# Prestressed Concrete Bridges

# Condition Assessment and Future Challenges

## A State-of-Art Report

BBT 2017-011, Trafikverket, TRV 2018/28332, ID 6686

Björn Täljsten, Björn Paulsson and Lennart Elfgren

Structural Engineering
Department of Civil, Environmental and Natural Resources Engineering
Luleå University of Technology
SE-971 87 Luleå

Cover Photo: Angeredsbron, Göta Älv, Gothenburg, 1978, Photo: Mikael Svensson/Trafikverket, www.trafiken.nu

Structural Engineering
Department of Civil, Environmental and Natural Resources Engineering
Luleå University of Technology
SE-971 87 Luleå

# Preface

This project is a pre-study on “Prestressed Concrete Bridges – Methods to assess the remaining prestressing force”, commissioned by BBT 2017-011, Trafikverket, TRV 2018/28332, ID 6686. During the study we found it motivated to widen of the scope.to include a more general treatment of the challenges with existing prestressed bridges. The reason for this is that we found that other problems are more severe in Swedish bridges than the remaining prestressing forces.

This report is a first attempt to answer questions regarding the severity and extent of the current situation for prestressed concrete bridges in Sweden. It also discusses possible social and economic consequences if no measures are taken. Hopefully this report will form a basis for future work regarding assessment, non-destructive testing and strengthening of existing prestressed concrete bridges.

The report was compiled during 2018 and the beginning of 2019 and is based on an extensive literature study and previous work and experience by the authors In addition contacts has been taken with many colleagues in Sweden and abroad and these discussion has been incorporated in the report.

The project is a pre-study commissioned by BBT/Trafikverket, TRV 2018/28332, ID 6686. The authors want to thank Ebbe Rosell, Robert Ronnebrant, Fredrik Olsson and Adriano Maglica at Trafikverket for good input and fruitful personal discussions.

Luleå, January 2019

Björn Täljsten, Björn Paulsson and Lennart Elfgren

# Abstract

Prestressed concrete bridges are susceptible to deterioration over time and it is vital to continually assess them in order to maintain their structural integrity and to prolong their service life. Important factors are corrosion of prestressing strands, wires and bars and concrete deterioration. In recent years, there has been an increased interest in monitoring and non-destructive testing to assess the state of bridges. It is essential to understand the behavior in ultimate and serviceability limit states and the level of safety, reliability and robustness. Calibration of Condition and Life Cycle Assessments are discussed by help of Full-Scale Tests of Bridges that are planned to be demolished. Repair and Strengthening methods are reviewed. Needs for Maintenance Strategies are outlined. Important factors are corrosion of prestressing strands, wires and bars, remaining prestressing forces and concrete deterioration.



Keywords: Prestressed concrete bridges, Pre-tensioning, Post-tensioning, Non-destructive testing, Monitoring and Inspection, Condition Assessment, Life Cycle Assessment, Ultimate and Serviceability Limit States (ULS, SLS), Calibration by Full Scale Testing to failure. Maintenance Strategies, Residual Prestressing.

# Executive Summary

Most concrete bridges of importance are prestressed as this is a technique to increase the length of the bridge spans. Society is dependent on their function and safety. They must be kept under control and in good condition and neglect can cause devastating damage and costs as in the recent collapse of the Morandi bridge in Genoa in August 2018. It illustrates the problems with corrosion of prestressing cables and how hard it is to determine its severity.

Deterioration occurs in all bridges and it is essential for bridge managers to be proactive and optimize maintenance without unnecessary traffic disturbances and accidents.

This report aims to identify how extensive and serious the problems are with prestressed bridges in Sweden and to propose actions to mitigate them. The remaining prestressing force is normally higher than what was expected and is not a major problem is Swedish bridges.

We have seen that a lot of progress was made in the late seventieths regarding materials and construction. This means that there is a larger risk for problems with the bridges built before 1980 than with the ones built after that. Out of the circa 2000 prestressed concrete bridges in Sweden there are some 400 built before 1980. A special investigation of the condition and safety of these bridges is recommended.

We have found that no single nondestructive monitoring or inspection techniques can give enough information regarding the safety of a prestressed concrete bridge. At the same time, we have found that there are promising combinations of techniques that can be used.

Today there are on the market strengthening and repair methods that are cost-effective, safe and environmentally friendly.

In order to meet long term demands we propose a concept for further studies in order to:

- better understand the behavior of prestressed concrete bridges in the Serviceability Limit State (SLS) as well as in the Ultimate Limit State (ULS). There are bridges with substantial higher load-carrying capacity than they were designed for as well as there are bridges with critical points with a low safety level.
- protect, detect and mitigate corrosion of prestressing reinforcement
- elaborate nondestructive monitoring and inspection methods.
- augment methods for risk analyses and for the study of the reliability and the robustness of prestressed concrete bridges
- develop methods for Condition Assessment and for Life Cycle Cost Assessment (LCCA) in order to optimize maintenance strategies. The methods ought to be calibrated by full scale tests of bridges decided to be demolished
- advance repair and strengthening methods
- promote proactive and adaptive maintenance strategies that support a sustainable society

An ambitious, manifold project like this could preferably be carried out in an international collaboration. However, parts of it need to be carried out on a national level without unnecessary delay.

# Sammanfattning

Rapporten är skriven på engelska då de problem som behandlas förekommer i hela världen och vi hoppas kunna bidra till att initiera ett internationellt utvecklingsprojekt inom området.

De längsta och ofta viktigaste betongbroarna är förspända eftersom förspänningstekniken möjliggör längre spännvidder. Detta betyder att samhället är beroende av att dessa broar fungerar på avsett sätt och att de är säkra. Därför måste de vara väl underhållna och i gott skick eftersom försummat underhåll kan leda till allvarliga olyckor som den tragiska kollapsen av Morandi-bron i Genua i augusti 2018. Problemen med spännarmering tydliggjordes där liksom hur svårt det kan vara att bedöma korrosionsskador på ingjutna kablar.

