CHALMERS

Conceptual Design of Prestressed Concrete Bridges Produced with the Incremental Launching Method

Analysis of prestressing arrangements

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

ERIK KARLSSON
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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 left: Side view of the Rybny Potok Bridge during launching
Middle: Front view of the Rybny Potok Bridge during launching
Bottom right: View of the casting yard and bridge deck of the Rybny Potok Bridge.

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ABSTRACT

Incremental launching is a repetitive construction method for prestressed concrete bridges. Approximately 15 to 30 m long segments of the bridge are cast, tensioned and launched forward. Next, another segment is cast against the previous one and after hardening, tensioned together with this. Thereafter the two segments are launched forward another segment length, over prebuilt piers. The launching system consists of hydraulic jacks and sliding bearings. The casting yard and launching jacks are placed behind one abutment. Incremental launching is a construction method that is used around the world since the 1960's. It has been used in Sweden four times in the 70's and 80's.

The purpose of this master's project was to find the specific characteristics of the construction method and from these draw conclusions and present recommendations for when the method is favourable to use. A profound analysis of alternative prestressing arrangements should also be conducted in this project. Since an incrementally launched bridge requires a certain need for prestressing because each cross section of the bridge is subjected to positive and negative bending moments during the launching. In service state the bridge is subjected to different load combinations, which requires a modification of the prestressing arrangement.

The investigation in this master's project was carried out through excessive literature studies, interviews with experienced bridge engineers and a site visit at an ongoing incremental launching project. The evaluation of prestressing arrangements was conducted through a comparison of various alternatives regarding the product ability, durability and economical aspects. From this evaluation four alternatives were chosen, which were subjected to a deeper study where the structural efficiency was tested through finite element analysis.

Incremental launching is an industrialized method by means of climate control, highly mechanical level and repetitive construction process. All work is conducted at the same location, the casting yard, why almost no work on high altitude is needed. The environmental aspect is also favourable since there is little risk of harming the nature under the bridge. The method makes it possible to span inaccessible areas and deep valleys where traditional scaffolding construction is prohibited. A negative aspect is that the launching limits the design of the bridge, both the vertical and horizontal radii of curvature needs to be constant, and the cross section of the bridge cannot vary along the length. Furthermore, a bridge length of at least 100 m is required to gain advantages due to repetitive work, and the method can be used on bridge lengths up to 2000 m. The profitable span range is 40 to 60 m.

The result from the investigation show that a prestressing arrangement with straight internal tendons in the flanges should be preferred during launching. Once the bridge is finally launched the bridge should be equipped with external polygonal tendons. The analysis was conducted on 45 m span, and should be valid for the medium span range of 40 to 60 m. Still, within this limited span range there are differences between shorter and longer spans, why the result in this thesis not can be applied without further investigation for the specific conditions.

Key words: Incremental launching, tacktschiebe, launching, prestressing, external prestressing, prestressed concrete, bridge, bridge construction.

Konceptuell Design av Förspända Betongbroar Producerade med Byggmetoden Etappvis Lansering
Analys av förspänningsarrangemang
Examensarbete inom civilingenjörsprogrammet Väg och vattenbyggnad

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SAMMANFATTNING

Etappvis lansering är en byggnadsmetod där 15 till 30 m långa segment gjuts, spänns samman och lanseras framåt. När ett segment är framlanserat gjuts nästa segment mot det föregående och när betongen härdat tillräckligt spänns de båda segmenten ihop med spännkablar. Härnäst lanseras de två segmenten ytterligare en segmentlängd framåt, ut över bropelare. Lanseringsystemet består av hydrauliska domkrafter och glidlager. Gjut- och lanseringsstationen upprättas bakom ett brofäste. Lansering av betongbroar är en byggnadsmetod som använts runtom i världen sedan 1960-talet. I Sverige har metoden nyttjats vid fyra tillfällen under 1970- och 1980-talen.

Syftet med examensarbetet var att finna och analysera byggnadsmetodens specifika egenskaper, samt att dra slutsatser och ge rekommendationer för när metoden är fördelaktig att använda. En analys av olika förspänningsutformningar skulle även genomförtas i projektet. Byggmetoden medför att varje tvärsnitt i bron utsätts av såväl negativa som positiva moment under lanseringsskedet, vilket skapar ett behov av centrisk förspänning. Då bron utsätts för andra lastfall i bruksskedet måste en förändring av förspänningen göras innan bron kan tas i bruk.

Utredningen i detta examensarbete genomfördes medelst litteraturstudier, intervjuer med erfarna broingenjörer samt studiebesök vid ett pågående laseringsprojekt. I utvärderingen av förspänningsarrangemang jämfördes olika alternativ med hänsyn till produktionsbarhet, beständighet samt ekonomiska aspekter. Från denna utvärdering valdes fyra alternativ ut, för vilka en djupare studie genomfördes. Denna studie syftade till att utvärdera alternativens effektivitet avseende spänningsbegränsningar och utfördes genom en finit element analys av alternativen.

Lansering av betongbroar erbjuder möjligheten till en industrialiserad tillverkningsprocess i form av klimatskydd, mekanisering och upprepning. Nästan allt arbete är lokaliserat till gjutstationen vid ett landfäste, vilket innebär att nästan inget arbete behöver utföras på hög höjd. Dessutom är produktionsmetoden fördelaktig ur miljösynpunkt eftersom det föreligger liten risk att skada naturen under bron. Metoden erbjuder många fördelar såsom möjligheten att överbrygga dalgångar och andra svårtillgängliga områden där traditionellt ställningsbyggande är omöjligt. Utformningen av en lanserad betongbro är dock begränsad då lanseringen kräver att bron har en konstant krökning både i vertikal- och horisontalled, samt att tvärsnittet på

bron är konstant längs hela bron. Vidare behövs minst en brolängd på 100 m för att uppnå de fördelar som bygger på repetitiva processer, men metoden kan användas för broar på upp till 2000 m. Spannlängder på mellan 40 och 60 m är mest ekonomiskt.

Resultatet av undersökningen påvisar att ett förspänningsalternativ med raka interna spännkablar bör användas under lanseringsskedet. Med hänsyn till brukskedes krav behöver externa polygonala spännkablar adderas när bron är färdiglanserad. Det bör dock tilläggas att strukturanalysen är genomförd med 45 m spannlängd och att utredningen generellt innefattar spannlängder kring 40 till 60 m. Även i detta begränsade spannområde finns betydande konstruktionsmässiga skillnader, vilket gör att informationen i arbetet bör ses som generell och ej direkt applicerbar utan en projektspecifik omvärdering.

Nyckelord: Etappvis lansering, Tacktschiebe, lansering, förspänning, extern förspänning, förspänd betong, bro, brobyggnad.

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Preface

This master's project focuses on the incremental launching method for concrete bridge construction and aims to investigate and develop the method. The work has been conducted at Skanska Teknik's Gothenburg office between June 2005 and November 2005, in cooperation between Skanska Teknik's Malmö office, Skanska Teknik's Gothenburg office and the Department of Civil and Environmental Engineering, Chalmers University of Technology, Sweden. The thesis completes the authors' Masters of Science Degree in Civil Engineering, specialising in Structural Engineering, at Chalmers University of Technology.

Firstly we would like to thank the initiator, as well as our supervisor, M.Sc.C.E. Karl Lundstedt, head of the bridge group at Skanska Teknik's Malmö office, who has been helpful and supportive throughout the whole project. We would also like to thank our reference group at Skanska Teknik where, besides Karl Lundstedt, M.Sc.C.E. Techn. Lic. Gunnar Holmberg and M.Sc.C.E. Jan Olofsson participates. They have all been of great help and a constant partner to discuss and ventilate both theory and practice. We would also like to thank for the opportunity to perform our master's project at Skanska Teknik's Gothenburg Office, where we have had the advantage to be in direct contact with many skilled and experienced engineers. We would especially like to thank M.Sc.C.E. Marcus Davidson and M.Sc.C.E. Rasmus Eklund for their support and expert guidance regarding the software BRIGADE. Scanscot Technology, and especially Jan Olsson, also deserves to be mentioned for their help regarding modelling problems in BRIGADE.

We would also like to thank the helpful and supportive bridge engineers at the Swedish National Road Administration and the Swedish National Rail Administration. Without them we would never have been able to collect information and drawings regarding the incrementally launched bridges in Sweden.

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We would like to send an invitation to visit Sweden, as well as our gratitude, to M.Sc.C.E. Jiří Bešta and M.Sc.C.E. Pavel Smíšek at VSL Systems (CZ) Ltd. for the very interesting site visit at the Rybny Potok project that they arranged in early October.

Finally we would like to thank our supervisor and examiner at Chalmers University of Technology Professor M.Sc.C.E. Ph.D. Björn Engström for his help, support and enthusiasm. Our colleagues Boldi Kisch and Per Langefors shall also be thanked for their help and views during the project. We would also like to thank our opponents, Audrey Farreyre and Jean Bernard Journot, for their help and views during the project.

Göteborg November 2005
Erik Karlsson & Erik Lööv

Notations

Roman Upper Case Letters

A	Area
E_{ck}	Young's modulus, concrete characteristic
E_p	Young's modulus, prestressing steel
I_t	Torsional constant
I_y	Moment of inertia, y-axis
I_{yz}	Sectorial moment
I_z	Moment of inertia, z-axis
I_γ	Warping constant
$P_{1,2,3\ldots}$	Force deriving from prestressing 1,2,3…
P_x	Force deriving from prestressing, y-direction
P_z	Force deriving from prestressing, z-direction
W_{yuf}	Sectional modulus, y-axis, upper flange
W_{ylf}	Sectional modulus, y-axis, lower flange

Roman Lower Case Letters

z_g	Distance to centre of gravity
f_{cck}	Concrete compressive strength, characteristic value
f_{ccd}	Concrete compressive strength, design value
f_{yk}	Steel yield strength, characteristic value
k	Unintended change of slope, prestressing tendon
s	Length of duct from active end to section under consideration

Greek Lower Case Letters

α	Change in slope
μ	Frictional coefficient
σ_b	Concrete tensile stress, load combination V:B
σ_{cc}	Concrete compressive stress
σ_{ct}	Concrete tensile stress
σ_{lf}	Concrete stress, lower flange
$\sigma_{p\max}$	Prestressing steel maximum stress
σ_{uf}	Concrete stress, upper flange

1 Basis of Incremental Launching

"Incremental launching", IL, is a construction technique for prestressed concrete bridges that was developed in the construction market of Germany during the rapid development of prestressed concrete bridges in the 60:s. The consulting engineers Leonhardt and Andrä played a major part in adapting the prestressing technique into a well functioning incremental launching way of construction. The first launched concrete bridge was the Caroni River Bridge at Ciudad Guyana/Venezuela in 1964. This bridge was, however, built entirely on land before the complete bridge was launched. A few years later, with the experienced gained at the Rio Caroni Bridge, the first incrementally launched bridge was built over the river Inn in Austria. Since then numerous bridges have been build and the method is now spread worldwide.

In Sweden four incrementally launched bridges have been built between 1972 and 1987 and after that the method has never been used. Today, the interest for the construction method increases again in Sweden, when subjects such as environmental aspects and industrial construction processes are of higher interest. With this in mind, the IL method is once again discussed.

1.1 Incremental launching

The general principle of the construction technique is to cast approximately 15 to 30 m long segments of the bridge in factory like conditions at the site then launch the segment forward over the piers. Next, another segment is cast against the previous and after hardening, tensioned to the previously cast segment. Thereafter the two segments are launched forward another segment length. This is done in a casting yard that is placed behind the abutment.

Figure 1.1 View over the casting yard and the bridge deck at one abutment of the Rybny Potok Bridge.

The IL method does not require any scaffolding or other temporary structures that interfere with the environment under the bridge. That is why the method offers advantages when it comes to crossing deep valleys, water paths or other obstacles such as roads and railways. Figure 1.2 shows an incremental launching construction in progress crossing deep valley in Czech Republic.

The IL method is schematically illustrated in Figure 1.3. In this figure the casting yard can be seen on the left side, and a launching nose can be seen at the front end of the structure, leading the way over the supports. How the casting, tensioning and launching are done can vary from project to project, still the key steps are described in the following sections.

Figure 1.2 The incremental launching method provides advantages when it comes to crossing obstacles, in this case a deep valley in Czech Republic.

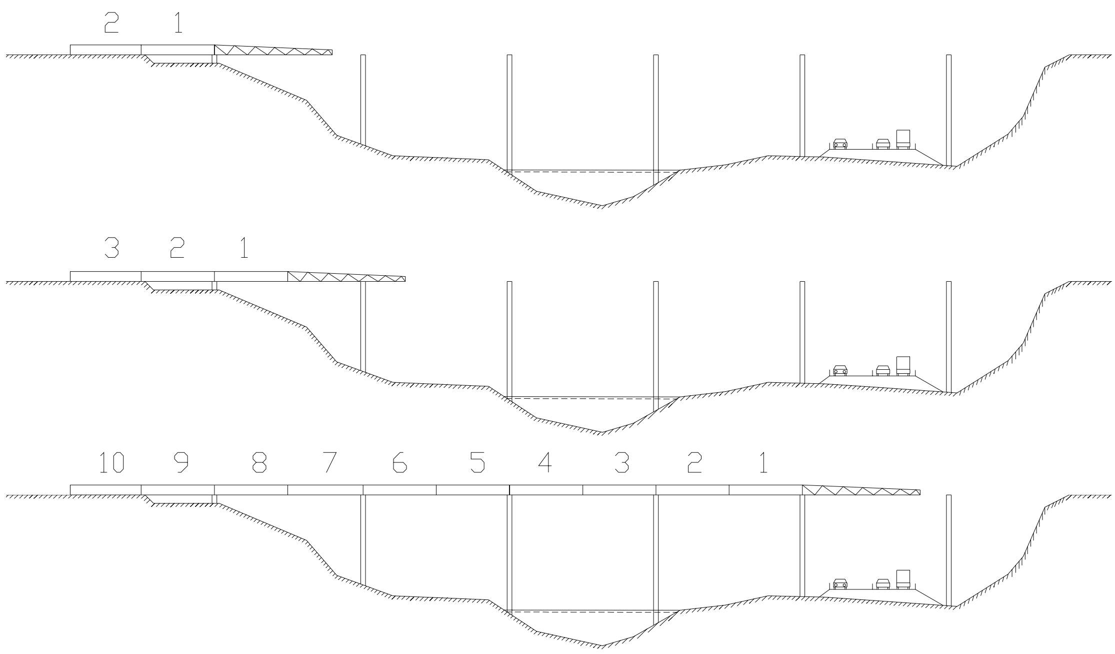


Figure 1.3 *Schematic example of the IL method. To the left the casting yard is placed where the segments are produced and launched.*

1.2 Casting of the cross section

In Figure 1.4 an example of the production of the cross section is shown. In this example the bottom slab is cast first, where after the webs are cast. In some cases this can be done simultaneously, but this depends on what way is more effective and time saving. When the bottom and sides of the box have been cast the outer formwork is moved, either manually or automatically by means of hydraulic pistons. The inner formwork is dismantled and the formwork for the top slab is mantled. In Figure 1.5 one can see that the bottom slab and the webs have been cast and the formwork is moved. The inner formwork for the top slab is being mounted inside the box (to the right in this picture).

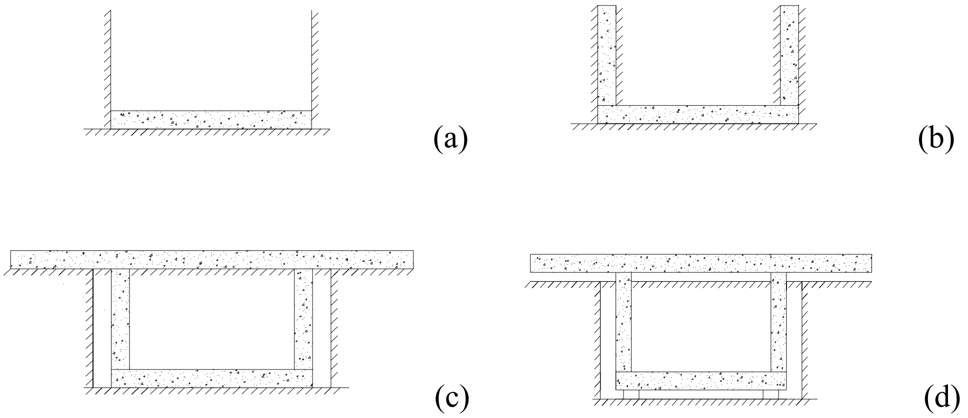


Figure 1.4 *Example of how the cross section is cast. In this example the bottom slab is cast first (a), thereafter the webs (b) and finally the top slab are cast (c) before the whole formwork is lowered to enable launching (d).*

Figure 1.5 *The bottom slab and the webs have been cast and the formwork is moved. The inner formwork for the top slab is being mounted inside the box (to the right in this picture).*

1.3 Launching moment

As the bridge slides over the supports during launching all segments will be situated both over supports and in mid span. This means that each section of the bridge will be subjected not only to the maximum positive bending moment, but also the maximum negative bending moment. When the bridge is launched this will be the case for all segments in the bridge. In Figure 1.6 a graph of all bending moments during the launching is shown.

This figure also illustrates one of the difficulties with the IL method, and that is the increased bending moments in de front zone due to the cantilever action. In Figure 1.6 this is seen to the right. In this case the bridge is launched with a launching nose that reduces the weight of the cantilever, which can be seen in the relative small bending moment incensement in the front zone. For further details see Chapter 1, Section 1.6.

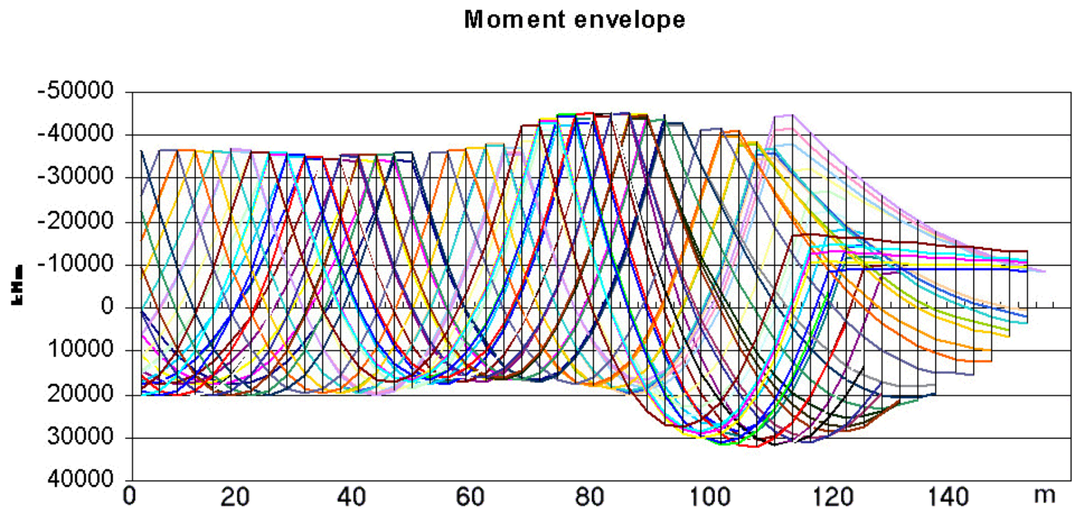


Figure 1.6 Example of moment envelope induced during launching of a bridge. Notice the increased moments in the front zone due to the cantilever action. Kisch Langefors (2005)

1.4 Prestressing

To be able to keep the segments together and also resist the bending moment envelops induced during the launching sequence, see Figure 1.6, a launching prestressing is provided from the beginning. The tendons should be stressed for each segment before the launching process starts. The location of the prestressing tendons varies depending on the prestressing arrangement. However, the arrangement used should result in a prestressing force acting in the centre of gravity of the bridge beam.

The launched bridge is often a concrete box girder. Some incrementally launched concrete bridges have been built with other type of sections, such as T-profiles, but the box girder is totally dominant, due to its relative low weight compared to its structural characteristics.

When the launching sequence is finished the bridge has to be equipped for the final state where it should be able to resist, not only its dead weight, but also all other loads such as traffic and surface loads. To be able to carry these loads in the final state additional prestressing is needed. Once the bridge is placed in its final position the final prestressing can be arranged in the zones where the highest tensile stresses due to bending moment are expected, which is at the top edge over support and bottom edge in mid span.

Both the launching prestressing and the final prestressing can be of either internal or of external type. Internal prestressing lies inside the concrete and is tensioned in concrete heels inside the box girder, or at anchors hidden in the joints between the concrete segments. External prestressing is placed outside the concrete, for example inside the box girder. Concrete deviators are used to deviate and anchorage the external prestressing tendons.

1.5 Launching

To be able to launch the rather heavy concrete bridge hydraulic jacks are needed. The launching system is generally one of following three types. One type of system is a pushing jack, which pushes the bridge from behind. These jacks are the least commonly used. Another type of launching system that is frequently used is the lift-and-push jack. This jack lifts the bridge with one hydraulic piston and slides it forward before the jack is lowered and retracted, see Figure 1.7 and Figure 1.8. Another way of launching the bridge is to pull it by the means of prestressing strands connected to the segment and pulled by hydraulic jacks, see Figure 1.9 and Figure 1.10.

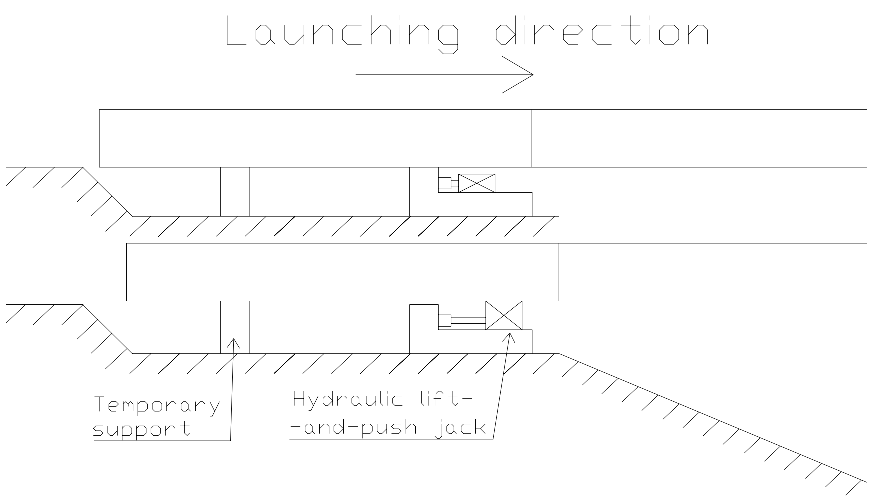


Figure 1.7 Schematic figure of the launching procedure in case of a hydraulic lift-and-push jack. In the upper figure the jack is in its start position. In the lower figure the jack has lifted the bridge and slid it forward. After that the jack is lowered and retracted, and the procedure is repeated.

Figure 1.8 *A lift-and-push jack. The vertical jack lifts the bridge and the horizontal pistons push the jack forward. Then the vertical jack is lowered and the sledge is retracted and the procedure is repeated.*

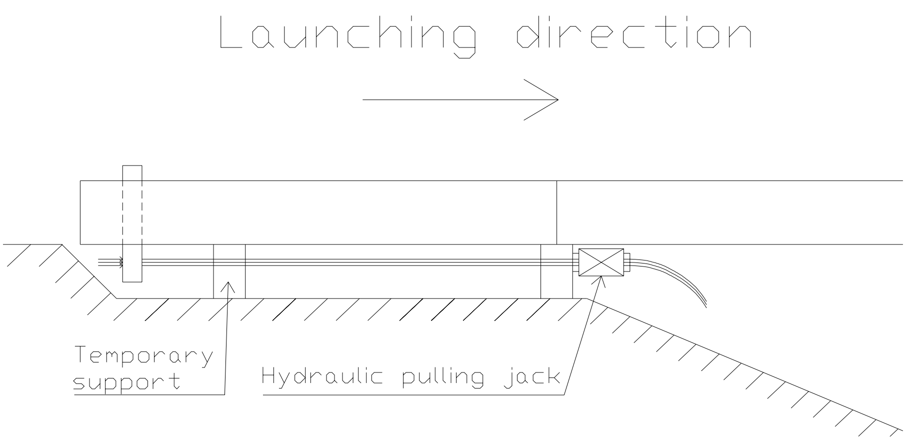


Figure 1.9 *Schematic figure of a hydraulic pulling jack launch equipment*

Figure 1.10 A pulling jack. The steel strands are connected to the bridge section 20 m to the left of the jack in this picture, and the jack pulls the strands towards it. The launching direction is to the right.

The bridge slides over the intermediate supports on low friction sliding bearings. This bearing consists of two parts, a stainless steel plate placed on top of the pier and a plate consisting of neoprene rubber and a Teflon coating on one side that slides between the bridge beam and the stainless steel plate. The Teflon coating slides on the stainless steel and the neoprene rubber sticks by friction to the concrete superstructure. When the bridge is launched forward the plates are installed manually, and it is of highest importance that the neoprene rubber side turned upwards and the Teflon coating downwards. Otherwise very high friction will be created and the pier could be damaged due to the high bending moments that will be induced. In Figure 1.11 the principle is shown, and Figure 1.12, Figure 1.13 and Figure 1.14 show real sliding bearings.

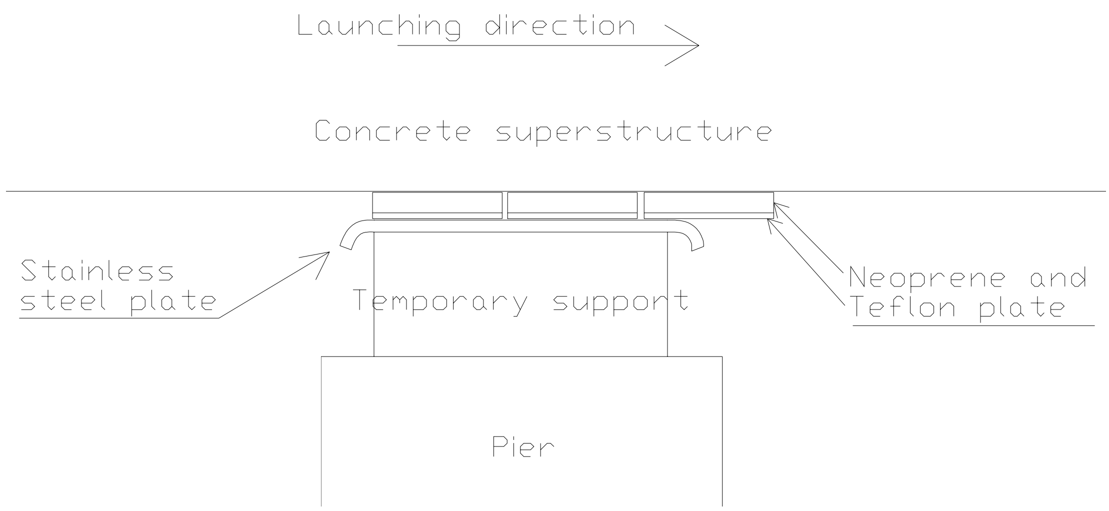


Figure 1.11 Schematic figure of a sliding bearing for launching of concrete bridges. Notice the Teflon coating on the lower part of the neoprene rubber plate, which slides with very low friction against the stainless steel.

When the launching is complete and the bridge is placed in its final position the temporary sliding bearings are replaced with permanent bearings, which can vary depending on the type of structural system. Recent trends show that sliding bearings can be used in the final state as well, but that depends on the specific structural system. Bešta Smíšek (2005).

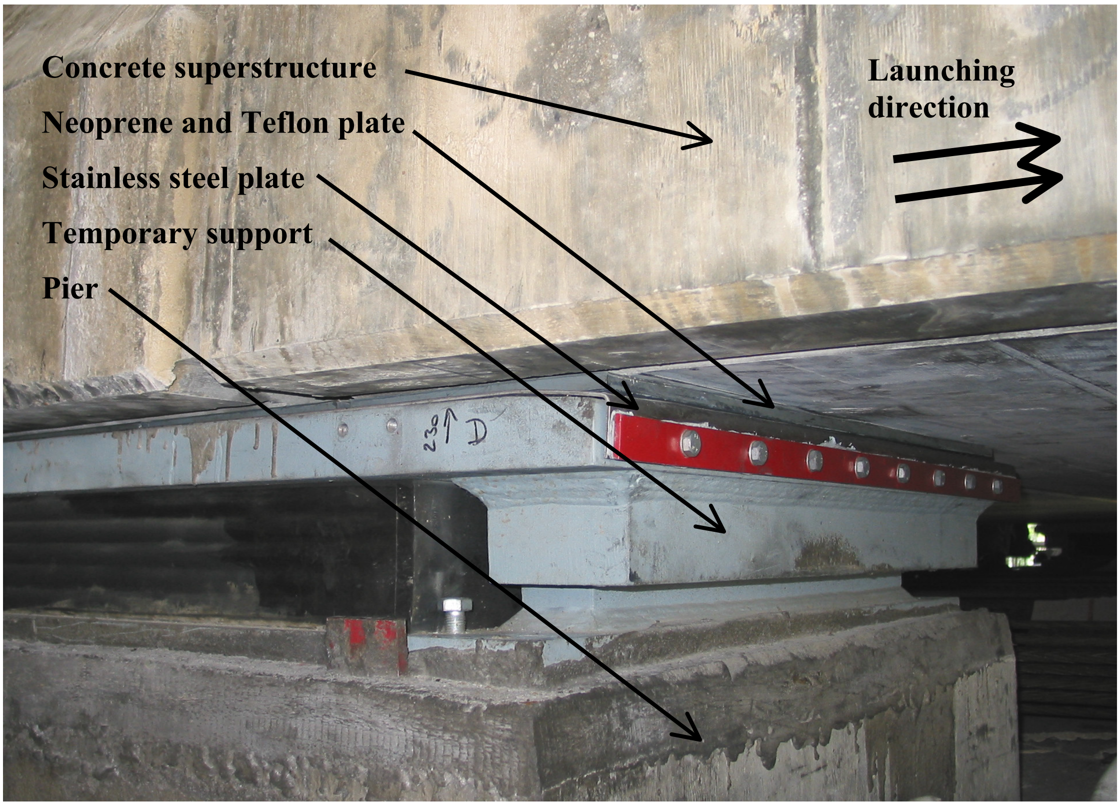


Figure 1.12 Side view of sliding bearing during launching of concrete superstructure

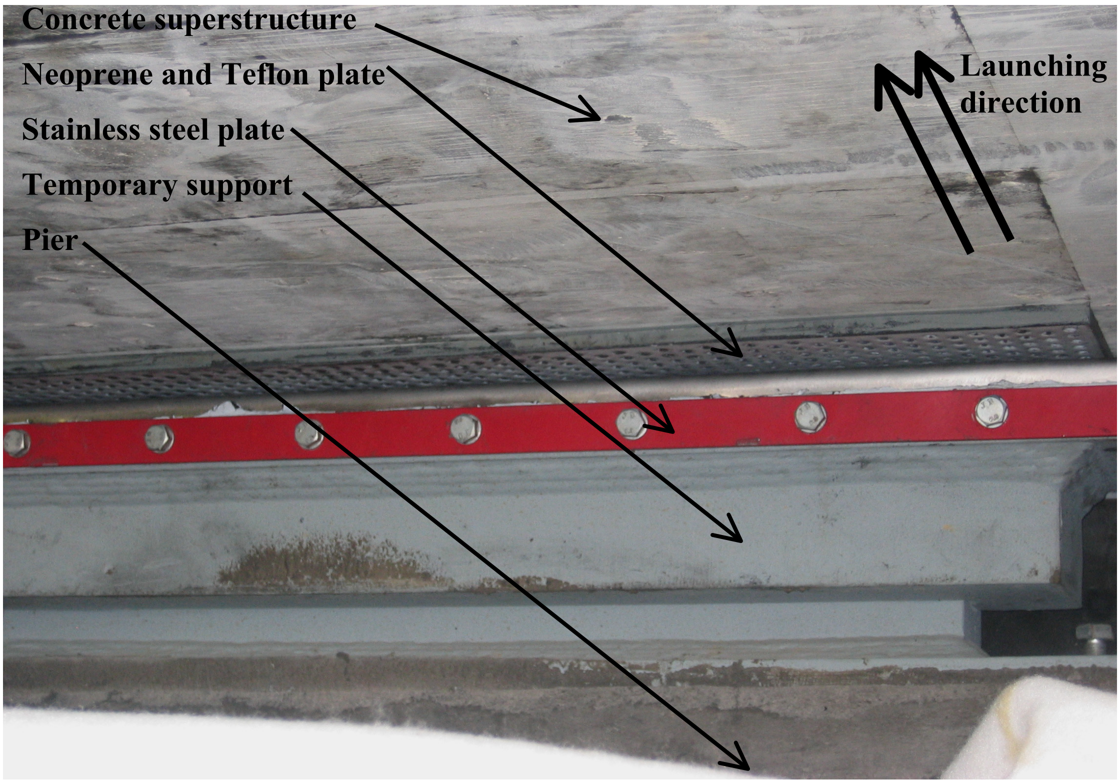


Figure 1.13 Front view of sliding bearing during launching of concrete superstructure

Figure 1.14 Preparation of a sliding bearing at the Rybny Potok Bridge.

