Repair of Prestressed Concrete Bridges
Stress Distribution and Prestressing Losses

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Repair of prestressed concrete bridges
– Stress distribution and prestressing losses

Reparation av broar i förspänd betong
- Spänningsfördelning och spänningsförluster

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ABSTRACT
Today we are faced with the problem of a growing number of existing prestressed concrete bridge structures that are in need of repair work. With prestressed concrete in particular, it is difficult to anticipate how the stress distribution in an existing structure will be affected by the removal or addition of concrete due to repair work. With the impending implementation of Eurocode in Europe, the development of suitable calculation methods on how to deal with this problem in accordance with the standard scripture, are compelling. This thesis is part of a larger project, which will develop suitable methods for the evaluation of damages on bridges, and suitable repair work, in accordance with Eurocode.

This thesis investigates the effect of repair work on a prestressed concrete bridge beam. The repair work constitutes the removal of old and/or damaged concrete and the addition of new concrete to repair the damage. The consequences of the repair work is investigated and presented by looking closely at the stress distribution in the cross section, and the losses in prestressing over a time-period of 100 years from construction. Concrete creep and shrinkage are taken into account, as well as prestressing steel relaxation. The results indicate that quite extensive repair procedures can be carried out without severe consequences to the structure. The time of the repair procedure is the most crutial to the structure, as large changes in stress are obtained when concrete is removed and replaced. The detrimental effects on the structure are more prominent the more extensive the repair procedure is.

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PREFACE

The work presented in this Master's Thesis was carried out at Destia Oy, Helsinki, Finland, in close collaboration with the Division of Structural Engineering, at Lund Institute of Technology in Lund, Sweden, from September 2009 to February 2010. I hope that my work will provide my readers with additional insight into the effects of repair work in prestressed concrete structures, as well as inspire others to carry out further studies on the subject.

Many people have helped me finish this thesis. I would like to thank Torsten Lunabba at Destia for giving me the opportunity to work with this project and for developing the method by which it has been carried out. Working with this thesis has presented me with many challenges, and I have learnt a lot. A great source of guidance throughout this project sprung from LTH, namely Sven Thelandersson. Thank you for the fruitful discussions and for your time and patience.

Olli-Pekka Tynkkynen and Kalle Nikula have also helped me a great deal by providing insight through discussions. Furthermore, I would like to thank Kristian Tammo at CBI for lending me literature. Lastly, I would like to acknowledge the help my parents have provided through correctional reading and Kim for his endless moral support.

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SUMMARY

Today we are faced with the problem of a growing number of existing prestressed concrete bridge structures that are in need of repair work. Engineers today, however, have very little experience in repairing such structures, much due to the fact that our present prestressed concrete bridges have previously all been in good working condition. With prestressed concrete in particular, it is difficult to anticipate how the stress distribution in an existing structure will be affected by the removal or addition of concrete due to repair work. With the impending implementation of Eurocode in Europe, the development of suitable calculation methods on how to deal with this problem in accordance with the standard scripture, are compelling. This thesis is part of a larger project, which will develop suitable methods for the evaluation of damages on bridges, and suitable design of repair work, in accordance with the Eurocode.

The purpose of this thesis is to determine how the stress distribution in a bridge made from prestressed concrete changes as the bridge is repaired. An existing bridge has been taken as an example, and this structure has been used as model for calculations. In the calculations, concrete is removed from the cross section of a prestressed bridge beam after the bridge has been operational for 30 years. The removed concrete is replaced with new concrete shortly after the removal. The difference in creep and shrinkage between the old and new concretes will affect the stress distribution in the structure, and must therefore be taken into consideration. When performing calculations in accordance with Eurocode, as supposed to many older European standards, the time-dependent parameters of creep and shrinkage continue to change throughout the lifetime of the concrete structure.

The changes in the stress distribution when concrete is removed will, naturally, also affect the losses in prestressing in the prestressing cables. The extent of this will be investigated. Furthermore, after some time has passed since new concrete has been cast to repair the existing damaged structure, the new and the old concrete castings will start to interact. The extent of the interaction is tightly linked to the bond between the different castings. The new casting will thus not be load-carrying to the same extent as the concrete in the original structure. Hence, the cross section can not be regarded as fully restored as soon as the repair concrete has been cast.

Notations

A_c	Cross-sectional are of the concrete in the cross section [m^2].
A_e	Equivalent cross-sectional area taking steel stiffness into account, before the injection of the prestressing ducts [m^2].
A_{ee}	Equivalent cross-sectional area taking steel stiffness into account, after the injection of the prestressing ducts [m^2].
$A_{ee,r}$	Equivalent cross-sectional area taking steel stiffness into account, for the reduced cross section after the removal of concrete [m^2].
A_p	Cross-sectional area of prestressing steel [m^2].
A_{sl}	Cross-sectional area of upper reinforcing steel [m^2].
A_{su}	Cross-sectional area of lower reinforcing steel [m^2].
E_p	Modulus of elasticity of prestressing steel [Pa].
E_{cm}	Modulus of elasticity of concrete at 28 days [Pa].
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days [Pa].
f_{ctk}	Characteristic axial tensile strength of concrete [Pa].
I_c	Second moment of inertial of the concrete in the cross section, around the centre of gravity of the concrete [m^4].
I_e	Equivalent second moment of inertia taking steel stiffness into accont, before the injection of the prestressing ducts [m^4].
I_{ee}	Equivalent second moment of inertia taking steel stiffness into accont, after the injection of the prestressing ducts [m^4].
$I_{ee,r}$	Equivalent second moment of inertia taking steel stiffness into accont, for the reduced cross section after the removal of concrete [m^4].
M_0	Total moment at the equivalent centre of gravity of the cross section just before the removal of concrete [Nm].
N_0	Total cross-sectional normal force just before the removal of concrete [N].
r	Amount of concrete removed and replaced from each upper flange [m].
z_e	Equivalent location of the centre of gravity for the cross section taking steel stiffness into account, before the injection of the prestressing ducts [m].

z_{ee} Equivalent location of the centre of gravity for the cross section taking steel stiffness into account, after the injection of the prestressing ducts [m].

$z_{ee,r}$ Equivalent location of the centre of gravity for the cross section taking steel stiffness into account, for the reduced cross section after the removal of concrete [m].

α_p Parameter which takes the stiffness of the prestressing steel into account [-].

α_s Parameter which takes the stiffness of the reinforcing steel into account [-].

$\Delta M(t_i)$ Change in moment at equivalent centre of gravity, at instant t_i [Nm].

$\Delta N(t_i)$ Change in normal force at equivalent centre of gravity, at instant t_i [N].

$\Delta M_{G,plate}$ Moment due to the additional weight of the repair concrete [Nm].

ΔM_{joint} Moment at the centre of gravity of the cross section due to ΔNjoint [Nm].

ΔM_{rem} Moment induced by removal of concrete [Nm].

ΔN_{joint} Normal force at the joint between new and old concrete after the addition of repair concrete [N].

$\Delta\varepsilon(t_i)$ Change in strain in the structure, at instant t_i [-].

$\Delta\kappa(t_i)$ Change in curvature in the structure, at instant t_i [m^{-1}].

$\Delta\sigma$ Change in stress, at instant t_i [Pa].

$\Delta\sigma_{add}$ Change in stress due to addition of new concrete [Pa].

$\Delta\sigma_{crs}$ Change in stress due to the reduction of the cross section when concrete is removed [Pa].

$\Delta\sigma_{rel}$ Prestressing steel relaxation stress [Pa].

$\Delta\sigma_{rem}$ Change in stress due to removal of concrete [Pa].

$\Delta\sigma_{surf}$ Change in stress due to the addition of a surfacing structure [Pa].

$\Delta\varepsilon_p(t_i)$ Strain in the prestressing steel at an instant ti [-].

ε_c Total strain in concrete [-].

ε_{ca} Autogenous shrinkage strain of concrete [-].

ε_{cd} Drying shrinkage strain of concrete [-].

ε_{cs} Total shrinkage strain of concrete (sum of autogenous and drying shrinkage) [-].

$\varepsilon_{ctk,add}$ Tensile strain in new repair concrete [-].

$\varepsilon_{ctk,ok}$ Tensile strain corresponding to the tensile strength of the concrete [-].

ε_s Total long-term strain of steel [-].

$\sigma_{0,l}$ Initial stress at the centre of gravity of the lower reinforcement, just after prestressing (before injection of tubes and after initial losses) [Pa].

$\sigma_{ctk,add}$ Tensile stress in new repair concrete [Pa].

$\sigma_l(t)$ Overall stress at the centre of gravity of the lower reinforcement, at time t [Pa].

$\sigma_{0,u}$ Initial stress at the centre of gravity of the upper reinforcement, just after prestressing (before injection of tubes and after initial losses) [Pa].

$\sigma_u(t)$ Overall stress at the centre of gravity of the upper reinforcement, at time t [Pa].

σ_{Poi} Initial stress in the prestressing steel, after initial losses in prestressing [Pa].

$\varphi(t,t_0)$ Concrete creep coefficient for load initiated at t0 [-].

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INTRODUCTION

1.1 Background

The building of prestressed concrete bridges in Scandinavia started in the 1950's. At present, many of these bridges, built in the 1950's and 1960's, are in need of repair work. A common form of damage is the deterioration of concrete, which would then need to be replaced. With prestressed concrete in particular, it is difficult to anticipate how the stress distribution in an existing prestressed concrete structure will be affected by the removal or addition of concrete due to repair work. Below, the effects on the prestressing losses and stress distribution of concrete removal from and addition to an existing prestressed concrete bridge structure are investigated. Both long-term and immediate effects are taken into consideration.

1.2 Problem and Limitations

The problem constitutes determining the effects of repair work on a prestressed concrete bridge beam. The focal point of the study is to investigate how the long-term losses in tension in the prestressing tendons are influenced by the repair work. However, immediate effects will also be investigated. The following questions are to be answered:

1. What changes in stress distribution, and thus also in long-term losses in tension, arise due to concrete removal and/or erosion?
2. To what extent does the altered stress distribution (1) inflict further long-term losses in the tendons?
3. What changes in stress distribution, and thus also in long-term losses in tension, arise due to the addition of new concrete?
4. How much concrete can be removed from the cross section?

Repair work is here limited to the removal of aged and/or damaged concrete, and the addition of new concrete to take its place. This repair work is carried out on a simply supported bridge structure, subjected to evenly distributed loads constituting the weight of the structure itself and the weight of a surfacing structure. Concrete is removed and added to the structure instantaneously. As new concrete is added, the bond between the new and old castings is assumed to be perfect. However, the new casting is not assumed to have any initial stiffness of its own.

The concrete is here assumed to be a homogenous material behaving linearly as it is subjected to loading. The cross section of the bridge beam is T-shaped and made from concrete, equipped with prestressing steel, upper and lower reinforcement. The same calculations are carried out on three bridges, with varying span-lengths, amounts of reinforcement and prestressing steel and with varying cross-section heights and widths. Temperature differences between new and old concrete, arising from the hydration of the new concrete, are not considered.

1.3 Method and Modelling

Three bridge models were chosen for further calculations, with span lengths of 20 metres, 28 metres and 36 metres. These bridge models were designed by case study of an existing bridge in Särkijärvi, Finland. Based on the Särkijärvi bridge geometry and boundary conditions the three models, all with different cross section heights, amounts of reinforcement and amounts of prestressing, were created. Bridge model dimensioning was done in a simple Finnish computer program called Tassu, which utilizes the Finnish National Standards. The model bridges are presented in section 5.2.

The bridges are subjected to repair work 30 years after their construction. Repair work here implies the removal of damaged concrete and the addition of new concrete to take the place of the removed. For each bridge, calculations determining the stress at the level of the upper and lower reinforcement, respectively, and determining the long-term losses in the prestressing, are carried out. These parameters will best describe what happens in the cross section when the bridge is repaired.

All calculations after the dimensioning are carried out in accordance with Eurocode. Creep, shrinkage and steel relaxation are all time-dependent properties. Thus, in order to fully understand what happens in the structure, stresses and prestressing losses must be calculated in time steps. The life-cycle of the bridge as a function of time is presented in Figure 0-1 below. The calculations start at the time of prestressing, 14 days after concrete casting. 28 days after casting, a surfacing structure is added to the bridge. After 30 years of service the bridge is assumed to require repair work and damaged concrete is removed from its upper flanges. 22 days after the removal of the concrete new repair concrete is added to the structure. The life-span of the structure ends after a service life of 100 years. The calculations, after the initial dimensioning, were carried out in MATLAB.

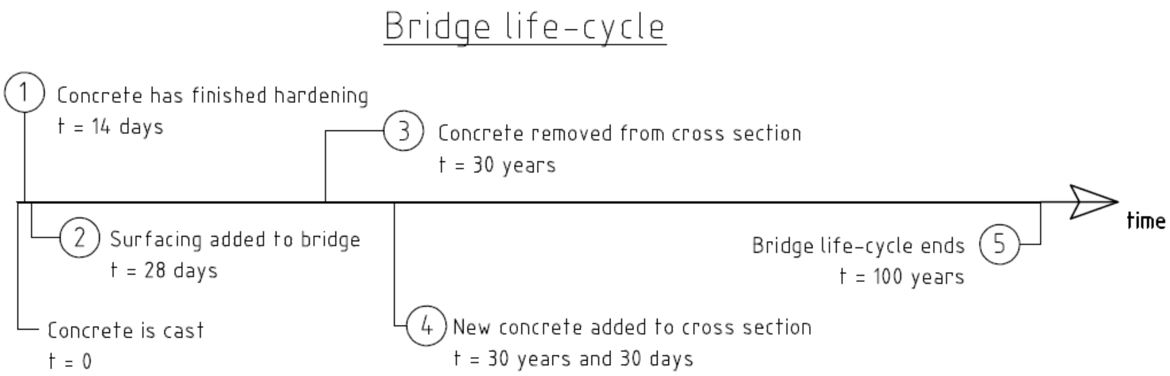


Figure 0-1: Presumed life-cycle of a bridge, from initial construction to end of service life at 100 years.

Three calculations are carried out for each bridge. Firstly, a reference calculation, evaluating only the effects of steps 1, 2 and 5 from Figure 0-1 above. This calculation represents a bridge left intact for 100 years. Secondly, the effect of just removal of concrete is evaluated. In this calculation steps 1, 2, 3 and 5 were evaluated (Figure 0-1). This calculation can be compared to a bridge beam with deteriorated concrete, left unattended. Lastly, a full calculation determining the effects of the entire life-cycle is carried out. Here the damaged concrete is first removed and then replaced, thus evaluating all steps from step 1 to step 5 (Figure 0-1).

The bridge is assumed to require repairs 30 years after its initial construction. The concrete is removed symmetrically from the edges of the upper flange of a T-shaped bridge beam cross section (Figure 5-7). Three cases are considered: The removal of 300 mm, 500 mm or 1800 mm (the entire upper flange), from each upper flange. The removal of concrete is in the calculations modeled as a change in stress corresponding to the dead weight that is removed. Moreover, as concrete is removed the cross section properties, namely cross-sectional area, location of centre of gravity and bending stiffness are all reduced due to the now smaller cross section.

As new concrete is added the cross section properties remain reduced in the calculations. The modeling is done this way since the new concrete casting is not assumed to contribute to the bearing capacity of the bridge, as such. The newly cast repair plate is rather modeled as a normal force at the joint between old and new concrete, summing up the effects of the plate's creep and shrinkage. Creep and shrinkage of the newly cast concrete will, naturally, be more pronounced than that of the older concrete structure. The normal force in the joint generates a bending moment and a normal force at the centre of gravity of the cross section, hence affecting the stress distribution.

When calculating losses in prestressing the effects of concrete creep, shrinkage and relaxation are taken into account. Furthermore, the effects of the different material properties of steel and concrete are considered. Firstly, equivalent cross section properties are calculated, in order to take steel stiffness into account. Secondly, a change in stress is determined, which arises due to the different behavior of the two structural materials, when they are subjected to loading. This change in stress will determine the prestressing losses in the tendons.

The repair work on the cross section is divided into three parts in the calculation, all occurring at different points in time. The first part constitutes the removal of concrete, resulting in a stress that corresponds to the decreased weight of the structure. The cross section is, in this stage, assumed to remain intact. The second part is a stress induced by the change in cross section properties. Here the weight of the structure remains the same and just the geometry is altered. Lastly, the third part constitutes the addition of new concrete, constituting the weight of the added concrete. Dividing the repair work into three parts is done in order to isolate the individual effects of the repair work, which simplifies the comprehension of the results.

2 PRESTRESSED CONCRETE

Prestressed concrete is concrete cast around prestressing tendons that are either already tensioned or post-tensioned after concrete casting. Prestressing results in the bridge structure being curved slightly upwards before it is subject to any loading, thus minimizing the downward curvature once the loading is in place. The prestressing is utilized to compensate for the poor tensile strength of concrete. As a result of the prestressing, compressive stresses are introduced into the material, which will be gradually relieved as soon as the bridge structure is subjected to loading, and begins to curve downward. Thus, the tendons carry the tensile stresses rather than the concrete, thereby delaying the initiation of cracking in the material. By prestressing concrete, it can hence carry far more extensive loads than a non-pretensioned structure (1, p. 1-4).

2.1 Post-tensioning

Post-tensioning implies casting the concrete before the tensioning steel is tensioned. In bridge construction, several tensioning cords are put into post-tensioning ducts that serve as protection against corrosion, for example. Once the cables have been tensioned the tubes are injected with concrete, in order to establish full interaction between the concrete and the tendons. By achieving full interaction the structural materials can be regarded as concurrent, rather than just individual structural elements. After the tendons have been tensioned and the tubes have been injected, the prestressing steel is thus activated. The cables are normally placed along the moment diagram of the structure, in a parabola shaped curve. The curvature of the cables, however, initiates friction between pre-tensioning steel and the post-tensioning tube, as the cables are being tensioned. This results in a loss in the prestressing force applied to the tendons (1, p. 25-26).

2.2 Immediate losses of prestress

The immediate losses of prestress during post-tensioning include, firstly, loss due to friction. If the tendons are curved inside the concrete structure, friction will occur at the bends of the strands, which results in a loss of tension. Secondly, wedge draw-in of the anchorage devices anchoring the tendons at the ends of the beam, results in a loss of tensioning. Finally, losses are induced due to the instantaneous deformation of the concrete as the tendons are stressed. The effect of the latter depends on the order in which the strands are tensioned (2, 5.10.5).

2.3 Time-dependent losses of prestress

The prestressing strands do not only suffer immediate losses, but tension is also lost gradually over time. Engström (1) maintain that when the prestressing cable of a simply supported beam structure is tensioned, the structure will respond with an elastic strain:

$$\varepsilon_1 = \frac{\sigma_c}{E_c} = \frac{P}{A_c \cdot E_c},$$

where P is the tensile force in the prestressing cable. This initial deflection will increase with time, due to creep. The strain related to creep is:

$$\varepsilon_{cr} = \varphi \varepsilon_1,$$

where φ is the creep coefficient. Furthermore, the concrete will shrink, which will result in decreased tension in the prestressed tendon, and thus a strain ε_{cs}. The creep and the shrinkage will both result in a loss in prestressing tension. This loss can be calculated by summing up the strains, $\Delta\varepsilon = \varepsilon_{cr} + \varepsilon_s$, and multiplying them with the modulus of elasticity of the prestressing tendons (1, p. 11).

Creep and shrinkage are both time-dependent properties of concrete. Thus, the loss in prestressing and the stress distribution in a prestressed concrete bridge structure are also time-dependent variables. According to Engström (1), creep and shrinkage reach final values in a long-term perspective, leaving the effective prestressing force amounting to about 75-80% of its initial value. Time-dependent losses should thus be calculated for a structure, taking the reduction in strain due to concrete deformation by creep and shrinkage under permanent loads, and reduction in stress due to steel relaxation under tension into account (2, section 5.10.6).

3 REPAIR OF PRE-STRESSED CONCRETE STRUCTURES

Publications concerning the reparation of concrete structures are plentiful. However, literature related to the repair of prestressed concrete bridges in particular, is scarce. This is due to the fact that the technology for building prestressed concrete bridges was not fully developed until the beginning of the 1950's. Many of the bridges built in the 1950's and 1960's are still in good working condition, and thus few cases of repair work on prestressed concrete bridges have been encountered in the Nordic countries before recent years.

3.1 Defects and damage

Damages to prestressed concrete structures can occur as damage to the prestressing tendons, deterioration of the concrete or member displacement. Damage to the tendons can be articulated as exposed, damaged or severed strands. Member displacement implies, for example, the rotation of the deck due to heavy vehicle impact (11, p. 532). Damage to prestressing tendons and member displacement will not be further discussed in this thesis.

The severity of damage depends on its location. The most severe damage case is when the concrete protecting the prestressing strands is deteriorated, leaving the tendons partially visible through the remaining structural concrete (11, p. 621). Concrete can deteriorate by scaling, cracking or spalling. Scaling is a progressive disintegration process which dissolves the cement paste, starting from the surface and moving downward. This occurs due to repeated freeze-thaw cycles, poor drainage or deicing chemicals, for example. Cracking can occur in multiple locations on the structure, and have a variety of directions, but is essentially initiated due to excessive tensile stresses. The most prominent in bridges are longitudinal cracks, which often appear between prestressed concrete box beams, and random cracks, which occur due to, for example, poor curing methods or load deflection. Finally, spalling is a breaking out process of the concrete, which is initiated at the top reinforcing steel. Spalling appears as a result of corrosion of the rebars and is accelerated with the presence of chlorides (11, p. 147).

3.2 Repair procedure briefing

Wear and tear to concrete bridge decks occur continuously due to, for example, repeated freeze-thaw cycles and heavy traffic loads. In the Northern countries, additional deterioration of bridge decks is invoked by use of deicing salts on roads and studded tires on vehicles. If damage to the concrete deck is too extensive for mere patching with a sealer, the dilapidation of the concrete bridge deck requires damaged concrete to be removed, and to be replaced by new concrete (7, p. 1 and 11, p. 534). The removal of the concrete can be done in multiple ways, for example, by different kinds of blasting, scrabbling, milling or waterjetting (8, p. 6-7).

3.3 Bond

When damaged concrete is removed from an existing bridge deck, the method of removal is of uttermost importance to how the structure will perform once new concrete has been cast to replace the deteriorated concrete. Full interaction between overlay and base structure is desirable, but can normally only be achieved if the base surface, after the removal of deteriorated concrete, is rough to the surface, clean and free from microcracking. Selective removal of damaged concrete, leaving a surface with the properties described above, can only be achieved by waterjetting (8, p. 6-7). An alternative method to improve interaction between the new and old concretes is to attach the overlay to the base with a mechanical device, such as reinforcement (6, p. 36). To attain full interaction, the damaged concrete should first be removed by waterjetting and, when the new concrete is cast, the repairing concrete should be sufficiently attached to the existing structure with reinforcement (6, p. 49).

3.4 Interaction between repair concrete and base structure

Full interaction between the existing concrete structure and the newer repair concrete is desirable when repairing a bridge structure. Full interaction between the newer and the older concretes will allow the repaired bridge deck to work as a homogenous structural element. (6, p 73) When repairing the compression zone of a beam, additional concrete is cast, which results in a differential shrinkage between the different concretes. This differential shrinkage will, however, mainly affect the stiffness of the repaired structure, rather than remarkably affecting its strength (6, p 49).

Cracks may develop as a result of change in forces that arise due to the differential shrinkage between the new and the old concrete castings. Furthermore, cracks may develop due to differences in temperature between the old and new structures, as the new concrete hydrates. The difference in temperature can be minimized by cooling the new concrete casting during hydration and/or by pre-heating the old structure. The more repair concrete is cast, the more extensive the heating due to hydration (10, p. 278-279).

The interaction between the old structure and the new casting is also affected by vibrations from traffic during the hardening of the new concrete. The strength of the new casting may reduce, and the bond between the old and the new concrete may weaken, due to vibrations. Hence, it is of importance to minimize vibrations during the hardening of the new concrete, possibly by temporary speed limits (10, p. 278-279).

The extent of the transference of the shrinkage and creep strains from the repair concrete to the old structure is determined by the elastic moduli (E) of the two materials, or, rather their relative stiffness:

$$\frac{E_{repairconcrete}}{E_{oldstructure}}$$

If the repair material is stiffer than the existing structure, strains can be transferred from the repair concrete to the old concrete. If the relation is the opposite, the two castings display no or negligible structural interaction, which implies that either tensile stress or no stress at all will be present in the repair concrete. The shrinkage of the repair concrete is the most extensive during a few weeks just after the casting. During this time, shrinkage is either transferred or restrained, depending on the modulus of elasticity of the old structure. Since the original structure is, at least initially, stiffer than the repair concrete, the restrained shrinkage of the repair patch will induce tensile forces in the repair concrete. If this tensile stress exceeds the tensile strain of the new concrete, cracking will occur (5).

