Feasibility of incorporating the concept of demountable (modular) construction to an existing integral bridge design

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Declaration

I, Harikrishnan Sushin declare that this additional thesis titled 'Feasibility of incorporating the concept of demountable construction to an existing integral bridge design' is based on my work throughout the summer and the month of September 2020 under the supervision of Dr. ir. Herbert van der Ham. This report is done to fulfill the criteria of Additional Graduation Work (CIE5050-09) to the Delft University of Technology. The work done has used references from various sources and have been given due credit by adding these sources in the references.

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Abstract

Adopt the concept of demountable and rebuildable brides to an existing idea of an integral bridge. An understanding of the different forces and moments that the bridge will have to take up and how the critical connections (deck-abutment) can be realized. Applying the understanding of how integral and demountable bridges behave and try to integrate the characteristics of both to achieve rebuildable/demountable construction. In this additional thesis, the first steps to make an integral bridge demountable is looked at in detail. The concepts of vertical prestressing with unbonded bars are explored to achieve the demountable abutment, and abutment- deck connection. The different forces that arise due to this method of prestressing have been accounted for to ensure the basic safety of this design is met.

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1.Introduction

1.1. Motivation

Most of the bridges built in the Netherlands are coming to their end of service life and will need to be demolished and deposited in a landfill. There is a pressing need to reduce this waste of materials (steel and concrete) and one way to do this is to start building more circularly. Without going too much into detail about sustainable and circular construction, it can be said that the goal is to move from a cradle-to-grave construction process to a cradle-to-cradle construction process. This additional thesis will also focus on the individual precast components that allow the construction to become demountable. Some terms are used repeatedly and to avoid any confusion, the definitions of these terms are given in the next section.

1.2. Definitions

Some definitions that need to be understood before we proceed are:

1. Sustainability: Roughly put, it means development in which the needs of the present are met without compromising the ability of future generations to meet their own needs.
2. Circular construction: A construction where cradle-to-cradle is the path that the life cycle of a structure follows. Instead of the last step being demolition and landfilling.
3. Integral bridges: A bridge without movable or expansion joints[1].
4. Accelerated Bridge Construction (ABC): A practice that gives detailed methods on how to construct bridges at a much quicker rate, making use of more precast elements and less cast in place concrete.

1.3. Objective

The objective of this additional thesis is to understand how the concept of circular construction can be applied to an integral bridge (Global scope). For this thesis, the N69, KW13 Luikerweg Viaduct in the Netherlands is studied. It is an integral bridge with an overall span of 39.3m and a deck width of 19.3m. The deck is designed to be cast with C45/55 concrete grade and the abutments of C35/45[2].

1.4. Research Question

As there aren't any prototypes or existing structures that combine the concepts of integral and demountable bridges, the primary research question becomes: **Would it be possible to combine the model of an integral bridge with that of a demountable one in a practical manner such that it facilitates circular construction?**

1.4.1. Sub Questions

- What is an integral bridge?
- What is a demountable bridge?
- What are the critical connections within the bridge to try and come up with a demountable solution?
- What are the boundary conditions that exist in the current design?

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[1] A more detailed explanation is given in section 2.1
[2] More details about this can be found in section 3.1

1.5. Report Outline

This report will have four chapters with the first chapter focusing on the objective at hand and the second chapter focusing on the literature review done to better understand the concept of integral bridges before trying to come up with a solution. At the end of this report, all the relevant research papers, textbooks, articles, etc. that were used in this additional thesis are mentioned. The third chapter will touch upon the critical connections in the bridge. For this, documents from Boskalis [3] have been referred to. The fourth chapter will try to showcase a practical and realistic solution to overcome some of the shortcomings of the integral bridge.

2. Literature Review

2.1. Integral bridges

The term integral bridge now refers to single or multiple span bridges without expansion joints. The choice to have no expansion joints comes with several advantages and a few disadvantages as well. Some of the main advantages when it comes to integral bridges are the reduction in maintenance costs of the bridge once it is up and running as well as the option of a slender design due to the clamping effect.

2.1.1 Stufib report 25

The Stufib Report 25 [1] guides engineers and designers through the process of modeling integral bridges using prefab elements. The connection between the deck and the substructure is one to be considered at an early stage and this will be considered in this additional thesis[3]. There has been a shift in the construction of bridges towards a more integral type especially for small spans (approximately 10 meters). Integral bridges can have low as well as high based abutments. The bridge being inspected in this additional thesis is a low based abutment.

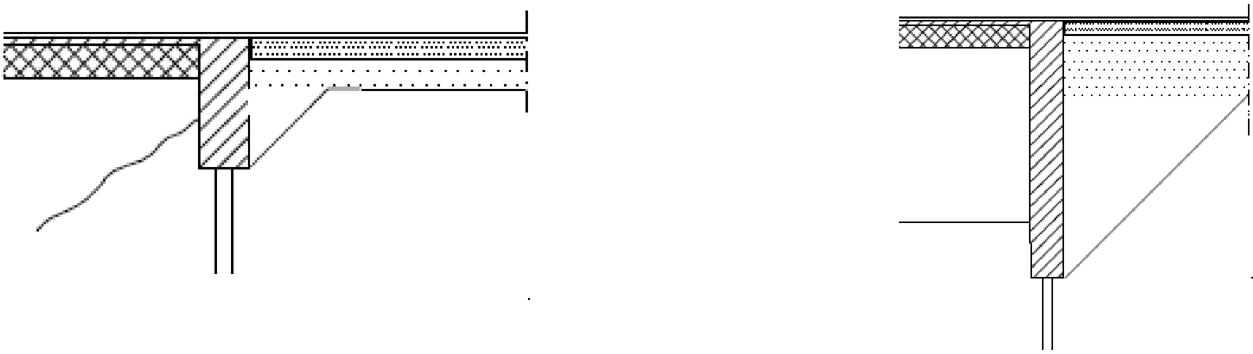


Figure 2.1-High based abutment (left) and low based abutment (right). Source: Stufib Report 25

Advantages of integral bridges

- Lesser maintenance costs due to lack of expansion joints
- No leakage due to expansions joints
- Slimmer deck and more robust design

Disadvantages of integral bridges

- Limited lengths
- Lack of expansion joints due to which deformations are transferred to the approach slab
- More complex designs and calculations

The lack of expansion joint features as an advantage as well as a disadvantage. This is because once the bridge is in use no maintenance is required for the expansion joints (as they do not exist). However, the lack of expansion joints also transfers the cyclic expansion and contraction forces of the bridge to the approach slab warranting a better design for the approach slab.

The Stufib reports talk about the various participants in the design process, their roles, and the organization of the design process. This is not going to be elaborated on in this additional thesis.

[3] Will be discussed further in Chapter 3

Like a conventional bridge (non-integral), the loads encountered are the same, self-weight, pretension, traffic loads, and time-dependent (creep, shrinkage). The horizontal forces (secondary forces) arise mainly due to daily and seasonal temperature variation as well as shrinkage, creep, earth pressure. These forces are directly transferred via the abutments to the foundation. The vertical loads of self-weight, traffic, asphalt, etc. are transferred via the abutments and will be taken as axial compressive loads (if piles are used) or as a load that the soil below the foundation must withstand (bearing capacity of soil). The interaction between the abutment and the soil is also an important aspect to keep in mind especially for integral connections. A clamping effect arises at the ends of the bridge due to the monolithic nature of the connection between the deck and the substructure. To treat this clamping effect, the ground pressure is taken as a load as well as a resistance. This additional thesis does not go into detail about this.

2.1.2 Attributes and Limitations

The publication by Burke [2] talks about the different attributes and some of the limitations constructors and designers can face when tackling the construction of an integral bridge.

Although integral structures have been around for centuries, the term integral bridge is used to refer to continuous jointless bridges with single and multiple spans with capped-pile stub-type abutments. (Burke, 1993)

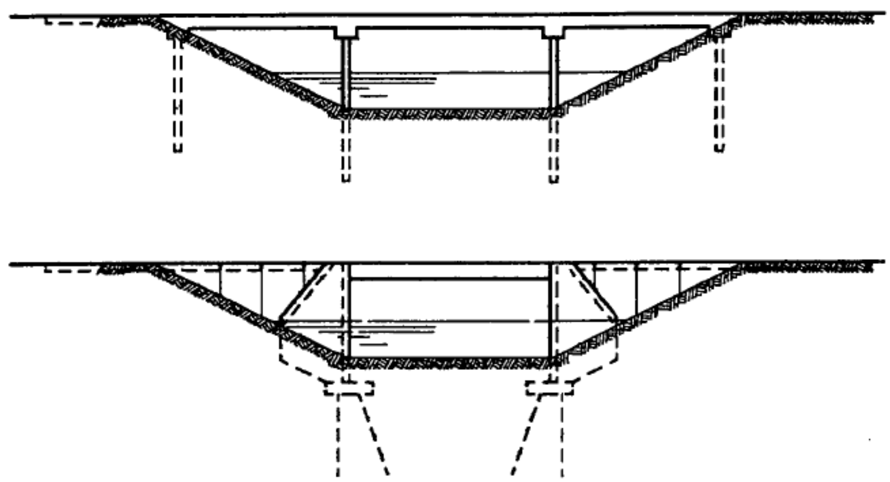


Figure 2.2-Integral bridges with single span (bottom) and multiple spans (top). Source: Burke (1993)

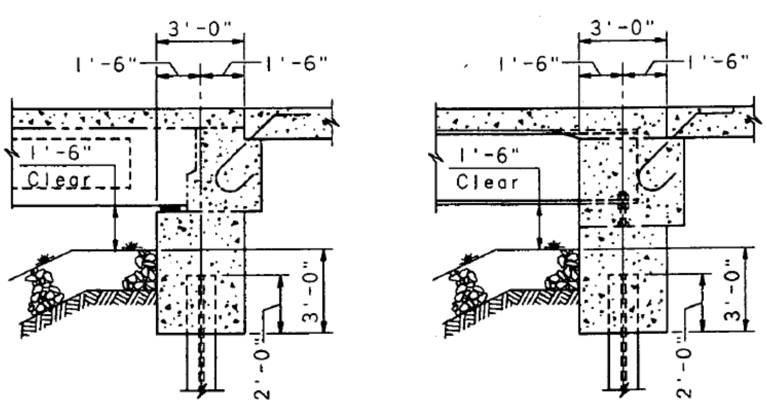


Figure 2.3- Capped-pile stub-type abutment: (left) for prestressed concrete box-beam stringers, (right) for steel I beam stringers. Source: Burke (1993)

The design of integral bridges is further simplified as the abutments and piers do not have to be designed to resist lateral or longitudinal loads. This is possible because the stiff concrete deck slab is rigidly attached to both abutments, which are restrained by the confining embankments [2]. Hence, it is the piles that eventually must take the lateral and longitudinal loads acting on the superstructure. A comparison between an integral bridge and a conventional bridge (contains expansion joints) is done to understand the various attributes (and limitations) of both.

