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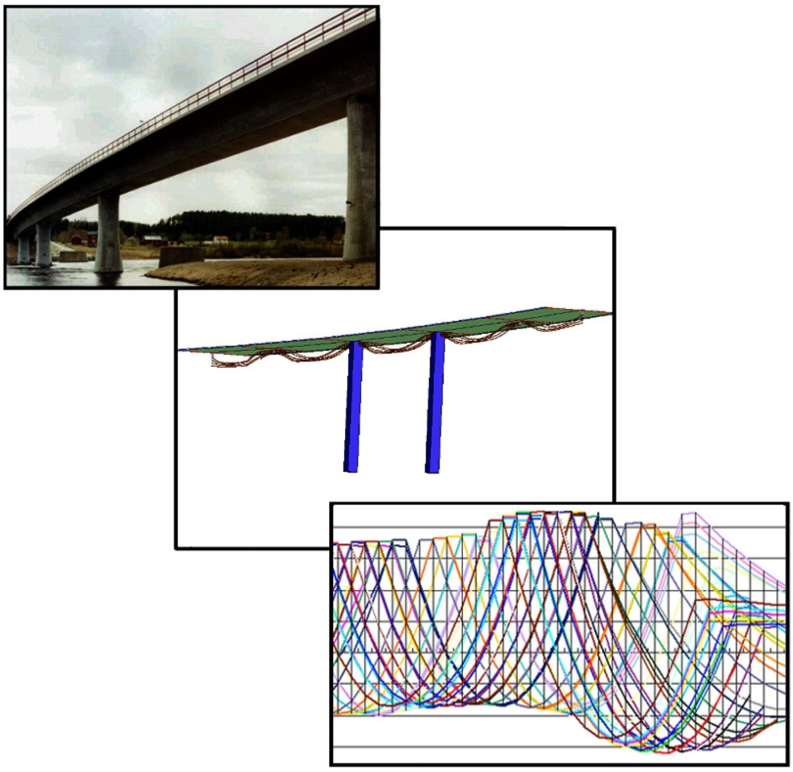

Incremental launching versus scaffolding for construction of prestressed concrete bridges

Master's Thesis in the International Master's Programme Structural Engineering

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Department of Civil and Environmental Engineering
Division of Structural Engineering
Concrete Structures
CHALMERS UNIVERSITY OF TECHNOLOGY

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Cover:
Top figure: The bridge over the river Vindeln, (Anna Lindell, Vägverket, 2005).
Middle figure: Model of the reference bridge in BRIGADE/Standard, Chapter 6.
Bottom figure: Result of the moment envelopes during launching, Chapter 6.

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ABSTRACT

Incremental launching (IL) of concrete bridges is a construction technique that has not been used in Sweden for a long time. The purpose of this master's thesis is to gather information that makes it possible for Skanska Sverige AB to decide if incremental launching is advantageous to use compared to other construction methods. An existing bridge built with scaffolding is used as reference bridge in order to compare incremental launching with scaffolding.

The main task is to compare the needed amounts of concrete and prestressing tendons depending on whether the reference bridge is built with the IL technique or with traditional scaffolding. The cross-section of an IL bridge has to be designed to handle the varying bending moments that are induced during launching. An IL bridge requires therefore larger cross-section, additional prestressing and a launching nose in order to manage the launching process.

To be able to compare scaffolding and IL a model is created of the reference bridge in the finite element software BRIGADE/Standard. In order to create a model as correct as possible values and constants are used from ELU Konsult AB's calculations on the reference bridge. The model is verified through a comparison of the sectional forces between the reference bridge and the model. After verification a simulation of the launching process is carried out in order to find the most critical section forces. From the sectional forces stresses can be calculated in the top and bottom part of the cross-section. A stress control is carried out to make sure that the stresses do not exceed the allowable values in the Swedish code BRO 94.

Different prestressing layouts and cross-sections are combined and analysed in order to find the optimal combination for incremental launching. From the obtained stresses the amounts of concrete and prestressing tendons are calculated for the different combinations of cross-sections and prestressing layouts. The result from the analysis shows that it requires least amount of concrete and prestressing tendons if the bridge is built with scaffolding. However, the difference is less than 10 % between the most optimal cross-section and prestressing layout for incremental launching in comparison with the bridge built with scaffolding. IL makes it possible to construct bridges in inaccessible areas and also in a shorter time than bridges built with scaffolding.

Key words: Incremental launching (IL), scaffolding, launching nose, internal and external prestressing, finite element modelling, stress analysis, amount of concrete and prestressing tendons

Etappvis lansering kontra ställningsbyggande för tillverkning av förspända betong broar
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SAMMANFATTNING

Etappvis lansering av betongbroar är ett byggnadssätt som inte har använts i Sverige på länge. Syftet med detta examensarbete är att ta fram information som gör det möjligt för Skanska Sverige AB att avgöra om det är fördelaktigt att bygga med etappvis lansering jämfört med andra byggnadssätt. En befintlig bro byggd på ställning används som referensbro för att kunna jämföra etappvis lansering och ställningsbyggande.

Huvuduppgiften är att jämföra erforderlig mängd betong och förspänd armering beroende på om referensbron byggs med etappvis lansering eller på traditionellt sätt med ställning. Tvärsnittet för en etappvis lanserad bro måste utformas så att den klarar av momenten som uppkommer under lansering. En lanserad bro kräver därför större tvärsnitt, extra förspänning och en lanseringsnos för att klara lanseringens processen.

För att kunna jämföra de två byggnadssätten skapas en modell av bron byggd på ställning i datorprogrammet BRIGADE/Standard. För att skapa en modell så verklighetstrogen som möjligt används värden och konstanter från ELU Konsult AB:s beräkningar på referensbron. Modellen verifieras genom att jämföra snittkrafterna från referensbron med snittkrafterna från modellen. Efter verifiering utförs en simulering av lanseringsprocessen för att hitta de kritiska snittkrafterna. Från snittkrafterna kan spänningar beräknas i över- och underkant för tvärsnittet. En spänningskontroll utförs för att säkerställa att spänningarna inte överskrider de tillåtna värdena enligt den svenska normen BRO 94.

Olika spännarmeringslayouter och tvärsnitt provas och analyseras för att hitta den optimala kombinationen för etappvis lansering. Från de beräknade spänningarna beräknas mängden betong och förspänd armering för de olika kombinationerna av tvärsnitt och spännarmeringslayouter. Resultatet från analyserna visar att det krävs minst mängd betong och förspänd armering om bron byggs på ställning. Skillnaden är dock mindre än 10 % mellan optimerad spännarmeringslayout och tvärsnitt för etappvis lansering i jämförelse med den ställningsbyggda bron. Etappvis lansering gör det möjligt att bygga broar i otillgängliga områden och på kortare tid än broar byggda på ställning.

Nyckelord: Etappvis lansering, ställningsbyggande, lanseringsnos, intern och extern förspänning, finite element modellering, spänningsanalys, mängd betong och förspänd armering

Contents

Appendices

Preface

This master's thesis focuses on a stress comparison between a bridge built on scaffolds and a simulation of the same bridge when it is constructed with the incremental launching technique. The project has been executed at Skanska Teknik´s office in Göteborg between June and November 2005 in cooperation with the Department of Civil and Environmental Engineering, Chalmers University of Technology.

We would firstly like to thank the initiator, as well as our supervisor, M.Sc.C.E. Karl Lundstedt for all guidance of our master's thesis. Also special thanks to the reference group consisting of M.Sc.C.E. Tech. Lic. Gunnar Holmberg, M.Sc.C.E. Tech. Lic. Jan Olofsson and M.Sc.C.E. Claes Bergsten. We would first of all thank them for the given opportunity to carry out the study at Skanska Teknik and develop a deeper knowledge in bridge design and construction. They shall also be thanked for their support throughout the duration of the project. The staff at Skanska Teknik's office in Göteborg shall also be thanked for their help and support. Special thank to M.Sc.C.E. Marcus Davidson, structural designer at Skanska Teknik, for always being available and his expert guidance regarding the software BRIGADE/Standard.

We would also like to thank the people involved in this master's project in one or another way. Especially M.Sc.C.E. Jiří Bešta and M.Sc.C.E. Pavel Smíšek at VSL Systems in the Czech Republic and M.Sc.C.E. Karl-Erik Nilsson at Internordisk Spännarmering for the very informative site visit to the Rybný Potok bridge. Jan Olsson and Johan Kölfors at Scanscot Technology AB for their patience and support with BRIGADE/Standard. Anders Bengtsson at ELU Konsult AB for providing us with information about the bridge across Vindeln.

We would also like to thank our examiner at Chalmers University of Technology Professor M.Sc.C.E Ph.D. Björn Engström for his help and support with our master's thesis. A special thank to our opponent Camilla Juthage for her help and comments during the project. Our colleagues Erik Karlsson and Erik Lööv shall also be thanked for their help and support throughout the project. Finally we would like to express gratefulness to our families and friends.

Göteborg, November 2005

Boldi Kisch & Per Langefors

Notations

Roman upper case letters

A	Cross-sectional area of concrete
A_s	Cross-sectional area of tendon
E	Young's modulus
H_{IL}	The depth of the superstructure in an IL constructed bridge
H_S	The depth of the superstructure in a bridge constructed with scaffolding
I	Moment of inertia
L	Length
M	Bending moment
M_{PS}	Bending moment caused by the prestressing
$M_{V:A}$	Bending moment caused by load case V:A
N	Normal force, axial force
$N_{V:A}$	Normal force caused by load case V:A
P	Force
P_{PS}	Axial force caused by the prestressing
W	Principal section modulus

Roman lower case letters

f_{cck}	Characteristic concrete compressive strength
f_{ctt}	Concrete tensile strength
f_{st}	Steel tensile strength
z	Lever arm
q	Deadweight

Greek letters

σ	Stress
σ_c	Compressive stress in the concrete
σ_{ct}	Tensile stress in the concrete
σ_r	Tensile stress at the same level as the prestressing tendons

1 Introduction

For thousands of years people have had the need to cross over obstacles. In the beginning fallen trees were used by people to cross over a small stream of water. Later on arches made of stones and suspension bridges made of vines and creepers made it possible to pass rivers and ravines.

Most arch bridges were made of stone, masonry or iron. Nowadays it is more common that arch bridges are built of reinforced concrete. Suspension bridges can be used if long spans are needed. This is however rather expensive. The most economical and common way of designing and building bridges is to use beam elements. Beam bridges allow spans up to 150 m - 200 m depending on if prestressed concrete or a steel box girder is used. These types of bridges are an economical way of building both small and long span bridges (The Hong Kong Polytechnic University, 2005).

One way of building beam bridges is with prestressed concrete. When prestressing is used, tensile stresses in the concrete are delayed or prevented in the service state and by this cracking is avoided. In Figure 1.1 four common bridge types are visualised.

*Figure 1.1 Various types of bridges **Top left:** Cable-stayed bridge, Kap Shui Mun bridge, Hong Kong. **Top right:** Balanced cantilever bridge, Lavanttal bridge, Austria. **Bottom left:** Arch bridge, Pitzalbrücke, Austria. **Bottom right:** Suspension bridge, Kwang Ahn bridges, Korea (TDV 2005).*

1.1 Problem description

The Swedish contractor Skanska Sverige AB is interested to find out the advantages with the building method called incremental launching (IL) of prestressed concrete bridges. Incremental launching is a construction technique where the bridge is built in segments behind one of the abutments and moved forward successively. IL is not a common way of building in Sweden but it is frequently used in countries abroad, like Germany and Poland.

In June 2005 Erik Karlsson and Erik Lööv carried out an interview study on why incremental launching is not used in Sweden. It seems like the general opinion among designers in Sweden is that IL is not the most economical way to construct bridges today (Karlsson and Lööv 2005). Incremental launching requires some additional equipment and a larger cross-section than a bridge built on scaffolds. Perhaps there are not enough new projects suitable for IL, so the equipment cannot be reused in a satisfactory way? This master's project will compare the amounts of concrete and prestressing tendons needed for incremental launching and scaffolding.

Until now only four bridges have been built in Sweden with IL. Hopefully the result of this master's thesis can be valuable for the future progress of the incremental launching technique in Sweden.

1.2 Aim and scope

The aim of this master's project is to investigate and compare an existing traditionally built bridge on scaffolds with the same bridge built with IL, as a basis for decision-making at Skanska Sverige AB. Is it with Swedish codes advantageous to use incremental launching for the studied bridge? How are the needed amounts of concrete and tendons in the superstructure influenced by the production method?

The main focus of the master's project was to determine sectional forces such as axial forces and bending moments from computational analysis using a detailed model. From these data the important stresses needed to design the bridge superstructure should be calculated. Different prestressing layouts should be studied in order to find the most effective arrangement. From the stresses it should be possible to obtain the amount of concrete and prestressing needed for incremental launching and compare it with traditional bridge construction on scaffolds.

A detailed description of the launching nose should also be carried out. Information about the launching nose should be gathered from literature studies and from a study trip.

1.3 Limitations

The analysis will only be carried out on the superstructure of the reference bridge, while no effort will be put into analysing the deck, columns and foundation. Although these parts are included to some extent in the model they are only specified in order to define the boundary conditions for the superstructure more accurately. The superstructure is the most critical part when using incremental launching. The choice of construction method does not affect other parts as columns and foundation of the bridge in a significant way according to Karl Lundstedt. A different bearing is needed during launching but this is not considered in this thesis work.

Ordinary reinforcement is not considered at all in this master's project. The focus is on the longitudinal prestressing tendons. Different layouts of external and internal prestressing should be studied and compared. Larger amount of prestressing tendons might also be necessary in comparison with a traditionally built bridge, in order to deal with the high moments during launching of the bridge.

1.4 Method

Information was to be gathered from literature studies and a study trip to a bridge in the Czech Republic. The focus should be on FEM modelling and the structural response during the construction with IL of a concrete bridge. The optimal layout of prestressing tendons (temporary and final) should be evaluated. From this result the needed amounts of concrete and tendons can be calculated.

1.5 Outline of content

Well-known bridge construction techniques are presented and explained in Chapter 2 with special focus on incremental launching. In Chapter 3 the procedure when choosing the reference bridge is presented. Location and geometry for the bridge is also stated. The software that ELU Konsult AB used for design of the reference bridge is described. A description of the experiences from the study trip to Czech Republic is also presented in Chapter 3. Special focus is put on the launching nose used during incremental launching in Chapter 4. An introduction to prestressing and different prestressing schemes depending on whether the tendons are used to balance forces during erection or during the service state is explained in Chapter 5. Two prestressing alternatives are described in Chapter 5. The finite element model is presented in Chapter 6. In this chapter the geometry, material properties, loads and simplifications are explained as well as the working procedure used creating the model. The verification of the model can also be found in Chapter 6 as well as an analysis of the bending moments in the superstructure during launching and in the service state. Chapter 7 deals with the structural calculations carried out in order to determine the stresses during launching and in the service state. The simplifications and design criteria for the model are also clarified. Results from the stress analysis is presented and analysed in Chapter 7. The amounts of concrete and prestressing tendons are calculated in Chapter 7. In Chapter 8 the final conclusion of the master thesis is presented.

2 Different bridge construction techniques

In the middle of the 20^{th} century the steel prices begun to rise, and about the same time labour costs in the industrialised countries became a substantial part of the construction cost of a bridge. As a consequence of this, an interest for studying new more efficient ways of erecting bridges arose. The unique circumstances surrounding each project often decide which construction technique is the most efficient. This chapter will briefly describe the most common techniques used when constructing box girder prestressed concrete bridges. When designing a concrete bridge, several factors need to be considered:

- Economical aspects for both material and labour

- Regulations and design codes of practice that need to be fulfilled

- Advanced designs that shall be accomplished

In Section 2.1-2.3 the construction techniques of balanced cantilever, scaffolding, falsework, travelling gantry and girder are described. Incremental launching (IL) is covered in Section 2.4.

Important to have in mind is that the columns for a bridge have to be built with scaffolding independently of the construction method for the superstructure.

2.1 Scaffolding and falsework

Perhaps the most basic way of erection is to use scaffolds or falsework to support the construction of a bridge, see Figure 2.1. This technique gives also high flexibility to the design of the bridge, since the actual segment of the superstructure will be supported until the construction is completed. This means that the most slender and effective bridge designs often are constructed using scaffolding or falsework.

When a bridge is constructed using scaffolds closely spaced cross-braced struts are used. These are designed to support the load directly from the formwork down to the ground. The scaffolds are most often combined of standardised reusable cross-braced modules of steel that can be used over and over again (fib 2000). However, scaffolds of timber are often used because they are flexible and cheap.

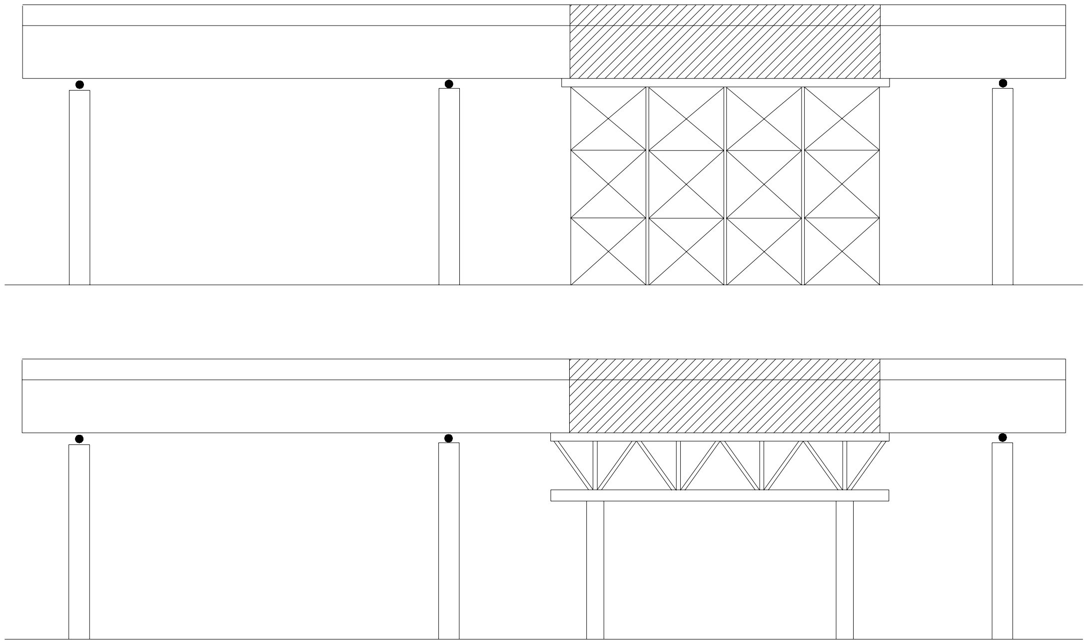

Figure 2.1 ***Top:*** *Bridge constructed with scaffolding.* ***Bottom:*** *Bridge constructed with falsework.*

The main difference between scaffolding and falsework is that falsework requires larger and heavier temporary supports for the superstructure. Instead of supporting the load via a large number of cross-braced struts, steel beams between temporarily established supports are used. Spans can have a range of 10 to 20 m. Falsework can therefore be used when obstacles like small rivers or roads need to be reached over (fib 2000). Scaffolds on the other hand can only be used if it is possible to place the temporary steel posts all the way under the superstructure. Both methods mentioned above are suitable to use if the free height of the bridge is not greater than up to 6 m.

2.2 Travelling gantry/girder

Scaffolding is a suitable choice for construction of short bridges. For bridges that are considerable longer the scaffolding needs to be moved between the different sections of the bridge during construction. This has developed the travelling gantry technique. The construction method uses a movable supporting beam, gantry, for the falsework that reaches over at least one span but usually over the length of two spans. With the supporting beam in place, transverse beams along the gantry secure the formwork and working platform and the building process can be carried out efficiently. With special roller bearings and launching jacks the gantry can easily be moved forward along the bridge as the construction proceeds. The travelling gantry system is most suited for spans of 30 to 60 m (fib 2000).

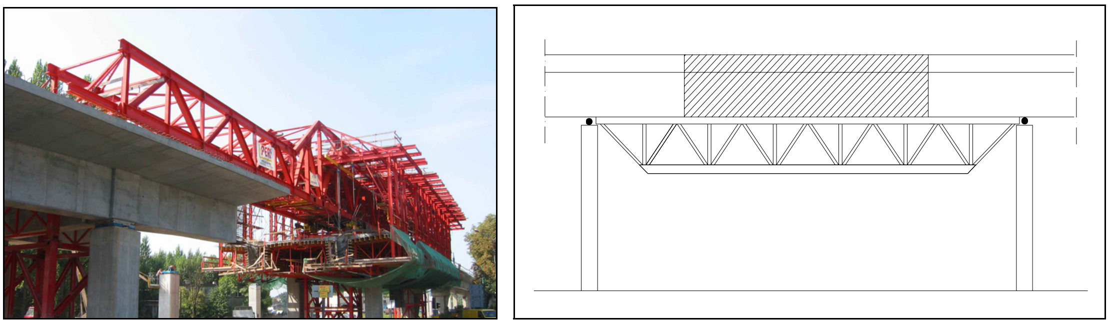

Figure 2.2 ***Left:*** *Travelling gantry (Skanska SA in Poland).* ***Right:*** *Travelling girder.*

When the supporting beam is placed below the superstructure, see Figure 2.2, the method is called travelling girder. The technique is very similar to the travelling gantry but more material is needed when the girder is placed below. A gantry is therefore cheaper and lighter with less material, utilising a more optimal design since the structural depth is larger compared with a girder (fib 2000).

These two systems are often used with precast concrete elements for fast erection. In this way several spans can be completed within a week, compared to insitu construction where one span per week can be built at its best (fib 2000).

2.3 Balanced cantilever

One of the most popular and frequently used building methods today is the balanced cantilever technique. Starting from a supporting pier the superstructure is produced step-by-step symmetrically outwards. This can be achieved both with precast and insitu construction. The segments support each other by cantilevering out from the support on opposite sides, see Figure 2.3. This method is often used for long spans but has a possible span range between 50 and 300 m (fib 2000).

Figure 2.3 *Balanced cantilever construction," Lavanttal bridge" (TDV 1984).*

The bending moments that arise during construction using this method can be up to five times larger then the moments created from the travelling gantry system. This often makes it necessary to use some kind of temporary support or cable-stays along the way. Due to the fact that the cantilevers should balance each other out, it is important that only small segments are added on each side of the support. Usually insitu segments not larger than 3 to 4 m or prefabricated segments between 1.8 and 3.5 m are used to limit the additional bending moments per step in order to avoid unbalance in the system (fib 2000).

2.4 Incremental launching

Incremental launching is another possibility to reduce the costs for construction of bridges longer than 150 m and with spans between 30 m - 60 m (VSL International Ltd. 1977).

Behind one of the abutments a construction yard is established, where one segment of the superstructure is manufactured at each time. Each segment is prestressed to the previous part of the structure and after hardening the bridge is launched forward the length of one segment. This building method is rather industrialised since the formwork can be reused and therefore less labour cost is needed. Also temporary piers, bearings and the launching nose can after a finished job be reused in new projects later on (VSL International Ltd. 1977).

One major advantage of incremental launching is that no scaffolding or falsework is used when the superstructure is constructed. The bridge can therefore pass over obstacles like rivers, buildings, railroads, etc without any problems see Figure 2.4. Another advantage is that the construction yard can be covered in order to enable a more protected environment against different weather conditions. The production of the bridge in the construction yard makes it easier to overview and control the production and leads to higher quality. Separate working steps recur in continuous cycles. Another benefit with incremental launching is the spared local transportations of all material and staff, since the whole construction of the superstructure is located in the casting yard (Rosignoli 2002).

Disadvantages of IL are the additional costs for launching jacks, launching nose, additional prestressing, the construction yard and extra amount of concrete needed to increase the cross-section due to extra stresses caused by the launching. It might be necessary to reuse equipment for projects in the future to make the present one economical (Göhler and Pearson 2000).

Figure 2.4 Launching nose during erection of the Gebergrund bridge in Germany (Doka 2003).

Incremental launching is a building technique where large amount of prestressing is needed to avoid tensile stresses in the concrete. When the bridge is launched a high bending moment is created from the cantilever. As seen in Figure 2.5 the sign of the moment differ between a support region and a span region. Since each cross-section must resist these continuously alternating bending moments centric prestressing is used. To reduce the moment from the cantilever a lightweight-launching nose of steel is normally used. For more information about the launching nose see Chapter 4.

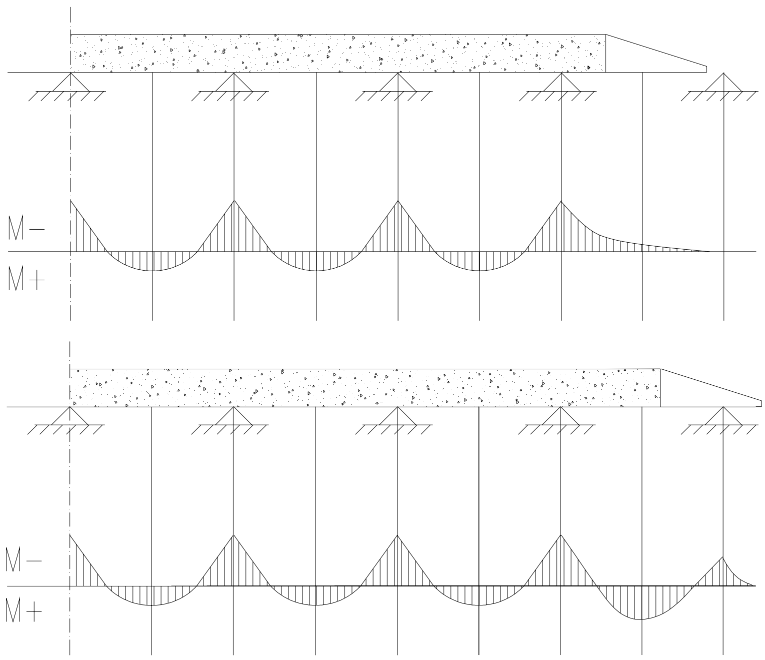

Figure 2.5 Bending moments during bridge launching.

The construction sequence in the casting yard is carried out according to the following steps:

1. Casting of the bottom slab
2. Casting of the webs
3. Casting of the deck slab
4. Further hardening of the concrete
5. Tensioning of the tendons
6. Launching of the bridge segment

The bridge segments are constructed in lengths that are determined so that the construction sequence can be approximately one week. The segment is pushed or pulled forward by means of hydraulic jacks. Low friction bearings on the piers make it possible to slide the heavy superstructure forward. It is important that the abutment can resist high horizontal forces from the hydraulic jacks during launching, especially in a late state of erection when the jack has to move a very long bridge forward (VSL International Ltd. 1977).