3.5 Effect of prestressing

In the case where only the concrete is damaged, and not the prestressing tendons, the stress distribution in the prestressed structural element will be affected by the repair work. For example, if the bottom part of a beam has suffered extensive losses in concrete, but the prestressing cables are still intact, the prestressing applied initially will still be present, whereas the concrete area will be reduced. In a single-span bridge, dead load alone will result in the maximum compressive stress, because of the initial prestressing. This problem can be solved by preloading the structure while it is repaired. The preloading is accomplished by adding tensile forces to make up for the extra compression. Calculations should be carried out, on the reduced cross section, in order to determine whether the stresses are at an acceptable level, before the bridge deck is repaired. (11, p. 546)

4 EUROCODE SPECIFIC CALCULATIONS

4.1 Initial prestressing losses in tendons

The initial losses are connected to the physical tensioning of the tendons. All of the tensioning initially applied to the tendons will not be preserved in the structure, but some will be lost due to wedge draw in, friction, and the instantaneous deformation of the structural concrete. The consequences on the structure due to these three phenomena are further explained below.

4.1.1 Wedge draw-in

The prestressing tendons are attached to the concrete by anchors. When the strands are tensioned they are first stretched, and then wedged into place. Well in place, they are held put by the anchors. As the wedging is done, the anchor may slip due to the massive force in the tendons. As a consequence, the strands suffer a loss in tensioning. In many cases, the tensioning is done from just one end of the beam, thus limiting the loss to that end only. Eurocode refers to ETA (European Technical Approval) for specification of the expected wedge draw-in. ETA 2.7.3 recommends a wedge draw-in of 5 or 6 mm, depending on whether the seating ram of the stressing jack is activated, or not (4).

4.1.2 Friction

The pre-tensioning strands are normally placed with a curvature within the concrete, which has the effect that friction occurs between pre-tensioning steel and the post-tensioning tube, as the cables are tensioned. The tensioning force P in the tensioning cables thus decreases with increasing distance from the anchor. The losses in tensioning force due to friction is larger the more pronounced the curvature of the cords is. (1, p. 25-26)

4.1.3 Instantaneous deformation of concrete

Since the strands are not all tensioned at the same time, the earlier a strand is stressed, the more prestressing tension it will loose as the following tendons are tensioned. This can be taken into consideration by applying a larger stress to the tendons that are stressed first, and then successively decreasing the stresses applied to the following tendons. If done carefully, all tendons can then have the same initial prestressing force P_0, after all strands have been tensioned (1, p. 71).

4.2 Time dependent prestressing losses

The long-term prestressing losses are determined by the creep and shrinkage of the concrete, and by the relaxation of the prestressing steel. Traditionally, creep and shrinkage are assumed to reach a final value after some time has passed after the casting of the concrete, and the parameters are hence most often assigned final values in design calculations (as mentioned in section 2.3). In repair work, however, especially as new and old concrete must interact, the time dependent effects of these two parameters are of great importance, and thus, this simplification cannot be made.

4.2.1 Creep

The creep coefficient, $\varphi(t,t_0)$, and the corresponding creep strain in the concrete is calculated in accordance with Eurocode EN 1992-1-1:2004 3.1.4 and Annex B. The value of the creep coefficient is highly sensitive to which relative humidity is chosen for the calculations. A relative humidity value of 70% is chosen for the calculations, which is the relative humidity for outdoor air according to the Concrete Code of the Finnish National Standards *Betoninormit* (9, p. 15).

4.2.2 Shrinkage

The shrinkage strain is divided into two parts, the drying shrinkage strain, $\varepsilon_{cd}(t)$, and the autogenous shrinkage strain, $\varepsilon_{ca}(t)$. The total shrinkage strain at a time t is $\varepsilon_{cs}(t)=\varepsilon_{cd}(t)+\varepsilon_{ca}(t)$. The shrinkage strains are calculated in accordance with the method presented in Eurocode EN 1992-1-1:2004 3.1.4 and Annex B. The same relative humidity is chosen for the calculation of shrinkage strain, as in the calculation of creep strain.

4.2.3 Relaxation

The stress lost in the prestressing tendons due to prestressing steel relaxation $\Delta\sigma_{rel}(t)$ is determined according to Eurocode EN 1992-1-1:2004, equation 3.29. $\Delta\sigma_{rel}(t)$ is an accumulative value of the lost tension, in other words a value which increases with time.

4.2.4 Losses due to stress variation

A sum of instantaneous and creep deformations due to a variation in stress in the structure may inflict further losses in tension. The strain due to this sum of deformations is defined in the calculation of concrete strain in Eurocode EN 1992-2 (see Equation 1 section 4.3).

4.3 Calculation of concrete strain

The concrete strain, determined at a time t, is calculated according to the bridge specific Eurocode scripture (3, Annex KK. 3), as:

$$\varepsilon_c(t) = \underbrace{\frac{\sigma_0}{E_c(t_0)}}_{1} + \underbrace{\varepsilon_{cs}(t,t_s)}_{2} + \underbrace{\varphi(t,t_0)\frac{\sigma_0}{E_c(28)}}_{3} + \underbrace{\sum_{i=1}^{n}\left(\frac{1}{E_c(t_i)} + \frac{\varphi(t,t_i)}{E_c(28)}\right)\cdot \Delta\sigma(t_i)}_{4} \qquad (1)$$

In equation 1, the first term (marked 1) represents the instantaneous deformation, due to an initial stress. The second term (marked 2) determines the concrete creep due to the initial stress, and the third term (3) is the strain due to concrete shrinkage. The fourth term (4) of the equation is a sum of the instantaneous and creep deformations that occurs due to a variation in stress in the structure, at an instance t_i. The variation in stress is further investigated in section 6.2.3.

5 BRIDGE MODELLING

Models for calculating the effects of repair work on a bridge structure were made, studying an existing bridge structure in Särkijärvi, Finland. The original bridge and the geometrical models used in calculations are presented below.

5.1 Description

The reference bridge in Särkijärvi, Finland is a single-span bridge made from prestressed concrete (post-tensioning), built in 2009. The span-length of the bridge is 28.5 meters, and the bridge beam height is 1.35 meters. Särkijärvi Bridge drawings and pictures can be found in Appendix 1.

5.2 Geometrical modelling

The original bridge in Särkijärvi above has been used to create three simple bridge models, with different span lengths of 20, 28 and 36 meters. All three models are equipped with prestressing tendons and both upper and lower reinforcement. In order to simplify the calculations, the cross section was simplified to a T-beam cross section with perpendicular corners. The thickness of the concrete in the model is the average width of the non-plane members of the original structure.

The cross section height and width was somewhat altered for the bridges of span-lengths 20 and 36 meters, whereas the bridge with span-length 28 meters is a simplified copy of the original bridge in Särkijärvi. A preliminary dimensioning was made in a simple computer program, namely Tassu, which utilizes the rules and regulations of the Finnish National Standards. The tensioning of the tendons is done in a way which provides all the strands in the same cross section with the same initial prestressing force. The prestressing force is assumed to remain the same in all tendons after the effects from initial losses have been assessed.

The initial prestressing force, however, differs between the different bridges. The post-tensioning is done from one end of the bridge only. Wedge draw-in for the specific wedges used in Särkijärvi has been defined as 10 mm.

5.2.1 20-meter span-length

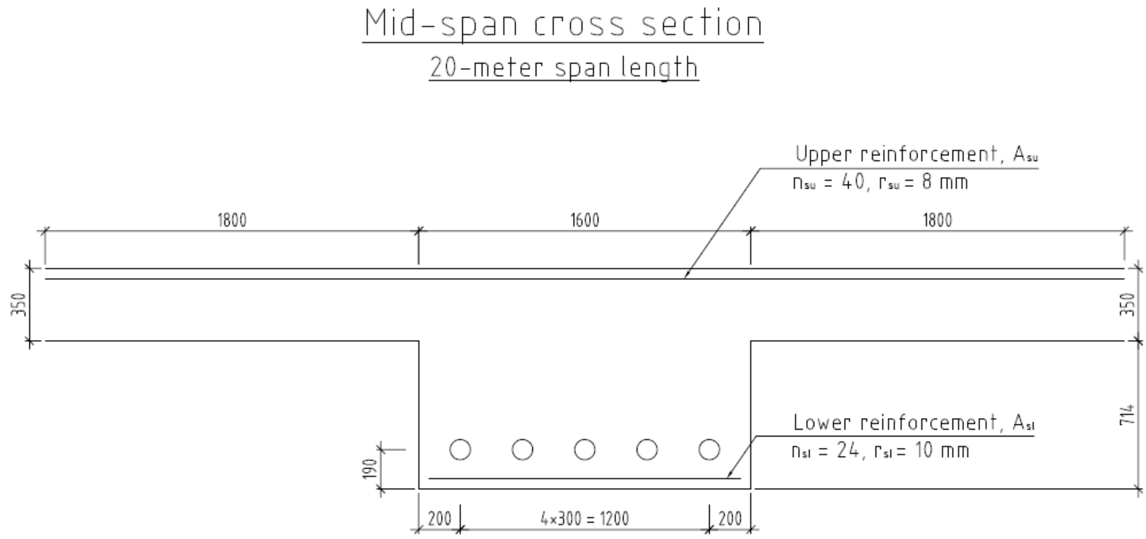


Figure 5-1: Mid-span cross section of bridge model with span-length 20 meters.

The 20-meter beam T-section has a smaller width than the Särkijärvi Bridge and the other two cross sections (1.6 meters), a total cross section height of 1.064 meters and 5 ducts for prestressing cables (Figure 5-1), in two layers near the ends of the beam (Figure 5-2). Each duct containing prestressing cables has 14 individual cables inside. The curvature of the prestressing is depicted in below.

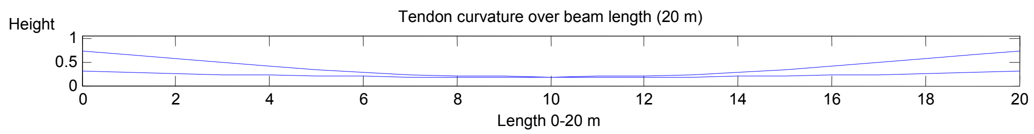


Figure 5-2: Tendon curvature for bridge model with span-length 20 metrrs.

5.2.2 28-meter span-length

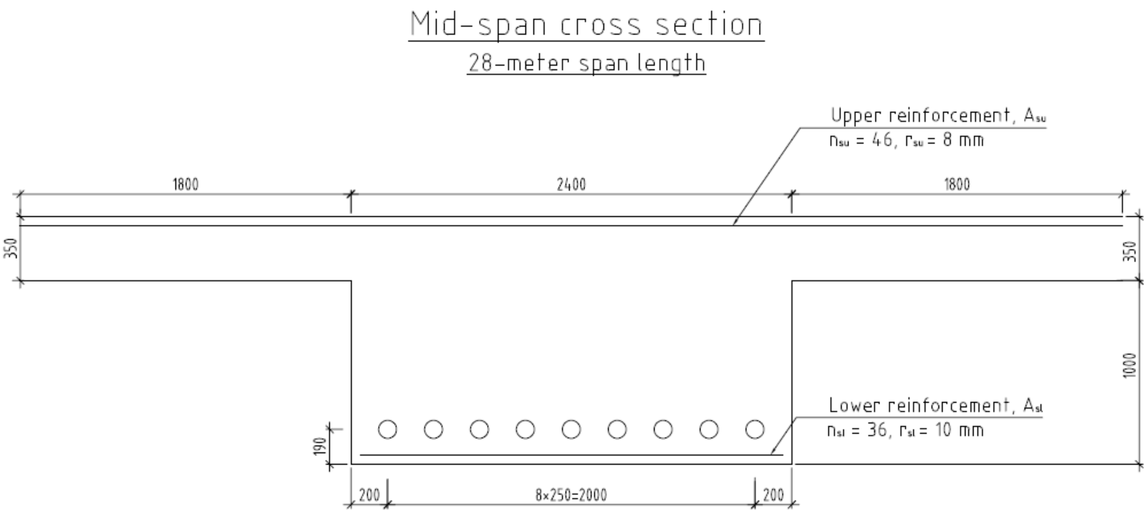


Figure 5-3: Mid-span cross section of bridge model with span-length 28 meters.

The cross section of the 28-meter bridge model is a simplified version of the cross section of the actual Särkijärvi Bridge, with a total height of 1.35 meters, a width of 2.4 meters and 9 ducts for prestressing cables (Figure 5-3), in two layers near the ends of the beam (Figure 5-4). Each duct containing prestressing cables has 14 individual cables inside. The curvature of the tendons, over the length of the beam, is depicted below.

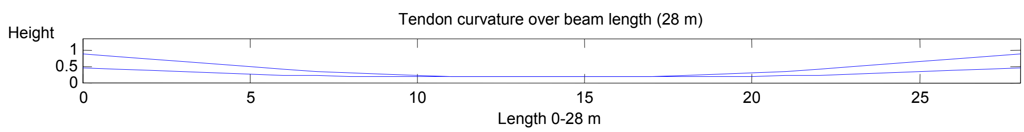


Figure 5-4: Tendon curvature for bridge model with span-length 28 meters.

5.2.3 36-meter span-length

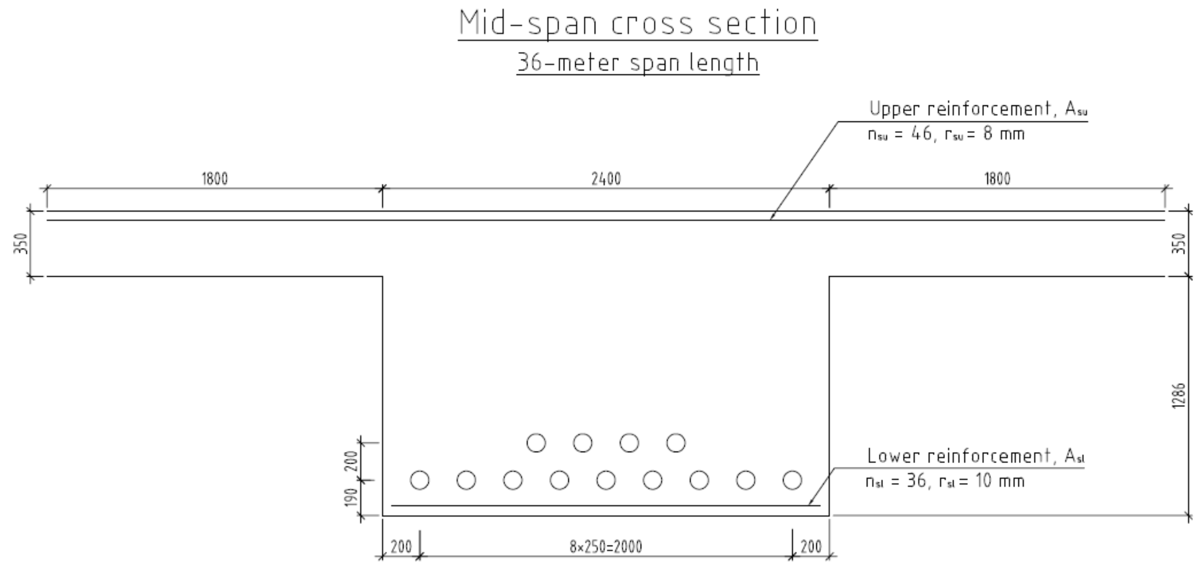


Figure 5-5: Mid-span cross section of bridge model with span-length 36 meters.

The longest beam, with a span length of 36 meters, required one additional layer of prestressing cable. The beam hence includes 13 ducts for cables, in three layers near the ends of the beam and in two layers midspan (Figure 5-6). The total height of the cross section is 2.4 meters (Figure 5-5). As in the other two beam models, each tube containing prestressing cables has 14 individual cables inside. The tendon curvature is depicted in Figure 5-6 below. Since two layers of ducts mid-span complicates the calculations, the ducts are assumed to be placed at the same level in the cross section, namely at the centre of gravity of the ducts mid-span.

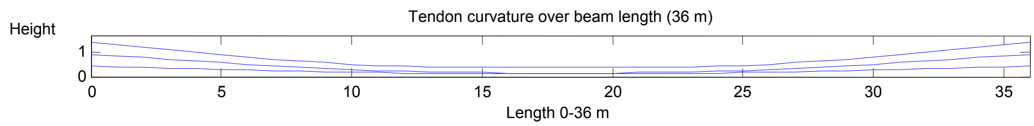


Figure 5-6: Prestressing curvature for bridge model with span-length 36 meters.

5.3 Case modelling

The life of the three bridge models follow the presumed life-cycle of a bridge, presented in Figure 0-1. Calculations were thus carried out on the structures, presuming that the bridges will require repairs 30 years after their construction.

Before the bridge requires any repair work, from the end of concrete curing at 14 days after casting, until just before 30 years has passed, the creep, shrinkage and relaxation effects exert an influence on the losses in prestressing and the stress distribution of the structure. Only the weight of the structure is considered in the calculations of the prestressing loss and the stress distribution, since only long-term effects are considered. As concrete is removed and replaced, the meddling with the cross section geometry will further affect the prestressing loss and the stress distribution.

Prestressing loss and stress distribution are determined by looking closely at the two different structural materials of the structure. Steel and concrete behave very differently when they are subjected to loading. Concrete is subject to creep and shrinkage effects, and prestressing steel is affected by prestressing steel relaxation. These time-dependent effects will all contribute to changing the stresses in the structure over time. The concrete is here assumed to be a homogenous material behaving linearly as it is subjected to loading. In reality, concrete has a highly non-linear behavior in loading, especially at high stresses approaching 45% of the concrete compressive strength, $0.45f_{ck}$. The Eurocode states that if the compressive stress in the concrete structure reaches a permanent value exceeding this limit, the non-linearity of creep must be considered (2, section 5.10.2.2 (5)). Hence, it is important that the stresses in the structure never reach a permanent value exceeding the limit of $0.45f_{ck}$.

The bridge beam structure is subjected to to a change in loading twice during its life-cycle. Firstly, at 28 days after concrete casting, a surfacing structure is added, and secondly, 30 years after the construction of the beam it will require repair work. The repair work is divided into three phases of reparation for each bridge. Firstly, the symmetrical removal of concrete from the tip of each flange is considered. Secondly, the reduction of the cross section and hence also the reduction in stiffness of the cross section, is taken into account. This will initially inflict an elastic stress in the structure, and then, consequently affect all stresses which are calculated with the reduced cross section parameters. Thirdly, the symmetrical addition of concrete at the tip of each flange is considered. Each phase causes a difference in stress to arise, which contributes to the creep of the structural concrete.

The amount of concrete removed is here always the same amount of concrete which is added. In other words, the object of the procedure of adding concrete is to restore the cross section to its original state. However, full interaction between new and old concrete cannot be presumed. Thus, the cross section is assumed not to get its cross section properties restored from the addition of the new concrete. The added concrete plate is rather modeled as a force, affecting the stress distribution with its more pronounced shrinkage and creep, but which has no bending stiffness of its own.

Three different removal and addition cases are considered: the removal and addition of 300 mm, 1300 mm and 1800 mm from each upper flange. The removal of 1800 mm on each side entails removing both of the upper flanges (see Figure 5-7). Removal and addition of concrete, and the reduction of the cross section, are taken into account by calculating the change in stress, which the different effects invoke on the structure. The parameters chosen to specify what happens in the structure due to the repair work are losses in prestressing in the prestressing steel and the stresses at the centers of gravity of the upper and lower reinforcement bars, respectively.

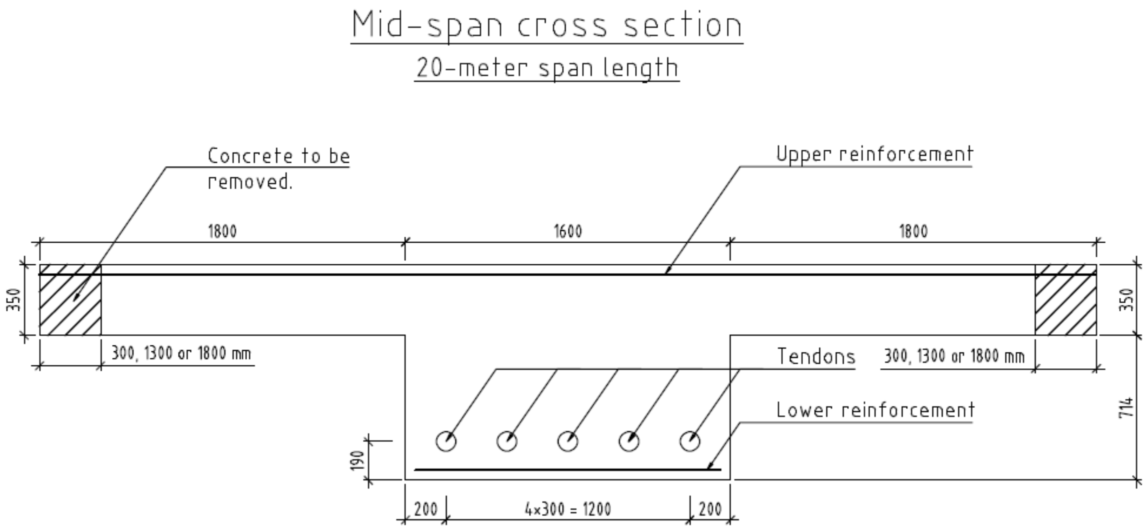


Figure 5-7: Mid-span cross section of a bridge model with a span-length of 20 metres. The concrete is removed and replaced symmetrically from the edges of the flanges.

More detailed theory behind the modeling of the repair procedure is presented in section 7.2. Concrete creep and shrinkage, as well as prestressing steel relaxation are calculated in accordance with Eurocode, as specified in section 4 above.

6 THEORY

6.1 Equivalent cross section properties

The equivalent cross section properties (cross-sectional area, centre of gravity and area moment of inertia) take the stiffness of both concrete and steel parts in the cross section into account. Since steel has a higher modulus of elasticity than the concrete, the structure will gain stiffness from the structural reinforcement and prestressing steel. Two sets of equivalent cross section property values must be calculated: one before the prestressing steel is activated and another after the activation of the prestressing steel (see section 2.1 about post-tensioning).

The cross section mainly consists of concrete. Some additional stiffness is added to the structure in the calculations by taking the steel into consideration.

The reinforcing steel stiffness is taken into account by a parameter $\alpha_s = \frac{E_s}{E_{cm}}$.

The prestressing steel stiffness is taken into account by a parameter $\alpha_p = \frac{E_p}{E_{cm}}$.

E_s = reinforcing steel modulus of elasticity
E_p = prestressing steel modulus of elasticity
E_c = concrete modulus of elasticity at 28 days after casting

The equivalent cross-sectional area, before the activation of the steel is:

$$A_e = A_c + (\alpha_s - 1) \cdot (A_{su} + A_{sl}),$$

where A_c, A_{su} and A_{sl} = the cross-sectional areas of the concrete, upper and lower reinforcing steel.

The equivalent centre of gravity of the cross section (the distance from the bottom of the cross section to the centre of gravity), before the activation of the steel is determined as:

$$z_e = \frac{A_c \cdot z_{bc} + (\alpha_s - 1) A_{su} \cdot z_{bu} + (\alpha_s - 1) A_{sl} \cdot z_{bl}}{A_e},$$

where z_{bc}, z_{bu} and z_{bl} = the distances from the bottom of the cross section to the centre of gravity of the concrete, upper and lower reinforcing steel.

The equivalent area moment of inertia, at the centre of gravity, before the prestressing is activated, is:

$$I_e = I_c + A_c \cdot (z_{bc} - z_e)^2 + (\alpha_s - 1)A_{su} \cdot (z_e - z_{bu})^2 + (\alpha_s - 1)A_{sl} \cdot (z_e - z_{bl})^2,$$

where I_c = the area moment of inertia of the concrete only.

After the activation of the prestressing steel the equivalent cross section parameters are:

$$A_{ee} = A_c + (\alpha_s - 1)A_s + (\alpha_p - 1)A_p,$$

where A_p is the cross-sectional area of the prestressing steel.

After the activation of the prestressing steel, the location of the centre of gravity (the distance from the bottom of the cross section to the centre of gravity) is determined as:

$$z_{ee} = \frac{A_c \cdot z_{bc} + (\alpha_s - 1)A_{su} \cdot z_{bu} + (\alpha_s - 1)A_{sl} \cdot z_{bl} + (\alpha_p - 1)A_p \cdot z_{bp}}{A_{ee}},$$

where z_{bp} = the distance from the bottom of the cross section to the centre of gravity of the prestressing steel.