Attributes

Jointless construction

Open deck joints allow for contaminated deck drainage to seep into the joints and cause extensive damage. Closed and sealed joints do provide some modicum of safety but are still vulnerable to extensive damage due to deck drainage. Therefore, the absence of expansion joints allows for more safety of the deck from contaminated drainage and reduces the need for maintenance. It also allows for a better driving experience.

Pressure Resistance

The integral bridge (being jointless) allows for a better distribution of longitudinal pavement pressure over the whole superstructure area, as compared to an approach pavement cross-section. Bridges that make use of expansion joints need to make sure that these joints can handle the thermal cycling of the bridge and the attached approach slabs. If not, they are more likely to be in extreme distress early into the lifespan of the bridge due to pavement pressure. Integral bridges do not face this issue as any pressure relief joint used by maintenance forces to relieve pavement pressure would be suitable for them.

Rapid Construction

Integral bridges enjoy the added advantage of lesser overall construction time due to the easy to assemble components it consists of. Consider the following:

a) Embankments: Simple compaction equipment can be used after placing soil. Very little hand compaction is required.
b) Cofferdams: Integral bridges do not require the use of cofferdams due to inclement weather conditions and stream flooding due to the use of drilled shaft piers or capped pile. It can even be done generally without the need for dewatering.
c) Excavations: Generally, not too deep (0.3-0.6 m)
d) Vertical Piles: Single row of piles, usually uniformly spaced. Jointed bridges have one or more rows of piles, sometimes the need for battered piles also arises.
e) Simple forms: Piers and abutments consist of simple shapes (rectangular or square)
f) Very few joints: Lack of joints leads to lesser overall time designing elaborate joints needed to relieve pavement pressure.
g) Fewer parts: Overall lesser number of bearings, deck joint seals, etc. are unnecessary, and the delays due to the installation of these components are avoided.
h) Improvement in Live load distribution: Slightly better wheel distribution load as compared to jointed bridges where the deck is separated from the pier by bearings.
i) Earthquake resistance: Since integral bridges derive their stability from the embankments, they are part of the ground and hence move with the ground. Consequently, when they are built on stable embankments and subsoils, they should have an adequate response to most earthquakes.

j) Simplified widening and replacement: Most bridges (jointed) have wall type abutments and flared wing walls. When growing traffic speeds and population demand for wider roads the replacements of such structures become complex and expensive. Integral bridges on the other hand have straight capped pile substructures that are easier to replace in contrast to regular bridges. The foundation of integral bridges can be easily replaced, withdrawn, or left in place, and hence do not need to be demolished.

Limitations

High Abutment Pile stress

Along with resisting the longitudinal and transverse stress due to loading (dead load, live load, etc.), the vertical piles of an integral bridge also must resist the stresses caused by the cycling temperature deformations. This causes much higher stress than would be found in a normal jointed bridge. This could lead to the yielding of the vertical piles and thus only suitable piles should be used (steel H piles or appropriately reinforced concrete piles or prestressed concrete piles).

Limited application

Integral bridges are known for being simple, durable, and safe. With these features comes a lack of adaptability (design, length, etc.), where it cannot be used in every case. Only in certain scenarios where conditions are favorable can integral bridges be erected. For example, integral bridges cannot be used for a skew angle greater than 30 degrees. Integral bridges cannot be used where the subsoil is not stable.

Buoyancy

Integral bridges being one monolithic structure face the problem of floatation. Being connected to the soil as a whole structure the bridge may feel some uplift force and design considerations will have to be made.

Despite the limitations stated above, there are more positives than negatives when it comes to the use of integral bridges. Bridges with a certain limited length that do not make use of curved beams and do not have an angle of skew greater than 30 degrees are best suited for integral type construction. The next section focuses on the demountable prototype that has been used and tested in the Netherlands by Rijkswaterstaat.

2.2 Demountable Bridge: Realized prototype in the Netherlands

To understand the concept of circular bridges, the prototype designed by Rijkswaterstaat is studied [3]. Below, a short description of the key structural elements will be given, but for more information regarding project details and construction procedures, the link is given below[4].

The prototype consisted of five beams (each composed of 8 elements) placed side by side with steel bars inserted transversely to keep them intact as a deck. Each beam is composed of eight elements, six elements (hollow) and two solid headers at the beam ends. Ducts in the beam elements allowed for unbonded prestressing tendons to pass through. Shear keys (along with some grouting material) are used to hold the elements together both longitudinally and transversely.

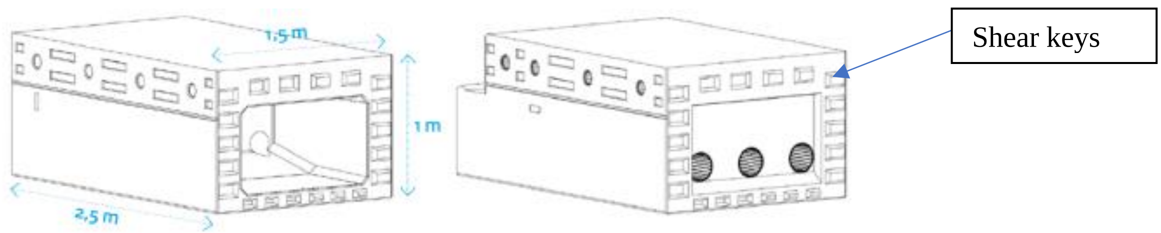


Figure 2.4- Hollow elements (left), Solid header element (right). Source: https://www.rijkswaterstaat.nl/zakelijk/duurzame-leefomgeving/circulaire-economie/bouw-circulair-viaduct-bij-kampen/index.aspx

The concepts of post-tensioning with unbonded tendons and 'shear keys' are the most important takeaways from this prototype. At this point, it becomes very clear that there can never be one 'basic' design when it comes to modular (circular) construction.

2.3 Accelerated Bridge Construction (ABC)

Another area of construction to consider is ABC (Accelerated Bridge Construction) practice. The ABC practice in no ways ensures that the construction will be demountable, rather it encourages the incorporation of more precast elements that would improve upon some features, namely:

- Site Constructability
- Total project delivery time
- Work-zone safety for the traveling public

One interesting design concept (D1) is the use of Precast Abutment elements. The Utah DOT (Department of Transport) system provides some unique details. The abutment elements contain large cavities (reduce the overall weight) which then allow for easier transport of these elements from the factory to the site. These cavities are then filled on-site with cast in place concrete after the required reinforcement is placed.

[4] https://www.rijkswaterstaat.nl/zakelijk/duurzame-leefomgeving/circulaire-economie/bouw-circulair-viaduct-bij-kampen/index.aspx

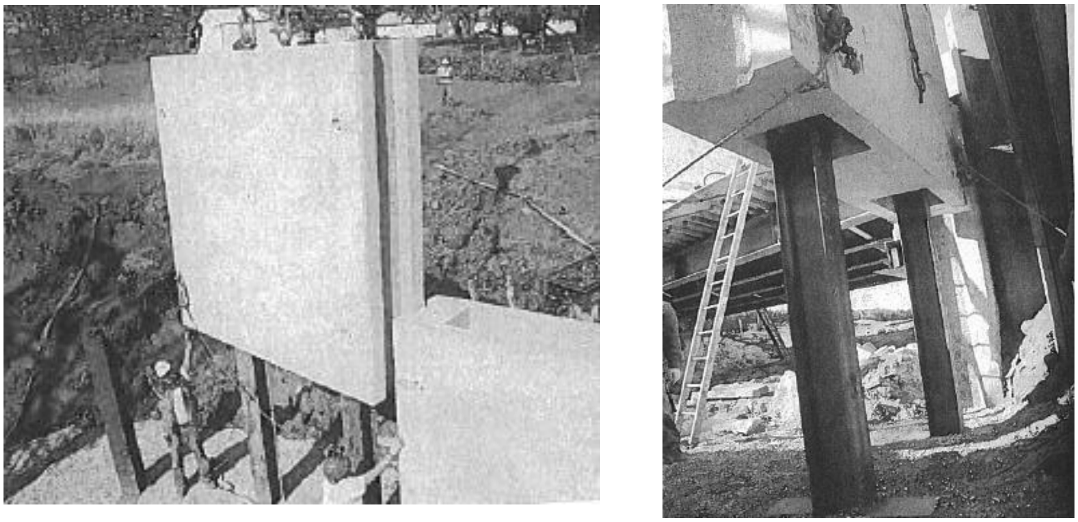


Figure 2.5 - Precast abutment elements with cavities (eventually filled with cast in place concrete. Source: [4]

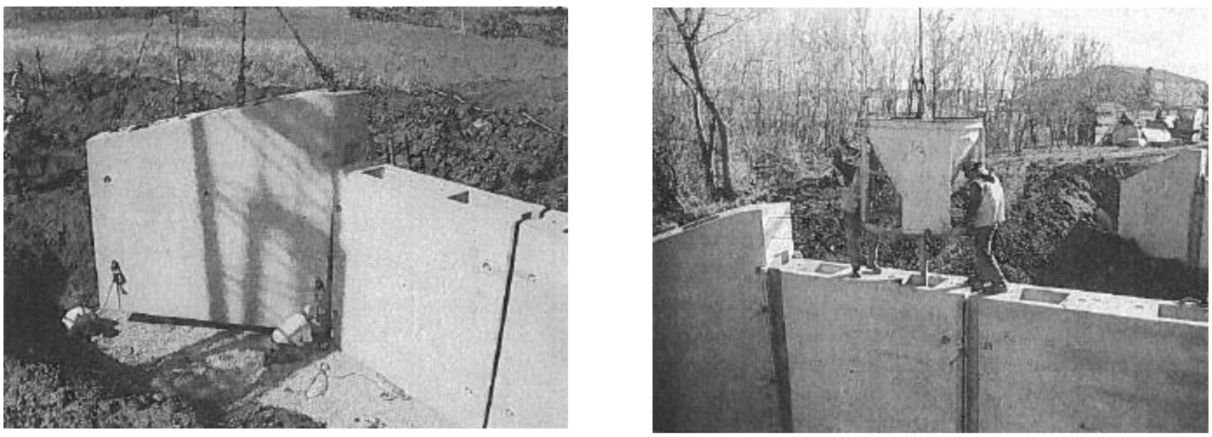


Figure 2.6 - Lightweight and easy to lift and place on site (left), Cavities filled with cast in place concrete on site (right). Source: [4]

This concept of precast abutment elements could help achieve a more demountable construction depending on the characteristics of the bridge in question and is investigated in later chapters. The different precast elements can be held together using transverse post-tension reinforcement or with vertical shear keys [6].

