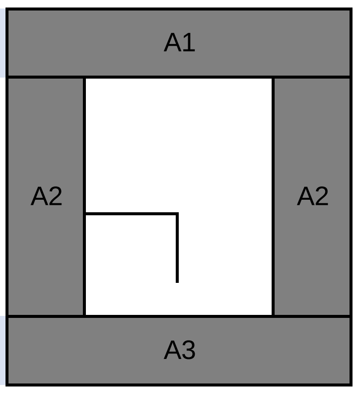
Appendix H Design calculation reconsidered design

This calculation contains the design approach without reducing the pre-stress force and the amount of surface area of the pre-stress tendons according to NEN-EN 1992-1-1+C2 chapter 6.1 and the accompanying National Annex chapter 6.1 sub (9). It is allowed to apply this reduction for the amount of reinforcement or pre-stressing steel which is required to resist the predefined load. Since this is the case, this reduction could be applied, because it is beneficial. However, it doesn't mean that the calculation is wrong. It is a less efficient calculation and therefore it's still attached in this Appendix.

Box beam design

SLS **Ap 20700 mm2**

h:				
3000	ht:	615		
	hi:	2015		1452
				1548
	hb:	370		



	150	700	150
	bw	bi	bw
b:	1000		

A tendon	150	mm2
Number of tendons	138	[-]
A1	615000	mm2
A2	302250	mm2
A3	370000	mm2
z tov top	1452	mm

INPUT IN MAPLE:

Atot	1,5895	m2
Izz	1,7728	m4
Wbot	1,1453	m3
Wtop	1,2207	m3
ep	1,3628	m
L	75	M
q_ligger_eg	41,327	kN/m
q_ligger_eq	30,2	kN/m

Econcrete	44000	MPa
Esteel	195000	MPa
Span L	75	m
rho C90/105	26	kN/m3
q eq	30,2	kN/m2
rho steel	78	kN/m3
G	3099,525	kN
Geinds	198,0342	kN
Gtot	3297,559	kN
Mg:=	29058	kNm
Mq:=	21234	kNm
Mgq:=	50292	kNm

z = (O2*K4*0,5*C4+2*O2*K5*(C4+0,5*C6)+O2*(K6-K2*K3)*(C4+C6+0,5*C13)+O3*K2*K3*(C4+C6+0,5*C13))/(O2*K4+2*O2*K5+O2*(K6-K2*K3)+O3*K2*K3)

SUM: EA*a/EA

z = 1452 mm

1452

Homogeneous material:

z = 1391

Density of both material:

z = 1427

> #Calculation xu, M[Rd]

> $restart;$

> **#input parameters C90/105, height flange=380 mm, dp=2815 mm on average:**

> $h\_fl := 615\,mm; dp := 2815\,mm; \varepsilon\_c3 := 0.0023; \varepsilon\_cu3 := 0.0026; f\_cd := \frac{60\,N}{mm^2}; Ec := \frac{f\_cd}{\varepsilon\_c3}; Ep := \frac{195000\,N}{mm^2};$

$$h\_fl := 615\,mm$$
$$dp := 2815\,mm$$
$$\varepsilon\_c3 := 0.0023$$
$$\varepsilon\_cu3 := 0.0026$$
$$f\_cd := \frac{60\,N}{mm^2}$$
$$Ec := \frac{26086.95652\,N}{mm^2}$$
$$Ep := \frac{195000\,N}{mm^2} \tag{1}$$

>

> **#Pm_inf is determined in initial calculation as well as Ap; #However, a reduction is applied of 80.8% according to NEN-EN 1992-1-1+C2/NB chapter 6.1 sub (9);**

> $Pm\_inf := 0.8 \cdot 28800000\,N; Ap := 138 \cdot 150\,mm^2; \sigma\_pinf := \frac{Pm\_inf}{Ap}; \varepsilon\_p := \Delta\varepsilon\_p + \frac{\sigma\_pinf}{Ep};$

$$Pm\_inf := 2.30400000\,10^7\,N$$
$$Ap := 20700\,mm^2$$
$$\sigma\_pinf := \frac{1113.043478\,N}{mm^2}$$
$$\varepsilon\_p := \Delta\varepsilon\_p + 0.005707915272 \tag{2}$$

>

> **#Delta epsilon p is the unknown. Strains are calculated as function of this unknown:**

> $eq1 := \frac{\Delta\varepsilon\_p}{dp - xu} = \frac{\varepsilon\_cu3}{xu};$

$eq2 := \frac{\Delta\varepsilon\_p}{dp - xu} = \frac{\varepsilon\_c3}{xu - h\_epsilon\_c3};$

$eq3 := \frac{\Delta\varepsilon\_p}{dp - xu} = \frac{\varepsilon\_flange}{xu - h\_fl};$