Incremental launching are preferable to use when the bridge is straight or has a constant curvature throughout the length, since all segments has to be of equal size and shape (it is possible to have a small variation of the segments, but it is not a standard procedure). Bridge segments with different curvatures can be launched from opposite abutments and connected in the middle of the bridge. Greater mid spans can be constructed when launching is performed from both abutments (Rosignoli 2002).

3 Reference bridges

Two bridges were studied in order to acquire enough knowledge to be able to carry out the stress comparison between a bridge built on scaffolds and an incrementally launched one. The bridge described in Section 3.1 was used as the reference bridge. A study trip was also conducted to the Rybný Potok bridge which is described in detail in Section 3.3.

The process of choosing a reference bridge included a brief study of eight different bridges built in Sweden. Common for all these bridges was that they were prestressed concrete box girder bridges built using scaffolding. However, all of these bridges were not suited for the incremental launching technique, since some of them were too short others were not suited for a comparative study because they were too old and therefore built with old regulations that would not allow a good comparison for the master's project. Different advantages and disadvantages for the eight bridges were discussed with Karl Lundstedt (supervisor for the master's project) in order to find the best-suited reference bridge. Finally the bridge across the river Vindelälven in Vindeln was selected, see Figures 3.1 - 3.3 below.

3.1 The bridge over the river Vindeln

The major criteria why this bridge was chosen as reference bridge were:

- A box girder cross-section. The cross-section is visualised in Figure 3.4
- Suitable spans; 41 m in the two end spans of the bridge and 54 m in the other spans, see Figure 3.2
- The total length of the bridge (244 m) makes the bridge suitable for IL
- A constant radius of 900 m makes it possible to use IL
- Built according to the Swedish code BRO94, edition 2 (relatively modern code)

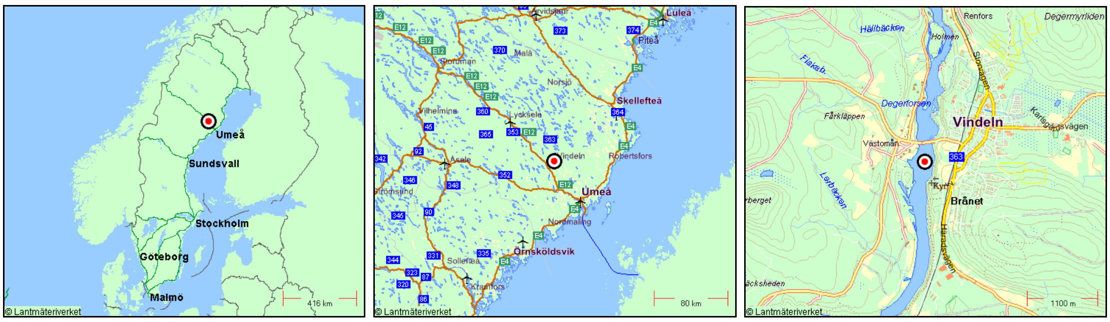

Figure 3.1 Location of the reference bridge (www.eniro.se).

When the reference bridge was built it replaced an old steel bridge built in 1922 that did not fulfil the current requirements for load carrying capacity concerning heavy road traffic. ELU Konsult AB carried out the design of the new bridge, which was built in 1997. ELU Konsult AB supplied some of the drawings and calculations concerning the reference bridge. As mentioned earlier, the bridge across Vindelälven is a prestressed box girder bridge built on scaffoldings. It consists of five spans see Figure 3.2. The total length of the bridge is 244 m and the bridge has a constant horizontal radius of 900 m and vertical radius of 9500 m.

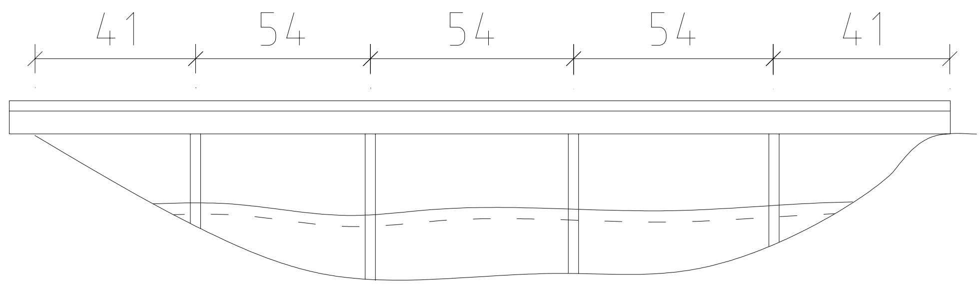

Figure 3.2 Elevation of the reference bridge.

Figure 3.3 The bridge across Vindelälven (Anna Lindell Vägverket 2005).

The reference bridge has two different cross-sections; one for support regions and another one for span regions see Figure 3.4. The cross-section properties were calculated with Section Editor in the FEM Design software and verified with the calculations from ELU Konsult AB. The Section Editor makes it possible to define a cross-section and calculate the cross-sectional properties. Results from these calculations can be found in Appendix 1.

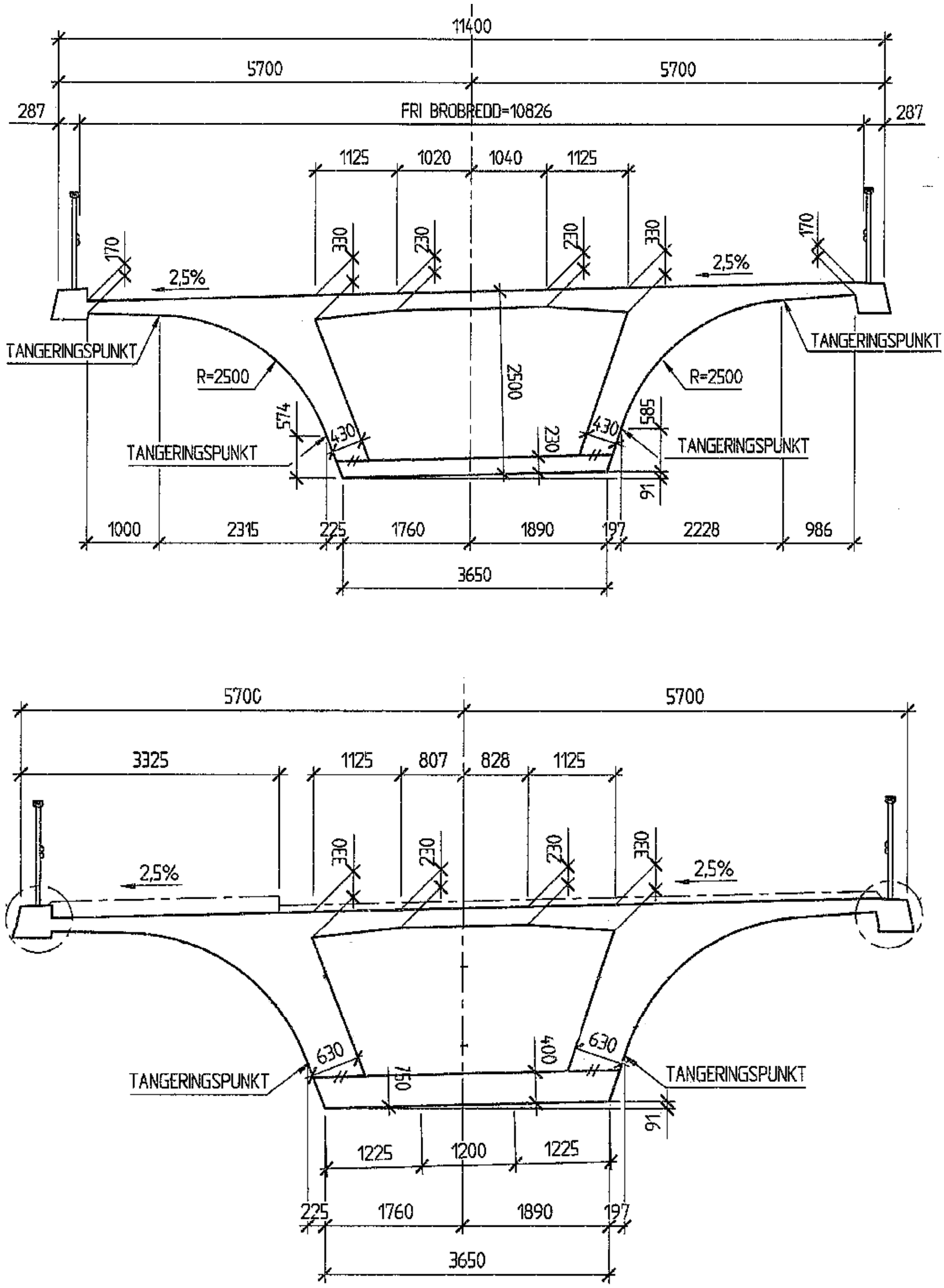

Figure 3.4 ***Top:*** *cross-section in span regions* ***Bottom:*** *cross-section in support regions, notice the in circled wing walls. These walls are not considered in this master's thesis.*

3.2 Structural analysis used for the design

ELU Konsult AB used the software ‘Strip Step 3’ for structural analysis of the bridge over Vindeln. This is an old program for frame analysis that has been under constant development since the 1960’s. Strip Step 3 is a completely text based software with no graphic options.

Strip Step 3 handles three-dimensional structures like trusses and frames. It manages an arbitrary geometry on a global level and a varying cross-section. The members can be eccentrically attached to each other and the joints can be elastic. Relevant loads such as, surface loads, concentrated loads, support settlement, thermal load and traffic loads are included in Strip Step 3. Load cases can be combined arbitrary into envelopes. Creep deformations under constant loads considering relaxation the prestressing tendons can be estimated (NORDCAD AB 1989).

The result output from Strip Step 3 includes deformations and sectional forces like bending moments, shear forces and normal forces. It delivers influence lines for section forces, deformations and reactions both for external and internal forces (NORDCAD AB 1989).

Strip Step 3 is based on theory of elasticity (Hooks law) with linear relation between stress and strain. Furthermore, plane sections remain plane according to Bernoulli hypothesis. All deformations are treated as being very small. Like conventional methods of calculation the law of superposition is utilised (NORDCAD AB 1989). The law of superposition states that if two solutions are given to one equation, then the sum of the solutions will be a third solution to the equation (Wikipedia 2005).

Calculations can be carried out with normal force and shear force. The numerical integration that determines element- and load constants is performed with the Simpson method where the steps do not exceed 1/20 of the element length (NORDCAD AB 1989). The Simpson method is an accurate way to reach a value through iteration where information from four previous points is used when computing the following point (Mathews 2003).

3.3 The bridge over Rybný Potok

In October 2005 a study trip was conducted to the Czech Republic and a bridge that was constructed with the incremental launching technique. The study trip was organised by Karl-Erik Nilsson from Internordisk Spännarmering in corporation with Jiří Bešta and Pavel Smíšek at VSL. VSL is one of the leading companies concerning prestressing systems and were among the pioneers using the incremental launching method.

The bridge, seen in Figure 3.5, is situated 200 km from Prague (near the border to Germany) and is a part of the D8 highway. The highway will when the project is completed, be a link between Dresden and Prague. Skanska is the main contractor for this project, but a subcontractor called Metrostav construct the bridge. The design is made by SHB (a local design company), while the launching and prestressing is made by VSL.

Figure 3.5 Two different views of the bridge over Rybný Potok.

The bridge reaches over Rybný Potok (*full of fish creek*) and has a total length of 356 m, the span lengths can be seen in Figure 3.6 below. The superstructure of the bridge consists of a box-girder with the deck on top. The deck is quite wide (30 m) and therefore prefabricated inclined struts were used in order to transfer the vertical load into the girder. The large width of the bridge hints that it might be better with two separate bridges next to each other, however according to the designer at SHB it is more economical with one large bridge.

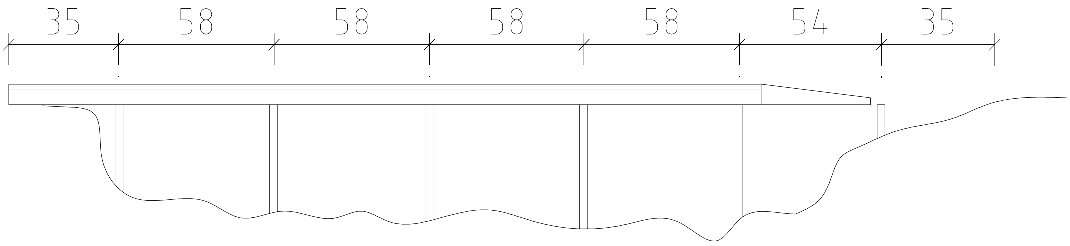

Figure 3.6 Elevation of the Rybný Potok bridge.

3.3.1 Construction yard

The incremental launching part of the construction work started in June 2005 and was completed in October 2005. Like described in Chapter 2, Section 2.4 the casting of the superstructure was carried out in a casting yard located at the lowest abutment of the bridge. The casting yard was not covered since casting and launching of the bridge was carried out during the summer of 2005 and therefore favourable weather was predicted. Although heavy rainfall during casting of one segment resulted in some smaller quality problems. The surface was not as smooth for that specific segment as for the rest of the bridge.

The yard was divided into two parts, in the first section the bottom flange and the webs of the box girder were cast. In the second one the deck was cast and the prefabricated diagonal struts between the web and the deck, seen in Figure 3.8 below, were assembled. The casting yard was about 60 m long since in each segment except the segments at the beginning and end of the bridge had a length of 30 m. The two segments at the ends of the bridge had a length of 24 m in order to avoid joints close to the supports. As seen in Figure 3.7, the casting yard was positioned on top of a steel grid made out of transversal and longitudinal beams. The steel grid rested on 12 hydraulic jacks.

Figure 3.7 ***Left:*** *View of the casting yard.* ***Right:*** *The steel grid is resting on hydraulic jacks.*

Hydraulic adjustable formwork was used for the outer sides of the webs. For the inside of the webs and top flange the formwork was made out of movable carts. The web thickness was constant 600 mm along the whole bridge.

The concrete used in this bridge corresponds to strength class K45 in the Swedish standard, with the concrete density of 28.5 kN/m^3. No reinforcement cages were used but all the ordinary reinforcement was placed and mounted by hand. Since the bridge is rather slim but still has a wide deck 250 kg ordinary reinforcement was used per m^3 concrete. Transversal prestressing reinforcement bars were placed with a spacing of 0.5 m. Extra amount of ordinary reinforcement was used for the first 50 m of the bridge. This was necessary in order to resist the increased moments in the front zone due to the cantilever action while launching the bridge.

Heels were used to anchor the external polygonal tendons inside the box girder. The reinforcement for the heels was assembled in the casting yard, see Figure 3.8, but the heels were not cast until the launching of the bridge was completed. In this way the weight of the superstructure could be kept as low as possible during launching. Diaphragm walls over each support provided saddle points for the external tendons.

The working cycle for this bridge was a ten-day process, divided into 12 hours shifts. The work proceeded 24 hours a day.

Figure 3.8 ***Left:*** *Reinforcement for heels assembled in the casting yard.* ***Right:*** *The diagonal struts between the box and deck of the superstructure.*

3.3.2 Design criteria

The design criteria during launching allowed 2 MPa in tension (compare to 1 MPa used in BRO 94). Maximum crack width allowed in the service state was 0.1 mm. When the launching of the bridge was completed and the tendons were stressed the whole bridge is in compression. The diagonal struts positioned along the bridge were designed to resist 8 MPa in compression but have a capacity of 20 MPa. The struts can be seen in Figure 3.8 above.

During launching the tolerance for settlement of the supports was set to 10 mm. In the service state the same tolerance was increased to 20 mm. The supports in the middle of the bridge were designed as flexible piers.

According to the designer at SHB the most difficult part of the superstructure design was the intersection between the wide deck and the small box. The torsion acting on this part was the most challenging problem to solve.

3.3.3 Prestressing

For the bridge over Rybný Potok a prestressing arrangement very similar to prestressing alternative 2 described in Section 5.5.2 was used. This means that straight internal prestressing was used to manage the launching and external polygonal tendons were used for the service state. Each centric tendon is 2 segments long and half of the tendons are anchored at every second segment. As mentioned in Section 3.3.1 the casting of the cross-section was carried out in two steps. Since the deck was cast in the second step it was necessary to put some of the prestressing in the webs in order to deal with the bending moments during the first step.

Since the bridge over Rybný Potok is rather short the external tendons could be continuous along the whole bridge length without any overlapping joints. The external polygonal tendons were assembled and tensioned after the bridge was launched and the heels were cast.

3.3.4 Launching procedure

When casting of a segment was completed and the concrete reached required strength the launching procedure could begin. First the steel grid was lowered slightly by the hydraulic jacks and at the same time a support in the middle of the section was raised. The support had a low friction sliding bearing on the top. The same type of bearing was used on top of the columns. When the superstructure was clear of the steel grid the bridge could be launched forward on the sliding bearings. The bridge was launched uphill with a slope of 3 % and a horizontal radius of 1700 m, the radius of the vertical alignment was 2400 m. When the current segment was launched the steel grid was free and fabrication of the next segment could begin.

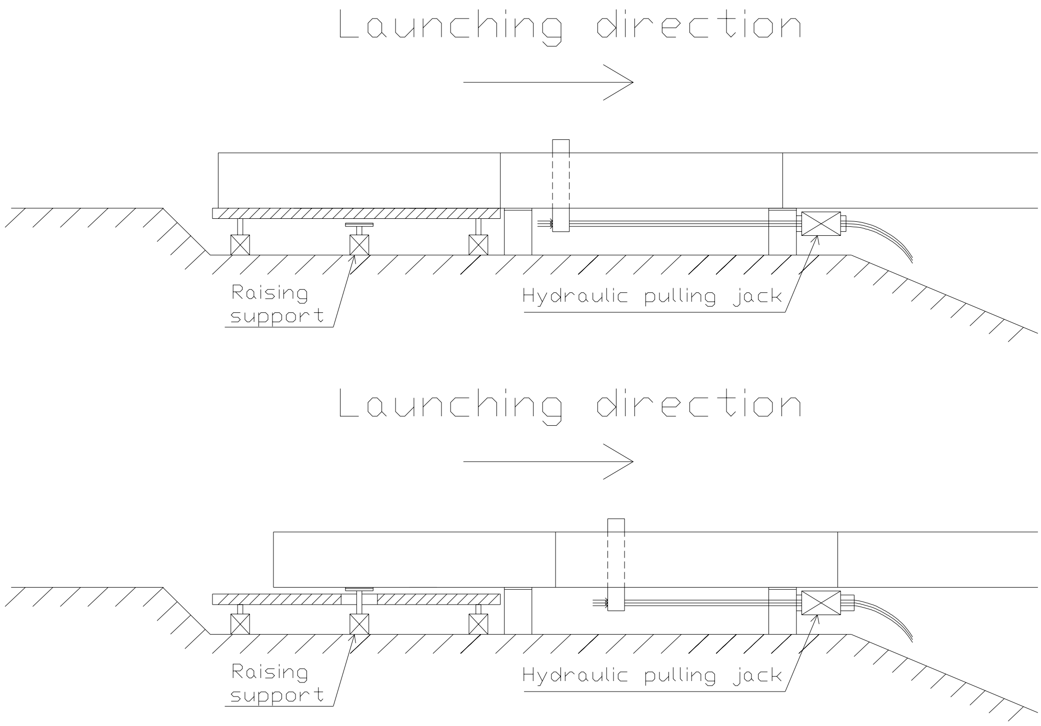

Figure 3.9 Schematic sketch of the launching procedure (Karlsson and Lööv, 2005).

A lift and pull system was used instead of a lift and push system, which is more frequently used according to Pavel Smíšek at VSL. The launching procedure can be seen in Figure 3.9. At the end of every new segment four steel brackets were assembled, see Figure 3.10. The brackets were mechanically attached to the bridge segment by passing through premade holes in the flanges of the box girder. These steel brackets should secure a proper grip of the segment during the launching. At the bottom of each steel bracket a 31-strand tendon was attached and linked to a horizontally positioned hydraulic jack. All together four jacks pulled the strands when the bridge was launched forward. Each pull was 200 mm long and therefore the launching speed was about 6 m/hour. An advantage with the lift and pull system is that the connection between the jacks and the bridge is not based on friction. The mechanical attached steel brackets provide a solid connection that can move heavier bridges than the friction based system. Another advantage with this system is the constant prevention of back sliding created by the tensioned strands.

Figure 3.10 Steel brackets mechanically attached to the concrete segment. The steel tendons pulling the bridge forward during launching are also visualised.

During the launching two workers at each support were checking the low friction pads. One person managed to control the hydraulic jacks that pulled the bridge forward. A steel wire rope reaching between the two ends of the bridge was used to make sure that the launching process did not diverge. If the launching nose diverged more then a certain distance, the launching would automatically stop.

3.3.5 Launching nose

The launching nose was 35 m long, which corresponds to 60 % of the longest span. Since it was such a massive bridge, the nose was specially designed for this project. The nose was designed in such way that it is easy to dismantle and reuse it for other bridges with smaller dimensions according to Jiří Bešta and Pavel Smíšek at VSL. The large steel beams are equipped with one hydraulic jack each in the front part of the launching nose. The hydraulic jacks could compensate for the maximum deflection (200 mm) of the launching nose.

The nose was attached to the box girder with prestressing bars, see Figure 3.11 below. This caused large strains in the box-girder and as described in Section 3.3.1 an extra amount of reinforcement was added to the first 50m of the bridge.

Figure 3.11 ***Top:*** *The launching nose.* ***Bottom:*** *The joint between the launching nose and the superstructure.*

4 The launching nose

The role of the launching nose is more important for a heavy superstructure than for a lighter one. During the last 30 years launching noses have been designed in a large variety of different ways; they have been built up by truss beams or plate girder beams; they have been made out of concrete or steel; they have had front realignment sledges or a hydraulic nose lifting system in order to compensate for the elastic deflections. Sometimes these noses have been designed out of the best knowledge and experience available but most often old noses have been reused (Rosignoli 2002). An example of a launching nose is visualised in Figure 4.1.

Figure 4.1 Launching nose used during launching of the Rybný Potok bridge.

The launching nose is one of the biggest investments when erecting a bridge with incremental launching. Therefore it is very important to design and construct the nose so it can be reused in other IL projects. The launching nose has to be constructed in such a way that only a small amount of work is required to adjust the nose for the next project (Göhler and Pearson 2000).

4.1 Structural layout

The optimal length of a launching nose is 60 % of the longest span according to Göhler and Pearson (2000), however Rosignoli (2002) claims that 65 % is the most optimal. If a shorter nose is used, the cantilever moment on the bridge superstructure will increase, which demands more centric prestressing. If a very large amount of centric prestressing is used, then problems will occur concerning the arrangement of all the prestressing tendons. The large amount of prestressing will also result in high compressive forces in the bottom of the flange. A longer nose will reduce the cantilever moment, but will not affect the overall cost for the prestressing so much that it is worth the effort of creating a longer nose (Göhler and Pearson 2000). The nose needs to be very stiff with a small weight therefore it is more difficult to create such a nose when it gets very long. When the nose is reused in another project, it will often be necessary to change the length of the nose to fit the current situation, since the nose should not be shorter than 60 % of the longest span. If it is shorter additional segments has to be added to the nose. When a nose is too long it will often also be too high in the end that will be attached to the bridge deck. The solution to this problem is to add two concrete blocks behind the top flanges of the main nose beams. These blocks will be vertically prestressed against the bridge deck and the nose will be longitudinally prestressed against these blocks. If a reused launching nose has to be widened considerably it will probably be more cost effective to make a new nose. The nose and the nose bracing, should however be designed in such a way that a future widening will be as easy as possible (Göhler and Pearson 2000).

Since a launching nose can be used in several projects its cost can also be amortised over several projects. According to Rosignoli (2002) it may therefore be smart to design the nose from the beginning for the longest probable span that might appear in future projects. By dividing the nose into spliced segments its length can easily be adjusted (Rosignoli 2002).

Most often a launching nose is designed with two main longitudinal girders at each side. These are often huge I-beams with an inclined top flange see Figure 4.2.

Figure 4.2 Example of launching nose with I-beams.

According to Rosignoli (2002) a launching nose design based for a current bridge should have the same depth as the bridge at the section of attachment and further on for a few metres. It is only in this segment that large negative moments will occur and only here the full flexural stiffness is needed. As a result the rest of the nose can be made lighter by decreasing the height and web thickness of the main girders (Rosignolli 2002). Some launching noses have been designed with truss-beams for the main girders instead of large I-beams. Such truss-noses result in very large reaction forces in the bottom chord of the main truss beams. This means that the bottom struts in the main truss beam need to be very strong and heavy. According to Göhler and Pearson (2000) this always makes a truss-nose more expensive than a launching nose made of I-beams (Göhler and Pearson 2000). Another drawback for truss noses is the large amount of manual welding required (Rosignoli 2002).

There are always high stresses in the bottom part of the launching nose close to the bridge superstructure. According to Rosignoli (2002) these stresses are:

- Longitudinal flexural stresses caused by the cantilever and the continuous beam action

- Vertical compressive stresses caused by the support reactions led into the webs

- Shear stress on the shear keys welded on the end nose plate

- Stress due to prestressing used for anchoring the nose to the superstructure

4.2 Bracing

Although it is very important that the launching nose is very stiff, it should better not be braced horizontally both in the top and the bottom. It becomes too expensive to adjust the length of the nose between different projects if there are both top and bottom bracings. It is common practise to brace the nose horizontally only in the bottom with diagonal vertical bracings. Transverse vertical bracings are positioned throughout the nose to brace the top flanges of the main beams. Vertical elements are integrated in the web of the main I-beams in these locations (Göhler and Pearson 2000). The vertical bracings distribute uneven load effects between the two girders, caused by non uniform support reaction. The bracings will also reduce distortion when the nose is twisted. Another task for the bracings is to carry the walkway used by the workmen, often positioned in the middle and at the top of the nose (Rosignoli 2002).

The launching nose should have the same width as the bottom of the box girder since it uses the same support bearings (Göhler and Pearson 2000). Since most bridges are designed with different widths it is always a need for adjusting the width of the nose. The cost for this additional work, new horizontal bracings and cross frames should always be accounted for when using a previously built launching nose (Rosignoli 2002).

The web thickness of the main I-beams is determined with regard to the shear force and safety against buckling; most often the web thickness will be 20 mm or greater. Vertical stiffeners will not brace the bottom flange due to the fact that elastic deformations in the bottom help compensate for local deviations caused during manufacture. The width and thickness of the bottom flange will be determined by the tension and compression at the bearings as well as the transverse bending of the flange. Often a width of 300 mm is used according to Göhler and Pearson (2000). Because of the defects and potential differences in load effect due to, in some cases, narrow spaced main girders, each nose girder needs to be designed for 75-100 % of the support reaction transferred by the nose. The same is valid for the local prestressing system used for connecting the nose to the bridge superstructure (Rosignoli 2002).