Besides providing a more maintenance-free and durable structure, continuity and elimination of joints can lead the way to more aesthetically pleasing and innovative bridge designs. (*Innovative Bridge Designs for Rapid Renewal- Transport Research Board*. *Page166*) The Utah DOT has had good experiences with precast elements and ABC practice [6]. In this additional thesis, the use of the precast elements and the ideas behind accelerated construction are to be kept in mind when moving forward. The use of precast elements (that can be disassembled and then reused) and the concepts of transverse post-tension and shear keys to join the numerous abutment elements together will be carried forward to the next chapter to try and realize a demountable solution.

3. KW 13: The first step to realize a demountable concept

3.1 KW13 Luikerweg Viaduct

This chapter makes use of documents from the firm Boskalis, who have designed the KW13 Viaduct. The documents supplied help in understanding the plans, geometry, design procedures involved in this integral bridge. The KW13 is constructed as an integral concrete construction with the deck made of C45/55 and abutments C35/45. The arched deck is prestressed in situ and integrated with the abutments. The abutments based on steel are low-lying to tension the deck structure between the ground bodies. The bridge has a span of approximately 39.3 m and a width of approximately 19.3 m (figures 3.1 and 3.2 help to better understand the geometry of the bridge).

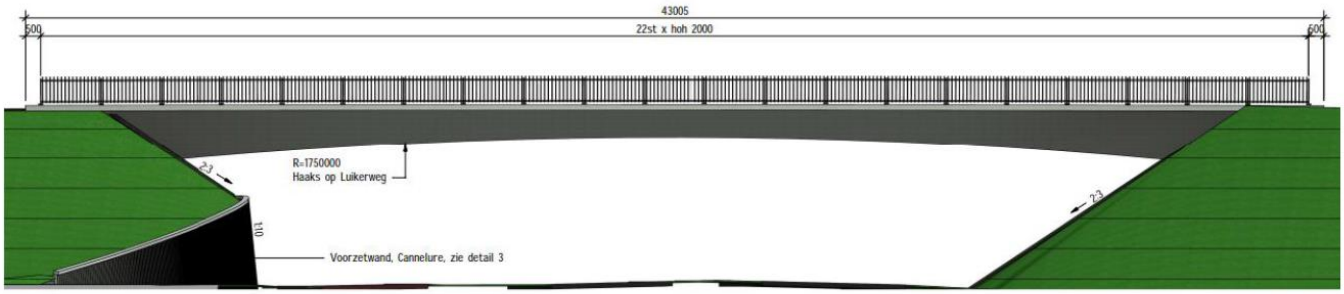


Figure 3.1 - Viaduct KW13. Source : Boskalis documents N69-DO-KW00-RAP-0009_KW13 v1.0

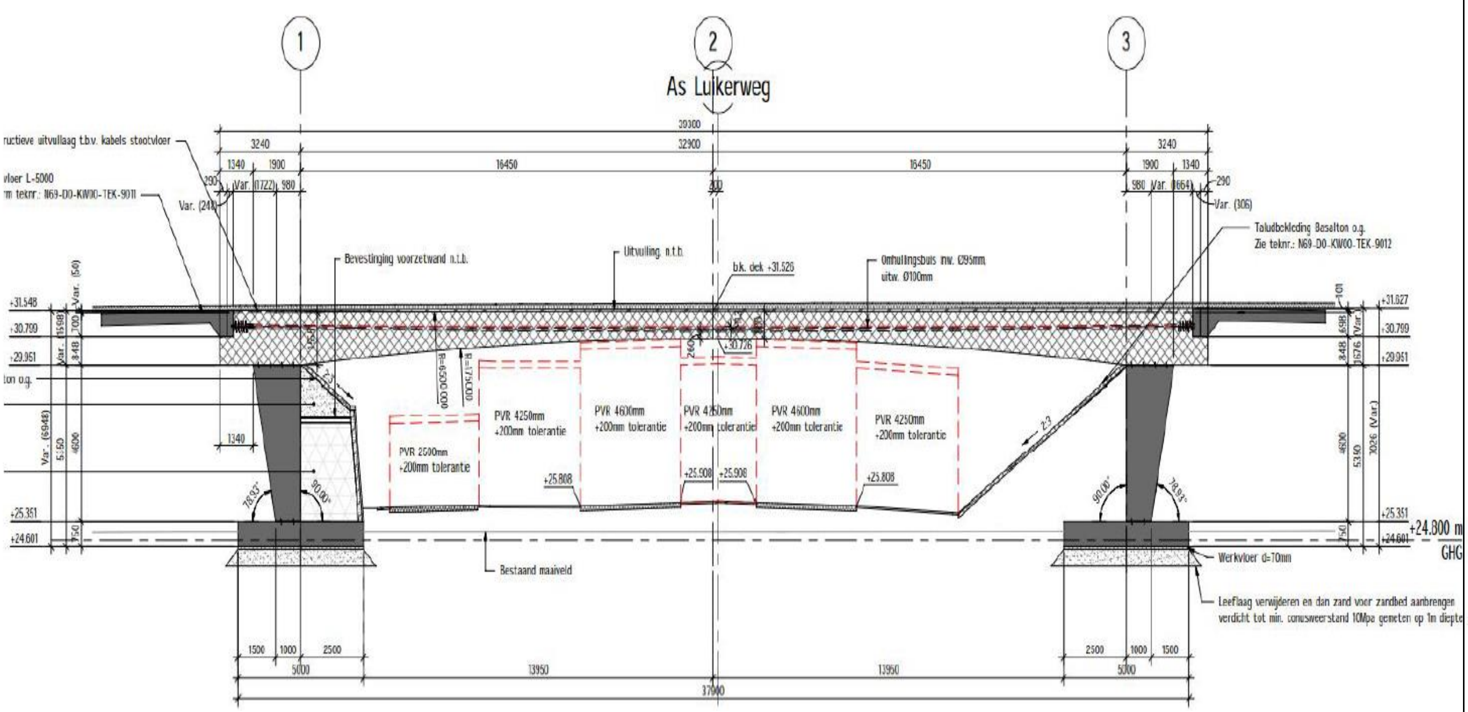


Figure 3.2 - Geometry Viaduct KW13. Source : Boskalis documents N69-DO-KW00-RAP-0009_KW13 v1.0