$eq4 := \Delta\sigma\_p = \left(\frac{1522\,N}{mm^2} + \left(\frac{(\varepsilon\_p - 7.8 \cdot 10^{-3})}{(35 \cdot 10^{-3} - 7.8 \cdot 10^{-3})}\right) \cdot \left(\frac{1691\,N}{mm^2} - \frac{1522\,N}{mm^2}\right)\right) - \sigma\_pinf;$

$$eq1 := \frac{\Delta\varepsilon\_p}{2815\ mm - xu} = \frac{0.0026}{xu}$$

$$eq2 := \frac{\Delta\varepsilon\_p}{2815\ mm - xu} = \frac{0.0023}{xu - h\_epsilon\_c3}$$

$$eq3 := \frac{\Delta\varepsilon\_p}{2815\ mm - xu} = \frac{\varepsilon\_flange}{xu - 615\ mm}$$

$$eq4 := \Delta\sigma\_p = \frac{408.956522\ N}{mm^2} + \frac{6213.235294\ (\Delta\varepsilon\_p - 0.002092084728)\ N}{mm^2} \tag{3}$$

> $Answer := solve(\{eq1, eq2, eq3, eq4, \}, \{xu, h\_epsilon\_c3, \Delta\sigma\_p, \varepsilon\_flange, \});$

$$Answer := \left\{ xu = \frac{36595.\ mm}{5000.\ \Delta\varepsilon\_p + 13.}, h\_epsilon\_c3 = \frac{4222.500000\ mm}{5000.\ \Delta\varepsilon\_p + 13.}, \Delta\sigma\_p = \frac{1.600000000\ 10^{-17}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p)}{mm^2}, \varepsilon\_flange = 0.002031971581 - 0.2184724689\ \Delta\varepsilon\_p \right\} \tag{4}$$

> $xu := rhs(Answer[1]); h\_epsilon\_c3 := rhs(Answer[2]); \Delta\sigma\_p := rhs(Answer[3]);$
> $\varepsilon\_flange := rhs(Answer[4]);$

$$xu := \frac{36595.\ mm}{5000.\ \Delta\varepsilon\_p + 13.}$$

$$h\_epsilon\_c3 := \frac{4222.500000\ mm}{5000.\ \Delta\varepsilon\_p + 13.}$$

$$\Delta\sigma\_p := \frac{1.600000000\ 10^{-17}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p)}{mm^2}$$

$$\varepsilon\_flange := 0.002031971581 - 0.2184724689\ \Delta\varepsilon\_p \tag{5}$$

>

> **#input: width beam 'b' and web width b_web, and equations of the forces:**

> $b := 1000\ mm; b\_web := 150\ mm;$

$$b := 1000\ mm$$

$$b\_web := 150\ mm \tag{6}$$

> $Nc1 := b \cdot f\_cd \cdot h\_epsilon\_c3;$
> $Nc2 := b \cdot \varepsilon\_flange \cdot Ec \cdot (h\_fl - h\_epsilon\_c3);$
> $Nc3 := 0.5 \cdot b \cdot (f\_cd - \varepsilon\_flange \cdot Ec) \cdot (h\_fl - h\_epsilon\_c3);$
> $Nc4 := 0.5 \cdot 2 \cdot b\_web \cdot \varepsilon\_flange \cdot Ec \cdot (xu - h\_fl);$
> $Pm\_inf;$
> $\Delta Np := Ap \cdot \Delta\sigma\_p;$

$$Nc1 := \frac{2.533500000\ 10^{8}\ N}{5000.\ \Delta\varepsilon\_p + 13.}$$

$$Nc2 := \frac{1}{mm} \Bigg(2.608695652\ 10^{7}\ (0.002031971581 - 0.2184724689\ \Delta\varepsilon\_p)\ N\ \Bigg(615\ mm$$

$$-\frac{4222.500000\,mm}{5000.\,\Delta\varepsilon\_p+13.}\Bigg)\Bigg)$$

$$Nc3 := 500.0\,mm\left(-\frac{26086.95652\,N\,(0.002031971581-0.2184724689\,\Delta\varepsilon\_p)}{mm^2}+\frac{60\,N}{mm^2}\right)\left(615\,mm-\frac{4222.500000\,mm}{5000.\,\Delta\varepsilon\_p+13.}\right)$$

$$Nc4 := \frac{1}{mm}\left(3.913043478\;10^6\,(0.002031971581-0.2184724689\,\Delta\varepsilon\_p)\,N\left(\frac{36595.\,mm}{5000.\,\Delta\varepsilon\_p+13.}-615\,mm\right)\right)$$

$$2.30400000\;10^7\,N$$

$$\Delta Np := 3.312000000\;10^{-13}\,N\,(2.474736921\;10^{19}+3.883272059\;10^{20}\,\Delta\varepsilon\_p) \quad \textbf{(7)}$$

\>

\> **#Horizontal equilibrium and determine the increase of strain in pre-stressing steel:**