4.3 Joints and connections

The main nose girders must be designed in such a way that they can be divided in two or three parts. This is necessary when transporting the nose. One way to attach the parts to each other is with high strength bolts. Another method is to use prestressing bars through the launching nose (Göhler and Pearson 2000). It is important that it is quite easy to take the joint apart when the launching is completed. The connection between the nose and the concrete bridge deck is done with prestressing bars or tendons see Figure 4.3. Prestressing bars are more common then tendons, mainly because it is difficult to relieve the tendons for monitoring and maintenance. Within about 5 m next to the bridge deck, or twice the length of the box depth, the webs of the main beams are widened in order to provide place to anchor the prestressing bars and tendons (Rosignoli 2002). If tendons are used they are covered with grease for easy removal when the launching is complete. Often the tendons are tensioned from their anchorage in the launching nose. It is then necessary to have a certain distance between the main girders and the tendons so there is enough space for the prestressing equipment (Göhler and Pearson 2000).

Figure 4.3 Joint between the launching nose and the box girder.

The largest tensile forces will appear in the bottom of the joint. This is because the positive moment is almost always larger then the negative moment caused by the cantilever effect (Rosignoli 2002). Often six to ten prestressing bars need to be used and anchored at the bottom flange at each side of the launching nose. Close to the bridge deck the web needs further thickening in the top, to manage the large compressive forces caused by the cantilever moment (Göhler and Pearson 2000).

It is important that the nose can transmit large shear forces to the bridge deck. This is managed today with indentations; transverse horizontal steel plates are welded to the end plate of the nose in order to create a notched pattern (Göhler and Pearson 2000). This notched surface will create an interlocking effect between the bridge deck and the launching nose. According to Göhler and Pearson (2000) it is very important that the vertical reinforcement in the webs near the joint can resist the shear force.

4.4 Need for nose realignment

Often the nose diverges from its theoretical position, equal to the support level. This results in elastic deflection that needs to be considered. In case of curved bridges there can also be a horizontal deviation from the correct position.

The launching nose is most often constructed straight in both elevation and in the plan. The nose is aligned straight with regard to the centroidal axis of the bridge deck but this is not always the case for curved decks. The bottom flange of the nose should be arranged tangential to the bottom of the deck (Göhler and Pearson 2000).

For a bridge curved in elevation the nose reaches the next support either too high above the bearing or too low below the bearing. This difference in elevation between the nose and the support is denoted Δu. The launching nose must be lifted up or pressed down this distance in order to reach the support in a correct position. This correction of elevation will add additional stresses to the incremental launching and needs to be carefully considered in the calculations (Göhler and Pearson 2000).

Most often the nose must be corrected upwards. Inclined bottom flanges in the front of the nose could be used to steer the nose into the correct position. Such inclined bottom flanges that will realign the nose are called a hoisting wedge or realignment sledge. However, this is not a good solution since the horizontal force component acting on the support will most often be too large for the support to handle. Instead a hydraulic lifting device, see Figure 4.4, is used to realign the nose into the right position (Rosignoli 2002).

Figure 4.4 Hydraulic jack integrated in the launching nose.

Most often the radius of highway bridges curved in elevation is very large and Δu will be rather small. For these types of bridges no real problems will arise due to the curved sections. Smaller bridges, often situated in urban areas, occasionally have smaller radius that will generate larger Δu values. When Δu is expected to be large, the launching nose can be attached to the bridge deck with a small angle in order to decrease Δu. For these cases the launching bearings needs to be able to rotate this angle (Göhler and Pearson 2000).

When the nose reaches a support, it has to be lifted in place, because of the distance Δu caused by the deflection in the nose (Göhler and Pearson 2000). There are two ways to lift the nose hydraulically into place. The least common method is to have hydraulic presses attached to each pier. When the nose reaches the support the jacks lift and push the nose tip upwards into the correct position. A drawback with this method is that the jacks have to be moved between the piers. These jacks are flat jacks that only have capability for short lifting; which often make several hoisting cycles necessary. This results in a waste of time since the repeated lifting cycles are time consuming. A more commonly used method is to integrate the jack into the nose tip, see Figure 4.4 (Rosignoli 2002). At the front of the nose the hydraulic lifting equipment is positioned. When the nose reaches the support the lifting device will lift the nose into its correct position. Because of the great flexibility of the nose no great forces are needed for this lifting job. However, the lifting device needs to be able to withstand both vertical and horizontal forces (Göhler and Pearson 2000).

When the bridge is curved in the plane, it is not wise to put the nose along the tangential direction of the curved line, see line number 1 in Figure 4.5 below. If the nose is positioned along this line it will reach the next support with a too large horizontal distance between the nose tip and the support, see distance A in Figure 4.5.

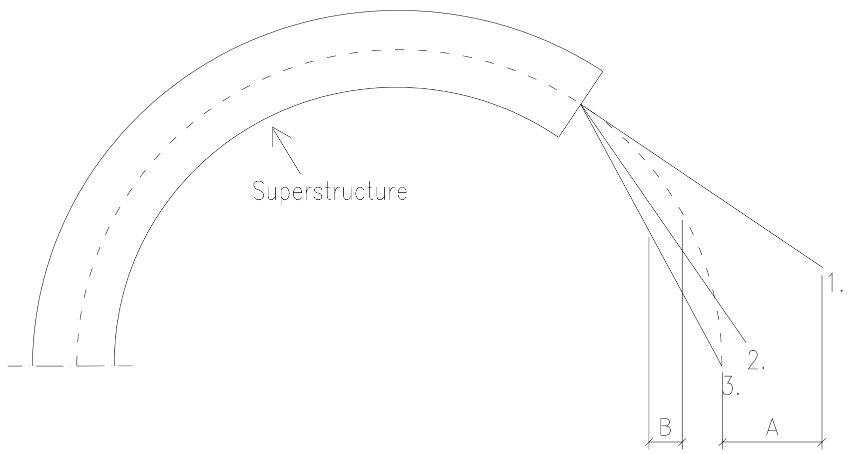

Figure 4.5 *Nose position for superstructure curved in the plane. Number 1. represents the nose direction along the tangent of the curved line. Number 2. represents the preferred direction for the nose. Number 3. represents a straight line between two supports.*

If the support is positioned like line number 3 in Figure 4.5, with a straight line between the two supports, problems will occur. The nose tip will reach the support but when the section is launched further the horizontal distance between the middle of the nose and the support will be too large, see distance B in Figure 4.5. The preferred position to attach the nose into the superstructure is according to line number 2 in Figure 4.5. This is a compromise between the two extremes, number 1 and number 3. With this solution there is a distance between the nose tip and the support, but the distance will be short enough to still be acceptable; the same applies when the support is under the middle of the nose.

4.5 Approximate nose design

Rosignoli (2002) has developed an approximate way to design a launching nose, thus avoiding making a time consuming computational analysis. The model make use of three important relationships, nose length compared to longest span L_{nose}/L_{span}, nose weight compared to the weight of the front zone of the deck q_{nose}/q_{deck} and nose flexural stiffness compared to flexural stiffness of the front zone of the deck $(EI)_{nose}/(EI)_{deck}$.

The model is based on several assumptions such as, constant weight and stiffness of both the nose and the superstructure, equal spans and total centric prestressing throughout the deck (Rosignoli 2002).

According to the assumptions based on Rosignoli's equations some important results can be concluded. Perhaps the most interesting fact is that the launching nose must fulfil two requirements that contradict each other. The launching nose needs to be of light weight and long in order to make the cantilever moment at the last support small. However, it also has to be very stiff to avoid large elastic deflections and by that ensure a large reaction force at the support closest to the nose front that will help to balance out the cantilever moment acting on the launched bridge superstructure.

5 Prestressing

It is well known that concrete has excellent properties in compression but poor capacity for tensile stresses. This is the main reason why reinforcement is needed for almost every concrete structure built. Using incremental launching requires a lot of prestressing combined with closely spaced ordinary reinforcement. Although in this master's project the focus has been on the prestressing used in IL bridges. In this chapter different prestressing systems and layouts are described.

5.1 Prestressing arrangement during the service state

Due to prestressing cracking is limited or totally eliminated in the service state, which reduces the risk of corrosion. With the use of prestressing, the whole cross-section can be under compression after completion. The bending stiffness is also higher since the whole cross-section is active. Due to this fact it is possible to create more slender structures with use of prestressing compared to structures using only ordinary reinforcement (Engström 1999).

Internal prestressing is the most common approach when designing concrete structures in Sweden. However, internationally it is more common to use external prestressing or a combination of the two methods. In the following sections these prestressing systems will be explained more in detail.

5.1.1 Internal prestressing

When the prestressing tendons are placed into the concrete cross-section it is called internal prestressing. The prestressing tendons are placed into ducts made of steel or plastic. The ducts are embedded in the structure, with or without the prestressing tendons, during construction. After the bridge has been launched to its final position, the prestressing tendons are inserted into the ducts and anchored, at one fourth of the span. The tendons have a length up to 150 m and extend over several spans, depending on the prestressing tendons area and the friction between strand and duct. When the structure is cast and the concrete has hardened, the tendons are tensioned to the needed prestressing level. After tensioning the strands the ducts can either be filled with grease (which results in post-tensioning without bond) or grout (this will result in post-tensioning with bond).

The grout creates bond between the concrete and the prestressing but it also prevent corrosion, see Figure 5.1 below (Rosignoli 2002). The grout is injected into the duct in the lowest point in the span through a vent. To ensure that the whole duct is filled with grout there are vents over the supports where the ducts reach their highest point. The grout is pressing the air out from the duct and when grout is coming out from the vents one can be sure that the duct is properly filled with grout.

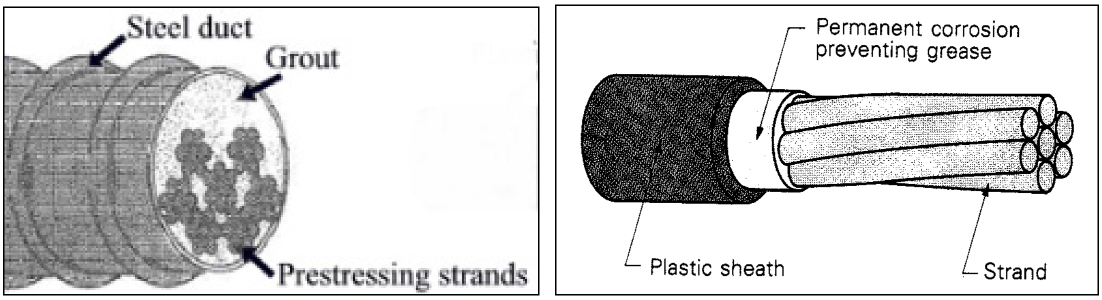

Figure 5.1 ***Left:*** *Steel duct containing prestressing strands after tensioning and grouting (Engström 1999).* ***Right:*** *Steel duct containing prestressing strands covered with grease inside a plastic duct (VSL 2004).*

One advantage when using internal prestressing is that the prestressing tendons can be placed close to the concrete surface and in this way create large eccentricity. The large eccentricity is favourable in the service state since the balancing moment from the prestressing force increases with the eccentricity.

There are some drawbacks when using internal prestressing; the lack of possibility to inspect the tendons is one of them. Another one is the limited tendon length due to the large frictional losses when tensioning the tendons (Göhler and Pearson 2000).

5.1.2 External prestressing

External prestressing is when the prestressing tendons are placed outside the concrete cross-section see Figure 5.2. The method requires so-called heels and deviators in which the tendons are attached and anchored. This results in a more complicated formwork then using internal prestressing but it also eliminates the costs for ducts and grouting. Furthermore, the mechanical properties of the cross-section will improve when removing the holes for the ducts. Using external prestressing also reduces the dead load of the superstructure if the cross-section area is decreased and therefore increases the cross-sectional efficiency.

Figure 5.2 *External prestressing inside a box girder (VSL Constructions Systems).*

Another advantage of using external prestressing is the possibility of adjusting and controlling the tendon forces. External prestressing allows long tendons since the frictional loss is low. The use of longer tendons reduces the number of anchors and therefore also the labour costs.

There are some difficulties using external prestressing. As mentioned above the formwork can be rather complicated due to the heels and deviators. Another problem is the need to resist the large local moments and forces near the anchorage (Rosignoli 2002). In case of fire external prestressing is more exposed than internal and can be seriously damaged.

5.2 Need of prestressing during incremental launching

The prestressing arrangement for an IL built bridge must counteract the most severe bending moment distribution until the bridge is completed. During the launching sequence every section has a need for prestressing in both the bottom and the top flange, since all cross-sections will be subjected to both large positive and negative bending moments, see Figure 5.3 below. These bending moments are produced from the dead weight of the concrete structure. In order to balance these moments by prestressing the tendons are arranged such that the resulting prestressing force falls as close to the centre of gravity of the girder as possible (Karlsson and Lööv 2005).

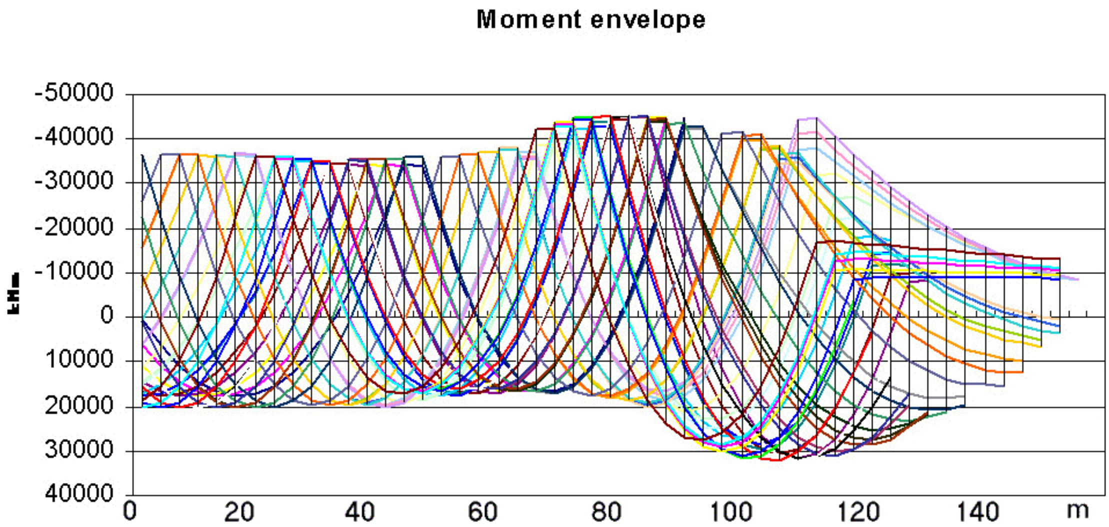

Figure 5.3 Envelope of the bending moment during launching, every cross-section of the superstructure will be exposed to positive and negative moments.

5.3 Need of prestressing in service state

After launching the bridge, additional tendons are needed in order to balance the effects of the service loads. During the service state the prestressing should balance the effects of permanent and variable loads. The tendons should therefore be arranged in such a way that the largest eccentricity of the resulting prestressing force is obtained. The additional tendons are needed in the top and in the bottom of span regions unless the launching prestressing (centric prestressing) provides sufficient capacity for the service state.

In the service state there is no difference between an IL built bridge and a bridge built on scaffolds concerning the amount of prestressing needed. Although an incrementally launched bridge requires a larger cross-section area witch results in larger dead weight and therefore also more reinforcement. Obviously one of the advantages with scaffolding is that one can construct the bridge with just the service and ultimate state in mind. In this way the prestressing arrangement can be tailormade to fit the worst load case in the service state, see Figure 5.4 (Göhler and Pearson 2000).

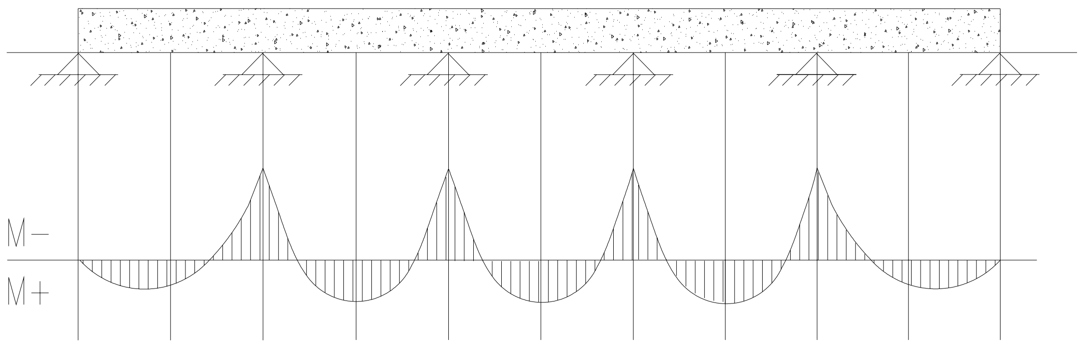

Figure 5.4 Bridge with moment diagram in the service state.

5.4 Different prestressing layouts

Karlsson and Lööv have in their master's thesis (Conceptual Design of Prestressed Concrete Bridges Produced by the Incremental Launching Method, 2005) carried out an investigation of which prestressing schemes are best suited for incremental launching. Ten different prestressing layouts were analysed. At first all alternatives was compared to each other with regard to parameters such as; web thickness, number of heels and shear forces resisted by parabolic tendons. Then the four most promising alternatives were analysed in the finite element software BRIGADE/Standard.

The two best-suited prestressing schemes for incremental launching were used for further studies in the present project. These two alternatives, called alternative 1 and alternative 2 are described in this section.

5.4.1 Alternative 1

All bridges built with incremental launching in Sweden are built with prestressing arrangement according to alternative 1. In this alternative internal prestressing is used both during the launching sequence and for the service state see Figure 5.5. Straight tendons placed in the top and bottom flanges and tensioned before launching achieve centric prestressing for the launching stage. Additional prestressing for the service state is achieved with parabolic tendons placed in the web and tensioned after launching. (Karlsson and Lööv 2005)

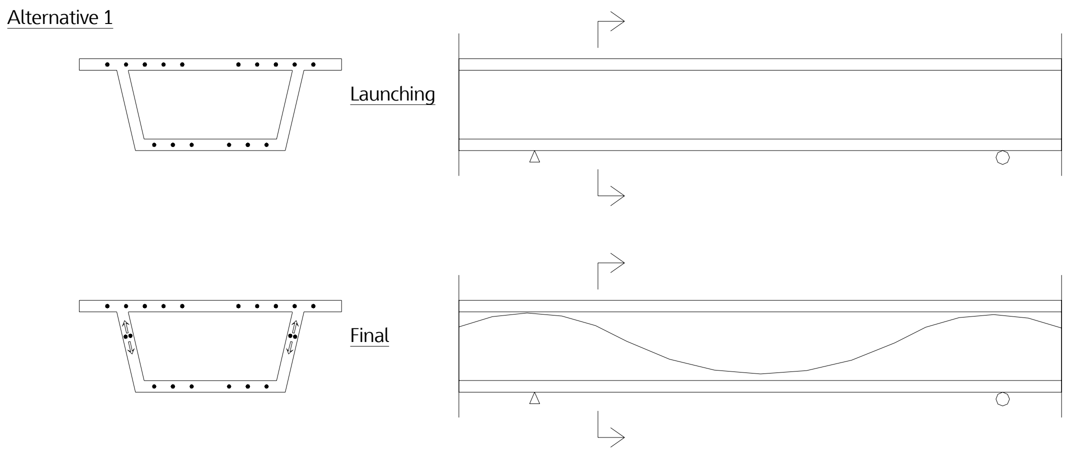

Figure 5.5 Schematic drawing of alternative 1, with internal prestressing (Karlsson and Lööv 2005).

A major advantage with alternative 1 is that fewer heels and no deviators are needed; and in this way the formwork is kept simple. In this alternative maximum eccentricity for the centric prestressing is achieved which is favourable during launching.

One disadvantage is that there is no possibility to detension the tendons after the launching. This can result in unfavourable stresses in the service state and might govern the length of the span. Another drawback of alternative 1 is the rather thick webs needed to hold the ducts for the internal prestressing.

5.4.2 Alternative 2

When designing incrementally launched bridges a combination of internal and external prestressing can be an excellent solution. Alternative 2 has also internal centric prestressing during the launching. These tendons are just as in alternative 1 tensioned before launching. The difference compared to alternative 1 is in the service state; where external tendons with polygonal profile are used, see Figure 5.6.

In order to not delay the casting sequence the heels and deviators are not manufactured in the casting yard. After launching the whole bridge the heels and deviators are cast and when this concrete has hardened the external tendons can be tensioned.

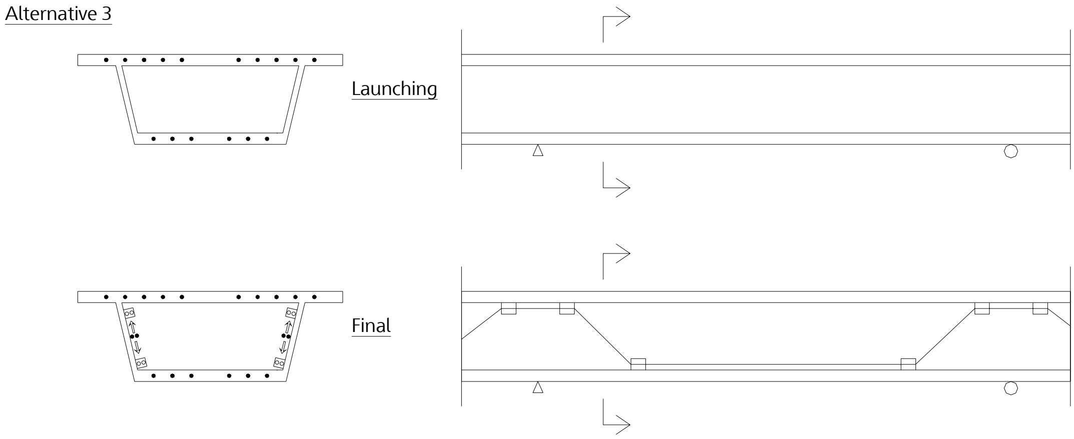

Figure 5.6 Schematic drawing of alternative 2, with both internal and external prestressing (Karlsson and Lööv 2005).

An advantage when using the prestressing layout according to alternative 2 is that the web thickness of the box girder can be made approximately 50 % thinner than in alternative 1 (Karlsson and Lööv 2005). In this way savings are made regarding concrete.

In long bridges overlapping is necessary of the external polygonal tendons and this is located over the supports. If the total bridge length is less than 300 m the tendons can remain continuously through the whole bridge.

6 Modelling

For the finite element modelling the software BRIGADE/Standard version 3.4 from Scanscot Technology was used. This finite element software is especially developed for analysing and designing bridge structures. It has a modern graphical windows interface and is capable of 3D analysis (Scanscot Technology AB 2005).

6.1 Working procedure

The model was created with the reference bridge across the river Vindeln as template. Several variations of the model were created; the first one was a complete model of the reference bridge without the prestressing tendons in order to verify the model. The second one was a model of the reference bridge with all the prestressing tendons. This model was used to analyse and compare the sectional forces with ELU Konsult AB's results and verify that the prestressing tendons were modelled in an appropriate way. Then several models, simulating different stages during launching, were created. From these models the largest sectionals forces were identified to magnitude and location. The two prestressing alternatives were modelled and analysed as well.

When the reference bridge was designed, BRO 94 edition 2 (the Swedish code for design bridge) was used. Different codes have different loads, load groups and load combinations as well as different partial coefficients on the loads. In order to make comparisons possible, BRO 94 edition 2 was also used as the input code for BRIGADE/Standard when the model was developed.

6.2 Geometry

Perhaps the most important feature when creating a model is to be accurate when defining the geometry. In this section the key factors that influence the model geometry are explained.

A **stakeout line** was used as a reference line. Other parts of the bridge were referred to the stake out line. The stakeout line was placed in the middle of the bridge, which made it easy to define the rest of the bridge. On each side of the stakeout line, a left and right borderline were placed, see Figure 6.1. The borderlines were used as the left and right edge of the bridge. Since the transversal slope of the bridge was 2.5 %, it was introduced as left and right banking of 2.5 %.

The reference bridge is curved both in the horizontal and longitudinal plane. One of the simplifications made when creating the model was to disregard from the horizontal radius of the bridge. Since the radius is rather large (900 m) and constant its influence on bending moments and axial forces can be neglected according to Karl Lundstedt. If this radius is included in the model the coordinates for the bridge become more difficult.

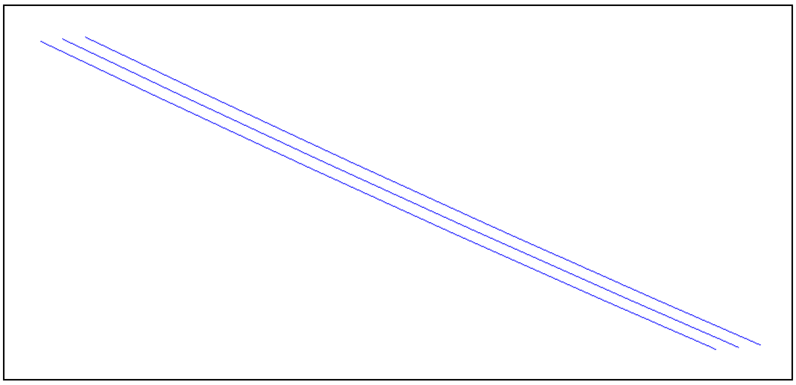

Figure 6.1 Stakeout line in the middle with one border line on each side of it.

Support lines are needed in order to define the supports. These lines were placed perpendicular to the stakeout line, as seen in Figure 6.2, and were used as "local stakeout lines" for supports.

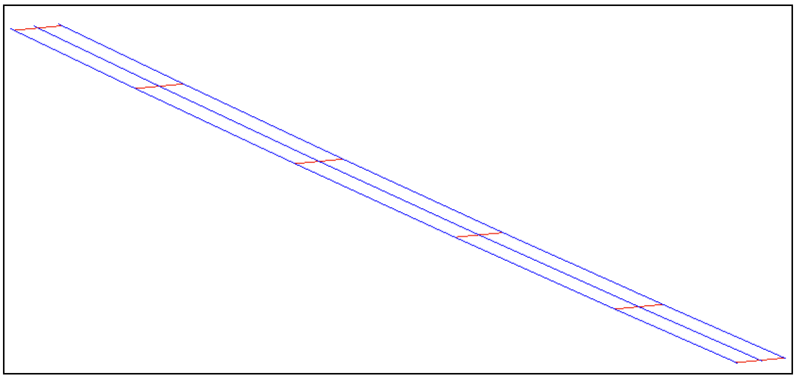

Figure 6.2 Support lines transversal to the stakeout line.