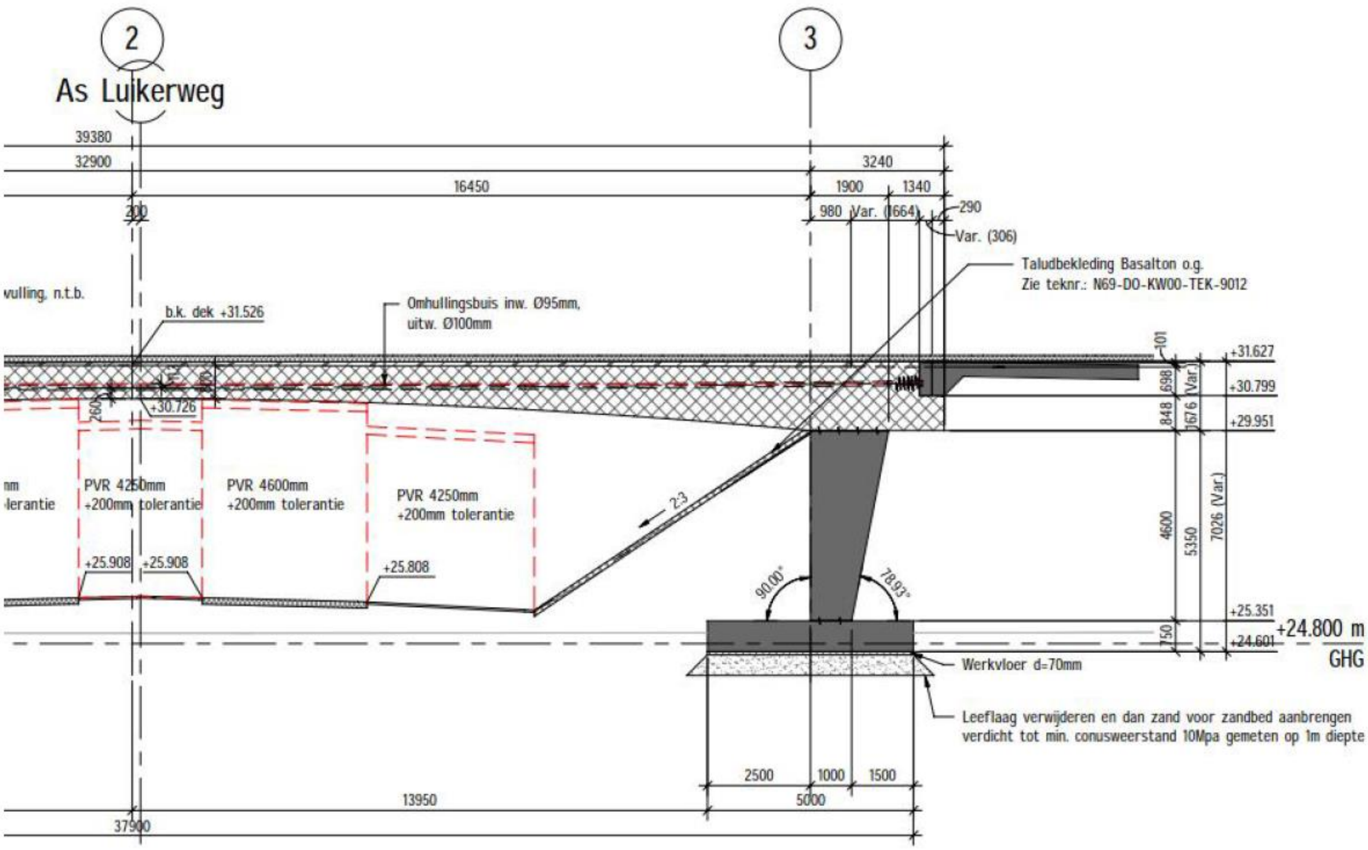


Figure 3.3 - KW13 Viaduct, Half span. Source : Boskalis documents N69-DO-KW00-RAP-0009_KW13 v1.0

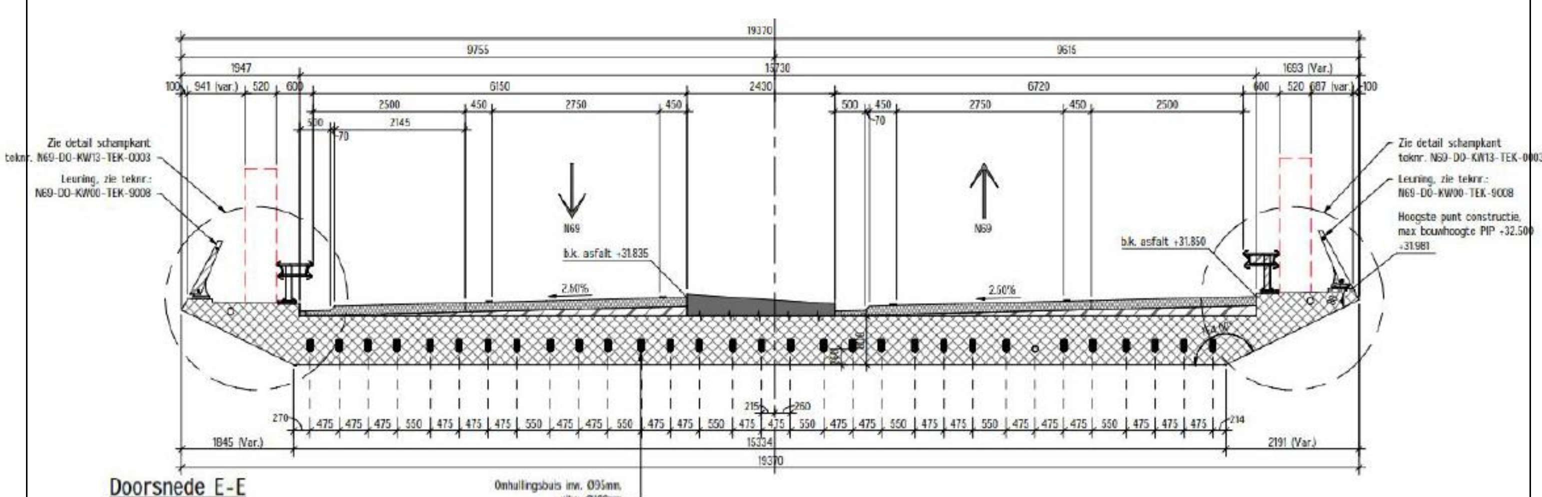


Figure 3.4 - KW13 cross section at midspan. Source : Boskalis documents N69-DO-KW00-RAP-0009_KW13 v1.0

Figures 3.3 and 3.4 show the half span of the bridge in more detail and the cross-section at midspan showing the position and spacing of the various prestressing cables and the width of the deck.

Construction phasing is as follows:

1) Phase 0 – Groundwork
2) Phase 1 - Realization of the footing and wall
3) Phase 2 - Applying soil backfill between abutments for foundation formwork deck
4) Phase 3 - Realization of the formwork deck construction
5) Phase 4 - Cast deck construction
6) Phase 5 - Removal of the formwork on the ground side of the wall for partial backfilling
7) Phase 6 - Ground backfill behind walls approx. 2 m, phased per side with a layer thickness of 0.5m
8) Phase 7 - Tensioning the deck construction
   - 20% prestressing 2 days after pouring depending on strength development (F_{cm} 20N /mm ^{2})
   - 100% prestressing only after complete curing of concrete
9) Phase 8 - Removal of the formwork near the wall along the length of approximately 3m to 5m.
   - Removing part of the formwork will exert pressure on the rest of the formwork. This should be considered when designing formwork.
10) Phase 9 - Further soil backfill to at least the bottom of the deck construction
11) Phase 10 - Removal of the complete formwork
   - Remove formwork after curing and clamping concrete
12) Phase 11 - Realize soil backfill to the final situation

<u>Loads acting on the bridge</u>

The different loads considered for the calculation of the BM diagram at ULS (figure 3.5) across the span are calculated by Boskalis using Scia software. A summary of the loads and combinations are given below. These loads are:

1. The self-weight of the structure (concrete, steel, etc.) $\rightarrow Load\ case\ 1$
2. Central reservation load (9.5 kN/m^2 for the entire deck)
3. Asphalt layer (100mm) Volumetric weight = 23 kN/m^3
4. Guide rails + Protruding deck part = 6 kN/m
5. Impact floor = 22 kN/m
6. Cantilever + bump floor = 22.5 kN/m
7. Wingwalls lead to load as well as a moment
8. Vertical and Horizontal ground pressure

Point numbers 2-7 (Resting load) $\rightarrow Load\ case\ 2$

<u>Variable loads</u>

1. Load Model 1- The evenly distributed load on the first lane and the other lanes is:

   Lane 1: UDL = 9.0 kN/m^2
   Others: UDL = 2.5 kN/m^2
   To determine the maximum moments, three tandem systems are applied halfway through the span and moved to normative positions of the maximum sagging moment, maximum hogging moment, and maximum shear force.
2. Load Model 2- Load model 2 concerns a single axle load of 400 kN without evenly distributed load.

3. Load Model 3- Special vehicle load
4. Load Model 4- Crowd Loading, following the Euro code the design load from pedestrians consists of an evenly distributed load (UDL, Uniform Distributed Load).
5. Horizontal brake loads- Following NEN-EN1991-2 article 4.4.1 applies to the braking or starting load: 495 kN
6. Temperature loading
7. Wind load
8. Handrail load
9. Impact load, a collision of the deck, an accident on the bridge deck, etc have been considered as well

This report does not go into detail about the load calculations, detailed information about the loads and combinations can be found in the documents of Boskalis [3].

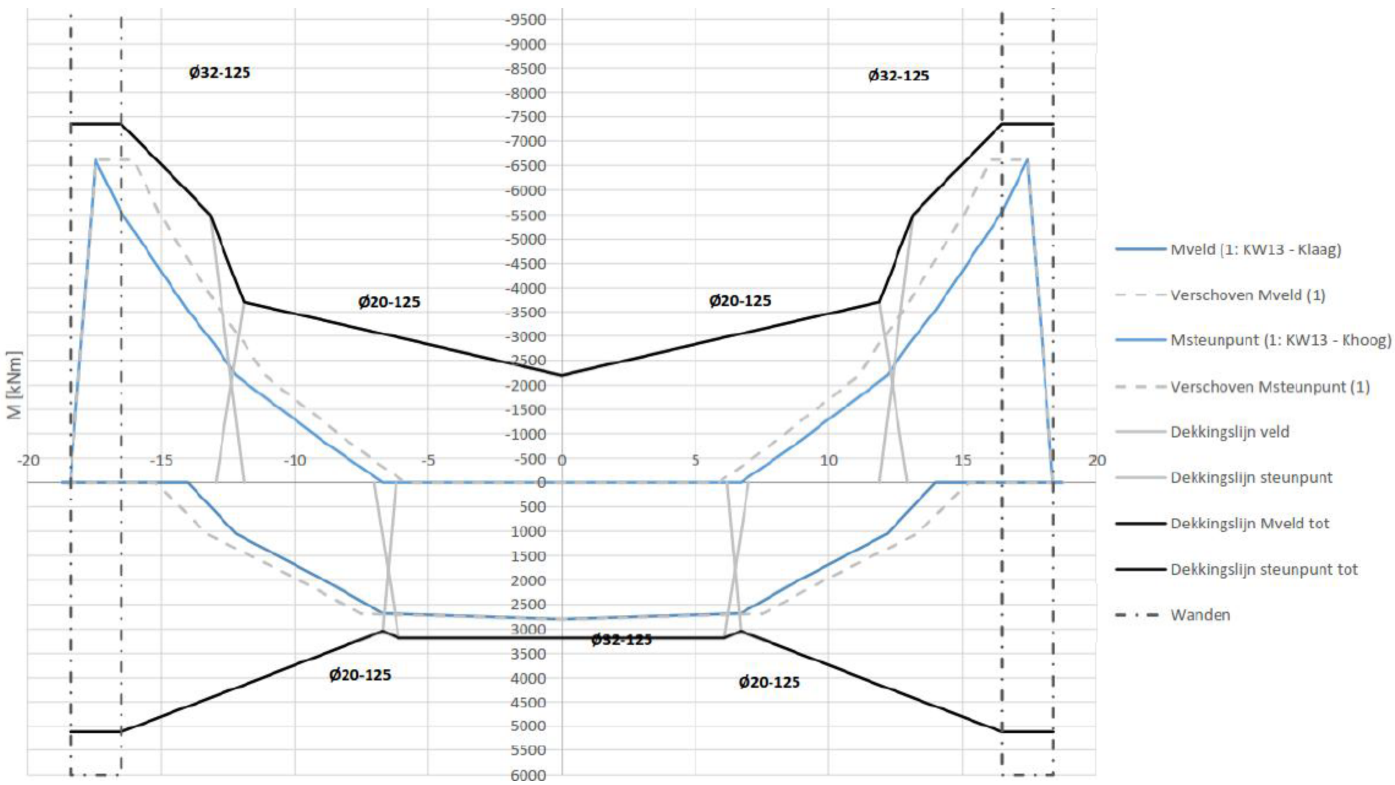


Figure 3.5 - Bending moment distribution along the bridge at ULS. Source: Boskalis documents N69-DO-KW00-RAP-0009_KW13 v1.0