The equivalent value for the second area moment of inertia, after prestressing steel activation is:

$$I_{ee} = I_c + A_c \cdot (z_{bc} - z_{ee})^2 + (\alpha_s - 1)A_{su} \cdot (z_{ee} - z_{bu})^2 + (\alpha_s - 1)A_{sl} \cdot (z_{ee} - z_{bl})^2 + (\alpha_p - 1)A_p \cdot (z_{ee} - z_{bp})^2$$

The equivalent cross section values calculated before the activation of the prestressing steel is used for calculating initial stresses in the cross section. After the initial stage, when the prestressing has been activated (14 days after concrete casting), the second set of equivalent cross section values are used, which take the prestressing into account.

After the removal of concrete from the cross section, the cross section parameters change to reduced values. These are simply calculated for the new cross section, which is smaller due to the removal of concrete. The cross section parameters after removal, $A_{ee,r}$, $z_{ee,r}$ and $I_{ee,r}$, are calculated the same way as above, but for the smaller cross section.

6.2 Time-dependent effects

6.2.1 Prestressing losses

An important parameter for the calculations is the loss in prestressing. The loss in prestressing is here defined as all changes in stress in the prestressing throughout the life-cycle of the bridge to the losses. Hence, the elastic loss is not necessarily a loss but can also be an increase in prestressing tension. Usually loss is defined as merely loss in tension in the prestressing tendons, due to long-term effects only. The reason to why all changes in stress are considered in the losses here, is that the effects due to the repair work is better understood when also the decrease in tension is included. The loss is calculated from a point in time just after the prestressing has been tensioned, before the injections of the prestressing ducts but after the initial losses have occurred:

$$Loss = \frac{\sum_{i=1}^{n} \Delta\sigma_p(t_i)}{\sigma_{Poi}} + \frac{\Delta\sigma_{rel}(t)}{\sigma_{Poi}}, \qquad (2)$$

where:

$$\sum_{i=1}^{n} \Delta\sigma_p(t_i) = \sum_{i=1}^{n} \Delta\varepsilon_p(t_i) \cdot E_p \cdot A_p$$

n = calculations step referring to a certain point in time t.

$\sum_{i=1}^{m} \Delta\varepsilon_p(t_i)$ = the sum of all the changes in strain in the prestressing, from t_1 to t_n.

$\Delta\varepsilon_p(t_i)$ = the change in strain in the prestressing occurring at a certain time step t_i.

$\Delta\sigma_{rel}(t)$ = the prestressing steel relaxation which has occurred up until a certain time t.

σ_{Poi} = the initial stress in the prestressing tendon after initial loss, due to prestressing only.

The total strain in the prestressing steel,

$$\varepsilon_p(t) = \frac{\sigma_{Poi}}{E_p} + \sum_{i=1}^{m} \Delta\varepsilon_p(t_i),$$

is calculated in accordance with section 6.2.2 below.

6.2.2 Changes in strain

At a certain point in time t_j the strain in the concrete is defined by Equation 1, section 4.3 as:

$$\varepsilon_c(t_j) = \frac{\sigma_0}{E_c(t_0)} + \varepsilon_{cs}(t_j, t_s) + \varphi(t_j, t_0)\frac{\sigma_0}{E_c(28)} + \sum_{i=1}^{n}\left(\frac{1}{E_c(t_i)} + \frac{\varphi(t_j, t_i)}{E_c(28)}\right) \cdot \Delta\sigma(t_i)$$

The change in concrete strain that occurs between the time of t_j and another point in time t_{j+1} is:

$$\Delta\varepsilon_c(t_j) = \Delta\varepsilon_{cs}(t_j, t_s) + \Delta\varphi(t_j, t_0)\frac{\sigma_0}{E_c(28)} + \sum_{i=1}^{n}\left(\frac{1}{E_c(t_i)} + \frac{\Delta\varphi(t_j, t_i)}{E_c(28)}\right)\cdot\Delta\sigma(t_i)$$

Since the steel and the concrete in the structure are perfectly attached to one another, and regarded as a uniform structure, the strain somewhere in the cross section must be the same regardless of which structural materials are present at that location (see **Fel! Hittar inte referenskälla.**). In other words, the change in strain in the concrete from $t = t_j$ to $t = t_{j+1}$ at a given location in the cross section will equal the change in strain in any steel present at the given location, during the same time interval.

By assuming that the changes in strain behave in accordance with Bernoulli-Euler beam theory, the strain curve over the cross section can be defined by determining the strain at two locations.

6.2.3 Changes in stress

The change in stress $\Delta\sigma(t_i)$ in the structure between two points in time $t = t_i$ and $t = t_{i+1}$ can be derived from the in strain described in section 6.2.2 above. This change in stress is described as the result of a change in normal force $\Delta N(t_i)$ and moment $\Delta M(t_i)$, where:

$$\begin{cases} \Delta N = \int_A \Delta\sigma \cdot dA \\ \Delta M = \int_A z \cdot \Delta\sigma \cdot dA \end{cases}$$

$\Delta N(t_i)$ and $\Delta M(t_i)$ should equal zero. However, since the structure is continually subjected to small changes in stress due to concrete shrinkage, concrete creep and prestressing steel relaxation, small changes in normal force and moment will affect the structure during each time step. Larger changes in stress will occur when some external loading is added.

The changes in moment and normal force are determined by first determining the change in strain and curvature, as described in section 6.2.2 above. The equivalent cross-sectional parameters are then used to calculate $\Delta N(t_i)$ and $\Delta M(t_i)$:

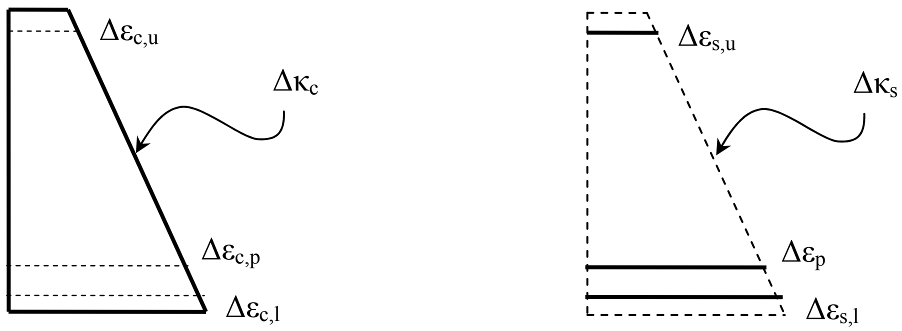
$$\begin{cases} \Delta N(t_i) = A_{ee}\cdot E_c\cdot\Delta\varepsilon(t_i) \\ \Delta M(t_i) = \Delta\kappa(t_i)\cdot E_c\cdot I_{ee} \end{cases}$$

The change in stress $\Delta\sigma(t_i)$ in the structure between two points in time $t = t_i$ and $t = t_{i+1}$ can thus be determined as:

$$\Delta\sigma(t_i) = -\left(\frac{\Delta N(t_i)}{A_{ee}} + \frac{\Delta M(t_i)}{I_{ee}}\cdot z\right)$$

where z is the vertical distance from the point in the cross section that is being considered, to the centre of gravity for the equivalent cross section.

CONCRETE STEEL



The strain at a certain location in the cross section is the same regardless of structural material ⟶

At level of upper reinforcement bars: $\Delta\varepsilon_{c,u} = \Delta\varepsilon_{s,u}$

At level of prestressing: $\Delta\varepsilon_{c,p} = \Delta\varepsilon_{p}$

At level of upper reinforcement bars: $\Delta\varepsilon_{c,l} = \Delta\varepsilon_{s,l}$

⟶ Gradient of strain curve = curvature: $\Delta\kappa_{c} = \Delta\kappa_{s}$

6.2.4 Total stress

The stresses in the concrete structure at the centres of gravity of the upper and lower reinforcement bars are used as parameters representing what happens in the structure over time. The stress in the structure at a certain time t (at a corresponding calculation step n) is the sum of the initial stress σ_0 and the sum of all changes in stress occurring from $t = t_l$ to $t = t_n$:

At upper reinforcement level:

$$\sigma_u(t) = \sigma_{0,u} + \sum_{i=1}^{n} \Delta\sigma_u(t) + \Delta\sigma_{rel,c,u}(t)$$

At lower reinforcement level:

$$\sigma_l(t) = \sigma_{0,l} + \sum_{i=1}^{n} \Delta\sigma_l(t) + \Delta\sigma_{rel,c,l}(t)$$

Since the relaxation of the prestressing steel is separated from the other changes in stress $\sum_{i=1}^{n} \Delta\sigma(t)$, the extra parameter $\Delta\sigma_{rel,c}(t)$ is added to the overall stresses. $\Delta\sigma_{rel,c}$ is the stress in the concrete inflicted by a relaxation force ΔP_{rel} in the prestressing steel:

$$\Delta P_{rel}(t) = \Delta\sigma_{rel}(t) \cdot A_p$$

7 CALCULATIONS

Initial stresses at the levels of the upper and lower reinforcement and losses in prestressing are determined for the three bridges from the bridge modelling in section 5. This initial stage takes place after the tensioning of the prestressing, after initial losses have occurred but before the injections of the prestressing ducts. After this, a calculation procedure futher described in section 7.3 is initialized. Three different calculations are carried out: one taking only addition of surfacing into account (reference calculation), another taking the surfacing and the removal of concrete into consideration, and finally, a calculation taking the whole bridge life-cycle into account, including the addition of repair concrete (see Figure 0-1).

7.1 Addition of surfacing

The addition of the surfacing structure is added to the calculation 28 days after concrete casting by determining the moment inflicted in the structure due to the added weight applied. This moment induces a change in stress in the cross section. The surfacing structure is assumed to be added instantaneously and thus the change in stress inflicted by the additional weight is added to the remaining changes in stress occurring during the timestep in question.

7.2 Repair work

The repair work entails removing a certain amount of concrete from the upper flange of the cross section, and replacing it with new concrete after some time has passed (in this case 22 days). Both the removal and the addition of concrete affect the stresses in the structure. After the removal of the concrete, the cross section properties cross-sectional area, location of centre of gravity and second area moment of inertia, will have changed. These reduced cross section parameters inflict further changes in the stress distribution. The changes in stress that the three repair stages have on the stress distribution are investigated separately below.

7.2.1 Removal of concrete

The effect of concrete constitutes a change in moment ΔM_{rem}, due to the change in weight. The removal stress due to ΔM_{rem} inflicts compression at the bottom of the cross section and tension at the top. This can be compared to subjecting the structure to an evenly distributed load, directed upwards. When calculating the stress due to concrete removal, the cross section parameters for the intact cross section (before removal of concrete) are utilized. This facilitates the isolation of the different repair effects. The removal stress is thus determined to:

$$\Delta\sigma_{rem} = \frac{\Delta M_{rem}}{I_{ee}} \cdot z \ ,$$

where z is the distance from the equivalent centre of gravity to the location in the cross section where the stress is determined. The removal stress is calculated at the levels of the upper and lower rebars, and is added to the change in stresses of the following calculation step at calculation step 5.

7.2.2 Cross section reduction

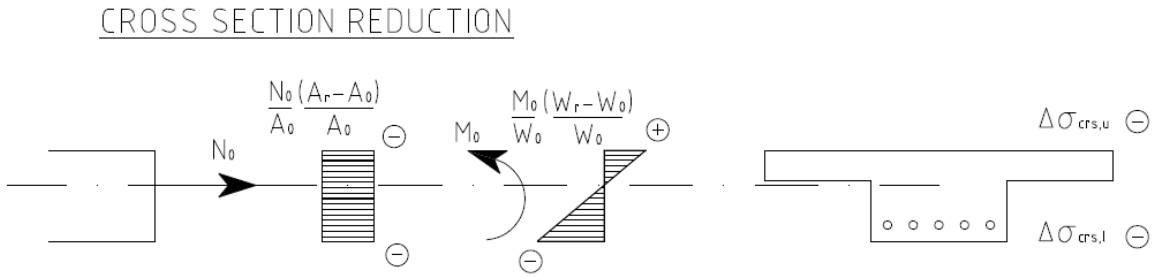


Figure 7-1: The change in stress inflicted by the reduction of the cross section parametes cross-sectional area, location of centre of gravity, and second moment of inertia.

The stress inflicted in the cross section due to the reduction of the cross section properties is depicted in Figure 7-1 above. The total moment and the total normal force in the structure directly before the removal of concrete (M_0 and N_0) are determined. M_0 is multiplied by the change in bending stiffness and N_0 with the change cross-sectional area inflicted by the removal of the concrete. The bending stiffness W is calculated as:

$$W = \frac{I}{z},$$

where z is the distance from the centre of gravity of the cross section to a specific location in the cross section (upper and lower reinforcement levels). W_0 is the bending stiffness before the removal of concrete, and W_r is the bending stiffness after the removal has taken place (see Figure 7-1). The reduction of the cross section inflicts tension at both the upper and the lower level of the cross section.

The change in stress due to cross section reduction is added to the change in stresses of the following calculation step. Every change in stress after the cross section reduction has taken place is calculated for the reduced cross section, with cross-sectional properties $A_{ee,r}$, $z_{ee,r}$ and $I_{ee,r}$ (as specified in section 6.1).

7.2.3 Addition of concrete

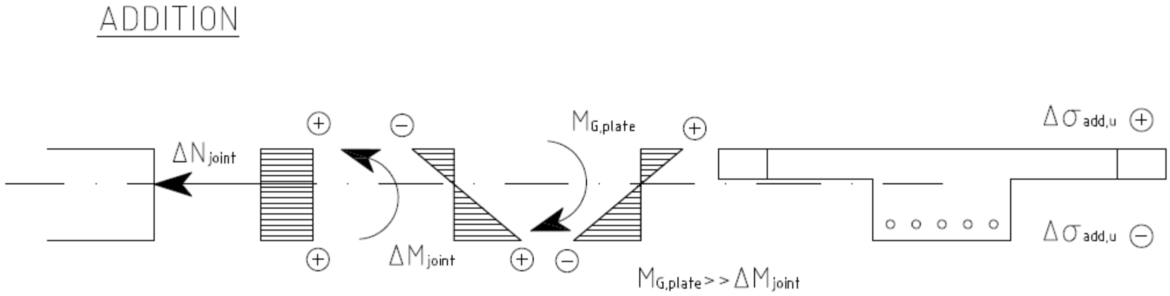


Figure 7-2: Stress inflicted by the addition of concrete.

In the calculations, the new repair concrete is modelled as a plate when it is added to the cross section. A normal force at the joint between new and old concrete (ΔN_{joint}) is inflicted, which results in a moment and a normal force at the centre of gravity of the cross section (ΔN_{joint} and ΔM_{joint}), see Figure 7-2 above. A moment due to the weight of the newly cast plate $\Delta M_{G,plate}$ also arises, at the centre of gravity of the structure. The new repair plate is not assumed to have any bending stiffness of its own. ΔM_{joint} and ΔN_{joint} arise due to the difference in creep and shrinkage between the two concrete castings. The moment due to the additional weight $\Delta M_{G,plate}$ is dominating in its effect on the stress distribution. The addition stress is calculated as:

$$\Delta\sigma_{add} = \frac{\Delta N_{joint}}{A_{ee,r}} + \frac{(\Delta M_{joint} + \Delta M_{G,plate})}{I_{ee,r}} \cdot z_r ,$$

where z_r is the distance from the centre of gravity of the reduced cross section to a specific location in the cross section (upper and lower reinforcement levels). The addition of concrete inflicts compression at the top of the cross section and tension at the bottom (see Figure 7-2 above). The stress inflicted at the upper and lower rebar levels due to the forces ΔN_{joint}, ΔM_{joint} and $\Delta M_{G,\,plate}$ are added to the change in stresses of the following calculation step. Since ΔM_{joint} and ΔN_{joint} are directly linked to the difference in creep and shrinkage between the existing structure and the repair plate, ΔM_{joint} and ΔN_{joint} will continue to change as time passes and remain as contributors to the total change in stress in the structure, throughout the remaining life-span of the bridge structure.

7.3 Calculation procedure

Firstly, before the calculation in time steps is initiated, the initial stresses in the cross section are determined. The creep strains due to the initial stresses are determined, and the shrinkage of the structure. At the very first calculation step the change in strain from $t = t_0$ until $t = t_1$, is assumed to be the change in creep and the change in shrinkage occurring during this time span:

$$\Delta\varepsilon_c(t_1) = \Delta\varepsilon_{cs}(t_1, t_s) + \Delta\varphi(t_1, t_0)\frac{\sigma_0}{E_c(28)}$$

The changes in strain at the levels of the upper and lower reinforcement are used to determine the change in strain at the equivalent centre of gravity of the cross section, and the change in curvature. These parameters are utilized to calculate the change in moment and the change in normal force for the first time step:

$$\begin{cases} \Delta N(t_1) = A_{ee} \cdot E_c \cdot \Delta\varepsilon(t_1) \\ \Delta M(t_1) = \Delta\kappa(t_1) \cdot E_c \cdot I_{ee} \end{cases}$$

ΔN and ΔM should, in fact, equal zero since no external change in stress has taken place between $t = t_0$ and $t = t_1$. They do, however, not equal zero, which indicates that the assumed value for the change in strain was an incorrect guess. In order to correct the mistake, the change in strain due to ΔN and ΔM, is calculated and added to the calculation of change in strain of the following time step:

$$\Delta\sigma(t_2) = -\left(\frac{\Delta N(t_1)}{A_{ee}} + \frac{\Delta M(t_1)}{I_{ee}} \cdot z\right)$$

For an arbitrary time step j, the change in strain is:

$$\Delta\varepsilon_c(t_j) = \Delta\varepsilon_{cs}(t_j, t_s) + \Delta\varphi(t_j, t_0)\frac{\sigma_0}{E_c(28)} + \sum_{i=1}^{n}\left(\frac{1}{E_c(t_i)} + \frac{\Delta\varphi(t_j, t_i)}{E_c(28)}\right)\cdot \Delta\sigma(t_i)$$

where $\Delta\sigma(t_i) = \Delta\sigma(t_{j+1})$. A full list of calculation steps is presented below.

7.3.1 Calculation steps

During an arbitrary time step, $t = t_i \rightarrow t = t_{i+1}$, the following calculation steps are carried out:

1. The change in strain at the levels of the upper and lower reinforcement levels (see section 6.2.2) is determined to: $\Delta\varepsilon_u(t_i)$ and $\Delta\varepsilon_l(t_i)$
2. Having determined the strain curve, the change in strain at the level of the prestressing steel $\Delta\varepsilon_p(t_i)$ is calculated. The change in strain in the prestressing is used to determine the prestressing loss (section 6.2.1).
3. $\Delta\varepsilon_u(t_i)$ and $\Delta\varepsilon_l(t_i)$ are used to determine the change in strain at the equivalent centre of gravity of the cross section $\Delta\varepsilon_c(t_i)$ and the change in curvature $\Delta\kappa_c(t_i)$.
4. $\Delta\varepsilon_c(t_i)$ and $\Delta\kappa_c(t_i)$ are used to determine the change in moment and normal force $\Delta N(t_i)$ and $\Delta M(t_i)$ (section 6.2.3)
5. $\Delta N(t_i)$ and $\Delta M(t_i)$ are used to determine the change in stress at levels of the upper and lower reinforcement $\Delta\sigma_u(t_{i+1})$ and $\Delta\sigma_l(t_{i+1})$ (section 6.2.3), which will be used to calculate the change in strain for the following time step $t = t_{i+1} \rightarrow t = t_{i+2}$.

8 RESULTS

The results from the calculations in MATLAB for the three bridges are presented below. Three different calculations are examined separately: Firstly, a reference calculation, where no repair work is carried out; secondly, a calculation including the removal and addition of concrete; and, thirdly, a calculation taking just the removal of concrete into account.

8.1 Reference calculation

In the reference calculation no repair work is carried out. The calculation takes steps 1, 2 and 5 from Figure 0-1 into consideration. This corresponds to a bridge left intact for 100 years after its construction. Below, the overall stresses, at the upper and lower reinforcement levels, for the three bridges, are presented. Moreover, the losses in prestressing for the bridges are depicted. These results are used for means of comparison with the results from the two other calculations. Hence, further results from the reference calculation are presented in sections 8.3 and 8.2.

8.1.1 Stresses

The stresses in the structure, at the levels of the upper and lower reinforcement, throughout the life-cycle of the bridge are presented below. For all three calculations, a surfacing structure is added 28 days after concrete casting. The effects the surfacing has on the stress distribution for all three calculations, is presented below.

8.1.1.1 Upper reinforcement level

The compression at the upper reinforcement increases with time. An increase in compression arises at 28 days after concrete casting due to the added surface structure. The magnitude of the stresses connected to the addition of the surfacing, as well as the initial and end stresses in the structure, are presented in Table 8-1. The stress at upper reinforcement level throughout the life-cycle of each bridge, is presented in Figure 8-1 - Figure 8-3 below.

Table 8-1: Stress distribution at upper reinforcement level. Stresses are presented for t = 14, 27 and 28 days, and for t = 100 years (end of calculation). $\Delta\sigma_{u,surf}$ is the stress contribution from the addition of the surfacing structure. Positive stress is compression [MPa].

Span length L	$\sigma_{0,u}$	σ_u(27 d)	$\Delta\sigma_{surf,u}$	σ_u(28 d)	$\sigma_{end,u}$
20 m	1.40	1.52	1,40	2.93	3.09
28 m	2.98	3.02	1.51	4.53	4.64
36 m	5.36	5.48	1.83	7.31	7.71

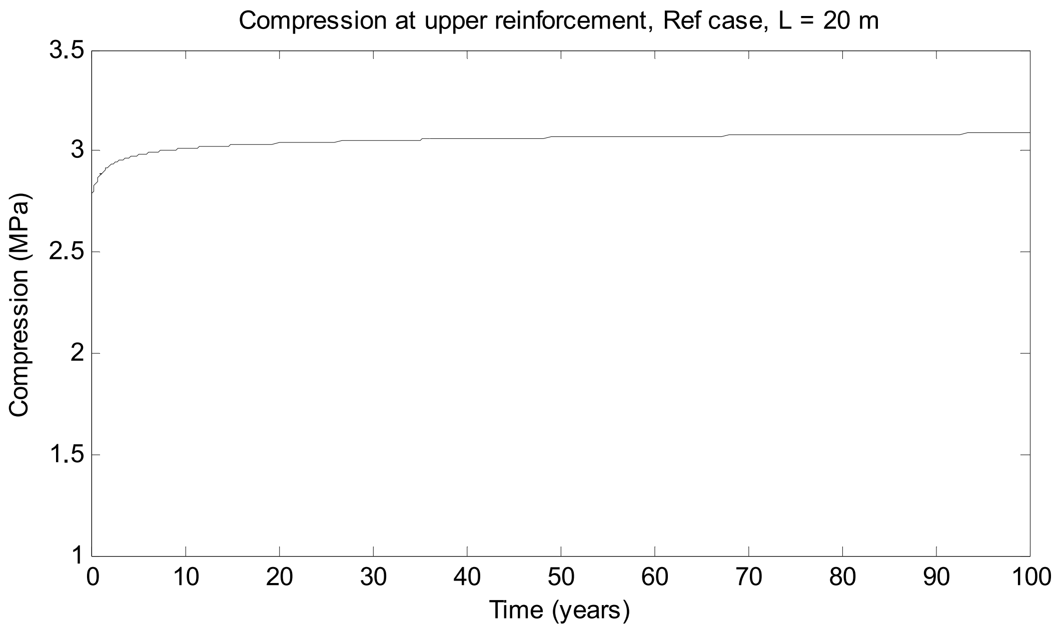


Figure 8-1: The compression at the level of the upper reinforcement for span-length L = 20m, according to the reference calculation.

Figure 8-2: The compression at the level of the upper reinforcement for span-length L = 20m, according to the reference calculation.