Nedbrytning sker i alla broar och det är väsentligt för brohållare att vara proaktiva och att optimera underhållsåtgärder utan att de medför onödiga trafikstörningar och olyckor. När sådana ändå sker får vi inte stå handfallna utan vi måste kunna hantera de problem som uppstår. Den kvarvarande spännkraften är normalt högre än vad man dimensionerat för och är därför vanligen inte den mest kritiska faktorn. Rapporten syftar till att identifiera hur omfattande och allvarliga de problemen är som kan uppstå vid spännarmerade broar i Sverige och att förslå åtgärder för att bemästra dem.

Vi har konstaterat att det under sent 70-tal gjordes viktiga förbättringar av de barnsjukdomar som förelåg beträffande material och produktionsmetoder. Det betyder att broar byggda före 1980 kan ha problem. Av de ca 2000 spännbetongbroar som finns i Sverige är ca 400 byggda före 1980. Vi rekommenderar en fördjupad studie av dessa broar baserad på kunskapen från arbetet med denna rapport. Vi har också konstaterat att ingen oförstörande undersöknings- och inspektionsmetod ensam kan ge tillräcklig information som beslutsunderlag eftersom noggrannheten är otillräcklig. Samtidigt har vi hittat lovande kombinationer av tekniker som kan nyttjas i framtiden.

På marknaden finns redan förstärknings och reparationsmetoder som är kostnadseffektiva, säkra och miljövänliga men som kan behöva utvecklas och prövas.

För att möta frågan långsiktigt föreslår vi ett koncept för fortsatta studier. Det innehåller följande utvecklingsområden så att vi:

- bättre förstår hur spännarmerade broar beter sig såväl i bruks- som i brottgränstillståndet. Det finns såväl broar som håller betydligt mer än det de dimensionerats för, som broar med kritiska punkter där säkerheten är låg.
- skyddar, upptäcker och motverkar korrosion i spännarmeringen
- utvecklar icke förstörande undersöknings- och inspektionsmetoder
- förbättrar metoder för riskanalys och för studium av pålitlighet och robusthet
- utvecklar metoder för tillståndsbedömning och livscykelkostnadsanalys för att optimera underhållsstrategier. Metoderna bör kalibreras genom fullskaleförsök på broar som dömts ut och ändå skall förstöras
- vidareutvecklar reparations- och förstärkningsmetoder
- verkar för proaktiva och adaptiva strategier för underhåll som stöder ett mer hållbart samhälle

Ovanstående ambitiösa och mångfacetterade projekt borde med fördel utföras i internationellt samarbete. I avvaktan härpå bör delar startas på nationell nivå utan onödigt dröjsmål.

# Notations and abbreviations

## Notations

The process of **prestressing** consists in applying forces to the concrete structure by stressing **tendons** relative to the concrete member. "**Prestress**" is used globally to name all the permanent effects of the prestressing process, which comprise internal forces in the sections and deformations of the structure. (Eurocode 2, EN 1992-1-1:2005, section 1.5.2.4).

The prestress is applied by **tendons** made of **high-strength steel** (**wires, strands** or **bars**). Tendons may be **embedded** in the concrete. They may be **pre-tensioned** and **bonded** or **post-tensioned** and **bonded** or **unbonded**. Tendons may also be **external** to the structure with points of contact occurring at deviators and anchorages. (Eurocode 2, EN 1992-1-1:2005, section 2.3.1.4). **Couplers** may be used to anchor and/or join the tendons.

Resilience (from Latin *resilere*, of *re* = once more; back; *salire* = jump) the capability of a strained body to recover its size and shape after deformation.

Robustness (from Latin *robus*, *robur* = oak, strength) the capacity of a structure to function also with accidental or exceptional events.

Redundancy (from Latin *redundere* = to flow over, be in excess, from *unda* = wave) indicates that a structure can carry loads also if one part is removed.

## Abbreviations

| | |
|---|---|
| CFRP | Carbon Fibre Reinforced Polymers |
| CM | Cover Meter |
| EIS | Electrochemical Impedance Spectroscopy |
| ES | Endoscope |
| GPR | Ground Penetrating Radar, Georadar, Impulse radar |
| IE | Impact Echo |
| IT | Infrared Tomogreaphy |
| MFL | Magnetic Flux Leakage |
| MMFM | Permanent Magnet |
| MMFM | Solenoid |
| NDT | Non-destructive Testing |
| PM | Potential Mapping |
| RPF | Residual Prestressing Force |
| RT | Radar Tomography |
| SASW | Spectral Analysis of Surface Waves |
| SLS | Service Limit State |
| ULS | Ultimate Limit State |
| UPE | Ultrasonic Pulse Echo |
| VI | Visual inspections |

# Table of content

# 1 Introduction

## 1.1 Background

Civil infrastructure and structures are susceptible to different kinds of deterioration processes and defects once built and used. Examples of damages these defects and deterioration processes might lead to are cracking, bond loss, voids, reduction of cover layer, corrosion, delamination etc. which in the long run, if nothing is done, leads to lowering of the performance level and eventually unsafe structures. This necessitates methods to continuously assess the quality of structures in order to avoid problems that might lead to shorter service life or reduction of structural integrity. With a proper and continuous assessment of the state of a structure maintenance can be planned in advance and the structural safety can be increased. The service life can also be increased if the structural integrity of a structure can be proven to meet the requirements, saving both money and decreasing the environmental impact of the structure.