To be able to apply loads on the structure a **deck** must be created, see Figure 6.3 below. In this case it was made very thin in order to not influence the rest of the structure, since the properties of the bridge cross-section were defined as beam elements. Shell elements were used for the deck structure in the software. In the IL model the pavement thickness was set to zero during erection since no pavement was provided before or during the launching of the bridge.

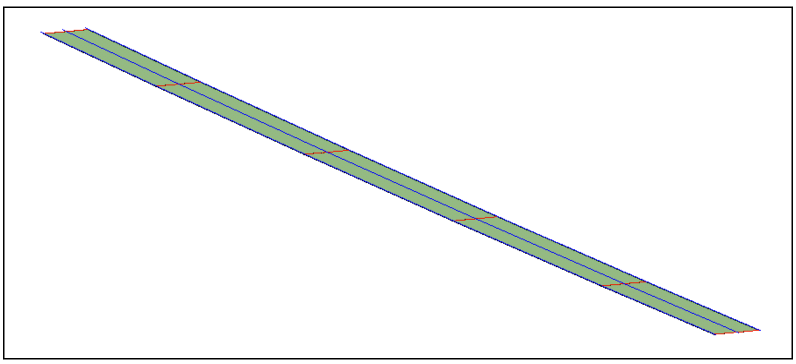

Figure 6.3 The bridge deck is added to the model.

As stated above the properties of bridge cross-section were included in the beam elements, which were used to model the superstructure. Another simplification was to not include the wing walls in the model. The wing walls are visualised in Section 3.1 Figure 3.4. These walls do not affect the sectional forces significantly and can therefore be excluded from the model according to Karl Lundstedt.

Beam elements are good to use when bending moments and axial forces are of interest. Another alternative is to use shell elements, which are good to describe bending in two directions according to M.Sc.C.E. Ph.D. Karin Lundgren. Beam elements were used since bending in only one direction is of interest. Therefore a longitudinal beam was created as the basis for the superstructure. It was defined as a general beam, where the cross-sectional constants as area, moment of inertia and torsional rigidity were used as input.

The distance to the cross-sectional centre of gravity was calculated with the Section Editor in FEM Design and used as input BRIGADE/Standard. This distance was defined in the model by adding a distance between the stakeout line and the beam element. One "disadvantage" with the general beam model is that the whole superstructure is modelled as one beam element and therefore it will be visualised only as a thin line in the model. This is however only a visualisation disadvantage and does not affect the results.

The **substructure** in the model consists of end supports, columns and foundation. Although the columns were of no interest for this analysis they were needed in order to receive appropriate results of the normal force in the superstructure. Supports 1, 2, 5 and 6 were modelled as bearings, one in each support. Supports 3 and 4 were modelled as columns see Figure 6.4. The boundary conditions for the bearings and columns were modelled according to the drawings from the reference bridge. Bearing 3 and 4 (on top of the columns) were defined as fixed and the others were movable along the bridge.

In the reference bridge the substructure rests on two bearings in each support in order to resist twisting moments. In the analysis the twisting forces were not important according to Karl Lundstedt. Therefore one of the simplifications in the model was to use only one bearing in each support. Simplifications were also made for the columns and foundations. The dimensions and properties of these objects were only roughly estimated from the reference bridge, since the main focus of this project was the behaviour of the superstructure.

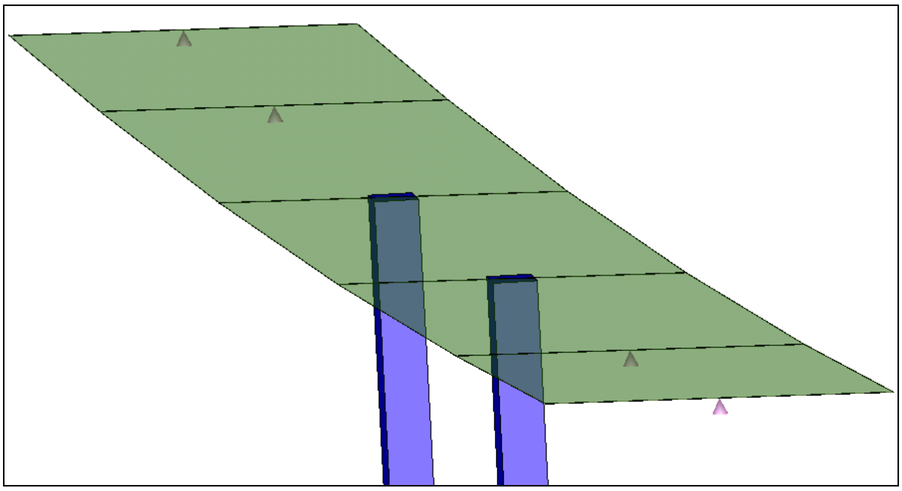

Figure 6.4 Substructure modelled as columns and bearings.

6.3 Prestressing tendons

One of the most important features of a prestressed concrete bridge is the arrangement of the prestressing tendons. In the model of the reference bridge the real tendon layout from the actual design was used, see Figure 6.5. In each span the layout of the tendons was defined in 14 points in order to have an accurate tendon profile. The tendons were fitted through these points using the spline curve function in BRIGADE/Standard. Spline curves are composite curves adjusted by an arbitrary amount of key points that the tendons will pass through (Scanscot Technology 2005). Full interaction was assumed between the tendons and the concrete, which means that the effect of bondslip was not included.

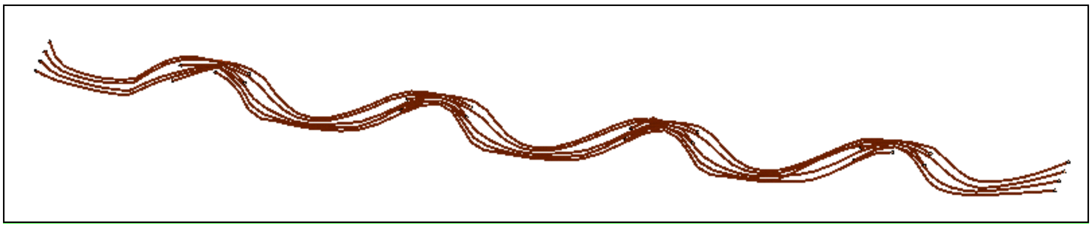

Figure 6.5 The profile of the prestressing tendons was defined with spline curves.

Due to the fact that the cross-section is symmetric, all tendons could be placed in the centre of the bridge along the stakeout line. In reality half of the tendons are positioned in the left web of the box girder and the other half in the right side. There were as most 26 tendons running in a cross-section, or 13 tendons on each side. At least two of these tendons are always positioned on the same distance from the bottom of the bridge, except in the edges near the anchorage. Because of this, these tendons were merged together in the model with increased cross-sectional area instead. As seen in Figure 6.6 two tendons on each side and on a certain level were modelled as one tendon in the middle with four times larger area. One pair of tendons split on each side at a certain level was modelled as one tendon in the middle with twice the area.

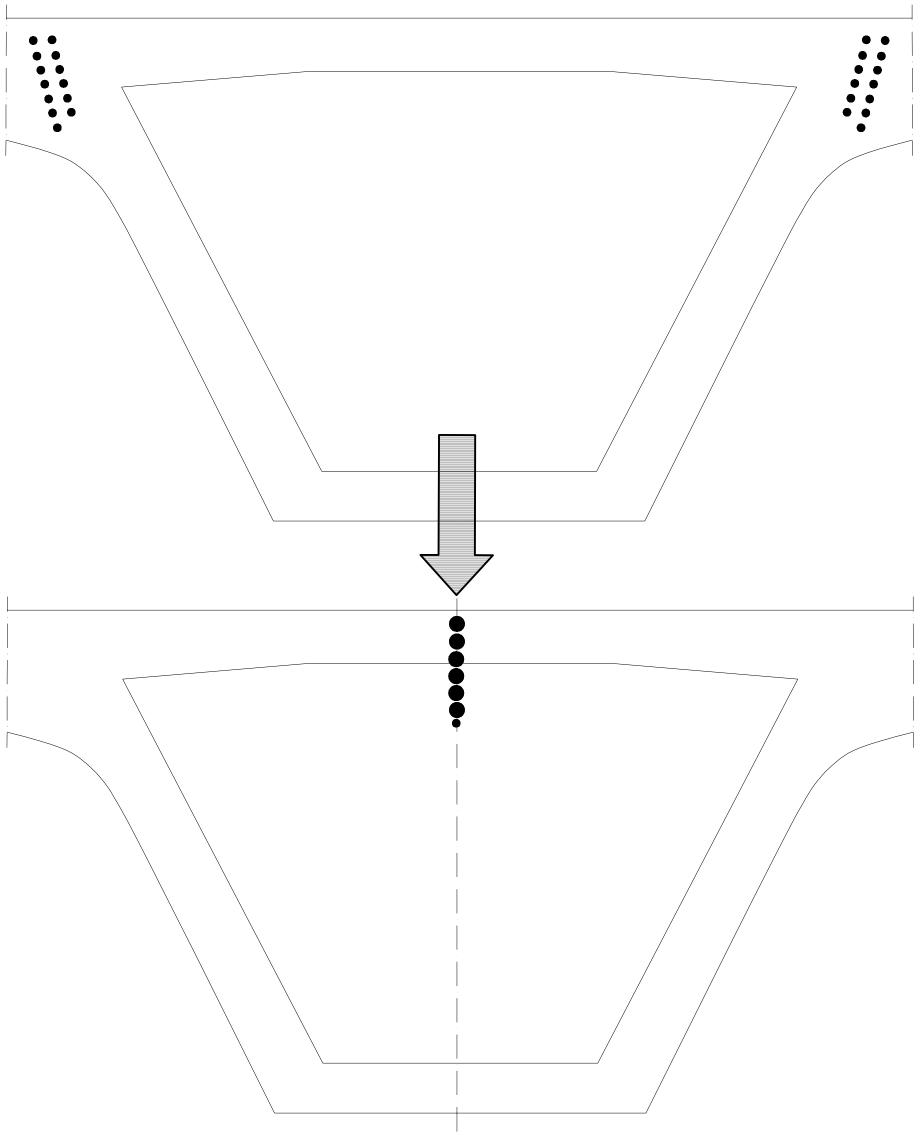

Figure 6.6 ***Top:*** *The cross-section of the box girder.* ***Bottom:*** *Simplified cable layout used in model.*

All tendons were tensioned from both sides in reality as well as in the model. In the reference bridge the tendons were curved near the anchorage in the web blisters, heels, that protrude from the web, see Figure 6.7 below. These web blisters were not included in the model.

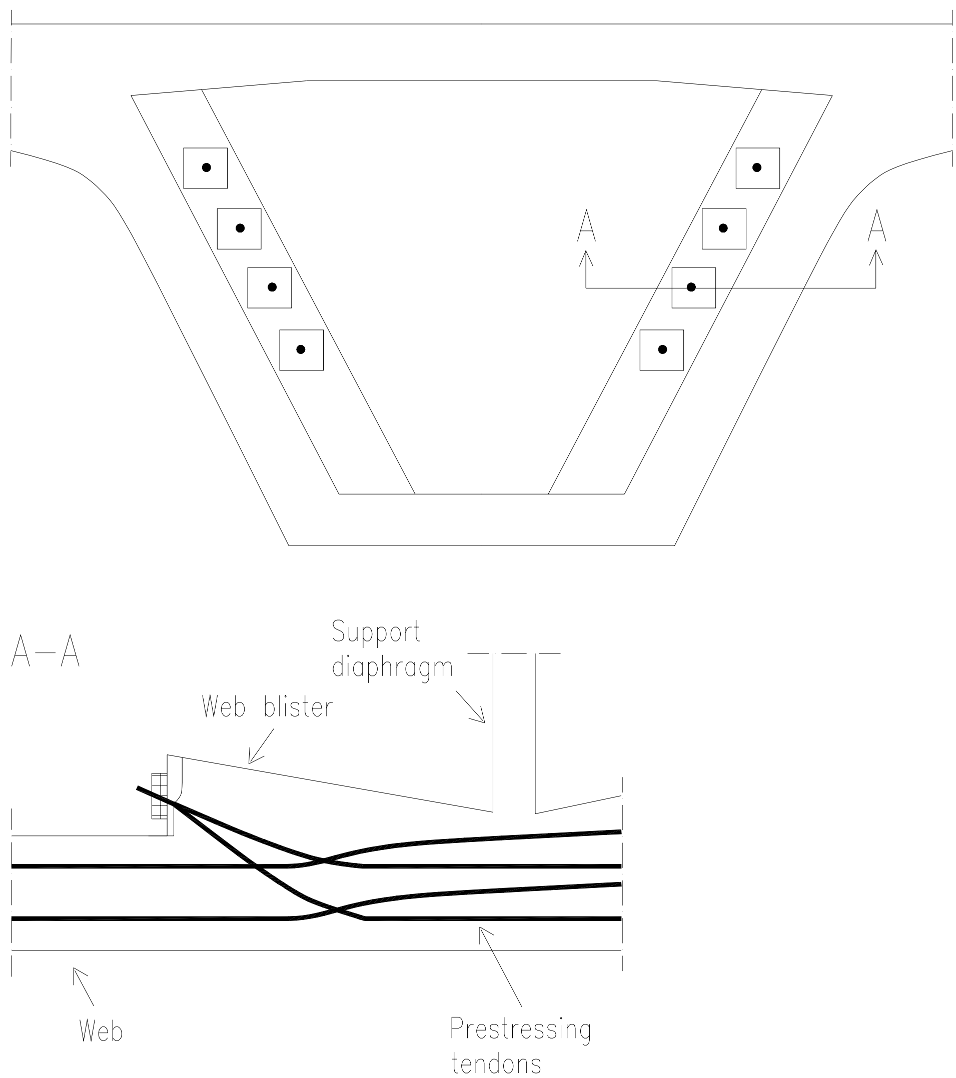

Figure 6.7 Web blister near one of the supports.

When the tendons are anchored in the web blister they are separated from each other, with only one tendon at each level. In the model the average height of such a pair was calculated and used, visualised in Figure 6.8 below.

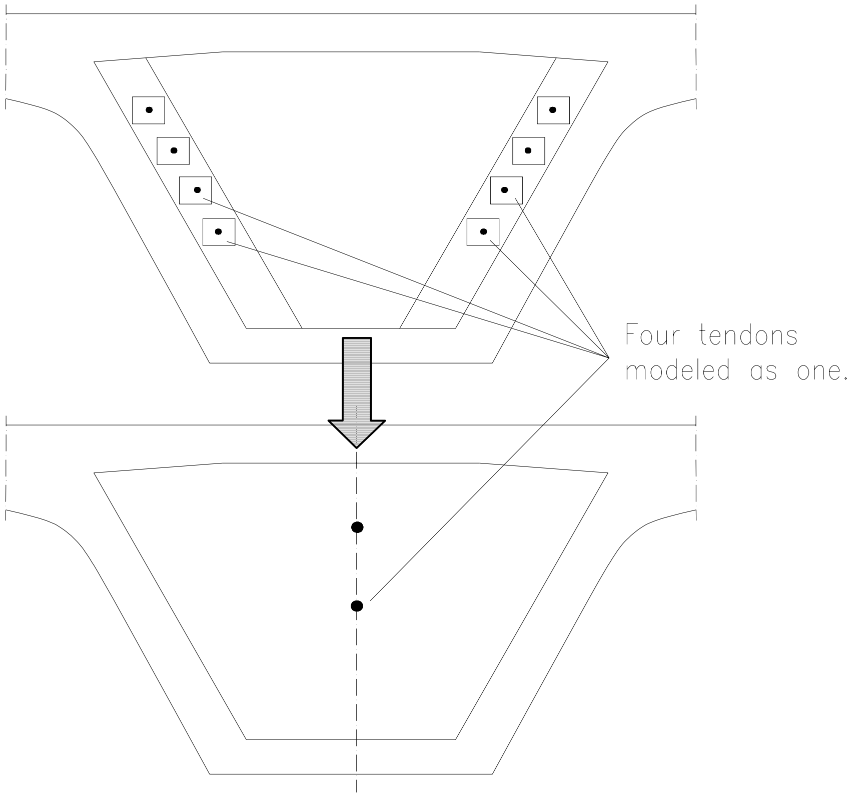

Figure 6.8 ***Top:*** *A web blister from the bridge.* ***Bottom:*** *how the web blister was modelled.*

The model was verified by comparing moment and normal force results in the superstructure from the prestressing force, with results from ELU Konsult AB's Strip Step 3 analysis, see Appendix 2. In order to receive a result close to the one from ELU Konsult AB support 3 and 4 were modelled as columns, see Section 6.2 for further information. If both supports 3 and 4 are modelled as fixed bearings, no axial normal force will be generated. With these settings and the simplifications mentioned above a reliable result was obtained. The final model of the reference bridge is visualised in Figure 6.9.

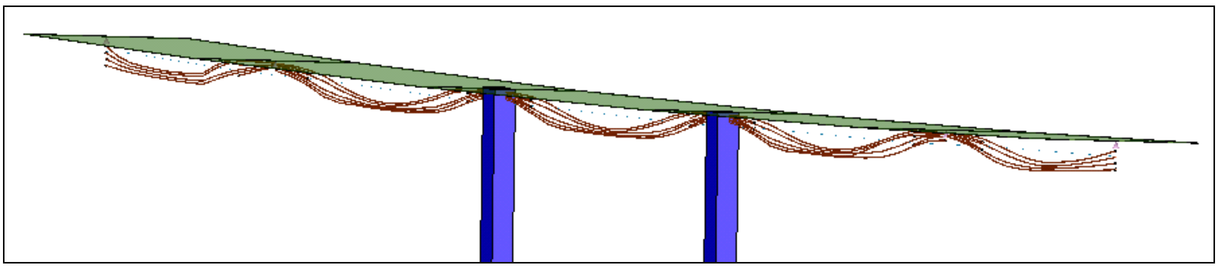

Figure 6.9 *The final model of the reference bridge in BRIGADE/Standard.*

It was necessary to create a model of the **external polygonal tendons** used in prestressing alternative 2. The prestressing force applied to the polygonal tendons was modelled as longitudinal (x) and vertical (z) external force components acting on every deviator positioned along the superstructure. The magnitude of the external forces was calculated by simple equilibrium equations. The calculations were based on the same basic prestressing force as ELU Konsult AB used in the designed reference bridge, see Figure 6.10 below for more details concerning the external forces. The derivation of the values can be seen in Appendix 3. Friction losses in the deviators were neglected. No special stress analysis was carried out for local stress peaks in the deviators, experience from the study trip show that it is possible to use massive external prestressing without failure problems in the deviators.

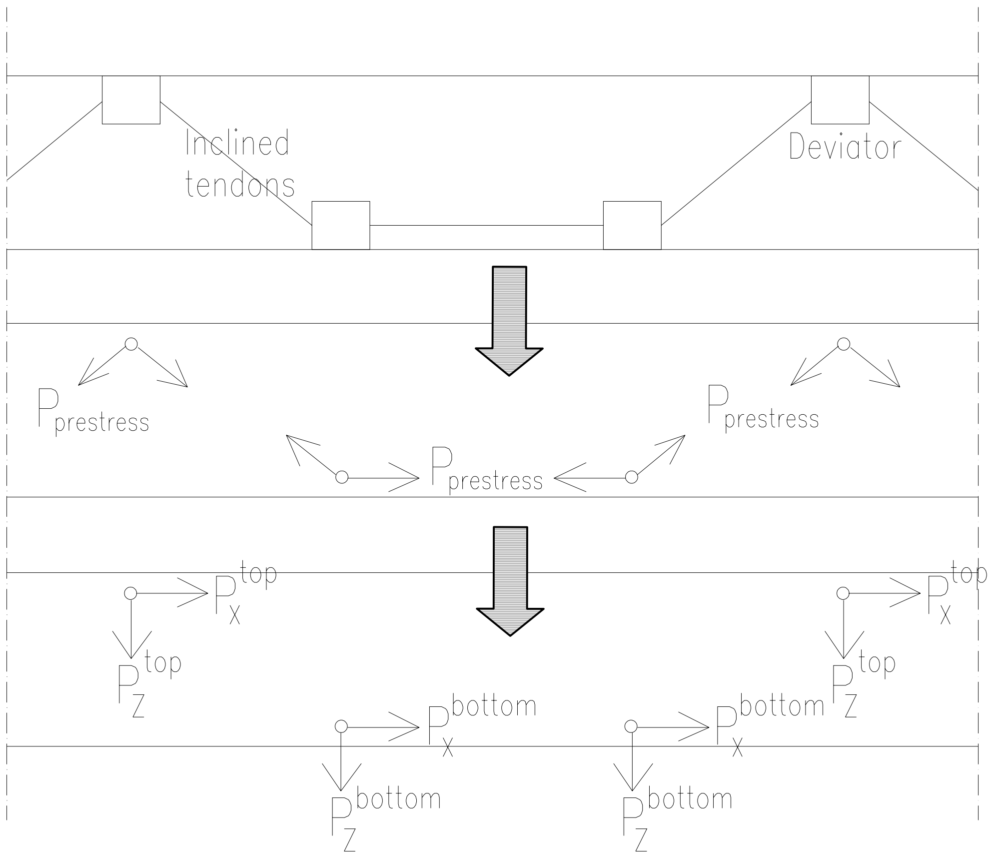

Figure 6.10 External prestressing modelled as external forces ***Top:*** *The polygonal tendons and their deviators.* ***Middle:*** *The prestress force acting on the superstructure.* ***Bottom:*** *The force components applied to the beam element in BRIGADE/Standard.*

The **centric prestressing** was modelled as straight tendons positioned in the centre of gravity of the cross-section. Naviers formula was used to calculate the prestressing force applied to the centric prestressing. The most critical moments and normal forces during launching were used as input. Also losses were taken into consideration; see Appendix 7 for complete derivation of the centric prestressing needed.

6.4 Material

All the material properties needed such as Young's modulus, Poisons ratio, density and thermal expansion were defined for the model. As mentioned before the deck is only needed to be able to apply the loads acting on the structure. Therefore the properties for the deck were minimised in such a way that they should not influence the result of the analysis.

6.5 Loads

When designing the superstructure it is important to have all critical load cases clearly specified, both for the service state and for temporary load-cases during erection. As mentioned in Chapter 1, every cross-section of the bridge must be able to resist a huge variation of load effects during erection.

Loads specified in the Swedish code BRO 94 are predefined in BRIGADE/Standard. A selection of the most important loads was made to limit the work. The chosen loads had the largest contribution to bending moments and axial forces in the superstructure. Forces like braking force, wind acting on the structure, accidental loads and seismic loads were neglected.

The governing load in the SLS is the load combination V:A according to BRO 94. This combination consists of four permanent loads and two variable loads that give the most unfavourable effect. In this case the permanent loads were deadweight, surfacing, support yielding and shrinkage. The variable loads used were the traffic and temperature loads. The traffic load was applied to the model by a load surface, shown in Figure 6.11. The load surface is the defined area upon the deck where the load acts.

The most important load during the launching is the dead weight of the structure. To reduce the bending moment the first section is equipped with a launching nose. This structure is made of steel by a weight that is only 10 % of the concrete beam.

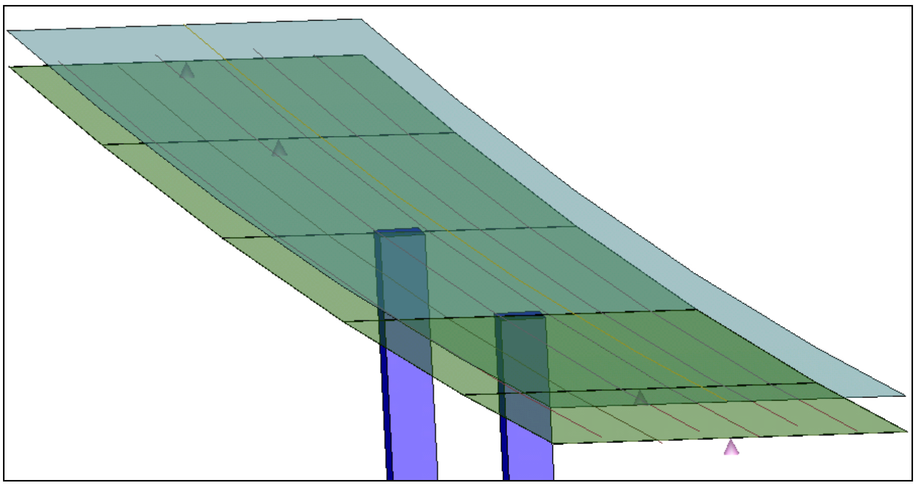

Figure 6.11 Loads applied to the model using the load surface.

6.6 Verification of the model

The model was verified by comparing results from BRIGADE/Standard concerning bending moments and reaction forces with the calculations carried out by ELU Konsult AB. The results from the most significant load cases can be seen in Appendix 2 and a summary of the results from the moment and reaction forces comparison can be found below:

- Moments caused by temperature difference differ at most with 2.1 % and an average difference of 0.98 %.
- Moments caused by deadweight differ at most with 7.5 % and an average difference of 3.6 %.
- Moments caused by surfacing differ at most with 6.9 % and an average difference of 2.9 %.
- Support reaction forces caused by deadweight differ more then 15 % at the first and last support but there is only an average difference of 0.55 % for support 2 through support 5.
- Moments caused by prestressing differ at most with 12 % and an average difference of 6.4 %.

The conclusion drawn from the results of the comparison is that errors are rather small and therefore an acceptable equivalence between the values has been reached. The model is accurate and can be used in further analyses.

6.7 The incremental launching stage

The moment distribution during the launching of the superstructure is too complex to just assume. The different stages of the launching sequence needs to be analysed by means of proper modelling in order to receive some of the most critical moments. A model that accurately approximates the behaviour of the real nose was needed. Subsequently an analysis where the superstructure with the nose was moved in small steps towards one of the abutments was carried out.

In the reference bridge the concrete superstructure consists of two different cross-sections, one support section and one span section. For the launching model of the bridge these two sections were replaced by one cross-section with average properties from the two sections as the new cross-section data. This cross-section will be referred to as the original cross-section from now on.

6.7.1 Modelling the launching nose

To simulate the launching process and some of its most critical steps, a detailed model of the launching nose was created. An optimal length of a launching nose for the reference bridge is around 65 % (35 m) of the longest span and has the average weight that is 10 % of the concrete section (Rosignoli 2002). Some assumptions were made regarding the geometry of the nose. The height was set to 2,5 m high in the end were it is attached to the concrete section and approximately 1,0 m high at the nose front.