1.6 Accessories to reduce launching moment

To reduce the bending moments obtained during the launching sequence in the front zone, there are some different accessories available. The most common item is the launching nose, which is used to reduce the cantilever weight and thereby the moment. In Figure 1.15 the launching nose for the Rybny Potok Bridge is shown. A typical launching nose should have a length of about 2/3 of the span to optimise the structural behaviour. Rosignoli (2002). To further reduce the cantilever moment auxiliary piers can be used together with the launching nose.

Instead of a launching nose a guiding mast with stayed cables can be used, though it is a solution that is more cumbersome and therefore not frequently used. Due to this guiding mast will not be further analysed. The moment envelope developed during the launching sequence can be seen in Figure 1.6 above. The characteristic shape of the front zone derives from the influence of the cantilever.

Figure 1.15 *The launching nose used for the Rybny Potok Bridge in the Czech Republic. Height of the nose is 4,2 m at the concrete connection and 1,0 m at the front end. The nose is 10 m wide The length of the nose is 36 m.*

2 Project Description

2.1 Purpose

The purpose of this master's project is to acquire knowledge and understanding of the incremental launching method for prestressed concrete bridges. Experience from the past and present should be gathered to present an objective picture of the construction method, and its positive and negative characteristics.

Development and adaptation of the incremental launching method to the trends and the codes of today was also the purpose of the thesis. This should be carried out through a comparison between old reliable prestressing arrangements and new innovative alternatives. The intention should be to find the best prestressing arrangements for general cases.

This increased understanding and knowledge of how to design and construct an incrementally launched bridge today should hopefully lead to that the method can be investigated for further use.

2.2 Goal and objectives

The main goal of this project was to increase the knowledge about the IL method in Sweden and learn how to design for the method. We hope that this Masters thesis will be a useful guide that presents positive and negative aspects regarding the IL method and also presents helpful advises. A profound investigation should be carried out regarding the prestressing arrangement, which hopefully results in some general advises concerning how to best arrange the complicated prestressing in incrementally launched bridges.

The project can be divided into four major parts and two subparts:

1. Implementation of a profound interview study.

2. Evaluation of the IL method and its features such as preconditions and characteristics. Considerations about when the method is favourable should also be presented.

3. Presentation and discussion of the different aspects and alternatives that need to be considered when making the choice of prestressing. Presentation of some possible prestressing arrangements.

    a. Analysis of the prestressing arrangements regarding some general aspects such as production aspects, corrosion protection and durability problems.

    b. Analysis of the prestressing arrangements regarding the structural behaviour. Establishing of a general model where the alternatives can be evaluated in the commercial Finite Element program BRIGADE.

4. Evaluation of a study trip and exchange of experience from incremental launching projects of today.

2.3 Scope and limitations

This thesis concentrates on the analysis and development of the construction method “incremental launching”. The method can be employed on a span range between very short spans of 20-40 m up to long spans such as 60-90 m. This analysis will however focus on the medium span range between 40 and 60 m because most incrementally launched bridges have such a span. Another reason for this choice of span is that the thesis will become more general when bridges of this span are analysed. If shorter or longer spans are to be analysed, the result from this thesis can be interpreted and adopted for that specific span. Some comments about longer spans will however be presented in the conclusions.

The analysis is limited to incremental launching and no other construction method should be evaluated. However other methods should be compared with the incremental launching method in this thesis, but not be described or analysed in detail. Such subjects are treated in another master’s project. Kisch Langefors (2005).

The FE-analyses should be conducted in the commercial bridge FE-program BRIGADE, and to save time, mainly in the calculations but also in the modelling, some general simplifications are needed. Still this analysis should be a base for a comparison and since the alternatives are quite similar the simplifications will affect the different alternatives in almost the same way. A detailed description is found in each specific chapter.

2.4 Method

To reach the aim and the goals of the master’s project, along with creating a comprehensive and logical report, a well defined method is needed. This is the subject of this section.

Firstly, the literature studies concerning bridge engineering, the IL method and prestressing theory and practices are explained. Next, the importance of exchange of experience is briefly explained, and the interview studies that have been conducted are demonstrated. The manner in which discussions with the supervisors, the reference group and other experts is accounted for next. Software used during the project is briefly described next. Finally the aim of the study trip is mentioned.

The methodology used in the FE calculations, as well as the analyse scheme, are presented in Chapter 7.

2.4.1 Literature studies

An extensive literature study to gather information regarding the subject of the thesis has been carried out. The sources of the information necessary to perform this project have been, for example, books, publications, codes, lecture publications, master's theses and scientific papers. The material has been borrowed from personal collections, Skanska Teknik's library, the Department of Civil and Environmental Engineering's library, Chalmers' library and the library of Luleå University of Technology.

The supplier of the FE program BRIGADE, Scanscot Technology AB, has provided manual and tutorial for BRIGADE. These tools were a great help in learning the basics of the program.

There are a couple of sources that have been of great importance, which deserves to be mentioned. The book "Incrementally Launched Bridges - Design and Construction", Göhler Pearson (2000), contains valuable information regarding the history and development of the IL method. "Bridge Launching", Rosignoli (2002), is a book that focuses highly on the structural aspects of bridge launching, and from which we have gathered much knowledge about prestressing in incrementally launched bridges. Publications from VSL regarding internal as well as external prestressing have also provided experience and technical solutions to this thesis. Finally, material such as papers, brochures and letters borrowed from Karl-Erik Nilsson's personal collection has been a good source for this work.

2.4.2 Interview studies

The importance of exchange of experience is easy to understand. Skilled and experienced engineers provide a high degree of knowledge, and their experience must not be underestimated. Nor should it be the only decisive factor; one must always try to think in new patterns. Still, experience from people who have designed and constructed incrementally launched bridges is an exceptional source for information to this project.

The interview study was conducted through a series of meetings with skilled and experienced engineers, which gave an opportunity to gain knowledge and experience. These interviews are also one of the greatest sources to the part of the thesis where the incrementally launched bridges built in Sweden are described.

2.4.3 Discussions

During the project discussions have been carried out with numerous experts within the field of incremental launching, prestressed concrete and general bridge engineering. As previously mentioned, our supervisor, Karl Lundstedt, has been of great help in questions concerning prestressing and bridge design. Skanska Teknik's BRIGADE experts Marcus Davidson, at the Gothenburg office, and Rasmus Eklund, at Skanska Teknik's Malmö office, have both been available in assisting in questions concerning modelling and FE-calculations in BRIGADE. Specific questions and information regarding BRIGADE have also been obtained by correspondence with BRIGADEs support team at Scanscot Technology and especially Jan Olsson.

Discussions regarding prestressing have been conducted with Karl-Erik Nilsson, prestressing specialist at Internordisk Spännarmering, and the skilled engineer Jan Olofsson, bridge division manager at Skanska Teknik's Gothenburg office.

2.4.4 Analysis tools

As mentioned earlier, this thesis is conducted at Skanska Teknik's Gothenburg office, and Skanska Teknik has provided all software needed in this study. There are many different types of software that have been used during this thesis. BRIGADE/Standard version 3.4 was mainly used to conduct the finite element calculations. The program FEM Design's help module Section was used to calculate cross sectional constants. Microsoft Word and Microsoft PowerPoint were used for the written presentation and slides. Microsoft Equation, a support program to Microsoft Word, was used to express mathematical equations. Finally, Microsoft Excel was used for stress calculations and to illustrate results graphically.

2.4.5 Study trip

To gain complementary information about recent incremental launching trends and focuses a study trip to an ongoing IL project was performed. While the interview study gave important knowledge and experience from the past, the study trip provided essential information about how the IL method is used today. The study trip also gave the opportunity to study the casting process and casting yard better, which was necessary since the literature does not cover this subject sufficient.

Another reason is of course the opportunity to get a better feeling for, and visualize, what we have been working on for several weeks.

2.5 Outline

The different methods used to carry out this master's project, such as, the way in which information is gathered, knowledge obtained and results produced, are explained in Chapter 2. Chapter 3 consists of a part concerning exchange of experience. This chapter includes an interview study conducted with experienced bridge engineers as well as a study of the incrementally launched bridges in Sweden.

A detailed description and analysis of the IL method and its features is found in Chapter 4. The beginning of the chapter includes considerations about when the method is favourable, where after a presentation of how to build an incrementally launched bridge follows. Finally, Chapter 4 contains the characteristics of a bridge built with the IL method.

The basics and theory of prestressing is described in Chapter 5, where after a presentation of the different prestressing schemes, which are to be analysed, is made. When the alternatives are presented, an analysis of the prestressing arrangements regarding some general aspects such as production aspects, corrosion protection and durability is carried out in Chapter 6.

A detailed analysis of the prestressing arrangements regarding the structural behaviour follows in Chapter 7. First the establishing of the model and the simplifications are described. Thereafter FE-calculations and results are evaluated and discussed in the following part of Chapter 7. Once the analyses are performed and the alternatives have been evaluated, regarding both general as well as structural aspects, some general conclusions are stated and presented in the end of Chapter 7.

A study trip and conclusions and impressions from this are described in Chapter 8. Finally, in Chapter 9, the thesis is concluded and future applications and improvements are discussed.

3 Exchange of Experience, Interview Study

When building bridges, as for all civil engineering projects, experience and knowledge are of highest importance. Experienced and skilled workmen and engineers make fewer mistakes and avoid simple traps.

To gather experience to this thesis an interview study was conducted in an early phase of the work. The study included a survey of the incrementally launched prestressed concrete bridges that have been constructed in Sweden through the history. Furthermore the study also included interviews with experienced engineers who have been involved in the incrementally launched bridges in Sweden and/or in many other bridge projects.

3.1 Incrementally launched bridges in Sweden

This chapter is based on the interviews that were conducted in May 2005 and described in Chapter 3, Section 3.2. The material comes from different sources and in many cases it is impossible to assign one information to one source. Therefore no references will be put out in the text, except when the material sources from other than the interviews. The rest of the information is gathered from Carlsson (2005), Lysedal (2005), Nilsson (2005) and Pettersson (2005)

3.1.1 General

In Sweden the IL method has not been frequently used. The first bridge was constructed in 1972 and since then a handful of bridges has been erected by this method. The latest incrementally launched bridge that was constructed was the railway bridge in Tomteboda in 1987. The bridges built in Sweden by the IL method are presented in Table 3.1. VSL (1977).

All the incrementally launched bridges in Sweden have been box girder bridges with a span to depth ratio of approximately 17-18.

Table 3.1 Incrementally launched bridges in Sweden

Bridge	Built year	Type of bridge	Length	Span	Height above ground	Vertical radius	Horizontal radius	Slope
Bridge crossing Dalälven in Avesta	1972	Road bridge	350 m	40/6x45/40	15,8-24,8 m	25000m	3500m	2,9 %
Bridge crossing Lule Älv in Gäddvik	1978	Highway bridge	614 m	37/12x45/37	16,0 m	60000m	7000m	1,0 %
Bridge crossing Göta Kanal in Norsholm	1987	Highway bridge	348 m	35/6x44/35	12,0-24,0 m	∞	∞	2,5 %
Bridge crossing railway yard in Tomteboda	1987	Railway bridge	391/411m (two girders)	~40 (average)	~6 m	Not available	Not available	1,7 %

3.1.2 Auxiliary equipment

The same type of launching equipment, a lift-and-push jack system, has been used for all bridges except the Norsholm Bridge. For launching the Norsholm Bridge tie rods and pulling jacks were used. Further on all bridges have been provided with a launching nose made of steel to reduce the cantilever moment. In fact, the very same launching nose was used to produce the two first launched bridges.

The supports have all been equipped with temporary sliding bearings that were replaced when the bridges were placed in their final positions and the launching was complete. The sliding bearings were of traditional type with manually inserted neoprene rubber and Teflon plates. They have all had a friction coefficient of about 3 %. No temporary supports have been used in any of the bridges.

3.1.3 Prestressing

All the Swedish IL bridges have the same prestressing arrangement, though with different amounts of prestressing tendons of course. For the launching prestress straight internal tendons have been employed to provide centric prestressing. The prestressing tendons were placed inside the both flanges, the top and the bottom slab. These tendons have been tensioned at each segment joint or overlapping at each second segment joint, before the launching of the segments was conducted.

For the final state prestressing, parabolic tendons were pushed into corrugated steel ducts, which were placed in the webs before casting of the concrete. They were all tensioned and grouted when the bridges were placed in their final positions.

3.1.4 Working cycle

The working cycle is as previously mentioned quite important for the IL method. The incrementally launched bridges in Sweden have all been cast in a casting yard at one of the abutments. The casting yards have been provided with a crane and a movable roof to cover and protect the casting station. All bridges have been cast in two steps where the bottom slab was cast first, then the bridge was launched one segment, and the webs and the upper flange were cast. Meanwhile the next bottom slab was produced.

The formworks have generally been of two different kinds, manually assembled formwork or hydraulic movable formwork. The first three bridges have all used a kind of manually assembled and reassembled formwork, which have been built up and torn down by hand at each segment. For the Tomteboda Bridge the formwork was movable with hydraulic pistons that could lower and raise the complete form sides. The internal formwork was manually installed though.

The cycle itself was kept as short as possible, and in the last two projects, for which we have information, the cycle was one week. Tensioning and launching took place each Monday when the concrete had cured enough during the weekends.

3.1.5 Known problems

One problem that many of the bridges had was the short curing time for the concrete. The cold climate in Sweden makes it hard for the concrete to reach the often-used limit, 70% of the final strength. Ehnström et al (1972). Therefore it was of highest importance to keep the climate controlled and warm enough inside the casting yard. If the concrete had not reach sufficient strength the concrete could easily crush locally during tensioning.

3.2 Interview study

The interviews were conducted in late May 2005 and the people who were interviewed were chosen on the basis of their experience within bridge engineering and especially incrementally launched bridges in Sweden. The aim of the interviews was to gather knowledge about the IL method and in that way fortify the information gathered from the literature study.

The interviews were performed as meetings with discussions rather than regular questionings. Therefore the material from the interviews is put together more as a discussion summary, where important key issues are stated in a summary of each interview. After the interviews some general conclusions from them are drawn.

In some cases the technical level of the interviews are quite high and some aspects and characteristics are not described in detail. For more information regarding these subjects please see Chapter 4, Chapter 5, Chapter 6 and Chapter 7.

3.2.1 Christer Carlsson, head of bridge department, Tyréns

3.2.1.1 Presentation

Christer Carlsson is working as the head of the bridge department at Tyréns main office in Stockholm. Carlsson is a well-known and skilled bridge engineer who designed the incrementally launched railway bridge in Tomteboda in 1985.

3.2.1.2 Interview

A discussion about the incrementally launched bridges built in Sweden was the first subject of the interview. Carlsson meant that there were two main reasons why the IL method was used, first there was some kind of obstacle to overcome, for the Norsholm Bridge there was a deep valley and for the Tomteboda Bridge a railway yard to pass. Secondly there was an experienced engineer, a “strong man”, who could persuade and convince the client that the IL method was the most appropriate construction method for the bridge. Both of these reasons were present in the bridge that Carlsson was involved in, the Tomteboda Bridge. Carl Cappé was the strong man who had been involved in all incrementally launched bridges in Sweden, and some in Norway, so far. As previously mentioned there was also a railway yard to pass, which limited the possibility to use scaffolding dramatically. Also the free height was limited so that the usage of launching girder was impossible.

Carlsson mentioned one thing that was new for the Tomteboda Bridge, which was the usage of a movable formwork. On the previous bridges the formwork had been manually assembled and then dismantled when the segment was cast. The same formwork was then reassembled once again for the next segment. However, for the Tomteboda Bridge the outer formwork was never dismantled manually, instead it was lowered and raised by the usage of hydraulic pistons. The interior formwork however was manually mounted.

Prestressing

The launching state requires a launching prestressing that must be designed for maximum positive as well as negative bending moment. This prestressing can both be permanent or dismantled when the bridge is in the final position. In the Tomteboda Bridge the prestressing was not dismantled since there was no need for this. The launching prestressing is useful, because it does create compression in the tensile edge of the beam. However, it can also be a problem in the compressive edge, since the total compression can be too high.

According to Carlsson there are two main principle ways of designing a box girder. Either the final prestressing is placed in a parabolic shape with continuous tendons that follow the bending moment curve due to dead weight. The other way is when the number of tendons varies according to the bending moment diagram with a certain overlapping. Some tendons are placed along the whole bridge while some short tendons are placed in the positions where the bending moment peaks. These short tendons are anchored at heels inside the box girder. This is a more effective way to consider the bending moment and shear forces. For schematic description of the two main principles, see Figure 3.1.

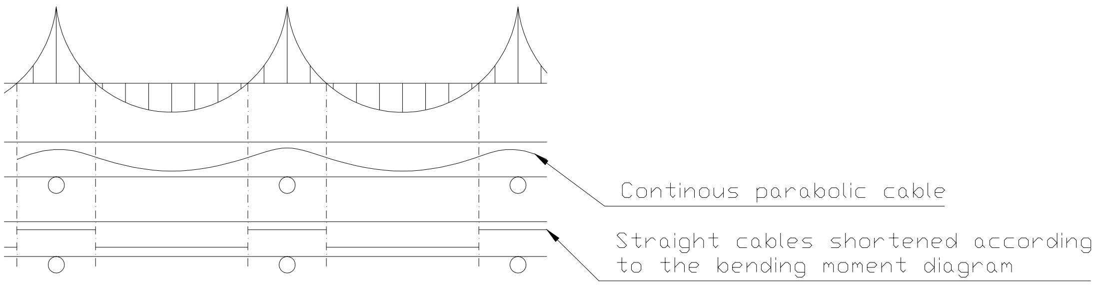


Figure 3.1 The two main principles of prestressing according to Carlsson (2005).

If only external tendons are to be used it enables the web thickness to be reduced approximately from 600-800 mm to 400 mm compared with only internal prestressing. The flange thickness however, can hardly be reduced if external prestressing is used instead of internal. That is because the overall beam behaviour, the moment of inertia and radius of gyration, depends mainly on the flange thickness. This demand makes it hardly worth the effort to use external prestressing instead of internal in the flanges. Only if there would be any advantages in removing some launching prestressing it would be favourable to use external tendons for the flange.

Carlsson does not believe in detensioning tendons only to save money. The procedure is rather time consuming and besides it can be quite hard to reuse the tendons. Only if there is a need to detension due to too high compressive stresses it might be economical to do it. However, in general stiff box girders seldom need to be detensioned.

Auxiliary equipment

Regarding the launching nose Carlsson states that it should be light and stiff enough to keep the moment peak in the front zone of the bridge to acceptable small levels during launching. When the cantilever is long, the nose needs to land on the support before it has deflected too much. It must thereafter lift the bridge up on the support, and a stiff nose is then more of help fore the bridge, since it relieves the cantilever moment more than a weaker nose.

If you want to study the nose behaviour you should study the phase when the nose nearly reaches the support. If no nose were used, the moment would be the unit weight times $L^2/2$ instead of $L^2/12$ in a service state middle support. Therefore it is important to use a stiff but light nose. Then the cantilever moment at the last support is only approximately 30 % higher than the middle support moment, and has a shape of a swan head and neck. The fact that it is only 30 % is due to the smaller weight of the steel nose instead of the concrete.

Sliding bearings are of highest interest since they can provide a smooth and low friction launching. On all bridges neoprene and Teflon plates have been used. These plates are coated with Teflon on one side to slide well against a stainless steel plate that is grouted to the support top. The other side of the plate is neoprene rubber, which create high friction between the concrete superstructure and the sliding plate. These plates have to be manually inserted at each support, and it is of highest importance that they are inserted the right way. If the plates would be inserted in the wrong direction, or even be slightly irregular, the superstructure would fasten to the support, which would most likely crack due to the horizontal force that is created.

The jacking equipment in the Tomteboda Bridge was of the traditional lift-and-push jack system. On the Norsholm Bridge tie-rods were used that were pulled by hydraulic pulling jacks. This solution requires more work with the foundation of the construction yard since all the compressive forces that are needed to push the bridge needs to be transferred from the yard to the ground.

IL in general

According to Carlsson the most important thing when building an IL bridge is the repetitive work. In a working shed on the construction yard bridge segments can be produced in a quite short cycle since the concrete can mature for only three days before the segment is tensioned and launched. As the construction precedes the workmen get more skilled since they are doing the same thing over and over again. Of highest importance is though the climate so that the concrete is cured enough during these three days.

When discussing the IL method one has to ask oneself about what other bridge types that are competing with the IL method. Of course a traditional bridge built on scaffoldings needs to be considered. Thereafter the competition from a launched steel bridge has to be evaluated. Further on, there are many ways of building concrete bridges without scaffoldings; launching girder, launching gantry, match casting or prefabrication of whole segments. In Sweden prefabrication has not been so accepted because of old traditions as well as foreseen problem with icing in the joints.

A positive aspect of incremental launching is the possibility to not disturb the nature and environment. The IL method provides a smooth and non-visible construction since all you can see is the bridge that softly is launched out over the supports. The risk for harming the environment underneath is small. And of course the fact that IL provides an industrialised way of construction where the workmen get more and more skilled for each segment is of positive value.

3.2.1.3 Summary

- The cross-section cannot be dramatically changed if external prestressing is used, only the thickness of the web.

- IL is a good way of constructing bridges when it comes to the environment, but many other methods are competitive.

- According to Carlsson the IL method has economically disadvantages compared to other ways of bridge construction.

- Relieving of launching prestress is only good when the compressive stresses must be reduced.

- Carlsson does not believe that designing an incrementally launched bridge today should be much different from the way the Tomteboda Bridge was designed.

3.2.2 Bengt Lysedal, project manager, ELU Konsult

3.2.2.1 Presentation

Bengt Lysedal is an experienced engineer who is working at ELU Konsult as a project manager. Lysedal is the engineer who designed the bridge crossing Lule Älv in Gäddvik in 1976.

3.2.2.2 Interview

First of all a general discussion about the IL method was held. Lysedal is quite certain that he never heard about any cross section other than box girders that have been launched when discussing concrete bridges. Lysedal has designed the bridge crossing Lule Älv in Gäddvik in 1976 and a presentation of this bridge was made. Lysedal tells that his office, ELU Konsult, designed the bridge in Avesta in 1971 and that he used those calculations as a base for his calculations. In fact the two bridges are quite similar, the same launching nose was even used. The front end of the bridge had to be a little modified to fit the nose. For example a concrete heel was cast on the top flange to connect the nose, which was a little too high for the box girder beam.

Further, the bending moment diagram for incrementally launched bridges was discussed. The diagram has a shape of a snake where the head is the extra moment due to the cantilever. The head of the diagram is usually subject to a special investigation where the bridge has to be provided with extra prestressing compared to the rest of the bridge.

Prestressing

The launching prestressing, which was used in the construction state, was kept in the final state as well. Final prestressing in a parabolic shape was pushed into ducts when the bridge was in its final position. The ducts were made of corrugated steel and were placed inside the bridge webs before casting.

Lysedal does not think that the use of external prestressing would provide any difficulties other than that the structure has to be provided with deviators to anchor the prestressing tendons. However Lysedal does not think that there will be any reason to detension any external tendons. There will be no economy in such an operation.

Auxiliary equipment

For jacking equipment on the Gäddvik Bridge a traditional lift-and-push jack system was used. No temporary supports were ever considered, since the span was quite short.

On the piers temporary sliding bearings were used of the traditional type. These were replaced when the bridge was fully launched.

IL in general

Why the IL method is not frequently used today is due to the comprehensive competition from other construction methods according to Lysedal. Economy is increasingly important today, and Lysedal think that the key to make IL profitable today is to reuse the equipment and the knowledge in many projects. Another important aspect that can make IL interesting is when the bridge must span over protected environments. Then the IL method provides a way of constructing the bridge without harming or disturbing the environment.

Problems with building incrementally launched bridges in Sweden can be the climate. Since the bridge must cure during such a short time, it is of highest importance to have a controlled climate. This can be done inside a shed, but still one must be careful not to tension the prestressing tendons to early. This was the case in one segment of the Gäddvik Bridge, which resulted in cumbersome repair work before the bridge could be launched further.

3.2.2.3 Summary

- IL is a good production method when concerning the environment and surroundings.

- It is hard to have good economy in incrementally launched bridges, other than if many projects provide reuse of equipment and knowledge.

- It is important to have good control over the hardening of the concrete so that sufficient strength is reached before tensioning.

3.2.3 Karl-Erik Nilsson, project engineer, Internordisk Spännarmering

3.2.3.1 Presentation

Karl-Erik Nilsson is an expert in prestressing who has been involved in numerous bridge and offshore projects. Karl-Erik Nilsson is a project engineer at Internordisk

Spännarmering, which is a company that has provided prestressing and/or launching equipment to some of the IL bridges built in Sweden.

3.2.3.2 Interview

First a general discussion about the incrementally launched bridges built in Sweden was held. Nilsson informed that Internordisk Spännarmering have been involved in the bridges in Gäddvik and in Tomteboda. Nilsson visited the Gäddvik Bridge several times during construction, and he had good contact with the persons who actually conducted the design.

One advantage of the IL method is that the bridge can easily pass obstacles such as railway yards and/or deep valleys. No scaffolding is needed and therefore savings in both foundations and work can be made. IL also provides a way of construction that does not interfere with the nature and environment.

Nilsson believes that economy is the answer to why the IL method is not used in Sweden today. There are many competitive methods that need to be evaluated before any method can be chosen, and Nilsson thinks that it often can be found that a launched steel bridge with concrete deck is more economical. Other methods such as launching girder, launching gantry and match casting can also be found economical instead of the IL method. This is because that an incrementally launched bridge requires rather much concrete to handle the high compressive stresses that the launching prestressing creates. Also the fact that a part of the prestressing steel is not used in the most effective way increase the cost. According to Nilsson this is what makes an IL bridge more expensive that other bridges.

Prestressing

External prestressing was discussed and Nilsson talked about a project in England where Skanska used only external prestressing with success. However, the Swedish Road and Rail administrations do not permit this, not now anyway. Nilsson believes that exceptions from this rule can be made if a proposal and a good tender can be provided.

For external prestressing strands are placed in plastic ducts, which are filled with grease. This creates a sufficient corrosion protection according to Nilsson. If durability is of extra concern the plastic ducts can be placed inside other plastic pipes, which finally are grouted. Then grease, plastic cover, grout and another layer of plastic protect a single tendon. This is a more expensive solution and according to Nilsson the corrosion protection of the simpler one should be good enough in most of the cases. One could argue that this creates a better corrosion protection than just the ordinary grouted steel ducts inside a concrete section. A cracked section with internal prestressing can result in corrosion attack on the strands, while an external prestressing tendon always have the same protection whether the concrete is cracked or not.

Also the fact that the web in the concrete section can be reduced when using external prestressing is an advantage. The reduction of dead weight and savings in concrete mass could reduce the costs.

External tendons also have some disadvantages. First of all the eccentricity of the tendon is limited due to that they have to be placed inside the box and not inside the concrete section. This means that the prestressing steel area must be increased to create the same moment. Further on, the use of external tendons will result in wider cracks. Internal tendons are grouted which results in almost full interaction between the concrete and the steel. When cracking occurs, the crack opening will result in a higher stress in the tendon. These stresses will hold the crack together. With external tendons no interaction can be counted for. The deformation is smeared out between the anchors of the tendon with only a small stress increase as a result.

Tensioning external tendons can sometimes be quite hard to perform. The height in a box girder is limited, often the internal space is not much more than two or maybe two and a half meter. The jacks for tensioning tendons can weight 300 to 500 kg (12 strand respectively 19 strand tendons), and to handle such a jack require some sort of lifting equipment. Such work takes time and requires skilled personnel.

Regarding the size of the tendons Nilsson states that the Swedish Road Administration previously allowed only 12 strands in one tendon. Nowadays 19 or even 24 strands can be used. Though a special permission from the Swedish Road administration is needed when using more than 12 strands. This enables that the concrete sections can be smaller since two 12 strand tendons requires more space than a single 24 strand tendon. According to Nilsson tendons with more strands is more economical not only by the possibility to have smaller concrete sections. Fewer tendons means less tensioning points and also less work with placing the tendons in the casting formwork. But still, too many strands in one tendon can cause too high stresses in the concrete under the anchor devices. To decrease the stresses, larger anchor devices must be used which take more room and therefore can be problematic to use.

Concrete deviators for tensioning

To create deviators in the concrete quite a lot of reinforcement is needed to transfer the prestressing force into the concrete structure. Generally ordinary reinforcement is enough to handle these forces, but in some cases there can be a need to tension the deviators to the structure with tie rods or prestressing tendons.

It is of great importance to avoid curving the prestressing tendon with small radius since this will result in large friction losses. Therefore it is important to make deviators and saddles smooth and round.

Detensioning of tendons

Nilsson means that it is technically possible to detension temporary tendons when the bridge is finally launched, but he also points out that in most cases there is no economy in reusing detensioned tendons. It is costly to detension tendons due to that it takes time since one has to be careful not to detension too much at the same time. The ends of the tendon are damaged once they are tensioned. Therefore the reusable tendon length is reduced compared to the initial length. If they are to be reused they have to be collected and checked before they can be tensioned again. One could say that the only time detensioning could be economical is when it is necessary to reduce the compressive stresses in the structure in the final state. If then the detensioned tendon should be reused in an economical way, depends on the specific situation and how the price of the tendons develop.

3.2.3.3 Summary

- The IL method is not frequently used because the bridges tend to have too much concrete and therefore become too expensive.
- External prestressing can provide a better corrosion protection.
- With external prestressing the eccentricity is less than with internal prestressing.
- When tendons should be tensioned inside the box there can be problem with limited space for accommodating the prestressing jack. It is also hard to transport and lift the jacks inside the box.
- Once cracking occurs, internal prestressing provides a better crack control.
- Detensioning tendons is usually only economical when it is needed to reduce concrete compressive stresses in the final state.

3.2.4 Lars Pettersson, bridge engineering specialist, Skanska Sverige AB

3.2.4.1 Presentation

Lars Pettersson is a former bridge designer who currently is working as an bridge engineering specialist at Skanska Sverige AB. Pettersson is involved in all larger bridge and tunnelling projects that Skanska is contracting and he has a great knowledge about engineering and construction work.

3.2.4.2 Interview

First the IL method in general was discussed, and Pettersson meant that a bridge already in the early phase has to be adapted for the IL method if it shall be suitable to use this method for production. A standard concrete bridge cannot be erected with the IL method unless it is designed for it from the beginning. That is the case for many production methods, but according to Pettersson, especially important when using the IL method, since the vertical and horizontal alignment has to have a constant curvature.

Regarding the economy Pettersson believes that it is enough with one project to make the method profitable. No repetitive projects are needed to gain economical advantages. Before choosing the IL method, all other methods have to be evaluated to determine that the IL method is the most suitable. First of all the height and the obstacles that are to be passed have to be evaluated so that a traditional concrete bridge built on scaffoldings can be disregarded. Then a comparison between a launched steel bridge and an incrementally launched bridge has to be done. Launching a steel bridge requires fewer people than when producing an IL bridge. Besides this, launching a steel bridge can be as fast as building an incrementally launched bridge. In the end it can come down to material costs and material savings according to Pettersson.

Advantages

Some of the advantages that the IL method provide are that the IL bridges hardly have any detoriation problem, and that one can build bridges that crosses sensitive environment without making any harmful influence. The good durability behaviour comes from that the bridge is subjected to compression from the launching prestressing, and therefore less cracking occurs compared to traditionally prestressed concrete bridges.

Disadvantages

One of the main disadvantages with the IL method is that the IL bridges are of box girder type. A box girder of concrete is structurally a very good solution, but since it is a very stiff structure a small settlement in any support will increase the bending moment in the nearby support with a factor two. And since the bridge is quite heavy, support settlements can be hard to avoid if not a very rigid foundation is made. This is of course quite costly.

Prestressing

External prestressing is not a good solution. The very high stresses that the tendons must transfer to the structure will cause a need for very strong concrete deviators that requires much work and reinforcement. Therefore there are no benefits in using external prestressing, the only result will be more labour time and material costs.

3.2.4.3 Summary

- To produce an economical bridge, the bridge has to be designed for the method. Especially when regarding the IL method since the bridge alignment is limited.
- One project can result in good economy, no repetitive effects are needed.
- Low detoriation problems since the bridge is subjected to compression from the launching prestress.
- Support settlements increase the moments.

- External prestressing is not a good solution when concerning production aspects.

3.2.5 Conclusions from the interviews

The interview study was worthwhile and a lot of valuable experience and knowledge were collected and compiled. Although one should be aware of that most of the practical experiences of IL in Sweden is rather old and limited to some few examples. Some of the interviewed people expressed that it was hard to recall all specific details, but the key steps is correct Further the development of the IL method outside Sweden is excluded since no experience of this could be found.