The figure above (figure 3.5) shows the bending moment distribution (ULS) across the span. The dotted dark grey line (wanden) refers to the walls (abutments). The black lines refer to the capacity of the reinforcement at the supports and the middle of the deck. The dark blue line (Mveld, KW13 -Klaag) corresponds to the M field line in the middle of the deck. The light blue line corresponds to the moments at the support points (Msteunpunt KW13-Khoog). The light blue lines are the M lines of interest as they are the overall bending moment line of the structure at the support with the highest moment (hogging) at the deck abutment connection. All the loads acting on the bridge (found on the previous page), require a large amount of two-layer reinforcement (figure 3.6) to take up the forces at this junction as well as within the abutment. Once the reinforcement is placed in the right positions, concrete is poured, and the prestressing of the deck is done to 100 percent.

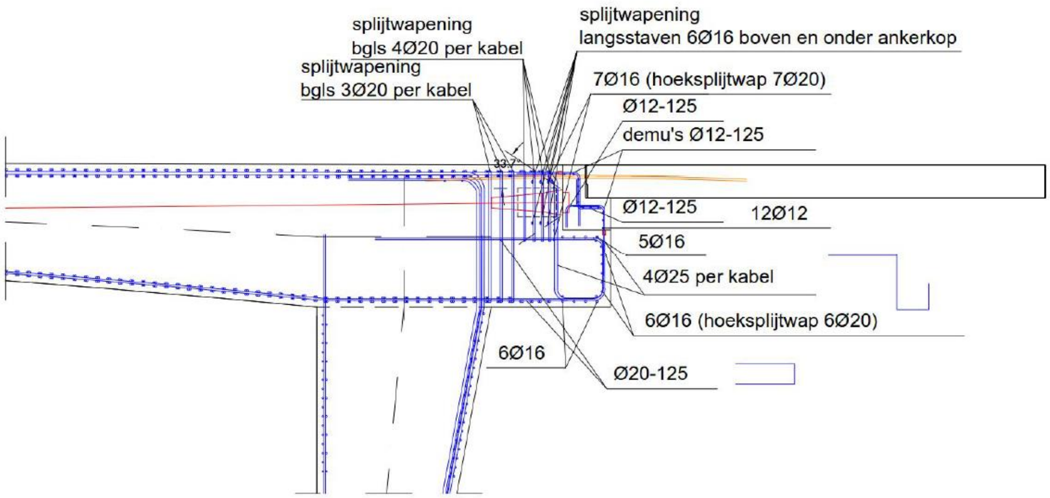


Figure 3.6 - Deck-abutment connection. Source: Boskalis documents N69-DO-KW00-RAP-0009_KW13 v1.0

The figure shown above (figure 3.6) is a detailed drawing of the abutment-deck connection. For this design concept (shown above) to be realized a lot of reinforcement (used to connect the deck and abutment) along with the cast in place concrete is required to make it a rigid (monolithic) connection. This makes it possible to transfer the moment from the deck to the abutment but also makes it difficult to realize the concept of demountable construction. Analyzing the data provided by Boskalis, a few areas of concern become evident, and addressing these concerns would be a step in the right direction in achieving a demountable structure. The analysis carried out by the engineers at Boskalis [3] has helped in understanding the finer details that go into the bridge construction process. Using this information, some of the preliminary steps are modified to provide a practical solution in making an already integral bridge demountable.

4. Practical Solutions

4.1 Frame like behavior

Integral bridges essentially behave like frames and therefore the entire hogging moment at the end of the deck is transferred to the abutment at the deck-abutment junction (see figure 4.1). For this additional thesis, the first three phases of construction are looked at more closely. Alternative solutions (that work) to realize the concept of demountable bridges are looked at in more detail. Due to this, construction details and procedures are not discussed as the area of interest is the abutment-deck (critical) connection. The abutment will have to take the moment transferred to it by the deck and hence a suitable abutment design is required that can be disassembled as well. Studying the prototype done by Rijkswaterstaat, it becomes clear that to achieve demountable construction unbonded tendons must be used. One solution that is explored in this report is the use of precast abutments (ABC Concept D1). If this design concept is to be incorporated and be demountable at the same time, no bonded reinforcement or prestressing cables can be used. Due to this, the concept of vertical prestressing (unbonded) is explored further in this report. A few basic checks such as:

1. The prestressing force required to counter the hogging moment (see figure 3.5)
    a) Without eccentricity
    b) With an eccentricity of +0.3m (assumed value)
2. Check to see that the force calculated above is within the concrete compressive range
3. Choice of bars/cables used for post-tension system and whether the required (calculated) force can be delivered with the chosen system

are carried out to see if the hogging moment (see figure 3.5) can be carried by an abutment that is vertically prestressed. If these checks are satisfactory then the amount of steel required longitudinally in the abutment is reduced. The next step would be to model the deck similar to the prototype done by Rijkswaterstaat and ensure that the connection between the deck and the abutments are rigid. This report will focus on the abutment and how it can take the hogging moment from the deck.

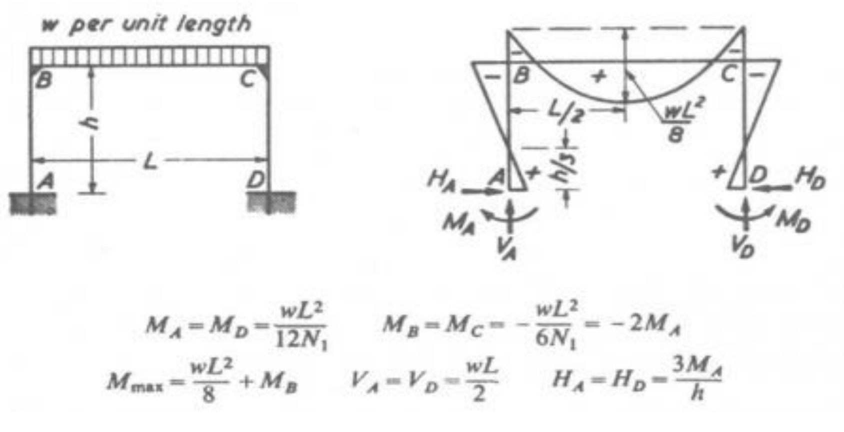


Figure 4.1 - Bending moment transfer in frames. Source: https://www.steelconstruction.info/Modelling_and_analysis

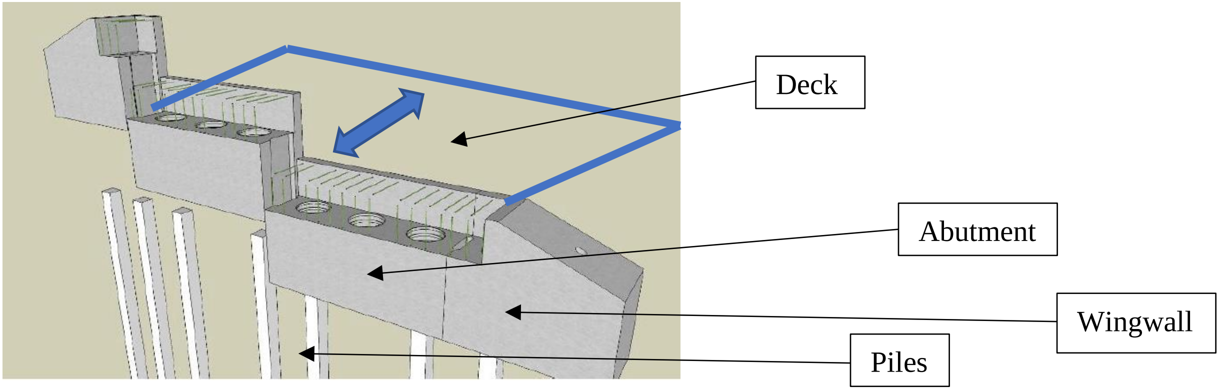


Figure 4.2 – Prefabricated Integral Abutment Bridge without deck (3-D). Source: [6]

Recalling from section 3.1 the first three phases involved in construction, the focus is now on the stage after the footing has been laid (phase 1). The choice is made to use precast abutment elements, and they are considered as vertical beams. Taking inspiration from the system used by the Utah DOT [6], a system using precast abutment elements would make the option of disassembly possible. To achieve this demountable feature, unbonded tendons will have to be used.