\> $eq5 := Nc1+Nc2+Nc3+Nc4 = \Delta Np + Pm\_inf$

$$eq5 := \frac{2.533500000\;10^8\,N}{5000.\,\Delta\varepsilon\_p+13.}+\frac{1}{mm}\Bigg(2.608695652\;10^7\,(0.002031971581-0.2184724689\,\Delta\varepsilon\_p)\,N\left(615\,mm-\frac{4222.500000\,mm}{5000.\,\Delta\varepsilon\_p+13.}\right)\Bigg)+500.0\,mm\left(-\frac{26086.95652\,N\,(0.002031971581-0.2184724689\,\Delta\varepsilon\_p)}{mm^2}+\frac{60\,N}{mm^2}\right)\left(615\,mm-\frac{4222.500000\,mm}{5000.\,\Delta\varepsilon\_p+13.}\right)+\frac{1}{mm}\left(3.913043478\;10^6\,(0.002031971581-0.2184724689\,\Delta\varepsilon\_p)\,N\left(\frac{36595.\,mm}{5000.\,\Delta\varepsilon\_p+13.}-615\,mm\right)\right)=3.312000000\;10^{-13}\,N\,(2.474736921\;10^{19}+3.883272059\;10^{20}\,\Delta\varepsilon\_p)+2.30400000\;10^7\,N \quad \textbf{(8)}$$

\> $answer\_\Delta\varepsilon\_p := evalf(solve(eq5, \Delta\varepsilon\_p));$

$$answer\_\Delta\varepsilon\_p := 0.004044736203,\ -0.01050106020 \quad \textbf{(9)}$$

\> $\Delta\varepsilon\_p := answer\_\Delta\varepsilon\_p[1];\ xu;\ h\_epsilon\_c3;\ \Delta\sigma\_p;\ \varepsilon\_flange;\ \sigma\_pinf;$

$$\Delta\varepsilon\_p := 0.004044736203$$

$$1101.473373\,mm$$

$$127.0930815\,mm$$

$$\frac{421.0888051\,N}{mm^2}$$

$$0.001148308077$$

$$\frac{1113.043478\,N}{mm^2} \quad \textbf{(10)}$$

> $Nc1; Nc2; Nc3; Nc4; \Delta Np; Pm\_inf;$

$$7.625584890\ 10^{6}\ N$$
$$1.461567275\ 10^{7}\ N$$
$$7.329371180\ 10^{6}\ N$$
$$2.185909448\ 10^{6}\ N$$
$$8.716538266\ 10^{6}\ N$$
$$2.30400000\ 10^{7}\ N \tag{11}$$

>

> **#rotational capacity for C90/105:**

> $f := \frac{\left(\frac{1691\ N}{mm^{2}} - \sigma\_pinf\right) \cdot Ap}{Ap} \frac{mm^{2}}{N}; f2 := \frac{max\_height\_xu}{dp} \leq \frac{\varepsilon\_cu3 \cdot 10^{6}}{\varepsilon\_cu3 \cdot 10^{6} + 7 \cdot f};$

$$f := 577.9565222$$
$$f2 := \frac{max\_height\_xu}{2815\ mm} \leq 0.3912306755 \tag{12}$$

>

> $xu\_max := evalf[5](solve(f2, max\_height\_xu));$

$$xu\_max := \begin{cases} [\{1101.3\ mm \leq max\_height\_xu\}] & mm < 0. \\ [\ ] & mm = 0. \\ [\{max\_height\_xu \leq 1101.3\ mm\}] & 0. < mm \end{cases} \tag{13}$$

>

> **#xu must be equal or lower than xu_max:**

> $eq5 := xu \leq xu\_max;$

$$eq5 := 1101.473373\ mm \leq \begin{cases} [\{1101.3\ mm \leq max\_height\_xu\}] & mm < 0. \\ [\ ] & mm = 0. \\ [\{max\_height\_xu \leq 1101.3\ mm\}] & 0. < mm \end{cases} \tag{14}$$

>

>

> **#Bending moment capacity:SOM|delta_Np: Yellow hatched parts manual input via Excelsheet**

> **#z_top is the distance from top fibre compression zone to neutral axis, calculated via Excelsheet:**

> $z\_top := 1452\ mm;$

$$z\_top := 1452\ mm \tag{15}$$

> $M[Rd] := evalf[5]\Big(\Big(Nc1 \cdot (dp - 0.5 \cdot h\_epsilon\_c3) + Nc2 \cdot (dp - (h\_epsilon\_c3 + 0.5 \cdot (h\_fl - h\_epsilon\_c3))) + Nc3 \cdot \left(dp - \left(h\_epsilon\_c3 + \frac{1}{3} \cdot (h\_fl - h\_epsilon\_c3)\right)\right) + Nc4 \cdot \left(dp - \left(h\_fl + \frac{1}{3} \cdot (xu - h\_fl)\right)\right) - Pm\_inf \cdot (dp - z\_top)\Big)\Big);$

$$M_{Rd} := 4.8265\ 10^{10}\ N\ mm \tag{16}$$

> $M[Eg, beam] := \frac{1}{8} \cdot \frac{41.327\,N}{mm} \cdot (75000\,mm)^2; M[Eg, park] := 0.125 \cdot \frac{27\,N}{mm} \cdot (75000\,mm)^2;$
> $M[var] := \frac{1}{8} \cdot \frac{3.2\,N}{mm} \cdot (75000\,mm)^2;$

$$M_{Eg,\ beam} := 2.905804688\ 10^{10}\ N\,mm$$

$$M_{Eg,\ park} := 1.898437500\ 10^{10}\ N\,mm$$

$$M_{var} := 2.250000000\ 10^{9}\ N\,mm \tag{17}$$

>

> **#M[Ed] including safety factors as described in the Preliminary Study Appendix D:**