To state some general conclusions that are pointed out in all interviews is rather difficult. But one thing that all agree on is that IL not has been used in Sweden because of the insensitive competition with other construction methods. Competition regarding the economical and characteristic advantages and disadvantages. They also agree on that there is potential in incremental launching as a construction method, but the optimal conditions have not existed in Sweden for a success. Our opinion is that it is possible to use IL with an economical profit, since the method is frequently used outside Sweden. But one problem in Sweden is that there is a lack of recent experience.

One more thing all agreed on is the doubt in rearranging prestressing tendons as well as their full convenience that it is not economical beneficial to reuse prestressing tendons. The indication was that IL should be as rational and straightforward as possible to be profitable. However, you can differentiate the discussion conducted by Karl-Erik Nilsson compared to Lars Pettersson regarding the use of complicated external prestressing arrangements. Nilsson states that external prestressing is rather easy and straightforward, while Pettersson means that external prestressing is a difficult and time-consuming solution. We suppose that it is their different jobs and previous experiences that base their opinions. It is hard to say whose opinion is closest to the truth, but one can believe that it is somewhere between them, and that this is a major conflict in recent design of incrementally launched bridges.

One advantage stated by both Carlsson and Lysedal is the environmental benefits when using the IL method compared with for example the usage of scaffolding. Also their point of view how the economical issue could be treated is similar. Both of them propose to make use of reusable equipment that deprecates in several projects. Pettersson on the other hand means that it should be possible and necessary to include the equipment in one single project. One can argue if it is risky or even possible to deprecate the equipment on several projects depending on how sure you can be on the inflow of projects. But it would of course be more profitable to depreciate the equipment over several projects.

4 Application and Characteristics of the Incremental Launching Method

The process of the incremental launching method is, as mentioned earlier, a cyclic procedure where 15 to 30 m long segments are cast and launched at one or both abutments. The segments are cast against the previous segment and tensioned together with prestressing before launching.

Figure 4.1 Launching of the Woronora Bridge in Australia. Radius of curvature was 460 m, launching downwards with 6,0% slope and the superstructure had a total weight of 20 000 tons. Typical span length 58,7 m. (Courtesy VSL)

The IL process and its characteristics will be the subject of this chapter. Typical features and applications of the technique will also be presented. In Figure 4.1 one amazing project from Australia is shown where the favourable characteristics of the IL method were fully utilized.

4.1 Performance

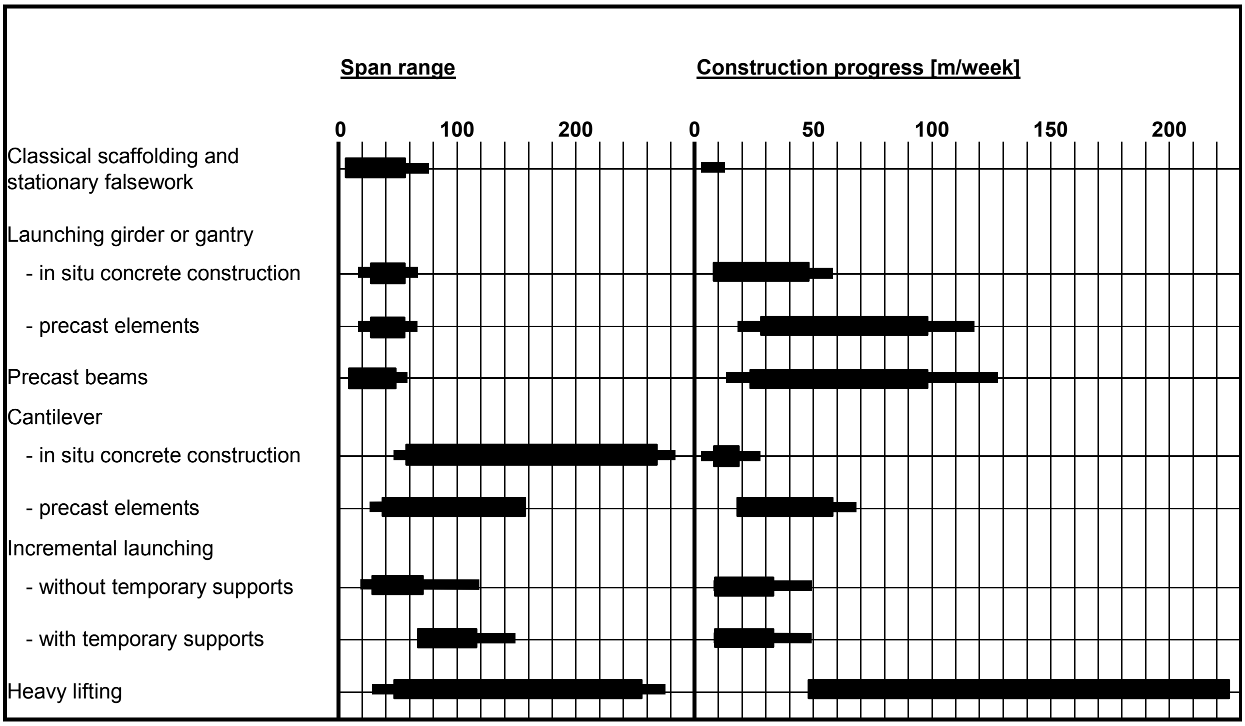
It is outside the scope of this project to carry out any deeper comparison with other construction methods. However it is impossible not taking it into account when evaluating the range of applications and the different characteristics of the IL method.

4.1.1 Span

The optimal span range for the IL method is according to Göhler Pearson (2000) about 45 to 50 m, but it could even be economic with span up to 65 m when the height of the piers increases. The span in the final state is not limited due to the method itself because arranging auxiliary piers can be used during construction. This is not frequently used due to economical reasons. It is quite expensive to construct these temporary structures. The spans should be of almost the same length otherwise additional measures, such as auxiliary piers, have to be used.

An optimal span to depth ratio during the launching state is somewhere around 15:1. When using temporary auxiliary piers the final span to depth relation will become 30:1, which according to Göhler Pearson (2000) is the economic limit for a continuous girder with constant depth. In fib (2000) the span lengths for different construction methods are stated. The comparison can be seen in Table 4.1.

Table 4.1 Typical span ranges and construction progress for different concrete bridge construction methods. (Source fib (2000))



4.1.2 Construction progress

When building a bridge today the time is always a decisive parameter. How fast can the bridge be erected and is it possible to increase the speed if unforeseen things happen? In Table 4.1 where the possible span ranges are compared also a general overview of the erection speed of the most used construction methods is presented. When studying the table closer, it is obvious that the construction methods using precast concrete elements are relative fast compared to methods where the concrete is cast in-situ, see Table 4.1. According to this the IL method is, and should be, a rather slow method, since the concrete is cast in the casting yard at the abutment.

Noticeable is also that, according to Table 4.1, the IL method is a bit slower than the travelling gantry cast in-situ method. To speed up the IL method one possibility is to launch the bridge from both the abutments. However launching from both abutments could make the IL method too costly compared to other methods. IL is far faster than construction on scaffoldings though.

One should be aware of that it is rather hard to speed up an ongoing IL process if something unforeseen happen. This is due to that the seven-day cycle is optimised for a specific project and the concrete curing time often is decisive. The only possibility to increase the effectiveness is to further industrialise the work in the casting yard, but this is as mentioned hard to do during an ongoing project. One example is to use movable formwork. According to Carlsson (2005) this was used for the Tomteboda Bridge.

4.1.3 Length of the bridge

The bridge length has a direct influence on the selection of construction method. When building with the IL method, a general minimum length of approximately 100 m is needed. For shorter bridges scaffolding or for example precast T-beams are competitive. Göhler Pearson (2000). Yet IL can, in some cases, be of interest also for shorter bridges when some special qualities are required, for example when building over a railway area or other inaccessible areas.

The IL method is as mentioned earlier an industrialised process and it has many repetitive steps. This is an advantage especially when the number of segments and span increases. Although Göhler Pearson (2000) claims that when the bridge length approaches 1000 m the travelling gantry construction method has more favourable effects. An incrementally launched bridge section cannot be optimised or haunched but for the travelling gantry method this is possible, why the IL method generally uses more material. The travelling gantry method is nevertheless more expensive regarding the equipment, but at approximately 1000 m break-even is reached. However they also declare that, if the bridge is launched from both abutments a total length of 2000 m could be an economical proposition. Göhler Pearson (2000).

4.2 Advantages

The advantages with the IL method are several. The industrialized process, the possibility to span over inaccessible areas and a good working environment are all, among others, factors that are positive with this construction method.

4.2.1 No scaffoldings

Bridge building today requires passing both over and under obstacles, such as roads, existing bridges and railway yards where the free height often can be limited, and spanning over deep valleys and waters. In some of these cases traditional construction on scaffolding can be hard to accomplish, either due to economical or physical reasons. If, for example, a bridge has to span over a very deep valley or over rapid flowing water it can be very expensive, if possible, to construct scaffolding to hold the formwork for the bridge.

Another situation where it is hard to use scaffolding is when areas with clay or unstable soil are to be passed over. Then the scaffolding itself requires a stabile foundation, which of course also is quite costly.

IL is a construction method that offers a solution for this problem. Since the bridge is launched over the entire span, no scaffolding is needed and the bridge can be built passing many different and challenging obstacles without even being affected of them. In Figure 4.2 a good example of this advantage with the incremental launching method is shown. The bridge is spanning over a 40 m deep valley, which would have make construction on scaffolding quite costly.

Figure 4.2 One of the main advantages with the IL method is that a bridge can be erected where no scaffolding is possible or economical to build. In this picture the Rybny Potok Bridge is crossing a 40 m deep valley.

4.2.2 Working environment

There is however some other construction methods that offers the contractor the possibility to build a bridge without the usage of scaffolding. The travelling gantry method or launched formwork method both provide the same possibility as IL. When constructing a bridge with a travelling gantry or launched formwork the workmen have to work at a platform hanging high over the ground. When using the IL method the labour works at one abutment and the risks with construction works on high levels are decreased. In Figure 4.3 a travelling gantry is shown.

Further advantages with the working environment are the fact that the method implements a working cycle that induces a repetitive process. The workmen do the same tasks over and over again at the same place with the same tools. One could of course argue that the repetitive work results in negligence and slackness that can result in accidents. This is however a question that concerns all works and, nevertheless, the fact that one avoids works on high level is preferable.

Figure 4.3 Construction of a concrete bridge with a travelling gantry. In this specific case the height of the bridge is quite low, why the risk of working at the gantry is limited. However, when constructing with the IL method almost no high level work is needed. (Courtesy Skanska SA Poland (2002))

4.2.3 Environment

Another positive aspect with launching the complete bridge, and not building on scaffolding or with a travelling gantry or launched formwork, is that the IL process does not disturb the environment. The risk of dropping tools or material from a working platform is avoided and the risk of harming the environment is decreased, since the work is performed at one station on land only.

An additional environmental aspect is the esthetical. When constructing an incrementally launched bridge the only thing that is visible is a slowly flowing bridge, which carefully and with minimal environmental disturbance slides over a number of previously constructed piers.

4.2.4 Industrialized process

The working process is, as mentioned earlier, repetitive and cyclic in a rather short cycle. This enables the workmen to learn and master their tasks after only a couple of cycles, a couple of weeks, and due to this, increase the quality of the work rapidly. The cycle is often called the “seven days cycle” and briefly described in Table 4.2. Of course the schedule is not valid in all IL projects since the process depends on the length of the segment and the weather conditions.

Table 4.2 One example of the seven days working cycle

Monday	Testing the concrete strength Applying the launching prestress Lowering/dismantling the formwork Launching one segment length Cleansing and adjustment the formwork Reinforcing of bottom slab Grouting the tensioned tendons
Tuesday	Installing ducts in bottom slab Reinforcing the webs Installing ducts in webs
Wednesday	Finishing formwork and casting bottom slab (and webs)
Thursday	Mantling formwork for top slab Reinforcing the top slab
Friday	Reinforcing the top slab Installing the ducts in top slab Finishing formwork and casting top slab (and web)
Saturday	Curing of concrete
Sunday	Curing of concrete

The “seven days cycle” combined with a fixed working environment, at the casting yard, provide for an effective industrialized process under factory-like (covered and heated) conditions. Under such conditions the risk that unforeseen problems occur can be decreased. It is also possible to further industrialize the process to improve both the quality and the efficiency of the process. This is done by using rigid, mechanically handled formwork, as well as to establish efficient systems for placing reinforcement, concrete and prestressing.

4.2.5 Logistic

When building a bridge the terrain is often hard to overcome and it is expensive and time consuming to have good accessibility out to the construction site. IL is in this aspect extremely favourable as all the construction work is kept at one place, except from the production of the substructure. To have almost the whole production at one place provides for less logistic problems and could for example avoid the need of temporary roads, or large cranes as well. With excellent logistic also favourable effects are received for the concreting, since it never has to be pumped long to be cast. Also that it is easy to estimate the time it takes for the trucks to deliver the concrete to the construction site is favourable regarding to the concrete quality.

Furthermore, the fact that only a minimal amount of additional land area is required to accommodate the concreting facilities because it is usually located in the area required for the approach embankment.

4.2.6 Good quality and control

IL is a construction method that is easy to measure and oversee. A better control is provided since the IL method is a more industrialised process than other bridge construction methods. Many repetitive steps can easily be standardised and systematic controlled. This is of course a main advantage for the IL method since quality and quality control is of highest importance.

When casting concrete it is very positive to be able to control the climate. The covered and heated casting area provides for a concrete cured under controlled conditions, which results in a higher concrete quality.

One additional advantage with the IL method compared with other methods is the usage of additional prestressing tendons. These extra tendons provide an improvement of the quality and by this the durability of the bridge according to Göhler Pearson (2000). The additional prestressing tendons increase the robustness of the bridge with regard to unforeseen but possible loadings during the service life. According to Göhler Pearson (2000) also the quality of the joint between the segments is of better quality when building with the IL method in relation to the coupling joint for other span-by-span methods.

In fib (2000) it is stated that the IL method provides a good geometry control. The geometry of the bridge is well controlled since the formwork is robust and reused in the construction yard.

4.2.7 Safety and detoriation

An additional advantage is the fact that the IL method produces a continuous bridge beam. When a new segment is cast and tensioned together with the previous the surface of the first segment is soaked with water and there will be a good quality joint between the two segments without any cracks. This is favourable since the behaviour and the robustness of the bridge will be superior to a precast alternative in which segment joints will be visible. There are two advantages within this subject.

First, the long time behaviour will be better when segment joints are closed. If the joints are open dirt and water can easily gather, where after detoriation and freezing cracks easily can occur. This will especially be a problem in Sweden with the cold climate and salt on the roads according to Carlsson (2005).

Another advantage is the safety aspect. When using precast elements that only are placed on top of the piers there will be a risk that segments can fall down, for example during an earthquake. This will not be the case when the bridge is a continuous beam. Structural Engineering International (2005)

4.3 Disadvantages

The IL method has of course not only favourable aspects. The characteristic of the method has also unfavourable effects, which are important to be aware of if the construction method should be used in a successful way. In the following sections the major subjects that may limit the performance of the construction method are stated. Most of the limitations are due to the bridge launching sequence.

4.3.1 Launching alignment

When building with incremental launching the general launching requirements are that the superstructure should be launched over the piers without additional deviations. To be able to launch the bridge over the existing piers both the horizontal and the vertical radius have to be constant. To have a constant radius in both planes limits the ranges where IL is possible to use. To increase the usability of the IL method it could be combined with other construction methods. One further option is to launch the bridge from both abutments, or from any two points of the alignment, and by this increase the possibilities of the method since you can use different radii for the two launching parts.

One other opportunity, that could be used to obtain a certain alignment, is to either increase or reduce the width of the cantilever deck. According to Göhler Pearson (2000) variations up to 0,75 m are possible but in between 0,75 and 1,0 m it could be rather comprehensive. This allows having a slightly different alignment of the launching compared to the final road.

Figure 4.4 shows a schematic description of how the flange width can be changed during launching to obtain a small difference in curvature.

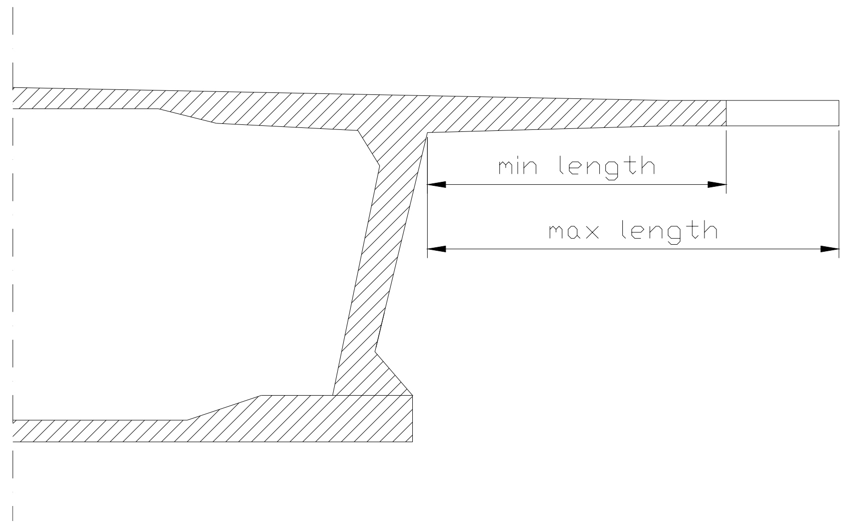


Figure 4.4 Schematic description of how a varied curvature can be arranged through widening of the top slab.

Also the geometry in elevation can be slightly manipulated to obtain the deck gradient required. As long as the bottom of the box is constant, it is possible to vary the height of the cross section. One should be aware of that both these measures only slightly increase the usability range. Still the limitation due to the restricted alignment is noticeable. Furthermore these measures require additional work in the process at the casting yard.

4.3.2 Structural demand due to launching

To be able to launch a heavy concrete bridge out over its support the design has to consider not only the final state but also the launching sequence. As mentioned in Chapter 1, Section 1.3 all sections in the bridge will be exposed to positive and negative moments during the launching sequence. To resist the varying moment a launching prestressing is needed. In most cases the launching prestressing is kept in the final state with additional parabolic or polygonal prestressing to increase the structural performance. However, when the bridge is in its final state, the launching prestressing is not utilized in the most effective way, which result in a larger amount of prestressing steel compared with other construction methods.

The fact that the cross-section of the bridge needs to be constant results in an overcapacity of many sections in the final bridge. There will not be any possibility to make a haunched bridge as well. The amount of used concrete and prestressing will therefore increase.

4.3.3 Aesthetic

The aesthetic aspect of a bridge always has to be considered since a bridge will be a part of the surroundings for decades. Bridge design has according to fib (2000) two different aspects: the integration of the structure into its surroundings, either urban area or natural site, and the intrinsic bridge architecture itself.

When building a bridge with the IL method the bridge cross section is constant. It is hard to judge whether it is aesthetically a benefit or not regarding to the two different aspects mentioned above. Still it must be seen as a negative consequence since the architectural design is limited to a fixed constant height of the section for the whole bridge.

4.3.4 Foundation and substructure

According to Pettersson (2005) the substructure and the foundation have to be of high quality since the superstructure is really stiff. Even a small settlement would create a significant increased amount of stress in the superstructure and there could be a certain risk of cracking. When launching a steel bridge the foundation and substructure is less expensive to construct, since the weight and the stiffness of the bridge is less.

Another dilemma is the launching procedure, which induces horizontal forces on the sliding bearings and down in the piers. The substructure and the foundation have to be carefully analysed for all the construction states where all possible combinations of movements and forces are considered.

4.4 Conclusions

Incremental launching is a bridge construction method with several characteristics both favourable and unfavourable. To be able to carry out a good design and construct the bridge in a successful way these special characteristics all have to be considered. According to Pettersson (2005) it is of highest importance, if a bridge should be constructed with the IL method, that it is designed for it from the very beginning. We insist in what Pettersson (2005) says and think it is of greater importance if the design team and the contractors are inexperienced in the IL method.

One can say that the dilemma for the IL method is the increased amount of material usage compared with other modern construction methods. This negative aspect has to be compensated for by the favourable characteristics that the IL method has. To be able to minimize the material usage and preserve the rational way of construction a proper design of the prestressing arrangement has to be done. An investigation of different prestressing arrangement is conducted and can be found in Chapter 5, Chapter 6 and Chapter 7.

If the right preconditions for IL are acquired and the product ability is considered in the design, already from the feasibility study, we believe that the method is competitive. Also the fact that it is frequently used outside Sweden confirms this conclusion.

5 Prestressing

When using the incremental launching method you need to arrange the prestressing tendons with regard to the launching sequence during construction as well as the final service state. The prestressing in the construction state has to accommodate the varying moment during the different launching states. Since all cross-sections of the bridge will be subjected both to maximum positive as well as maximum negative bending moments, the section has a need for prestressing to reduce the tensile stresses in both the top and the bottom flange. Therefore the launching prestressing should be arranged so that the resultant is placed as near the centre of gravity of the whole box girder section as possible. The final configuration of the prestressing has to be designed to take care of the moment obtained from the different loads such as dead weight, traffic and accident loading similar as for ordinary constructed bridges.

In this text we will refer to *launching prestressing* as the prestressing required during launching, and to *final prestressing* as the prestressing in the final service state.

In Chapter 5, Chapter 6 and Chapter 7 an investigation of the prestressing arrangement is carried out. First ten different prestressing alternatives are presented in Chapter 5, where after an investigation is conducted through a discussion that compares the different arrangements, Chapter 6. The results from this investigation are used to determine the most interesting alternatives, which will be evaluated by FE-analysis to find the most effective solution in terms of amounts of materials (concrete and prestressing steel) in Chapter 7.

5.1 Prestressing methods

Generally the tendons can be placed in several different ways that less or better fulfil the different kinds of demands. One choice that has to be done is whether internal or external tendons should be used, or a combination of both. The choice of prestressing arrangement has to be done based on an investigation that clarifies the different aspects and effects that have to be considered. A general discussion about internal and external prestressing follows in Chapter 5, Section 5.1.1 and Section 5.1.2.

5.1.1 Internal tendons

Internal tendons are placed inside the concrete section in corrugated steel ducts. In some cases, as when corrosion protection by electro-chemical methods is needed, plastic ducts can be used. Nilsson (2005). The tendon can be inserted into the duct immediately when the bridge is launched or pushed into the duct when the bridge is in final position depending on what effect is needed from the prestressing. In both cases it is of highest importance that the ducts are placed in their right positions and that no concrete or slurry emerge into the ducts while casting the bridge. When the tendons have been inserted into the ducts and tensioned, the ducts are grouted with cement grout to ensure the corrosion protection and to create interaction between the tendons and the surrounding concrete.

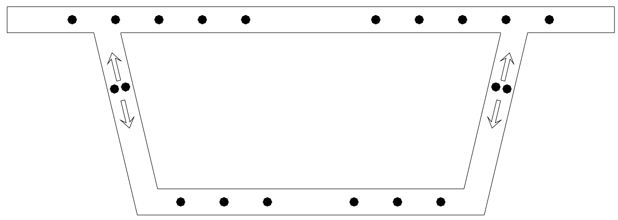


Figure 5.1 Section with internal tendons. The arrows show the paths of the internal parabolic tendons along the bridge. The tendons are placed in the upper part of the webs over supports and in the lower part in mid span.

The bridges built in Sweden so far have been provided with internal prestressing were the parabolic tendons have been inserted when the bridge is in its final position. The typical section can be seen in Figure 5.1. An important thing to take notice of is that the grouting of the ducts will take place outside the shed. In cold regions such as Scandinavia this could cause problem and you have to grout when the temperature is high enough. If such a problem occurs the steel has to be protected against corrosion until grouting is done. Göhler Pearson (2000).

Another disadvantage with internal prestressing is that the tendon length is limited due to the friction losses, especially for parabolic tendon profiles. For the straight launching tendons this will not be any problem since the length of the segments sets the length of the tendons. The tendons are never longer than two, or in special cases three, segments. Therefore the friction losses will not determine this length. For further information regarding the tendon length see Chapter 6, Section 6.1.5.

When using eccentric internal tendons in the flange the lever arm to the prestressing force could be somewhat increased compared to external prestressing inside the box section and the prestressing is used more efficiently. Though a minimum distance of ~150 mm from the bottom edge of the slab to the bottom edge of the recess has to be kept to avoid problem with high local compression over supports during launching, see Figure 5.2. Göhler Pearson (2000)

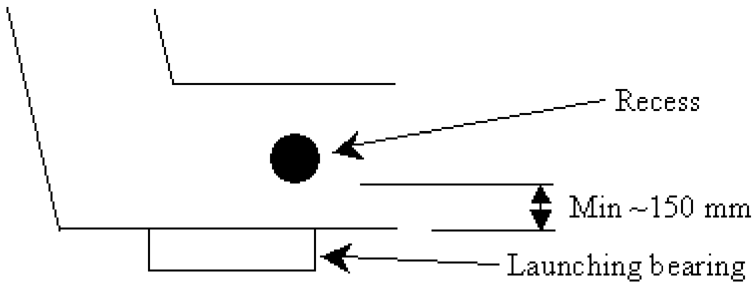


Figure 5.2 Minimum distance of ~150 mm due to high local compression.

A major advantage with internal prestressing is the ability to restrain cracks. When a crack occurs the bond between the prestressing steel and the concrete limits the crack width by strain compatibility in a way that external prestressing is unable to do.

Long-term losses of the prestress of approximately 15 % of the initial prestressing force can be expected for internal prestressing. This value derives from experience and includes concrete creep and shrinkage as well as relaxation of the prestressing steel. Davidson (2005)

5.1.2 External tendons

The use of external tendons complicates the formwork and reinforcement, but offers advantages such as easier inspections and possibilities to remove tendons. The improvement of the mechanical properties of the cross section due to the lower number of holes and also the fact that the cross section can be made smaller is also an advantage. These main advantages and disadvantages have to be investigated carefully to be able to carry out an appropriate design. In Figure 5.3 and Figure 5.4 schematic figures of external prestressing can be seen.

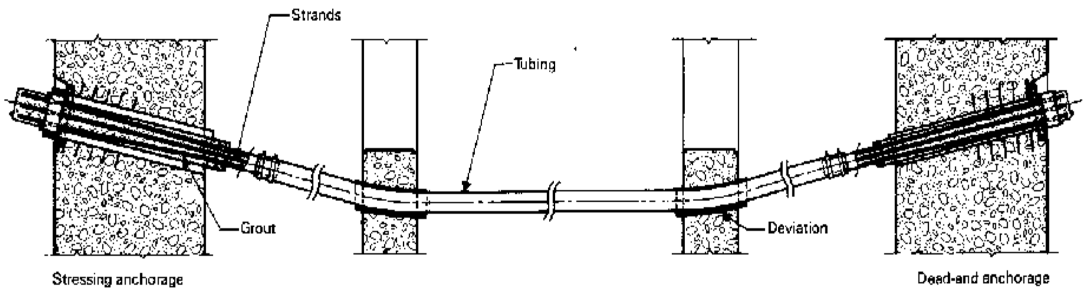


Figure 5.3 Schematic figure of a external prestressing tendon and deviators. VSL (1992)

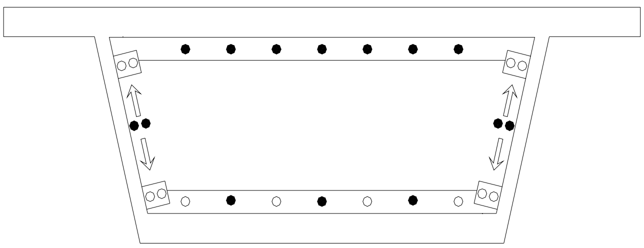


Figure 5.4 Section with external tendons.

Another aspect that needs to be considered is the maintenance cost. According to Rosignoli the maintenance cost are lower when using external prestressing since the tendons can be inspected easily and the advantage of the bundles of strands guarantees their optimum alignment both in straight zones and inside deviators, and this allows a good control of tensioning and elongation during the whole service life. Rosignoli (2002).

External prestressing has low friction losses, which leads to higher average tensile stresses in the strands and therefore a higher efficiency of prestressing. Low friction losses also allow using tendons as long as several spans. This reduces the number of anchorage and also the labour cost. To further reduce extra work when using external tendons the support diaphragms can be used as anchorage.

The long terms effect for external prestressing is same as for internal prestressing. The losses are approximately 15 % according to Nilsson (2005). This value includes concrete creep and concrete shrinkage as well as relaxation of the steel.

5.2 Different prestressing schemes

As mentioned earlier the prestressing can be divided into two parts. One is the prestressing for the launching sequence and the other one is for the final state of the bridge. The following section presents different tendon arrangements. Some of the alternatives are taken from the literature and from the old Swedish bridges, while some of the alternatives are new combinations suggested by the author's.

5.2.1 Alternative 1

This alternative is used in all of the incrementally launched bridges in Sweden. Internal prestressing is used both for the launching and final prestressing. The tendons for launching prestressing are placed in the top and bottom flanges and the final tendons, tensioned after launching, are placed parabolic in the web.

The launching tendons are tensioned before launching. In the final state the straight launching tendons work together with the polygonal final tendons without being detensioned. A schematic sketch can be seen in Figure 5.5.

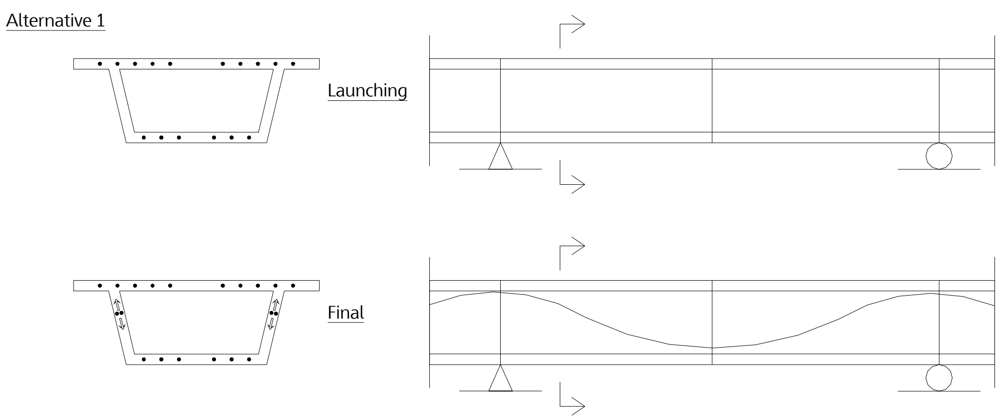


Figure 5.5 Alternative 1, internal tendon arrangements in both launching and final state.

5.2.2 Alternative 2

This prestressing arrangement has external tendons for the launching prestressing and internal tendons for the final state with a polygonal shape. The deviators have to be placed in such a way that the coupling of the segment is possible.

The internal tendons for the final prestressing are tensioned after launching and placed in the web. A schematic sketch can be seen in Figure 5.6.

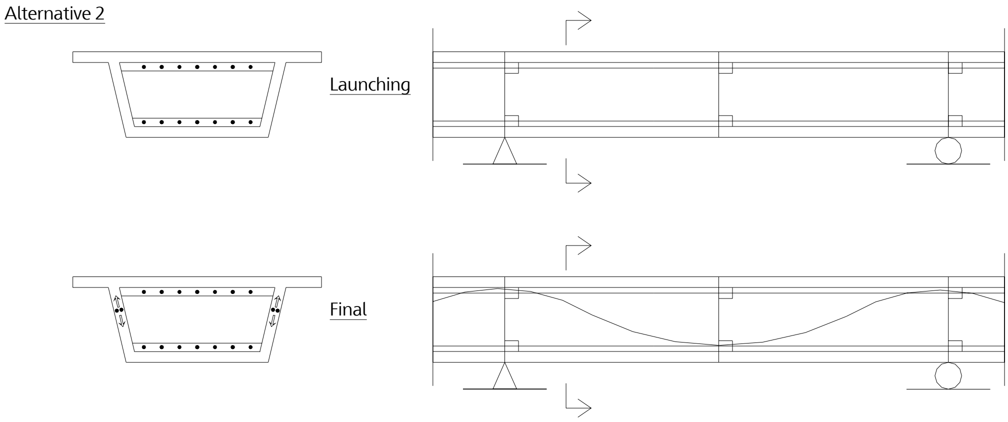


Figure 5.6 Alternative 2, both internal and external prestressing.

5.2.3 Alternative 3

The launching prestressing is obtained in the same way as in the first alternative, but the final arrangement has external polygonal prestressing tendons instead of internal.

The tensioning of the external tendons is made after launching. Also the manufacturing of the deviators could to be made outside the casting yard to speed up the launching process. A schematic sketch can be seen in Figure 5.7.