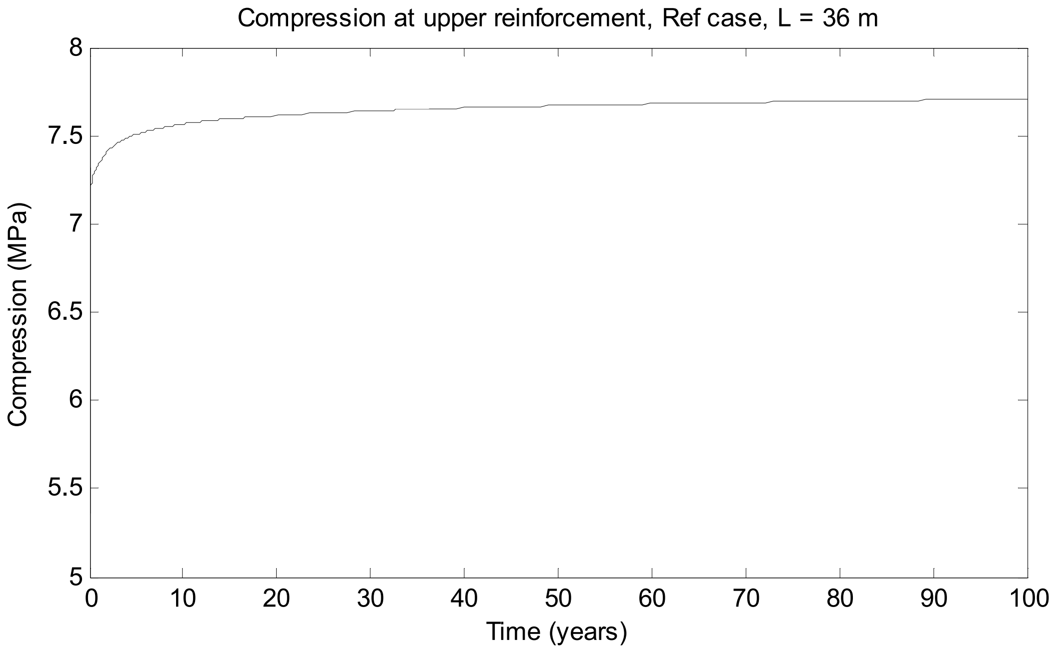


Figure 8-3: The compression at the level of the upper reinforcement for span-length L = 36m, according to the reference calculation.

8.1.1.2 Lower reinforcement level

The compression at the lower reinforcement slowly decreases with time. A decrease in compression arises at 28 days after concrete casting due to the added surface structure. The magnitude of the stresses connected to the addition of the surfacing, as well as the initial and end stresses in the structure, are presented in Table 8-2. The stress at lower reinforcement level as a funtion of time, for each bridge, is presented in Figure 8-4 - Figure 8-6 below.

Table 8-2: Stress distribution at lower reinforcement level. Stresses are presented for t = 14, 27 and 28 days, and for t = 100 years (end of calculation). $\Delta\sigma_{surf,l}$ is the stress contribution from the addition of the surfacing structure. Positive stress is compression [MPa].

Span length L	$\sigma_{0,l}$	σ_l(27 d)	$\Delta\sigma_{surf,l}$	σ_l(28 d)	$\sigma_{end,l}$
20 m	9.67	9.04	-2,64	6,39	3,22
28 m	8.38	7.61	-2.29	5.31	2.04
36 m	7.92	7.00	-2.64	4.34	3.78

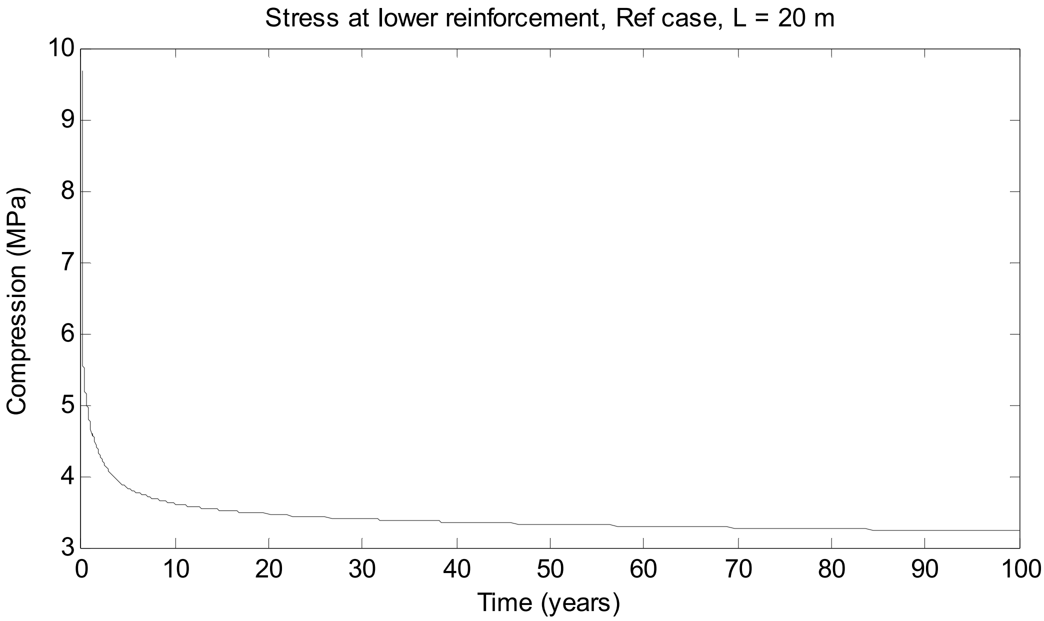


Figure 8-4: The compression at the level of the lower reinforcement for span-length L = 20 m, according to the reference calculation.

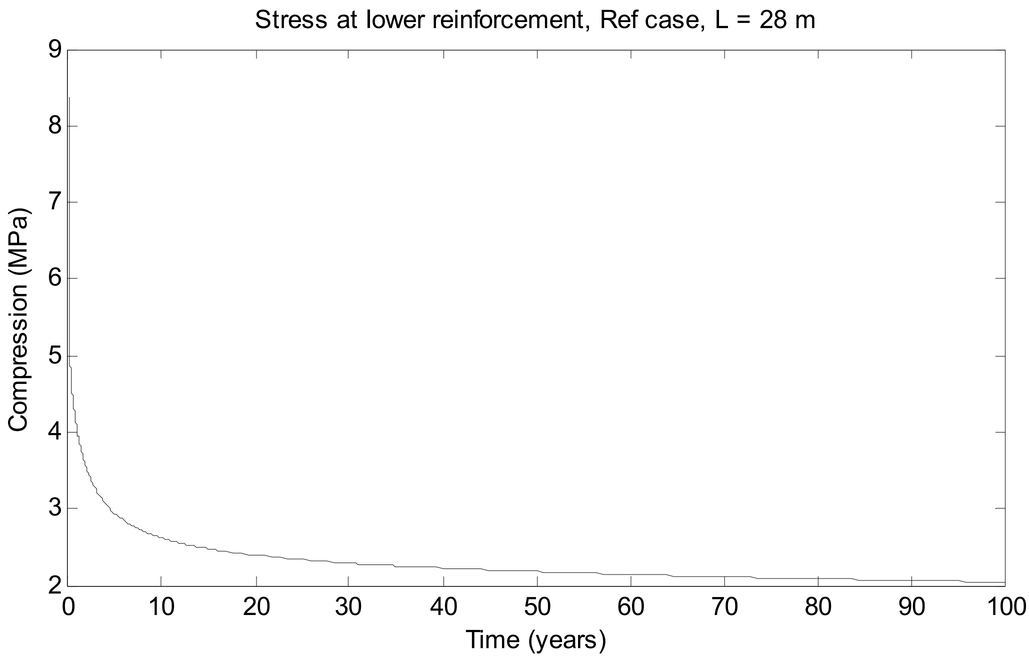


Figure 8-5: The compression at the level of the lower reinforcement for span-length L = 28 m, according to the reference calculation.

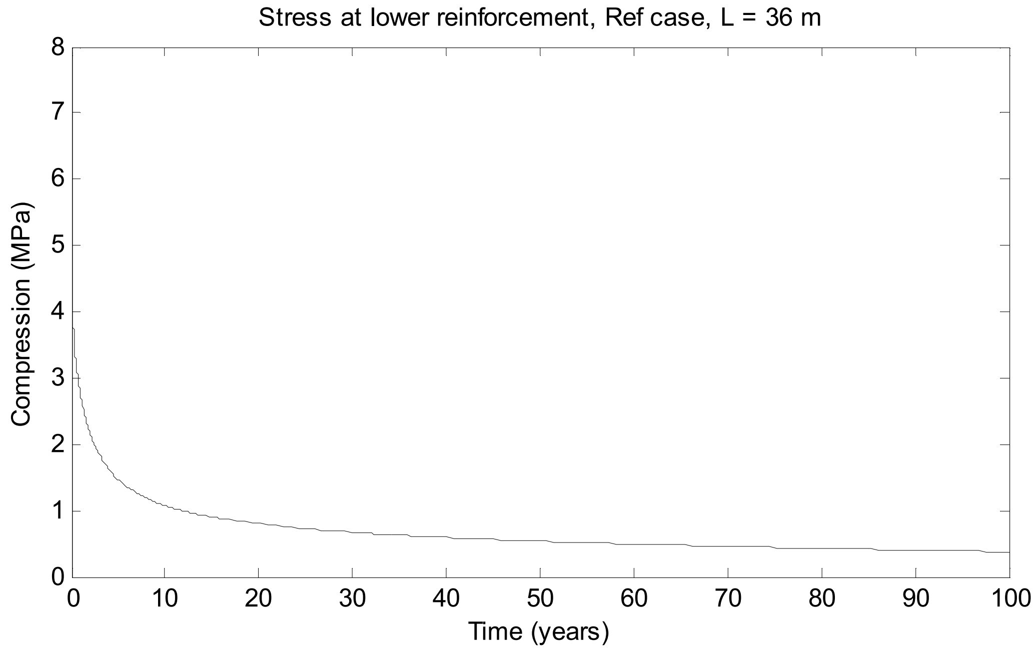


Figure 8-6: The compression at the level of the lower reinforcement for span-length L = 36 m, according to the reference calculation.

8.1.2 Loss in tension

In Figure 8-7 - Figure 8-9 below, the loss in prestressing for the three bridges is depicted. For all three span-lengths the loss after 100 years is about 18%. The lowest loss is obtained for the bridge with the shortest span-length (20 meters).

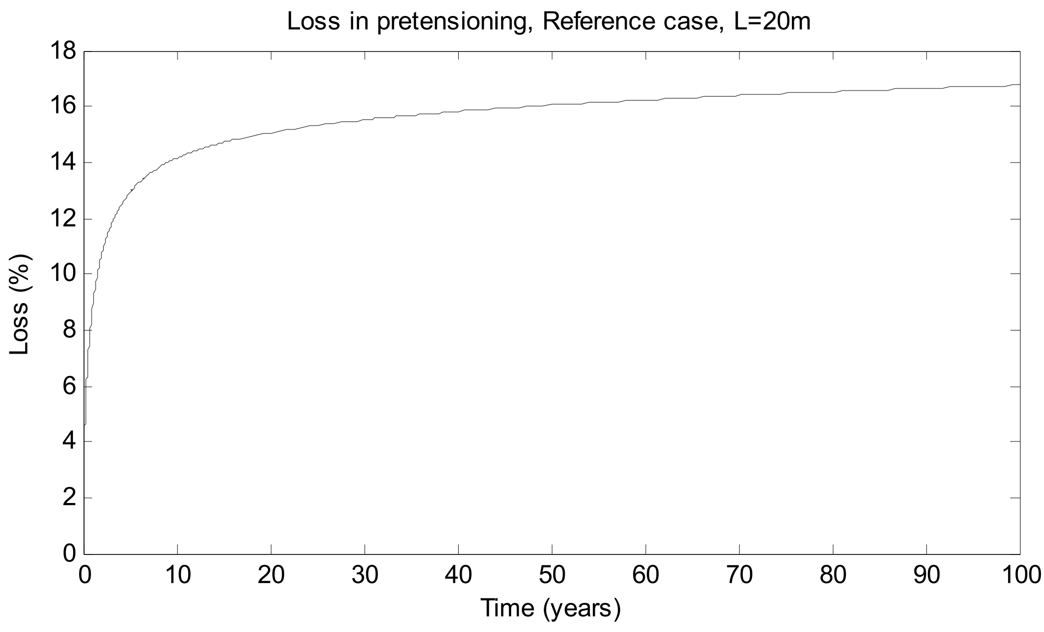


Figure 8-7: Loss in tension (%) in the prestressing, for bridge with span-length L = 20 m.

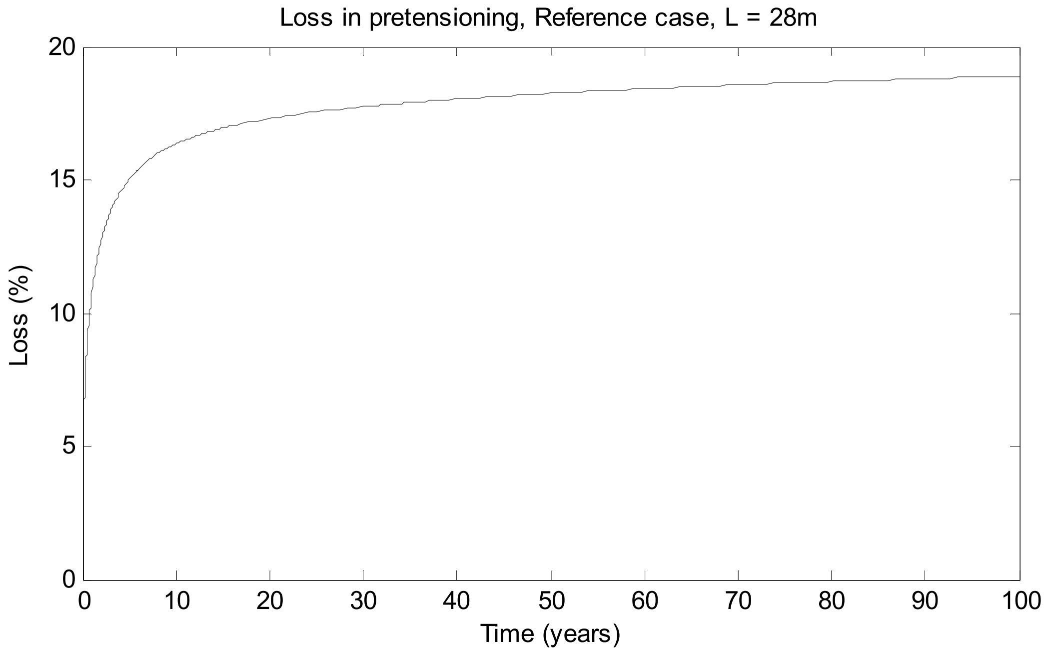


Figure 8-8: Loss in tension (%) in the prestressing, for bridge with span-length L = 28 m.

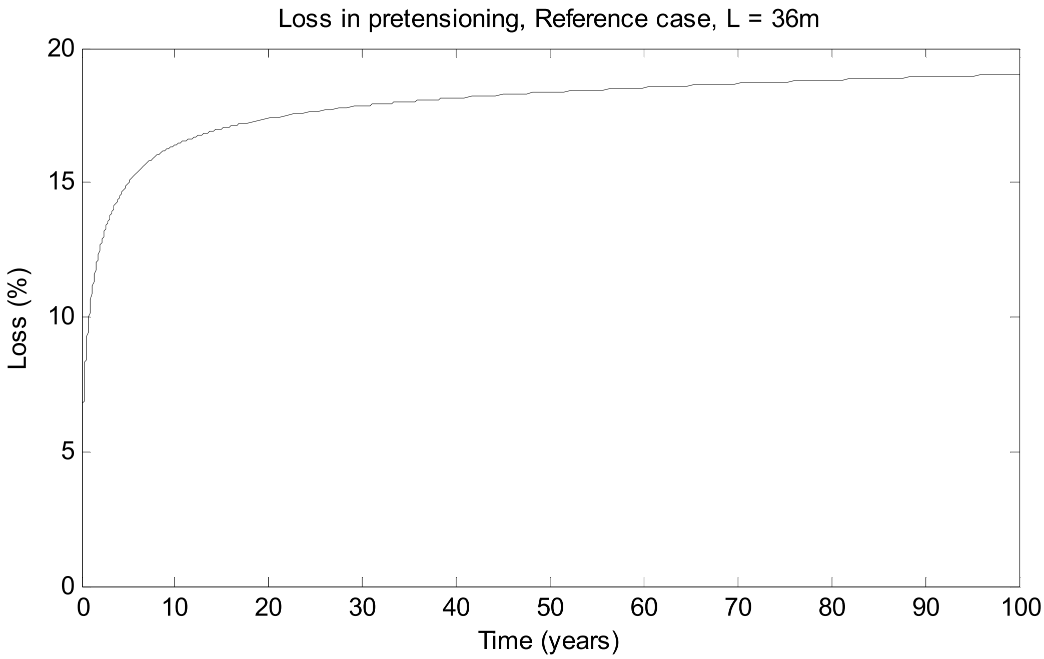


Figure 8-9: Loss in tension (%) in the prestressing, for bridge with span-length L = 36 m.

8.2 Removal of concrete

The second calculation involves concrete removal after 30 years, with reduction of cross section properties as a consequence, but no addition of new concrete to the structure. The calculation takes steps 1, 2, 3 and 5 from Figure 0-1 into account, and can be compared to a structure where deterioration of concrete has occurred but where the structure has been left unattended. Calculating the stresses and the prestressing losses due to just removal is of interest in respect of comparison.

8.2.1 Stresses

Below, the overall stresses as concrete is removed are presented. These stresses must remain within the range between the tensile strength of the concrete $f_{ctk,\,0.05}$ (5% fractile), and the upper limit of $0.45f_{ck}$. The change in stress due to the removal of concrete and the reduction of the cross section geometry still remain the same as presented in Table 8-8.

8.2.1.1 Upper reinforcement level

In Table 8-3 below, the maximum and minimum stresses, and the end stress at the end of the calculation (at 100 years) are compared to the corresponding values from the reference calculation (calculation without removal) and the stress limits defined above. In Figure 8-10 - Figure 8-15 the compression at the upper reinforcement level for two removal cases are depicted; removal of 300 mm and removal of 1800 mm (entire upper flange). The compressive stress clearly decreases at the removal of the concrete, and remains at a permanently lower level than the compression in the reference calculation, for all three bridges.

Table 8-3: Maximum, minimum and end stress (at 100 years) at upper reinforcement (in bold) compared to limit for un-linear behaviour and cracking (in italics), respectively, as well as to reference calculation (without removal of concrete). Compression is positive. r is the amount of concrete removed from each flange.

	L = 20m			L = 28m			L = 36m		
r (m)	1.80	1.30	0.30	1.80	1.30	0.30	1.80	1.30	0.30
Max σ_u	**3.30**	**3.30**	**3.30**	**4.59**	**4.59**	**4.59**	**7.64**	**7.64**	**7.64**
LIMIT 0.45 f_{ck}	15.75	15.75	15.75	15.75	15.75	15.75	15.75	15.75	15.75
Max σ_u reference	*3.32*	*3.32*	*3.32*	*4.64*	*4.64*	*4.64*	*7.71*	*7.71*	*7.71*
Min σ_u	**0.59**	**1.34**	**1.40**	**0.54**	**1.67**	**2.98**	**2.03**	**3.60**	**5.36**
LIMIT $f_{ctk0.05}$	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20
Min σ_u reference	*1.40*	*1.40*	*1.40*	*2.98*	*2.98*	*2.98*	*5.36*	*5.36*	*5.36*
End σ_u	**0.66**	**1.41**	**2.88**	**0.87**	**1.88**	**4.00**	**2.47**	**3.88**	**6.81**
End σ_u reference	*3.32*	*3.32*	*3.32*	*4.64*	*4.64*	*4.64*	*7.71*	*7.71*	*7.71*

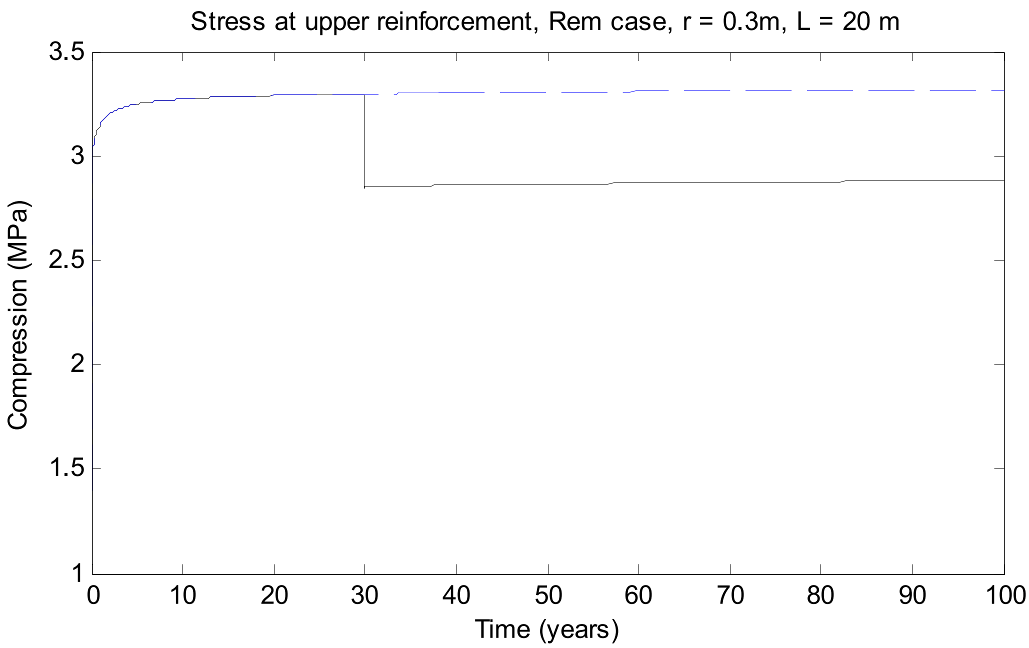


Figure 8-10: Compression at upper reinforcement level as a function of time for span-length L = 20m, when 300 mm is removed from each flange. The dashed line is the corresponding stress from the reference calculation.

Figure 8-11: Compression at upper reinforcement level as a function of time for span-length L = 20m, when 1800 mm is removed from each flange (entire flange). The dashed line is the corresponding stress from the reference calculation.

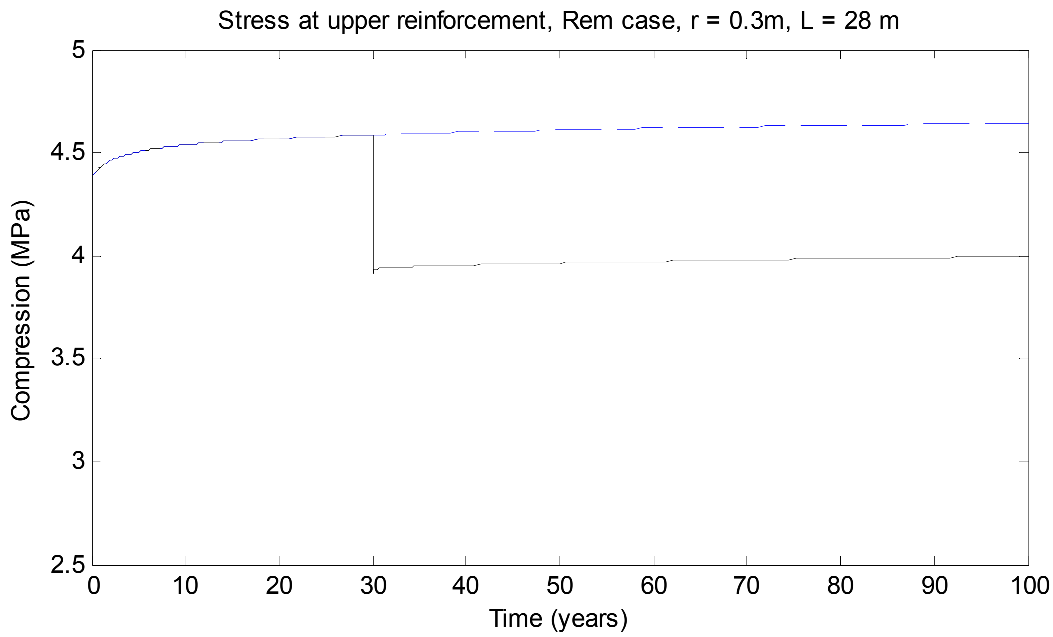


Figure 8-12: Compression at upper reinforcement level as a function of time for span-length L = 28m, when 300 mm is removed from each flange. The dashed line is the corresponding stress from the reference calculation.