One difficulty when modelling the launching nose in BRIGADE/Standard is that there is no possibility to create beam elements with discontinuous cross-sectional properties. This problem was solved by creating a number of general beams with different cross-sectional dimensions that are merged together in the software see Figure 6.12.

In order to not make this procedure too time consuming the number of these average cross-sections was limited to five different segments. With values for the cross-sectional area, density of steel, length of the nose and the alternating height of the nose, an average cross-section could be calculated for the whole launching nose. With the calculated average nose cross-section and the knowledge of the geometry of the launching nose, properties for the five different nose segments could be calculated.

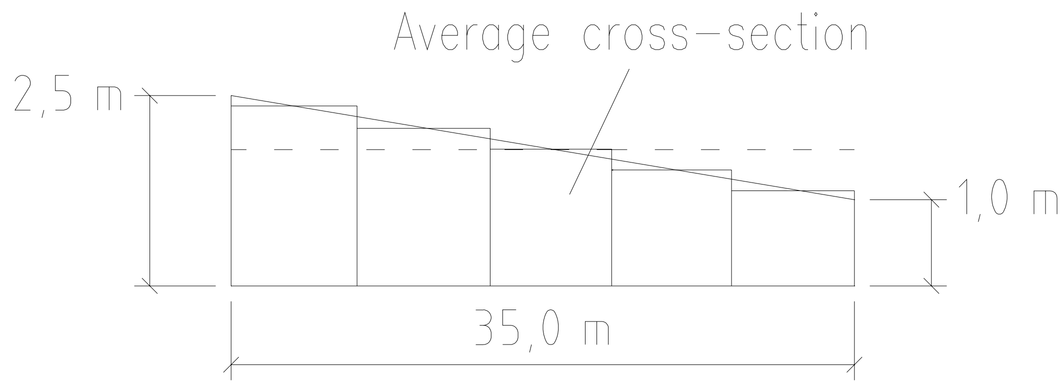

Figure 6.12 Visualisation of the five nose segments including the average cross-section used to model the launching nose in BRIGADE/Standard.

In reality a launching nose often consists of two parallel I-beams both with inclined top flanges, see Figure 6.13. These I-beams are connected with each other via transversal steel bars. The flanges of the two I-beams could be modelled as four beams, see Figure 6.13. The model was simplified to consist of four quadratic homogenous beams. The summation of the area over these four homogenous beams was equal to the calculated area for the different nose segments. These simplified nose cross-sectional models were defined in FEM Design Section Editor, were properties like torsional rigidity and product of inertia were calculated. When all the data for the simulated launching nose was obtained, five different beam elements were created in BRIGADE/Standard and attached to the superstructure of concrete.

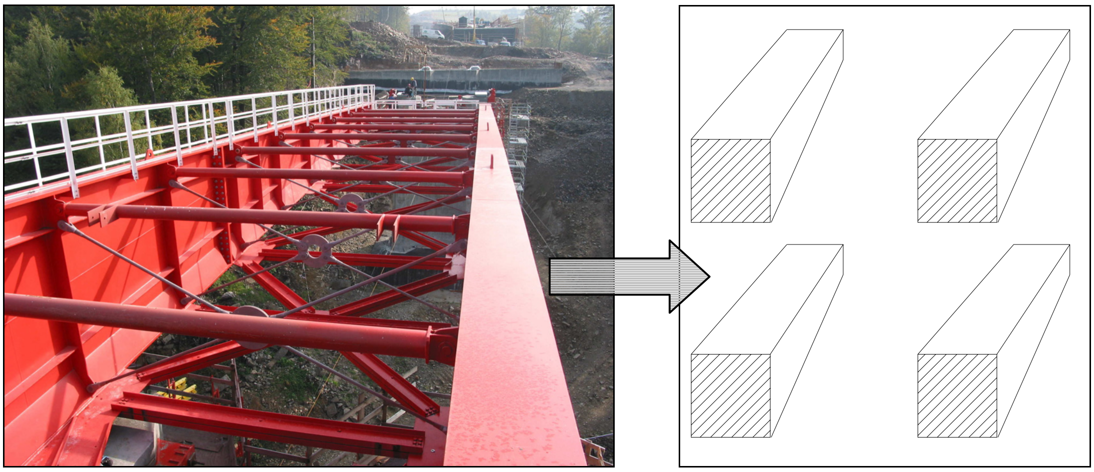

Figure 6.13 The cross-sectional model of the launching nose used in BRIGADE/Standard consists of four quadratic homogenous beams. The large I-beams in the left figure were modelled as the four beams in the right figure.

The new concrete superstructure and launching nose model was then placed in different positions in relation to the locations of the support. From this analysis the most critical moments during the launching stage can be acquired.

6.7.2 Bending moments during launching

During the incremental launching of a bridge every part of the superstructure will be exposed to a huge variation of bending moments. In order to find the magnitude of these moments and obtain knowledge of when the most critical ones arise, an investigation was carried out. The model described in the previous section was used. The analysis was carried out in BRIGADE/Standard and all the results were collected in Microsoft Excel see Appendix 4 for an example.

The starting point for the analysis was when the superstructure had been launched as far as 141 m, thus passing over three supports with the concrete section and passing over support 4 with the launching nose, see Figure 6.14. Starting from this stage the bridge was moved forward in steps towards the final abutment, support 6, and all the moments along the bridge were calculated for each step.

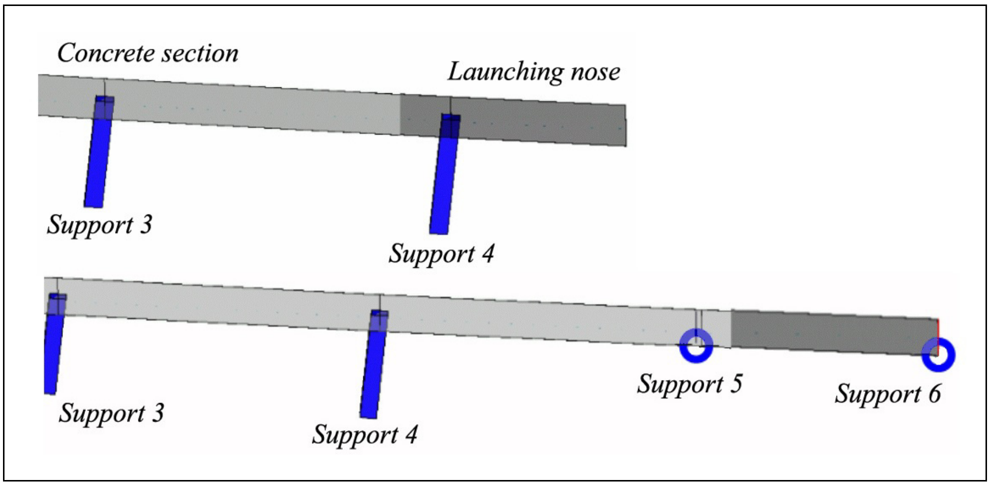

Figure 6.14 ***Top:*** *The start point for the moment analysis during launching.* ***Bottom:*** *The end point for the moment analysis during launching.*

The result from this analysis was that the most critical section regarding negative moment was when the concrete part had 5 m left to support 5, thus the nose had passed support 5 with 30 m see Figure 6.14. Critical section regarding positive moments was when the concrete section had 4 m left to support 4, thus the nose had passed support 4 with 31 m see Figure 6.15.

Most surprising with these results is that the maximum negative moment is not received when the launching nose is in the position of just reaching support 5, like one might have guessed. The reason for this is most likely the very light nose structure, with a weight being only 10 % of the concrete section.

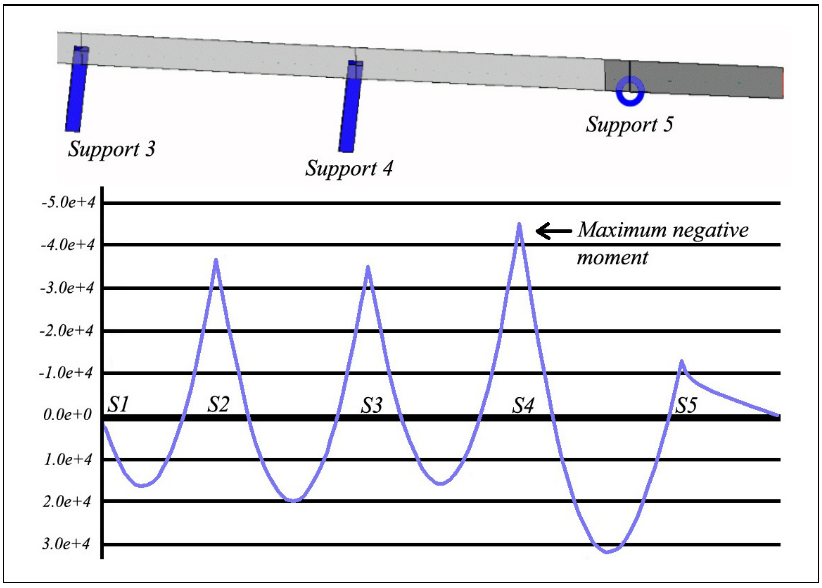

Figure 6.15 ***Top:*** *at this point in the launching process maximum negative moment is reached.* ***Bottom****: moment diagram belonging to the launching stage in top picture (exact moment values only for the concrete part due to limitations in BRIGADE/Standard).*

The fact that the largest positive moment, see Figure 6.16, was acquired at a stage when the concrete section was about to reach support 4 was not that surprising. The heavy weight concrete section deflects the span between supports 3 and 4 with significant power. While the lightweight launching nose does not counteract that much when cantilevering out from support 4, thus large positive moment occurs.

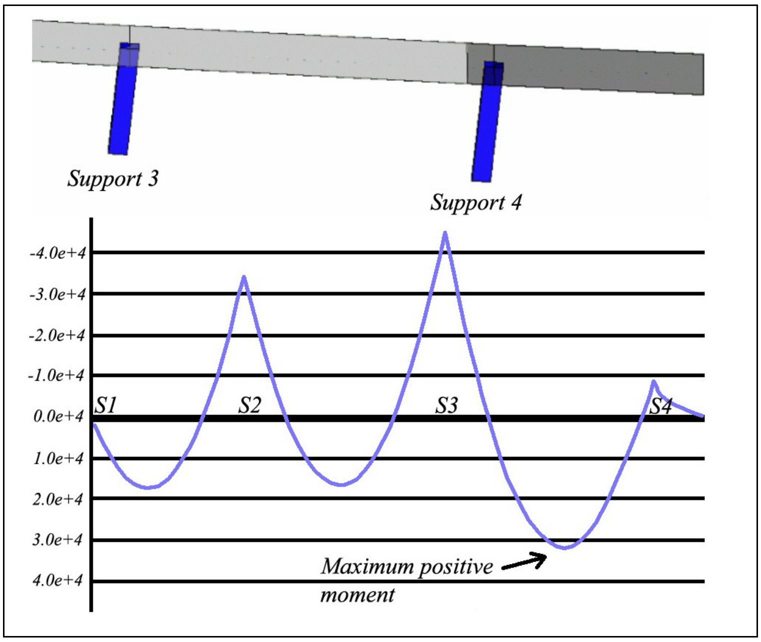

Figure 6.16 ***Top:*** *at this point in the launching process maximum positive moment is reached.* ***Bottom:*** *the moment diagram belonging to the launching stage in top picture (exact moment values only for the concrete part due to limitations in BRIGADE/Standard).*

6.7.3 Moment envelope diagram

If all the moments from all the different steps in the analysis are put together in a graph a moment envelope, seen in Figure 6.17, is obtained with a good visualisation of moments during the launching process. The last 140 meters of the bridge including the launching nose was studied. The moments in the left part of the diagram (the first 60 m) are rather constant moments from the part of the bridge that is resting on supports, while the larger moments on the right side (the last 80 m) are from the front of the bridge where the nose is step by step moving towards the final abutment, with the nose acting as cantilever. Every part of the superstructure is exposed to all these moments. As seen in Figure 6.17 the moment envelope diagram has an interesting shape. Notice the leap in the in-circled part in Figure 6.17. It occurs when the nose reaches support 6 and the moment at support 5 decreases.

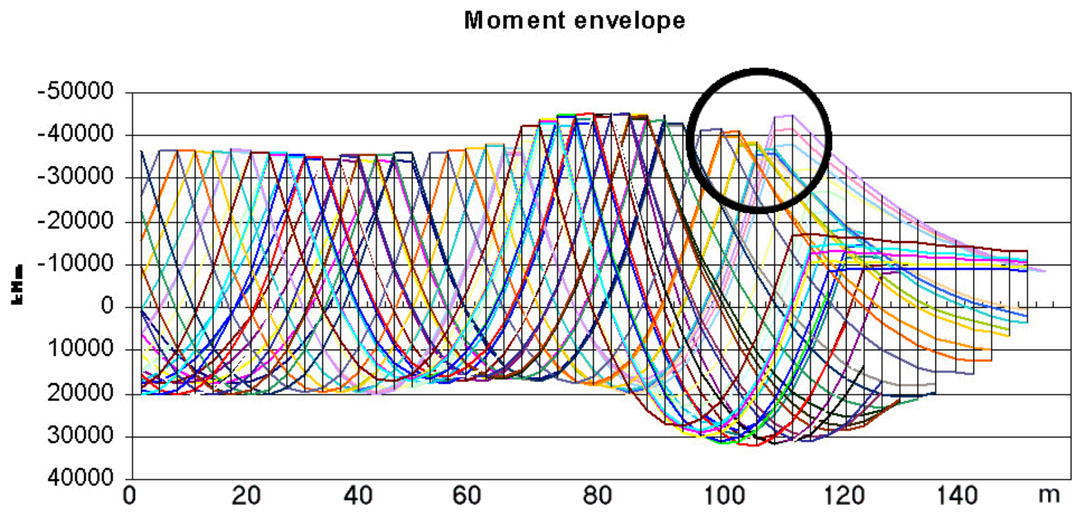

Figure 6.17 Moment envelope from the launching analysis, launching direction from left to right. The in-circled part indicates when the moment in support 5 decreases since the launching nose reaches support 6.

6.8 Important results

For the interested reader moment and axial force results from several BRIGADE/Standard runs can be viewed in Appendix 5. A summary of the largest moments follows below:

- Largest positive moment during launching, dead weight only, original cross-section, 32190 kNm
- Largest negative moment during launching, dead weight only, original cross-section, -45210 kNm
- Largest positive moment in service state, dead weight only, original cross-section, 19030 kNm
- Largest negative moment in service state, dead weight only, original cross-section, -37170 kNm
- Largest positive moment in service state, load case V:A, original cross-section, 44510 kNm
- Largest negative moment in service state, load case V:A, original cross-section, -62530 kNm
- Largest positive moment in service state, load case V:B, original cross-section, 27510 kNm
- Largest negative moment in service state, load case V:B, original cross-section, -47320 kNm

7 Structural calculations

The moments and axial forces received from the analysis of the bridge in BRIGADE/Standard were used to calculate stresses along the bridge and across the cross-section. These stresses were the basis for the evaluation and comparison that was the main purpose of this project. A prewritten Excel document based on Naviers formula was provided by Skanska (Karl Lundstedt), see Appendix 6, and was used for the stress analyses of the cross-section. The formulas used in the Excel document are seen in formula 7.1 and 7.2 below.

Naviers formula:

$$\sigma_{top} = \frac{\sum M}{W_{top}} + \frac{\sum N}{A} \qquad (7.1)$$

$$\sigma_{bottom} = \frac{\sum M}{W_{bottom}} + \frac{\sum N}{A} \qquad (7.2)$$

Formula 7.3 is an example of how the calculations were performed for the stress in the top part of the cross-section for load case V:A:

$$\sigma_{top}^{V:A} = \frac{1}{W_{top}}(M_{PS} + \Delta M_{PS} + M_{V:A}) + \frac{1}{A}(P_{PS} + \Delta P_{PS} + N_{V:A}) \qquad (7.3)$$

7.1 Simplifications

For the launched bridge only the critical sections, over the support and in the span, were analysed. Here the prestressing was entirely placed in the upper and lower flanges. In these sections all the stresses were assumed to be normal stresses along the bridge. Sections that have prestressing in the web are more complicated to analyse since the shear stresses need to be included in the evaluation.

7.2 Different cross-sections analysed

In order to make the evaluation as thorough as possible different cross-sections were analysed together with the two prestressing alternatives described in Section 5.4. To only consider the original cross-section used in the existing bridge is not fair against the incremental launching technique. Bridges constructed with IL will most often have a greater depth of the box than a bridge constructed on scaffolding. This is necessary because of high cantilever moments during launching. In order to fulfil these requirements for IL three new cross-sections were designed with help of Rosignoli’s design formulas. Regard was taken to the spacing needed for all the tendons and reinforcement. The new cross-sections are described in detail in Section 7.2.1 and 7.2.2.

The following formula (7.4) from Rosignoli was used when calculating the depth of the new superstructure:

For bridges constructed with incremental launching:

$$H_{IL} = 0.94 + \frac{L_{span}}{22.7} = 0.94 + \frac{54}{22.7} = 3.3m \tag{7.4}$$

It can be compared to Rosignoli's formula (7.5) for other methods of building, like scaffolding:

$$H_S = 0.04 + \frac{L_{span}}{21.8} = 0.04 + \frac{54}{21.8} = 2.5m \tag{7.5}$$

Notable is that formula 7.5 results in the same height for scaffolding as the reference bridge. The conclusion drawn is that Rosignoli's design formulas are acceptable.

Where concrete cut outs have been made the new thickness was limited due to the space needed for the ordinary reinforcement and in some cases also for prestressing tendons. The required distances between different reinforcement layers as well as required distance between reinforcement and the surface of the section were considered. The minimum values for these different distances were taken from BRO 94 and BBK 94 and can be found in Table 7.1 below.

Table 7.1 Minimum values regarding required distances between reinforcements and free edges of the cross-section (BRO 94 and BBK 94).

Concrete cover, the distance between the reinforcement and the edge of the concrete	45 mm
Distance between crossing reinforcement	0 mm
Distance between reinforcement in the same direction and between reinforcement and tendons	30 mm
Concrete cover in bottom and top flange for prestressing tendons	150 mm

Table 7.1 above is used in order to obtain the dimensions seen in Figure 7.2 - 7.5. To show how the thickness of the bottom flanges (184 mm and 370 mm) was calculated see Table 7.2 and Table 7.3 below. To get a better understanding for the dimensions of the bottom flange see also Figure 7.1.

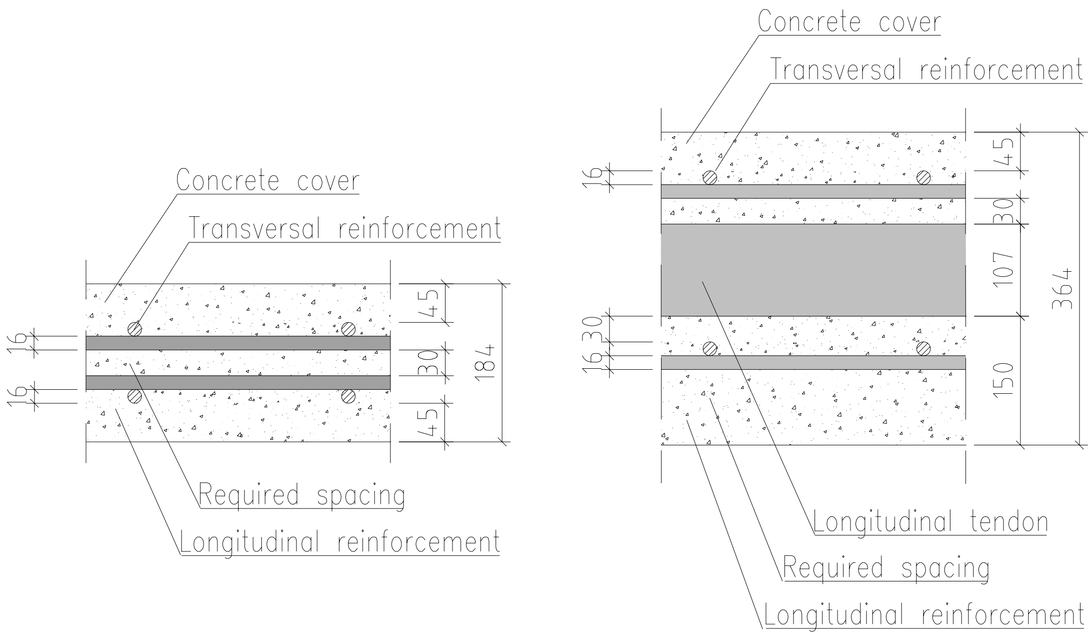

Figure 7.1 ***Left:*** *Detailed drawing of the middle part of the bottom flange for alternative A, B and C.* ***Right:*** *Detailed drawing of the side part of the bottom flange for alternative A, B and C. Distances in mm.*

Table 7.2 *Example of how the calculated thickness used in the middle part of the bottom flange in the alternative cross-sections was obtained.*

Concrete cover, the distance between the reinforcement and the edge of the concrete	45 mm
Transversal reinforcement diameter	16 mm
Distance between crossing reinforcement	0 mm
Longitudinal reinforcement diameter	16 mm
Distance between reinforcement in the same direction and between reinforcement and tendons	30 mm
Longitudinal reinforcement diameter	16 mm
Distance between crossing reinforcement	0 mm
Transversal reinforcement diameter	16 mm
Concrete cover, the distance between the reinforcement and the edge of the concrete	45 mm
Total thickness needed, summation:	**184 mm**

Table 7.3 Example of how the calculated thickness used in the side part of the bottom flange in the alternative cross-sections was obtained.

Concrete cover, the distance between the reinforcement and the edge of the concrete	45 mm
Transversal reinforcement diameter	16 mm
Distance between crossing reinforcement	0 mm
Longitudinal reinforcement diameter	16 mm
Distance between reinforcement in the same direction and between reinforcement and tendons	30 mm
Longitudinal tendon diameter	107 mm
Concrete cover in bottom for prestressing tendons	150 mm
Total thickness needed, summation:	**364 mm**
The value was rounded of upwards to 370 mm	

In a similar way, as the two examples presented in Table 7.2 and Table 7.3, all other dimensions in the cross-sections were calculated to fulfil the minimum requirements.

7.2.1 Cross-sections used with prestressing alternative 1

For prestressing alternative 1, which has internal parabolic and straight tendons, the following cross-sections were studied:

- The original cross-section that is the average between the span and support cross-sections on the reference bridge, see Figure 7.2.

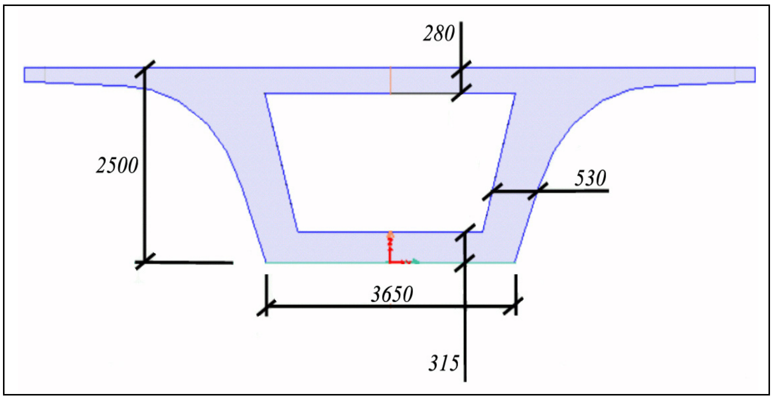

Figure 7.2 Geometry of the original cross-section.

- Alternative cross-section A, where the height have been increased from 2,5 m to 3,3 m and some concrete cut outs have been made in the top and bottom of the box in order to optimise the cross-section, see Figure 7.3.

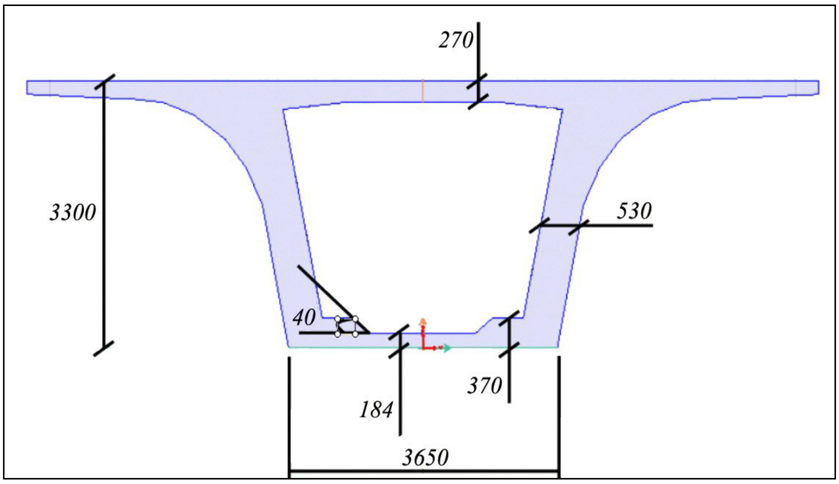

Figure 7.3 Geometry of alternative cross-section A.

- Alternative cross-section B is a further optimisation of alternative A see Figure 7.4. The arched outer sides of the webs were removed for more concrete savings.

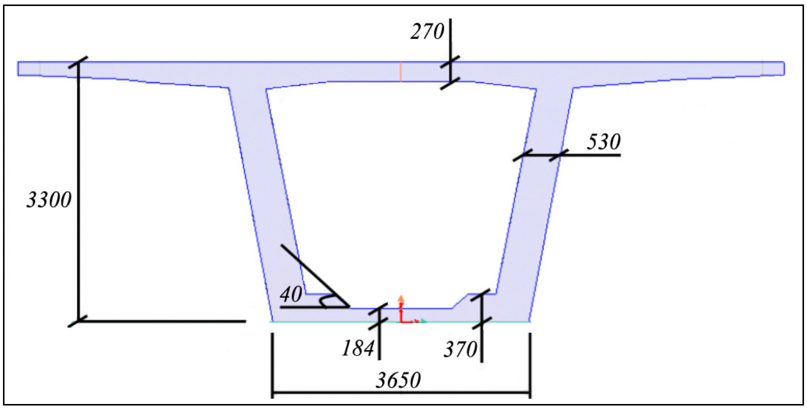

Figure 7.4 Geometry of alternative cross-section B.

7.2.2 Cross-sections used with prestressing alternative 2

For prestressing alternative 2, which has internal straight tendons in the bottom of the box as well as in the deck and external polygonal tendons, the following cross-section alternatives were studied:

- The original cross-section.

- Alternative cross-section B.

- Alternative cross-section C that has an increased height of 3,3 m and arched webs like alternative A, see Figure 7.5. The difference from alternative A is the web thickness that was reduced since the prestressing tendons were attached externally inside the box girder.