> $M[Ed] := evalf[5](1.4 \cdot (M[Eg, beam] + M[Eg, park]) + 0.8 \cdot 1.5 \cdot M[var] - 1.0 \cdot Pm\_inf \cdot (dp - z\_top));$

$$M_{Ed} := 3.8555\ 10^{10}\ N\,mm \tag{18}$$

> $Eq6 := M[Ed] < M[Rd];$

$$Eq6 := 3.8555\ 10^{10}\ N\,mm < 4.8265\ 10^{10}\ N\,mm \tag{19}$$

> $Unity\_Check := evalf[3]\left(\frac{(M[Ed])}{M[Rd]}\right);$

$$Unity\_Check := 0.799 \tag{20}$$

>

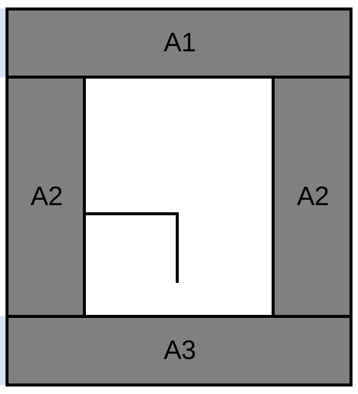
Appendix I Design calculation with applied reduction

The next calculation contains the same design procedure as the previous appendix. However, in this calculation a reduction is applied to the pre-stress force and the surface area of the pre-stressing steel.

Box beam design

SLS **Ap** **20700** **mm2**

h:	3000	
ht:	380	
hi:	2250	
hb:	370	



1557

1443

150	700	150
bw	bi	bw

b: 1000

A tendon	150	mm2
Number of tendons	138	[-]
A1	380000	mm2
A2	337500	mm2
A3	370000	mm2
z tov top	1557	mm

Econcrete	44000	MPa
Esteel	195000	MPa
Span L	75	m
rho C90/105	26	kN/m3
q eq	30,2	kN/m2
rho steel	78	kN/m3
G	2778,75	kN
Geinds	245,7	kN
Gtot	3024,45	kN
Mg:=	26051	kNm
Mq:=	21234	kNm
Mgq:=	47285	kNm

INPUT IN MAPLE:

Atot	1,425	m2
Izz	1,5855	m4
Wbot	1,0989	m3
Wtop	1,0182	m3
ep	1,2578	m
L	75	M
q_ligger_eg	37,05	kN/m
q_ligger_eq	30,2	kN/m

z = (O2*K4*0,5*C4+2*O2*K5*(C4+0,5*C6)+O2*(K6-K2*K3)*(C4+C6+0,5*C13)+O3*K2*K3*(C4+C6+0,5*C13))/(O2*K4+2*O2*K5+O2*(K6-K2*K3)+O3*K2*K3)

SUM: EA*a/EA

z = 1557 mm

1557

Homogeneous material:

z = 1494

Density of both material:

z = 1532

> #Calculation xu, M[Rd]

> *restart*;

> #input parameters C90/105, height flange=380 mm, dp=2815 mm on average:

> $h\_fl := 380\ mm$; $dp := 2815\ mm$; $\varepsilon\_c3 := 0.0023$; $\varepsilon\_cu3 := 0.0026$; $f\_cd := \frac{60\ N}{mm^2}$; $Ec := \frac{f\_cd}{\varepsilon\_c3}$; $Ep := \frac{195000\ N}{mm^2}$;

$$h\_fl := 380\ mm$$
$$dp := 2815\ mm$$
$$\varepsilon\_c3 := 0.0023$$
$$\varepsilon\_cu3 := 0.0026$$
$$f\_cd := \frac{60\ N}{mm^2}$$
$$Ec := \frac{26086.95652\ N}{mm^2}$$
$$Ep := \frac{195000\ N}{mm^2} \qquad (1)$$

>

> #Pm_inf is determined in initial calculation as well as Ap; #However, a reduction is applied of 80.8% according to NEN-EN 1992-1-1+C2/NB chapter 6.1 sub (9);

> $Pm\_inf := 0.808 \cdot 0.8 \cdot 28800000\ N$; $Ap := 0.808 \cdot 138 \cdot 150\ mm^2$; $\sigma\_pinf := \frac{Pm\_inf}{Ap}$; $\varepsilon\_p := \Delta\varepsilon\_p + \frac{\sigma\_pinf}{Ep}$;

$$Pm\_inf := 1.861632000\ 10^7\ N$$
$$Ap := 16725.600\ mm^2$$
$$\sigma\_pinf := \frac{1113.043478\ N}{mm^2}$$
$$\varepsilon\_p := \Delta\varepsilon\_p + 0.005707915272 \qquad (2)$$

>

> #Delta epsilon p is the unknown. Strains are calculated as function of this unknown:

> $eq1 := \frac{\Delta\varepsilon\_p}{dp - xu} = \frac{\varepsilon\_cu3}{xu}$;

$eq2 := \frac{\Delta\varepsilon\_p}{dp - xu} = \frac{\varepsilon\_c3}{xu - h\_epsilon\_c3}$;

$eq3 := \frac{\Delta\varepsilon\_p}{dp - xu} = \frac{\varepsilon\_flange}{xu - h\_fl}$;