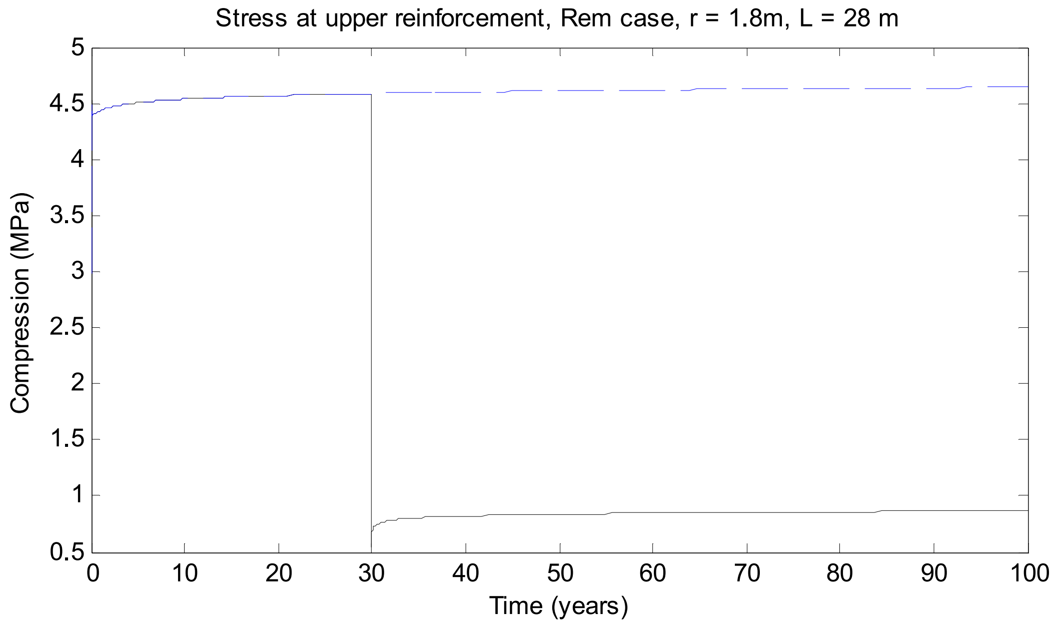


Figure 8-13: Compression at upper reinforcement level as a function of time for span-length L = 28m, when 1800 mm is removed from each flange (entire flange). The dashed line is the corresponding stress from the reference calculation.

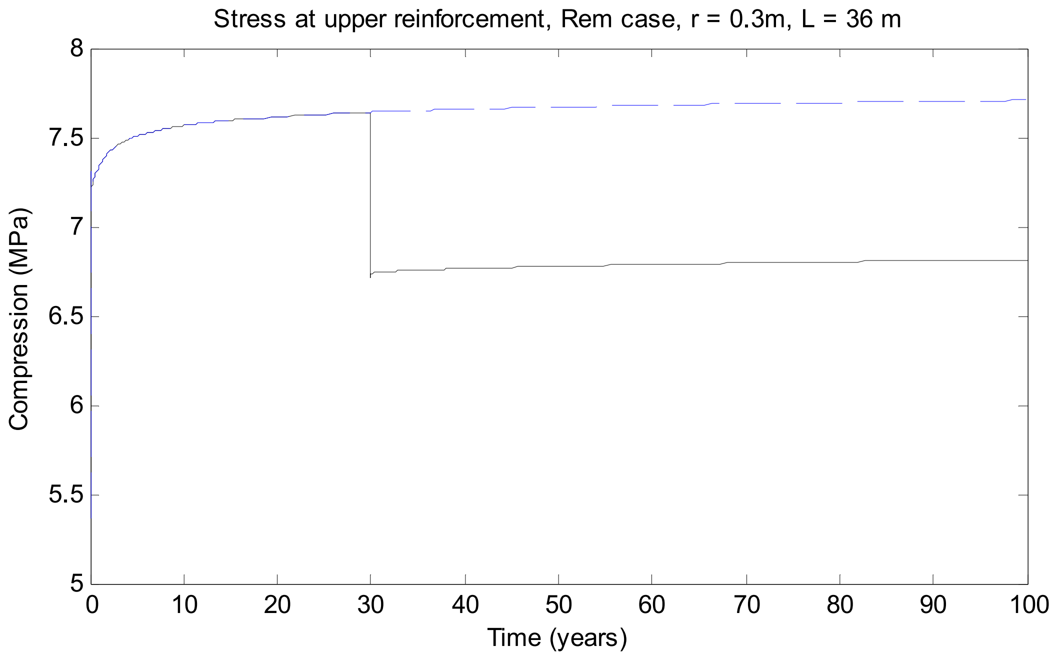


Figure 8-14: Compression at upper reinforcement level as a function of time for span-length L = 36m, when 300 mm is removed from each flange. The dashed line is the corresponding stress from the reference calculation.

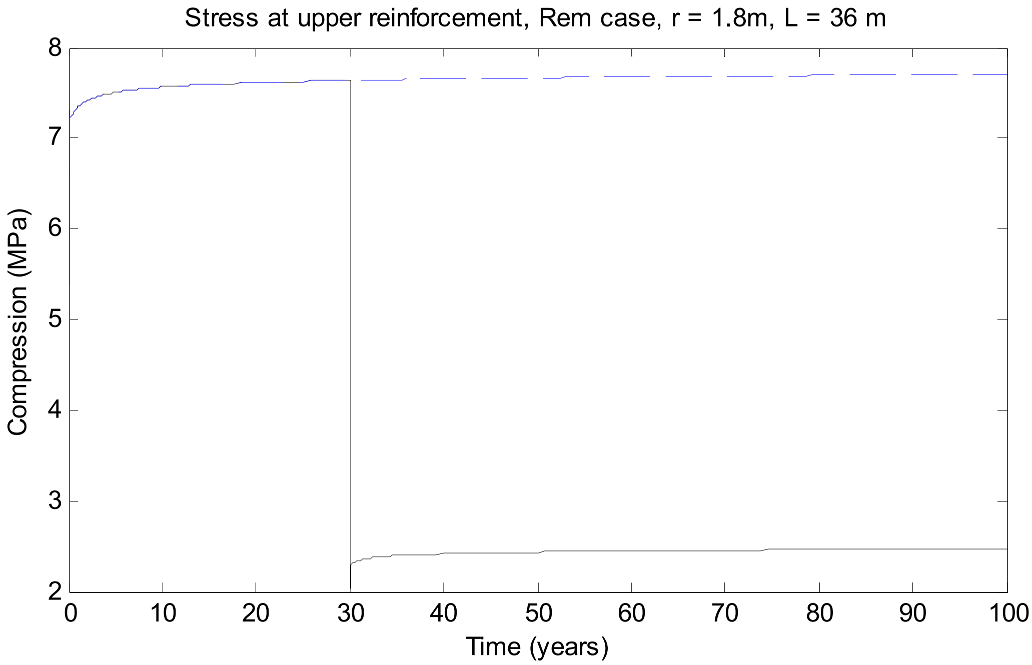


Figure 8-15: Compression at upper reinforcement level as a function of time for span-length L = 36m, when 1800 mm is removed from each flange (entire flange). The dashed line is the corresponding stress from the reference calculation.

8.2.1.2 Lower reinforcement level

In Table 8-4 below, the maximum and minimum stresses, and the end stress at the end of the calculation (at 100 years) are compared to the corresponding values from the reference calculation (calculation without removal) and the stress limits defined above. In Figure 8-16 - Figure 8-21 the compression at the lower reinforcement level for two removal cases are depicted; removal of 300 mm and removal of 1800 mm (entire upper flange). The compressive stress clearly increases at the removal of the concrete, and remains at a permanently elevated level than the compression in the reference calculation, for all three span-lengths.

Table 8-4: Maximum, minimum and end stress (at 100 years) at lower reinforcement (in bold) compared to limit for un-linear behaviour and cracking (in italics), respectively, as well as to reference calculation (without removal of concrete). Compression is positive. r is the amount of concrete removed from each flange.

	L = 20m			L = 28m			L = 36m		
r (m)	1.80	1.30	0.30	1.80	1.30	0.30	1.80	1.30	0.30
Max σ_l	**9.67**	**9.67**	**9.67**	**8.38**	**8.38**	**8.38**	**7.92**	**7.92**	**7.92**
LIMIT 0.45 f_{ck}	15.75	15.75	15.75	15.75	15.75	15.75	15.75	15.75	15.75
Max σ_l reference	*9.67*	*9.67*	*9.67*	*8.38*	*8.38*	*8.38*	*7.92*	*7.92*	*7.92*
Min σ_l	**3.40**	**3.40**	**3.40**	**2.29**	**3.40**	**3.40**	**0.67**	**0.67**	**0.67**
LIMIT $f_{ctk0.05}$	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20
Min σ_l reference	*3.22*	*3.22*	*3.22*	*2.04*	*2.04*	*2.04*	*3.76*	*3.76*	*3.76*
End σ_l	**9.07**	**7.41**	**4.18**	**5.09**	**4.23**	**2.54**	**4.39**	**3.21**	**1.01**
End σ_l reference	*3.22*	*3.22*	*3.22*	*2.04*	*2.04*	*2.04*	*3.78*	*3.78*	*3.78*

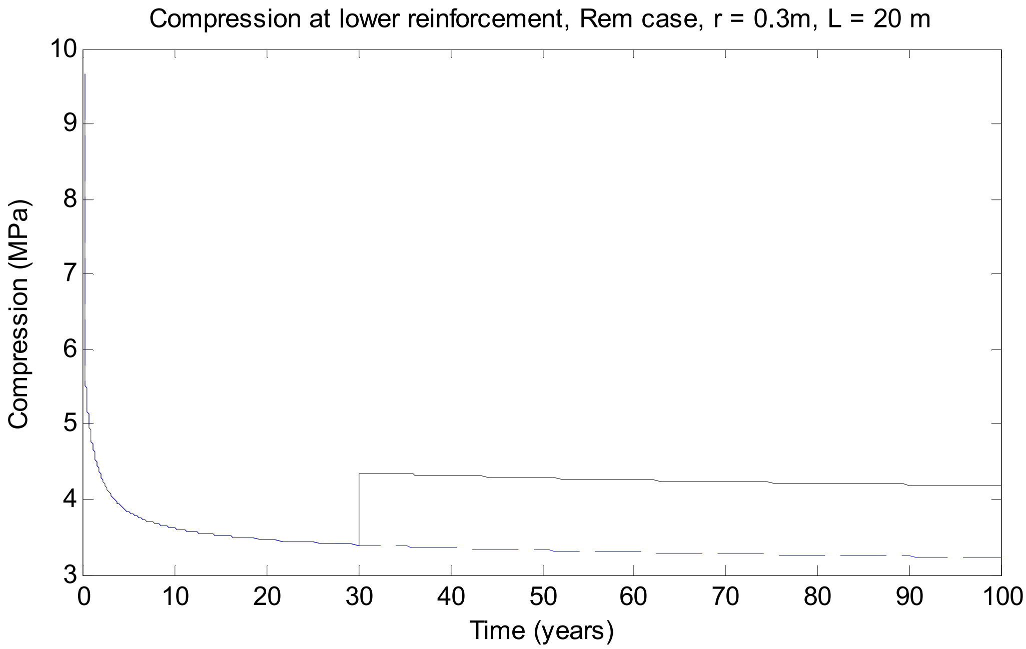


Figure 8-16: Compression at lower reinforcement level as a function of time for span-length L = 20m, when 300 mm is removed from each flange. The dashed line is the corresponding stress from the reference calculation.

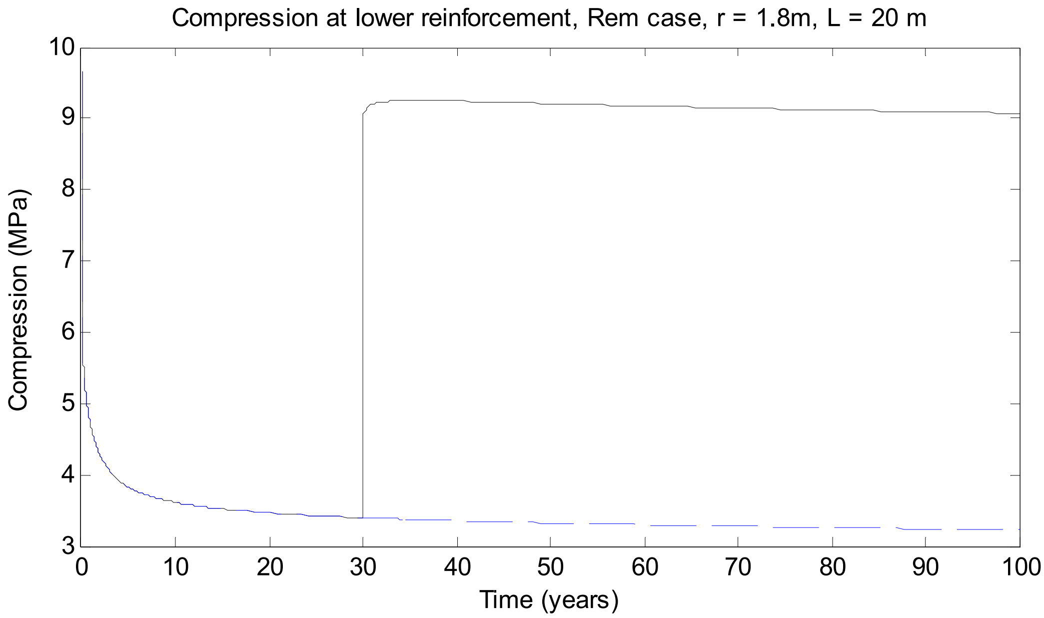


Figure 8-17: Compression at lower reinforcement level as a function of time for span-length L = 20m, when 1800 mm is removed from each flange (entire flange). The dashed line is the corresponding stress from the reference calculation.

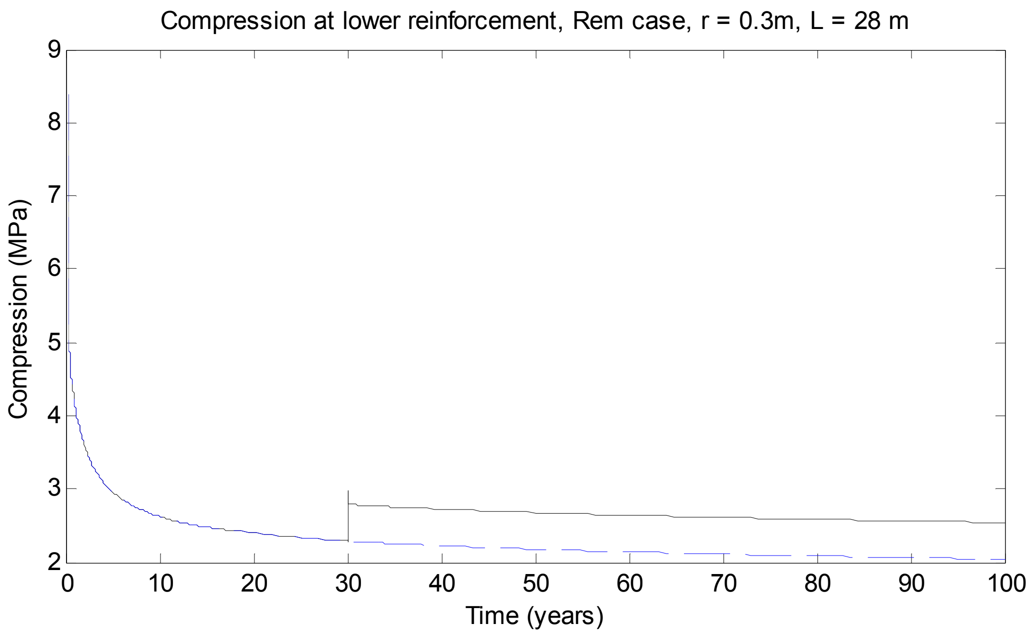


Figure 8-18: Compression at lower reinforcement level as a function of time for span-length L = 28m, when 300 mm is removed from each flange. The dashed line is the corresponding stress from the reference calculation.

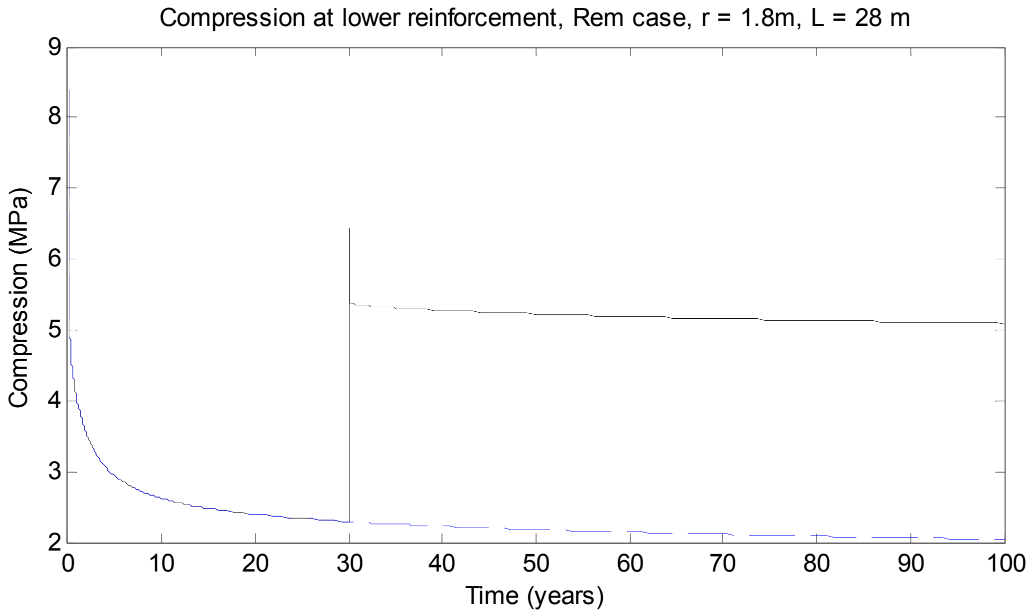


Figure 8-19: Compression at lower reinforcement level as a function of time for span-length L = 28m, when 1800 mm is removed from each flange (entire flange). The dashed line is the corresponding stress from the reference calculation.

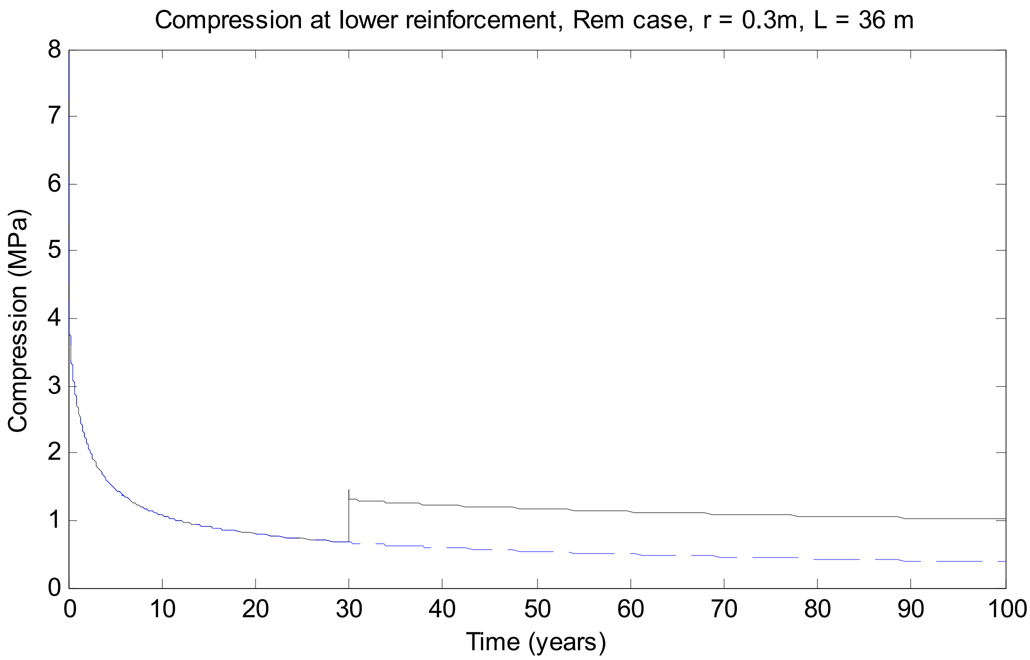


Figure 8-20: Compression at lower reinforcement level as a function of time for span-length L = 36m, when 300 mm is removed from each flange. The dashed line is the corresponding stress from the reference calculation.

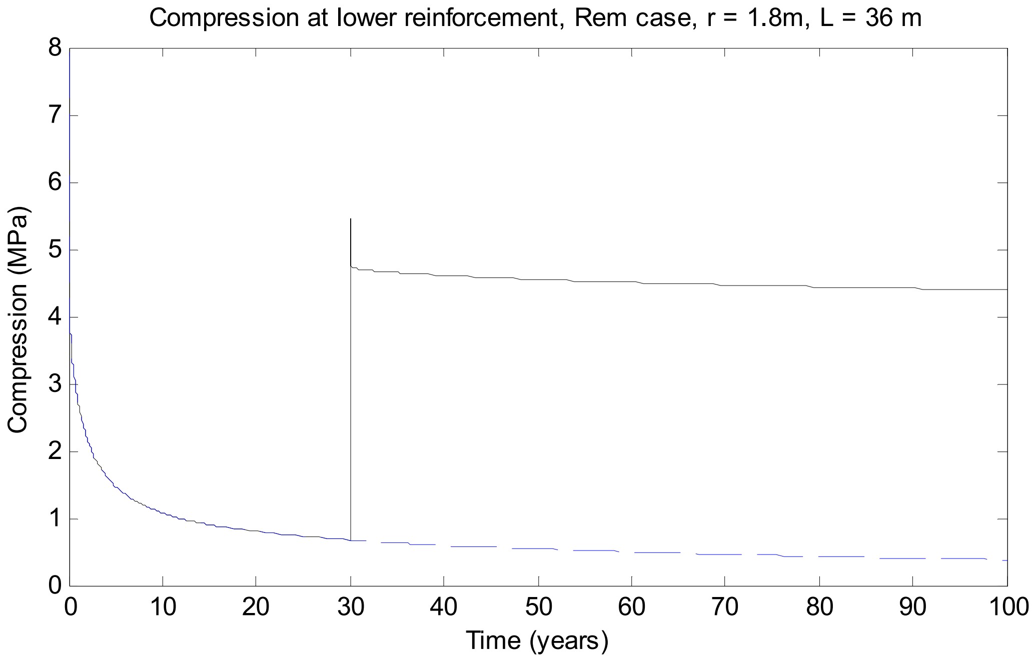


Figure 8-21: Compression at lower reinforcement level as a function of time for span-length L = 36m, when 1800 mm is removed from each flange (entire flange). The dashed line is the corresponding stress from the reference calculation.

8.2.1.3 Loss in prestressing

The long-term loss in prestressing is presented in Table 8-5 below. The loss is directly linked to the stresses invoked due to the repair procedure, presented in Table 8-8. The end loss (at 100 years) for the three removal cases of each bridge span-length, are compared to the end loss of the reference calculation. In Figure 8-22 - Figure 8-27 below the losses are depicted. The prestressing losses for two cases are presented, for each span-length; the removal of 300 mm and the removal of the entire upper flange (1800 mm). The figures show that, the more concrete is removed, the larger the decrease in prestressing loss. The loss decreases since the structural weight decreases due to the removal of concrete.

Table 8-5: Loss in prestressing with elastic effects included. The maximum loss, the end loss at 100 years and the end loss according to the reference calculation (calculation without removal of concrete) are presented in %.