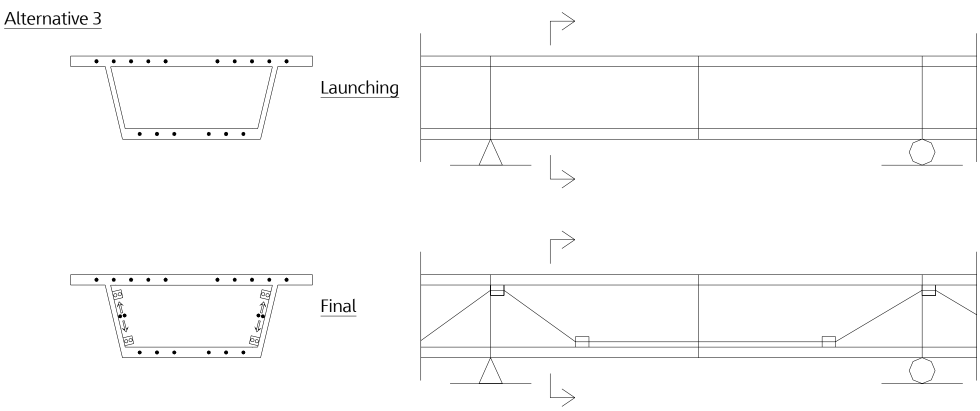


Figure 5.7 Alternative 3, both internal and external prestressing.

5.2.4 Alternative 4

This alternative has internal launching prestressing arranged just like in alternative 1 and 3. For the final state the launching prestressing is combined with external straight tendons placed in zones exposed to high bending moments.

As mentioned earlier the deviators for the final arrangement can be manufactured outside the casting yard if it is favourable. A schematic sketch can be seen in Figure 5.8.

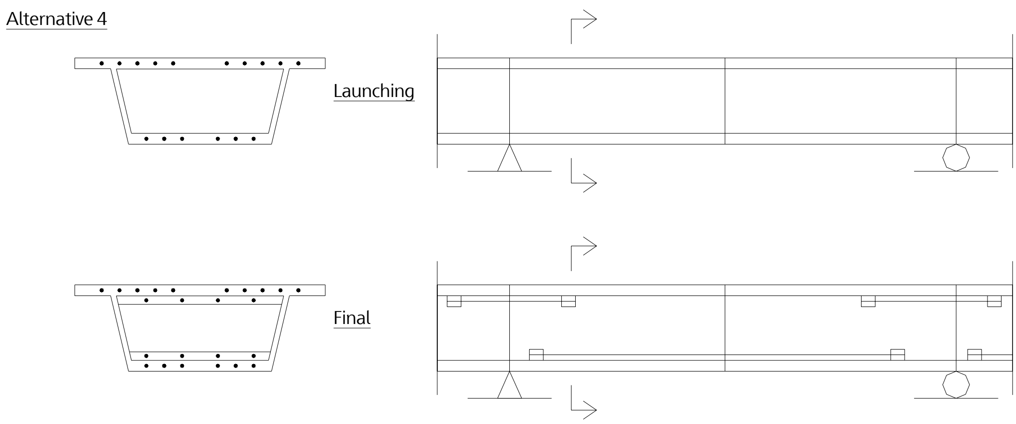


Figure 5.8 Alternative 4, both internal and external prestressing.

5.2.5 Alternative 5

This alternative has a launching prestressing realised by both internal and external straight tendons. When the bridge is in its final position, the external tendons needed in the launching state but now placed in unfavourable positions are detensioned and arranged into a more efficient arrangement.

Instead of reusing the launching tendons it is possible to introduce new tendons, depending on the cost and in what condition the old tendons are. A schematic sketch can be seen in Figure 5.9.

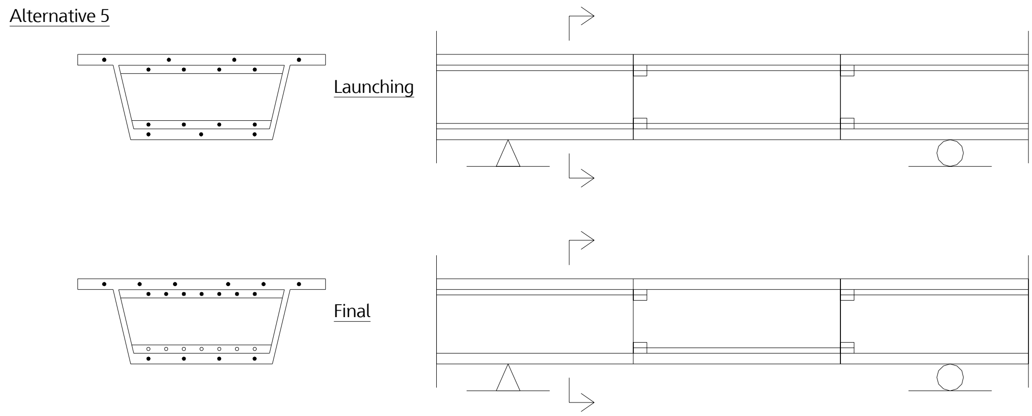


Figure 5.9 Alternative 5, both internal and external prestressing.

5.2.6 Alternative 6

All the tendons are provided before launching. To obtain the launching prestressing the final internal parabolic tendons are balanced with antagonistic external polygonal tendons during launching. When the bridge is in the final position the external antagonist tendons are removed.

When using this kind of prestressing scheme the launching prestressing must be obtained before launching the segments. This means that the deviators need to be placed in a proper manner so that the external prestressing can be applied correctly. A schematic sketch can be seen in Figure 5.10.

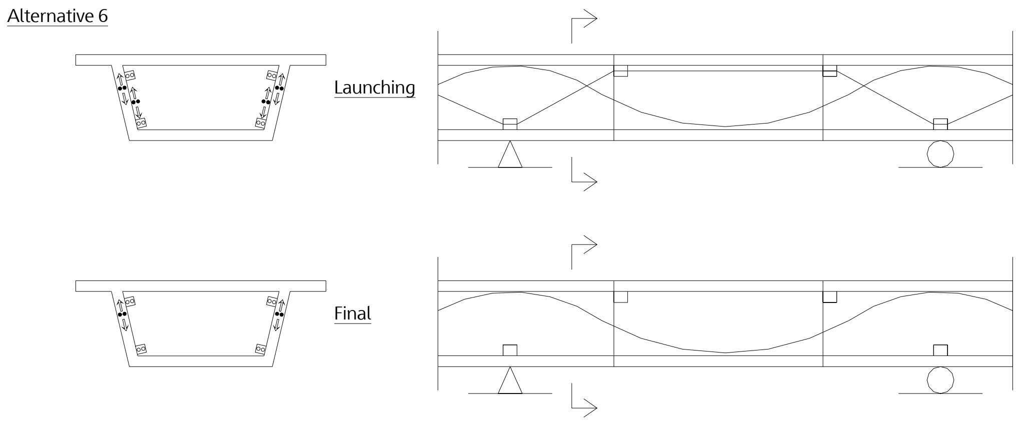


Figure 5.10 Alternative 6, both internal and external prestressing

5.2.7 Alternative 7

This alternative has almost the same prestressing configuration as alternative 6, except from the extra straight external tendons placed in zones exposed to high bending moments. This additional prestressing makes it possible to design the launching prestressing only regarding the launching sequence.

Straight tendons accomplish the extra bending capacity needed when the bridge is in its final position. A schematic sketch can be seen in Figure 5.11.

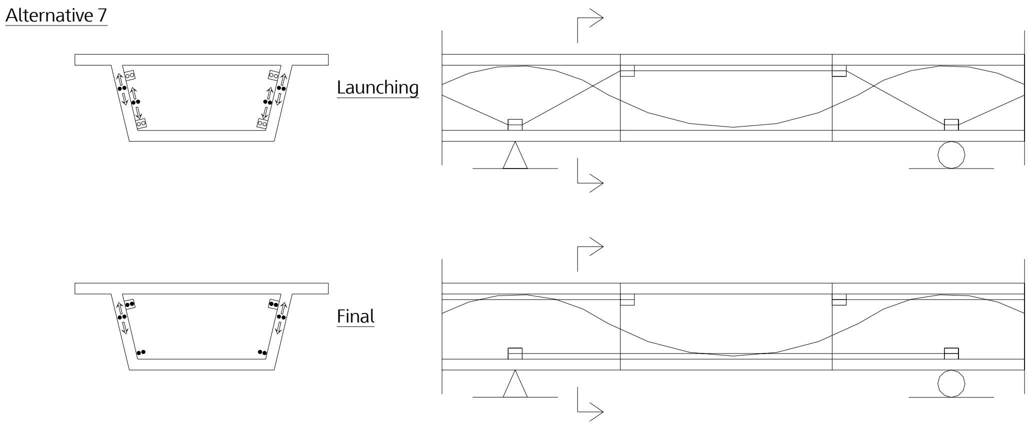


Figure 5.11 Alternative 7, both internal and external prestressing

5.2.8 Alternative 8

This alternative is arranged with only external prestressing. The launching prestressing is obtained with straight external tendons anchored to the concrete in a number of deviators. The amount of deviators is determined by the length of the segments contra the spans.

When the bridge has been launched unfavourable tendons are detensioned and extra polygonal tendons are assembled in the existing deviators. A schematic sketch can be seen in Figure 5.12.

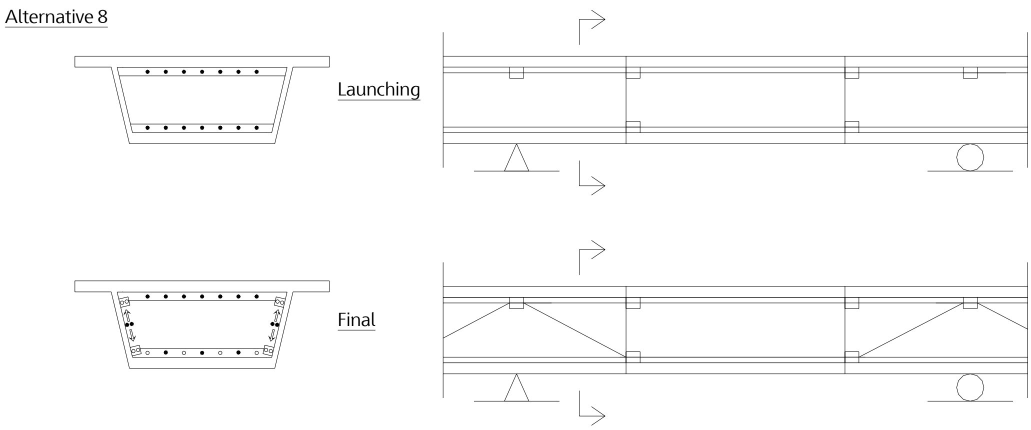


Figure 5.12 Alternative 8, only external prestressing

5.2.9 Alternative 9

Rosignoli presents this alternative with a low amount of internal prestressing to permit sizing the polygonal tendons to resist most of the permanent loads. According to Rosignoli this alternative has an extremely efficient permanent prestressing. Rosignoli (2002).

The launching prestressing is realised by straight internal tendons. Polygonal external tendons are inserted before launching, and during the launching temporary external antagonist polygonal tendons are used. Extra straight temporary external tendons lift the prestressing force to the level of the centre of gravity.

In the final arrangement the external straight tendons are detensioned as well as the antagonist half of the polygonal tendons. The antagonist tendons are rearranged, span-by-span, and retensioned. Straight tendons provide the extra prestressing that is needed in case of long spans. A schematic sketch can be seen in Figure 5.13.

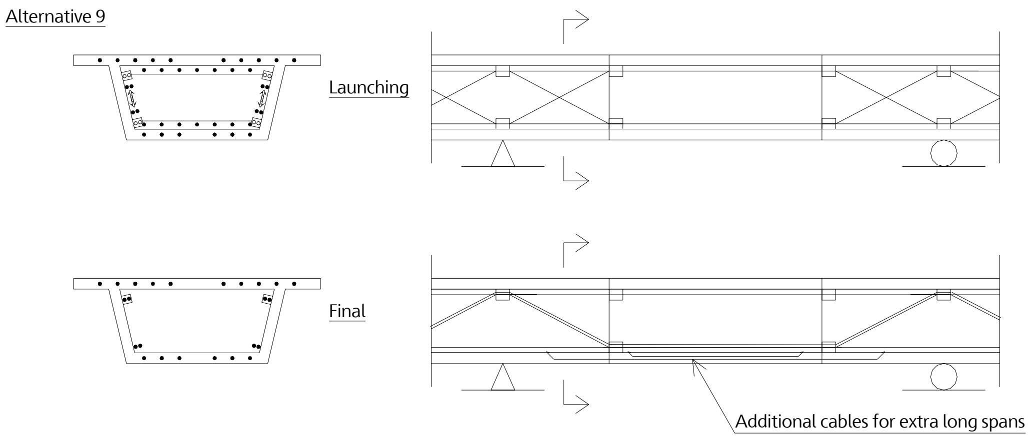


Figure 5.13 Alternative 9, both internal and external prestressing

5.2.10 Alternative 10

This alternative has similarities to alternative 3 but with extra straight tendons in the final arrangement. As in alternative 3 the launching prestressing is realised only by internal straight tendons.

The final prestressing has two different layout opportunities. Both alternatives have external polygonal tendons. This prestressing are inclined and designed to handle the shear force and balance a certain amount of bending. Straight tendons fulfil the extra prestressing that is needed in case of long spans. In alternative A the tendons are internal and in alternative B the tendons are external with anchorage in existing deviators. A schematic sketch can be seen in Figure 5.14.

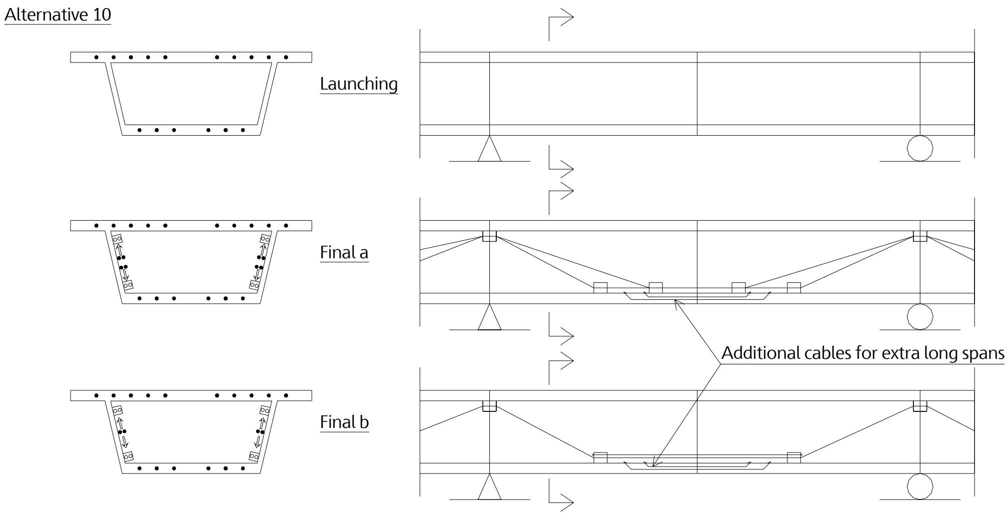


Figure 5.14 Alternative 10, both internal and external prestressing.

6 Evaluation of Prestressing Arrangements

6.1 Decisive parameters

This chapter will present some decisive parameters regarding prestressing and from these make an analysis of the prestressing alternatives presented in Chapter 5, Section 5.2. The analysis will result in a grading, from which the four best alternatives will be selected. Table 6.1 shows the grading system for the different decisive parameters. A compilation of the grading is found in Chapter 6, Section 6.2, and conclusions from the analysis are stated in Chapter 6, Section 6.3.

Table 6.1 Grading system for the decisive parameters

--	=	Negative
-	=	Slightly negative
0	=	Moderate
+	=	Slightly positive
++	=	Positive

6.1.1 Flange thickness

One of the main aims is to reduce the concrete volume as long as the total cost for prestressing and reinforcement does not increase. If the thickness of the flange can be reduced this of course results in savings. According to Rosignoli (1999) the thickness of the top flange depends on the ability to handle the creep deformations that springs from the large local compression stresses at the anchors. This local compression requires extra reinforcement to prevent cracking due to the transverse tensile stresses that occur. The thickness also depends of the actual size of the dead end and active anchors. A typical active anchor is shown in Figure 6.1, and the anchor spacing can vary between 310mm (for 12 strands) and 430mm (22 strands). Besides this a minimum concrete cover has to be included in the height. If the prestressing in the flanges could be removed, by use of external prestressing, then the flange thickness might be reduced.

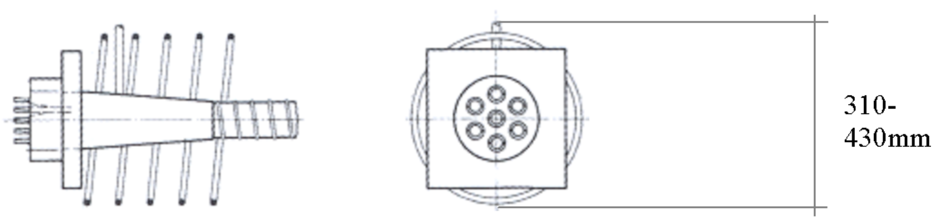


Figure 6.1 Active anchor. VSL (2005)

Another thing that might be governing is that the flange functions as a transversal slab and has to be able to transfer the loads transversally to the webs. Therefore the flange must have a sufficient moment and shear capacity in the transversal direction. Lundstedt (2005). Further on, for the structural behaviour of a box girder, as for all structural members, the moment of inertia is one decisive parameter. The moment of inertia depends mainly on the flange area and distance between the flanges of the box girder. Since the centre of gravity is placed closer to the top of the beam, the upper flange needs a certain area to create a high moment of inertia. Carlsson (2005) also states this, and that the possibility to reduce the thickness of the flange is limited.

The conclusion is therefore that the effect of removing the internal prestressing in the flange might be neglect able when it concerns the overall structural behaviour and the structural behaviour of the flange. Summarizing that there might not be any possibility to reduce the flange thickness if tendons are removed. Therefore all alternatives will be unaffected by this. Table 6.2 shows a listing of the grades.

Table 6.2 Grades regarding flange thickness

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	0	0	0	0	0	0	0	0	0	0	0

6.1.2 Web thickness

In the same way as for the flange thickness a reduction of the web size without increasing the cost for prestressing and reinforcement gives an overall cost saving. When using internal prestressing in a box girder bridge the web thickness is approximately 600 to 800 mm when the span is about 40 to 60 m. This corresponds to the needed space to accommodate the prestressing tendons according to the codes. If the tendons instead are placed externally, the thickness of the web could be reduced by approximately 50 % to about 400 mm for the same spans. Carlsson (2005). It can be quite cumbersome to reduce the thickness more since the webs need to handle the shear forces and the torsion.

Figure 6.2 shows a principal drawing of a web section with internal tendons. It shows that he absolute minimum thickness just to fit all the ducts and the reinforcement is 550 mm. The measures are found in VSL product catalogue. VSL (2005)

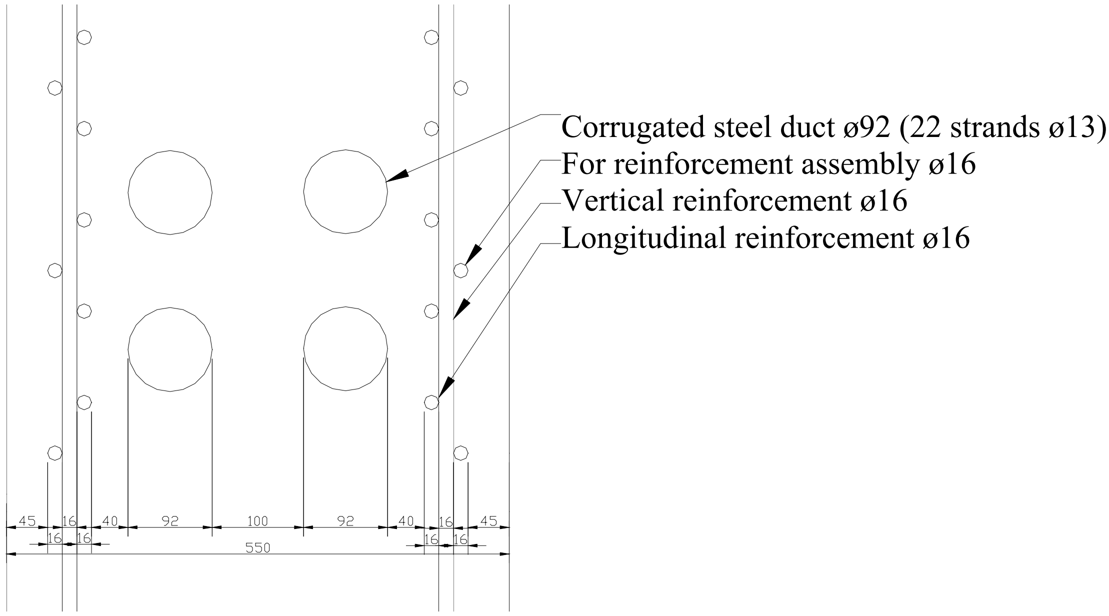


Figure 6.2 Schematic layout of prestressing ducts in a box girder web. Minimum web thickness is 550 mm for four tendons, each with 19 strands

This result will affect alternatives with internal prestressing in a bad way, while alternatives with external prestressing will get "++" since the effect of reducing the web thickness is quite high concerning the dead weight. Table 6.3 shows a listing of the grades.

Table 6.3 Grades regarding web thickness

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	-	-	++	++	++	-	-	++	-	++	++

6.1.3 Number of deviators

When designing prestressed concrete bridges for incremental launching, one of the main aims is to make the work process lean and as repetitive as possible. Therefore obstacles such as deviators and anchorages for tensioning should be avoided. Carlsson (2005). By using internal straight tendons for the launching prestressing this could be avoided totally since both the dead end anchors and the active anchors are placed within the concrete section at the joints. If external prestressing is to be used instead, the number of deviators and anchors would increase dramatically. This is due to that each segment needs to be tensioned to the previous and therefore needs at least one deviator in the top and bottom. Figure 6.3 shows how the number of deviators increases when external tendons are used.

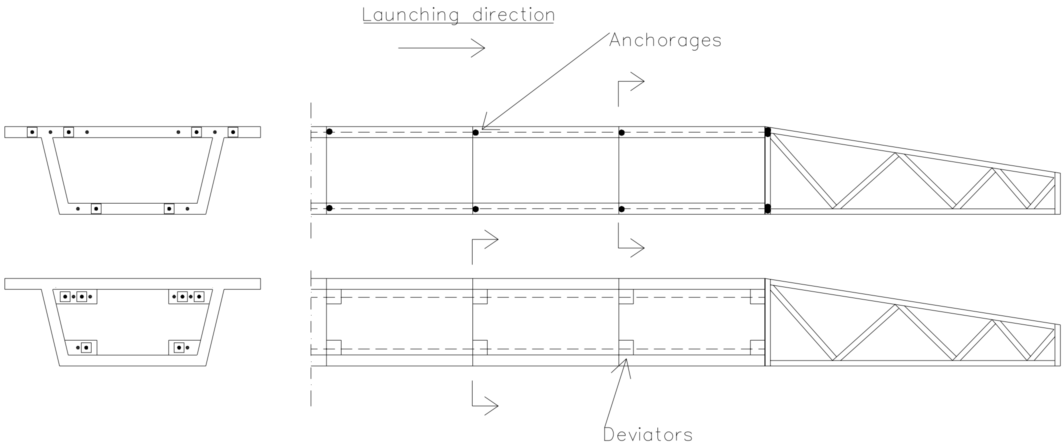


Figure 6.3 Principal figure that shows the increased number of deviators when using external instead of internal launching prestressing.

The final prestressing can be arranged either internally parabolic or externally polygonal. Due to frictional losses and production matters the tendons cannot be longer than a couple of spans for internal tendons and several spans for external tendons. Therefore the tendons need to be tensioned and overlapped at deviators in some points within the structure. For the parabolic and polygonal tendon arrangements the overlap is generally located in regions where the bending moment is close to zero for internal tendons and over the supports for external tendons. Rosignoli (2002).

For internal prestressing the tensioning of the tendons is usually realised by inserting heels in the structure where the tendons emerge into the box girder. These heels require extra reinforcement and are cumbersome to produce, but in general quite well known. For external prestressing the structure has to be provided with deviators to assure that the tendon can be placed at its nodes that the prestressing force can be transferred into the structure. A typical way of obtaining this is shown in Figure 6.4. Figure 6.5 shows reinforcement for an external deviator. Of course the support diaphragms can be used for anchorage of the external tendons as well, but still they also have to be produced. As one can conclude, the use of external prestressing requires more deviators than the use of internal, and because of this more work time and material cost.

The effect of this will be quite governing for the whole construction procedure and the classification for this will be as seen in Table 6.4. Table 6.5 shows a listing of the grades.

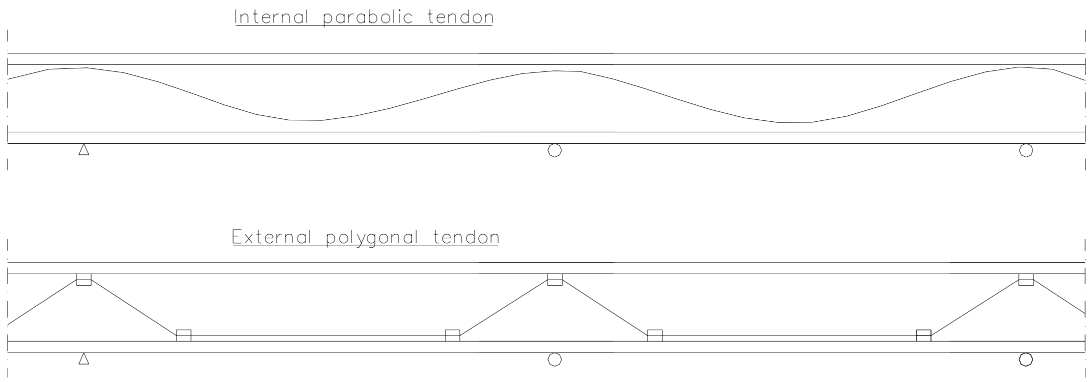


Figure 6.4 Example of tendon arrangement, internal and external prestressing. Notice the deviators required when using external polygonal prestressing

Figure 6.5 Picture from inside a box girder where the reinforcement for two concrete deviators is seen. The deviators will be cast when the bridge is totally launched.

Table 6.4 Grades for number of deviators

Alternatives with…	will get
external launching prestressing, straight or polygonal tendons	--
external final prestressing, polygonal tendons (3-4 deviators/span)	-
external final prestressing, straight tendons (~2 deviators/span)	0
no external prestressing at all	++

Table 6.5 Grades regarding number of deviators

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	++	--	-	0	--	--	--	--	--	--	0

6.1.4 Corrosion protection

One aspect that always has to be considered in bridge engineering is the durability regarding concrete degradation and corrosion of reinforcement. The concrete degradation will be the same for any type of prestressing arrangement as long as the cracking aspects are the same, micro cracking and widths of macro cracks. More about this can be found in Chapter 6, Section 6.1.8. But regarding the corrosion the different alternatives provide various protections.

When using internal prestressing, the tendons are placed in corrugated steel ducts, which later on are grouted. This provides a sufficient corrosion protection, since the steel duct has to be corroded and then the grouting degraded before the actual strand corrosion starts. However, one might argue that if a crack occurs in the concrete structure, the duct is no longer protected from the environment and the detoriation can start. Rosignoli (2002).

In external prestressing the tendons are placed inside a plastic duct filled with grease. This plastic duct can be placed inside a larger plastic pipe that can be grouted to get double corrosion protection. VSL (1992). Further on the pipes are placed inside a concrete box girder that could be climatically protected and even controlled. According to Rosignoli (2002) this provides a better corrosion protection than for internal prestressing. The maintenance costs also will be lower for external prestressing, since the tendons can easily be inspected, according to Nilsson (2005). The pipes and ducts can just be cut open and the strands can be checked.

Still these effects are not sufficiently well understood and considered differently around the world, even within Europe. Therefore, we conclude that only the alternative with totally external prestressing will be slightly better since the control and maintenance will be easier to carry out. As a consequence only alternative 8 will be more favourable in this aspect, and the other alternatives will be neutral. Table 6.6 shows a listing of the grades.

Table 6.6 Grades regarding corrosion protection

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	0	0	0	0	0	0	0	+	0	0	0

6.1.5 Tendon length, launching tendons

The cost is proportional to the tendon length in such way that if a tendon is longer, there will be less cost in tensioning the bridge since there will be fewer tensioning points. When providing the structure with launching prestress, it is important to tension each segment to the previous one. This can be done by tensioning all tendons at each joint, or by overlapping the tendons so that they are tensioned at each second or each third joint. In Figure 6.6 a typical layout of the launching tendons can be seen. The common solution is to tension the tendons at each second joint. This requires less work than tensioning at each joint and it is cheaper since fewer anchorages are needed, but it also keeps the process quite smooth since it can be hard to handle such long tendons as three segments. This also reduces the compressive forces in the young concrete. If the full launching prestress was to be tensioned at each joint the concrete would have to be cured longer to enable a longer growth of the strength. And since a rapid casting-tensioning-launching cycle is preferred when constructing incrementally launched concrete bridges, a short curing period of only about three days is wanted. Tensioning only at each second joint is therefore often preferred.

Since the structure has to be provided with launching prestressing in the same way both with internal and external prestressing there will be no difference in tendon length. Even when polygonal tendons are used for the launching prestress they have to be tensioned at each or at each second joint. Therefore there will be no difference between internal and external prestressing with regard to tendon length. The only thing that could save tendon length is to detension and rearrange unfavourable launching prestressing tendons, and in that way reuse some tendons. However, this operation requires quite high work cost and the overall economical aspect of this is questionable since it is not assured that the strands can be reused. Nilsson (2005). Table 6.7 shows a listing of the grades.

Table 6.7 Grades regarding tendon length, launching tendons

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	0	0	0	0	+	0	+	0	-	0	0

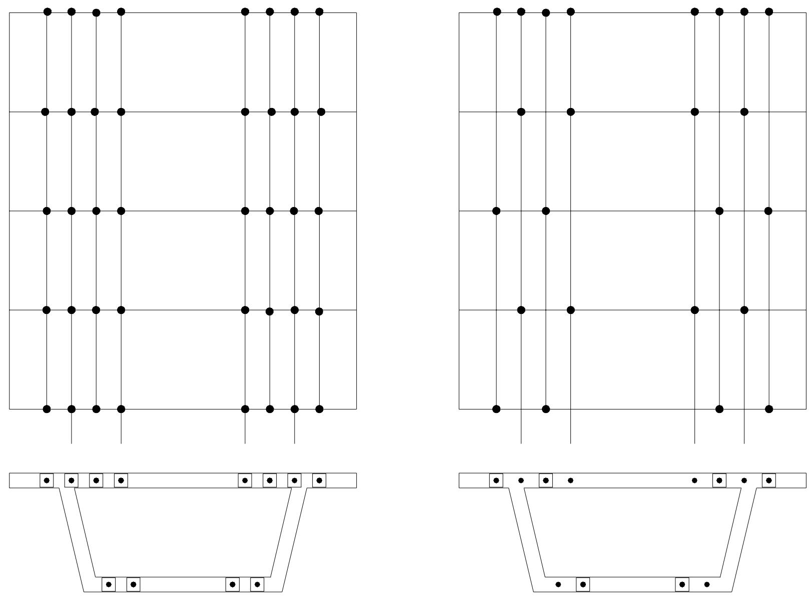


Figure 6.6 Typical layout of launching prestress with straight tendons (upper flange) tensioned at each joint (left) and overlapping tendons tensioned at each second joint (right). The dots symbolize anchorages.

6.1.6 Tendon length, final tendons

The length of the final tendons depends on which system of prestressing is adopted. If the final tendons are placed in a parabolic or polygonal shape, the length of them is depending on if they are internal or external. Due to frictional losses and production matters internal tendons cannot be longer than a couple of spans. If the tendons instead are placed externally they can be much longer since there will be very small frictional losses. The tendons can be up to several spans. Therefore external polygonal tendons will be more economical compared to internal when only considering the length of the tendon. This will result some benefits for Alternative 3, 8, 10a and 10b. The rest will be unaffected. To be able to look deeper into the difference in frictional loss between internal and external prestressing and verify the thesis stated above, some examples are calculated in Chapter 6, Section 6.1.6.1.

The final prestressing can be conducted through the use of straight tendons at the upper and lower flange in the regions of high positive and negative moments. The tendons would of course be quite short and therefore this would be less economical.. Table 6.8 shows a listing of the grades.