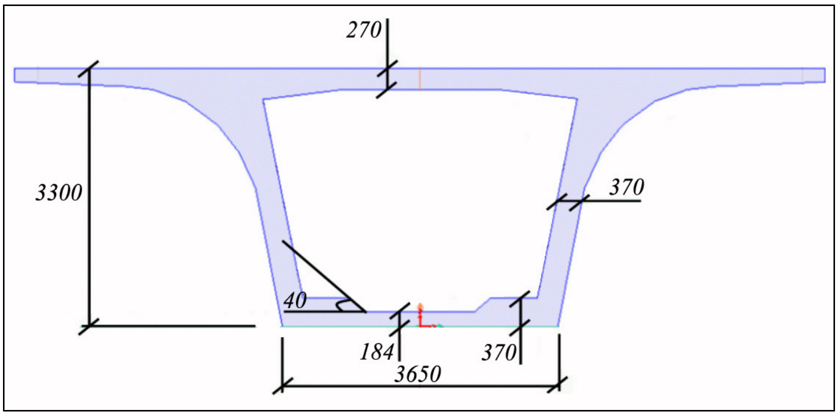

Figure 7.5 Geometry of alternative cross-section C.

7.3 Design criteria

The stress analysis for the bridge in its service state was performed in two critical sections along the bridge, one support section and one span section. Support 3 was chosen 95 m from the abutment, and span 3 was chosen 122 m from the abutment. These specific sections were chosen since large positive and negative moments were found here, see Figure 7.6. For these two sections stresses were evaluated according to the design criteria for load case V:A (main load case in the serviceability limit state) and V:B (load case for allowable crack width) according to BRO 94.

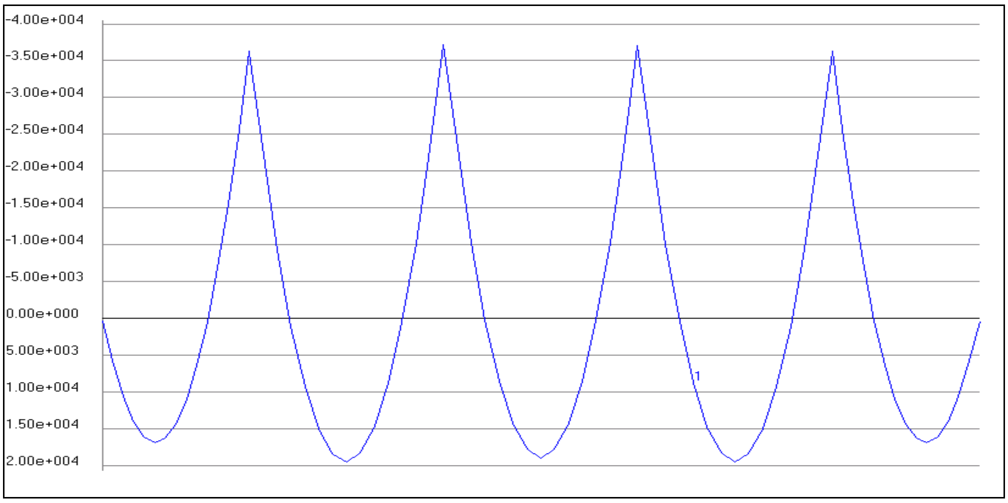

Figure 7.6 Moment distribution along the bridge in the service state.

During launching the two most critical sections described in Chapter 6 were evaluated according to the design criteria for load case V:A and V:B (BRO 94). The key values received from the design criteria:

- V:A allowable compressive stress: $|\sigma_c| < 0.6 f_{cck} = 0.6 \cdot 32 = 19.2 MPa$
- V:A allowable tensile stress in the top: $\sigma_{ct} < \sigma_r = \frac{k \cdot f_{ctk}}{\zeta} = \frac{1 \cdot 2.1}{1.5} = 1.4 MPa$
- V:A allowable tensile stress in the bottom: $\sigma_{ct} < \sigma_r = \frac{k \cdot f_{ctk}}{\zeta} = \frac{1 \cdot 2.1}{2.0} = 1.0 MPa$
- V:B allowable compressive stress: $|\sigma_c| < 0.6 f_{cck} = 0.6 \cdot 32 = 19.2 MPa$
- V:B allowable tensile stress: $\sigma_{ct} < 0$ MPa in level with the prestressing tendons.

7.4 Stress analysis

The two different prestressing alternatives studied together with the different cross-sections, resulted in six different sectional analyses. As mentioned before prestressing alternative 1 was analysed for the original cross-section and also for the two improved cross-sections A and B. Prestressing alternative 2 was also analysed for the original cross-section and the two improved ones denoted cross-section B and C. The focus of the analysis was the design criteria for load case V:A and V:B.

Normally the service limit state analysis, load cases V:A and V:B, determines the amount of prestressing needed. This means that often the ultimate limit state analysis, load case IV, will not be decisive according to Karl Lundstedt.

In all the analyses made tendons with 12 Ø 12.9 mm strands in each tendon and a tendon area of 1200 mm^2 were used for the centric prestressing. Calculations of the centric prestressing needed seen in Appendix 7 led to that 48 tendons for the original cross-section, 37 tendons for cross-section A, 33 tendons for cross-section B and 37 tendons for cross-section C was used in the analysis. For internal parabolic and external polygonal prestressing the original amount of tendons designed for the reference bridge was used as a starting point and then in some cases modified for optimal design. This means an average of 26 tendons with 12 Ø 15.7 mm strands in each tendon and a tendon area of 1800 mm^2.

The prestress force was assumed to be close to the capacity of the tendon:

$$P_{prestress} = fst \times As \tag{7.6}$$

7.4.1 Original prestressing arrangement with original cross-section

The average cross-section of the reference bridge was evaluated with the original prestressing layout. This analysis was carried out to simulate scaffolding as the construction method. In this case no centric prestressing was used. By this example both the model and the stress calculations could be verified. The stresses were evaluated with the design criteria in BRO 94. If the stresses were close to the permitted values given in BRO 94 one could be convinced that the model and calculations were carried out in a correct way. To make the verification more significant, one could remove some of the parabolic prestressing tendons and see if the stresses still are within allowable limits.

The table below shows case V:A when the bridge is finalised and the original cross-section as well as original prestressing arrangement was used, see Table 7.4.

Table 7.4 Stresses in load case V:A in the two critical sections with the original cross-section and the original prestressing arrangement. Parabolic prestressing: 26 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]	
Cross-section part:	Top	Bottom
Span 3:	-7.4 Mpa OK	1.0 Mpa OK
Support 3:	-4.6 MPa OK	-15.0 MPa OK

As seen in Table 7.4 all stresses are within permitted values and the sections satisfy the design criteria for load case V:A. At span 3 in the bottom of the cross-section the stress is equal to the allowable value of 1.0 MPa. At support 3 the margins are quite large between the stresses and their allowable values both for top and bottom. For the verification purpose it is the bottom part of span 3 that is interesting. This value is close enough to its allowable value and the conclusion is therefore that the model and calculation seems to be appropriate.

To convince the reader further that the calculations are acceptable 15 % of the parabolic tendons were removed. The stresses for this case (original prestressing arrangement with the original cross-section) with load case V:A for the finalised bridge are presented in Table 7.5.

Table 7.5 Stresses in load case V:A in the two critical sections with original cross-section and 15 % less prestressing than the original prestressing. Parabolic prestressing: 23 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]	
Cross-section part:	Top	Bottom
Span 3:	-7.2 Mpa OK	**3.0 Mpa** **NOT OK**
Support 3:	-2.6 MPa OK	-15.8 MPa OK

In Table 7.5 above it can be seen that the stress for the bottom part of the cross-section in span 3 is not within the allowable range. Thus when removing 15 % of the parabolic tendons the design criterion for load case V:A is not fulfilled. The conclusion of this is that the model and stress calculations are appropriate since the original cross-section is close to optimal and therefore could not manage a 15 % decrease of the prestressing without problem.

When studying how the original cross-section with the original prestressing layout behaves in load case V:B, see Table 7.6, it can be seen that the whole cross-section remains compressed and there is no risk of cracking.

Table 7.6 Stresses in load case V:B in the two critical sections with original cross-section and original prestressing arrangement. Parabolic prestressing: 26 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom
Span 3:	-5.0 Mpa OK	-3.3 MPa OK
Support 3:	-6.5 MPa OK	-10.2 MPa OK

7.4.2 Prestressing alternative 1 with original cross-section

The first parts of the analysis considered the superstructure during launching. The only load acting was the concrete deadweight. The two most critical sections obtained in Chapter 6, see Figure 6.15 and Figure 6.16 were analysed. In Table 7.7 the stresses and design criteria for load case, V:A, can be seen.

Table 7.7 Stresses in load case V:A in the two most critical sections during launching with original cross-section and prestressing arrangement according to alternative 1. Centric prestressing: 48 tendons with 12 Ø 12.9 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]	
Cross-section part:	Top	Bottom
Span 3 where maximum positive moment during launching appear	-13.7 Mpa OK	**1.8 Mpa** **NOT OK**
Support 4 where maximum negative moment during launching apear	-2.2 MPa OK	**-23.8 MPa** **NOT OK**

When studying the stresses and the allowable values from load case V:A in Table 7.7, it can be seen that there are two sections where the stresses reaches unacceptable values. The tension is too high in the bottom of the cross-section in span 3. This occurs when the superstructure including the launching nose has reached 180 m from the abutment, in the point where maximum positive moment during launching is obtained. There is also too high compression in the bottom of the cross-section at support 4. This occurs when the superstructure and its launching nose has reached 233 m from the abutment, in the point where maximum negative moment during launching is obtained.

Since the design criteria for load case V:A is not fulfilled, there is no reason too further analyse the behaviour in load case V:B. The cross-section or the procedure must be changed before the IL technique is applicable. Suggestions for changes could be to use high performance concrete or temporary supports.

However, it could be interesting to see how the original cross-section behaves with prestressing alternative 1 when the bridge is in the service state. The result can be found in Table 7.8 for load cases V:A and V:B.

Table 7.8 Stresses in the service state load cases V:A and V:B in the two critical sections with original cross-section and prestressing arrangement according to alternative 1. Centric prestressing: 48 tendons with 12 Ø 12.9 mm strands. Parabolic prestressing: 26 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-13.9 MPa OK	-6.1 Mpa OK	-11.5 Mpa OK	-10.1 MPa OK
Support 3:	-11.3 MPa OK	**-21.4 MPa** **NOT OK**	-13.4 MPa OK	-16.6 MPa OK

In Table 7.8 it can be seen that at support 3 in the bottom of the cross-section there are too large compressive stresses during load case V:A. Some modification must be made also here in order to fulfil allowable stresses. The most obvious one is to decrease the prestressing force.

7.4.3 Prestressing alternative 1 with cross-section A

As seen in Section 7.4.2 it is not possible to use IL for the original cross-section without changing the material capacities or use temporary supports. Therefore it was interesting to see how a cross-section behaves during the launching and in the service state when it is designed according to Rosignoli's design formulas. The stresses obtained during the launching of the bridge with cross-section A and prestressing arrangement according to alternative 1 can be seen in Table 7.9.

Table 7.9 Stresses in load case V:A in the two most critical sections during launching with cross-section A and prestressing arrangement according to alternative 1. Centric prestressing: 37 tendons with 12 Ø 12.9 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]	
Cross-section part:	Top	Bottom
Span 3 where maximum positive moment during launching appear	-12.2 Mpa OK	**1.6 Mpa** **NOT OK**
Support 4 where maximum negative moment during launching appear	0.1 MPa OK	-18.6 MPa OK

When studying Table 7.9 it can be concluded that there is a problem to fulfil the allowable stress in the bottom of the cross-section in span 3.

One might also notice in Table 7.9 that a slight increase of the centric prestressing could improve the situation. The centric prestressing is increased by 20 % by recommendation of Karl Lundstedt. This means that instead of the original calculated amount of straight tendons (37) a 20 % increased amount of tendons is used (45) in the model. The result from this change can be seen in Table 7.10.

Table 7.10 Stresses in load case V:A in the two most critical sections during launching with cross-section A and prestressing arrangement according to alternative 1. With 20 % increase of the original calculated centric prestressing. Centric prestressing: 45 tendons with 12 Ø 12.9 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]	
Cross-section part:	Top	Bottom
Span 3 where maximum positive moment during launching appear	-13.6 Mpa OK	0.2 Mpa OK
Support 4 where maximum negative moment during launching appear	-1.5 MPa OK	-20.2 MPa OK

It can be concluded when studying Table 7.10 that there is no problem to launch this type of system consisting of cross-section A and prestressing alternative 1. However, it requires slightly more centric prestressing than any of the other modified cross-sections described in this chapter.

For the service state with load cases V:A and V:B the stresses can be viewed in Table 7.11.

Table 7.11 Stresses in the service state under load case V:A and V:B in the two critical sections with cross-section A and prestressing arrangement according to alternative 1. Centric prestressing: 45 tendons with 12 Ø 12.9 mm strands. Parabolic prestressing: 26 tendons with 12 Ø 15.7 mm strands.

Load case an design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-14.1 Mpa OK	-7.5 MPa OK	-11.6 Mpa OK	-11.2 MPa OK
Support 3:	-12.7 MPa OK	-17.2 MPa OK	-14.6 MPa OK	-13.8 MPa OK

The first thing to notice when studying Table 7.11 is that all stresses are within allowable limits. No problems occur and the whole cross-section is under compression. One might also notice that all the values have a large margin to their respective limits.

In an attempt to optimise the model and get the stresses closer to their allowable limits, a decrease of parabolic tendons seem to be a logical approach. At first the parabolic tendons were decreased with 50 %. This action led to that the compressive stress in the bottom of support 3 grew to large as seen in Table 7.12.

Table 7.12 Stresses in the service state under load cases V:A and V:B in the two critical sections with cross-section A and prestressing arrangement according to alternative 1 and with the parabolic tendons reduced by 50 %. Centric prestressing: 45 tendons with 12 Ø 12.9 mm strands. Parabolic prestressing: 13 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-14.1 MPa OK	-2.2 MPa OK	-11.6 MPa OK	-5.9 MPa OK
Support 3:	-5.6 MPa OK	**-19.8 MPa** **NOT OK**	-7.6 MPa OK	-16.4 MPa OK

In an attempt to optimize the prestressing and receive values within allowable limits the parabolic prestressing force was reduced by 40 %. In Table 7.13 below the result from this change can be seen.

Table 7.13 Stresses in the service state under load cases V:A and V:B in the two critical sections with cross-section A and prestressing arrangement according to alternative 1 and with the parabolic tendons reduced by 40 %. Centric prestressing: 45 tendons with 12 Ø 12.9 mm strands. Parabolic prestressing: 16 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-14.1 MPa OK	-3.3 MPa OK	-11.6 Mpa OK	-7.0 MPa OK
Support 3:	-7.0 MPa OK	-19.2 MPa OK	-9.0 MPa OK	-15.9 MPa OK

When studying Table 7.13 it can be seen that the stress in the bottom of support 3 for load case V:A is equal to its allowable limit.

The conclusion of this analysis was that cross-section A is a good cross-section to use together with prestressing alternative 1. It needs a slightly larger amount of centric prestressing than the other modified cross-sections in this chapter. However, the fact that the parabolic tendons can be decreased with 40 % compensates this.

7.4.4 Prestressing alternative 1 with cross-section B

When more effort was put into optimising cross-section B was obtained. With even less concrete but still with a great depth this should be a very effective cross-section. In Table 7.15 the stresses in the most critical sections during launching can be found.

Table 7.15 Stresses in load case V:A in the two most critical sections during launching with cross-section B and prestressing arrangement according to alternative 1. Centric prestressing: 33 tendons with 12 Ø 12.9 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]	
Cross-section part:	Top	Bottom
Span 3 where maximum positive moment during launching appear	-12.9 MPa OK	0.2 MPa OK
Support 4 where maximum negative moment during launching appear	-0.5 MPa OK	-17.5 MPa OK

The most important fact received from this analysis is that no stresses exceed their allowable limits. Cross-section B together with prestressing alternative 1 and the estimated amount of centric prestressing could therefore be used with the incremental launching technique without any problems.

The next step was to analyse how this cross-section and prestressing layout will behave in the service state. In Table 7.16 it can be seen that the whole cross-section is under quite large compressive stress.

Table 7.16 Stresses in the service state under load cases V:A and V:B in the two critical sections with cross-section B and prestressing arrangement according to alternative 1. Centric prestressing: 33 tendons with 12 Ø 12.9 mm strands. Parabolic prestressing: 26 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-13.5 MPa OK	-7.9 MPa OK	-10.6 Mpa OK	-11.4 MPa OK
Support 3:	-13.3 MPa OK	-15.3 MPa OK	-15.5 MPa OK	-12.1 MPa OK

It is quite easy to draw the conclusion that the prestressing force is too large. The prestressing force was reduced by 50 % in an attempt to improve the situation. As seen in Table 7.17 below the decrease of the parabolic prestressing force in the service state results in values more close to their allowable limits.

Table 7.17 Stresses in the service state under load cases V:A and V:B in the two critical sections with cross-section B and prestressing arrangement according to alternative 1 with the parabolic tendons reduced by 50 %. Centric prestressing: 33 tendons with 12 Ø 12.9 mm strands. Parabolic prestressing: 13 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-13.6 MPa OK	-2.7 MPa OK	-10.7 MPa OK	-6.2 MPa OK
Support 3:	-5.5 MPa OK	-17.2 MPa OK	-7.8 MPa OK	-14.0 MPa OK

Although all values are acceptable it could be interesting to see if the prestressing achieved by parabolic tendons could be decreased even further. This prestressing force was reduced by 75 % and the stresses obtained are presented in Table 7.18.

Table 7.18 Stresses in the service state under load cases V:A and V:B in the two critical sections with cross-section B and prestressing arrangement according to alternative 1 with the parabolic tendons reduced by 75 %. Centric prestressing: 33 tendons with 12 Ø 12.9 mm strands. Parabolic prestressing: 7 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-13.6 MPa OK	0.0 MPa OK	-10.8 Mpa OK	-3.6 MPa OK
Support 3:	-1.3 MPa OK	-17.8 MPa OK	-3.6 MPa OK	-14.7 MPa OK

Even with this large decrease of the prestressing force all stresses are within their allowable limits for both load cases V:A and V:B. When the prestressing force was reduced by 90 % this section failed to fulfil its allowable value, see Table 7.19 below.

Table 7.19 Stresses in the service state under load cases V:A and V:B in the two critical sections with cross-section B and prestressing arrangement according to alternative 1 with the parabolic tendons reduced by 90 %. Centric prestressing: 33 tendons with 12 Ø 12.9 mm strands. Parabolic prestressing: 3 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-13.7 MPa OK	**1.6 Mpa NOT OK**	-10.8 Mpa OK	-2.0 MPa OK
Support 3:	0.7 MPa OK	-18.8 MPa OK	-1.5 MPa OK	-15.6 MPa OK

A conclusion for this cross-section and prestressing arrangement is that it is possible to reduce the amount of parabolic tendons radically (75 %) in comparison with the original bridge designed by ELU Konsult AB.

7.4.5 Prestressing alternative 2 with the original cross-section

If the original cross-section should be combined with prestressing alternative 2, which has external polygonal tendons and internal straight tendons. One could expect that the slight decrease in eccentricity for the polygonal tendons would lead to higher stresses than the original cross-section together with prestressing alternative 1. On the contrary the straight tendons in the top and bottom flanges might compensate this decrease in eccentricity, and without tendons in the web the greater volumes of concrete could in fact result in an improved performance compared to the one tested in Section 7.4.2.

Since the only difference between prestressing alternatives 1 and 2 are that internal parabolic or external polygonal tendons are used there will be no difference in the stresses caused during launching. These tendons are applied to the bridge after the launching is completed. See Table 7.7 for the stresses during launching. Of course the same conclusion drawn in Section 7.4.2 regarding the launching can be drawn here. The bottom part of the cross-section has difficulties to fulfil the allowable values for both maximum positive and maximum negative moments regarding the design criteria of V:A. The stress analysis for the bridge in the service state can be seen in Table 7.20 below.

Table 7.20 Stresses in the service state under load case V:A and V:B in the two critical sections along the bridge with original cross-section and prestressing arrangement according to alternative 2. Centric prestressing: 48 tendons with 12 Ø 12.9 mm strands. Polygonal prestressing: 26 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-9.5 MPa OK	-1.5 MPa OK	-7.0 Mpa OK	-6.3 MPa OK
Support 3:	-3.1 MPa OK	-15.4 MPa OK	-5.3 MPa OK	-10.5 MPa OK

In Table 7.20 above it can be observed that in difference from the system of original cross-section together with prestressing alternative 1 all values are within allowable limits. As with the case of prestressing alternative 1 and the original cross-section it can be stated that the prestressing can be decreased since large compressive stresses are induced in the whole cross-section. In order to check how large reduction one could make the prestressing force for the external polygonal tendons was reduced by 25 %. The results from this analysis can be viewed in Table 7.21 below.

Table 7.21 Stresses in the service state under load case V:A and V:B in the two critical sections with original cross-section and prestressing arrangement according to alternative 2 with the parabolic tendons reduced by 25 %. Centric prestressing: 48 tendons with 12 Ø 12.9 mm strands. Polygonal prestressing: 20 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-10.7 MPa OK	**1.1 Mpa** **NOT OK**	-8.3 Mpa OK	-3.8 MPa OK
Support 3:	-1.9 MPa OK	-18.1 MPa OK	-4.1 MPa OK	-13.2 MPa OK

It is only in the bottom of the cross-section in span 3 during load case V:A that the allowable stress is exceeded. The allowable stress of < 1.0 MPa was exceeded with just 0.1 MPa in this section it is reasonable to argue that some reduction (about 20 %) of the polygonal tendons can be made for the original cross-section and prestressing alternative 2.

7.4.6 Prestressing alternative 2 with the cross-section C

Since the original cross-section cannot be launched without necessary measures taken, it was interesting to analyse a cross-section that is better optimised for IL and prestressing alternative 2. Since the polygonal tendons are positioned externally it is possible to make the webs thinner and thereby save concrete and decrease the deadweight. This new cross-section was denoted C and can be found in Section 7.2.1.

Since this cross-section was designed with help of Rosignoli’s formulas developed especially for IL, one can expect no problem during launching. The stress analysis from the launching can be seen in Table 7.22 below.

Table 7.22 Stresses in load case V:A in the two most critical sections during launching with cross-section C and prestressing arrangement according to alternative 2. Centric prestressing: 37 tendons with 12 Ø 12.9 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]	
Cross-section part:	Top	Bottom
Span 3 where maximum positive moment during launching appear	-12.5 MPa OK	0.0 MPa OK
Support 4 where maximum negative moment during launching appear	-1.8 MPa OK	-18.5 MPa OK

As expected with this cross-section there was no difficulties during the launching. The stresses in the service state under load cases V:A and V:B are presented in Table 7.23 below.

Table 7.23 Stresses in the service state under load case V:A and V:B in the two critical sections with cross-section C and prestressing arrangement according to alternative 2. Centric prestressing: 37 tendons with 12 Ø 12.9 mm strands. Polygonal prestressing: 26 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-10.2 MPa OK	-1.5 MPa OK	-7.7 MPa OK	-5.6MPa OK
Support 3:	-3.1 MPa OK	-13.5 MPa OK	-5.3 MPa OK	-9.7 MPa OK

When analysing the results in Table 7.23 it can be stated that all values are within their allowable limits and most of the values have some margin as well. In order to optimise the prestressing layout the external polygonal tendons were reduced.

The prestressing force was reduced by 50 %. The result of this modification is presented in Table 7.24 below. As seen the stresses in the bottom of span 3 exceeds the allowable value. Because the value only exceeds the allowable limit with 0.2 MPa one could assume that a reduction with 40 % would be possible.

Table 7.24 Stresses in the service state under load case V:A and V:B in the two critical sections with cross-section C and prestressing arrangement according to alternative 2 and with the polygonal tendons reduced by 50 %. Centric prestressing: 37 tendons with 12 Ø 12.9 mm strands. Parabolic prestressing: 13 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-11.9 MPa OK	**1.2 MPa NOT OK**	-9.4 Mpa OK	-2.8 MPa OK
Support 3:	-0.6 MPa OK	-18.0 MPa OK	-2.9 MPa OK	-14.2 MPa OK

The conclusion here is that with regard to the design criteria for load case V:A and V:B the polygonal prestressing could be halved and still result in acceptable stresses in the critical sections.

7.4.7 Prestressing alternative 2 with the cross-section B

Cross-section B was not designed for prestressing alternative 2 but it could be interesting to see how this system would behave. During launching it will behave the same as seen in Section 7.4.4 Table 7.15. The stresses obtained in the service state can be seen in Table 7.25.

Table 7.25 Stresses in the service state under load case V:A and V:B in the two critical sections with cross-section B and prestressing arrangement according to alternative 2. Centric prestressing: 33 tendons with 12 Ø 12.9 mm strands. Polygonal prestressing: 26 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-10.1 MPa OK	-2.1 MPa OK	-7.3 MPa OK	-5.7 MPa OK
Support 3:	-2.5 MPa OK	-12.3 MPa OK	-4.8 MPa OK	-9.2 MPa OK

The whole cross-section is under compression and there is room for a decrease of the polygonal prestressing tendons. The amount is decreased with 50 % and the result of this can be found in Table 7.26.

Table 7.26 Stresses in the service state under load case V:A and V:B in the two critical sections with cross-section B and prestressing arrangement according to alternative 2 and with the polygonal tendons reduced by 50 %. Centric prestressing: 33 tendons with 12 Ø 12.9 mm strands. Polygonal prestressing: 13 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-11.9 MPa OK	0.2 MPa OK	-9.1 Mpa OK	-3.4 MPa OK
Support 3:	0.1 MPa OK	-16.2 MPa OK	-2.1 MPa OK	-13.0 MPa OK

With this decrease in polygonal prestressing there are two section that reach tension stresses in the results shown in Table 7.26 above. However, all the results are within allowable limits.

The polygonal prestressing is reduced by 65 %. Results from this change can be seen in Table 7.27.

Table 7.27 Stresses in the service state under load case V:A and V:B in the two critical sections with cross-section B and prestressing arrangement according to alternative 2 and with the polygonal tendons reduced by 65 %. Centric prestressing: 33 tendons with 12 Ø 12.9 mm strands. Polygonal prestressing: 10 tendons with 12 Ø 15.7 mm strands.