Communication between people and transportation of goods constitute for exchange of ideas and growth in society. Without a well function and reliable infrastructure this would not be possible. Here our bridges provide an essential part for our physical transports. The most used material in our bridges is concrete, and more precise reinforced concrete, and for our large bridges, prestressed concrete. During the last two to three decades more and more focus has been placed on the life of our bridge and ongoing deterioration processes. Also considerably development regarding assessment and strengthening methods has been made during this time span. In addition, today we also have stronger calculation tools and a better understanding of our existing bridges and their behavior – at least for RC bridges where existing assessment and repair/strengthening methods is quite well understood. However, this is not the case with our existing prestressed concrete bridges, despite the fact that these bridges are critical for transportation and communication in our modern society. One large challenge with prestressed bridges is the possibility to assess the inner parts, i.e. ducts, anchorage and tendons, without creating damages. In the literature, challenges and problems with prestressed concrete bridges has been shown. However, very few problems have been reported in Sweden, even though we have the same type of post-tensioning systems installed. In addition our codes before the 70-ties allowed calcium chloride in the grout, and not until the 80-ties thicker ducts was raised due to changed standards. This means that we potentially could have problems we prestressed bridges built before 1980-ties. In Sweden we have approximately 2000 prestressed concrete bridges and 400 of these are built before 1980. It is then of utmost importance that these bridges are investigated more thoroughly. Most likely they are in good quality, but that has most commonly been verified in visual inspections.

## 1.2 Objectives

International experience shows that today there are problems with prestressed concrete structures. We do not have detailed knowledge regarding the depth of the problems since the bars, strands and wires are protected by grout and also cast into the structure they are not easily visible for inspection. Add to this that state of the are inspection methods (NDT or visual) are not accurate enough.

The overall objective regarding assessment of prestressed concrete structures is to work more proactive so that the society can handle the situation in advance and handle undesired events without catastrophic and/or disturbing effect on society. At the same time work adaptive so future possible undesired events are better considered.

The objective with this study is to investigate the Swedish situation in a wider perspective. This is done by an initial study including compilation of post-stressed bridges and existing post-tensioning systems. It will also investigate problems seen today as well as how today's condition assessment is done in Sweden. In this project the objectives are to decide

- the seriousness of the problem
- and, the extent of the problem

However, this has not been possible within this limited project, and in Chapter 9 we will discuss activities for the future actions.

### 1.3 Outline of a structure for future investigations

For prestressed concrete bridges the most important problems are summarized in Figure 1.1 together with a proposed way of dealing with them through a joint research project. This report can be seen as an initial State-of-Art in order to map the need for knowledge in the different main areas:

- Corrosion of strands wires and bars. This is a major problem as the reinforcement cannot easily be observed and the prestressing of the concrete prevent it to crack so the structure may look more sound than what it is
- Behavior in Serviceability and Ultimate Limit States (SLS) and (ULS). We need to learn what the load-deflection diagram looks like for different levels of deterioration and how the stiffness of the structures is changed.
- Monitor and inspection methods. There is a lot of development going on and the most relevant and promising methods ought to be tested.

With a good background from these three kinds of investigations it would be possible to engage in studies of the following three areas:

- Safety, Reliability and Robustness. How safe is an existing structure? Reliability based methods can be used to find out the robustness of an existing structure
- Full scale tests to Calibrate Condition and Life Cycle Assessment methods. When an interesting type of an old bridge is going to be demolished there is a good opportunity to check its real condition by a full-scale test. This gives a chance to calibrate existing code models and numerical methods.
- Repair and Strengthening methods ought to be developed and tested on structures.

Finally the results should be summarized to Maintenance Strategies for prestressed concrete bridges.

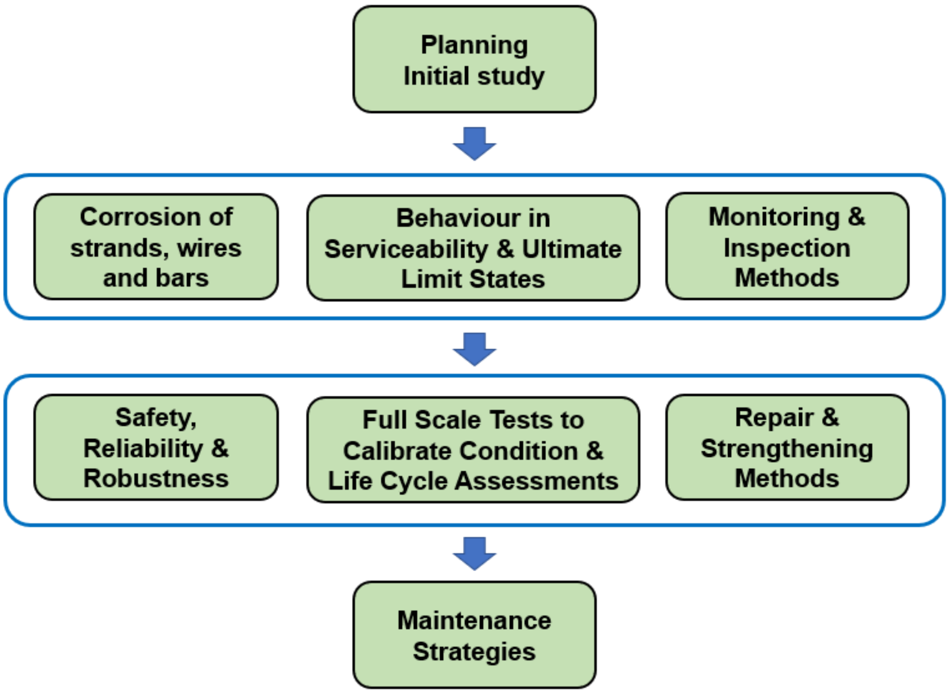



Figure 1.1. Outline of a possible way to structure investigations in different areas which need to be explored for prestressed concrete structures. This report can be seen as an initial State-of-Art in order to map the need for such investigations

## 1.4 Limitations

This state of the art has been a preliminary study regarding the research needs of prestressed concrete bridges. However, during the work the study has grown, partly based on the interest of the authors but also depending on the complicated nature of the topic studied. Nevertheless, with bounded resources the study is limited in its description. For example we have not in detail investigated any of the Swedish bridges, nor have we investigated the methods described in the report in laboratory tests or in field and, in addition, despite the vast amount of literature studied, more detailed analyses is needed. In particular studies of bridges built before the 80-ties.