$eq4 := \Delta\sigma\_p = \left(\frac{1522\ N}{mm^2} + \left(\frac{(\varepsilon\_p - 7.8 \cdot 10^{-3})}{(35 \cdot 10^{-3} - 7.8 \cdot 10^{-3})}\right) \cdot \left(\frac{1691\ N}{mm^2} - \frac{1522\ N}{mm^2}\right)\right) - \sigma\_pinf$;

$$eq1 := \frac{\Delta\varepsilon\_p}{2815\ mm - xu} = \frac{0.0026}{xu}$$

$$eq2 := \frac{\Delta\varepsilon\_p}{2815\ mm - xu} = \frac{0.0023}{xu - h\_epsilon\_c3}$$

$$eq3 := \frac{\Delta\varepsilon\_p}{2815\ mm - xu} = \frac{\varepsilon\_flange}{xu - 380\ mm}$$

$$eq4 := \Delta\sigma\_p = \frac{408.956522\ N}{mm^2} + \frac{6213.235294\ (\Delta\varepsilon\_p - 0.002092084728)\ N}{mm^2} \tag{3}$$

> $Answer := solve(\{eq1, eq2, eq3, eq4, \}, \{xu, h\_epsilon\_c3, \Delta\sigma\_p, \varepsilon\_flange, \});$

$$Answer := \left\{xu = \frac{36595.\ mm}{5000.\ \Delta\varepsilon\_p + 13.}, h\_epsilon\_c3 = \frac{4222.500000\ mm}{5000.\ \Delta\varepsilon\_p + 13.}, \Delta\sigma\_p = \frac{1.600000000\ 10^{-17}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p)}{mm^2}, \varepsilon\_flange = 0.002249023091 - 0.1349911190\ \Delta\varepsilon\_p\right\} \tag{4}$$

> $xu := rhs(Answer[1]); h\_epsilon\_c3 := rhs(Answer[2]); \Delta\sigma\_p := rhs(Answer[3]);$
> $\varepsilon\_flange := rhs(Answer[4]);$

$$xu := \frac{36595.\ mm}{5000.\ \Delta\varepsilon\_p + 13.}$$

$$h\_epsilon\_c3 := \frac{4222.500000\ mm}{5000.\ \Delta\varepsilon\_p + 13.}$$

$$\Delta\sigma\_p := \frac{1.600000000\ 10^{-17}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p)}{mm^2}$$

$$\varepsilon\_flange := 0.002249023091 - 0.1349911190\ \Delta\varepsilon\_p \tag{5}$$

>

> **#input: width beam 'b' and web width b_web, and equations of the forces:**

> $b := 1000\ mm; b\_web := 150\ mm;$

$$b := 1000\ mm$$

$$b\_web := 150\ mm \tag{6}$$

> $Nc1 := b \cdot f\_cd \cdot h\_epsilon\_c3;$
> $Nc2 := b \cdot \varepsilon\_flange \cdot Ec \cdot (h\_fl - h\_epsilon\_c3);$
> $Nc3 := 0.5 \cdot b \cdot (f\_cd - \varepsilon\_flange \cdot Ec) \cdot (h\_fl - h\_epsilon\_c3);$
> $Nc4 := 0.5 \cdot 2 \cdot b\_web \cdot \varepsilon\_flange \cdot Ec \cdot (xu - h\_fl);$
> $Pm\_inf;$
> $\Delta Np := Ap \cdot \Delta\sigma\_p;$

$$Nc1 := \frac{2.533500000\ 10^{8}\ N}{5000.\ \Delta\varepsilon\_p + 13.}$$

$$Nc2 := \frac{1}{mm}\Big(2.608695652\ 10^{7}\ (0.002249023091 - 0.1349911190\ \Delta\varepsilon\_p)\ N\ \Big(380\ mm$$

$$- \frac{4222.500000\ mm}{5000.\ \Delta\varepsilon\_p + 13.} \Big) \Big)$$

$$Nc3 := 500.0\ mm \left(- \frac{26086.95652\ N\ (0.002249023091 - 0.1349911190\ \Delta\varepsilon\_p)}{mm^2} + \frac{60\ N}{mm^2} \right) \left(380\ mm - \frac{4222.500000\ mm}{5000.\ \Delta\varepsilon\_p + 13.} \right)$$

$$Nc4 := \frac{1}{mm} \left(3.913043478\ 10^6\ (0.002249023091 - 0.1349911190\ \Delta\varepsilon\_p)\ N \left(\frac{36595.\ mm}{5000.\ \Delta\varepsilon\_p + 13.} - 380\ mm \right) \right)$$

$$1.861632000\ 10^7\ N$$

$$\Delta Np := 2.676096000\ 10^{-13}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p) \qquad \textbf{(7)}$$

```
>
> #Horizontal equilibrium and determine the increase of strain in
  pre-stressing steel:
```