	L = 20m			**L = 28m**			**L = 36m**		
r (m)	**1.80**	**1.30**	**0.30**	**1.80**	**1.30**	**0.30**	**1.80**	**1.30**	**0.30**
End loss (%)	16.5	17.5	19.4	17.6	18.0	18.7	17.5	18.0	18.8
End loss reference (%)	*20.0*	*20.0*	*20.0*	*18.9*	*18.9*	*18.9*	*19.0*	*19.0*	*19.0*

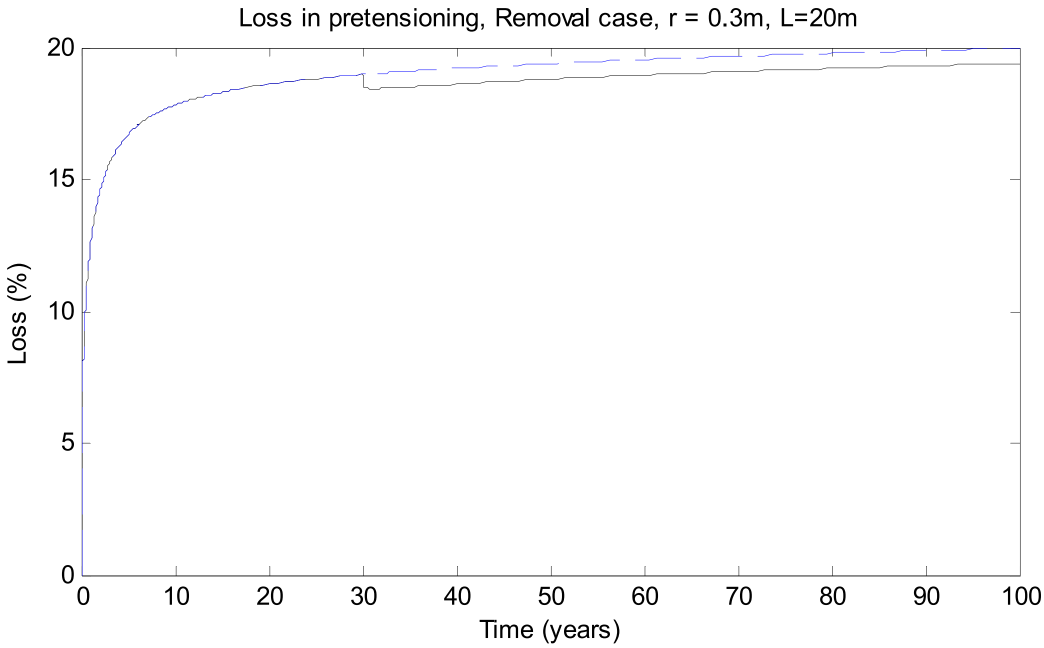


Figure 8-22: Prestressing loss with elastic effects included for bridge with span-length L = 20m, when 300 mm is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

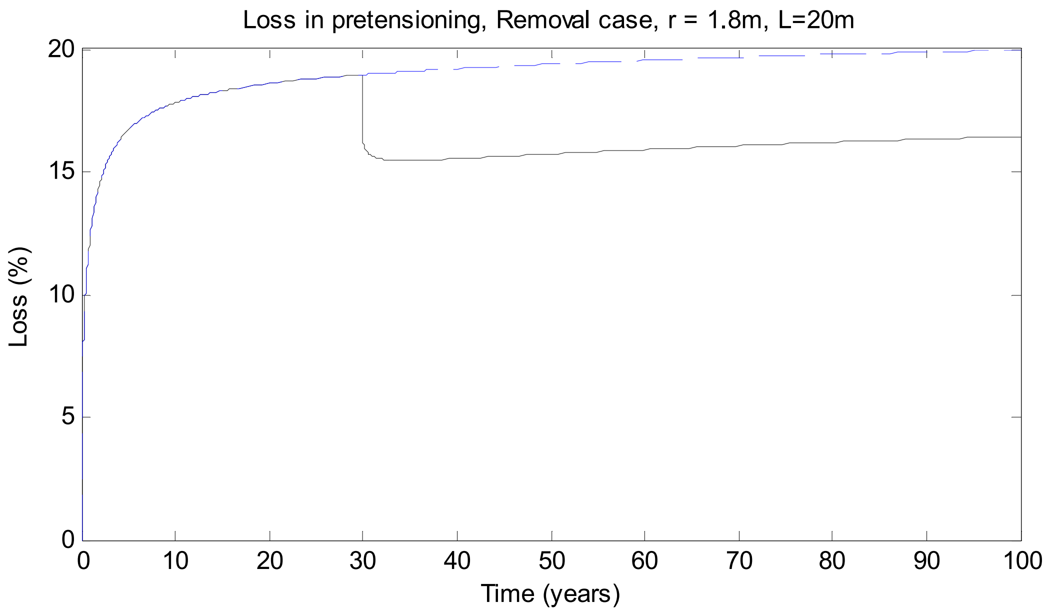


Figure 8-23: Prestressing loss with elastic effects included for bridge with span-length L = 20m, when 1800 mm (entire flange) is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

Figure 8-24: Prestressing loss with elastic effects included for bridge with span-length L = 20m, when 300 mm is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

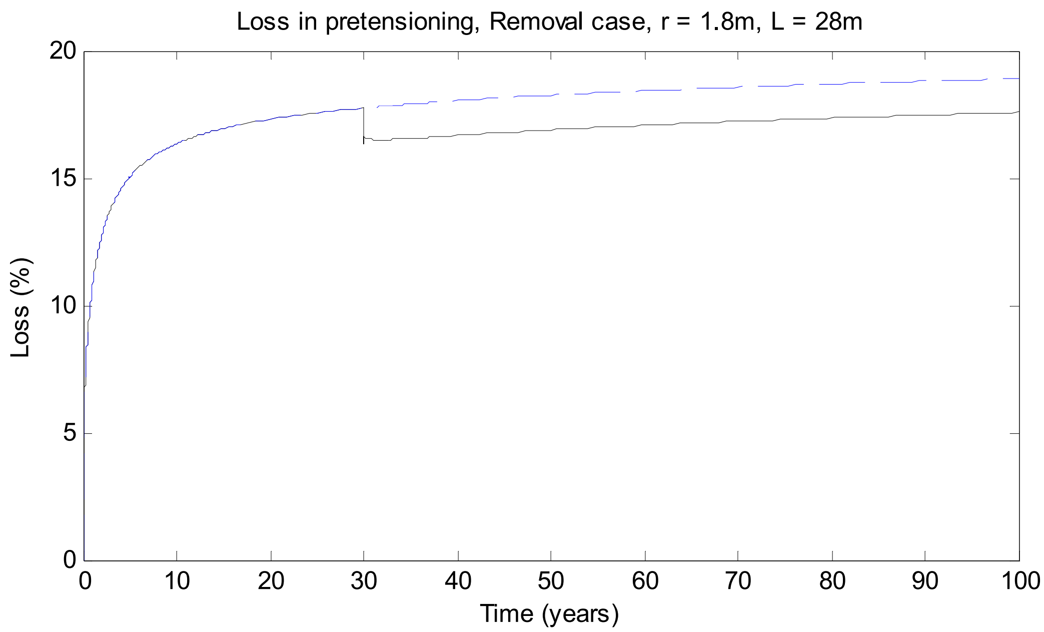


Figure 8-25: Prestressing loss with elastic effects included for bridge with span-length L = 28m, when 1800 mm (entire flange) is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

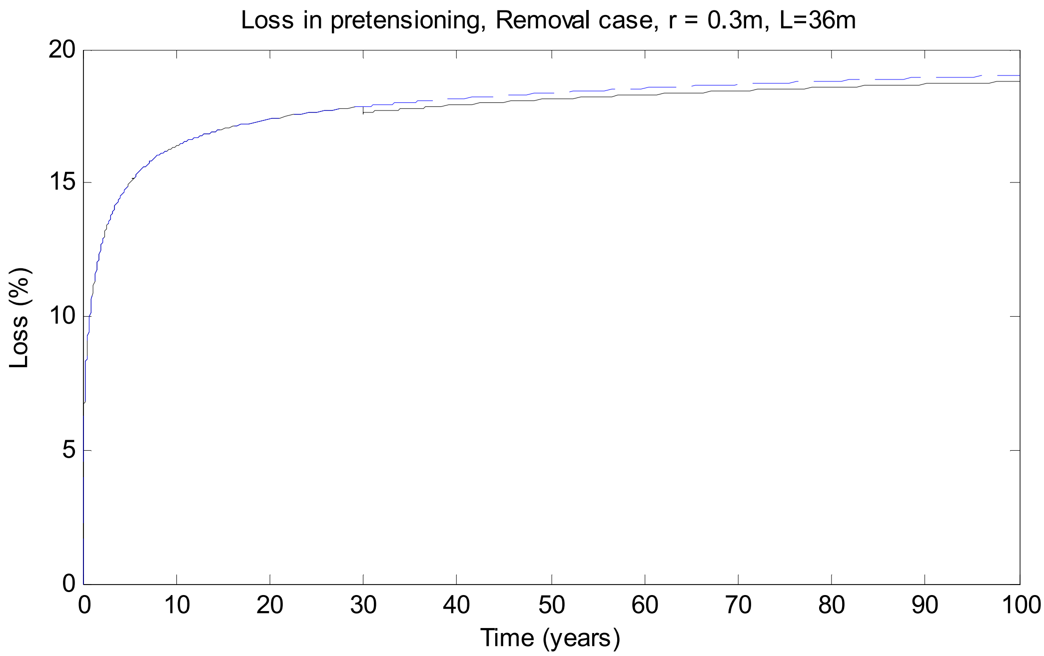


Figure 8-26: Prestressing loss with elastic effects included for bridge with span-length L = 20m, when 300 mm is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

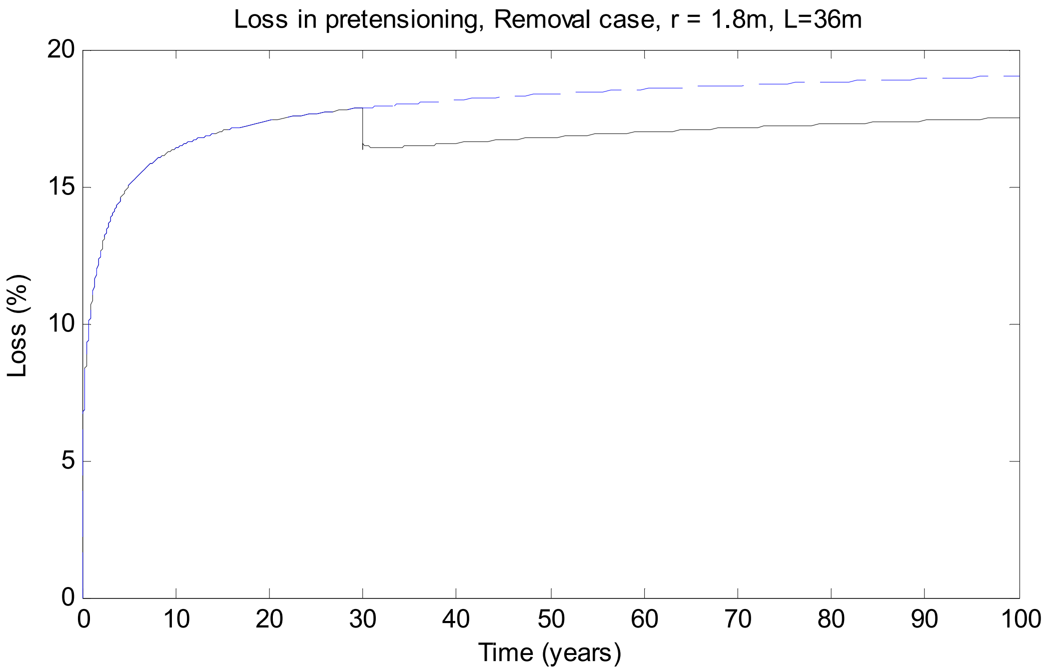


Figure 8-27: Prestressing loss with elastic effects included for bridge with span-length L = 36m, when 1800 mm (entire flange) is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

8.2.2 Summary

When concrete is removed, the compression at upper reinforcement level decreases due to the lower weight of the structure. According to the results, no tensile stresses arise. However, if any more concrete is removed there is a risk of tensile stresses arising. At the level of the lower reinforcement the result of the removal of concrete is an increase in compression, as the effect of the prestressing becomes more prominent with less weight. The compression never reaches the limit for when concrete creep starts to behave non-linearly. The losses decrease after the removal of concrete as a result of the lower weight. When very little concrete is removed, the decrease is negligible. When both the upper flanges are removed from the cross section, the decrease in loss after 100 years amounts to some 2-3%.

8.3 Removal and addition of concrete

Here, the entire repair work procedure is taken into account, corresponding to the entire life-cycle of the bridge in Figure 0-1 (steps 1-5). The bridge requires repairs 30 years after its construction. Concrete is removed, and then replaced, 22 days after the removal. Below, the effect on the stress distribution at the levels of the upper and lower reinforcement, throughout the life-cycle of the bridges, is examined. The changes in stress caused by the repair work procedure, and the losses in prestressing are, furthermore, presented. Lastly, the risk of cracking in the new repair concrete is evaluated.

8.3.1 Stresses

The stresses in the structure are presented below. First the overall stresses in the structure, at the upper and lower reinforcement levels, are presented. These stresses must remain within the range between the tensile strength of the concrete $f_{ctk,\,0.05}$ (5% fractile), and the upper limit of $0.45f_{ck}$. The Eurocode states that if the compressive stress in the concrete structure reaches a permanent value exceeding a limit of 45% of the concrete compressive strength f_{ck}, the non-linearity of creep must be considered (2, section 5.10.2.2 (5)). Since the non-linearity of concrete has not been considered in these calculations, the results can only be considered if the compression keeps below the limit value of $0.45f_{ck}$.

After the overall stresses, the change in stress due to repair work is examined. These stresses correspond to the effects presented in section 7.2. The overall stress distribution, at the calculations steps where the repair work is carried out, is here, furthermore, presented. Lastly, the risk for cracking of the new repair concrete is evaluated.

8.3.1.1 Upper reinforcement level

The magnitude of the stress at the centre of gravity of the upper reinforcement for the three bridges, with span lengths of 20, 28 and 36 meters, are presented below. In

Table 8-6, the stresses in the concrete are compared to the stress limits defined and to the results from the reference calculation presented in section 8.1 above. The maximum and minimum stresses are presented, as well as the end values, i.e. the stress at the end of the calculation when t = 100 years. The stresses neither approach the specified upper value of 15.75 MPa, which is the limit for concrete creep non-linearity specified in Eurocode, nor the lower value -2.2 MPa, which is the tensile strength of the concrete.

Table 8-6: Maximum, minimum and end stress at upper reinforcement compared to limit for un-linear behaviour and cracking, respectively, as well as to reference calculation (without repair work). Compression is positive. r is the amount of concrete removed from each flange.

	L = 20m			L = 28m			L = 36m		
r (m)	1.80	1.30	0.30	1.80	1.30	0.30	1.80	1.30	0.30
Max σ_u	**7.96**	**4.81**	**3.36**	**6.03**	**4.78**	**4.68**	**10.70**	**8.97**	**7.75**
LIMIT 0.45 f_{ck}	15.75	15.75	15.75	15.75	15.75	15.75	15.75	15.75	15.75
Max σ_u reference	*3.32*	*3.32*	*3.32*	*4.64*	*4.64*	*4.64*	*7.71*	*7.71*	*7.71*
Min σ_u	**0.59**	**1.34**	**1.40**	**0.54**	**1.67**	**2.98**	**4.33**	**5.25**	**5.36**
LIMIT $f_{ctk0.05}$	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20
Min σ_u reference	*3.32*	*3.32*	*3.32*	*4.64*	*4.64*	*4.64*	*7.71*	*7.71*	*7.71*
End σ_u	**7.96**	**4.81**	**3.36**	**5.49**	**4.56**	**4.68**	**10.20**	**8.77**	**7.75**
End σ_u reference	*3.32*	*3.32*	*3.32*	*4.64*	*4.64*	*4.64*	*7.71*	*7.71*	*7.71*

In
Table 8-6 above the impact of the different repair cases can also be seen. The more concrete that is removed and replaced, the larger the stress variations obtained in the structure. The largest compressive stress occurs after the addition of new concrete. The compression is larger the more concrete is removed. The time of occurrence of the smallest compressive stresses and the tensile stresses coincides with the reduction of the cross section.

In Figure 8-28 - Figure 8-33 below, the stresses at the level of the upper reinforcement, for when 300 mm and when the entire flange is removed, are presented for all three span-lengths. As the figures show, the stresses in the three bridges behave similarly. The repair procedure results in variations in stress, tension as concrete is removed and compression when concrete is added. From a long-term perspective, the removal and replacement of just 300 mm on each flange hardly affects the stresses. The removal and replacement of the entire flange invokes increased compression.

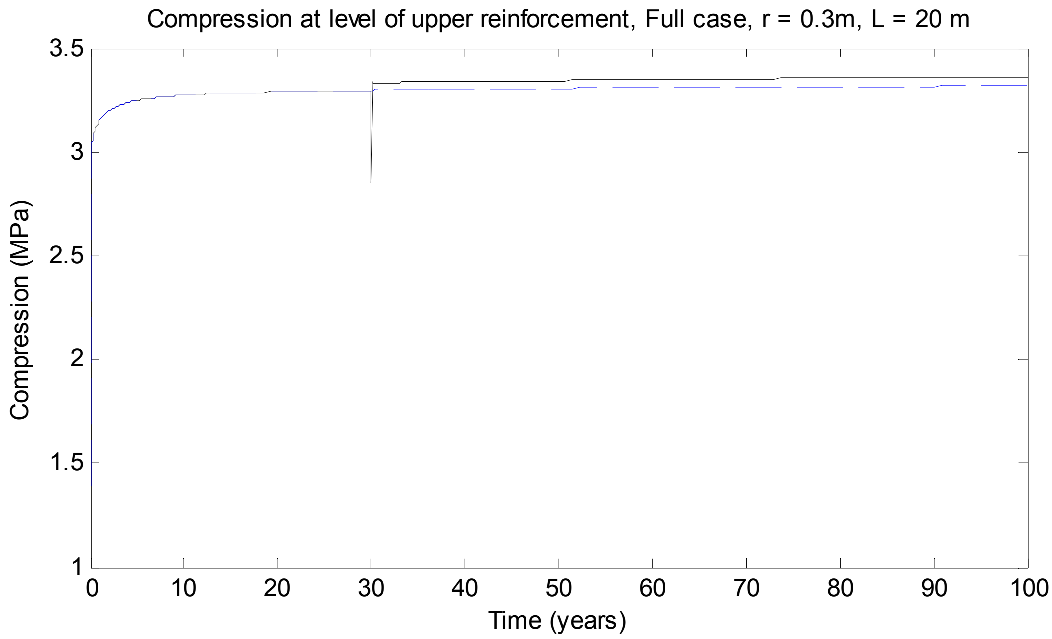


Figure 8-28: Compression at upper reinforcement level as a function of time for span-length L = 20m, when 300 mm is removed and replaced from each flange. The dashed line is the corresponding stress from the reference calculation.

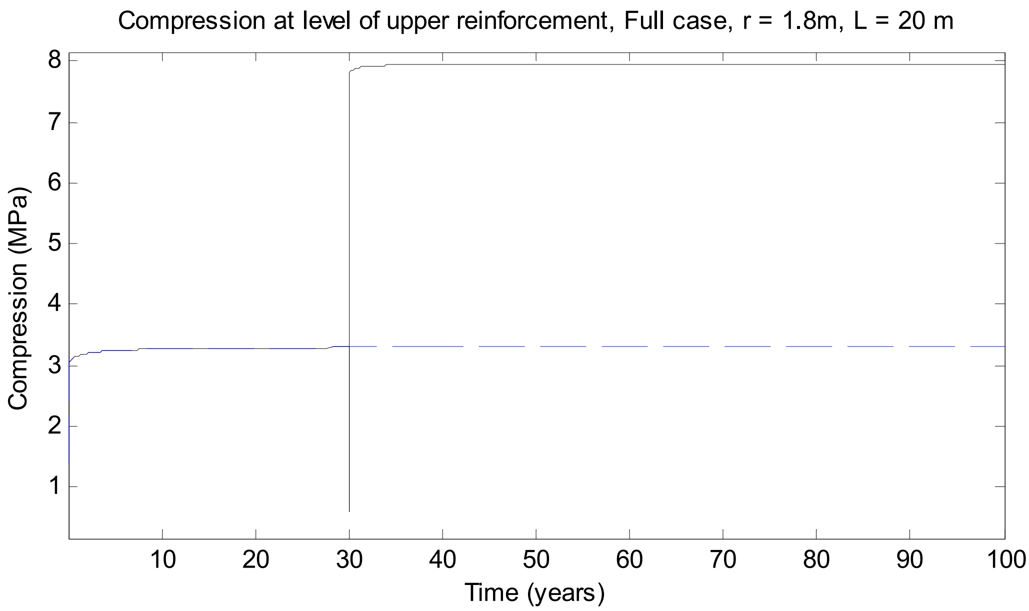


Figure 8-29: Compression at upper reinforcement level as a function of time for span-length L = 20m, when the entire flange is removed and replaced. The dashed line is the corresponding stress from the reference calculation.

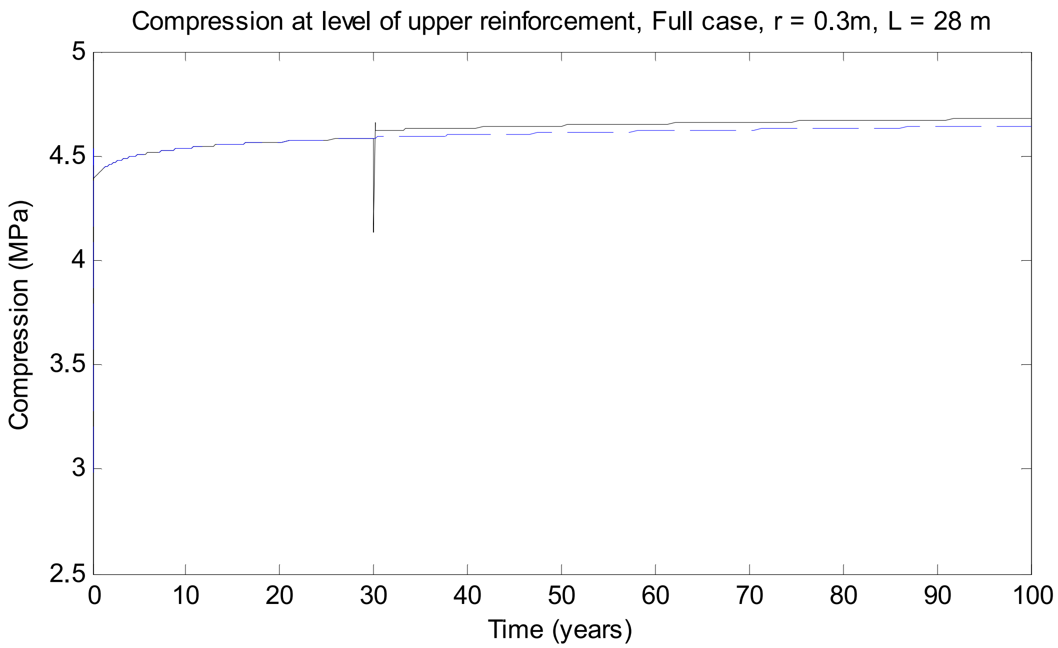


Figure 8-30: Compression at upper reinforcement level as a function of time for span-length L = 28m, when 300 mm is removed and replaced from each flange. The dashed line is the corresponding stress from the reference calculation.

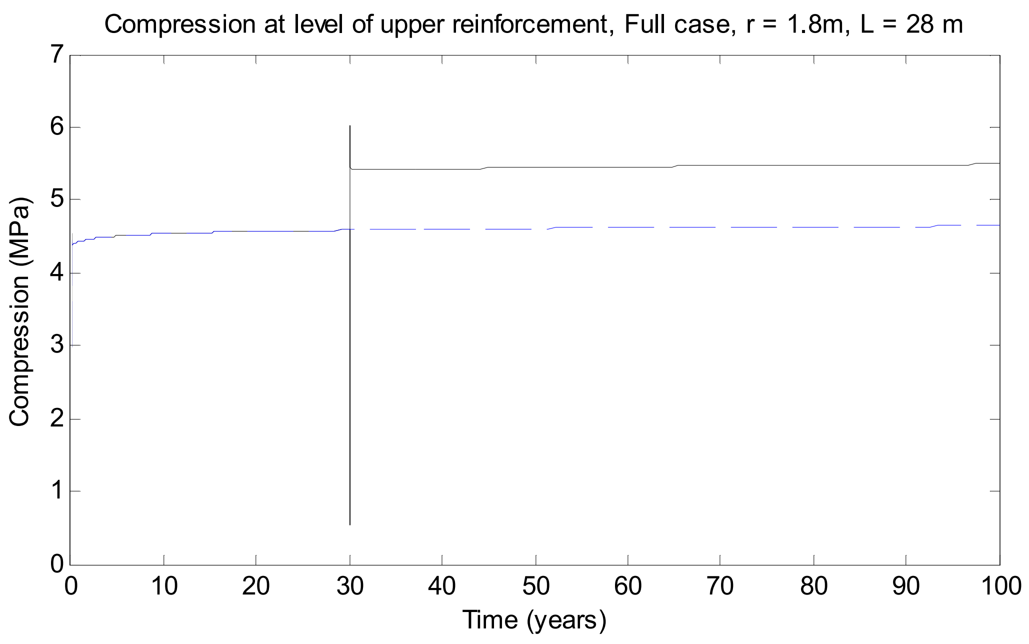


Figure 8-31: Compression at upper reinforcement level as a function of time for span-length L = 28m, when the entire flange is removed and replaced. The dashed line is the corresponding stress from the reference calculation.