Load case and design criteria:	**V:A** $-19.2 < \sigma_{Top} < 1.4$ [MPa] $-19.2 < \sigma_{Bottom} < 1.0$ [MPa]		**V:B** $-19.2 < \sigma < 0.0$ [MPa]	
Cross-section part:	Top	Bottom	Top	Bottom
Span 3:	-12.5 MPa OK	1.0 MPa OK	-9.6 Mpa OK	-2.6 MPa OK
Support 3:	1.0 MPa OK	-17.4 MPa OK	-1.2 MPa OK	-14.2 MPa OK

When studying Table 7.27 it can be seen that in the bottom of the cross-section in span 3 the allowable limit is precisely met. It can be concluded that if this cross section would be used together with prestressing alternative 2 the polygonal tendons could be reduced by 65 % compared to the parabolic tendons needed for the reference bridge.

7.5 Needed amounts of prestressing tendons and concrete

With the results of the stress analyses made in Section 7.4 it is possible to calculate the final amount of prestressing tendons needed for the different prestressing arrangements. The calculations for the different volumes of material were based on equations found in Appendix 7. The amount of prestressing tendons was calculated by multiplying the total area of the prestressing tendons used in the cross-section with the length of the bridge. The amount of concrete was calculated by multiplying the concrete area minus the area of the tendons with the length of the bridge. The results of these calculations can be seen in Table 7.28.

*Table 7.28 Summation of prestressing tendons and concrete for the alternatives evaluated. * The value could be reduced but was not considered important for this project since it failed the launching process.*

Prestressing arrangements:	**Cross-section type:**	**Prestressing tendons [m^3]:**			**Concrete [m^3]: (including ordinary reinforcement)**
		Centric:	**Parabolic and polygonal:**	**Total:**	
Reference bridge arrangement:	**Original**	**0**	**11.4**	**11.4**	**1537.1**
Prestressing alternative 1:	**Original (fail IL)**	*13,8*	*11.4**	*25.2*	**1546.7**
	A	**13.0**	**6.9**	**19.9**	**1766.4**
	B	**9,4**	**2.9**	**12.3**	**1628.6**
Prestressing alternative 2:	**Original (fail IL)**	*13.8*	*9.1*	**22.9**	**1558.2**
	C	**10.6**	**6.9**	**17.5**	**1556.2**
	B	**9.4**	**4.0**	**13.4**	**1631.5**

It can be concluded that the original cross-section was not possible to use with the incremental launching technique. The depth of this cross-section is most likely not high enough. The cross-sections that were more optimised A, B and C all managed the IL procedure regardless of prestressing alternative used. Alternative 1 with cross-section B is the one with least amount of tendons and the system used for the bridges built with IL in Sweden. Alternative 2 with cross-section C requires least amount of concrete, this is the result of the external prestressing that makes it possible to use thinner webs.

8 Conclusions

When summing up all the information gathered during this project some useful conclusions could be made which are presented in this chapter.

There are some obvious differences between incremental launched bridges and bridges built with scaffolding. For IL built bridges there will be larger volumes of concrete and prestressing. An IL bridge will have a constant cross-section that result in overcapacity in certain sections. The spans suited for incremental launching are limited and the total length of the bridge must also satisfy a certain range in order to make the incremental launching method an economical alternative.

On the other hand it is constructed in a factory like environment under more controlled and secure conditions than a bridge constructed with scaffolding. It will always be a faster and more labour efficient way of building a bridge. Using a construction yard provides a safer environment for the workers. The production will not be affected of different weather conditions. Since the bridge is launched there will be no interference below the bridge spans and therefore it is possible to have traffic and other activities under the bridge during construction. Much of the equipment used while launching the bridge can be reused in other bridge projects, thus making the method cheaper at each new project carried out.

The study trip to the Rybný Potok bridge in the Czech Republic gave valuable hands-on experience of the incremental launching technique. Interesting knowledge was obtained from the designer of the bridge such as the allowable limit for tensile stress was 2 MPa during launching. In the Swedish code BRO 94 the allowable tensile stress in the top and bottom of the cross-section is 1.4 MPa respectively 1.0 MPa. If 2 Mpa had been used for the simulation of the reference bridge during the launching it would have resulted in more values within the allowable limits.

The investigation of the launching nose resulted in a very informative chapter that can be used to acquire knowledge of the launching nose. The most important facts gathered were that the approximate length of a launching nose should be 60 - 65 % of the longest span and that the nose weight should be approximately 10 % of the concrete superstructure. This knowledge was used in the modelling of the launching nose in BRIGADE/Standard.

From the results obtained in Chapter 7 several conclusions can be drawn; one can understand that dimensions of the cross-section are very important. One of the most obvious conclusions concerning the different cross-section alternatives was that launching of the superstructure was not possible for the reference bridge at Vindeln. This is perhaps an obvious result but still an important conclusion since the focus of this master's project is the comparison between incremental launching and scaffolding. In order to be able to construct the bridge at Vindeln with IL the depth of the superstructure must be increased. The stress analysis shows that if the depth was increased to 3.3 m according to Rosignoli's formulas it will be no problem to launch the bridge.

Of course one could use high performance concrete or temporary supports during the launching and thereby decrease the spans and make it possible to launch the cross-section of the reference bridge. However, this is not the best solution since more supports will be expensive and disturb the environment below the bridge. The use of high performance concrete would also be expensive.

In the service state with prestressing alternative 1 and the original cross-section some decrease of the prestressing must be done in order to reach allowable limits. The centric prestressing together with the parabolic prestressing induces too large compressive stresses in the cross-section. If prestressing alternative 2 was used all the stresses were within their allowable limits in the service state without any modification. However, as with prestressing alternative 1 the stress calculations showed that some reduction could be made of the parabolic prestressing compared to the amount used for the reference bridge.

When the modified and more optimally designed cross-section A was used for IL together with prestressing alternative 1 the launching could be performed. The stress analysis shows that slightly more centric prestressing was needed for this cross-section in order to be launched than for any of the other modified cross-sections. In the service state it was concluded that the parabolic prestressing could be reduced with 40 % compared to the amount used in the reference bridge.

When cross-section B was put to test with prestressing alternative 1 the launching could be performed with slightly less amount of centric prestressing tendons then for cross-section A. The new cross-section design with less concrete and therefore smaller moments from the deadweight was most likely the reason for the success. In service state the compressive stresses were high and the parabolic prestressing could be reduced with 75 % compared to the parabolic prestressing used in the reference bridge. This made it possible to save large amounts of steel for the parabolic tendons. The conclusion drawn was that cross-section B and prestressing alternative 1 is a good solution if the bridge over Vindeln would be constructed with incremental launching.

When using cross-section C with prestressing alternative 2 neither the launching nor the service state of the bridge resulted in problems regarding the stresses. It was shown that the polygonal prestressing for alternative 2 with cross-section C could be reduced with 50 % compared to the prestressing used for the reference bridge. It could therefore be concluded that large savings regarding tendons could be made for the polygonal prestressing.

Cross-section B was also evaluated with prestressing alternative 2. The results from stress analysis were satisfying although this cross-section was not optimised regarding the web thickness for external prestressing. The polygonal prestressing could be reduced with 65 % compared to the amount of parabolic prestressing used in the reference bridge.

To sum up, the largest savings in material for parabolic tendons was made with cross-section B and prestressing alternative 1. As mentioned earlier more than 75 % of the parabolic tendons used for the reference bridge could be eliminated with this system. This would be the best choice for the bridge if it would be constructed with IL. In Chapter 7 a summation of the amounts of prestressing tendons and concrete were presented for the different alternatives evaluated. In order to make it easier for the reader the results are also presented below in Table 8.1.

*Table 8.1 Summation of prestressing tendons and concrete for the alternatives evaluated. * The value could be reduced but was not considered important for this project since it failed the launching process.*

Prestressing arrangements:	**Cross-section type:**	**Prestressing tendons [m^3]:**			**Concrete [m^3]: (including ordinary reinforcement)**
		Centric:	**Parabolic and polygonal:**	**Total:**	
Reference bridge arrangement:	**Original**	**0**	**11.4**	**11.4**	**1537.1**
Prestressing alternative 1:	**Original (fail IL)**	*13,8*	*11.4**	*25.2*	**1546.7**
	A	**13.0**	**6.9**	**19.9**	**1766.4**
	B	**9,4**	**2.9**	**12.3**	**1628.6**
Prestressing alternative 2:	**Original (fail IL)**	*13.8*	*9.1*	**22.9**	**1558.2**
	C	**10.6**	**6.9**	**17.5**	**1556.2**
	B	**9.4**	**4.0**	**13.4**	**1631.5**

As one would expect the reference bridge had the lowest amounts of tendons and concrete when comparing the different systems. Although if prestressing alternative 1 were used together with cross-section B the total amounts of tendons would differ 8 % and the amount of concrete 6 % compared with the reference bridge. This system has been used for all bridges constructed with IL in Sweden. However, cross-section B differs architecturally from the reference bridge, the arched webs was eliminated. If the architectural aspects should be considered and the bridge must be constructed with IL, prestressing alternative 2 with cross-section C or prestressing alternative 1 with cross-section A are good options. With Cross-section C and prestressing alternative 2 the amount of concrete was almost the same as for the reference bridge despite the greater depth. This was because the web thickness could be decreased for prestressing alternative 2, and thereby concrete savings were made. Regarding the total amount of prestressing, 43 % more tendons were required for prestressing alternative 2 with cross-section C then for the reference bridge. Slightly more tendons were required for cross-section A and prestressing alternative 1 and the amount of concrete was quite large.

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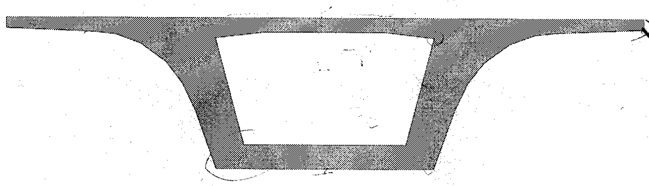
Appendix 1 - Cross-section properties

Extract from cross-section properties for the reference bridge calculated in FEM Design Section Editor



Sectional data

A	= 7.145 m2
P	= 34.06 m
A/P	= 0.2098 m
Yg	= 0,0000 m
Zg	= 1,527 m
Ys	= 0.0000 m
Zs	= 0.9605 m
Iy	= 5.427 m4
Ww Wy	= 3.555 m3
ez max	= 1.527 m
ez min	= 0.9732 m
iy	= 0.8716 m
Sy	= 2.801 m3
Rho y	= 0.5875
Iz	= 37.70 m4
W5 Wz	= 7.112 m3
ey max	= 5.300 m
ey min	= 5.300 m
iz	= 2.297 m
Sz	= 7.182 m3
Rho z	= 0.3136
It	= 10.04 m4
Wt	= 3.745 m3
Igamma	= 4.776 m6
Iyz	= 0.0000 m4
Rho yz	= 0.0000
alpha1	= 90.00 deg
I1	= 37.70 m4
W1 ok	= 7.112 m3
e2 max	= 5.300 m
e2 min	= 5.300 m
i1	= 2.297 m
S1	= 7.182 m3
So1	= 7.182 m3
c1	= 2.019
Rho 1	= 0.3136
alpha2	= 0.0000 deg
I2	= 5.427 m4
W2 ok	= 3.555 m3
e1 max	= 1.527 m
e1 min	= 0.9732 m
i2	= 0.8716 m
S2	= 2.801 m3
So2	= 2.710 m3
c2	= 1.525
Rho 2	= 0.5875

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Signature		Comments	

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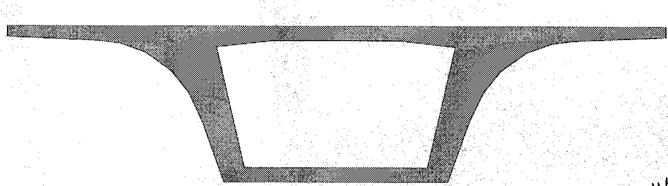

Sectional data

A	= 6.140 m2
P	= 34.85 m
A/P	= 0.1762 m
Yg	= 0.0000 m
Zg	= 1.650 m
Ys	= 0.0000 m
Zs	= 1.222 m
Iy	= 4.412 m4
Wy	= 2.673 m3
ez max	= 1.650 m
ez min	= 0.8497 m
iy	= 0.8477 m
Sy	= 2.267 m3
Rho y	= 0.5485
Iz	= 35.99 m4
Wz	= 6.790 m3
ey max	= 5.300 m
ey min	= 5.300 m
iz	= 2.421 m
Sz	= 6.587 m3
Rho z	= 0.3006
It	= 8.730 m4
Wt	= 3.300 m3
Igamma	= 4.065 m6
Iyz	= 0.0000 m4
Rho yz	= 0.0000
alpha1	= 90.00 deg
I1	= 35.99 m4
W1	= 6.790 m3
e2 max	= 5.300 m
e2 min	= 5.300 m
i1	= 2.421 m
S1	= 6.587 m3
So1	= 6.587 m3
c1	= 1.940
Rho 1	= 0.3006
alpha2	= 0.0000 deg
I2	= 4.412 m4
W2	= 2.673 m3
e1 max	= 1.650 m
e1 min	= 0.8497 m
i2	= 0.8477 m
S2	= 2.267 m3
So2	= 2.113 m3
c2	= 1.581
Rho 2	= 0.5485

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Signature		Comments	

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Appendix 2 - Verification of the model

Comparison of section forces between BRIGADE/Standard and ELU Konsult AB's Strip Step 3

Comparison of results concerning max-moments caused by Temperature difference			
Bridge section	Brigade/Standard	Strip Step 3	Difference
Support 1	-	-	-
Span 1	3772	3754	0.4 %
Support 2	7545	7495	0.7 %
Span 2	6514	6599	1.3 %
Support 3	5746	5701	0.8 %
Span 3	5728	5704	0.4 %
Support 4	5711	5701	0.2 %
Span 4	6544	6601	0.9 %
Support 5	7642	7495	2.0 %
Span 5	3821	3743	2.1 %
Support 6	-	-	-

Comparison of results concerning moments caused by Deadweight			
Bridge section	Brigade/Standard	Strip Step 3	Difference
Support 1	-	-	-
Span 1	13720	13779	0.4 %
Support 2	-35450	-35811	1 %
Span 2	16800	18017	7 %
Support 3	-37000	-38303	3.5 %
Span 3	15900	16763	5.4 %
Support 4	-37070	-38325	4 %

Span 4	16760	18021	7.5 %
Support 5	-35620	-35790	0.4 %
Span 5	13390	13843	3.3 %
Support 6	-	-	-

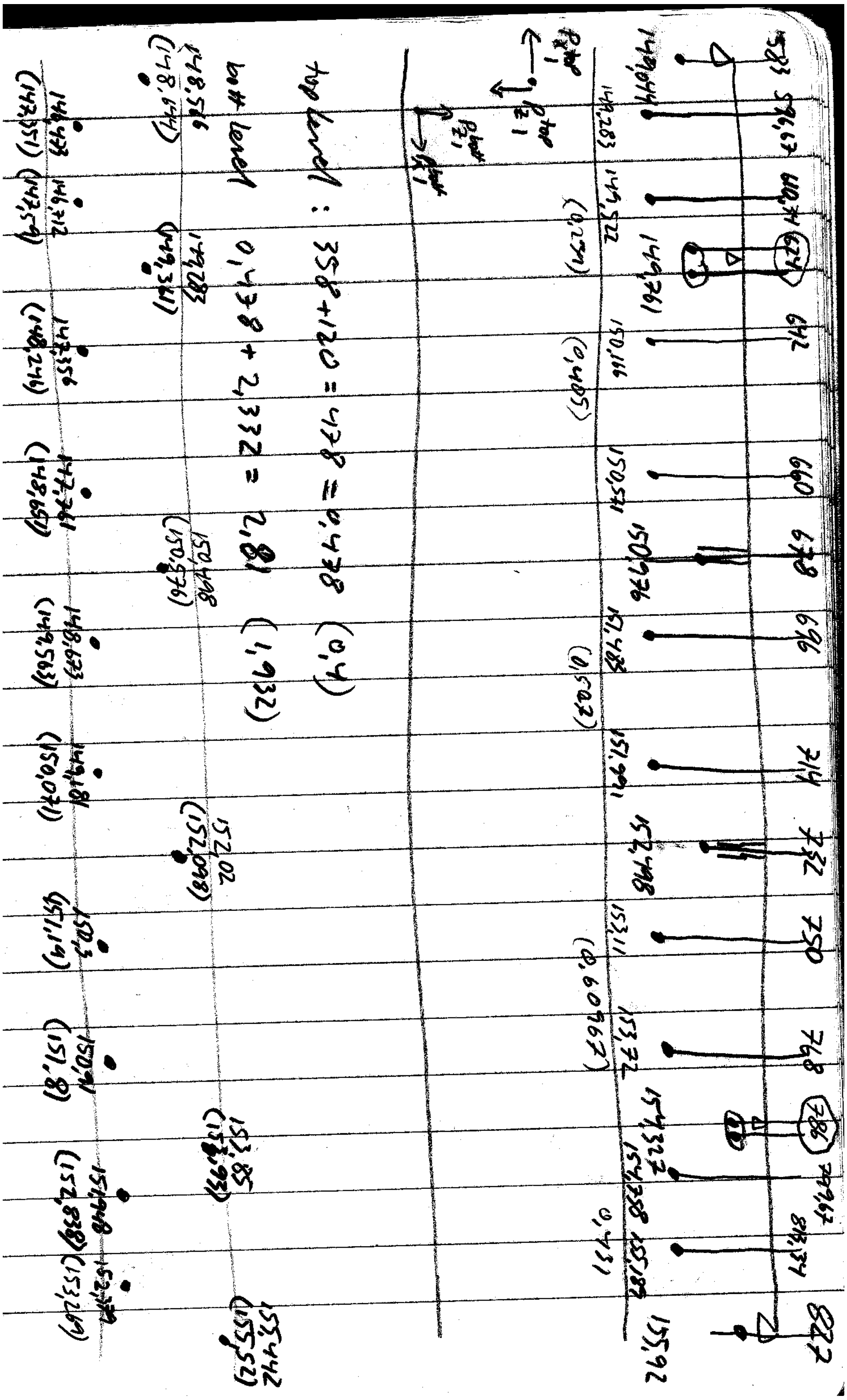
Comparison of results concerning moments caused Surfacing			
Bridge section	Brigade/Standard	Strip Step 3	Difference
Support 1	-	-	-
Span 1	2000	2077	3.9 %
Support 2	-5266	-5306	0.8 %
Span 2	2574	2720	5.7 %
Support 3	-5584	-5679	1.7 %
Span 3	2424	2532	4.4 %
Support 4	-5606	-5682	1.4 %
Span 4	2544	2720	6.9 %
Support 5	-5374	-5303	1.3 %
Span 5	2070	2087	0.8 %
Support 6	-	-	-

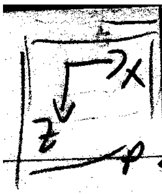
Comparison of results concerning reaction forces caused by Deadweight			
Bridge section	Brigade/Standard	Strip Step 3	Difference
Support 1	-2334	-2771	15.8 %
Support 2	-8407	-8397	0.1 %
Support 3	-8557	-8603	0.5 %
Support 4	-8538	-8604	0.8 %
Support 5	-8459	-8396	0.8 %

Support 6	-2409	-2772	15.1 %
Comparison of results concerning moments caused by Prestressing			
Bridge section	Brigade/Standard	Strip Step 3	Difference
Span 1	-26760	-30016	12 %
Support 2	44410	46438	4.6 %
Span 2	-27610	-29931	8.4 %
Support 3	47430	47636	0.4 %
Span 3	-26520	-29114	9.8 %
Support 4	47230	47784	1.2 %
Span 4	-28000	-30006	7.2 %
Support 5	44450	49015	10 %
Span 5	-29060	-30152	3.8 %

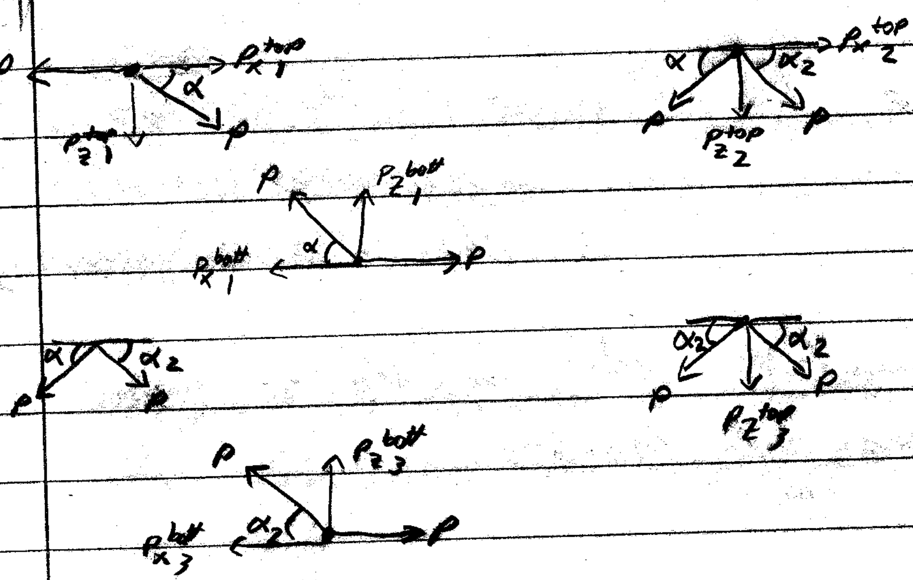
Appendix 3 - The derivation of the external prestressing force components

Extract from the derivation of the external force components for the modelling of the external prestressing





$\alpha = 7,7\ (6,4)$ $\quad P = 28499$ $\quad \alpha_2 = 7,4\ (4,9)$



$P_{x_1}^{top} = P - P \cdot \cos\alpha = (-407,4)\,kN \quad (-177,6)\,kN$

$P_{z_1}^{top} = P \cdot \sin\alpha = -4801,8\,kN \quad (3176,8)\,kN$

$P_{x_1}^{bott} = 407,4\,kN \quad (177,6)\,kN$

$P_{z_1}^{bott} = -4801,8\,kN \quad (-3176,8)\,kN$

$P_{x_2}^{bott} = (-407,4)\,kN \quad (-177,6)\,kN$

$P_{z_2}^{bott} = -4801,8\,kN \quad (-3176,8)\,kN$

$P_{x_2}^{top} = P \cdot \cos\alpha_2 - P \cdot \cos\alpha = 170\,kN \quad (73,5)\,kN$

$P_{z_2}^{top} = P \cdot \sin\alpha_2 + P \cdot \sin\alpha = -8472,3\,kN \quad (5611)\,kN$

$P_{x_3}^{bott} = P - P\cos\alpha_2 = 237,4\,kN \quad (104,2)\,kN$

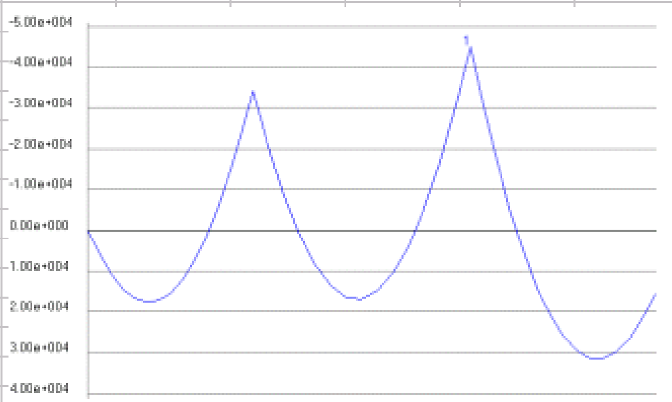
$P_{z_3}^{bott} = P \cdot \sin\alpha_2 = -3670,5\,kN \quad (-2434,3)\,kN$

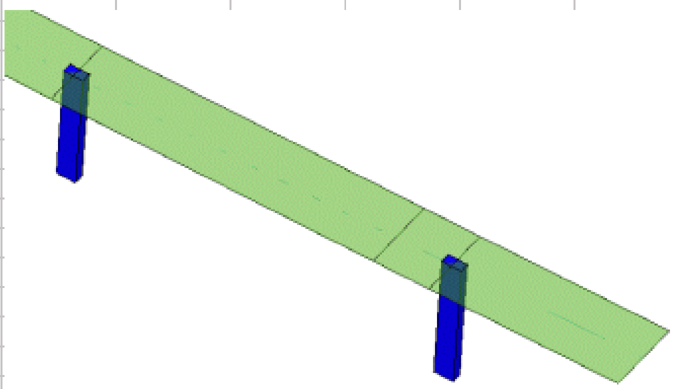
$P_{z_3}^{top} = 2P \cdot \sin\alpha_2 = 7341,1\,kN \quad (4868,6)\,kN$

Appendix 4 - Moments during incremental launching

Extract from the moment results during the incremental launching

Result Set Name: end of nose at 759		
Case 1: Dead Weight, Bending Moment Mh (kNm)		
Node	Position	Moment
1151	0	337,2
1152	2,93	5958
1153	5,86	10900
1154	8,79	14500
1155	11,72	16750
1156	14,65	17650
1157	17,57	17210
1158	20,5	15410
1159	23,43	12260
1160	26,36	7771
1161	29,29	1929
1162	32,22	-5262
1163	35,15	-13800
1164	38,08	-23690
1165	41,01	-34250
1166	41,01	-34000
1167	44,86	-20730
1168	48,72	-8622
1169	52,58	1144
1170	56,44	8571
1171	60,3	13660
1172	64,15	16400
1173	68,01	16810
1174	71,87	14880
1175	75,73	10610
1176	79,59	3998
1177	83,45	-4952
1178	87,3	-16240
1179	91,16	-29870
1180	95,02	-44670
1181	95,02	-44870
1182	98,31	-30360
1183	101,59	-16710
1184	104,88	-4749
1185	108,17	5512
1186	111,46	14070
1187	114,74	20940
1188	118,03	26110
1189	121,32	29580
1190	124,6	31350
1191	127,89	31420
1192	131,18	29790
1193	134,46	26470
1194	137,75	21450
1195	141,04	15580
ok	max:	31420
	min:	-44870