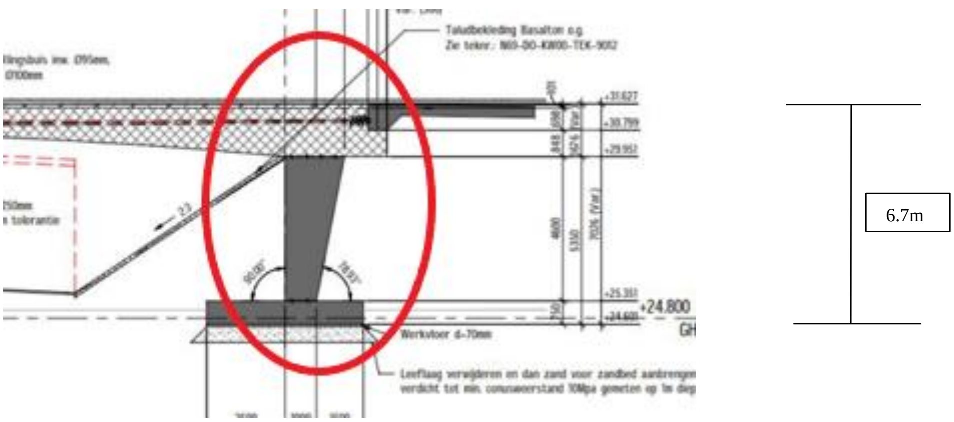


Figure 4.3 - Abutment as a vertical beam. Source: Boskalis documents N69-DO-KW00-RAP-0009_KW13 v1.0

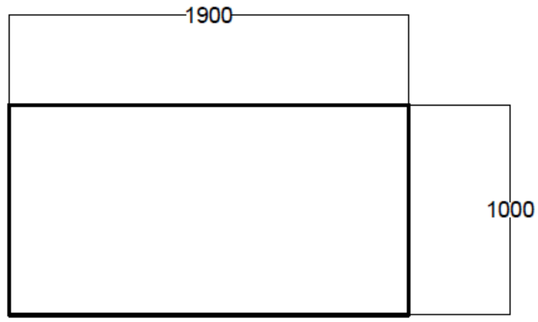
For the sake of simplicity, we assume this to be a vertical (prismatic) beam with the dimensions

Length: 6.7m

Width: 19.3m

Height 1.9m

To incorporate design concept D1, the abutment is made up of several precast elements that are transported to the site and joined to form the whole abutment structure. For KW13, the abutment width is split into 1m segments, leading to 20 (modular) elements to prefabricate. The number can be reduced by increasing the size of these elements up to the maximum weight that can be transported over the road, in this report the dimension of 1m is considered. Vertical prestressing is done from the top of the abutment with an anchor at the bottom. Now, the calculation for vertical prestressing is done keeping in mind the hogging moment (kNm per running meter) that is transferred to this abutment. Two cases of prestress force are looked at in this report.



b = 1m

h = 1.9m

W = 0.601 m^3

Figure 4.4 - Cross-section of one precast abutment element

A few cases were explored, one where a prestressing force with zero eccentricity was applied, as well as one with an eccentricity of +0.3m (assumed value), is applied to the abutment (considered as a vertical beam). The prestressing force required is calculated for each case below and the necessary compression check is carried out for each case.

4.2 CASE I: Zero eccentricity

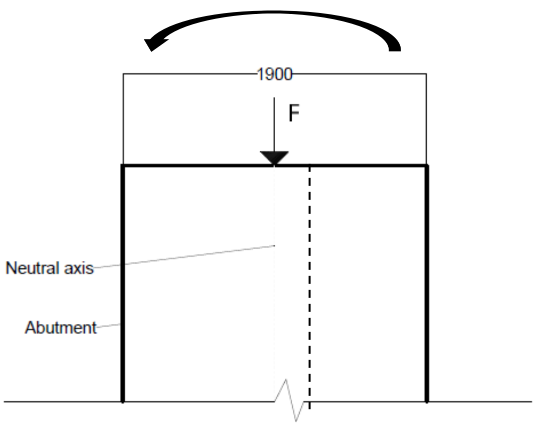


Figure 4.5 - Zero eccentricity case (front view of the abutment)

The stress at the bottom and top fibers are checked to obtain the value of the least prestressing force required to resist this moment. At the bottom fiber, the stress should not exceed $0.7f_{ck}$, and the maximum tensile stress at the top fiber must be ≤ 0. Assuming 15% of losses for time-dependent losses.

<u>Zero eccentricity</u>- Stress at bottom fiber (concrete under compression)

$$\sigma_b = -\frac{P_\infty}{A_c} + \frac{M_{hogging}}{W} \geq -0.7f_{ck}$$

$$\sigma_b = -\frac{0.85P_i}{1.9 * 1.0} + \frac{-6550}{\frac{bh^2}{6}} \geq -0.7 * 35$$

$$= -\frac{0.85P_i}{1.9 * 1.0} + \frac{-6550}{0.601} \geq -24.5$$

$$= P_i \leq 30403.35\ kN/m$$

Zero eccentricity- Stress at top fiber (concrete under tension)

$$\sigma_t = -\frac{P_\infty}{A_c} - \frac{M_{hogging}}{W} \leq 0$$

$$= -\frac{0.85P_i}{1.9 * 1.0} - \frac{-6550}{0.601} \leq 0$$

$$P_i \geq 24361.35\ kN/m$$

Taking the lower limiting value of force (24361.35 kN/m), the area of steel required per running meter is calculated.

$$\sigma_{pm,0} = 1395\ MPa$$

$$A_{p,required} = \frac{P_{initial}}{\sigma_{pm,0}} = \frac{24361350}{1395} = 17463.33 \frac{mm^2}{m}$$

Assuming standard Y1860 prestressing steel is used, (standard diameter of 15.7mm and area of $150mm^2$), 13 cables containing 9 strands each will be required.

$$A_{p,used} = 150 * 117 = 17550\ \frac{mm^2}{m}$$

Actual prestressing force ($P_{initial}$):

$$A_{p,used} * \sigma_{pm,0} = 17550 * 1395 = 24482250\ N/m$$

Check to see whether the compressive stress is between the range $0.45f_{ck}$ and $0.6f_{ck}$.

$\frac{P_{initial}}{A_c} = 12.88\ MPa < 0.45 * f_{ck} = 15.75\ MPa$ (Taking the lower limit value of $0.45f_{ck}$ into account)

Hence it seems that vertical prestressing for the zero-eccentricity case can withstand this large hogging moment and still be within the compressive stress range of concrete.

4.3 CASE II: +0.3m eccentricity

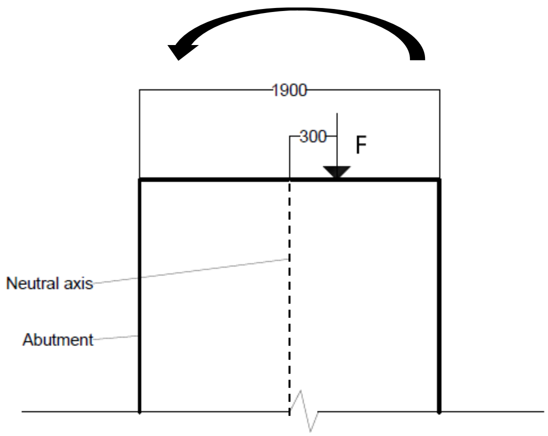


Figure 4.6 - +0.3m eccentricity case (front view of the abutment)

The same rules as the previous case apply, in this case, the moment that arises from the eccentricity of the prestressing force (P*e) needs to be considered. Assuming 15% of losses for time-dependent losses.

Check to see that the bottom and top fibers are within the allowable stress ranges:

Stress at bottom fiber:

$$\sigma_b = -\frac{P_\infty}{A_c} + \frac{M_{hogging}}{W} + \frac{P_\infty * e}{W} \geq -0.7 f_{ck}$$

$$\sigma_b = -\frac{0.85 P_i}{1.9 * 1.0} + \frac{-6550}{\frac{bh^2}{6}} + \frac{0.85 P_i * 0.3}{W} \geq -0.7 * 35$$

$$\sigma_b = -\frac{0.85 P_i}{1.9 * 1.0} - 10898.50 + \frac{0.85 P_i * 0.3}{W} \geq -24500$$

$$= P_i \leq 589575.20\ kN/m$$

Stress at top fiber:

$$\sigma_t = -\frac{P_\infty}{A_c} - \frac{M_{hogging}}{W} - \frac{P_\infty}{W} \leq 0$$

$$= -\frac{0.85 P_i}{1.9 * 1.0} - \frac{-6550}{0.601} - \frac{0.85 P_i * e}{0.601} \leq 0$$

$$= P_i \geq 12504.0156\ \ kN/m$$

The effect of eccentricity provides a moment that counters the initial hogging moment and reduces to a large extent the prestress force required.

Similarly, the amount of required steel is calculated, $\sigma_{pm,0} = 1395\ MPa$

$$A_{p,required} = \frac{P_{initial}}{\sigma_{pm,0}} = \frac{12504015.6}{1395} = 8963.452 \frac{mm^2}{m}$$

Assuming standard Y1860 prestressing steel is used (standard diameter of 15.7mm and area of 150mm^2), 7 cables containing 9 strands each would be required. Area of steel used is given by,

$$A_{p,used} = 150 * 63 = 9450\ \frac{mm^2}{m}$$

Actual prestressing force:

$$P_{initial} = A_{p,used} * \sigma_{pm,0} = 9450 * 1395 = 13182750\ N/m$$

Check to see whether the compressive stress is between the range 0.45f_{ck} and 0.6f_{ck}.

$\frac{P_{initial}}{A_c} = 6.93\ MPa < 0.45 * f_{ck} = 15.75\ MPa$ (Taking the lower limit value of 0.45f_{ck} into account)

Hence it seems that vertical prestressing along with the moment due to eccentricity can withstand this large hogging moment and still be within the safe compressive stress range of concrete.

One point to keep in mind is that the force of prestressing is an upper bound as the self-weight of the slab was not considered. The self-weight of the slab, acting downwards adds to the prestressing force on the abutments.

4.4 Post tension system to aid the demountable concept

In the previous section, the required force along with compressive checks was carried out to ensure that the calculated force did not exceed concrete capacity. Using the brochures provided by DYWIDAG [7], various post-tension options are investigated, and the most suitable system is chosen. One such interesting concept is the DYWIDAG Prestressing system using bars. From the example of the prototype by Rijkswaterstaat, unbonded tendons needed to be used to even consider the idea of demountable construction. Some important characteristics of prestressing using unbonded tendons are:

- Even though grouting is absent the tendons are protected from corrosion by grease and embedding in a plastic sheet.
- The corrosion protection allows for a lesser concrete cover leading to smaller size elements.
- Unbonded tendons generally use smaller bar diameters and this can allow for larger eccentricities of prestressing steel.
- Easy and quick installation of tendons.
- A key factor to keep in mind is that there will be no bond between the tendon and the duct and subsequently no bond between the duct and the concrete. Due to this, after the first crack occurs the deformation of the steel will be more in the case of unbonded tendons (due to lack of bond) and the tendon strain will be constant between the two anchorages. This leads to a larger deformation at increasing load. The concrete strain in the compression zone will be higher, and the concrete fails before steel fracture. All this leads to a much lesser overall resistance of a structure using unbonded tendons.