## 1.5 Report structure

Chapter 1 gives a short introduction to the report and why it is important to further investigate prestressed concrete bridges. In Chapter 1 we also present briefly a suggestion to a concept for further studies and discuss the limitations with the study. In Chapter 2 we give a historical overview regarding prestressed concrete bridges, their development, how prestressing works and their development over time including changes in Swedish codes. Chapter 3 deals with deterioration processes in reinforced concrete in general and in prestressed bridges in particular. Chapter 4 discusses prestressed concrete bridges in Sweden, statistics and developments. In Chapter 5 general assessment procedures for concrete structures is presented and in Chapter 6 a detailed description of NDT methods and their pro- and cons are presented. Chapter 7 gives an insight of strengthening possibilities of prestressed bridges. In Chapter 8 we present proposals for future actions and finally in Chapter 9 we discuss some social economic consequences if not measures are taken.

# 2 Development of prestressed concrete structures

## 2.1 Early development

Ever since concrete was started to be used by the Romans some 2000 years ago, the positive influence of externally applied compressive stresses have been utilized to avoid cracking. Early examples are arches and domes with mainly compressive stresses, which were balanced by forces from the abutments, see Figure 2.1. The structures were not only made of concrete but – more often – of stones and brick.

Figure 2.1. Load-bearing principle for an arch with mainly compressive stresses, www.wikimedia.org and Nilsson (2014).

After the Roman Empire collapsed, use of concrete became rare until the technology was redeveloped in the mid-18th century.

A great step forward in the modern use of concrete was Smeaton's Tower, built by the British engineer John Smeaton in Devon, England, between 1756 and 1759. It pioneered the use of hydraulic lime in concrete, using pebbles and powdered brick as aggregate, Davis (1924).

A industrialized method for producing cement was developed in England and patented by Joseph Aspdin in 1824. It is a fine powder, produced by heating limestone and clay minerals in a kiln to form clinker, grinding the clinker, and adding 2 to 3 percent of gypsum. Aspdin chose the name *Portland cement* for the product for its similarity to Portland stone, which was quarried on the Isle of Portland in Dorset, England. His son William continued developments into the 1840s, earning him recognition for the development of "modern" Portland cement, Davis (1924).

*Reinforced concrete* was invented in 1849 by the French gardener Joseph Monier. He began making concrete pots and tubs, but these were not stable enough. In order to strengthen the concrete containers, he experimented with embedded iron mesh and later on also built a small boat and a bridge. The first house with reinforced concrete was built by François Coignet in 1853, Haegermann et al (1964).

*Prestressed concrete* may in some way be considered as a new material, the difference compared with reinforced concrete being that the reinforcement in the latter is *passive*, i.e. it comes under load when outside forces act on the structure, while in *prestressed concrete* the reinforcement is active, i.e. it is *tensioned* before the loads of the structure will be receiving act (own weight, dead weight and traffic loads), compressing the concrete so it never has tensions or, if it does, they will be small, Troyano (2003). Tensioning of the reinforcement may be performed before concreting the elements, i.e. *pre-tensioning* them, or put them under load after concreting the item, i.e. *post-tensioning* them. The terminology used by the Euro codes will be used here.

The process of **prestressing** consists in applying forces to the concrete structure by stressing **tendons** relative to the concrete member. "**Prestress**" is used globally to name all the permanent effects of the prestressing process, which comprise internal forces in the sections and deformations of the structure. (Eurocode 2, EN 1992-1-1:2005, section 1.5.2.4).

The prestress is applied by **tendons** made of **high-strength steel** (**wires, strands** or **bars**). Tendons may be **embedded** in the concrete. They may be **pre-tensioned** and **bonded** or **post-tensioned** and **bonded** or **unbonded**. Tendons may also be **external** to the structure with points of contact occurring at deviators and anchorages. (Eurocode 2, EN 1992-1-1:2005, section 2.3.1.4). **Couplers** may be used to join the tendons.

The action of a post-tensioned concrete beam is illustrated in Figure 2.2.

The basic idea of prestressed concrete - to tension the reinforcement and compress the concrete in order to prevent cracking and to reduce deformations - appeared with more or less clarity almost at the same time as reinforcement for concrete was started to be developed around 1850.

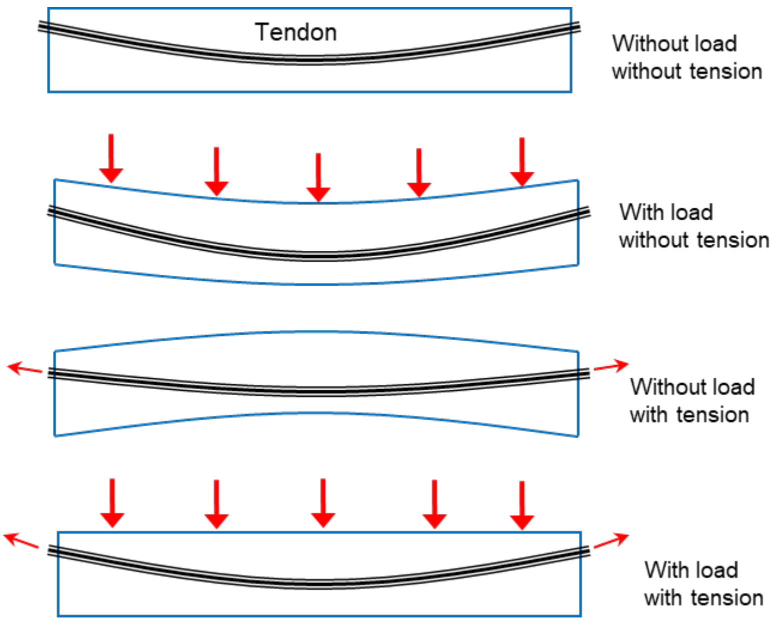



Figure 2.2 Principle for the behavior under load of a concrete beam without and with post-tensioned reinforcement, based on (Phoenix7777 - https://commons.wikimedia.org )

At the beginning of the 1860s, Armand Considére, a French builder, built a quay of granite blocks interlocked by vertical prestressed bars in Finisterre in France. The bars were anchored by their thread and a nut, a system that has been used in all attempts at prestressing bars and is still used in present-day bar systems, Troyano (2003).