Table 6.8 Grades regarding tendon length, final tendons

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	0	0	+	-	-	0	0	+	0	+	+

6.1.6.1 Frictional losses, calculations

The frictional losses along the prestressing tendon can for a certain section be calculated according to equation 6.1. The calculations are done to show the difference in frictional losses between internal and external prestressing. The length of the tendons varies in the example between one to four spans. One span length is assumed to be 45 m. Table 6.9 shows the parameters for the calculations.

$$P_i(s) = P_i(0) \cdot e^{-\mu(\alpha + k \cdot s)} \tag{6.1}$$

Table 6.9 Values to calculate frictional losses in prestressing

μ	=	0,19	For strands in duct (internal tendon). Eurocode (2004)
μ	=	0,12	For bare strand inside plastic tube running over saddle (external tendon). VSL (1992)
k	=	0,01	For internal parabolic. Eurocode (2004)
k	=	0	For external polygonal. VSL (1992)
		μ	Frictional coefficient
		s	The length of the duct from the active end to the section under consideration [m]
		α	Change in slope over the same distance [rad]
		k	Unintended change of slope per unit length [1/m]

The change in slope over the same distance differs between internal and external arrangements due to the variation in eccentricity of the tendons and the placement of deviators. For a span to depth ratio of 19 an internal tendon changes with 4-6° at each angle, see Figure 6.7. (5° used for calculations). For the external ones the change in slope is 6-8° for each angle, see Figure 6.8. (7° used for calculations).

For one span the change in slope is therefore four times the change in slope for the specific case since the tendons changes it angle four times over the span. This is valid for both external and internal tendons.

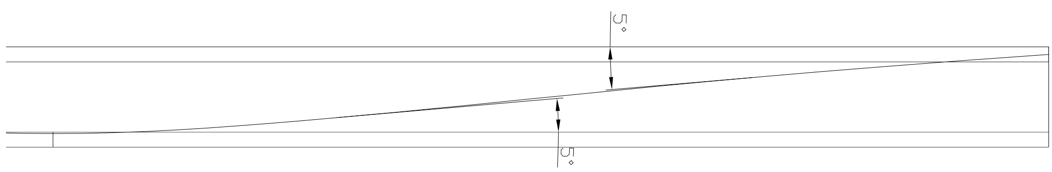


Figure 6.7 Deviation angle for internal prestressing, 5°

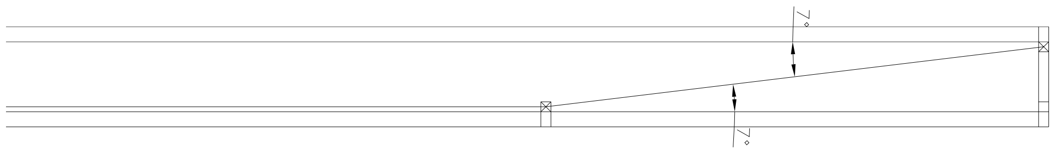


Figure 6.8 Deviation angle for external prestressing, 7°

The tendons are assumed to be stressed from both edges and the friction losses presented in the right column of Table 6.10 is mean values assumed to be half of the friction loss obtained in the section examined.

Table 6.10 Calculated frictional losses in prestressing

Type of arrangement	Tendon length	s	α	$e^{-\mu(\alpha+k\cdot s)}$	Mean friction loss
Internal	45 (1 span)	22,5	0,349	0,897	5,17 %
External	45 (1 span)	22,5	0,489	0,943	2,85 %
Internal	90 (2 spans)	45	0,698	0,804	9,80 %
External	90 (2 spans)	45	0,977	0,889	5,52 %
Internal	180 (4 spans)	90	1,396	0,649	17,7 %
External	180 (4 spans)	90	1,954	0,791	10,5 %

The calculation shows that the frictional losses are less for the external than for the internal tendons. The difference increases with the tendon length and a 45 m long internal tendon has about the same amount of fictional losses as a 90 m long external tendon.

6.1.7 Eccentricity of prestressing tendons

To effectively balance the bending moment of the bridge beam due to dead weight and loads, the prestressing tendons have to be placed as eccentric as possible. This is of course to get a large lever arm to the tendon. When using internal prestressing this is much easier to achieve since the tendon can be placed as near the surface as the minimum concrete cover and/or anchorages allow. Though a minimum distance of ~150 mm from the bottom edge to the tendon has to be kept to avoid problem with local crushing effect over supports during launching. Göhler Pearson (2000). See also Chapter 5, Section 5.1.1.

For external prestressing a big eccentricity is harder to obtain since there is actually a physical object, the flanges, that prevent the tendon from reaching the top or the bottom of the beam. Figure 6.9 and Figure 6.10 shows the maximum possible eccentricity of prestressing tendons.

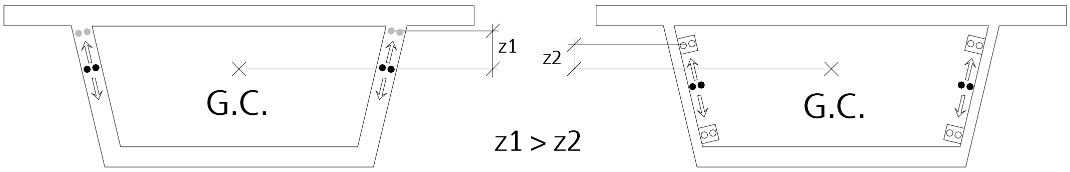


Figure 6.9 Maximum possible eccentricity of internal and external prestressing tendons placed in webs, parabolic (left) and polygonal (right) profile.

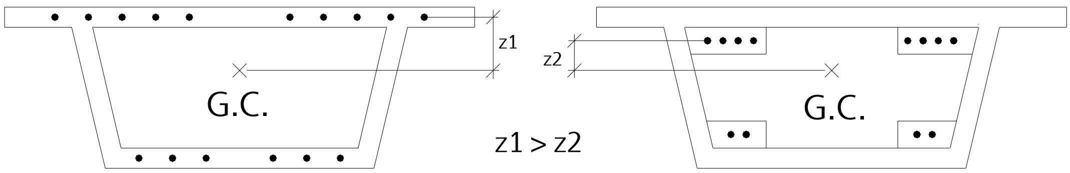


Figure 6.10 Maximum possible eccentricity of internal and external tendons placed in the flanges, straight profile.

Especially when using external straight tendons this will be obvious, and therefore the number of tendons or the prestressing force has to be increased.

The result from this is that alternatives with only internal prestressing will get "+", alternatives with eater launching or final prestress external will get "0" and alternatives with only external will get "-".Table 6.11 shows a listing of the grades.

Table 6.11 Grades regarding eccentricity of prestressing tendons

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	+	0	0	0	0	0	0	-	0	0	0

6.1.8 Crack widths/sustainability

One of the main differences between internal and external prestressing is the cracking behaviour. Internal prestressing tendons are always grouted to create a corrosion protection and a bond between the tendon and the surrounding concrete. When cracking occurs the tendon gets a certain strain localisation within the crack, but it also slips a little against the surrounding concrete. The bond resistance results in a resistance to pull out of the tendon near to the crack, and the crack is kept together resulting in small crack widths.

When using external prestressing a crack opening will not result in strain localisation in the tendon, but the strain will be more or less uniformly distributed between adjacent deviators or anchor. The steel strain will not increase significantly and the tendon cannot prevent opening of the crack in a controlled way.

Still there will be a better durability of incrementally launched concrete bridges compared to conventional constructed prestressed concrete bridges. This is due to the launching prestressing that is provided to the entire structure. The launching prestressing creates compression in all the bridge's cross sections that makes the bridge more uncracked than conventionally built prestressed concrete bridges. Pettersson (2005). Therefore all alternatives will get a good grade in the evaluation. However, prestressing alternatives with no internal strands will get "0", and alternatives with only internal will get "++". Table 6.12 shows a listing of the grades.

Table 6.12 Grades regarding crack widths/sustainability

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	++	+	+	+	+	+	+	0	+	+	+

6.1.9 Inclined tendons with regard to shear

It is good to have inclined tendons in a bridge, since the inclined tendons can balance some of the shear forces direct. Some of the alternatives have no inclined tendons why these will get a poor grade.

When building IL bridges there are quite high stresses introduced in the cross section due to the launching prestress. These stresses limit the possibility to introduce parabolic or polygonal tendons in the final state, especially in long bridges where the launching prestress is rather high. This is as mentioned not well, since the parabolic or polygonal tendons are good to take shear forces as mentioned. Therefore it can be quite practical to use the straight launching tendons to balance some of the bending moment and use extra parabolic or polygonal tendons to handle shear forces and achieve sufficient moment resistance.

Another way of producing a good behaviour of the bridge is to introduce some polygonal external or parabolic internal tendons in the final state, while detensioning some of the straight tendons in the upper flange in the span and in the lover flange over the support. More about this subject is treated in Chapter 6, Section 6.1.12.

Alternatives 7, 9 and 10a are provided with the feature that they can take the shear forces with inclined tendons and the moment with straight once and will therefore get a good grade from this. Table 6.13 shows a listing of the grades.

Table 6.13 Grades regarding inclined tendons with regard to shear

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	+	+	+	--	--	+	++	0	++	++	++

6.1.10 Need for extra reinforcement at couplings/anchors

The beam section needs extra reinforcement when internal couplings or anchorages are used. Due to the concentrated compressive force that occurs inside the concrete at the anchorages transverse tensile stresses occur that require reinforcement to keep equilibrium of the cracked concrete. This phenomenon will be most cumbersome when internal straight tendons are used, since the anchors are placed inside the concrete at both ends of the tendons. When using external straight and polygonal as well as internal parabolic tendons the problems with anchors will result in extra work with deviators, which is treated in Chapter 6, Section 6.1.3. Table 6.14 shows a listing of the grades.

Table 6.14 Grades regarding need for extra reinforcement at couplings/anchors

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	-	0	-	-	-	0	0	0	-	-	-

6.1.11 Launching

One of the more important things when building IL bridges is that the casting and launching process is smooth and rapid. This gives the contractor the possibility to create a short and effective working cycle that guaranties a well-built bridge in a short time. Therefore the prestressing has to be arranged in such way that the launching can take place without delays due to need for long curing times. This is only possible when the launching tendons are straight and internal since the tendons can be overlapped and only half the prestressing force is applied when the concrete is fresh. This is described in Chapter 6, Section 6.1.5. When using external straight tendons, the tendons can of course be overlapped as well, but problems with high local compression arises. The concrete will be subjected to very high compressive stresses when it is quite young and there will be risk for plastic deformations and/or crushing.

As described earlier the curing time for IL bridges is desirably less or equal to three days to create a good and effective working cycle. However, if for example deviators are to be used, the concrete in the deviators has to cure for approximately five days, which in total gives a longer construction time. Rosignoli (2002).

Alternatives with internal launching prestress will be positively grade due to this, whilst alternatives with external launching prestress will be negatively affected. Alternatives 6, 7 and 9 will be highly negative influenced, since both polygonal and antagonist tendons have to be tensioned and will therefore be graded “--“. Table 6.15 shows a listing of the grades.

Table 6.15 Grades regarding launching

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	+	-	+	+	-	--	--	-	--	+	+

6.1.12 Possibility to detension tendons

As previously mentioned in Chapter 3, Section 3.2.5, there is no economy in detensioning and rearranging tendons, the workload is generally too high. However, in one case there can be an advantage with this procedure, and that is when long spans are to be launched. Then the launching prestressing is generally very high a. This will result in extremely high compressive stresses in the section and only a few tendons can be added in the final prestressing, why some of the launching prestress at least needs to be detensioned. Rosignoli (2002).

The efficiency of the final prestressing is then not the most appropriate and the structural system is ineffective. On the contrary, if some tendons of the launching prestressing could be detensioned, a more effective final prestressing scheme could be achieved since more final tendons could be used.

Alternatives with external launching prestress will get a positive grade since they can be detensioned. Alternative 9 will get a very high grade since, it is possible to introduce more tendons in high moment areas. Table 6.16 shows a listing of the grades.

Table 6.16 Grades regarding possibility to detension tendons

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Grade	-	+	-	-	+	+	+	+	++	-	-

6.2 Comparison

The grades from the different decisive factors are complied in Table 6.17. Of course one could argue that some sort of weighting system should be used to give a further dimension to the comparison. Our opinion is that the comparison has been a way of gathering different aspects regarding prestressing more than a true comparison in every sense. We believe however that the grading system will present a rather fair picture of the comparison, indicating which alternatives that are better or worse. Selection of the most promising alternatives for further analysis, and comments regarding this, is found in Chapter 6, Section 6.3.

Table 6.17 Gathering of grades for the decisive factors

Alternative	1	2	3	4	5	6	7	8	9	10a	10b
Flange thickness	0	0	0	0	0	0	0	0	0	0	0
Web thickness	-	-	++	++	++	-	-	++	-	++	++
Number of deviators	++	--	-	0	--	--	--	--	--	--	0
Corrosion protection	0	0	0	0	0	0	0	+	0	0	0
Tendon length, launching tendons	0	0	0	0	+[1]	0	+[1]	0	-[2]	0	0
Tendon length, final tendons	0	0	+	-	-	0	0	+	0	+	+
Eccentricity of prestressing tendons	+	0	0	0	0	0	0	-	0	0	0
Crack widths/ sustainability	++	+	+	+	+	+	+	0	+	+	+
Inclined tendons with regard to shear	+	+	+	--	--	+	++	0	++	++	++
Need for extra reinforcement at couplings/ anchors	-	0	-	-	-	0	0	0	-	-	-
Launching	+	-	+	+	-	--	--	-	--	+	+
Possibility to detension tendons	-	+	-	-	+	+	+	+	++	-	-
Σ	+4	-1	+3	-1	-2	-2	0	+1	-2	+3	+5

[1] Might be possible to reuse some tendons by rearranging them
[2] Both straight and inclined tendons for launching prestressing

6.3 Conclusions

The different prestressing alternatives have been compared with regard to the different decisive parameters. The result from this comparison can be seen Chapter 6, Section 6.2. Some conclusions can be drawn from this and thereafter four qualified alternatives are to be chosen for a finite element analysis.

In the bottom row of Table 6.17 the overall result can be observed. This result will underlie the choice of which alternatives that are going to be further investigated, though it is not totally determining. The four selected alternatives are presented and motivated below.

Alternative 1 has received the best result with +4 and will be analysed further. This alternative has been used in all of the incrementally launched bridges built in Sweden and can therefore be seen as a good reference object for further modelling. Its primary weakness lies in the rather thick webs due to the internal parabolic tendons, which makes the bridge quite heavy. The inability to detension tendons to avoid unfavourable stresses in the final state is of course another weakness, which could be governing for the span length. The advantages with alternative 1 are, as Table 5.4 shows, several, primary its rational way of construction and good performance.

Alternative 3 got +3 with almost the same result pattern as the first alternative. The launching prestressing is the same as in the first alternative but the final arrangement has external polygonal prestressing tendons working together with the straight internal ones. One advantage when modelling both alternative 1 and 3 is the straightforward possibility to evaluate the effects of external polygonal prestressing compared to internal. Another reason that increases the interest to further analyse this alternative is that it has been used by Skanska in Poland, according to Szczepanik (2005).

The next arrangement that was selected for structural analysis is alternative 8. This alternative has been able to collect +1 point in the comparison matrix. As this alternative is arranged only with external prestressing it is an interesting alternative in the analysis. The favourable effects when using only external tendons are several, for example that the concrete section could be reduced as well as the possibility to detension and easy check the tendons. Its primary weakness is the way of construction due to the amount of deviators needed and that the eccentricity of the tendons is not optimised.

Finally alternative 9 is going to be analysed. One might argue why the alternative with such a poor result needs to be further investigated. Though reasons to analyse this alternative are several. The first one is that it is of interest to analyse a structurally optimised alternative to find out how efficient the alternative is. The comparison in Chapter 6, Section 6.2 focuses on a smooth and rapid production as well as on saving material but it may disregard the advantages with an optimised prestressing arrangement such as to avoid the need of extra temporary piers and also decrease the number of permanent piers. According to Rosignoli (2002) arrangements like this are the recent trends.

The reason why alternative number 10 is not further investigated is because of the similarities to alternative 3. If the additional straight tendons in the mid span are added they would have exactly the same arrangements.

7 Detailed Analysis of Prestressing Arrangements

The analysis presented in Chapter 6 regarding the different prestressing alternatives has only concerned different production and economical aspects. Only a few structural issues were included in the preliminary choices of the optimal prestressing arrangement. To obtain deeper understanding of the structural response the four chosen alternatives were evaluated through finite element calculations, referred as FE calculations. The results from this analysis will then form a basis for a total final comparison where the most effective prestressing solution regarding material usage and economical aspects will be selected.

One should be aware of that this is not an analysis with the intention to perform an accurate design of the prestressing tendons. The idea with the analysis was to clarify the difference in structural behaviours between the different alternatives. Due to this the analysis was kept on a global level. Simplifications, using experience data instead of making a totally accurate model, were made to improve the efficiency of the analysis.

In Chapter 7, Section 7.1, Figure 7.1 an overview of the way the analysis was done is presented and thereafter the modelling is described more in detail. Further on, conclusions will come and in the end a comparison of the different prestressing arrangements.

7.1 Analysis scheme

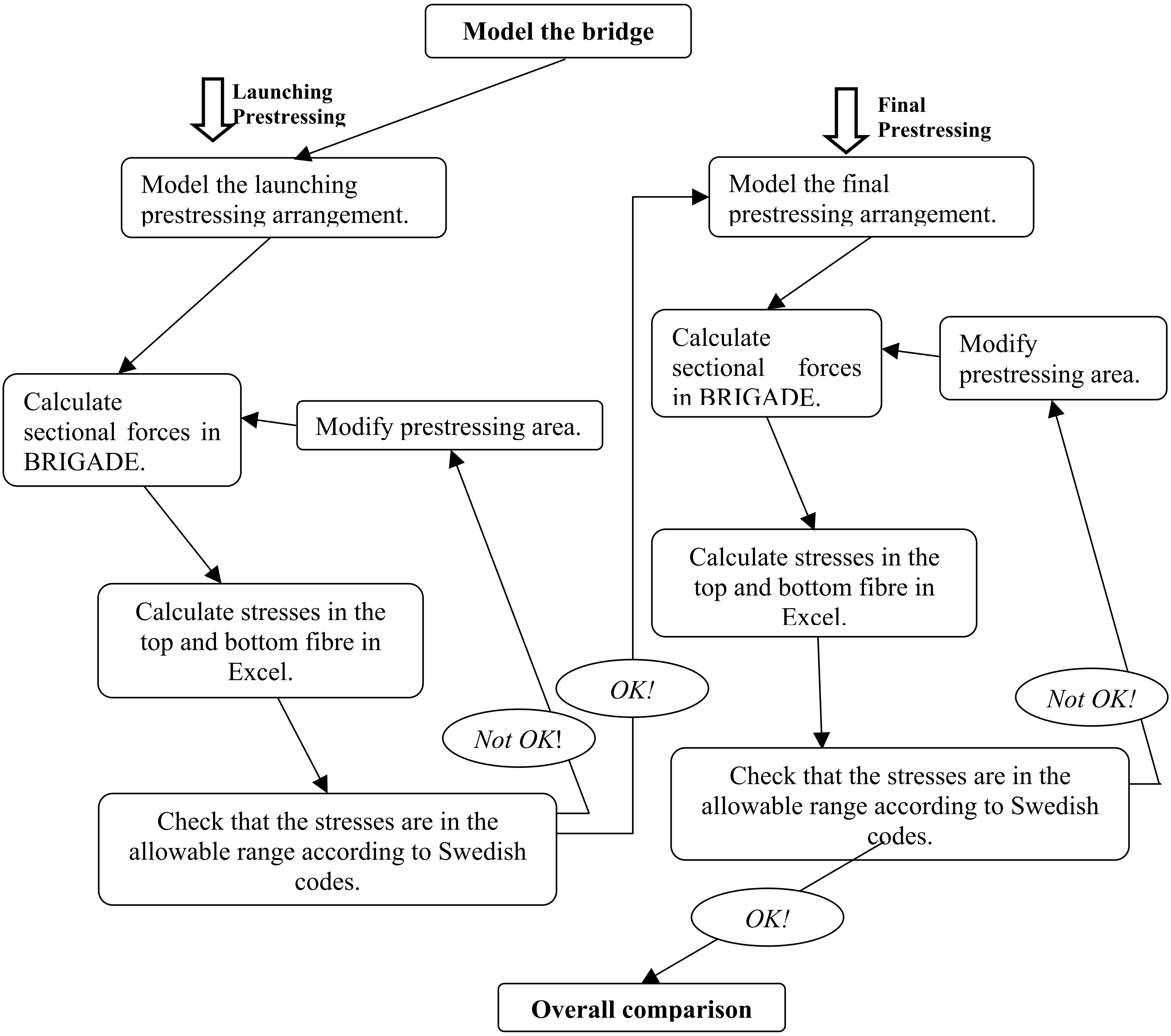


Figure 7.1 Overview of FE analysis scheme

7.2 FE program

For the FE calculations a program called BRIGADE/Standard was used. BRIGADE/Standard, hereafter called BRIGADE, is a FE program from Scanscot Technology AB that is especially developed for bridge calculations. BRIGADE is a program that uses Abaqus technology and runs in Microsoft Windows. It has a preprocess where bridge geometry, prestressing tendons and forces and loads are given. BRIGADE uses a three-dimensional geometry for both the physical geometry and the load handling, and the different loads acting on the bridge are automatically generated.

Results can be studied in an output module both in three dimensions and in sections across and along the bridge. Sectional forces and deformations are output that can be evaluated after the analysis. Scanscot Technology AB (2002).

7.3 FE modelling

7.3.1 General

The FE modelling is rather important, since a comparison between quite similar prestressing schemes is of interest. Still, a time consuming analysis could not be accepted since there were several alternatives that were about to be analysed. The aim of the FE calculations was to find out how much prestressing is needed for the different prestressing arrangement to obtain acceptable performance according to Bro 2004.

To do this comparison the section forces were obtained from the FE analysis and further analysed in an excel sheet. A FE beam model seemed to be a suitable way of modelling the bridge part. A beam model provides smooth and rapid analysis and gives the output needed to calculate the stresses and it was not so time consuming.

Both the launching and the final prestressing arrangement were modelled and analysed to distinguish the different advantages and disadvantages. The launching arrangements were exposed to only the dead weight, while the final arrangements were analysed with regard to loads automatically generated in BRIGADE. During the launching sequence the front zone will be exposed to additional stresses due to cantilever effects and needs to be extra reinforced and prestressed. This special case was not considered in this more general level.

The basis for the analysis was the Swedish IL concrete bridge over Lule Älv in Gäddvik. This bridge acts as a reference bridge, which was adopted with the different prestressing alternatives that were about to be analysed. The web in the concrete section was made thinner when the arrangement had no internal prestressing tendons in the webs.

The comparison between the different alternatives was conducted through a stress control obtained when the bridge was exposed to load combination V:A and V:B. Stresses that were compared were those in the top and bottom fibre in support section and span section respectively. This will be further explained in Chapter 7, Section 7.3.2 and Section 7.3.7.

7.3.2 Geometry of reference bridge

The bridge crossing Lule Älv is 610 m long with spans that are 45 m, except the end spans that are 35 m. See Figure 7.2.

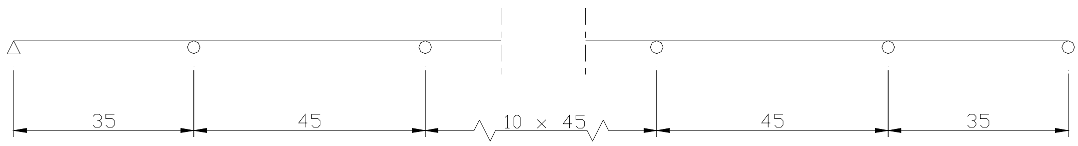


Figure 7.2 Structural model of the reference bridge

To simplify the analysis, but also reduce the effects from the end spans, the FE model consisted of five spans with an equal length of 45 m, which can be seen in Figure 7.3. The analysed bridge part is then 225 m long. Only critical sections in the mid span and over one of the interior supports were considered.

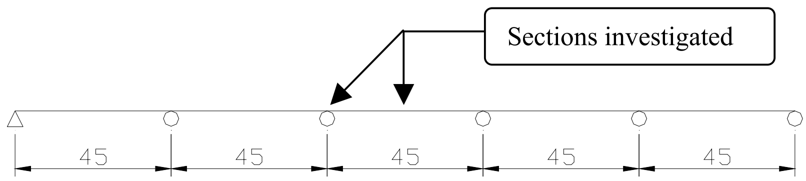


Figure 7.3 Simplified structural model of the reference bridge

The box girder cross section of the reference bridge is shown in Figure 7.4 it corresponds to the bridge over Lule Älv. This cross-section was analysed with different prestressing schemes. Only the thickness of the web decreased when external polygonal prestressing tendons were assumed.

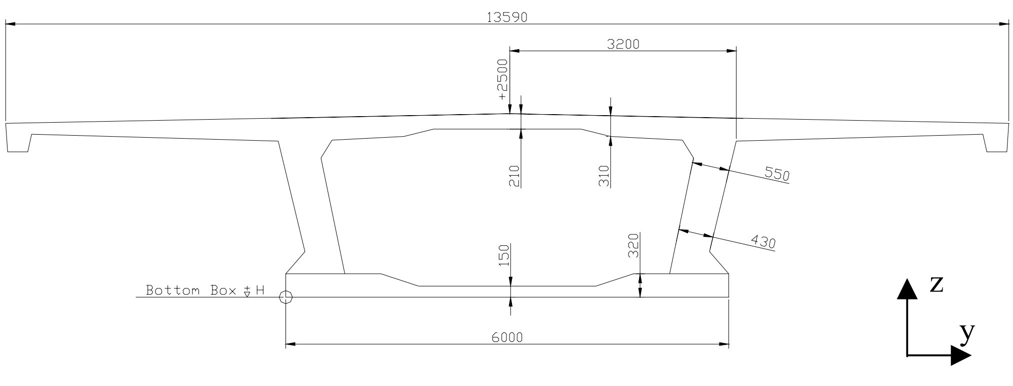


Figure 7.4 Cross section of the reference bridge

7.3.3 Sectional constants

For calculating the cross-sectional properties of the reference bridge the program FEM Design's help module Section was used. The effective flange width was calculated according to BBK 04 as 1/10 of the distance between moment zero sections.

The zero moment sections appear approximately at ¼ of the span. This gives an effective flange width of 2 m on each side, when calculating a bit on the safe side, see Figure 7.5.

The effective cross-sectional area was drawn and calculated in FEM Design. The result can be found in Table 7.1. Hand calculations were performed to verify that the result were acceptable.

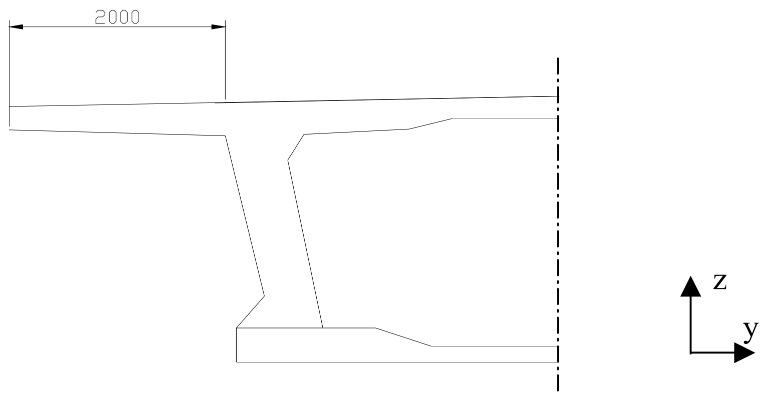


Figure 7.5 Effective flange width of the reference bridge

Table 7.1 Sectional constants of the reference bridge

A	6.072 m^2
I_y	5.173 m^4
W_{yuf}	-5.037 m^3
W_{ylf}	3.512 m^3
I_z	43.36 m^4
I_t	11.14 m^3
I_{yz}	0 m^4
I_γ	12.77 m^6
Z_g	1.055 m

When the prestressing arrangement had external tendons in the web, the concrete web thickness was reduced. The web thickness was reduced from 550 mm and 430 mm to 300 mm and 270 mm according to VSL (1992). These values seem to be reasonable regarding the interview with Carlsson (2005). The modified section with the thinner web can be seen in Figure 7.6 together with its sectional constants in Table 7.2.

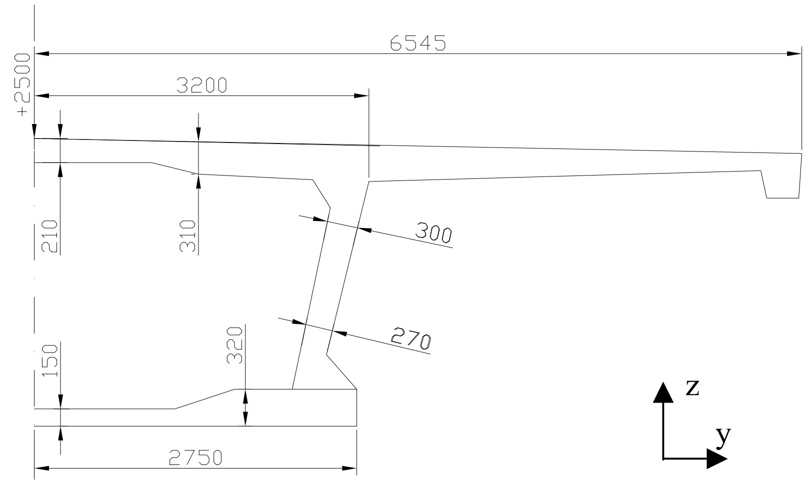


Figure 7.6 Modified section in case of no internal tendons in the web

Table 7.2 Sectional constants, modified section of the bridge crossing Lule Älv in Gäddvik

A	5.359 m^2
I_y	5.003 m^4
W_{yuf}	-4.929 m^3
W_{ylf}	3.369 m^3
I_z	38.51 m^4
I_t	10.57 m^4
I_{yz}	0 m^4
I_γ	8.488 m^6
Z_g	1.050 m

7.3.4 Boundary conditions

The model has five supports where the first support is fixed in all three directions and also consists of two extra movable bearings to prevent rotation around the longitudinal axis of the bridge. The other supports are set as movable along the bridge and fixed in all other directions. In Figure 7.7 a three-dimensional schematic sketch showing the boundary conditions is seen.

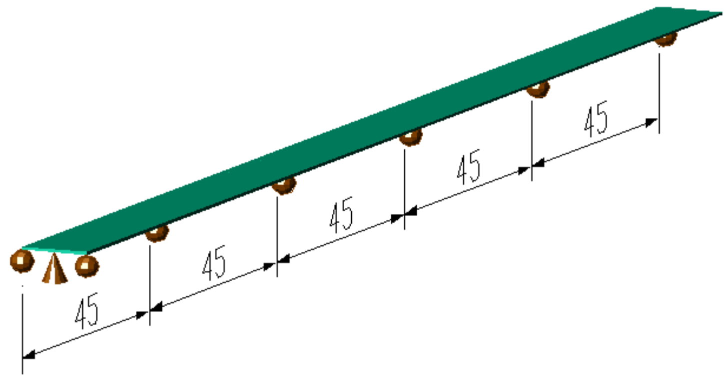


Figure 7.7 Visualization of boundary conditions

7.3.5 Material data

The following material data has been inserted into BRIGADE.

7.3.5.1 Concrete

The concrete used in the existing Lule Bridge was specified as K400T. The class is similar to the notation K40 used in the Swedish concrete code of today. The concrete was assumed to have the properties as shown in Table 7.3.

Table 7.3 Assumed properties of concrete

E_{ck}	f_{cck}	Density	Poisson's ratio	Shear modulus	Thermal expansion	Damping
32 GPa	28,5 MPa	25 kN/m^3	0,2	13,33 GPa	1e^{-5} 1/K	0,04

7.3.5.2 Prestressing steel

The assumed properties of the prestressing steel, modelled in BRIGADE, are shown in Table 7.4.

Table 7.4 Assumed properties of prestressing steel

E_{pk}	$f_{p0.1k}$	Density	Poisson's ratio	Shear modulus	Thermal expansion	Damping
200 GPa	1550 MPa	78 kN/m^3	0	1000 GPa	1e^{-5} 1/K	0

7.3.5.3 Deck

To make a correct model in BRIGADE a bridge deck has to be used to transfer the load transversally into the beam element, see Figure 7.8. Though the structural effects longitudinally along the bridge as well as the dead weight of the deck is already included in the cross sectional constants of the section. See Figure 7.4, Figure 7.5 and Figure 7.6. Therefore the stiffness of the deck in the longitudinal directions and the dead weight of the deck are set to zero, otherwise it would have been included twice. The transverse properties are set to correct material values to obtain a transverse load carrying capacity.