> $eq5 := Nc1 + Nc2 + Nc3 + Nc4 = \Delta Np + Pm\_inf$

$$eq5 := \frac{2.533500000\ 10^8\ N}{5000.\ \Delta\varepsilon\_p + 13.} + \frac{1}{mm} \left(2.608695652\ 10^7\ (0.002249023091 - 0.1349911190\ \Delta\varepsilon\_p)\ N \left(380\ mm - \frac{4222.500000\ mm}{5000.\ \Delta\varepsilon\_p + 13.} \right) \right) + 500.0\ mm \left(- \frac{26086.95652\ N\ (0.002249023091 - 0.1349911190\ \Delta\varepsilon\_p)}{mm^2} + \frac{60\ N}{mm^2} \right) \left(380\ mm - \frac{4222.500000\ mm}{5000.\ \Delta\varepsilon\_p + 13.} \right) + \frac{1}{mm} \left(3.913043478\ 10^6\ (0.002249023091 - 0.1349911190\ \Delta\varepsilon\_p)\ N \left(\frac{36595.\ mm}{5000.\ \Delta\varepsilon\_p + 13.} - 380\ mm \right) \right) = 2.676096000\ 10^{-13}\ N\ (2.474736921\ 10^{19} + 3.883272059\ 10^{20}\ \Delta\varepsilon\_p) + 1.861632000\ 10^7\ N \qquad \textbf{(8)}$$

> $answer\_\Delta\varepsilon\_p := evalf(solve(eq5, \Delta\varepsilon\_p));$

$$answer\_\Delta\varepsilon\_p := 0.004035266866,\ -0.02133950815 \qquad \textbf{(9)}$$

> $\Delta\varepsilon\_p := answer\_\Delta\varepsilon\_p[1];\ xu;\ h\_epsilon\_c3;\ \Delta\sigma\_p;\ \varepsilon\_flange;\ \sigma\_pinf;$

$$\Delta\varepsilon\_p := 0.004035266866$$

$$1103.045311\ mm$$

$$127.2744589\ mm$$

$$\frac{421.0299699\ N}{mm^2}$$

$$0.001704297901$$

$$\frac{1113.043478\ N}{mm^2} \qquad \textbf{(10)}$$

> $Nc1; Nc2; Nc3; Nc4; \Delta Np; Pm\_inf;$

$$7.636467535\ 10^{6}\ N$$
$$1.123616372\ 10^{7}\ N$$
$$1.963684374\ 10^{6}\ N$$
$$4.821983241\ 10^{6}\ N$$
$$7.041978865\ 10^{6}\ N$$
$$1.861632000\ 10^{7}\ N \quad \textbf{(11)}$$

>

> #rotational capacity for C90/105:

> $f := \frac{\left(\frac{1691\ N}{mm^2} - \sigma\_pinf\right) \cdot Ap}{Ap} \frac{mm^2}{N}; f2 := \frac{max\_height\_xu}{dp} \leq \frac{\varepsilon\_cu3 \cdot 10^6}{\varepsilon\_cu3 \cdot 10^6 + 7 \cdot f};$

$$f := 577.9565219$$
$$f2 := \frac{max\_height\_xu}{2815\ mm} \leq 0.3912306756 \quad \textbf{(12)}$$

>

> $xu\_max := evalf[5](solve(f2, max\_height\_xu));$

$$xu\_max := \begin{cases} [\{1101.3\ mm \leq max\_height\_xu\}] & mm < 0. \\ [\ ] & mm = 0. \\ [\{max\_height\_xu \leq 1101.3\ mm\}] & 0. < mm \end{cases} \quad \textbf{(13)}$$

>

> #xu must be equal or lower than xu_max:

> $eq5 := xu \leq xu\_max;$

$$eq5 := 1103.045311\ mm \leq \begin{cases} [\{1101.3\ mm \leq max\_height\_xu\}] & mm < 0. \\ [\ ] & mm = 0. \\ [\{max\_height\_xu \leq 1101.3\ mm\}] & 0. < mm \end{cases} \quad \textbf{(14)}$$

>

>

> #Bending moment capacity:SOM|delta_Np: Yellow hatched parts manual input via Excelsheet

> #z_top is the distance from top fibre compression zone to neutral axis, calculated via Excelsheet:

> $z\_top := 1557\ mm;$

$$z\_top := 1557\ mm \quad \textbf{(15)}$$

> $M[Rd] := evalf[5]\Big(\Big(Nc1 \cdot (dp - 0.5 \cdot h\_epsilon\_c3) + Nc2 \cdot (dp - (h\_epsilon\_c3 + 0.5 \cdot (h\_fl - h\_epsilon\_c3))) + Nc3 \cdot \left(dp - \left(h\_epsilon\_c3 + \frac{1}{3} \cdot (h\_fl - h\_epsilon\_c3)\right)\right) + Nc4 \cdot \left(dp - \left(h\_fl + \frac{1}{3} \cdot (xu - h\_fl)\right)\right) - Pm\_inf \cdot (dp - z\_top)\Big)\Big);$

$$M_{Rd} := 4.2064\ 10^{10}\ N\ mm \quad \textbf{(16)}$$

> $M[Eg, beam] := \frac{1}{8} \cdot \frac{37.05\ N}{mm} \cdot (75000\ mm)^2$; $M[Eg, park] := 0.125 \cdot \frac{27\ N}{mm} \cdot (75000\ mm)^2$;
$M[var] := \frac{1}{8} \cdot \frac{3.2\ N}{mm} \cdot (75000\ mm)^2$;

$$M_{Eg,\ beam} := 2.605078125\ 10^{10}\ N\ mm$$

$$M_{Eg,\ park} := 1.898437500\ 10^{10}\ N\ mm$$

$$M_{var} := 2.250000000\ 10^{9}\ N\ mm \quad \textbf{(17)}$$

>

> #M[Ed] including safety factors as described in the Preliminary Study Appendix D:

> $M[Ed] := evalf[5](1.4 \cdot (M[Eg, beam] + M[Eg, park]) + 0.8 \cdot 1.5 \cdot M[var] - 1.0 \cdot Pm\_inf \cdot (dp - z\_top))$;

$$M_{Ed} := 4.2330\ 10^{10}\ N\ mm \quad \textbf{(18)}$$

> $Eq6 := M[Ed] < M[Rd]$;

$$Eq6 := 4.2330\ 10^{10}\ N\ mm < 4.2064\ 10^{10}\ N\ mm \quad \textbf{(19)}$$

> $Unity\_Check := evalf[3]\left(\frac{(M[Ed])}{M[Rd]}\right)$;

$$Unity\_Check := 1.00 \quad \textbf{(20)}$$

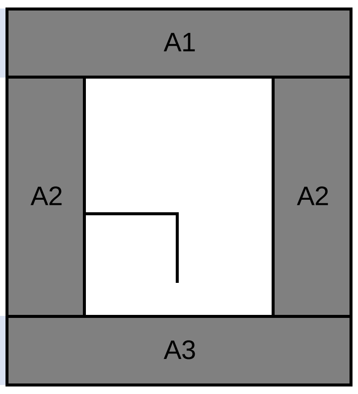
>

Appendix J Shear resistance

Box beam design

SLS **Ap** **20700** **mm2**

h:		
3000	ht:	380
	hi:	2250
	hb:	370



1557

1443

150	700	150
bw	bi	bw

b: 1000

A tendon	150	mm2
Number of tendons	138	[-]
A1	380000	mm2
A2	337500	mm2
A3	370000	mm2
z tov top	1557	mm

INPUT IN MAPLE:

Atot	1,425	m2
Izz	1,5855	m4
Wbot	1,0989	m3
Wtop	1,0182	m3
ep	1,2578	m
L	75	M
q_ligger_eg	37,05	kN/m
q_ligger_eq	30,2	kN/m

Econcrete	44000	MPa
Esteel	195000	MPa
Span L	75	m
rho C90/105	26	kN/m3
q eq	30,2	kN/m2
rho steel	78	kN/m3
G	2778,75	kN
Geinds	245,7	kN
Gtot	3024,45	kN
Mg:=	26051	kNm
Mq:=	21234	kNm
Mgq:=	47285	kNm

z = (O2*K4*0,5*C4+2*O2*K5*(C4+0,5*C6)+O2*(K6-K2*K3)*(C4+C6+0,5*C13)+O3*K2*K3*(C4+C6+0,5*C13))/(O2*K4+2*O2*K5+O2*(K6-K2*K3)+O3*K2*K3)

SUM: EA*a/EA

z = 1557 mm

1557

Homogeneous material:

z = 1494

Density of both material:

z = 1532

> #Shear check calculation

> #This calculation is performed to determine the shear realiability of the beam.
#The shear force is at its largest near the supports.
#Therefore, checks are performed there.

> #Formula determined via results bending moment calculation.
#The internal lever arm z is determined via this method.

> $Centre\_of\_gravity\_Nc := evalf[5]\left(\frac{1}{Nc1+Nc2+Nc3+Nc4}\left(Nc1\cdot 0.5\cdot h\_epsilon\_c3+Nc2\cdot(h\_epsilon\_c3+0.5\cdot(h\_fl-h\_epsilon\_c3))+Nc3\cdot\left(h\_epsilon\_c3+\frac{1}{3}\cdot(h\_fl-h\_epsilon\_c3)\right)+Nc4\cdot\left(h\_fl+\frac{1}{3}\cdot(xu-h\_fl)\right)\right)\right);$

$$Centre\_of\_gravity\_Nc := 262.89\ mm \quad (21)$$

>

> $z := dp - Centre\_of\_gravity\_Nc;$

$$z := 2552.11\ mm \quad (22)$$

> $V[Ed,1] := 1.4\cdot 0.5\cdot \frac{37.05\ N}{mm}\cdot 75000\ mm + 1.4\cdot 0.5\cdot\frac{27\ N}{mm}\cdot 75000\ mm + 1.2\cdot 0.5\cdot\frac{3.2\ N}{mm}\cdot 75000\ mm;$

$$V_{Ed,1} := 3.506625000\ 10^{6}\ N \quad (23)$$

> $\nu[\min] := \frac{0.6\cdot\left(1-\frac{90}{250}\right)N}{mm^2};\ k[1] := 0.15;\ \sigma[cp] := 0.2\cdot f\_cd;$

$$\nu_{\min} := \frac{0.3840000000\ N}{mm^2}$$

$$k_1 := 0.15$$

$$\sigma_{cp} := \frac{12.0\ N}{mm^2} \quad (24)$$

> $V[Rd,c] := (\nu[\min]+k[1]\cdot\sigma[cp])\cdot 2\cdot b\cdot dp;$

$$V_{Rd,c} := 1.229592000\ 10^{7}\ N \quad (25)$$

> $UC[Shear] := \frac{(V[Ed,1])}{V[Rd,c]}$

$$UC_{Shear} := 0.2851860617 \quad (26)$$

> #Solid part beam (first three meters at support) sufficient without stirrups.
#No stirrups not allowed, so apply practical (minimum) amount of stirrups.