In 1888 Peter H Jackson registered a patent in the US on prestressing of a concrete floor using bars anchored with nuts or wedges, Sanabra-Loewe & Capellá-Lloera (2014), see Figure 2.3.

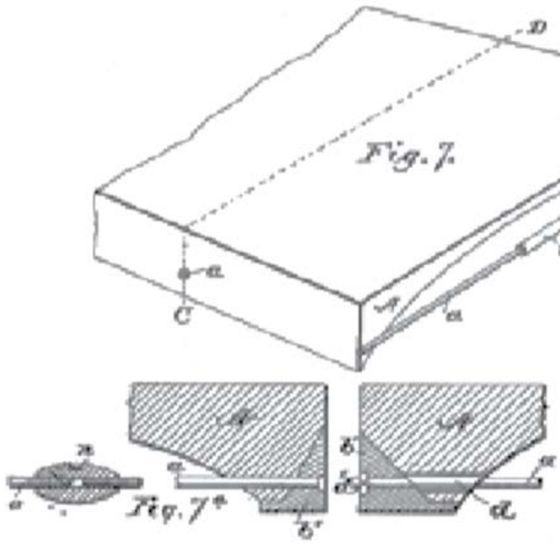


Figure 2.3. Prestressed concrete floor by P. H. Jackson, U.S. Patent 375999 (1888), Sanabra-Loewe & Capellá-Lloera (2014),

Many patents for prestressed reinforcements followed, although they were not much used. One of the fundamental problems was concrete shortening through shrinkage and creep. These deformations gave rise to the loss of the stress in the reinforcement and, therefore, prestressing lost all its effectiveness.

### 2.2 Introduction of high strength steel

Eugène Freyssinet in France started to use high-strength cold-drawn alloyed steel with a yield stress $f_y$ > 800 MPa (as compared to $f_y \approx 250$ MPa for ordinary reinforcement). The higher steel stress could compensate the losses due to shrinkage and creep. He registered a patent in France 1928 and in other countries shortly afterwards, Freyssinet (1950), Lacroix (2004), Troyano (2003), Shushkewich (2012). He later introduced the Freyssinet system comprising a conical wedge anchor for 12-wire tendons, Godart (2014), Troyano (2003), see Figure 2.4. Freyssinet also summarized the idea of prestressing in the foreword to a book by Yves Guyon (1951): "This idea is of an extreme simplicity in its foundation, even if it is not in the execution."

Figure 2.4. Freyssinet anchor circa 1935 with twelve 7 mm diameter wires. From Troyano (2003).

### 2.3 After World War II to 1960

During World War II and thereafter, it became necessary to reconstruct in a prompt manner many of the main bridges that were destroyed by war activities. Gustave Magnel of Ghent, Belgium, and Yves Guyon of Paris, France, extensively developed and used the concept of prestressing for the design and construction of numerous bridges in western and central Europe, Magnel (1948), Guyon (1951). The Magnel system used wedges to anchor the prestressing wires. They differed from the original Freyssinet wedges in that they were flat in shape, accommodating the prestressing of two wires at a time. P. W. Abeles in England introduced and developed the concept of partial prestressing between the 1930s and 1960s, see e.g. Abeles & Bardhan-Roy (1981).

Fritz Leonhardt in Stuttgart designed several bridges and other structures and published a first version of a text book in 1955, which was much use in Sweden, Leonhardt (1964). He has also published on aesthetics of bridges, Leonhardt (1984). Ulrich Finsterwalder, developed the cantilever method of construction using bars (Dywidag) that were successively connected to each other and pre-stressed as cantilevers on both sides of a column ("freivorbau"). His Bendorf Bridge over the Rhine at Koblenz, Germany, was completed in 1965 with thin piers and a centre span of 202 metres, Figure 2.5. The double cantilevering method saved money through the absence of scaffolding in the water and also by allowing for reduced girder depth and consequent reduction of material where the ends of the deck meet in the centre. The resulting girder has the appearance of a very shallow arch, elegant in profile

Figure 2.5. Free cantilevering bridge with a span of 202 m from 1962 by Ulrich Finsterwalder. Rhine near Bendorf and Koblenz Photo by Holger Weinandt 2014, Wikipedia.

In 1952 the International Federation for Prestressing (FIP, Fédération Internationale de la Précontrainte) was inaugurated at an international meeting held in Cambridge, England. In 1953 the European Committee for Concrete (CEB, Comité Européen du Béton) was founded, on an initiative of French contractors, by André Balency-Béarn (France), Louis Baes (Belgium), Emile Nennig (Luxembourg), Hubert Rüsch (Germany), Euardo Torroja (Spain) and Georg Wästlund (Sweden). Both organizations start4e to arrange conferences and publishing reports.

In 1953 the first codes for prestressed bridges were published in France, Godart (2014).

In Sweden the old concrete code from 1949 was still valid, SOU 1949:64 (1949).

## 2.4 From 1960 to 1980

During these decades the rules were refined with more detailed rules on material properties as steel relaxation and friction, injection grout properties, and that concrete compressive stresses were to be above zero (full prestressing). Accelerators as calcium chloride ($CaCl_2$) was forbidden for concrete and grout and steel properties were improved to reduce sensitivity for corrosion and fatigue.