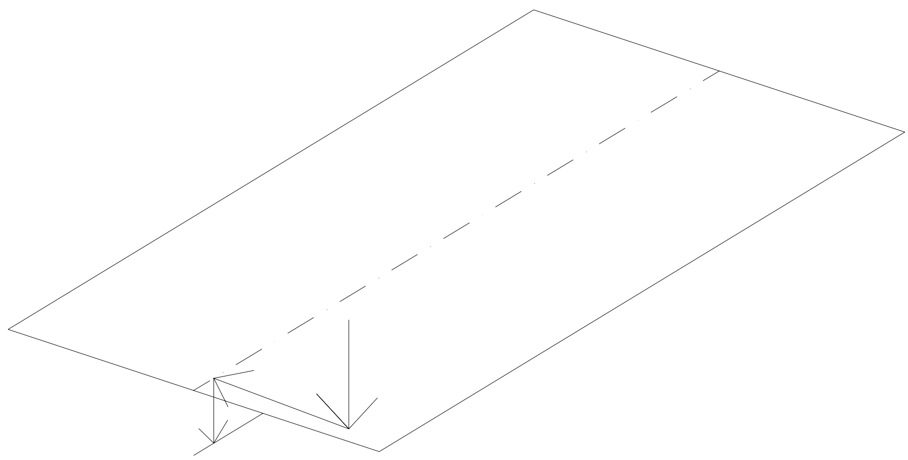


Figure 7.8 Principle figure of how the deck will transfer loads into the beam. The deck has no stiffness in the longitudinal direction but only in the transversal direction.

7.3.6 Prestressing

The prestressing tendons were assumed to be stressed up to 80 % of $f_{p0.1k}$. To keep the analysis on a general level the steel areas were considered without regarding the actual number and distribution of individual tendons. To translate the steel areas into tendons would not be relevant when the idea with the analysis was to distinguish the structural behaviour of the different alternatives. Though, the possibility to fit the tendon arrangement in the sections will be discussed.

The internal and external prestressing was modelled in two different ways. Modelling of the internal prestressing is quite straightforward in BRIGADE. Coordinates, steel areas and initial steel stresses are stated in the program. The tendons are not modelled as single ones, but instead one tendon group is modelled as one continuous tendon along the bridge. Due to this it is not possible to let BRIGADE automatically reduce for the slips and the frictional losses since it would be quite different from the reality. Neither will the effects be of any relevance when the choice of tendon length distinguish the difference in frictional losses between internal and external tendons according to Chapter 6, Section 6.1.6.1. Therefore the frictional losses were not taken into account in the FE analysis. Instead the friction losses were included in the overall analysis conducted in Chapter 6.

The external prestressing was modelled as external forces acting in the points where the tendon is fixed to the bridge in deviators. These external forces will not take the tendon elongation into account, which will underestimate the influence of the prestressing slightly, but is an approximation good enough.

Both the internal and external prestressing tendons were assumed to be retensioned once all tendons are tensioned. This is done to compensate for the stress reduction in the prestressing strands that comes from the shortening of the concrete due to successive prestressing. However it is impossible to retension the internal launching tendon as they are covered with concrete and grouted. Though this is not taken into account in the FE analysis and will be a simplification on the unsafe side.

7.3.6.1 Steel stress and long term effects

As mentioned earlier the prestressing tendon area was modified until the stresses were allowable according to Bro2004, see Figure 7.1 in Chapter 7, Section 7.1. All the tendons were initially stressed to a level of 1240 MPa for the different alternatives.

The long time effects were considered in the FE calculations by a reduction factor. For internal arrangements the steel stress was decreased with 15 % compared to the initial prestress. The reduction value is based on experience and includes the effect of the concrete creep and shrinkage as well as the relaxation of the steel. Davidson (2005).

For the external tendon arrangement the prestressing was also reduced with approximately 15 % for long-term effects. The long time effects should not differ significantly between internal and external prestressing. Nilsson (2005).

7.3.7 Loads

BRIGADE automatically generates the loads according to the Swedish Code Bro 2004. The loads were applied onto four lanes over the bridge deck. The load combinations that were used to analyse the behaviour of the bridge are shown in Table 7.5. The allowable stresses for the different states are shown in Table 7.6 in Chapter 7, Section 7.4.1.

Table 7.5 Load combinations

Launching		SLS			
		Initially		Long term	
Dead weight	Check of stresses	LC V:A	Check of stresses	LC V:A	Check of stresses, effective prestressing force
		LC V:B	Check of stresses		

7.3.8 Mesh

The mesh was set regarding to the time of the analysis versus a proper result. The mesh size was obtained by testing different configurations. Finally the deck was modelled using 16 elements in the transverse direction and 35 elements in each span for the longitudinal direction. The loads were applied with a mesh configuration of 16 nodes for the transverse direction and 175 nodes in each span for the longitudinal direction. The beam element had the same mesh configuration as the deck in the longitudinal direction.

7.3.9 Simplifications

In this section all simplifications are gathered, which are described in previous sections.

- The bridge modelled was not a full-length bridge. The model contained of 5 spans each with the length of 45 m.

- The effective flange width was calculated according to BBK 04 as 1/10 of the distance between moment zero sections, which approximately is placed at ¼ of the span. The cross-sectional parameters stated in Tables 7.2 are based on this effective area seen in Figure 7.6. One should also be aware of that the thickness of the webs due to the use of external prestressing was chosen on the basis of the interview with Carlsson (2005) and VSL (1992) and is verified in Chapter 6, Section 6.1.2 with a principal drawing. Though this is not in detail designed and should not be considered as finally determined.

- The concrete properties used in the modelling were chosen corresponding to the concrete strength class K40, but the existing bridge used for reference had K400T. This is of no importance in a comparative study.

- The launching prestressing was modelled as one continuous tendon located in the gravity centre of the beam. When using a beam model this is equivalent to a top and bottom placement of the launching prestressing.

- The polygonal prestressing was modelled as continuous tendons and the polygonal profile was the same as in Luleå Bridge. The only difference is that the tendon was adjusted towards the gravity centre in the end zones not to produce any moment over the supports.

- The external prestressing was modelled as external forces acting in the points where the tendon is fixed to the bridge and any elongation was not taken into account.

- Retensioning was carried out of all prestressing tendons to compensate for elastic losses due to successive prestressing.

- The long term effects were estimated to 15 % of the partial prestress for both the alternatives with internal and external tendons. This is not further investigated but should be proper enough for this type of analysis.

- The frictional losses were not included into the FE calculations, since a much more accurate model would have been needed. It is instead included in Chapter 6, Section 6.1.6, where a detailed investigation is performed.

7.4 Stresses

BRIGADE can be used to solve sectional forces for any section analysed as long as certain cross sectional constants such as moment of inertia and sectional rigidity for the beam are provided. From the sectional forces stresses must be calculated manually. These calculations could be quite comprehensive since many different alternatives as well as stresses are to be compared. Therefore a spreadsheet was used for these calculations.

In the spreadsheet Navier's formula was used to calculate the stresses in the top and the bottom fibre of the beam. Moments and shear forces were obtained from BRIGADE for the load cases V:A and V:B. Also the moments induced from the dead weight and from the surfacing as well as for the different prestressing schemes were inserted into the sheet.

7.4.1 Allowable concrete stresses

The allowable stresses in the concrete were determined according to BBK 04 and Bro 2004. The concrete strength class K40 has allowable stresses as shown in Table 7.6. Furthermore, the demand in the construction state is chosen to prevent the bridge from cracks before it is taken into use.

*Table 7.6 Allowable concrete stresses. **) 100 mm radius around the internal prestressing tendon.*

Compression	Tension	
Launching and Service state	Launching state	Service state
f_{cck} = 28,5 MPa	σ_{ct}<0 MPa	LC V:A σ_{uf}< 1,3MPa **
long term; $\lvert\sigma_c\rvert$<0,6·f_{cck}=17,1MPa		LC V:A σ_{lf}< 1,0MPa **
		LC V:B no tension allowed $\sigma_{bending} < 0MPa$

The allowable concrete tension should be fulfilled in a zone of 100 mm radius around the prestressing cable to reach necessary corrosion protection. The position of the tendon in the analysed sections is close to the upper and lower edges of the box. Therefore the stress control was done in the top and the bottom fibre of the box for both load combination V:A and V:B, which is on the safe side. The allowable concrete stresses are different for the top and bottom side of the concrete section in load case V:A, according to Bro 2004. The reason for this is that the concrete surface on the bottom side of the concrete box is unprotected while the asphalt layer protects the top surface of the concrete section.

7.5 FE calculations

As mentioned above the different prestressing alternatives were evaluated in the FE analysis program BRIGADE. The general information is also described in the previous sections. This section will explain the details in each of the alternatives and also present the result from the calculations.

7.5.1 Alternative 1

Alternative 1 consists of internal straight tendons for launching prestressing and internal parabolic tendons for the final prestressing, see Figure 7.9 and Figure 7.10.

In the launching state the prestressing tendons were modelled for full interaction with the surrounding concrete. The tendons were tensioned to 1240 MPa, and the only load that was acting was the dead weight of the bridge girder. The parabolic tendons for the final configuration were also modelled for full interaction and were tensioned to the same value as the straight tendons, 1240 MPa.

Figure 7.9 Schematic figure of how the launching prestressing tendons were modelled

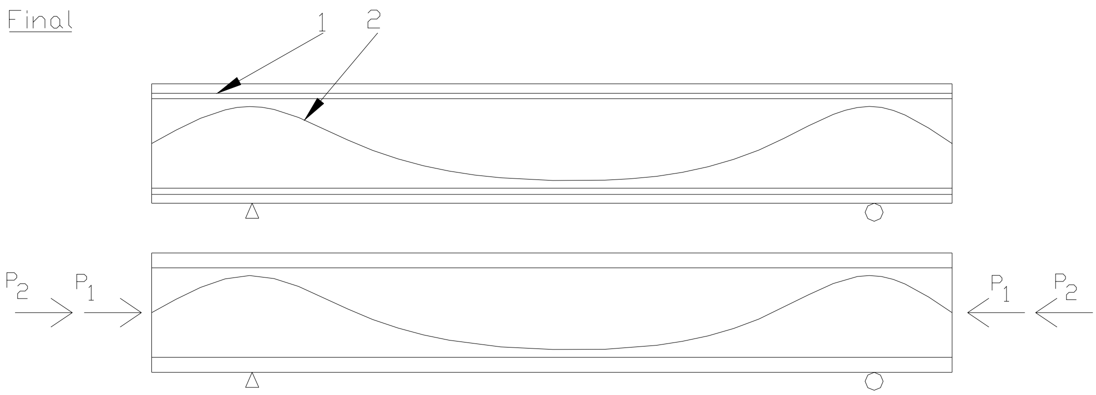


Figure 7.10 Schematic figure of how the final prestressing tendons were modelled

The first analysis that was performed was the launching state, which showed that to obtain only compressive stresses there is a need for launching prestressing according to Table 7.7. In this state only the launching prestressing and the dead weight were present. The final state analysis resulted in a need for parabolic tendons that also is shown in Table 7.7. Figure 7.11 shows the short term bending moment due to dead weight, surfacing load and load from the prestressing. Figure 7.12 shows the axial force due to the same loads as in Figure 7.11. Load combination V:A is also shown to verify the accuracy of the model, see Figure 7.13.

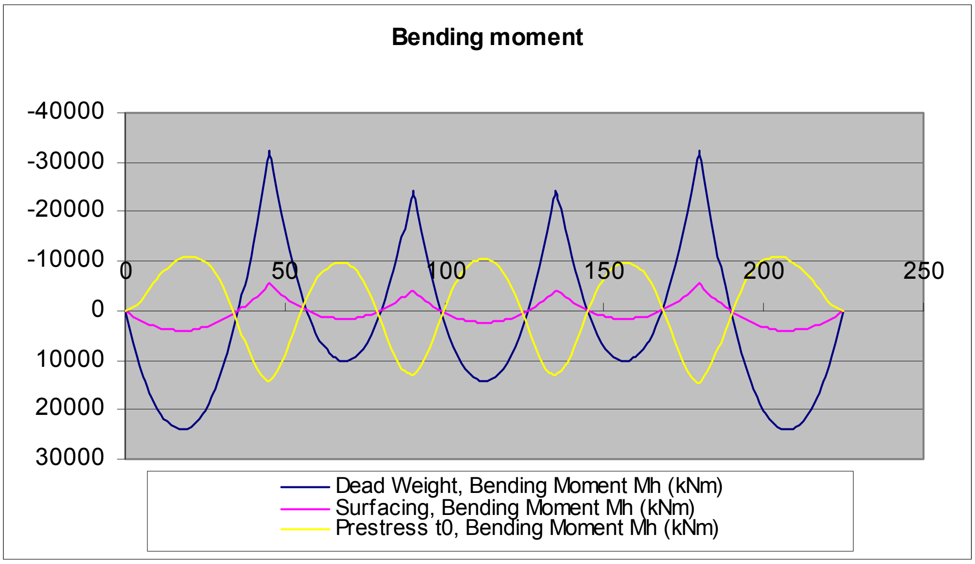


Figure 7.11 Bending moments due to dead weight, surfacing and prestressing

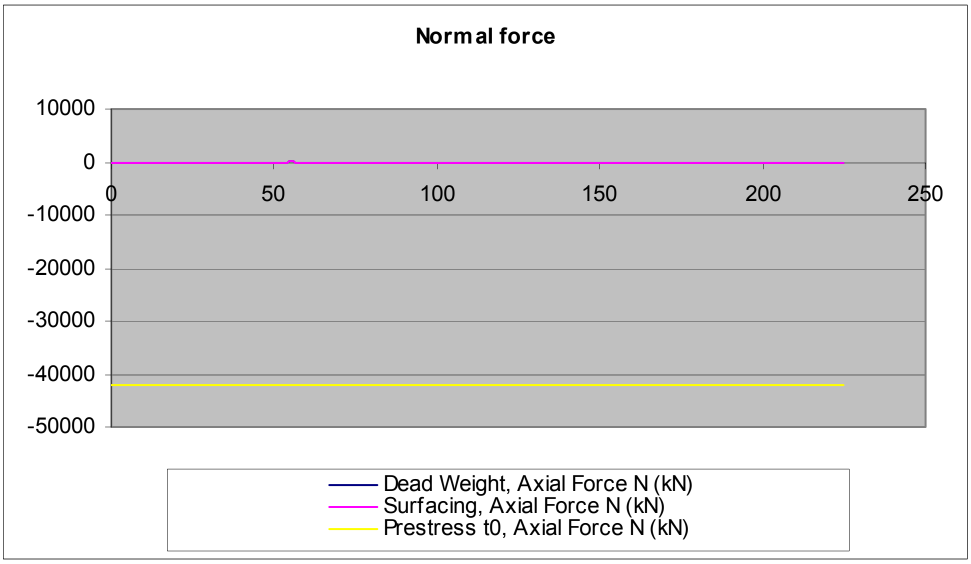


Figure 7.12 Normal force due to dead weight, surfacing and prestressing

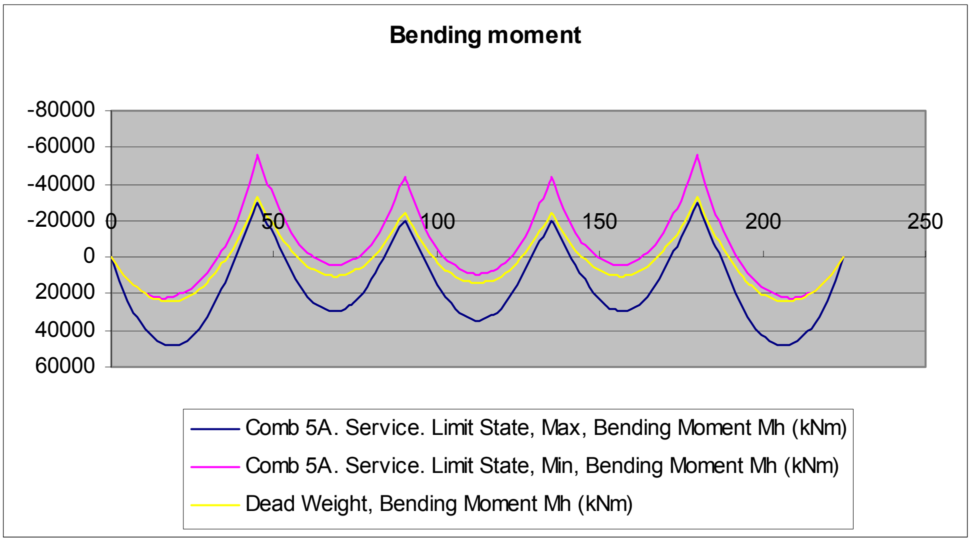


Figure 7.13 Bending moment from service load, dead weight, temperature and surfacing without bending moments from prestressing in load combination V:A

Table 7.7 Required prestressing steel and concrete area for alternative 1

Alternative 1		[m^2]	[kg]
Launching prestress	Internal straight	0,0245	42998
	External straight	0	0
	Parabolic/polygonal and antagonist	0	0
	Total	**0,0245**	**42998**
Final prestress	Internal straight	0,0245	42998
	Internal parabolic	0,0108	18954
	External straight	0	0
	External polygonal	0	0
	External/internal short straight tendons	0	0
	Total	**0,0353**	**61952**
	Wasted tendons	0	0
Total amount of used tendons		**0,0353**	**61952**
Concrete		6,072	

This analysis is as previously mentioned based on the Swedish incrementally launched bridge crossing Lule Älv in Gäddvik. The calculated required amounts of prestressing are slightly higher than for the actual bridge. This result can derive from changes in the code from 1977 to today or the simplifications that were made in this analysis. Still the deviation is not high, only about 10 to 15%, why the result is acceptable.

7.5.2 Alternative 3

Alternative 3 consists of internal straight launching tendons and external polygonal tendons for final prestressing. The launching state was modelled in the same way as for alternative 1, but the reduction of the cross section that was made resulted in a 12 % less need of prestressing steel. This reduction was found through an iteration process where new prestressing areas and their stress contribution were calculated. The results are shown in Table 7.8, and in Figure 7.16, Figure 7.17 and Figure 7.18.

The external prestressing in the final state was modelled as point loads acting in the position of the deviators. These forces were calculated by the assumption that the tendon force is the same before and after the deviator, and therefore simple equilibrium equations can be used. The principles of these calculations can be seen in Figure 7.15, and a schematic presentation of how external prestressing tendons in the final state are modelled is shown in Figure 7.14.

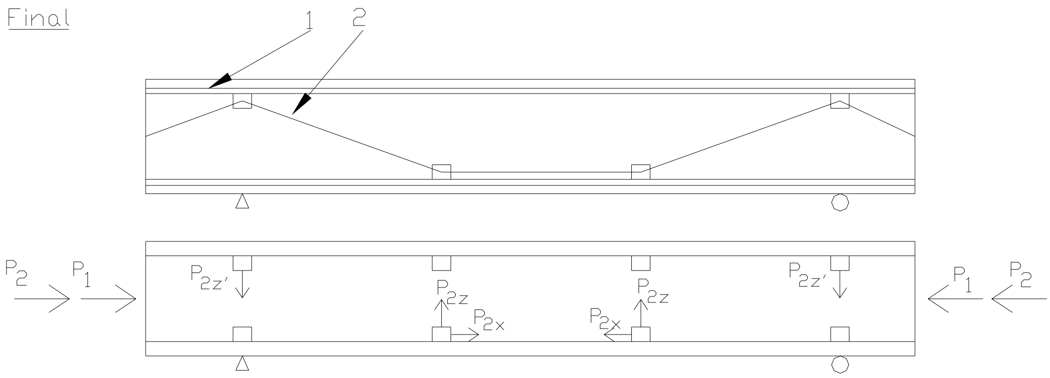


Figure 7.14 Schematic presentation of how alternative 3 was modelled.

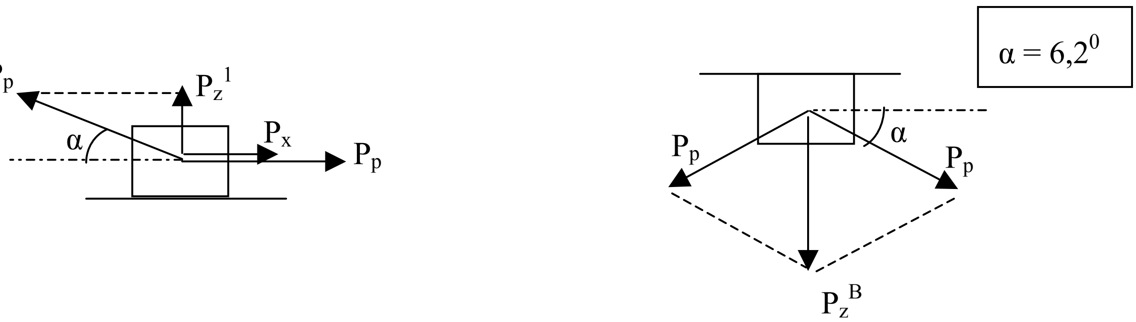


Figure 7.15 Principle of calculation of application forces in case of external prestressing

The first amount of polygonal prestressing tendons that was tried was the same area as for alternative 1, 0,0108m^2. With a stress of 1240 MPa in the tendons this gave the following forces:

$$P_p = 13392\text{kN}$$

$$P_Z^1 = 1446\text{kN}$$

$$P_Z^B = 2893\text{kN}$$

$$P_X = 78\text{kN}$$

The analysis showed that the compressive stresses obtained from this prestressing were lower than allowed. In load combination V:A there was some tension in the bottom fibre, which corresponded to a prestressing incensement of 5 %. This gave new reduced amount of prestressing steel as shown in Table 7.8.

Table 7.8 Required prestressing steel and concrete area for alternative 3

Alternative 3		[m^2]	[kg]	[%] of Alternative 1
Launching prestress	Internal straight	0,02156	37838	
	External straight	0	0	
	Parabolic/polygonal and antagonist	0	0	
	Total	**0,02156**	**37838**	**-12,0**
Final prestress	Internal straight	0,02156	37838	
	Internal parabolic	0	0	
	External straight	0	0	
	External polygonal	0,01139	19989	
	External/internal short straight tendons	0	0	
	Total	**0,03295**	**57827**	**-4,1**
	Wasted tendons	0	0	
Total amount of used tendons		**0,03295**	**57827**	**-4,1**
Concrete		5,359		**-11,7**

Figure 7.16 shows the short term bending moment due to dead weight, surfacing load and load from the prestressing. In Figure 7.17 the axial force due to the same loads as in Figure 7.16. Load combination V:A is also shown to verify the accuracy in the model, see Figure 7.18.

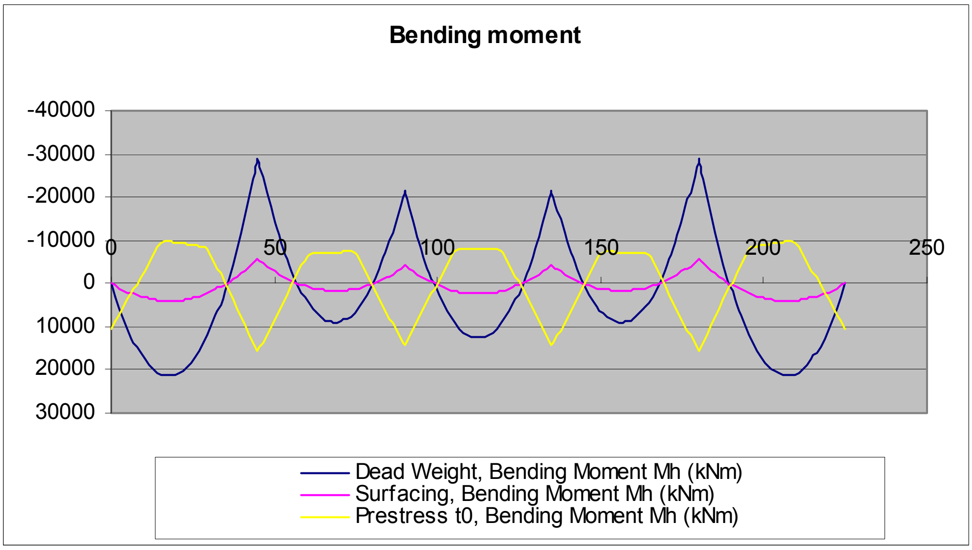


Figure 7.16 Bending moments due to dead weight, surfacing and prestressing

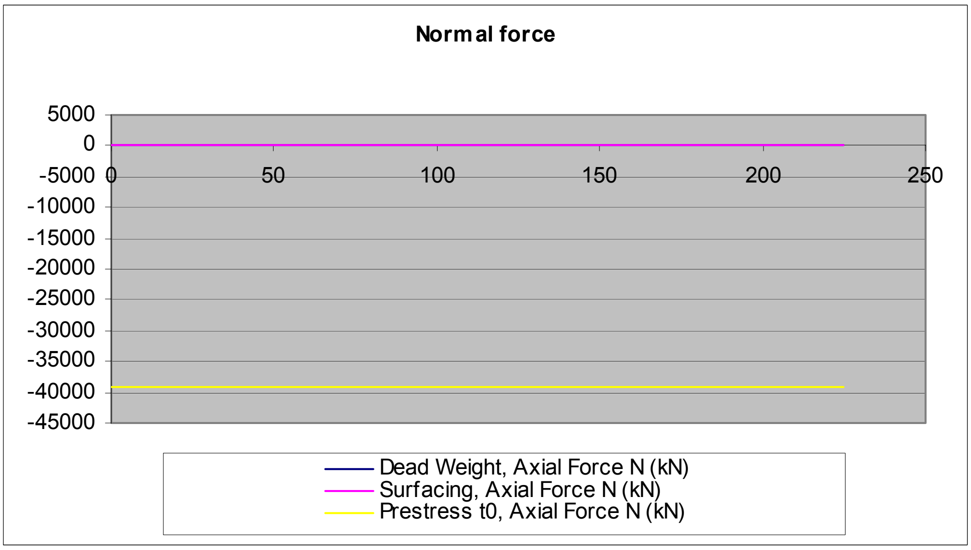


Figure 7.17 Normal force due to dead weight, surfacing and prestressing

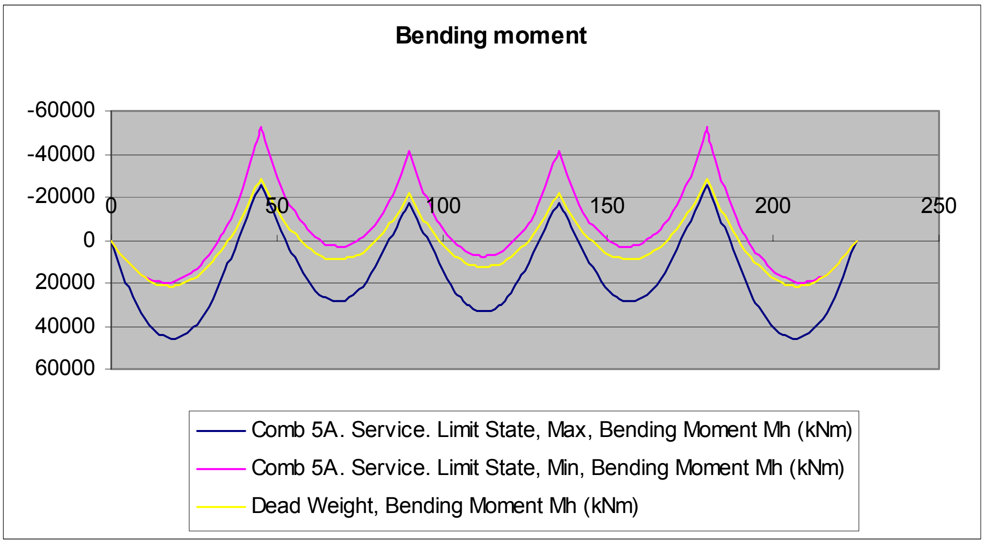


Figure 7.18 Bending moment from service load, dead weight, temperature and surfacing without bending moments from prestressing in load combination V:A

7.5.3 Alternative 8

This alternative includes only external prestressing. This effect the way that the stresses are checked as mentioned in the Chapter 7, Section 7.4. As there is no internal prestressing tendon there is no need to check stresses for load case V:A. The check of crack widths is as for the other bridges done for load case V:B.

This alternative is modelled as shown in Figure 7.19.

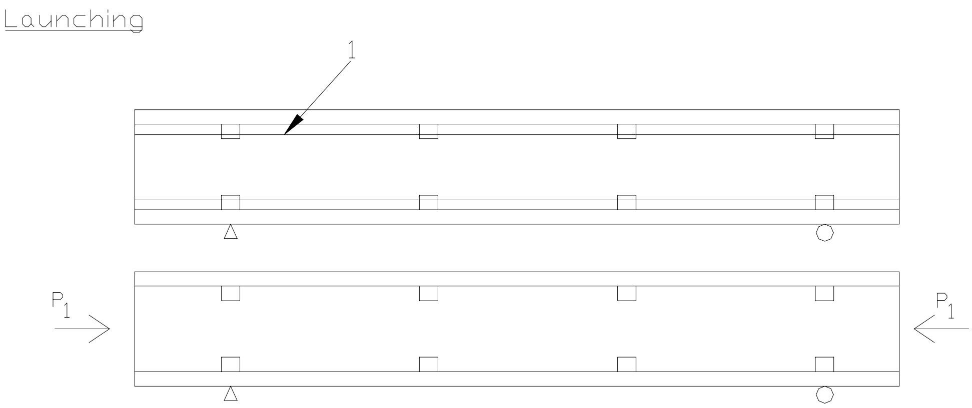


Figure 7.19 Schematic figure of how the launching prestressing is modelled in alternative 8

The result from this analysis showed that the need for launching prestress was the same as in alternative 3, which was quite expected.

To analyse the final state the same model as for alternative 3 was used. See Figure 7.14. However, when the results from this analysis were studied, the conclusion was that the launching prestressing almost could handle the tensile stresses. Therefore the idea of using polygonal tendons was disregarded. Instead the launching prestressing was increased to handle the load form temperature and surfacing, which was the only additional loads from the dead weight in load combination V:B. The amount of launching prestress needed was 9 % more than for the launching state, and can be found in Table 7.9.

Table 7.9 Required prestressing steel and concrete area for alternative 8 (first analysis - only straight external tendons)

Alternative 8 (first analysis)		$[m^2]$	kg	[%] of Alternative 1
Launching prestress	Internal straight	0	0	
	External straight	0,02156	37838	
	Parabolic/polygonal and antagonist	0	0	
	Total	**0,02156**	**37838**	**-12,0**
Final prestress	Internal straight	0	0	
	Internal parabolic	0	0	
	External straight	0,02156	41330	
	External polygonal	0	0	
	External/internal short straight tendons	0	0	
	Total	**0,02355**	**41330**	**-31,4**
	Wasted tendons	0	0	
Total amount of used tendons		**0,02355**	**41330**	**-31,4**
Concrete		5,359		**-11,7**

The fact that the launching prestressing almost is enough to avoid tensile stress for load combination V:B there is no need for additional prestressing related to cracking. On the other hand ultimate limit state also needs to be considered in a total analysis. If no additional prestressing is provided the remaining need to control stresses for the analysed section has to be taken care of by ordinary reinforcement. It could be hard to reduce the tensile stresses of 4,7 MPa as tension in the bottom fibre by adding ordinary reinforcement only. See Table 7.13.

Another thing that has to be evaluated is whether there is a good solution to take all shear force by reinforcement, since there is no contribution from the prestressing, except from the normal stresses, which prevents shear cracking.

Since there is no possibility to draw any conclusions whether this alternative is more or less economical than the other alternatives, the final state prestressing was modelled and analysed in the same way as for the other alternatives. To make a just comparison the same allowable stresses as for internal prestressing was used.

The result from this analysis was that for the launching state there was enough with 0,02156 m^2 of straight external prestressing. For the final state there was an additional need of 0,01139 m^2 of external polygonal tendons. Polygonal tendons were used due to that alternative 8 was prescribed with polygonal because of the shear capacity. This is the same needs as for alternative 3, which of course is quite easy to understand. The final prestressing is of the same kind, and the launching prestressing need to provide as much compression as in alternative 3. Production matters however are different. But that is included in the overall analysis. Table 7.10 shows the required amounts of materials.

Table 7.10 Required prestressing steel and concrete area for alternative 8 (second analysis - straight and polygonal tendons)

Alternative 8 (second analysis)		[m^2]	kg	[%] of Alternative 1
Launching prestress	Internal straight	0	0	
	External straight	0,02156	37838	
	Parabolic/polygonal and antagonist	0	0	
	Total	**0,02156**	**37838**	**-12,0**
Final prestress	Internal straight	0	0	
	Internal parabolic	0	0	
	External straight	0,02156	37838	
	External polygonal	0,01139	19989	
	External/internal short straight tendons	0	0	
	Total	**0,03295**	**57827**	**-4,1**
	Wasted tendons	0	0	
Total amount of used tendons		**0,03295**	**57827**	**-4,1**
Concrete		5,359		**-11,7**

Short term bending moment diagram due to dead weight, surfacing load and load from the prestressing and normal force diagram due to the same loads are found in Figure 7.20 and Figure 7.21. Figure 7.22 shows load combination V:A to verify the accuracy in the model.