>

> #Determine V_Ed at 3 meters from support, because hollow core part starts:

> $V[gradient] := \frac{(V[Ed,1])}{37.5\ m};\ V[Ed] := 34.5\ m\cdot V[gradient]$

$$V_{gradient} := \frac{93510.00001\ N}{m}$$

$$V_{Ed} := 3.226095000\ 10^6\ N \tag{27}$$

> $V[Rd, c] := (\nu[\min] + k[1] \cdot \sigma[cp]) \cdot 2 \cdot b\_web \cdot dp;$

$$V_{Rd,\ c} := 1.844388000\ 10^6\ N \tag{28}$$

> $UC[Shear] := \frac{(V[Ed])}{V[Rd, c]};$

$$UC_{Shear} := 1.749141179 \tag{29}$$

```
> #Unity check too high -> apply stirrups:
  #Input: theta conservative = 45 degrees and 150 mm c.t.c.
  distance for instance:
```

> theta := $evalf(convert(45\ degrees, radians))$; $s\_stirrup := 150\ mm$;

$$\theta := 0.7853981635$$

$$s\_stirrup := 150\ mm \tag{30}$$

> $V[Rd, s] := \frac{A[sw]}{s\_stirrup} \cdot \frac{z \cdot 435\ N}{mm^2} \cdot \cot(\text{theta});$

$$V_{Rd,\ s} := \frac{7401.119001\ A_{sw}\ N}{mm^2} \tag{31}$$

> $eq7 := V[Rd, s] = V[Ed]$; $A\_sw := solve(eq7, A[sw])$

$$eq7 := \frac{7401.119001\ A_{sw}\ N}{mm^2} = 3.226095000\ 10^6\ N$$

$$A\_sw := 435.8928697\ mm^2 \tag{32}$$

```
> #Two webs per cross-section, 1 stirrup per web implies 4 stirrup
  bar cross-sections:
```

> $A[sw, \text{phi}] = \frac{A\_sw}{4};$

$$A_{sw,\ \phi} = 108.9732174\ mm^2 \tag{33}$$

```
> #Choose stirrups diameter 12:
```

> $A[sw, applied] := 4 \cdot (6\ mm)^2 \cdot evalf(\text{Pi});$

$$A_{sw,\ applied} := 452.3893420\ mm^2 \tag{34}$$

> $eq8 := V[Ed] = \frac{A[sw, applied]}{s\_stirrupp} \cdot \frac{z \cdot 435\ N}{mm^2} \cdot \cot(\text{theta}); s[stirrup, c \bullet t \bullet c \bullet distance] :=$
> $solve(eq8, s\_stirrupp);$

$$eq8 := 3.226095000\ 10^6\ N = \frac{5.022281033\ 10^8\ mm\ N}{s\_stirrupp}$$

$$s_{stirrup,\ c \bullet t \bullet c \bullet distance} := 155.6767867\ mm \tag{35}$$

>

```
> #horizontal spacing is maximum 155 mm. Therefore, stirrups
  diameter 12-150 mm is adequate.
```

> **#Tensile tie check;**
#At 3 meters from support, 138 tendons in total, 26 are kinked, 60 should be unbonded at the support,
#At 3 meters from support, assumed that 40 tendons are unbonded. Thus:

> $number\_of\_tendons\_tensile\_check := 138 - 26 - 40; Ap\_one\_tendon := 150\ mm^2;$

$$number\_of\_tendons\_tensile\_check := 72$$

$$Ap\_one\_tendon := 150\ mm^2 \tag{36}$$

> **#rest capacity prestressing steel, the tensile tie;**
#f_pd=f_p,0.1k*gamma_s=0.9*1860/1.1 according to NEN-EN 1992-1-1+C2:2011 chapter 3.3.6 sub (7);

> $f[pd] := evalf[4]\left(\frac{\left(\frac{0.9 \cdot 1860\ N}{mm^2}\right)}{1.1}\right); evalf[4](\sigma\_pinf);$

$$f_{pd} := \frac{1522.\ N}{mm^2}$$

$$\frac{1113.\ N}{mm^2} \tag{37}$$

> **#Due to kink in 26 tendons, vertical component upwards, reducing V[Ed];**
#However, without reducing V[Ed], the Unity check is already sufficient;
#Therefore, tensile tie is sufficient!

> $Unity\_Check\_V[Ed] :=$
$evalf[3]\left(\frac{V[Ed]}{(number\_of\_tendons\_tensile\_check \cdot Ap\_one\_tendon \cdot (f[pd] - \sigma\_pinf))}\right);$

$$Unity\_Check\_V_{Ed} := 0.729 \tag{38}$$

>