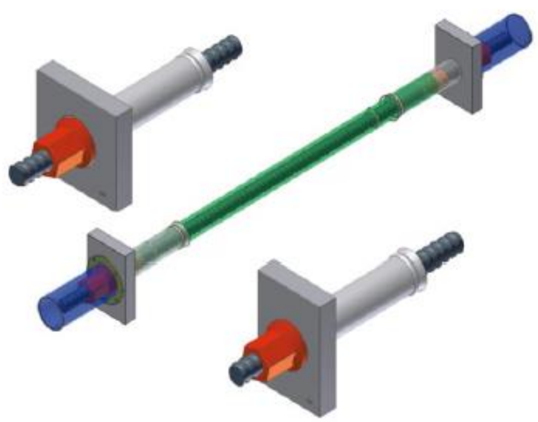



Figure 4.7a- Post tension bars without bonding along with anchor plates

Technical data

Designation			THREADBAR®						Plain bar	
			18 WR	26 WR	32 WR	36 WR	40 WR	47 WR	32 WS	36 WS
Nominal diameter	d_s	[mm]	17.5	26.5	32	36	40	47	32	36
Cross section area	S_n	[mm²]	241	552	804	1,018	1,257	1,735	804	1,018
Nominal mass per metre[1]	M	[kg/m]	1.96	4.48	6.53	8.27	10.20	14.10	6.31	7.99
Pitch	c	[mm]	8	13	16	18	20	21	3	3
Characteristic breaking load	F_m	[kN]	255	580	845	1,070	1,320	1,820	845	1,070
Max. initial stressing force[2] $P_{m0,max} = S_n \times 0{,}8 \times f_{p,k}$		[kN]	204	464	676	856	1,056	1,457	676	856
Max. overstressing force[3] $P_{0,max} = S_n \times 0{,}95 \times f_{p0,1k}$		[kN]	219	499	722	912	1,131	1,566	722	912

[1]The nominal mass per metre includes 3.5% not load bearing portion of ribs.
[2]The given values are maximum values according to Eurocode 2, i.e. min ($k_1 x f_{pk}$, $k_2 x f_{p0.1k}$) applies. The fulfillment of the stabilization criteria and the requirements fo cracks width in the load transfer tests were verified at $0.8 \times F_{pk}$.
$F_{pk} = S_n \times f_{pk}$
$F_{p0.1k} = S_n \times f_{p0.1k}$
[3]Overstressing is permitted if the force in the prestressing jack can be measured to an accurary of ±5% of the final value of the prestressing force.

Figure 4.7b - Technical data of the different bar systems

Note: The images and table shown above have been obtained from the DYWIDAG brochure mentioned in the references.

<u>Zero eccentricity case</u>

The first case of zero eccentricity leads to an initial prestressing force of 243631.35 kN per running meter. Making use of 47 WR bars a realistic spatial configuration of post-tension bars cannot be realized. The maximum initial stressing force (47WR) is 1457 kN. The number of bars required in one row to accommodate the force of 243631.35 kN/m is ~168 bars. With a bar diameter of 47mm that comes up to a total length of 7.89m which cannot be arranged in the space of one meter. Hence, not feasible.

Splitting up this one-row arrangement into three rows (to make use of 1.9m of breadth) we still come up short as the distance required to accommodate the number of bars is ~2.63m. Hence this system of bars would not work for the zero-eccentricity case.

For the remainder of this report, the focus will be on Option 2: +0.3m eccentricity case. The post-tension bar will be at a position of +0.3m (towards the right) from the neutral axis of the abutment. This leads to an additional moment arising from the eccentricity of the post-tension bar. This moment is opposite in sign to the hogging moment being transferred from the deck. The required values of forces have been calculated on page 23 of this report.

Now, looking at the second case, with an eccentricity of +0.3m, an initial prestressing force of $13182.750\frac{kN}{m}$ is required. Using the same bar (47WR) a total of ~9 bars (9.04 to be exact) is needed.

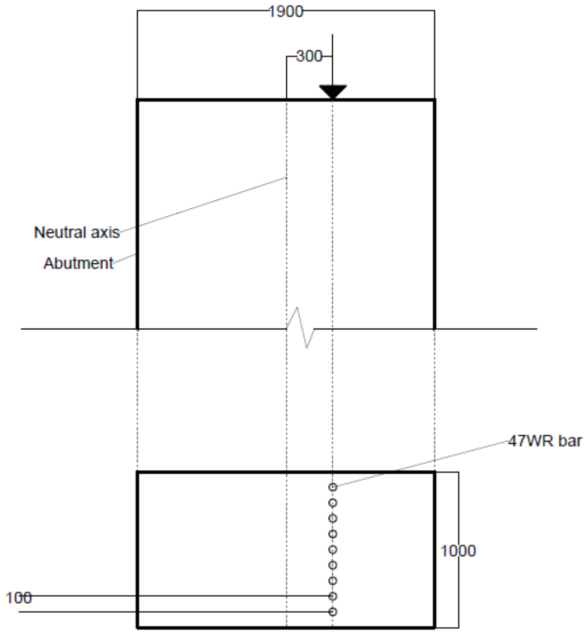


Figure 4.8 - Option 1 (9 bars in one row), +0.3m eccentricity

With this option (1), the concrete section is quite cramped with the steel bars and might hinder the horizontal post-tensioning of the deck. Another option is to use the 1.9m width of the abutment to incorporate more rows of bars hence reducing the number required in each row.

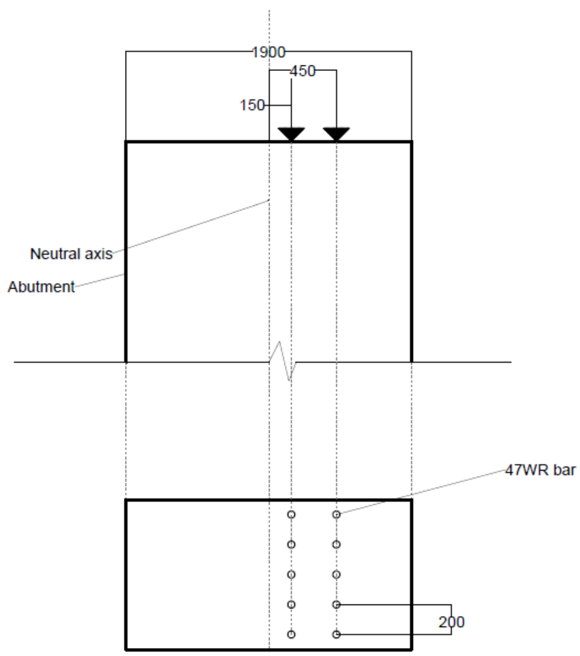


Figure 4.9 – Option 2, ten bars split equally into two rows, +0.3m

Arranging these 9 (47 WR) bars into two rows allows for a less cramped concrete cross-section and a much better arrangement of steel bars. With the second option (see figure above) a little more space is made available in the concrete cross-section for other necessary reinforcement (nominal longitudinal reinforcement, stirrups transverse post-tension bars/strands, etc.)

Table 1- Geometric characteristics of the Anchor plate. Source: DYWIDAG Prestressing using Bars

Geometrical Characteristics of Accessories

				THREADBAR®						Plain bar	
Bar designation				**18 WR**	**26 WR**	**32 WR**	**36 WR**	**40 WR**	**47 WR**	**32 WS**	**36 WS**
Domed Anchor Nut	2099	length	[mm]	55	75	90	100	115	135	46	60
		width across flat	[mm]	36	50	60	65	70	80	55	65
Hex nut[4]	2002	length	[mm]	60	80	90	110	120	140	55	80
		width across flat	[mm]	41	46	55	60	70	80	55	60
Coupler (Standard)	3003	length	[mm]	100	170	200	210	245	270	110	160
		outside diameter	[mm]	36	50	60	68	70	83	60	68
Square Solid Plate	2011	width	[mm]	110	150	180	200	220	260	180	200
		length	[mm]	110	150	180	200	220	260	180	200
		thickness	[mm]	25	35	40	45	45	50	40	45
Rectangular Solid Plate (Unbonded and Bonded)	2012	width	[mm]	100	130	140	150	160	200	140	150
		length	[mm]	130	150	180	220	250	280	180	220
		thickness	[mm]	30	35	40	50	60	60	40	50
Rectangular Solid Plate (Bonded)	2076	width	[mm]	80	120	140	160	180	210	140	160
		length	[mm]	90	130	165	180	195	235	165	180
		thickness	[mm]	25	30	35	40	45	55	35	40
QR-Plate	2074	width	[mm]	-	120	140	160	180	-	-	160
		length	[mm]	-	130	165	180	195	-	-	180
		thickness	[mm]	-	30	35	40	45	-	-	40
Corrugated Duct	4061	internal diameter	[mm]	25	38	44	51	55	65	44	51
		outside diameter	[mm]	30	43	49	56	60	70	49	56
Minimum Bar Protrusion at stressing anchorage			[mm]	60	75	90	100	115	135	46	60

[4] Hex nuts 2002 are not included in ETA-05/0123.

For the 47WR thread bar chosen an appropriate anchor plate would be a Rectangular Solid Plate (Unbonded). One large anchor plate (rectangular in shape) can be used as the use of single plates seems cramped [8]. This must be designed appropriately to account for the higher stresses that arise in the concrete face. This is not elaborated further in this report. The next section will give information about the strut and tie model used to visualize these forces.

4.4 Strut and Tie Model (Option 2)

Presented below is a strut and tie model showing the different forces arising in the concrete section and where the tensile ties need to be placed. The figure below (4.10) gives an overall flow of forces in the abutment as well as the deck. To avoid confusion these two individual strut and tie systems have been drawn separately to understand both systems a little better (see figure 4.11 and 4.12).