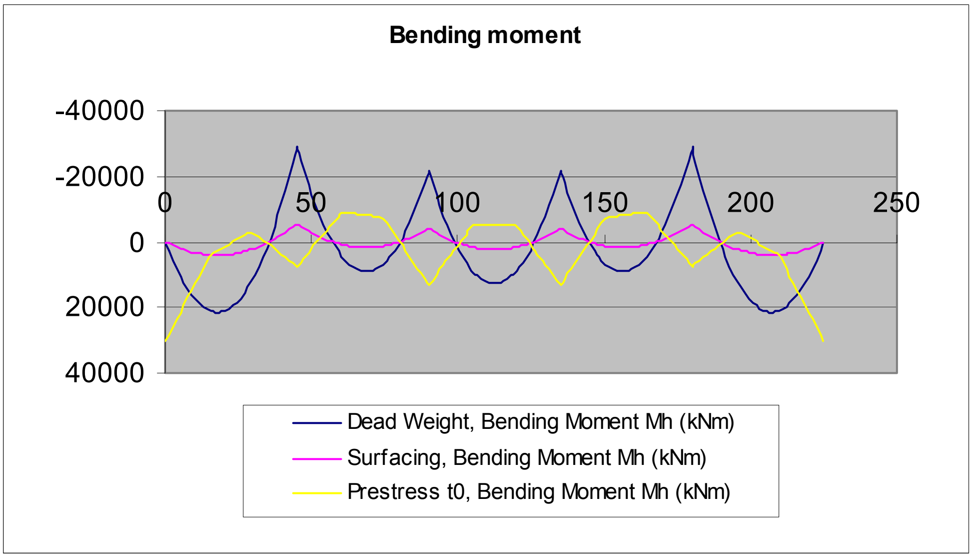


Figure 7.20 Bending moments due to dead weight, surfacing and prestressing

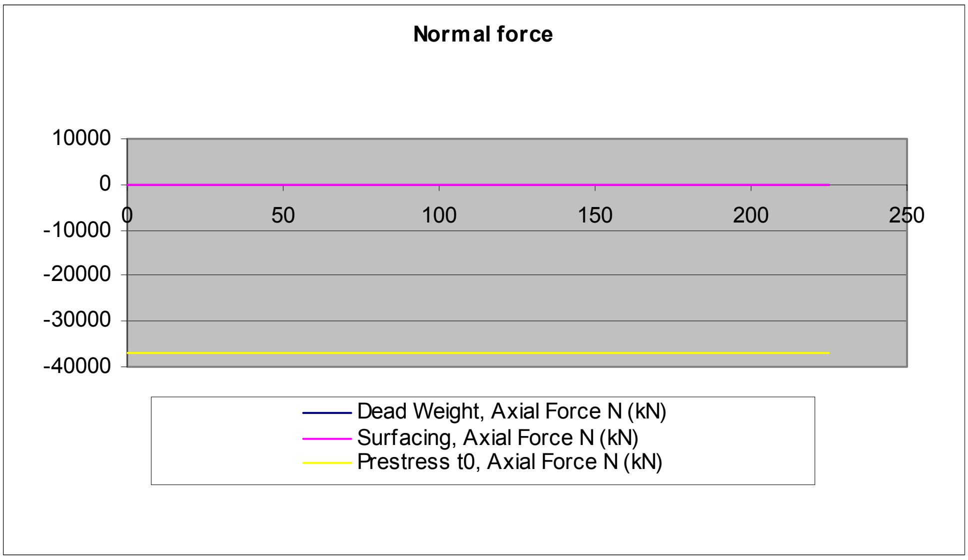


Figure 7.21 Normal force due to dead weight, surfacing and prestressing

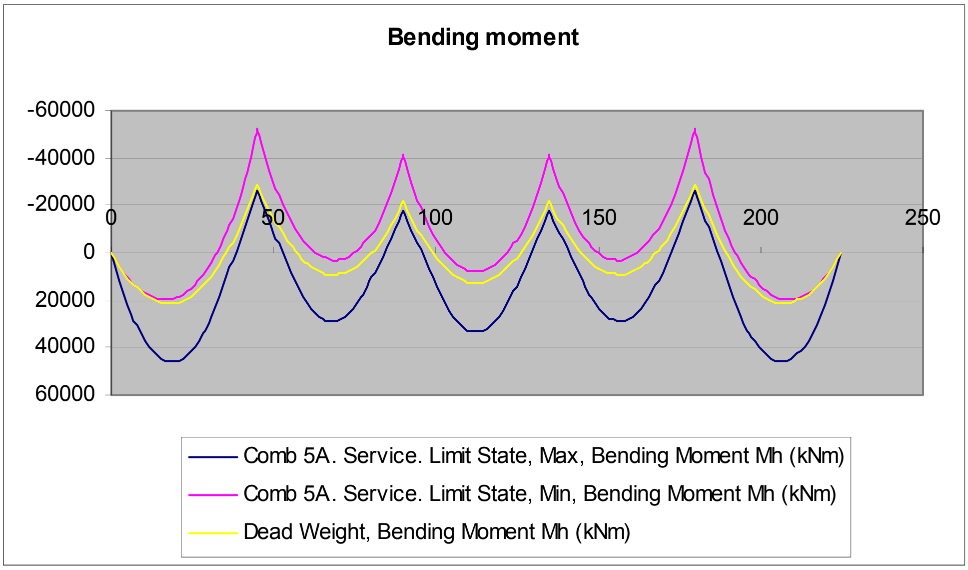


Figure 7.22 Bending moment from service load, dead weight, temperature and surfacing without bending moments from prestressing in load combination V:A

7.5.4 Alternative 9

Alternative 9 is the most complicated alternative. Here the launching prestress is produced by the usage of straight internal tendons, straight external tendons, polygonal external tendons and antagonist external polygonal tendons. First of all the same amount of launching prestressing as in alternative 3 was tested. These alternatives are quite similar structurally, though the ways they are constructed differ rather much, so it will be quite a good start value.

One of the advantages with this alternative is that the launching prestressing can be detensioned and therefore one of the goals is to place enough launching prestress externally. The polygonal external prestress that is used in the launching state will not be removed in the final state, therefore another goal is to place rather much launching prestress as external polygonal. One negative effect is that the antagonist tendons need to be tensioned as much as the polygonal, which can create too high compressive stresses.

The straight internal tendons were modelled to have full interaction and tensioned to 1240 MPa, and the straight external tendons were modelled as external normal forces. The modelling of the polygonal tendons and the antagonist tendons demanded quite a comprehensive model. Therefore only the resulting prestressing force was applied as an external normal force. This simplification is rather good since the force couples will cancel each other out except for the small horizontal force contribution that is created between the middle deviators. Though this is considered as a very small approximation, and the benefits of the simplified model are quite large. The launching state was modelled as shown in Table 7.11 and Figure 7.23.

Table 7.11 Required prestressing steel area for the launching state

Launching prestress	[m^2]	Total amount
Straight internal	0,00511	0,02156 m^2
Straight external	0,00511	
Polygonal and antagonist	0,01134	

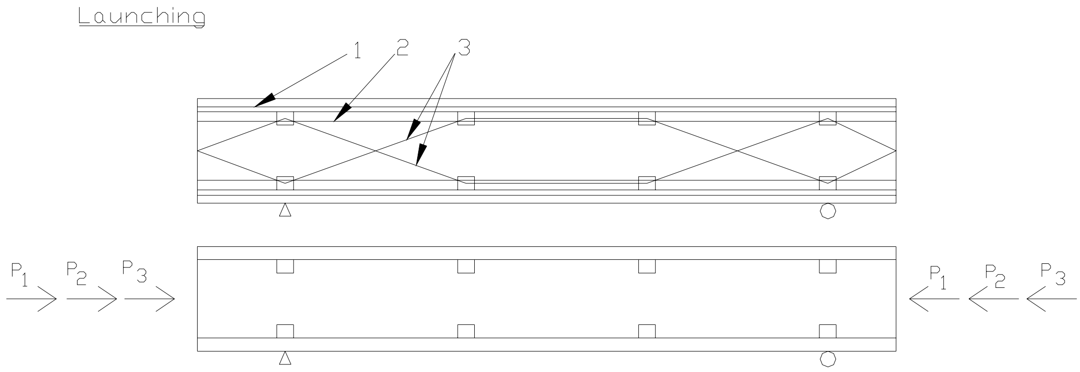


Figure 7.23 Schematic figure of how the launching prestressing tendons were modelled in alternative 9

The result from this analysis showed that the provided prestressing was enough and there was no tension in the structure during launching.

The final prestressing is more straightforward than the launching prestress. The polygonal tendons are still there, as the internal straight tendons. The external straight tendons are removed in all positions other than at the lower flange in the mid span. These tendons are, as previously mentioned, there to handle excessive bending moments that occur due to long spans. Since the spans are quite short in this analysis the short straight tendons do not provide as much efficiency as when the spans are long. Still they are kept since their effect is of interest. The antagonist tendons are also removed and rearranged in the polygonal tendon shape.

The first test was done with the same prestressing steel area as in alternative 3, 0,01134 m^2. This was of course not enough prestressing, since only about one fourth of the launching prestressing still is active. However it was a good start value. The structure was modelled as described in Figure 7.24.

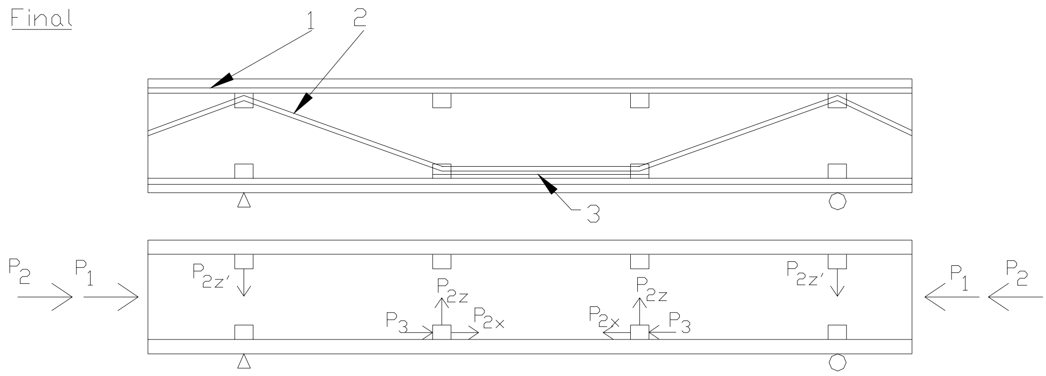


Figure 7.24 Schematic presentation of how alternative 9 was modelled for the final state.

The forces acting on the structure were as follows:

$P_2 = 14062\,\text{kN}$

$P_{2Z^1} = 3037\,\text{kN}$

$P_Z = 1519\,\text{kN}$

$P_X = 82\,\text{kN}$

$P_3 = 3168\,\text{kN}$

The result from this analysis was that there was not enough prestressing to limit the concrete stress to 1,0 MPa in the bottom fibre in the span. Therefore, an increase of the polygonal tendons was needed. An iteration process was started, and the final need of polygonal tendons was proved to be 0,0190 m^2, almost 1,6 times the prestressing steel area in the first run. The forces that correspond to this area can be found below, but then the straight external tendons in the bottom of the box in the span were not increased. This is due to the fact that there was too much tension in the top fibre at the supports as well, and that there must be a possibility to increase the number of tendons in this section when the span length increases.

$P_2 = 26500\,\text{kN}$

$P_{2Z^1} = 5724\,\text{kN}$

$P_Z = 2862\,\text{kN}$

$P_X = 155\,\text{kN}$

$P_3 = 3168\,\text{kN}$

The required amounts of prestressing steel are presented in Table 7.12.

Table 7.12 Required prestressing steel and concrete area in alternative 9

Alternative 9		$[m^2]$	kg	[%] of Alternative 1
Launching prestress	Internal straight	0,00511	8968	
	External straight	0,00511	8968	
	Parabolic/polygonal and antagonist	0,01134	19902	
	Total	**0,02156**	**37838**	**-12,0**
Final prestress	Internal straight	0,00511	8968	
	Internal parabolic	0	0	
	External straight	0	0	
	External polygonal	0,019	33345	
	External/internal short straight tendons	0,002555	1992,9	
	Total	**0,026665**	**44306**	**-26,5**
	Wasted tendons	0,01078	18918,9	
Total amount of used tendons		**0,037445**	**63225**	**4,9**
Concrete		5,359		**-11,7**

When the result is studied one obvious fact can be seen. The alternative provides a better structural behaviour both in the launching and the final state. This is seen if the amount of used launching prestressing steel and final prestressing steel is studied. Compared to alternative 1 this is far less, but still the overall usage of tendons is higher than in alternative 1, and therefore higher than all the other alternatives as well. This is a quite interesting result, which indicate that this alternative can be a successful one, once the span length increases. The alternative is more effective and the possibility to add straight tendons in the areas where the are moment peaks offers a great advantage.

Figure 7.25 shows the short term bending moment due to dead weight, surfacing load and load from the prestressing. Figure 7.26 the axial force due to the same loads as in Figure 7.25. Load combination V:A is also shown to verify the accuracy in the model, see Figure 7.27.

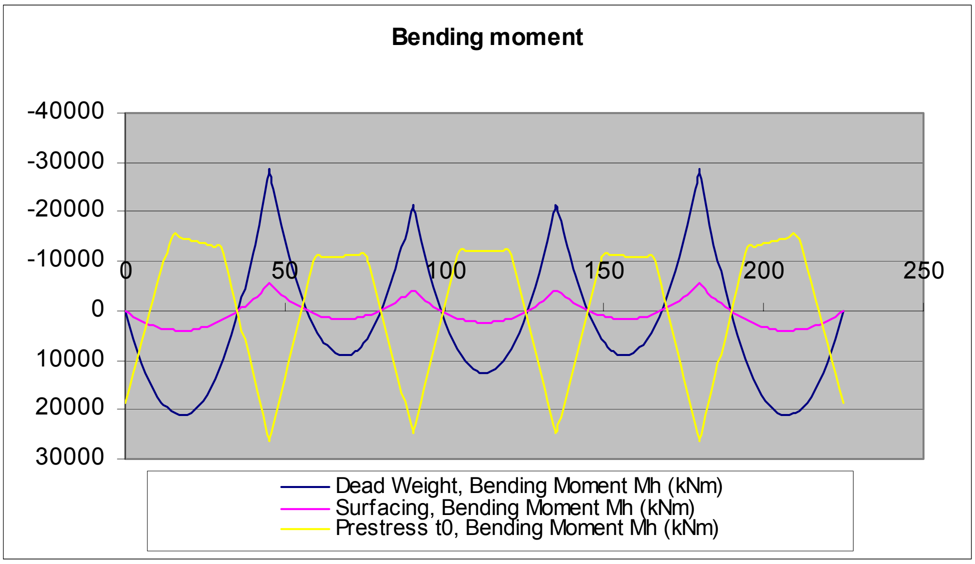


Figure 7.25   Bending moments due to dead weight, surfacing and prestressing

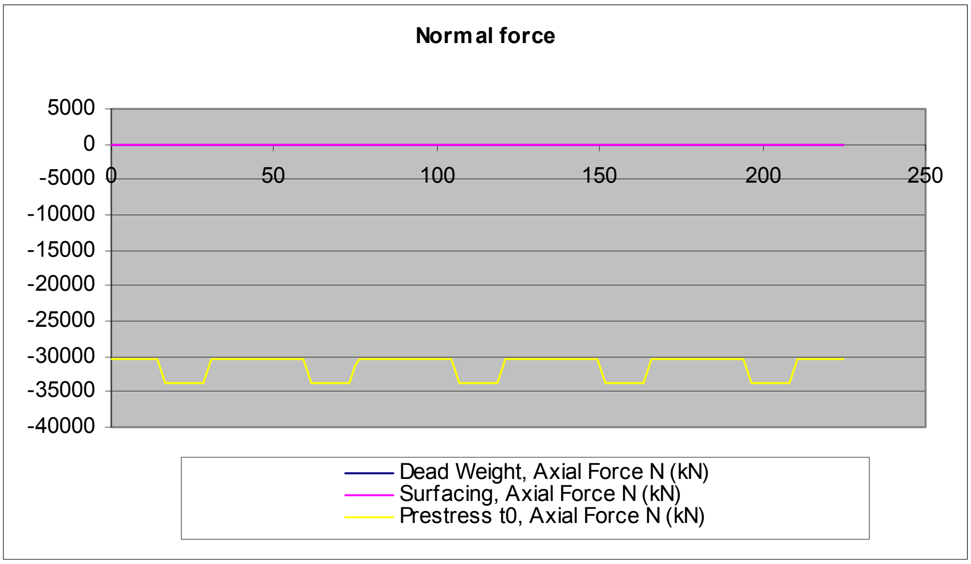


Figure 7.26   Normal force due to dead weight, surfacing and prestressing

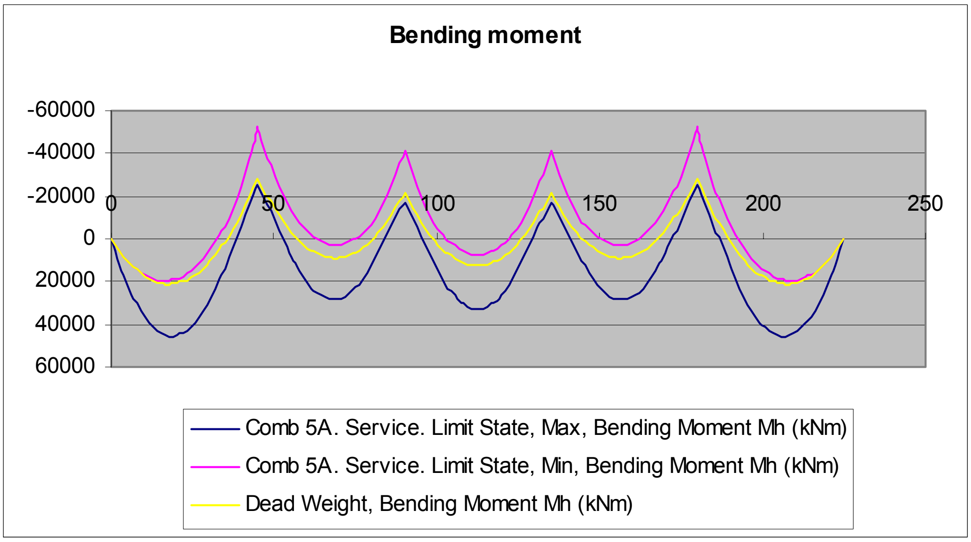


Figure 7.27 Bending moment from service load, dead weight, temperature and surfacing without bending moments from prestressing in load combination V:A

7.6 Results from the FE calculations

To compare the four different alternatives several aspects need to be considered. The structural behaviour is one of these aspects, and that comparison is the subject of this section. The concrete stresses, calculated from the different alternative, are presented in Table 7.13. The delta variable shows the difference compared with alternative 1, which is the reference alternative. However these stresses are only to control that the prestressing provides acceptable stresses, the comparison is made on the prestressing steel area.

Table 7.13 Calculated maximum stresses in the concrete upper and lower flanges at the support section and span section for the different alternatives

			Launching		V:A long term		V:A		V:B	
Alternative 1	Support	Uf	0,0		-0,1		-1,6		-3,7	
		Lf	-11,9		-14,4		-14,9		-11,7	
	Field	Uf	-7,7		-11,0		-11,7		-8,7	
		Lf	-0,8		0,9		-0,5		-4,5	
				delta		delta		delta		delta
Alternative 3	Support	Uf	0,0	0,0	-1,0	-0,9	-2,5	-0,9	-4,8	-1,1
		Lf	-10,5	1,4	-14,0	0,4	-14,4	0,5	-11,0	0,7
	Field	Uf	-6,8	0,9	-11,6	-0,6	-12,4	-0,7	-9,4	-0,7
		Lf	-0,7	0,1	1,0	0,1	-0,3	0,2	-4,5	0,0
Alternative 8	Support	Uf	-0,5	-0,5	3,1	3,2	2,3	3,9	0,0	3,7
(first analysis)		Lf	-11,0	0,9	-16,1	-1,7	-16,9	-2,0	-13,4	-1,7
	Field	Uf	-7,3	0,4	-11,3	-0,3	-12,1	-0,4	-9,1	-0,4
		Lf	-1,2	-0,4	4,7	3,8	3,9	4,4	-0,3	4,2
Alternative 8	Support	Uf	0,0	0,0	-1,5	-1,4	-3,1	-1,5	-5,4	-1,7
(second analysis)		Lf	-10,5	1,4	-13,5	0,9	-13,8	1,1	-10,4	1,3
	Field	Uf	-6,8	0,9	-11,7	-0,7	-12,6	-0,9	-9,6	-0,9
		Lf	-0,7	0,1	1,0	0,1	-0,3	0,2	-4,5	0,0
Alternative 9	Support	Uf	-0,1	-0,1	-1,4	-1,3	-3,0	-1,4	-5,2	-1,5
		Lf	-10,5	1,4	-10,0	4,4	-9,7	5,2	-6,3	5,4
	Field	Uf	-6,9	0,8	-10,0	1,0	-10,5	1,2	-7,5	1,2
		Lf	-0,8	0,0	1,0	0,1	-0,5	0,0	-4,7	-0,2

If the amounts of used prestressing steel are studied alternative 3 and alternative 8 are the most profitable. They not only need less prestressing steel than the two other alternatives, they also use less concrete than alternative 1. In Table 7.14 the differences between the used amounts of prestressing steel can be seen.

Table 7.14 Used prestressing steel amount in the examined alternatives

Total amount of used tendons (including wasted amount of cables)	$[m^2]$	kg	Compared to Alternative 1 [%]
Alternative 1	0,03435	60284	0
Alternative 3	0,03295	57827	- 4,1
Alternative 8	0,03295	57827	- 4,1
Alternative 9	0,03745	63225	+ 4,9

When the launching state is studied the differences between the alternatives are not significant other than that alternative 1 needs 12% more launching prestressing than the other alternatives. This is of course due to the fact that alternative 3, 8 and 9 have 11,6 % lesser concrete area than alternative 1.

Another interesting aspect in the comparison is the amount of steel used the final state. This gives an indication of how efficient the prestressing arrangement is. If one alternative can use less prestressing to fulfil the stress limits, this alternative is of course more effective. In Table 7.15 the amounts of prestressing steel in the final state are presented.

Table 7.15 Prestressing steel needed in the final state. Alternative 9 requires a smaller amount of prestressing than the other alternatives.

Prestressing in the final state	$[m^2]$	kg	[%] compared to Alternative 1
Alternative 1	0,03435	60284	0
Alternative 3	0,03295	57827	- 4,1
Alternative 8	0,03295	57827	- 4,1
Alternative 9	0,02667	44306	- 26,5

7.6.1 Discussion

The obvious fact from this result is that the prestressing arrangement in alternative 9 seems to be a more effective in the final state than the other alternatives, though this alternative uses more prestressing steel overall. This is due to the detensioning of the launching prestressing when the bridge is placed in its final position. Still the total amount of used steel is higher, and that is of course important to remember when designing a short to medium span bridge such as this. If the bridge would have longer spans the results might have been other, and alternative 9 could with its effectiveness be proved to be the most economical, and even possible, solution.

Concluding that alternative 3 and alternative 8 are the most economical solutions, regarding material, for a bridge with 45 m spans. With longer spans, however, alternative 9 could be more economical due to its effectiveness in the final state. In really long spans there might be no possibility to use alternative 1, 3 and 8, since they offer no possibility to detension tendons, which could cause too high compression in the flanges. Then alternative 9 would be the only possible solution, and because of this it must be considered as a needed alternative. Though it might be cumbersome to produce, and totally it uses more prestressing steel for short to medium spans compared to the other alternatives analysed.

The result from the FE analysis is that alternative 3 and alternative 8 are the most economical alternatives. Alternative 9 offers an effective solution that can be valuable in bridges with long spans, but in short to medium span bridges alternative 9 is not an economical solution.

7.7 Prestressing conclusions

Investigations have been performed to distinguish the most effective and economical way to arrange prestressing in an IL bridge. The scope of the analysis has been both the launching prestressing required to handle the changing positive and negative moment during launching, and the final prestressing.

In Chapter 6 a general analysis regarding production aspects, corrosion protection and durability and at last some general aspects is presented. This analysis resulted in four promising prestressing schemes, which later were subjected to a Finite Element analysis. The aim of the FE analysis was to study the structural effects of the four promising alternatives. The conclusions made in this section are based on the assumptions and analyses made in Chapter 5, Chapter 6 and Chapter 7, and are only valid for the medium span range, 40 to 60 m.

From the first general analysis the Swedish alternative, alternative 1, seemed to be the most appealing alternative. However, this alternative required quite a lot of prestressing, especially in the launching state, due to its heavy cross section that results from the usage of internal prestressing in the web. Therefore the first conclusion is that alternative 1 is not the most material saving solution. Nevertheless, if the use of external prestressing is prohibited, alternative 1 can be a good solution, since it is a well-known solution and has a quite straightforward production method.

The FE-analysis shows that alternative 3 and alternative 8 are the most effective solutions when designing short to medium span bridges. In the first run of FE analyses alternative 8 was considerably better than all the other alternatives, though this result was a consequence of an interpretation of the Swedish code Bro 2004. Since the demands are significantly different when the code is interpreted in this way the comparison is unfair. Besides, this alternative needs to be reinforced to reduce the flexural stress in the mid span with about 6 MPa. Further on, the alternative provides no possibility to prevent shear cracks since there are no inclined tendons, why the section has to be provided with comprehensive shear reinforcement. We have disregarded this result, since there was not enough time in this project. To analyse if it is better to exclude the polygonal prestressing tendons and take shear forces with ordinary reinforcement or include the polygonal tendons is not within the scope of work. The goal is to compare the different prestressing alternatives, why only the second analysis of alternative 8 will be discussed. This is simply because it is the only analysis that can be compared as mentioned above.

Alternative 3 and alternative 8 were the alternatives that required the least amounts of prestressing steel, combined with a smaller concrete cross section. If the general analysis is included in the comparison of these two alternatives a difference occurs however. Alternative 3 is then the more interesting alternative, since it provides a combination of internal and external prestressing, which results in several advantages. Not only the durability is better but also the eccentricity of the launching tendons. Alternative 3 also provides another advantage since the number of deviators that is needed is less. Alternative 8 requires deviators at each segment to enable the usage of launching prestressing, whilst alternative 3 only needs a few deviators. Therefore our recommendation is that alternative 3 should be preferred in IL bridges with medium span length, 40-60 m.

If the span would increase alternative 9 provides several advantages however, since it provides a possibility to detension tendons that would have a negative influence in the final state. For that reason we recommend that alternative 9 should be analysed further, if the span increase up to 70, 80 or even 90 m.

Also the possibility to provide straight tendons at the regions with peak moments should also be considered for alternative 3, if the span increases over the medium range. This means that alternative 3 could be competitive even for spans more than 60 m. However, this is not included in the scope of work.

Finally we can conclude that alternative 3 is the most appropriate and material saving solution, when designing IL bridges in the medium span range. If the usage of external prestressing is prohibited alternative 1 should be used. Furthermore, if the span length increases over the medium span range alternative 9 or alternative 3 with extra tendons in regions with peaks moments should be analysed.

8 Site Visit, Rybny Potok

In the beginning of October a study trip was conducted to acquire valuable practical information and if possible verify some theoretical statements. The destination for the study trip was the Rybny Potok Bridge located 200 kilometres north of Prague in the Czech Republic. The study trip was arranged through Karl-Erik Nilsson, project engineer at Internordisk Spännarmering, and his contacts with the VSL group. Figure 8.1 shows a view of the bridge.

8.1 General bridge information

The main contractor for the Rybny Potok Bridge project is Skanska however a subcontractor, Metrostav, constructs the bridge. The prestressing and the launching system were performed by VSL. VSL is a worldwide company specialised in prestressing systems and has long experience in the IL method. The bridge has been designed by SHB, a design company with office in Prague.

Figure 8.1 Front view of the Rybny Potok Bridge under construction.

8.2 General layout and Geometry

The bridge consists of seven spans where the spans are in the upper medium range, see Figure 8.2. The total length of the bridge is 356 m and it is a part of the new Highway D8, which goes from Prague heading north towards Dresden in Germany. The superstructure of the bridge is a concrete box girder that was incrementally launched out over the on forehand built piers. Noteworthy for the superstructure is the width of the bridge deck, 30 m, which makes the structure rather heavy. The height of the box girder is 4,2 m, and as seen in Figure 8.2, the average span is 58 m, why the span to depth ratio is 14. To be able to transfer the load from the wide bridge deck transversally into the box girder, inclined prefabricated struts were adopted every third meter. To further increase the structural performance, the bridge deck is prestressed in the transversal direction. The section of the superstructure can be seen in Figure 8.3.

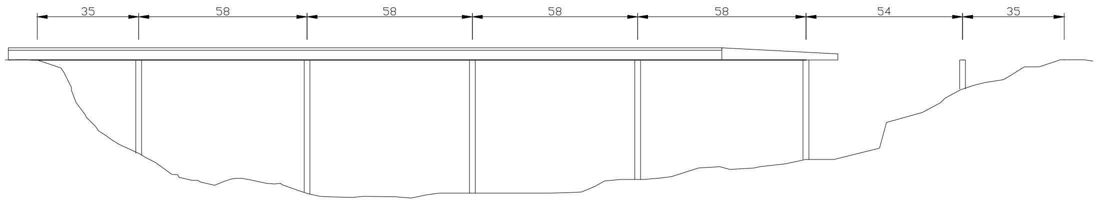


Figure 8.2 Elevation of the Rybny Potok Bridge

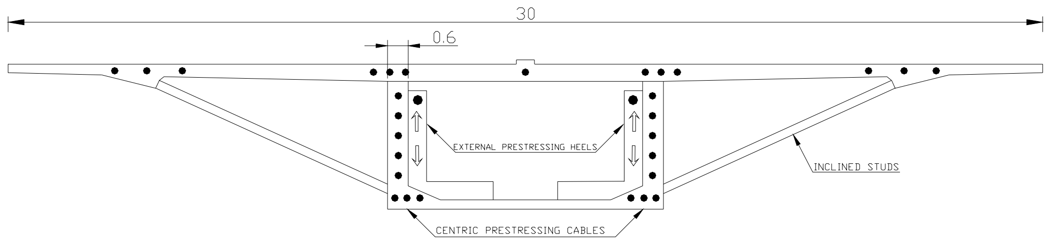


Figure 8.3 Bridge deck section of the Rybny Potok Bridge

The bridge has a reinforcement content of 250 kg/m^3, which is a rather high value but reasonable since it is a relatively slimmed structure compared to the large width. The concrete used in the bridge corresponds to strength class K45, according to the Swedish code, and has a density of 28,5 kN/m^3 since the aggregates are of rather heavy material. The area of the cross section is 18 m^2 why the total weight of the launched structured is 19 000 tons.

8.3 Prestressing

The prestressing arrangement for the bridge is similar to alternative 3 from the prestressing schemes. For the launching state straight internal prestressing tendons were used that were coupled at each second segment joint. In Figure 8.4 the tendons used for the launching prestressing can be seen. Half of the tendons were anchored in the segment joint and the other half continued to be cast into the next segment. According to the designer, external tendons were not considered for the launching prestressing since it is difficult to obtain sufficient resistance if only external tendons are used.

Figure 8.4 Rybny Potok Bridge section. Notice the tensioned prestressing anchorages and the continuous tendons that are to be tensioned after the completion of the next segment.

The final arrangement has external polygonal tendons working together with the straight internal ones. In Figure 8.5 the general prestressing arrangement is presented. The external tendons for the final state are tensioned after launching. Also the casting of the concrete deviators was made after the launching, to make the working process more rapid and smooth. Figure 8.6 shows the reinforcement for a deviator. The external polygonal tendons for the final state were chosen, not only due to their favourable structural behaviour regarding the friction losses, but also because of production aspects. According to Bešta and Smíšek (2005) the external polygonal tendons require less time to be installed compared to internal ones. Further on, the work with installing tendons is postponed to the phase when the bridge is finally launched. Also the fact that it would have been hard to find any suitable space for the tendons in the concrete section makes external tendons even more favourable. If internal tendons had been used, the concrete section would have had to be thickened and thereby the dead weight would have increased.

The launching prestress consist of two different tendon types and the final external prestressing is another tendon type. The tendons used can be seen in Table 8.1.

Table 8.1 Tendons used in the Rybny Potok Bridge

	Type	Number of strands	Diameter of strands
Launching tendons	Internal	7	16
	Internal	12	16
Final tendons	External	31	16

The similarities to the prestressing arrangement alternative 3 are several but there are also some differences. One thing is the position of the tendons for the launching state. In alternative 3, described in Chapter 5, Section 5.2.3, and in the bridges built by IL in Sweden, as well as in the literature from Rosignoli (2002) and Göhler Pearson (2000), these launching tendons are positioned in the top and bottom flanges. The advantages when having the tendons in the flanges are both that the distance to the gravity centre is longer and that it is possible to reduce the thickness of the web. Though the web thickness should still be sufficient to fulfil the structural demands such as shear resistance. Since the Rybny Potok Bridge is heavy, the web has to be rather thick due to the needed shear resistance. Therefore, it was also preferable to place some of the tendons for the launching state there since the flanges are quite slimmed.