In Sweden a handbook on design of prestressed concrete was published by Lorentsen (1963) as well as chapters in general handbooks, Müllersdorf (1961), BYGG 9 (1966).

## 2.5 From 1980 to 2010

The codes were further developed under influence of the first CEB-FIP Model Code (1978) and its successor CEB-FIP Model Code (1990).

Calculations started to be performed in the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS) with use of partial safety factors $\gamma$.

V. Mikhailov of Russia, and T. Y. Lin of the United States contributed to the art and science of the design of prestressed concrete. Lin's load-balancing method deserves particular mention in this regard, as it considerably simplified the design process, particularly in continuous structures, see e.g. Lin & Burn (1981). An influential American text book was published by Michael Collins and Denis Mitchell in Toronto, Collins & Mitchell (1991). In Europe, Christian Menn in Switzerland also wrote well known text book, Menn (1990). These twentieth-century developments have contributed to the extensive use of prestressing throughout the world.

In 1998 the International Federation for Structural Concrete (*fib*) was formed by merger of the Euro-International Committee for Concrete (CEB) and the International Federation for Pre-stressing (FIP). *fib* has published a third version of the international model code, *fib* Model Code (2010) and has a web page with all of its and its predecessor's publication at https://www.fib-international.org.

During this period common Eurocodes were developed. The work has now resulted in 10 Eurocodes: EC0 to EC 9 (2005-2007), see Johnson (2009) for a critical overview of the process.

Swedish textbooks on prestressed bridges were published by Petersson & Sundquist (1997, 2001) and Engström (2011). Håkan Sundquist also published a short history of teaching and research in bridge building at the Royal Institute of Technology in Stockholm, KTH, Sundquist (2016). A lot of information and references regarding the early development of bridge building in Sweden is given in Larsson (1997) and Ahlberg & Spade (2001).

Today, prestressed concrete is used in buildings, underground structures, TV towers, floating storage and offshore structures, power stations, nuclear reactor vessels, and numerous types of bridge systems including segmental and cable-stayed bridges. The success in the development and construction of all these landmark structures has been due in no small measure to the advances in the technology of materials, particularly prestressing steel, and the accumulated knowledge in estimating the short- and long-term losses in the prestressing forces.

## 2.6 Swedish codes

A summary of the development of the Swedish codes is given in Table 2.1. Earlier summaries are given in Bygginnovation (2010) and TRV Tåglast (2010).

Before 1960 the old concrete code from 1949 was still valid, SOU 1949:64 (1949). For bridges there were rules issued by the road and rail authorities (Kungliga Väg- och Vattenbyggnadsstyrelsen, KVVS, and Kungliga Järnvägsstyrelsen also called Statens Järnvägar, SJ)

In the 1960ies a new series of codes for concrete was published B5 (1965), B6 (1968) and B7 (1968). They were still based on allowable stresses.

In 1976 a new version was issued on the rules for bridges, VV (1976). It has formed the basis for many later editions and modern traffic loads were introduced.

The first codes for concrete structures with limit states and partial coefficients were published in the 1970ies: BBK (1979) and BBK 94 (1994). They were followed by revised editions of handbooks, BHB-K (1990), BHB-M (1994) and were also introduced in VV (1988) together higher demands on durability, crack widths and design for fatigue. This was a major change in the way structures were designed.

In 2002 classes for environmental exposure were introduced which resulted in higher demands on durability, concrete qualities and increased amount of surface reinforcement, BRO 2002 (2002). The risk for concrete cracking due to temperature effects during the hardening process should be calculated and actions during construction should be documented. The section on maintenance in earlier version was moved to a separate code, ATB Bro-UH (2002).

In BRO 2004 (2004), the revised concrete code BBK 04 (2004) was introduced with new rules for design for shear and fatigue in closer agreement with the Euro codes, which kept improving. The status of the codes was changed to Handbooks. The Swedish version of the Eurocode 2-1-1 for concrete was issued in 2008 and the version for concrete bridges in English in 2005, SS-EN 1992-2:2005.

Table 2.1 Development of Swedish Codes for Concrete Structures and Prestressed Concrete Bridges

| **Time** | **Concrete Structures** | **Road Bridges** | **Rail Bridges** |
|---|---|---|---|
| 1945-1960 | Design based on allowable stresses<br>SOU 1949:64 (1949) | Rules from the Royal Board of the Roads and Waterways (KVVS) | Rules from the Royal Board of the Railroads (SJ)<br>SJ Bro (1955) |
| 1961-1980 | B5 (1965), B6(1968), B7(1968) | KVVS (1961, 1965)<br>VV (1969) Bronorm 69<br>VV (1976) Bronorm 76<br>Base for modern traffic loads<br>VV (1988) Bronorm 88 | |
| 1981-2010 | Serviceability and Ultimate Limit States, SLS & ULS<br>BBK 79 (1979),<br>BBK 94 (1995):<br>Eurocode 2, General rules<br>SS-EN 1992-1-1:2005 (2008) | BRO 94 (1994)<br>BRO 97 (1997)<br>BRO 2002 (2002)<br>BRO 2004 (2004)<br>ATB Bro-UH 2002 (2002).<br>TK Bro (2009)<br>TR Bro (2009)<br>SS-EN 1992-2:2005 (2005) | Järnvägsbronorm (1992)<br>BV BRO (1995 - 2006)<br>BV Bärighet (1996, 2000)<br>TK Bro (2009)<br>SS-EN 1992-2:2005 (2005) |
| 2011- | BBK 04 (2004)<br>Further development of Eurocodes | Almost annual revisions of Rules and Advice for Design, Capacity assessment and Inspection, see e.g.<br>TRV Design Rules (2016)<br>TRV Design Advice (2016)<br>TRV Capacity Rules (2016)<br>TRV Capacity Advice (2017)<br>TRV Inspection Rules (2018)<br>TRV Inspection Advice (2018)<br>Further development of Eurocodes | |

In Table 2.2 the development of design loads is summarized.