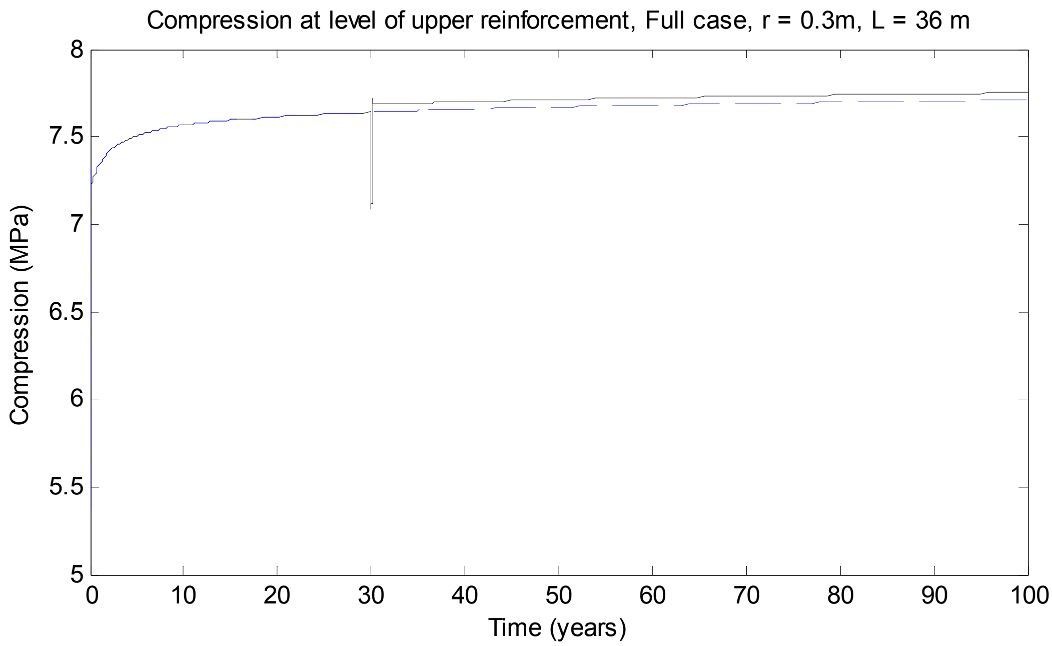


Figure 8-32: Compression at upper reinforcement level as a function of time for span-length L = 36m, when 300 mm is removed and replaced from each flange. The dashed line is the corresponding stress from the reference calculation.

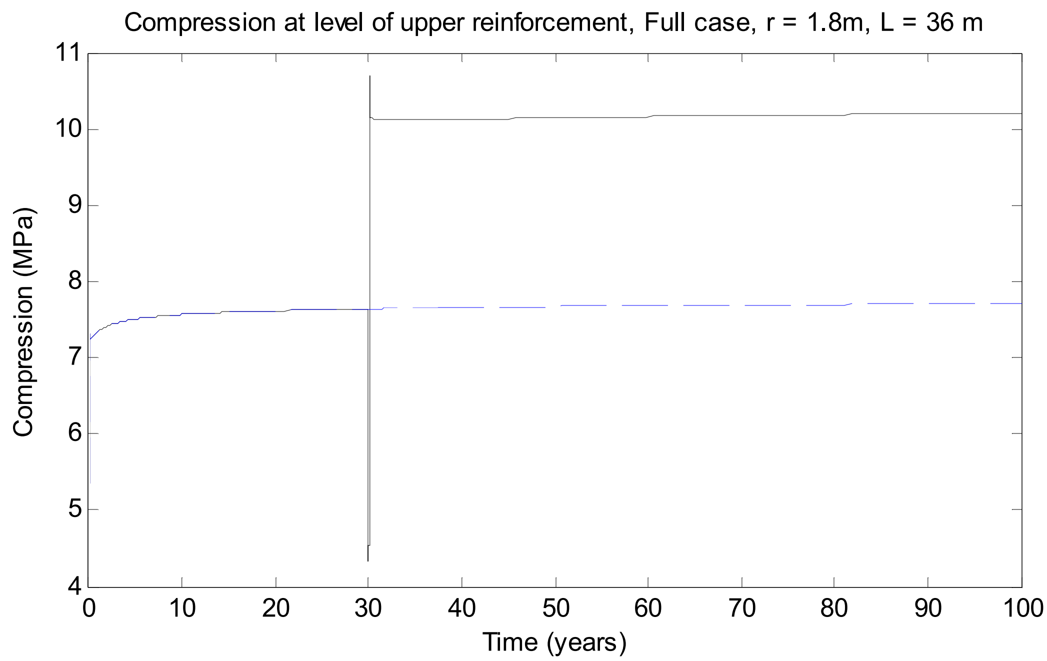


Figure 8-33: Compression at upper reinforcement level as a function of time for span-length L = 36m, when the entire flange is removed and replaced. The dashed line is the corresponding stress from the reference calculation.

8.3.1.2 Lower reinforcement level

The stress distributions at the centre of gravity of the lower reinforcement for the three bridges, with span lengths of 20, 28 and 36 meters, are presented below. In Table 8-7, the stresses in the concrete are compared to the stress limits defined and to the results from the reference calculation presented in section 8.1 above. The stresses neither approach the tensile strength of concrete nor the specified upper value of 15.75 MPa. However, some tensile stresses are obtained for the longest bridge (span-length 36 meters) when the entire flange is replaced.

In Table 8-7 below the magnitude of the different repair cases can be seen. When only 300 mm is removed from each flange, the maximum and minimum stresses are the same as the max and min from the reference calculation. The more concrete that is removed and replaced, the larger the stress variations obtained in the structure. The largest compressive stress occurs in connection with the reduction of the cross section. The smallest compressive stresses occur after the addition of the new concrete.

Table 8-7: Maximum, minimum and end stress at lower reinforcement compared to limit for un-linear behaviour and cracking, respectively, as well as to reference calculation (without repair work). Compression is positive [MPa]. r is the amount of concrete removed from each flange.

	L = 20m			L = 28m			L = 36m		
r (m)	1.80	1.30	0.30	1.80	1.30	0.30	1.80	1.30	0.30
Max σ_l	**9.67**	**9.67**	**9.67**	**8.38**	**8.38**	**8.38**	**7.92**	**7.92**	**7.92**
LIMIT 0.45 f_{ck}	15.75	15.75	15.75	15.75	15.75	15.75	15.75	15.75	15.75
Max σ_l reference	*9.67*	*9.67*	*9.67*	*8.38*	*8.38*	*8.38*	*7.92*	*7.92*	*7.92*
Min σ_l	**0.55**	**2.33**	**3.20**	**0.42**	**1.01**	**2.03**	**-0.36**	**0.06**	**0.36**
LIMIT $f_{ctk0.05}$	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20	-2.20
Min σ_l reference	*3.22*	*3.22*	*3.22*	*2.04*	*2.04*	*2.04*	*3.76*	*3.76*	*3.76*
End σ_l	**0.55**	**2.34**	**3.20**	**0.42**	**1.01**	**2.03**	**-0.35**	**0.59**	**3.64**
End σ_l reference	*3.22*	*3.22*	*3.22*	*2.04*	*2.04*	*2.04*	*3.78*	*3.78*	*3.78*

In Figure 8-34 - Figure 8-39 below, the stresses at the level of the lower reinforcement, for when 300 mm and when the entire flange is removed, are presented for all three span-lengths. As the figures show, the stresses in the three bridges behave similarly. The repair procedure results in stress variations, compression as concrete is removed and tension when concrete is added. From a long-term perspective, the removal and replacement of just 300 mm on each flange hardly affects the stresses. The removal and replacement of the entire flange invokes a permanently increased level of compression.

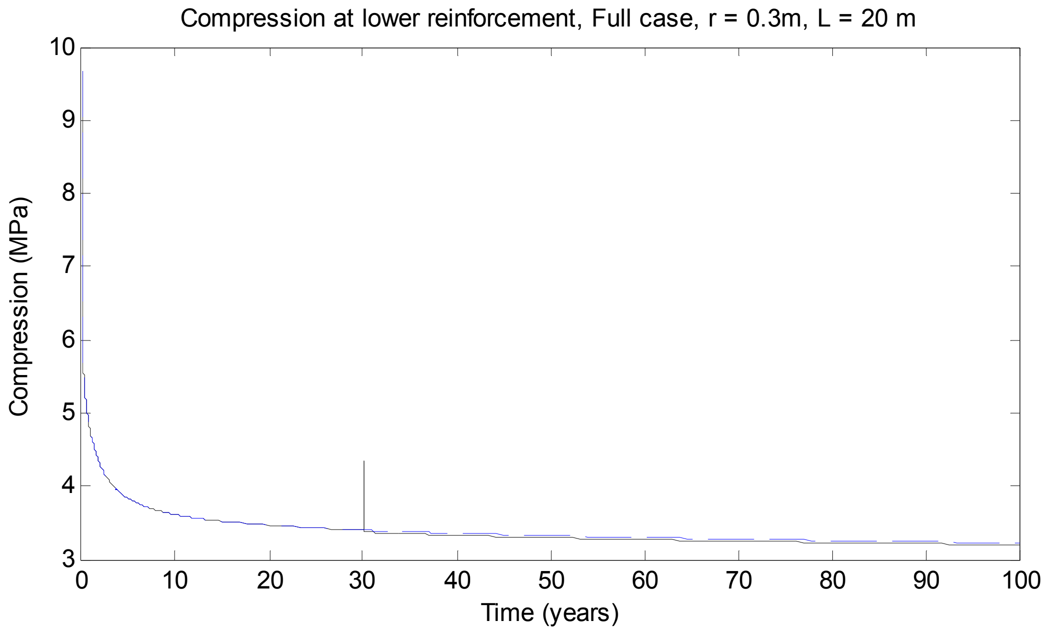


Figure 8-34: Compression at lower reinforcement level as a function of time for span-length L = 20m, when 300 mm is removed and replaced from each flange. The dashed line is the corresponding stress from the reference calculation.

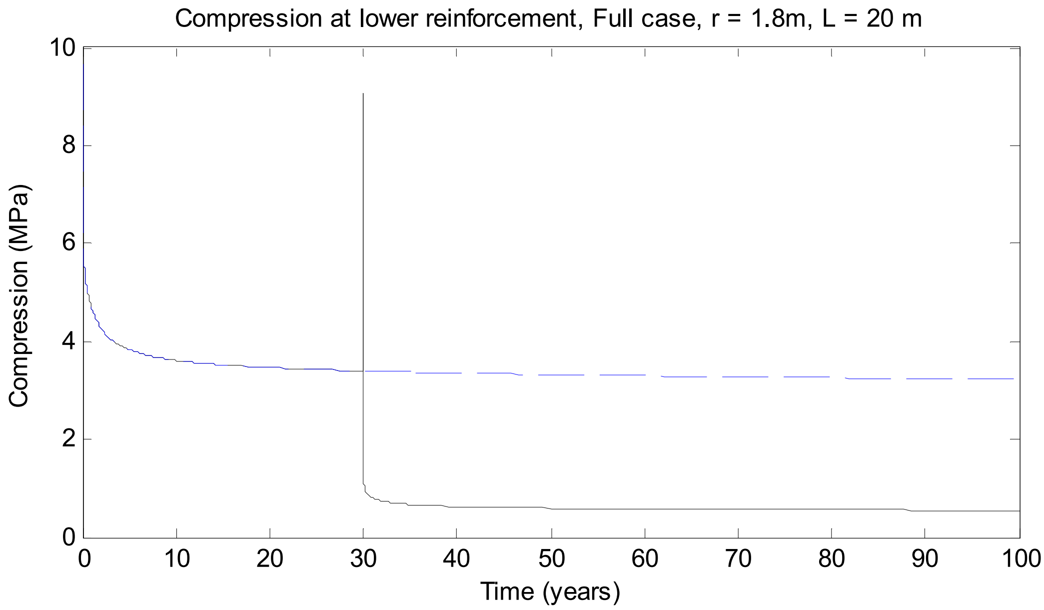


Figure 8-35: Compression at lower reinforcement level as a function of time for span-length L = 20m, when the entire flange is removed and replaced. The dashed line is the corresponding stress from the reference calculation.

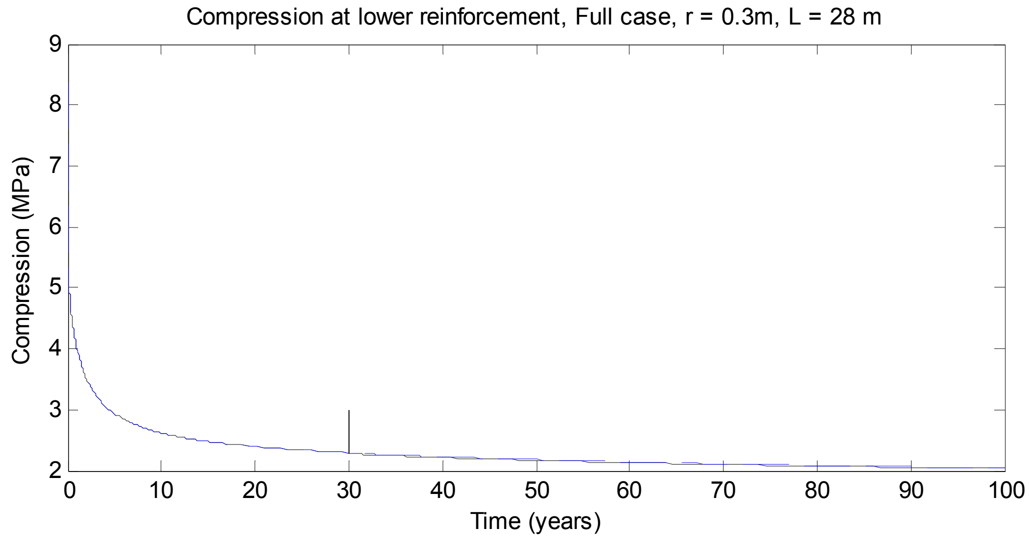


Figure 8-36: Compression at lower reinforcement level as a function of time for span-length L = 28m, when 300 mm is removed and replaced from each flange. The dashed line is the corresponding stress from the reference calculation

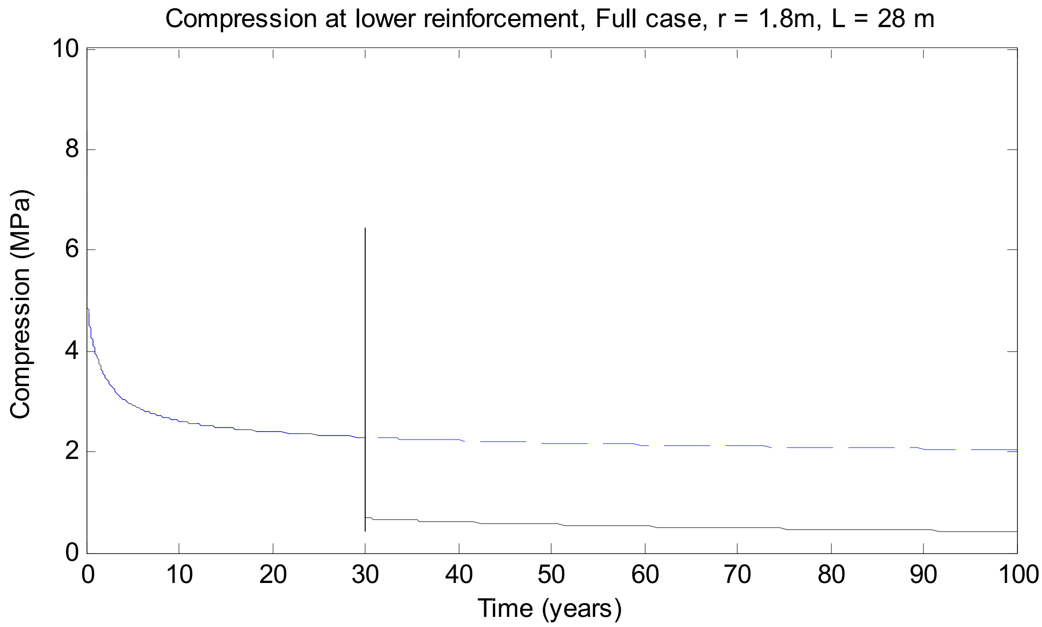


Figure 8-37: Compression at lower reinforcement level as a function of time for span-length L = 28m, when the entire flange is removed and replaced. The dashed line is the corresponding stress from the reference calculation.

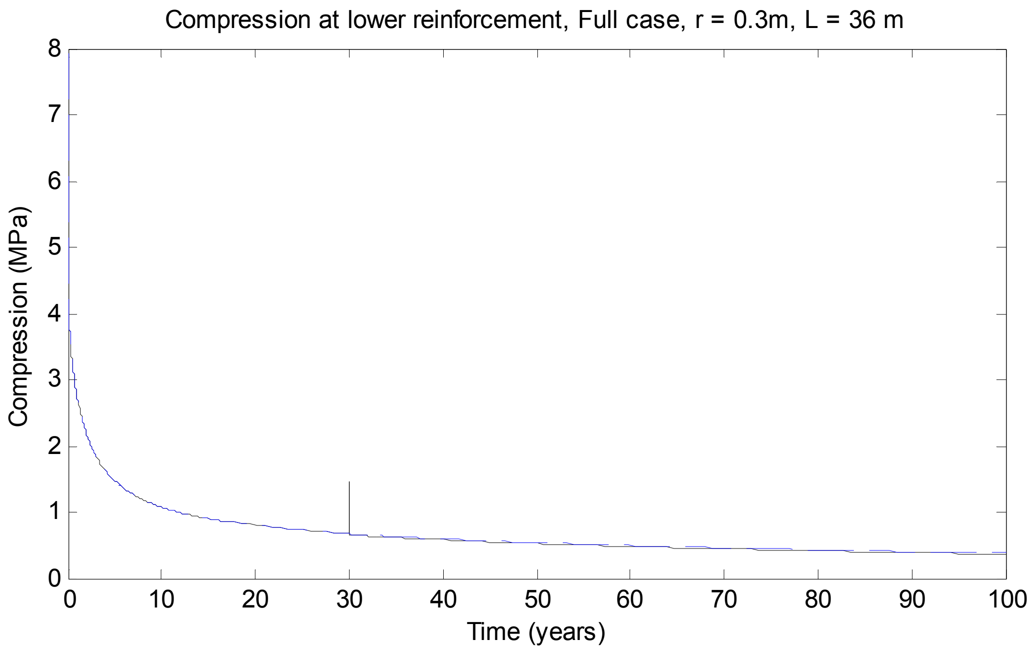


Figure 8-38: Compression at lower reinforcement level as a function of time for span-length L = 36m, when 300 mm is removed and replaced from each flange. The dashed line is the corresponding stress from the reference calculation.

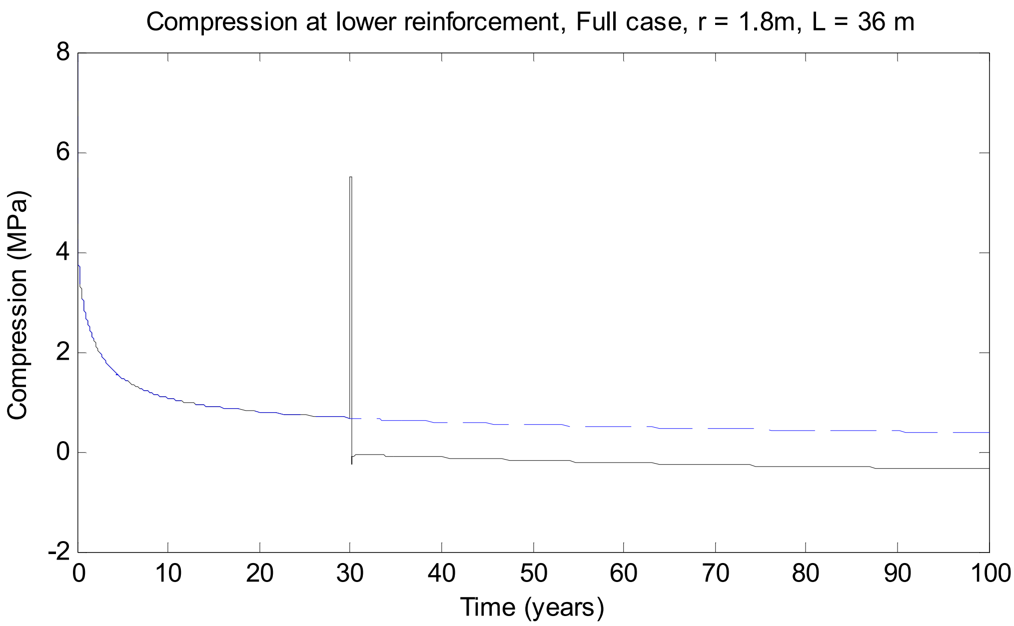


Figure 8-39: Compression at lower reinforcement level as a function of time for span-length L = 36m, when the entire flange is removed and replaced. The dashed line is the corresponding stress from the reference calculation.

8.3.1.3 Stress change due to repair

The immediate (elastic) stresses that occur due to the repair work are presented in Table 8-8 below. The directions of the stresses coincide with 7.2. Logically, the more concrete is removed and replaced, the larger the changes in stress. There is, however, no use in comparing the changes in stress to the limits for allowed stress presented in sections 8.3.1.1 and 8.3.1.2 above, since the changes in stress in Table 8-8 coexist with other stresses in the structure.

Table 8-8: Changes in stress due to the individual effects caused by repair work: Removal of concrete, reduction of cross section and addition of concrete. Compression is positive [MPa].

	L = 20m			L = 28m			L = 36m		
r (m)	**1.80**	**1.30**	**0.30**	**1.80**	**1.30**	**0.30**	**1.80**	**1.30**	**0.30**
$\Delta\sigma_{rem,u}$	-2.67	-1.93	-0.45	-2.74	-1.98	-0.46	-3.31	-2.39	-0.55
$\Delta\sigma_{rem,l}$	5.02	3.62	0.84	4.15	3.00	0.69	4.78	3.45	0.80
$\Delta\sigma_{crs,u}$	-0.32	-0.25	-0.07	-0.58	-0.43	-0.10	-1.30	-0.94	-0.22
$\Delta\sigma_{crs,l}$	-1.90	-1.28	-0.27	-1.34	-0.94	-0.21	-1.16	-0.84	-0.20
$\Delta\sigma_{add,u}$	6.54	3.30	0.50	5.22	2.98	0.49	6.05	3.53	0.59
$\Delta\sigma_{add,l}$	-5.77	-3.77	-0.79	-4.68	-3.13	-0.66	-5.49	-3.67	-0.77

8.3.1.4 Loss in prestressing

In Table 8-9 below the prestressing losses are presented. The prestressing losses take the stresses invoked due to the repair procedure, presented in Table 8-8, into account. The losses for the repair cases involving removal and addition of 300 mm and 1800 mm (entire upper flanges) are presented in Figure 8-40 - Figure 8-45. A permanent increase in loss after 100 years is obtained, in comparison to the reference calculation.This is a consequence of the cross section remaining reduced in the calculation even after the addition of concrete.

Table 8-9: Loss in prestressing. The maximum loss, the end loss at 100 years and the end loss according to the reference calculation (calculation without repair work) are presented in %.