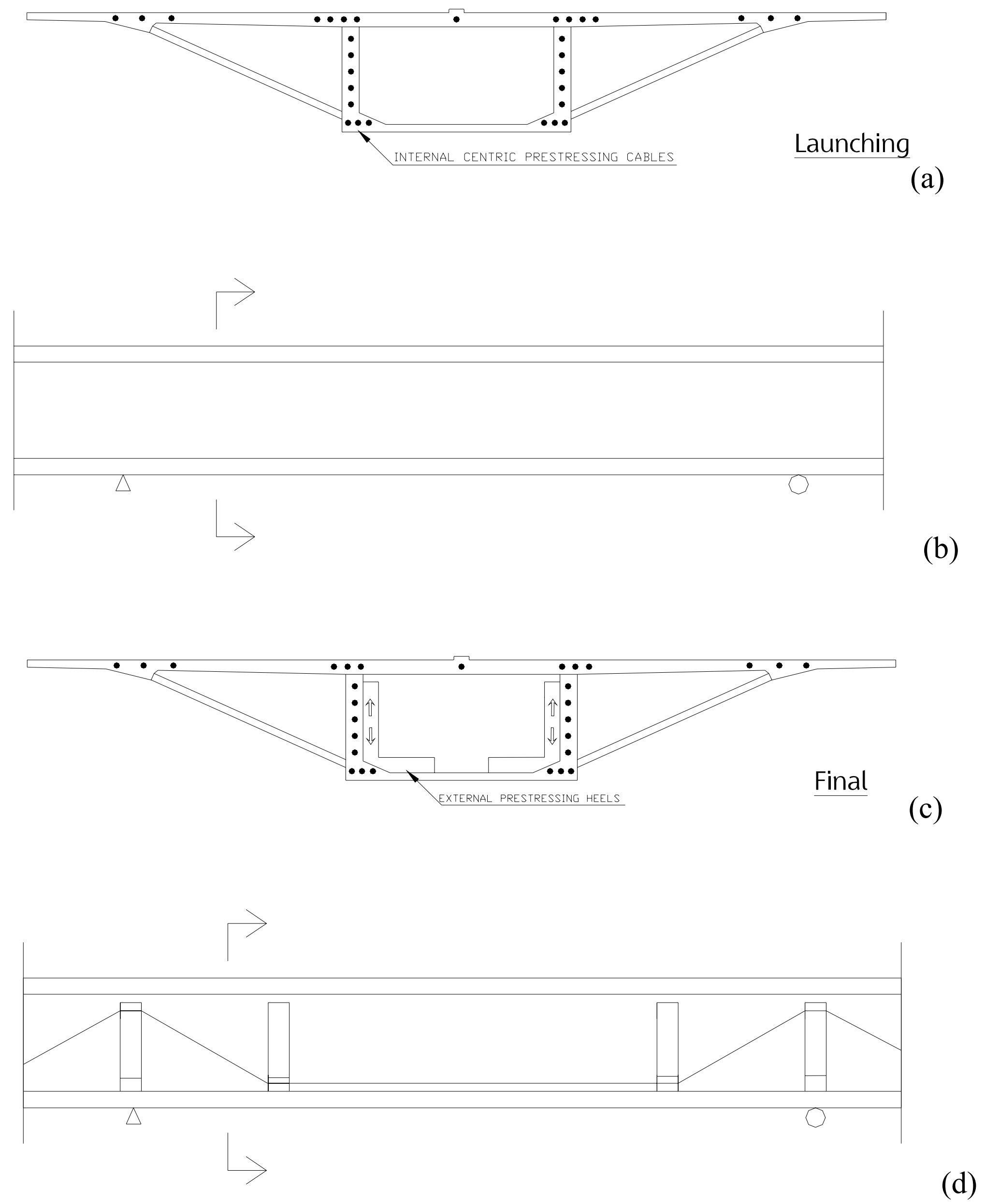


Figure 8.5 *General prestressing arrangement for the Rybny Potok Bridge. Internal launching tendons that were added during launching (a) and (b). External polygonal tendons that were added after completion of the launching (c) and (d).*

Figure 8.6 Reinforcement for deviators used for the external polygonal tendons.

8.4 Casting yard

The casting was a two-sequence system, based on two longitudinal steel beams with transversal beams on top, see Figure 8.7. The segment length was 30 m and the total length of the casting yard was somewhat above 60 m. From the beginning the segment length was supposed to be 15 m, but to be able to keep the time schedule, the length was increased to the double.

The casting yard was not covered with a shelter. Since the launching process was carried out from June to November the decision was to not cover the construction site. However the question was carefully considered and the decision was taken not without disagreements. At the time when the study trip was executed there had only been one segment with quality problem due to heavy rainfall during casting.

The first part of the casting sequence is to produce the bottom flange and the webs of the box girder. Then this U-shaped structure was prestressed together with the segment before and pulled forward to the next part of the casting yard. Of course the work was going on parallel in the both parts of the casting yard and after 9 days of construction work it was time for launching on the 10:th day. During the launching sequence the designer always participated to assist if any unforeseen incident occurred.

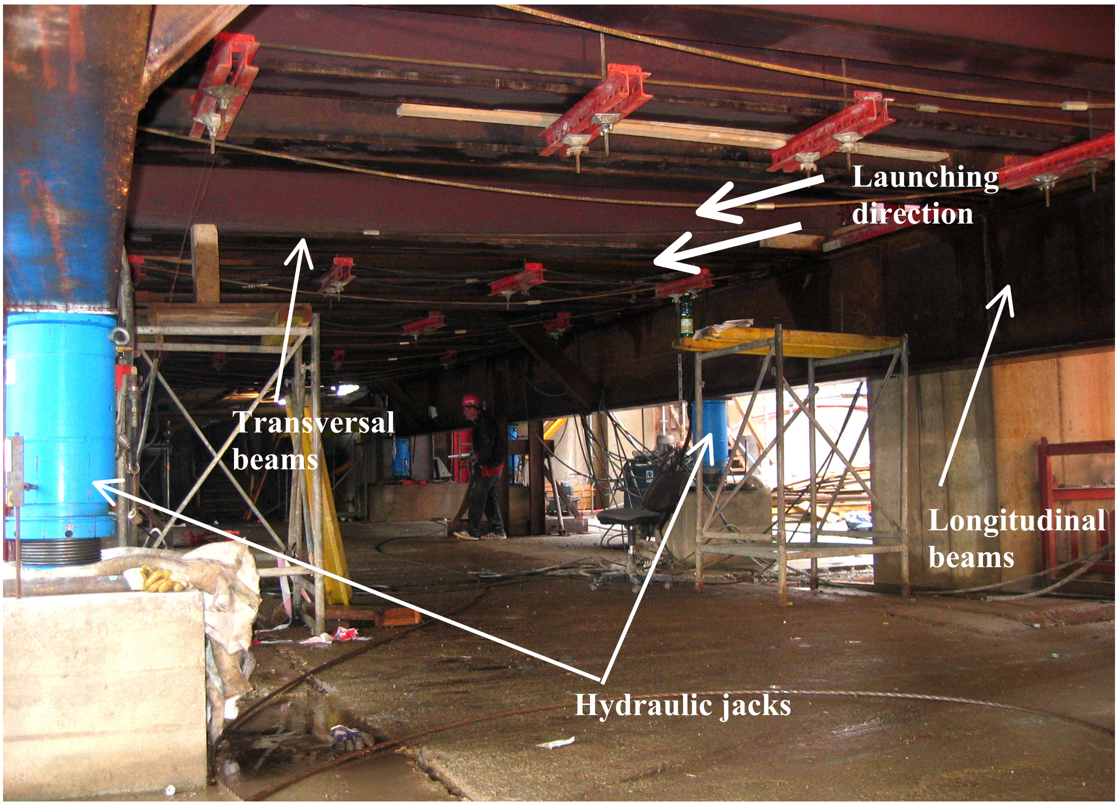


Figure 8.7 Underside of casting grid. The figure shows the longitudinal beams on which transversal beams are resting. The hydraulic jacks seen in the figure raise and lower the entire steel grid, which supports the bottom formwork.

This type of casting yard has been frequently used in Asia according to Bešta and Smíšek (2005). The large longitudinal beams, which were placed in the launching direction, were placed on hydraulic jacks, seen both in Figure 8.7 and in Figure 8.9. This made it possible to lower the bottom formwork 200 mm and at the same time raise a support, equipped with a sliding bearing on top, trough a hatch in the bottom formwork. The bottom of the U-section was then resting on the sliding bearing on the raised supports and also prestressed with the segment ahead, see Figure 8.10. Now the segment was ready to be launched out over the next part of the casting yard where the top flange of the box girder was cast and the bridge was equipped with the inclined struts. The whole launching process is described and illustrated more in next section.

The arrangement of the formwork can be seen in Figure 8.8, where the red vertical beams are attached to inclined turnbuckles acting as struts. The formwork is also attached to one horizontal strut in the lower edge. The sides of the formwork were mechanically removed but the system was not hydraulic. Though this type of casting yard has to been considered as a quite fashionable system especially in Swedish terms.

Figure 8.8 Formwork of the Rybny Potok Bridge. Notice the turnbuckles by which the formwork was manually mechanically moved to the side.

The working process was a ten-day repetitive working cycle. It was almost the same as the cycle described in Chapter 4, Section 4.2.4. The cycle length was longer than the one proposed in the literature and the one used in Sweden because of the size of the bridge. The cycle was divided into 12 hours shifts and the work was proceeding 24 hours a day.

8.5 Launching

8.5.1 Launching system

The bridge was launched with an average inclination of 3° upward and a horizontal radius of 1700 m and a vertical radius of 2400 m. Since the vertical and horizontal curvatures were constant there was no problem. The average inclination of 3° upward made the launching a little bit heavier, but also eliminate the need of a braking system. According to Bešta and Smíšek (2005) the type of launching system generally is chosen depending on the weight of the bridge and the vertical inclination of the launching.

The system used for the bridge was a pulling system. The pulling system is favourable for heavy bridges and different from the lift and pushing system, since it not depends on friction. The problems when using a system that makes use of friction for launching was in the beginning and in the end of the process when the pressure on the friction plate can be insufficient. This system is presented in Chapter 1, Section 1.4 as one of three common systems. In Figure 8.9 and Figure 8.10 the launching system and the movable formwork is shown.

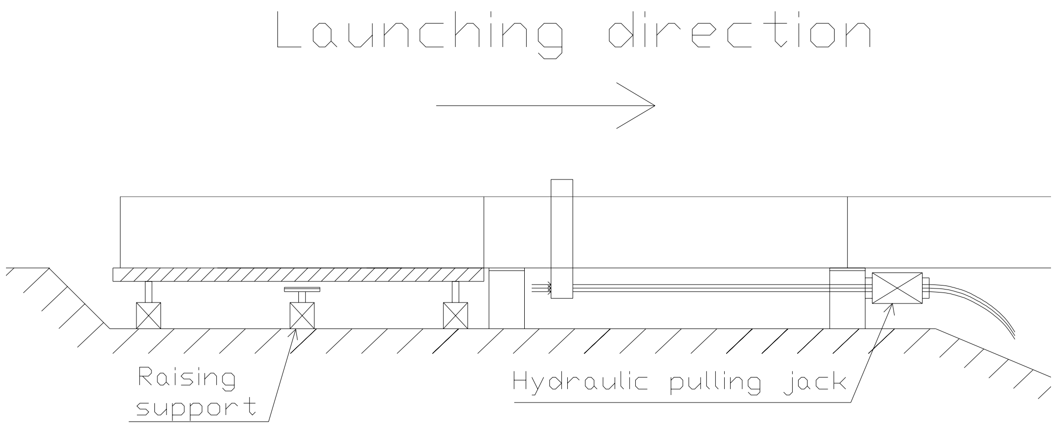


Figure 8.9 Principal figure of the launching system before lowering the formwork

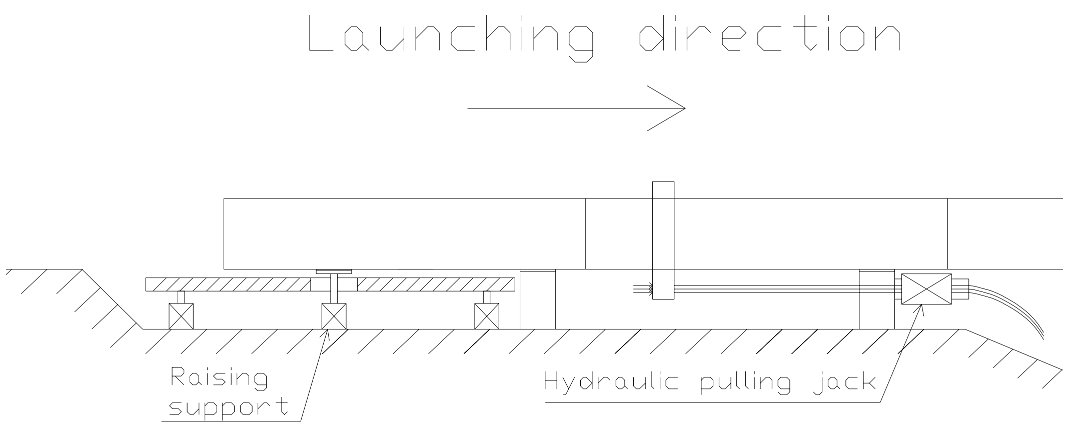


Figure 8.10 Principal figure of the launching system after lowering the formwork and lifting the support

The hydraulic pulling jack was launching the bridge forward. In the beginning when the bridge was not so heavy, only two hydraulic jacks were used but when the weight increased two additional jacks were used. Each jack was connected to a steel bracket, which was temporarily assembled into rectangular holes in the top and bottom flange of the box girder, see Figure 8.11. The speed of the launching was 6 m/h and each pull was 200 mm long. The frictional coefficient of the sliding bearings was, according to Bešta and Smíšek (2005), around 5-10 % for the first segment and 2-3 % for the rest of the segments. The frictional force together with the extra force needed for the inclination of the bridge gave the total necessary launching force. It is important to have the pulling jacks close to the bottom of the bridge to obtain an acceptable inclination of the pulling force when the steel bracket is close to the jack.

During the launching the piers were exposed to the horizontal force from the friction at the sliding bearings. If this force gets too high for one or another reason, the pier will bend and perhaps crack. To avoid this, each pier was equipped with a deflection gauge that automatically would stop the launching if an unacceptable deflection appears.

The piers of the bridge were constructed by means of climbing scaffolding. Sliding bearings were placed on the top of the piers during the launching, and after the entire launching the sliding bearings were exchanged to permanent ones. Figure 8.12 shows the sliding bearings. According to the designer, the bridge was designed to resist vertical movement in the piers, due to for example settlements, up to 10 mm during launching and 20 mm in the final state. This reduced possible problems with the foundation of the piers, as mentioned in Chapter 4, Section 4.3.4 and stated by Pettersson (2005).

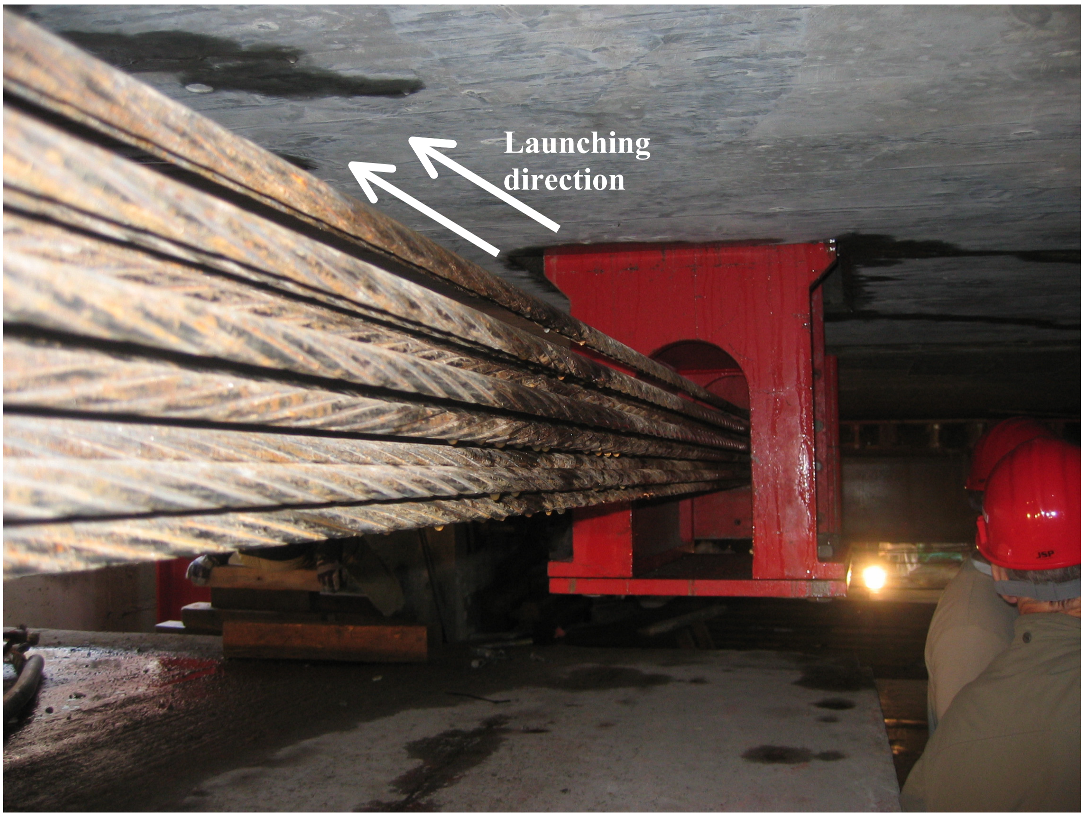


Figure 8.11 Steel bracket temporary assembled in the box girder and attached to the pulling tendons under the bottom flange of the U-section

The day we visited the site the ongoing launching was interrupted. The reason for this interruption was that one neoprene and Teflon plate had been inserted upside down. The friction had increased and the launching had been interrupted. The only problem with cracking they have had, in this project, was in the bottom of the piers during launching. Though, it was not a big problem, since the crack disappeared when the launching was finished. One measure to avoid problems with pier cracking is to connect the top of the pier to the abutment with cables, which was done on the Rybny Potok Bridge.

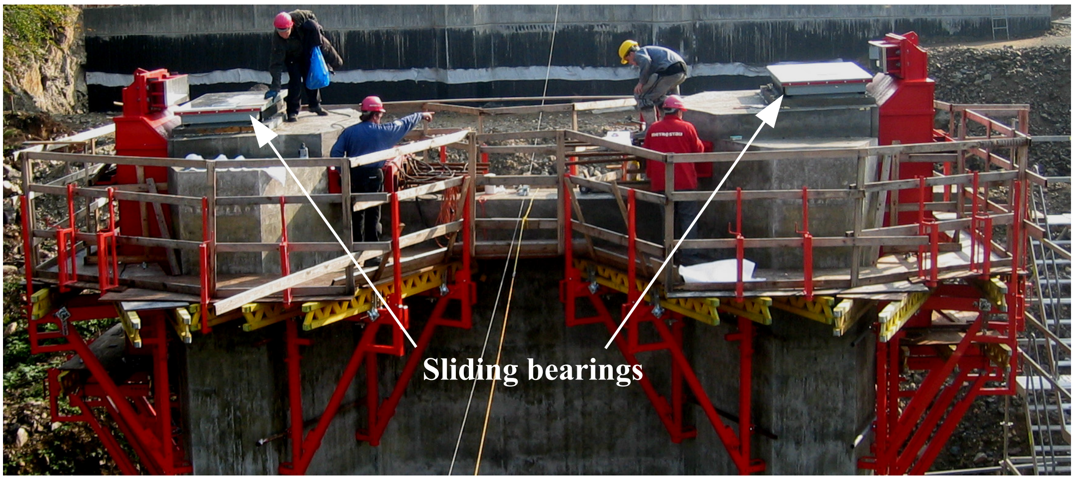


Figure 8.12 Preparation of the sliding bearings on one pier at the Rybny Potok Bridge

8.5.2 Launching nose

The launching nose was a steel nose constructed by Metrostav and designed for this specific project. It was mainly composed of two large steel beams and was equipped with two lifting jacks in the front to compensate for a maximum deflection of 200 mm, see Figure 8.13. The lifting jacks in the front of the nose are used to lift the front of the launching nose onto the support. Figure 8.14 shows the lifting jack when it reaches the support and when it has lifted the nose to the level of the support. This deflection was mainly caused by deflection of the nose itself. The nose was according to Bešta and Smíšek (2005) easy to reuse and will later be used in another project with some modifications. The nose is seen in Figure 8.15.

Figure 8.13 Lifting jack in front of the nose.

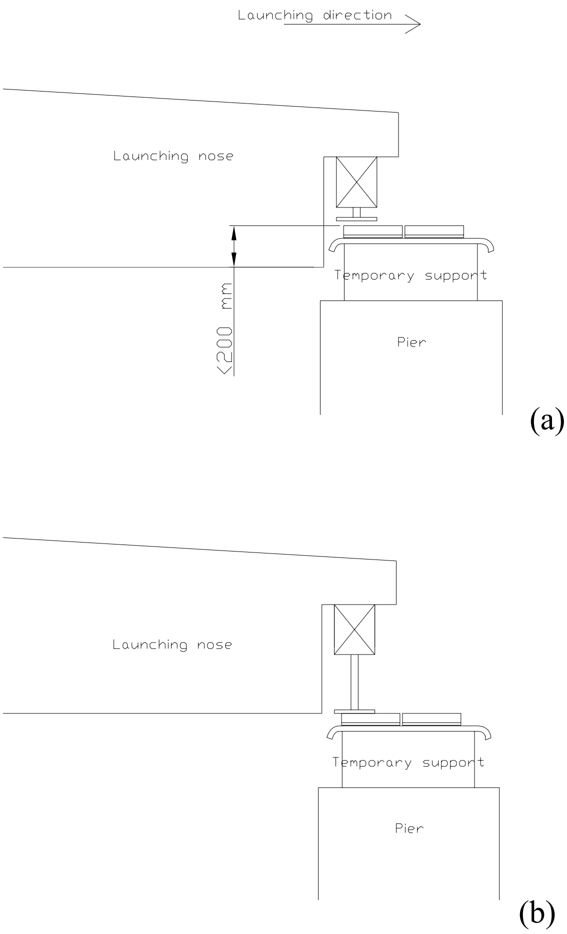


Figure 8.14 Lifting jack in the front of the launching nose when the nose is deflected (a), and when the jack has lifted the nose to the level of the support (b)

Figure 8.15 View of launching nose and the final pier in front of the end abutment.

8.6 Design engineering

As mentioned above the designer always participated to assist if any unforeseen incident occurred during the launching. Since our visit was executed a launching day we also had the possibility to meet the designer and ask some questions. The following information was received at a meeting with the designer on top of the bridge during the launching.

When designing the bridge several different analyses were performed. Time dependent FE calculations had been carried out with the FE program ANSYS. The launching had been analysed with the FE program RINO.

The main problem for the designer was the torsional resistance. Since the bridge is rather wide and the box girder is quite small and narrow the torsion was a problem, especially in the connections between the webs and the bridge deck where the force is concentrated.

The bridge was designed with a 50 m long front zone. This zone was designed to be able to resist the increased moments in the front zone due to the cantilever action during launching. Extra longitudinal prestressing was required in this zone.

The allowable tensile stress in the concrete was 2 MPa during launching and the maximum allowable crack width was 0,1 mm. For the final state the structure was designed to be fully compressed. The compression struts of the structure were designed for 8 MPa in normal cases but exceptionally 20 MPa.

8.7 Conclusions

The primary conclusion obtained from the study trip was the agreement between the result from our investigation of prestressing arrangements and the prestressing arrangement that was used in the Rybny Potok Bridge. Our investigation was carried out without knowing the type of arrangement used in the Rybny Potok Bridge. The fact that both the result of our investigation and that the Rybny Potok Bridge have similar type of prestressing arrangement do not necessary prove that alternative 3 must be the best alternative in all situations. Though it is a noticeable indication on the reliability of our analysis and an indication of the good performance that alternative 3 engender.

Though the launching prestressing with tendons in the web does seem to be a bit confusing with respect to our discussion in Chapter 7, Section 7.7 concerning the favourable aspect in external polygonal tendons for the final state. According to our discussion, one of the favourable aspects is the possibility to reduce the thickness of the web and decrease the dead weight of the structure. In the Rybny Potok Bridge, launching prestressing tendons were positioned both in the flanges and the webs. According to the designer, it was done because of the lack of space for prestressing strands in the cross section. Therefore the webs had to be utilized to position the tendons. Another reason is that when the formwork was lowered, the end section of the U-shaped box girder was a free cantilever and top prestressing in the U was needed to avoid tensile cracks in the top.

Our conclusion, drawn both regarding the discussion with the designer and our previous thoughts based on the interview with Carlsson (2005) concerning the web thickness, is that it must be the difference in span that induces the difference in prestressing arrangement. Also the fact that the Rybny Potok Bridge is quite wide and comparatively heavy supports this theory. The different preconditions for the prestressing arrangement applied on a wide bridge with 58 m spans and thin flanges compared to our reference bridges with 45 m spans and only 13,5 m wide deck was obvious. The increased weight and span required rather thick web, which were suitable for the launching prestressing tendons. The aim of this discussion is to illustrate how difficult it is to present general design suggestions valid for all incrementally launched medium span bridges.

The preconditions for the Rybny Potok Bridge were favourable for using the IL method. A deep valley, a bridge length of 356 m and constant curvature in both vertical and horizontal direction set the perfect preconditions. Even if the preconditions is the right ones the project still has to be designed and constructed in an excellent way to achieve full success, and our impression during the study trip was that the Rybny Potok bridge was designed in a effective and well considered way. The Rybny Potok Bridge seemed, with its rational casting yard and a design providing for an effective industrialized construction process, to be a successful project. Also to have the designer on site the launching day and the different control systems demonstrate professionalism and a common desire to perform well.

9 Future Applications and Recommendations for the Incremental Launching Method in Sweden

The previous chapters have contributed to an increased knowledge regarding the IL method. In this chapter the conclusions from each part will form a basis for a discussion where some final conclusions are to be stated. The aim is to point out some crucial details that have been discovered and analysed. Subsequently, some general advices for future applications of the incremental launching method in Sweden are presented.

9.1 Discussion

When evaluating a construction method, whether it is in bridge construction, car manufacturing industry or any other fields of applications, it is impossible to include and consider all different aspects. In this master thesis a general aspects have been considered and also a more detailed analysis regarding the prestressing arrangements was carried out.

The information that we have gathered and analysed is obtained from different sources that naturally have different opinions of the IL method but also several similarities. The interviews give a picture of how the IL method has been conducted in Sweden in the past and that the main problem for the IL method in Sweden so far has been to be profitable compared to other construction methods. Still the method is frequently used outside Sweden and therefore there must be some other reasons why it is not used in Sweden. One of these reasons might be the difference in markets between for example Sweden, Germany and Austria regarding the number of bridges that are suitable for IL. Another reason is that Swedish engineers and construction offices are more experienced in other construction methods and therefore obviously use them with less risk. Also the fact that the method requires certain investments is a negative aspect when it shall be used.

9.2 Adaptability

Incremental launching is a construction method with several distinctive characteristics, where the most governing ones are the geometric restrictions and a constant cross section. The bridge has to have a constant curvature in both vertical and horizontal plane to enable launching. This restriction limits the usability range for the method. As Pettersson (2005) mentioned, it is really important, already in the feasibility study when the stake out line of the road is established, to let the construction method influence the design. By that the usability for the IL method would increase.

Another effect from the second of the distinctive characteristics mentioned above, is the increased amount of material usage. Due to the varying moment during the launching, extra prestressing is required, and since the launching makes it impossible to optimise the concrete section additional concrete is also needed. A rational way of production with advantages in quality control and lean production must compensate for the increased amount of material usage. The economical advantages with the IL method naturally vary from one project to another but some general preconditions can be defined where IL is more advantageous to use. The obvious precondition is naturally when the underpass is inaccessible, but where it still is possible to have a span length in the medium range. Also the bridge length has to be long enough to obtain favourable effects due to repetitive process. According to Göhler Pearson (2000) a general minimum length of approximately 100 m is needed. When the bridge length approaches and increases 1000 m Göhler Pearson (2000) claim, that the travelling gantry construction method has even more favourable effects due to repetitive procedures and the possibility to construct a more optimised cross section.

9.3 Recommendations

When the preconditions indicate that the IL method is a possible and suitable construction method the next step is to carry out a good design. The design must fulfil not only the demands in the final state but also in the launching state. The investigation presented in Chapter 5, 6 and 7 considers both structural aspects as well as more general aspect such as durability and production matters. The conducted investigation resulted in some general advices for future applications of the incremental launching method in Sweden.

The conclusion is that out of the ten different prestressing alternatives that were investigated, alternative 3 is the most suitable one for the medium span range, which were the spans considered. For further details about the different alternatives, please see Chapter 5. Alternative 3 is a development of the prestressing arrangement used for the bridges built in Sweden before. The launching prestressing is similar but the final arrangement has external polygonal prestressing tendons operating together with straight internal ones. Please see Figure 5.7 in Chapter 5, Section 5.2.3. The favourable effects when using a combination of internal and external prestressing are several. According to Rosignoli (2002), a combination of internal and external tendons increases the robustness and improves the resistance against cracking compared to have only external prestressing. Regarding the production process it is favourable to use internal prestressing for the launching sequence, since the number of deviators drastically increases if external prestressing is used for launching prestress. Another favourable aspect when using final external polygonal tendons is the possibility to reduce the thickness of the web. This reduces the dead weight, which leads to a smaller need of launching prestress. It is also possible to increase the tendon length, since the friction losses decrease. When using external polygonal tendons one also avoid installation of internal polygonal ducts, which according to the site visit could be rather cumbersome and time consuming. Still, some work is required when using external polygonal prestressing. Deviators have to be produced, and the labour and material costs for producing these have to be compared with the efforts to install the internal ducts and internal tendons. The extra concrete needed in the webs when using internal prestressing must also be considered.

The negative effect when using internal launching prestressing is that it is impossible to detension the strands. However the possibility to detension tendons is not required when the span range is 40 to 60 m. On the other hand, if the span exceeds 60 m alternative 9 provides several advantages and could be competitive provided that IL still is a suitable construction method. One definitive conclusion is that there is no economical profit in reusing prestressing tendons. So unless the tendons induce too high compressive stresses in the concrete, there is no reason to detension prestressing tendons.

Each alternative has positive and negative aspects, which have to be considered and evaluated to be able to select the most favourable arrangement. The fact that at least two ongoing, or just finished, projects in Europe use the same prestressing arrangement as in alternative 3 clearly demonstrates the reliability of not only the arrangement itself but also of our analysis, that without any knowledge about the projects in Europe resulted in the same solution. The projects are the Rybny Potok project in the Czech Republic, which we visited, and the Thurrock Viaduct in the United Kingdom. In the near future, another three bridges will be constructed in Poland with the incremental launching method. All three bridges will be equipped with internal straight launching tendons and additional external polygonal tendons in the final state. Lundstedt (2005).

9.4 Final conclusions

When the industrialised process becomes more and more important for construction companies and the net price for steel increases at the same time, the incremental launching method seems more and more advantageous. We believe that if only the preconditions on a project are right, the IL method is a rational and competitive construction method. These preconditions can in short terms be described as when the bridge has to pass inaccessible areas or high above the ground and the span is somewhere about 30-65 m and the total length is between 100 and 1000 m. In these cases the IL method can be proved to be the most favourable. The curvature has of course to be constant to be able to have only one casting yard, but if the cost for constructing the bridge with any other method is very high, it can even be possible to launch the bridge from two positions and still make a profit on the construction.

Regarding the design of the bridge, we conclude that our alternative 3 should be adopted. That is, internal launching prestressing placed in the top and bottom flanges of the bridge and external polygonal prestressing tendons that are added when the bridge is in its final position. This is proved by both our investigation and our gathered experience from abroad.

If the usage of external prestressing is prohibited we would recommend alternative 1. It is a well-known and quite rational way of construction as well, though the structure will be heavier. Regarding external launching prestress it is our strongest recommendation that it is avoided, since the number of deviators and therefore the amount of work increases dramatically.

For launching, we recommend that lift and push jacks should be used on medium sized bridges, but pulling jacks should be used when the structures are large and heavy. This is only a conclusion from our site visit, since we have not made any own investigation on this subject. Regarding the sliding bearings more investigation has to be done to find out if it is possible, and favourable, to use the sliding bearings also in the final state.

We also would like to mention how difficult it is to present a general design valid for all incrementally launched medium span bridges. The different preconditions affect the design in several ways that make it complicated to cover all different aspect in a general design discussion. The information presented in this master's thesis cannot be applied for all medium span bridges without reconsidering the information with respect to the project specific conditions.

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