Table 2.2 Traffic Loads on Bridges, von Olnhausen (1991), Coric *et al.* (2018).

| **Building year** | **Axle load, kN** | **Bogie load, kN** | **Gross load, kN** | **Iron Ore Railway Line, Axle load, kN** |
|---|---|---|---|---|
| Before 1931 | 20-60 | - | 40-120 | 140 |
| 1931-37 | 28-75 | - | 36-100 | 140 |
| 1937-44 | 45-80 | 100 | 70-150 | 140 |
| 1944-77 | 196* | - | 330-650 | 250 |
| 1977-88 | 210* | 420*[1)] | Ca 720* | 250 |
| 1988-2009 | 210* | 420*[2)] | Ca 720* | 300 |
| 2009- | 270* | 540*[2)] | Ca 760* | 325 (2018) |

Notes: *includes dynamic amplification. Distance between bogies [1)]2.5 m [2)]1.5 m

# 3 Corrosion and other deterioration processes – examples of failures

## 3.1 General

Many of the prestressed concrete bridges designed after the Second World War, now have come to an age where their condition and safety have to be checked. Most of them are in good shape and show quite a high capacity when tested to failure. In Kiruna, in Northern Sweden, a 55-year-old bridge was going to be torn down due to ground settlements caused by mining. The bridge was before its demolition tested to failure and obtained a ductile bending-shear failure at a load more than twice the one predicted with the Euro codes, Bagge (2017), Bagge et al. (2017, 2018a, b), Elfgren et al. (2015). It will be further discussed in Appendix B.

However, besides ordinary concrete deterioration there is also a severe risk for corrosion and/or fatigue of the post-stressing reinforcement. There have been accidents, which have shown critical points in design and construction. A few of them are presented below but first a short introduction to corrosion will be given.

The corrosion of structural steel is an electrochemical process that requires the simultaneous presence of moisture and oxygen. Essentially, the iron in the steel is oxidised to produce rust, which occupies approximately six times the volume of the original material. The rate at which the corrosion process progresses depends on a number of factors, but principally on the 'micro-climate' immediately surrounding the structure, Sederholm (2006), Steel Construction (2019). The corrosion of steel can be considered as an electrochemical process that occurs in stages, see Figure 3.1. Initial attack occurs at anodic areas on the surface, where ferrous ions go into solution. Electrons are released from the anode and move through the metallic structure to the adjacent cathodic sites on the surface, where they combine with oxygen and water to form hydroxyl ions. These react with the ferrous ions from the anode to produce ferrous hydroxide, which itself is further oxidised in air to produce hydrated ferric oxide (i.e. red rust).

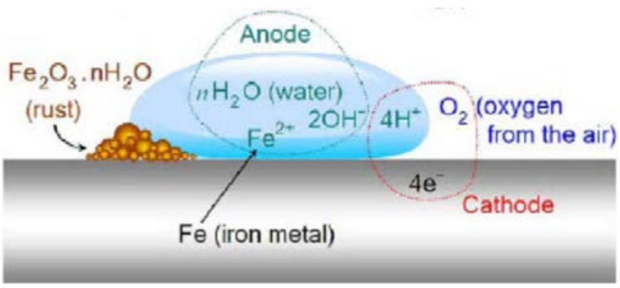



Figure 3.1 Corrosion principle. At the anode the iron loses electrons: $Fe \rightarrow Fe^{2+} + 2e^-$ and at the cathode the oxygen absorbs the electrons: $O_2 + 4e^- + 2H_2O \rightarrow 4OH^-$. The ferrous ions then react with the hydroxyl ions: $Fe^{2+} + 2OH^- \rightarrow Fe(OH)_2$ and the ferrous hydroxide forms together with oxygen hydrated ferric oxide (rust): $4Fe(OH)_2 + O_2 \rightarrow 2Fe_2O_3 + 4H_2O$. From Total Materia (2011)

A graphical representation of the thermodynamically stable regions of an aqueous electrochemical system for different potential and pH combinations for iron is illustrated in Figure 3.2 in a so called Pourbaix diagram, Pourbaix (1973), Berrocal (2017). In it three different thermodynamic corrosion regions can be identified: an immunity region, a passivity region and an active corrosion region From the diagram in Figure 3.2 it can be observed that at very low potentials, the steel is in the immunity region, which means that corrosion is not thermodynamically favored. When potentials increase, for very high pH values, as is the case with the pore solution of concrete, the steel is in the passivity region. This means that under high alkalinity conditions, a very thin, dense and stable iron-oxide film is formed on the surface of the steel. This film, often referred to as the passive layer, greatly reduces the ion mobility between the steel and surrounding concrete; thus, the rate of corrosion drastically drops and becomes negligible.

Therefore, under most conditions, well designed and executed reinforced concrete structures will present good durability as the concrete provides protection against corrosion of the reinforcing steel, Berrocal (2017). However, for lower pH values and higher potentials corrosion takes place.

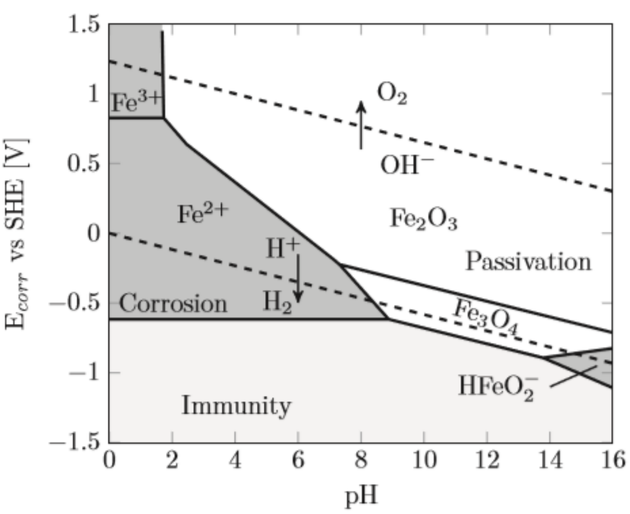



Figure 3.2. Corrosivity for iron, *Fe*, in water at 25°C for different potential and pH combinations. Pourbaix (1973), Berrocal (2017).