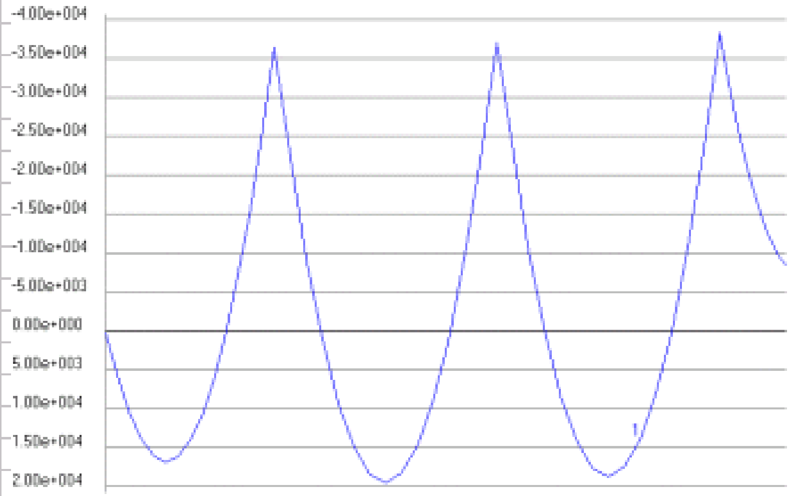


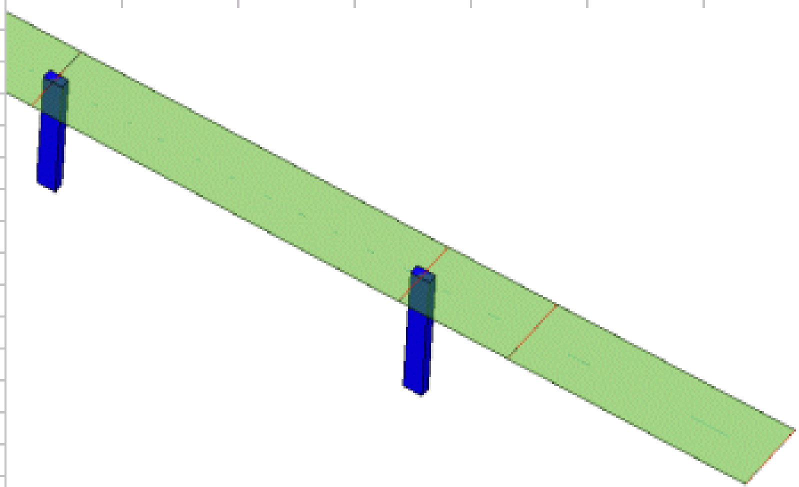


Result Set Name: end of nose at 783

Case 1: Dead Weight, Bending Moment Mh (kNm)

Node	Position	Moment
1166	0	337,2
1169	8,79	14050
1170	11,72	16150
1171	14,65	16890
1172	17,57	16300
1173	20,5	14350
1175	26,36	6405
1176	29,29	411,6
1177	32,22	-6931
1178	35,15	-15620
1179	38,08	-25660
1180	41,01	-36380
1181	41,01	-36130
1183	48,72	-9334
1184	52,58	1137
1185	56,44	9270
1186	60,3	15060
1187	64,15	18520
1188	68,01	19630
1190	75,73	14840
1191	79,59	8932
1192	83,45	687,3
1193	87,3	-9897
1194	91,16	-22820
1195	95,02	-36920
1196	95,02	-36920
1197	98,88	-22930
1198	102,74	-10120
1199	106,6	346,5
1200	110,45	8477
1201	114,31	14270
1203	122,03	18830
1204	125,89	17600
1205	129,75	14030
1206	133,61	8123
1208	141,32	-10710
1209	145,18	-23640
1210	149,04	-37740
1211	149,04	-38280
1212	150,18	-34890
1213	151,33	-31620
1214	152,47	-28540
1215	153,62	-25670
1216	154,76	-23010
1218	157,05	-18300
1219	158,19	-16260
1220	159,33	-14420
1221	160,48	-12780
1222	161,62	-11350
1223	162,76	-10130
1225	165,05	-8195





ok

max: 19630

min: -38280

The summation of the moment peaks from all the runnings:

Nose end	Max moment	Min moment	
759	31420	-44870	
761	32030	-44900	
763	32190	-44830	
765	31910	-44640	
767	31480	-44580	
769	30770	-44200	
771	29750	-43650	
773	28450	-42940	
775	26860	-42060	
777	24980	-41020	
779	23020	-39820	
781	21040	-38450	
783	19630	-38280	
784	19930	-41490	
785	20240	-44850	
786	20080	-37820	
787	19610	-37500	
788	19490	-37310	
789	19590	-37930	
790	19640	-38540	
792	19760	-39800	
794	19860	-40950	
796	19940	-41770	
798	20000	-42430	
800	20830	-42960	
802	23160	-43510	
804	25370	-43910	
806	27320	-44260	
808	28730	-44640	
810	30130	-44900	
812	31050	-45120	
814	31610	-45200	
816	32090	-45210	
818	31990	-45060	
820	31650	-44700	
822	31220	-44330	
824	30360	-43840	
826	29210	-43190	
827	28530	-42800	
827	27450	-42190	
Max moment	32190		Nose end 763
Min moment		-45210	Nose end 816

Appendix 5 - Sectional forces in the reference bridge

Extracts from the sectional force results from the BRIGADE/Standard runs

BRIGADE/Standard version 3.4.7	
Project File Name:	Bridge with average cross-section without parabolic
Case 1: Dead Weight, Axial Force N (kN)	
Case 2: Dead Weight, Bending Moment Mh (kNm)	
Case 3: Comb 4A. Ultimate Limit State, Max, Axial Force N (kN)	
Case 4: Comb 4A. Ultimate Limit State, Max, Bending Moment Mh (kNm)	
Case 5: Comb 4A. Ultimate Limit State, Min, Axial Force N (kN)	
Case 6: Comb 4A. Ultimate Limit State, Min, Bending Moment Mh (kNm)	
Case 7: Comb 5A. Service. Limit State, Max, Axial Force N (kN)	
Case 8: Comb 5A. Service. Limit State, Max, Bending Moment Mh (kNm)	
Case 9: Comb 5A. Service. Limit State, Min, Axial Force N (kN)	
Case 10: Comb 5A. Service. Limit State, Min, Bending Moment Mh (kNm)	
Case 11: Comb 5B. Crack Width, Max, Axial Force N (kN)	
Case 12: Comb 5B. Crack Width, Max, Bending Moment Mh (kNm)	
Case 13: Comb 5B. Crack Width, Min, Axial Force N (kN)	
Case 14: Comb 5B. Crack Width, Min, Bending Moment Mh (kNm)	

Node	Positoion	Case1	Case2	Case3	Case4	Case5	Case6	Case7
1051	0	-40,74	337,2	-34,47	1260	-97,46	312,8	-36,25
1054	8,79	-16,56	14060	-11,5	33930	-49,85	10650	-13,27
1055	11,72	-8,506	16170	-3,557	39860	-36,53	11400	-5,429
1056	14,65	-0,4545	16920	5,513	43030	-23,75	10700	3,165
1057	17,57	7,593	16330	17,57	43260	-12,72	8546	14
1058	20,5	15,64	14380	30,58	40900	-8,832	4933	25,48
1059	23,43	23,67	11090	45,12	35910	-11,98	-136,7	37,97
1060	26,36	31,7	6451	63,05	28330	-24,05	-6663	52,69
1062	32,22	47,69	-6875	129	6410	-108,3	-25070	102,1
1063	35,15	55,63	-15560	189,2	-7195	-210	-37510	144,8
1064	38,08	63,55	-25600	268,6	-18520	-351,2	-52860	200,3
1065	41,01	71,45	-36300	356,9	-29470	-505	-70000	261,7
1067	44,86	-81,75	-22100	34,54	-14910	-528,8	-44660	-9,023
1069	52,58	-54,19	1138	-44,05	16000	-204,2	-10010	-52,25
1071	60,3	-26,82	15020	-39,97	42190	-99,9	6254	-40,75
1072	64,15	-13,16	18450	-25,61	48840	-77,1	10580	-26,84
1074	71,87	14,13	18290	12,54	49090	-39,46	9728	7,705
1075	75,73	27,76	14700	34,67	42910	-36,99	4975	27,21
1077	83,45	54,95	501,3	110,9	17590	-96,64	-12400	87,43
1078	87,3	68,48	-10110	199,2	987,6	-214,6	-27260	150,9
1079	91,16	81,94	-23050	339,8	-15840	-420,8	-46810	249
1080	95,02	95,38	-37170	503,3	-30730	-666,1	-69300	362,3
1081	95,02	-119,6	-37170	397,7	-30970	-922,1	-69390	267
1082	98,88	-102,3	-23130	284,2	-16200	-625,2	-46540	197,5
1083	102,74	-85,04	-10250	197,6	606,3	-370,7	-26930	142,3
1085	110,45	-50,67	8481	190,2	32130	-123,8	-2449	117
1086	114,31	-33,55	14340	204,4	43010	-86,43	4611	130,1
1088	122,03	0,6539	19030	244,1	51940	-45,28	11090	172,9
1089	125,89	17,74	17860	266,6	49730	-28,81	9180	197,4
1090	129,75	34,82	14360	289,4	43230	-16,74	4689	222,3
1093	141,32	85,88	-10190	417,4	843,6	-161,1	-26700	349,4
1094	145,18	102,8	-23060	552,4	-15840	-353,4	-46300	444,7
1095	149,04	119,7	-37090	710,1	-30540	-583,9	-69100	555
1097	152,9	-124,2	-22980	164	-15570	-591,6	-46710	92,33
1098	156,76	-103,5	-10040	81,24	1200	-339	-27220	44,37
1099	160,62	-82,86	564,9	42,01	17650	-176,7	-12400	25,12
1101	168,34	-41,68	14750	50,25	42830	-40,62	4989	43,9
1102	172,2	-21,13	18330	71,52	49150	-7,649	9741	64,63
1103	176,06	-0,5829	19560	98,41	51040	22,59	11890	89,07
1104	179,92	19,95	18460	129	48740	49,37	10620	116,6
1105	183,78	40,48	15020	160,3	41980	67,28	6264	144,6
1107	191,49	81,47	1119	237,6	15720	50,58	-10060	210,3
1109	199,21	122,3	-22150	411,9	-15310	-148,1	-44770	339,9
1110	203,07	142,6	-36120	535,5	-29650	-305,5	-69770	428,8
1112	206	-142,5	-25650	31,26	-18930	-581,5	-53000	-17,82
1113	208,93	-124,5	-15610	-9,744	-7608	-410,2	-37630	-38,97
1114	211,86	-106,5	-6911	-33,34	6072	-278,8	-25180	-48,66
1115	214,8	-88,5	433,3	-37,96	17960	-191,7	-14740	-45,85
1118	223,59	-34,75	14370	2,823	40800	-67,15	4872	-1,418
1119	226,52	-16,85	16320	26,67	43260	-38,72	8497	20,17
1120	229,45	1,044	16920	51,57	42960	-11,79	10660	42,48
1121	232,38	18,93	16160	80,45	39760	8,406	11370	68
1124	241,17	72,58	5812	171,5	13690	59,81	4756	147,5
1125	244,1	90,47	337,4	213,8	1072	77	312,8	182

Node	Positoion	Case8	Case9	Case10	Case11	Case12	Case13	Case14
1051	0	977,6	-82,78	325	-44,29	555,5	-56,07	375,7
1054	8,79	28950	-42,58	11550	-20,1	19390	-27,44	14960
1055	11,72	33960	-30,88	12640	-11,99	22450	-18,41	16980
1056	14,65	36580	-19,55	12300	-3,665	23740	-9,504	17470
1057	17,57	36680	-9,636	10520	5,09	23230	-0,8082	16400
1058	20,5	34520	-4,521	7299	14	21000	6,443	13800
1059	23,43	30040	-4,11	2645	23,18	17020	12,25	9652
1060	26,36	23290	-9,68	-3447	32,97	11310	16,21	3961
1062	32,22	4069	-61,1	-21020	58,13	-5302	11,7	-12050
1063	35,15	-7790	-126,7	-33000	75,74	-16100	-2,86	-22430
1064	38,08	-19600	-218,8	-47460	96,9	-28000	-25,6	-34610
1065	41,01	-30540	-319,4	-63390	119,7	-40190	-50,93	-47750
1067	44,86	-16370	-394,9	-40610	-86,11	-24240	-196,3	-29750
1069	52,58	13680	-168	-7905	-77,71	3860	-107,9	-1557
1071	60,3	36670	-88,65	7918	-54,27	21550	-65,04	14860
1072	64,15	42430	-68,63	12080	-40,27	25950	-49,63	19050
1074	71,87	42410	-34,46	11630	-10,83	25820	-20,27	19010
1075	75,73	36810	-28,54	7270	4,293	21380	-8,8	14820
1077	83,45	14180	-59,94	-9225	40,51	3820	0,824	-1879
1078	87,3	-543,8	-134,7	-23440	68,1	-9166	-12,51	-14460
1079	91,16	-16700	-268,5	-41760	105,6	-24530	-44,02	-30070
1080	95,02	-31320	-428,5	-62530	147,4	-40400	-83,58	-47320
1081	95,02	-31480	-672,7	-62600	6,992	-40510	-258,1	-47410
1082	98,88	-17070	-468,3	-41610	-0,3246	-24660	-183,5	-30110
1083	102,74	-866	-292,3	-23240	-3,559	-9236	-117,7	-14520
1085	110,45	27120	-115,4	226	14,3	14170	-40,06	7564
1086	114,31	36770	-77,61	6959	30,61	21620	-16,84	14430
1088	122,03	44650	-34,24	12910	67,96	27640	19,7	20110
1089	125,89	42700	-17,65	11230	86,99	26160	36,87	18660
1090	129,75	36940	-7,326	7029	106,1	21710	52,12	14520
1093	141,32	-651,4	-95,58	-23040	178,2	-9069	60,97	-14300
1094	145,18	-16800	-218,8	-41380	217,4	-24450	34,94	-29860
1095	149,04	-31160	-367,5	-62330	260,9	-40260	1,015	-47120
1097	152,9	-16510	-427,7	-41690	-25,66	-24430	-172,5	-29960
1098	156,76	-355,8	-251,7	-23410	-24,24	-9067	-104,4	-14390
1099	160,62	14260	-136,1	-9225	-14,65	3882	-54,81	-1848
1101	168,34	36740	-30,88	7277	20,83	21330	5,439	14850
1102	172,2	42430	-1,685	11640	41,8	25770	28,34	19030
1103	176,06	44220	25,67	13430	63,83	27250	50,64	20470
1104	179,92	42300	50,04	12110	86,17	25780	72,61	19060
1105	183,78	36450	68,47	7926	108,6	21320	92,75	14860
1107	191,49	13380	70,21	-7946	156	3533	122,2	-1585
1109	199,21	-16790	-49,74	-40700	221,5	-24680	113,7	-29810
1110	203,07	-30740	-148,6	-63170	261,1	-40400	97,41	-47540
1112	206	-20020	-437,4	-47580	-102,2	-28450	-221,8	-34690
1113	208,93	-8234	-316,6	-33110	-94,9	-16530	-172,3	-22500
1114	211,86	3732	-222,6	-21120	-84,38	-5667	-131,1	-12110
1115	214,8	14040	-158,1	-11060	-70,37	3560	-99,06	-3318
1118	223,59	34390	-56,16	7248	-18,26	20800	-31,25	13780
1119	226,52	36630	-30,93	10480	1,047	23080	-11,38	16390
1120	229,45	36480	-6,746	12260	20,52	23600	8,189	17460
1121	232,38	33850	12,39	12620	40,44	22330	26,73	16980
1124	241,17	11690	63,65	5041	101	7927	80,46	6271
1125	244,1	852,3	80,77	325,1	123,6	518	98,39	375,9

Appendix 6 - Prewritten Excel document used for stress calculation

Extract from the prewritten Excel document used for the stress calculations. The formulas programmed into the document are based on Naviers formula. Losses such as friction and relaxation etc. are taken into consideration.

SPÄNNINGSKONTROLL I BRUKSSTADIET							Version :	0.1	
Objekt :	Referensbro stöd			Användare :	Per Boldi	Datum :	2005-11-07		
Brodel :	40	kablar		Lastfall :	**Mmax**				
Kabel area:	1200	mm^2							
INDATA :									
Balk:	Spann						M	N	
Nod :	3	122	$H =$	3,300	m	$M_{Egt} =$	1985	61,85	
$P_{SP} =$	-52170	kN	$H_{SP} =$	2,810	m	$M_{bel} =$	2801	32	
$M_{SP} =$	-13450	kNm				$\Delta M_{PK} =$	0	0	
$A_S =$	48000	mm^2	$H_{TP} =$	1,096	m	$\Delta M_{V:B} =$	24224	26	
	0,5					$\Delta M_{V:A} =$	42824	562	
							kNm	kN	
BERÄKNINGAR :									
Förluster :	Tidsberoende								
	$f_1 =$	4,65	%	$f_2 =$	15,16	%	$f_{TOT} =$	19,1	%
SPÄNNINGAR :				$\sigma_{ök}$	σ_{uk}				
Byggskedet							Oinjekterat Tvärsnitt		
Egt.	$M_{Egt} =$	1985		-0,3	0,5		$A =$	6,378	m^2
	$N_{Egt} =$	61,85		0,0	0,0		$Wö =$	-6,864	m^3
Försp.	$P_{SP} =$	-52170		-8,2	-8,2		$Wu =$	3,973	m^3
	$M_{SP} =$	-13450		2,0	-3,4				
Summa vid uppspänning :				**-6,5**	-11,1				
Förlust (t1)	$\Delta P_{SP} =$	2306		0,4	0,4				
	$\Delta M_{SP} =$	412		-0,1	0,1				
Summa t(1) :				-6,2	-10,6				
Bron tages i bruk :							Injekterat Tvärsnitt		
	$\Delta M_{bel} =$	2801		-0,4	0,7		$A =$	6,378	m^2
	$\Delta N_{bel} =$	32		0,0	0,0		$Wö =$	-6,864	m^3
	$\Delta M_{PK} =$	0		0,0	0,0		$Wu =$	3,973	m^3
	$\Delta N_{PK} =$	0		0,0	0,0				
Förlust (t2)	$\Delta P_{SP} =$	6903		1,1	1,1				
	$\Delta M_{SP} =$	848		-0,1	0,2				
Summa t(2) :				-5,6	-8,6		LK: III		

			I filerna Brinesb
H_{tp} =	Avst. från ök balk till balkens tyngdpunkt		med avrundat v
H_{SP} =	Avst. från ök balk till spännarmering		
			BLÅ TEXT (RU

ΔM_{III} = Beläggning + Överfyllnad + Jordtryck + Krympning

ΔM_{PK} = ΔM av Egt.+Bel.+Ö.F.+Krymp i LK V:A

	M	N
$M_{V:B}$ =	29010	119
$M_{V:A}$ =	47610	656

INDATA TILL FÖRLUSTBERÄKNINGEN :

Es = Esk =	195	GPa
Ec = Eck =	32	GPa

nolla Mvb för last komb 4a brott

	t(1)	t(2)	
ε_{cs}	0	0,25	o/oo
φ	0,4	1,6	
χ	2,475	3,025	%

Ändrat av Boldi & Per

σ_s(to) = 1087 Mpa

FÖRLUSTBERÄKNING :

	σcp	σsp	$\Delta\sigma s$	F	$\sigma s(t)$
t(1)	-9,94	1061,6	50,5	4,6	1036,4
t(2)	-8,15	957,7	157,2	15,2	879,2
	Mpa	Mpa	Mpa	%	Mpa

SPÄNNINGAR :				$\sigma_{ök}$	σ_{uk}				
Byggskedet							Oinjekterat Tvärsnitt		
Egt.	M_{Egt} =	1985		-0,3	0,5		A =	6,378	m^2
	N_{Egt} =	61,85		0,0	0,0		Wö =	-6,864	m^3
Försp.	P_{SP} =	-52170		-8,2	-8,2		Wu =	3,973	m^3
	M_{SP} =	-13450		2,0	-3,4				
Summa vid uppspänning :				**-6,5**	-11,1				
Förlust (t1)	ΔP_{SP} =	2306		0,4	0,4				
	ΔM_{SP} =	412		-0,1	0,1				
Summa t(1) :				-6,2	-10,6				
Bron tages i bruk :							Injekterat Tvärsnitt		
	ΔM_{bel} =	2801		-0,4	0,7		A =	6,378	m^2
	ΔN_{bel} =	32		0,0	0,0		Wö =	-6,864	m^3
	ΔM_{PK} =	0		0,0	0,0		Wu =	3,973	m^3
	ΔN_{PK} =	0		0,0	0,0				
Förlust (t2)	ΔP_{SP} =	6903		1,1	1,1				
	ΔM_{SP} =	848		-0,1	0,2				
Summa t(2) :				-5,6	-8,6		LK: III		
									Ar
	ΔN_{SP} =	-2135		-0,3	-0,3				
	ΔM_{SP} =	-3660		0,5	-0,9				
	$\Delta M_{V:A}$ =	42824		-6,2	10,8				
	$\Delta N_{V:A}$ =	562		0,1	0,1				
Summa LK V:A :		T2 :		-11,6	**1,0**		LK: V:A		
		T1 :		-12,6	-0,3				
	$\Delta M_{V:B}$ =	24224		-3,5	6,1				
	$\Delta N_{V:B}$ =	26		0,0	0,0				
Summa LK V:B :		T2 :		-9,2	-2,5		LK: V:B		
		T1 :		-10,1	-3,8				
							LK: V:A		

Appendix 7 - Centric prestressing and tendon/concrete amounts

Extract from the calculations done for the centric prestressing concerning the prestressing force. The critical bending moments and normal forces are the most important factors when calculating the amount of centric prestressing needed Calculations for the amounts of concrete and prestressing tendons are also presented.

	A	B	C	D	E	F	G
1	**Naviers formel**						
2	σ = (-Pb/Anet) + (Mb*z)/Inet						
3							
4	Inet:	8,76	m4				
5	Anet:	6,38	m2				
6	z-ök:	1,10	m				
7	z-uk:	2,20	m				
8							
9	σ-drag (ök):	1,4	Mpa =>	1,40E-06	N/m2	Tillåten spänning i ök	
10	σ-drag (uk):	1,0	Mpa =>	1,00E-06	N/m2	Tillåten spänning i uk	
11							
12	Mb-max:	32190	kNm =>	32190000	Nm	Sektion 145	
13	Mb-min:	-42670	kNm =>	-42670000	Nm	Sektion 198	
14							
15	Pb-ök	34054	kN				
16	Pb-uk	-51661	kN				
17							
18							
19	**Förlust beräkning**						
20	P(s)=P(0)*e^-μ(α+k*s)						
21							
22	e:	2,72					
23	μi:	0,19	Frictional coefficient, internal				
24	μex:	0,12	Frictional coefficient, external. Används inte i denna beräkning				
25	α:	0	Change in slope over distance				
26	ki:	0,01	Unintended change of slope per unit length, internal				
27	kex:	0	Unintended change of slope per unit length, external				
28	s:	25	Length of the duct from the active end to the section under consideration				
29							
30	Pi(s)ök:	32474	Diff:	1580	5%	Friction loss	
31	Pi(s)uk:	-49265	Diff:	-2396	5%	Friction loss	
32							
33	Ny Pb-ök:	35633		Spännkraft i Brigade:		-1239166,67	kN/m2
34	Ny Pb-uk:	-54057	kN			1239166,667	
35							
36			A=	0,0436239			

	A	B	C	D	E	F	G
33	Ny Pb-ök:	35633		Spännkraft i Brigade:		-1239166,67	kN/m2
34	Ny Pb-uk:	-54057	kN			1239166,667	
35							
36			A=	0,0436239			
37	**As(lina) 100 mm^2**						
38	Antal linor per kabel	Kabelarea [mm^2]	Tillåten spännkraft* [kN]	Antal kablar	Diameter [mm]	External duct diam [mm]	
39	4	400	496	109,0	22,6	45,0	
40	8	800	991	54,5	31,9	72,0	
41	12	1200	1487	36,4	39,1	72,0	
42	16	1600	1982	27,3	45,1	87,0	
43	20	2000	2478	21,8	50,5	92,0	
44	24	2400	2974	18,2	55,3	107,0	
45	28	2800	3469	15,6	59,7	107,0	
46	32	3200	3965	13,6	63,8	127,0	
47	36	3600	4460	12,1	67,7	137,0	
48	40	4000	4956	10,9	71,4	137,0	
49	44	4400	5452	9,9	74,9	150,0	
50	48	4800	5947	9,1	78,2	150,0	
51	52	5200	6443	8,4	81,4	150,0	
52	55	5500	6815	7,9	83,7	150,0	
53							
54	**As(lina) 150 mm^2**						
55	Antal linor per kabel	Kabelarea [mm^2]	Tillåten spännkraft* [kN]	Antal kablar	Diameter [mm]	External duct diam [mm]	
56	4	600	743	72,8	27,6	50	
57	8	1200	1487	36,4	39,1	87	
58	12	1800	2230	24,2	47,9	87	
59	16	2400	2974	18,2	55,3	107	
60	20	3000	3717	14,5	61,8	137	
61	24	3600	4460	12,1	67,7	137	
62	28	4200	5204	10,4	73,1	137	
63	32	4800	5947	9,1	78,2	150	
64	36	5400	6691	8,1	82,9	150	
65	40	6000	7434	7,3	87,4	150	
66	44	6600	8177	6,6	91,7	150	
67	48	7200	8921	6,1	95,8	150	
68	52	7800	9664	5,6	99,7	150	
69	55	8250	10222	5,3	102,5	150	
70							
71		Markerar det förslag som vi tror lämpar sig bäst för lansering					

	A	B	C	
72				
73	**Volumes**			
74				
75		Bro längd:	244	m
76	Tendons:			
77	Volume centrisk A:	0,0445*C75*1.2=	13,0296	m3
78	Volume centrisk B:	0,038524*C75=	9,3999536	m3
79	Volume centrisk C:	D36*C75=	10,64422618	m3
80	Volume centrisk org:	0,056604*C75=	13,811376	m3
81				
82	Volume parabolic:	26*1800*0,000001*244=	11,4192	m3
83	Volume polygonal A	0,6*C82=	6,85152	
84	Volume polygonal B	0,25*C82=	2,8548	m3
85	Volume polygonal org2	0,8*C82=	9,13536	
86	Volume polygonal C	0,6*C82=	6,85152	
87	Volume polygonal B 2	0,35*C82=	3,99672	
88				
89				
90	Concrete:			
91	Org:	6,4425*244=	1571,97	m3
92	A	7,321*244=	1786,324	m3
93	B	6,725*244=	1640,9	m3
94	C	6,378*244=	1556,232	m3
95				
96	Concrete minus tendons:			
97	Org:	C91-C80-C82=	1546,739424	
98	A	C92-C77-C83=	1766,44288	
99	B	C93-C78-C84=	1628,645246	
100	C	C94-C79=	1545,587774	
101	B2	C93-C78=	1631,500046	
102	org2:	C73-C80=	1558,158624	
103	Ref:	(B107*B114)+(C107*C114)-C82=	1537,0988	
104				
105				
106		supp	span	
107	A	7,069	5,816	
108	L	7	25	
109		20	34	
110		20	34	
111		20	34	
112		20	25	
113		7		
114		94	152	