	L = 20m			**L = 28m**			**L = 36m**		
r (m)	**1.80**	**1.30**	**0.30**	**1.80**	**1.30**	**0.30**	**1.80**	**1.30**	**0.30**
End loss (%)	22.0	20.7	20.0	19.7	19.5	18.9	19.1	19.1	19.0
End loss reference (%)	*20.0*	*20.0*	*20.0*	*18.9*	*18.9*	*18.9*	*19.0*	*19.0*	*19.0*

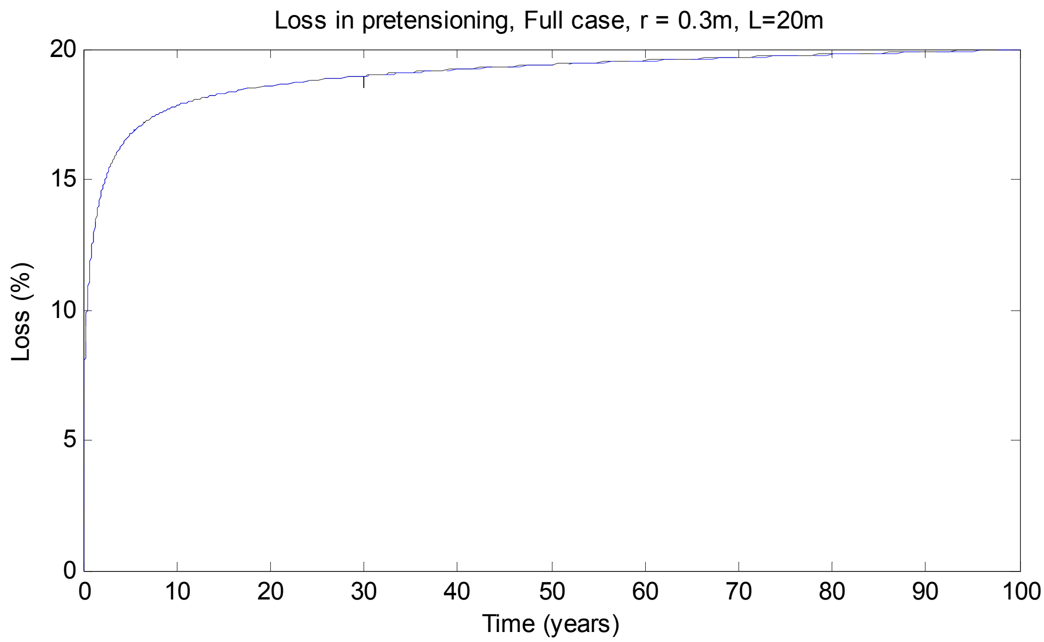


Figure 8-40: Prestressing loss for bridge with span-length L = 20m, when 300 mm is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

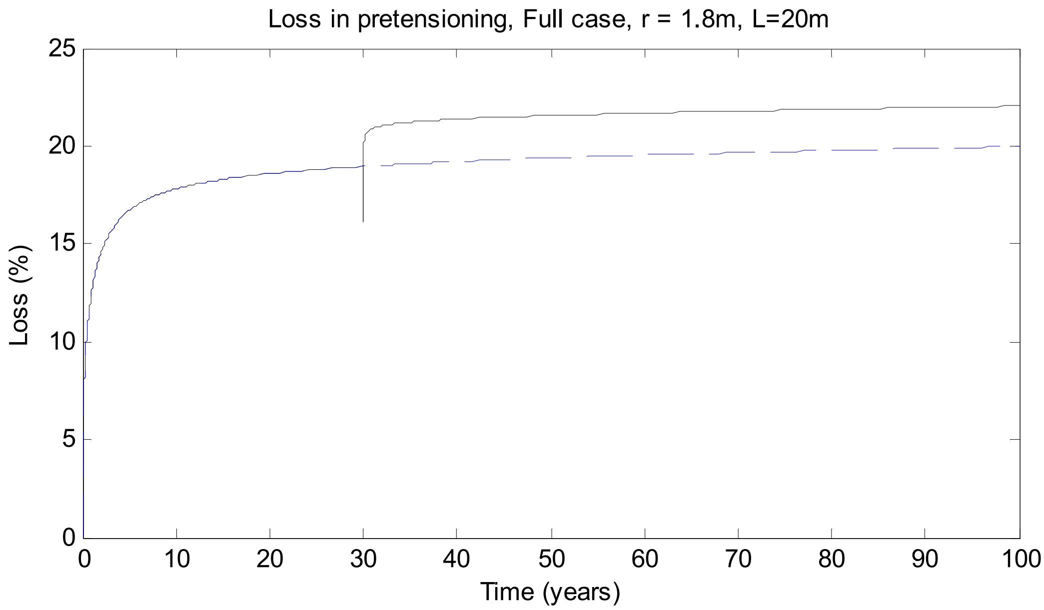


Figure 8-41: Prestressing loss for bridge with span-length L = 20m, when 1800 mm is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

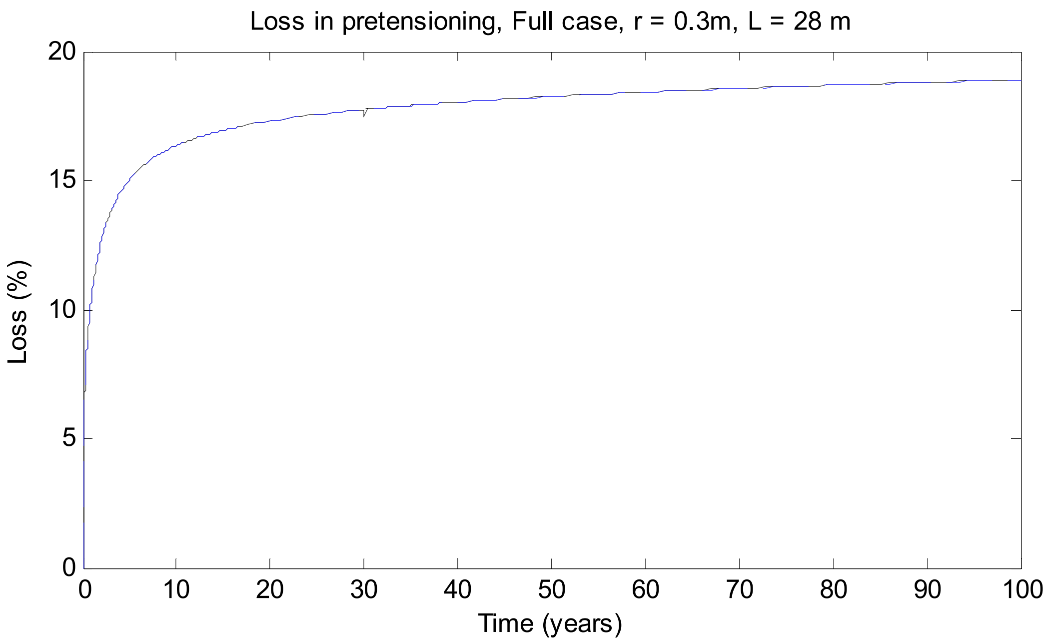


Figure 8-42: Prestressing loss for bridge with span-length L = 28m, when 300 mm is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

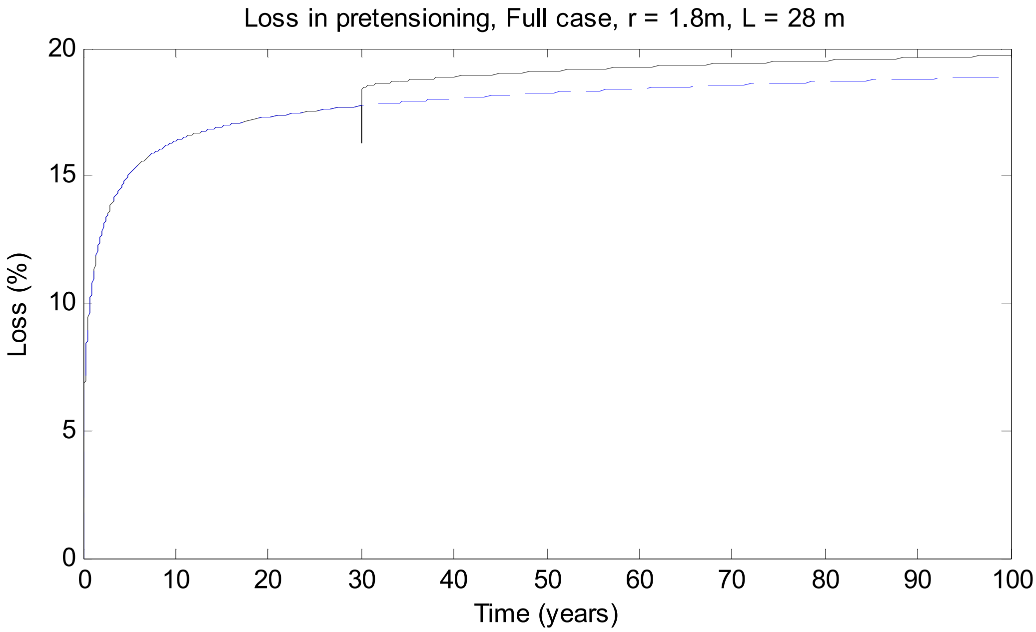


Figure 8-43: Prestressing loss for bridge with span-length L = 28m, when 1800 mm is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

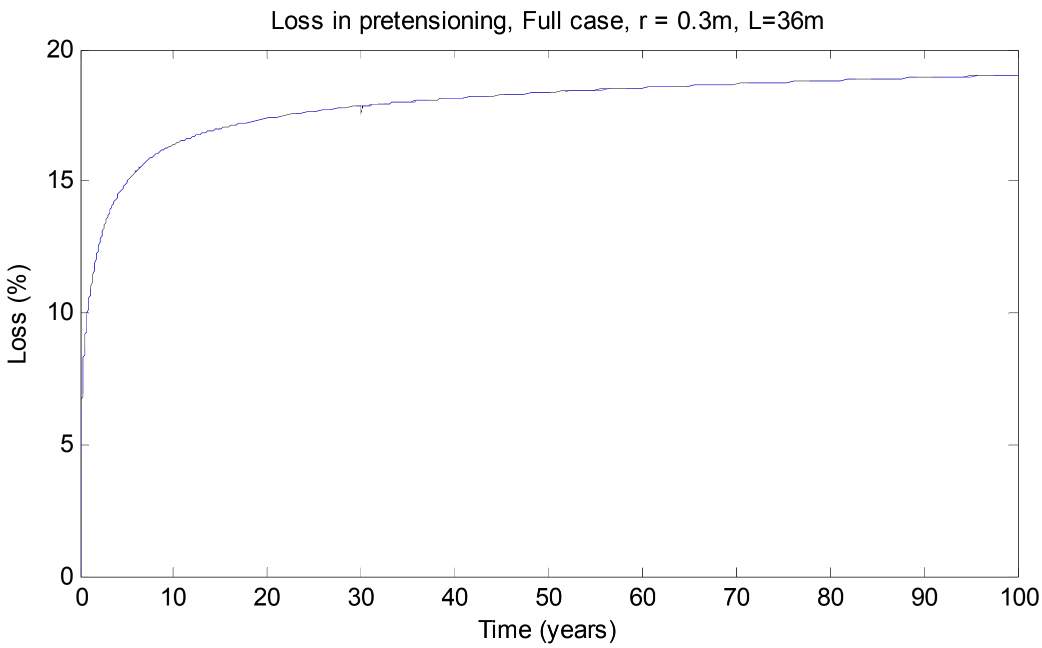


Figure 8-44: Prestressing loss for bridge with span-length L = 36m, when 300 mm is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

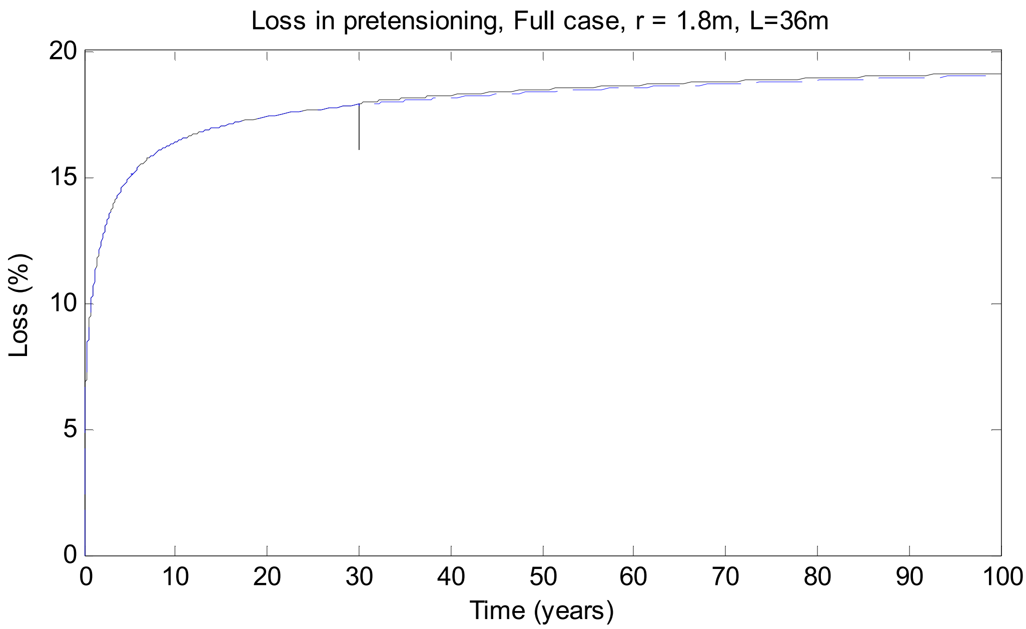


Figure 8-45: Prestressing loss for bridge with span-length L = 36m, when 1800 mm is removed from each flange. The dashed line is the corresponding loss in prestress from the reference calculation.

8.3.2 Cracking of new concrete

The change (decrease) in compression that occurs at the level of the upper reinforcement from the addition of the new repair concrete, until the end of the calculation at 100 years, will appear as tension in the new concrete. It is thus of interest to examine how big the risk of cracking of the new casting is.

The risk of cracking is evaluated by comparing the tensile strain in the new concrete with the tensile strength of the concrete:

$$\varepsilon_{ctk,ok} = \frac{f_{ctk,0.05}}{E_{cm}}$$, where:

$f_{ctk,\,0.05}$ is the tensile strength of the concrete (5% fractile), and E_{cm} is the modulus of elasticity of the concrete, at 28 days.

Similarly, the tensile strain in the new concrete is the tensile stress divided by the modulus of elasticity of concrete at 28 days:

$$\varepsilon_{ctk,add} = \frac{\sigma_{ctk,add}}{E_{cm}} .$$

As mentioned above, the tensile stress in the plate ($\sigma_{ctk,\,add}$) is the change in compression which has occurred in the structure, from the time of the addition of the plate, until the end of the bridge life-cycle at 100 years. The results from this comparison can be seen in Table 8-10 below.

The tensile strain limit in the concrete $\varepsilon_{ctk,ok}$ is compared to the tensile strain that arises due to the addition of concrete $\varepsilon_{ctk, add}$. The results in Table 8-10 does not indicate any risk that the repair concrete may crack. However, the extent of the interaction between the old structure and the new concrete is difficult to model. The tensile stresses that arise may be bigger than indicated here.

Table 8-10: Evaluation of cracking in new repair concrete. The tensile strain $\varepsilon_{ctk, add}$ due to the tensile stress in the new concrete plate $\sigma_{ctk, add}$ is compared to the tensile strength strain of the concrete $\varepsilon_{ctk, ok}$ by calculating the strain margin $\Delta\varepsilon_{ctk} = \varepsilon_{ctk, ok} - \varepsilon_{ctk, add}$.

	r	$\varepsilon_{ctk, ok}$ $(\cdot 10^{-5})$	$\sigma_{ctk, add}$ (MPa)	$\varepsilon_{ctk, add}$ $(\cdot 10^{-5})$	
L = 20m	0.3	6.46	0.68	1.98	OK
	1.3	6.46	0.12	0.36	OK
	1.8	6.46	0.02	0.06	OK
L = 28m	0.3	6.46	0.56	1.63	OK
	1.3	6.46	0.24	0.69	OK
	1.8	6.46	0.02	0.05	OK
L = 36m	0.3	6.46	0.49	1.45	OK
	1.3	6.46	0.20	0.60	OK
	1.8	6.46	0.03	0.09	OK

8.3.3 Summary

When concrete is removed, the compression at upper reinforcement level decreases. As concrete is added, the compression increases. Since the concrete is added to a smaller cross section than the one that concrete was removed from, a permanent increase in compression after the addition of concrete is obtained. At the lower reinforcement level, the compression first increases due to removal of concrete and then decreases as new concrete is added. Small tensile stresses arise when the entire upper flanges are replaced, in the bridge model with span-length 36 meters. The compression level after addition is lower than it was before the repair procedure due to the reduced cross section. The limit for concrete creep non-linearity is never reached.

The effect on the loss in prestressing is a small increase in losses for all cases. This increase sums up to 2% in the worst case, which is negligible. According to the calculations, the new repair concrete will not crack. However, since it is difficult to anticipate to what extent the old and new concrete castings interact, crack reinforcement in the new concrete is adviceable.

9 CONCLUSIONS

The most severe effects due to the reparation of a prestressed concrete beam are not obtained permanently in the structure, but rather as an elastic effect at the time of repair. The stresses increase and/or decrease very quickly as concrete is removed and replaced. From a long-term perspective there are no considerable changes in the prestressing losses, neither when the entire upper flanges are removed, nor removed and replaced.

9.1 Removal of concrete

The more concrete is removed, the lower the level of compression at the level of the upper reinforcement. However, no tensile stresses are obtained.The compression at the level of the lower reinforcement increases due to the removal of concrete but it never reaches the limit for non-linear creep.

The changes in stress distribution from a long-term perspective are small when very little concrete is removed. The compression at the level of the upper reinforcement reaches a permanently lower level after the removal of concrete. At the level of the lower reinforcement the compression is permanently elevated after the removal of concrete. The effect on the loss in prestressing is negligible. Thus, the entire upper flange can be removed from the cross section as long as the tensile stresses are monitored and kept below the tensile strength of the concrete.

9.2 Removal and replacement of concrete

After the immediate effects described above, due to the removal of concrete, an immediate increase in compression at the level up the upper reinforcement and an immediate decrease in compression at the level of the lower reinforcement, are inflicted. A permanent increase in compression is obtained at the upper reinforcement level, and at the lower reinforcement level the compression is permanently decreased.

The repair work has a very small effect on the loss in prestressing. The cross section properties are not considered to be restored after the addition of new concrete, and thus the loss in the results is higher than it would have been if the structure had never been repaired. This is the result of the same amount of weight as was removed is added to a much smaller cross section. Hence, the entire upper flange can be removed and replaced from the structure. However, the new concrete should be equipped with crack reinforcement, since it may be prone to crack even though the results fromt he calculations indicate that it is not.

9.3 Summary

The conclusions are summarized by answering the questions stated in section 1.2.

1. What changes in stress distribution, and thus also in long-term losses in tension, arise due to concrete removal and/or erosion?

The removal of concrete from the cross section results in a permanent decrease in tension at the top of the cross section, and a permanent increase in compression at the bottom of the cross section. This results in a decrease in loss, in other words more tension in the prestressing tendons.

2. To what extent does the altered stress distribution (1) inflict further long-term losses in the tendons?

The altered stress distributiondue to concrete removal inflicts a lower loss than for a cross section left intact.

3. What changes in stress distribution, and thus also in long-term losses in tension, arise due to the addition of new concrete?

The addition of new concrete inflicts a permanent increase in compression at the top of the cross section and a permanent decrease at the bottom. The addition of new concrete could possibly result in tension at the bottom of the cross section. An increase in loss of about 2-3% is obtained for the worst cases when the entire upper flanges are removed and replaced.

4. How much concrete can be removed from the cross section?

According to the results from these calculations, the entire upper flanges can be removed and replaced, without inflicting too severe consequences to the structure.

10 DISCUSSION

The results obtained are resonable. However, it is difficult to verify whether they are correct. Further investigations are necessary. The results could be verified by measurements. Further calculations should preferably be carried out addressing the problem with an alternative approach.

The extent of the interaction between new and old concrete is difficult to model accurately. Cracking in the new concrete is not obtained in the calculations described. Since the elastic modulus of the new repair concrete casting is smaller than that of the structure, more extensive change in forces may occur than what has been assumed in the calculations.

The effect of the hydration-heat of the new concrete might have a considerable effect on the stress distribution. This phenomenon has not been investigated in this thesis. Hydration-heat can possibly be managed by pre-heating the old structure and/or cooling the new casting as it is hardening.

Only the weight of the structure has been taken into account as a loading in the calculations above. It would be of interest to further investigate how the reduced and/or repaired structure reacts to the effect of traffic loading. Moreover, vibrations due to traffic may affect the strength of the new concrete, as well as the bond between the new concrete and the old structure. This can be dealt with by introducing speed limits during the hardening phase of the new concrete.

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APPENDIX 1: Särkijärvi Bridge

Bridge specifics

Length: 28.5 m
Beam height: 1.350 meters
Number of prestressing tendons: 9
Initial prestressing force, before anchoring: 2786 kN/ tendon

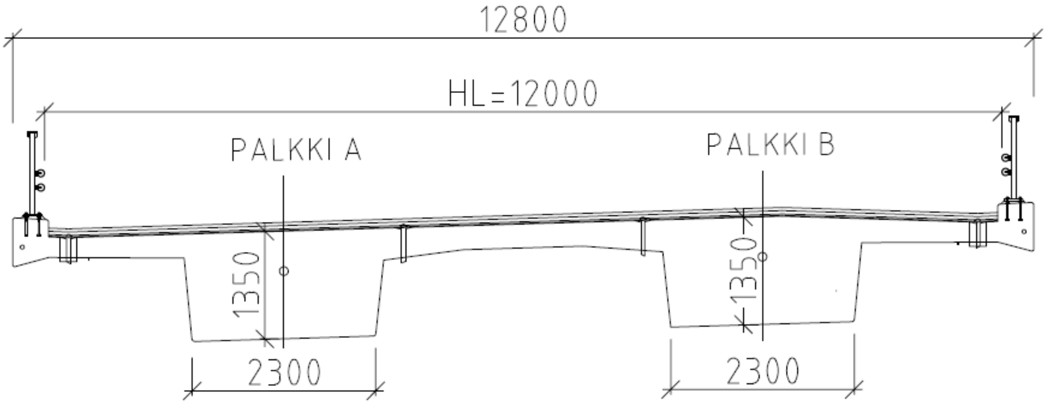

Figure A - 1: Mid-span cross section of the Särkijärvi Bridge construction plan.

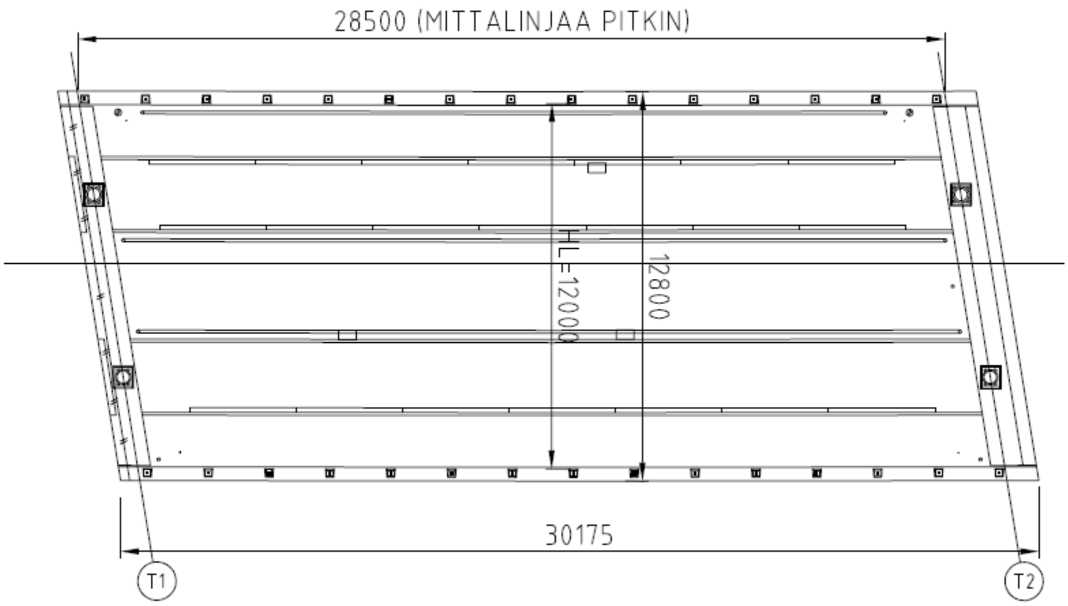

Figure A - 2: Plane figure of Särkijärvi Bridge from the construction plan.

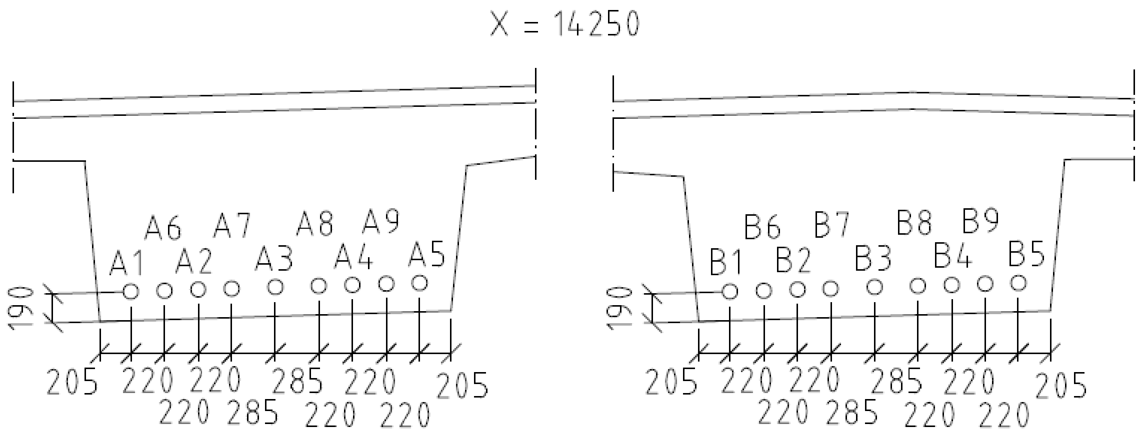

Figure A - 3: Placement of prestressing tendons mid-span.

Figure A - 4: The bridge deck of Särkijärvi Bridge when it was being constructed (photo by K. Nikula).

Figure A - 5: Särkijärvi bridge from underneath, taken when the bridge was being constructed (photo by K. Nikula).