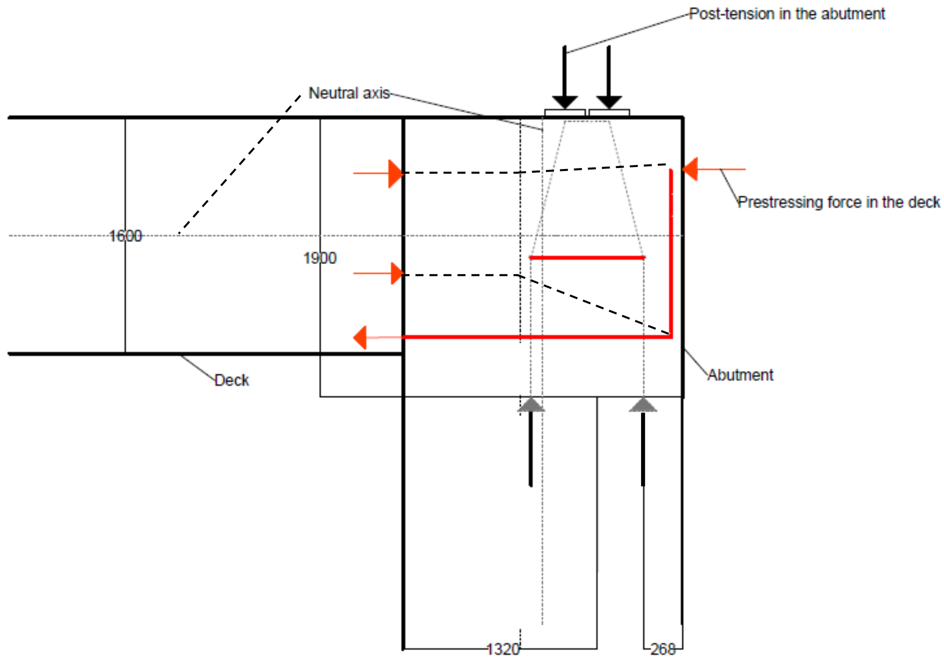


Figure 4.10- Strut and Tie of abutment and deck

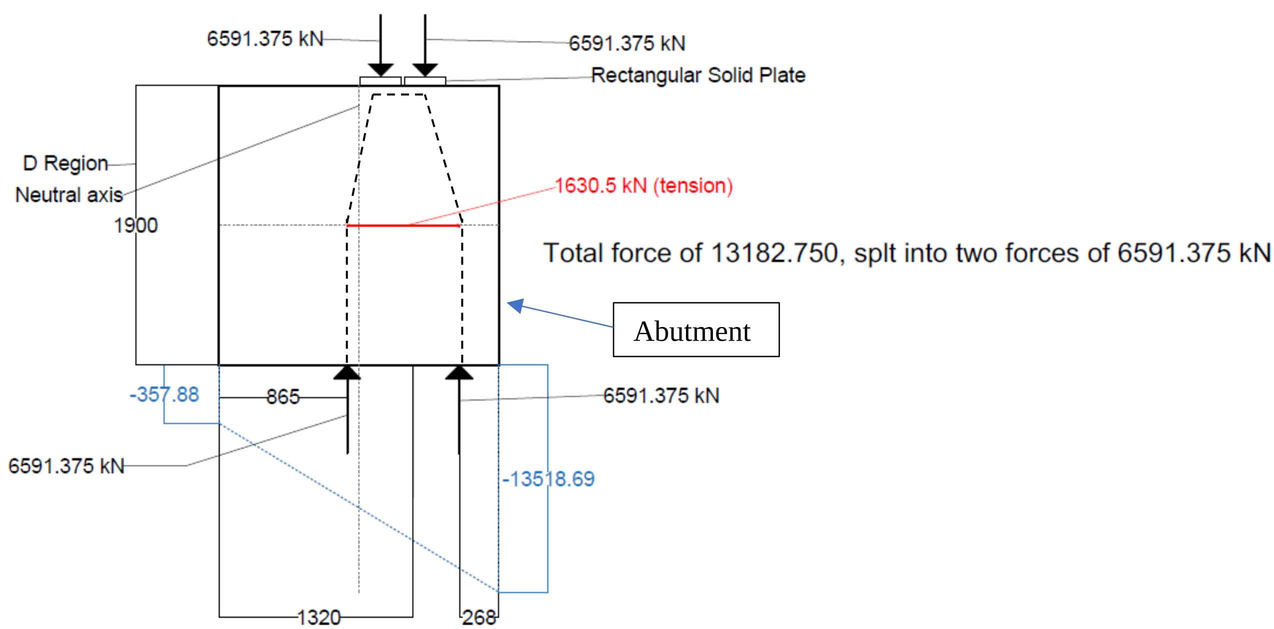


Figure 4.11 - Strut and Tie Model for the abutment

The force of the horizontal prestress cannot be ignored and has already been calculated for the integral bridge by Boskalis. Two strut and tie models will exist without any interaction between both models. The figures (figure 4.10 and 4.11) have been sketched separately for more clarity but will be one whole system. Figure 4.11 models the flow of forces in the abutment and 4.12 visualizes the forces in the deck.

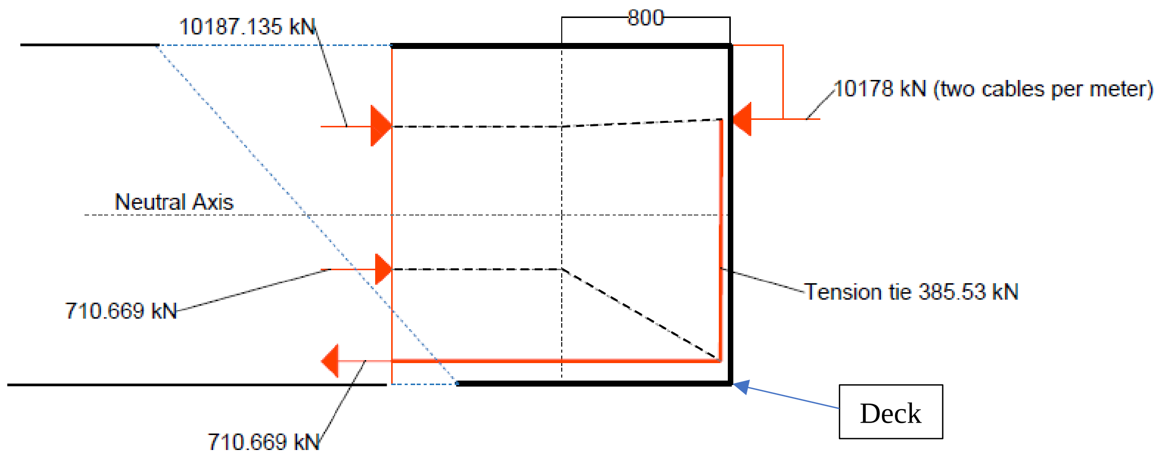


Figure 4.12 - Strut and Tie Model for the deck

4.5 Conclusion

In conclusion, the report above acts as a feasibility report in some sense as it shows the potential of incorporating the idea of un-bonded vertical post-tension into that of an integral bridge in a practical manner. The report shows how the moment transferred to the top of the abutment can be resisted using a vertical post-tension system. The merits of having an integral and demountable bridge far outweigh the negatives and helps sustainably control the afterlife of structures. However, at the present moment, certain improvements and innovative structural systems are still required to make the bridge completely demountable.

Shortcomings of the vertical prestressing system:

Abutments

The shear forces acting on the abutment due to its interaction with the deck and soil backfill must be discussed but this is a step in the right direction. The shear forces along the height of the abutment will have to be taken up by another system (most likely shear reinforcements) and this complicates the demountable feature. Anchoring of the vertical prestressing system once in place will be quite difficult to remove and will most likely be an expensive process to set up and dismantle.

Deck

The next stage would be realizing how the deck should be attached to this abutment and still be a rigid connection as well as demountable. Studying the necessary literature one can make a few assumptions regarding some of the ideas required to see this concept of a demountable integral bridge is completed. The deck will have to be split into segments, and these segments can be held together with a combination of shear keys and unbonded post-tensioning. The dimensions of these segments would approximately resemble the size of the blocks used by the demountable bridge prototype by Rijkswaterstaat (see section 2.2). An option for the order of post-tension will be abutments and then the deck (once a plan for the deck is decided). The reason for this is that once the deck is post-tensioned, the resulting hogging moments at the ends due to self-weight will have to be taken by the abutments (for which post-tension should have already been done).

A few interesting topics do come up and these topics could be an interesting follow-up for the graduation thesis. Some of the research questions are as follows:

- Would the use of Geosynthetic Reinforced Soil (GRS) soil abutments be an improvement?
- How would the deck be realized for a demountable bridge?
- Would the connection mimic the Boundary Condition of an integral bridge?
- What would the size of the deck elements be?
- Would a demountable post-tension deck of 40m span be possible?

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Appendix

1) Meeting on the 16th of July 2020 with Dr. ir. Herbert van der Ham

Discussion of interests and on-going projects. Pre-requisite knowledge that would be required was also discussed. The option of continuing this topic or rather coming up with an interesting topic for the graduation thesis was also discussed.

2) Meeting on the 24th of July 2020 with Dr. ir. Herbert van der Ham

The decision was taken to take up a project regarding demountable bridges. It was made clear that the concept behind integral bridges would play a role as the thesis progressed and hence was important to know. A few documents regarding the literature of integral bridges were shared and the goal of the next 3-4 weeks was a literature review to better understand the concepts.

3) Meeting on the 14th of August 2020 with Dr. ir. Herbert van der Ham

The main subject of this additional thesis is the KW13 viaduct and different concepts were discussed to come up with a demountable (but practical) solution for the bridge in question. The critical connections were discussed the previous week and this week's discussion focused on how those critical connections could be realized. The term ABC was introduced as well as the idea of introducing a vertical post-tension system.