The rate of corrosion will be augmented if the steel is subjected to tensile stresses. This is called **stress corrosion** and can lead to sudden failure of normally ductile metals subjected to a tensile stress, especially at elevated temperatures. The tensile stresses will lead to the growth of crack formation in a corrosive environment. This is called **stress corrosion cracking, SCC,** and has been a problem for early qualities of pipes and prestressing steels, FIP (1980), Davis (2000), Nürnberger (2002), Roth (2004), CEPA (2007).

Another problem is **hydrogen embrittlement** which is the process by which hydride-forming metal alloys such as titanium, vanadium, zirconium, tantalum, and niobium become brittle and fracture due to the introduction and subsequent diffusion of hydrogen into the metal. When assisted by a concentration gradient where there is significantly more hydrogen outside the metal than inside, hydrogen diffusion can occur even at normal temperatures. These individual hydrogen atoms within the metal gradually recombine to form hydrogen molecules, creating pressure from within the metal. This pressure can increase to levels where the metal has reduced ductility, toughness, and tensile strength, up to the point where it cracks open (*hydrogen-induced cracking*, or HIC), Ghasemi (2011), Recio et al. (2013).

Corrosion also reduces the fatigue capacity of a metal, so called **corrosion fatigue,** Davis (2000), Kühn et al (2008). ACI 222.2R-14 (2014).

Prestressing steel is often more sensitive to corrosion than reinforcement steel bars. This is because the diameter of the tendons is mostly relatively small and high-grade steel is more susceptible to corrosion compared to ordinary reinforcement steel. Even a small uniform corrosive layer or a corroded spot can substantially reduce the cross-sectional area of the steel. The exposition of unprotected steels to the environments, even for a few months, can produce a reduction of mechanical properties but also in the fatigue life. If unbonded tendons are used, they must be protected by anti-corrosive material such as asphalt, grease, oil or a combination of grease and plastic tubing, Nordin (2005)

### 3.2 Fatigue in cable couplers in 1977.

The use of cable couplers in construction joints of prestressed concrete bridges in order to transfer the prestressing force is a normal practice. In a study in Germany, damage was observed in some 1200 out of 2200 examined bridges, many in the form of a zone of cracking close to the coupling sections. Among other things, these cracks gave rise to an increased stress range in the tendons and a fatigue failure of a construction joint with a cable coupler was observed in a ten-year-old bridge in Heerdter Dreieck in Düsseldorf in 1977, see Figure 3.3, fib Bull 26 (2003). Two separate prestressed concrete bridges formed a flyover at the Heerdter crossing. The bridge decks were cast in place as continuous single-hollow-core girders. As the longitudinal prestressing, single smooth tendons made of 26mm diameter bar of steel grade St 850/1050 (yield stress $f_y$ = 850 MPa, ultimate strength $f_u$ = 1050 MPa) were used. At section joints the tendons were coupled with threaded connections using sleeve couplers.
In 1976 on one of the bridges fly overs (built in 1956), increased crack widths were clearly visible in the botton1 plate at several coupling joints in the curved region. Fine cracks (w < 0,2 mm), which had been detected earlier increased in widths up to 2 mm. Gamma-ray inspection confirmed that brittle fracture of the tendons in the lowest parts of the web was the reason for the cracks, Pfohl (1979), Kordina (1981),

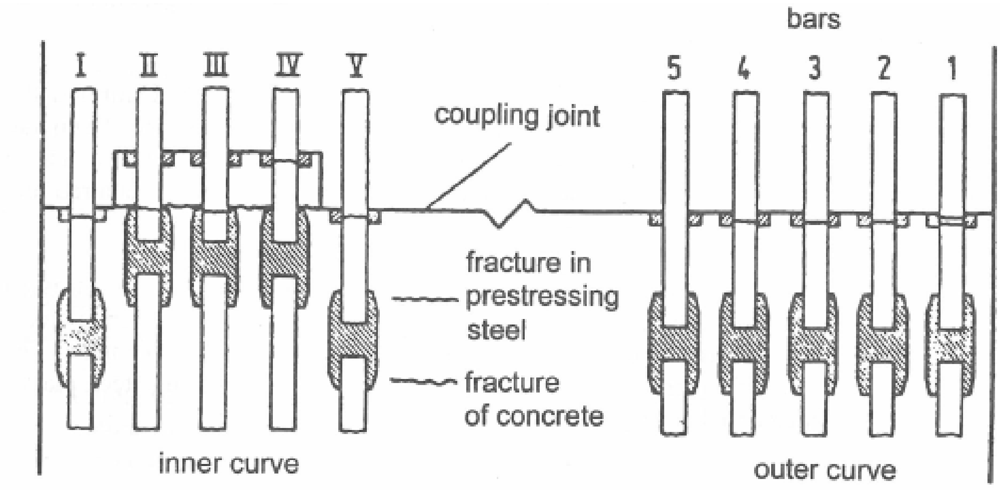



Figure 3.3. Location of Dywidag ϕ26 mm prestressing tendons and adaptors and development of fracture in the area of the coupling joint in Heerdter Dreieck.in Düsseldorf 1977. fib Bull 26 (2003).

A study of fatigue of cable couplers was subsequently initiated by the Swedish Road Administration, see Figure 3.4, Emborg et al. (1981), Emborg (1988). It was found that beams with couplers only could carry 10 to 15 % of number of load cycles for a beam without couplers. The following proposals were made:

(a) improve the design of the coupling units to avoid stress concentrations,

(b) avoid concrete tensile strains in the coupling area by better analysis, and

(c) introduce non-tensioned reinforcement in the coupling area to reduce cracking.

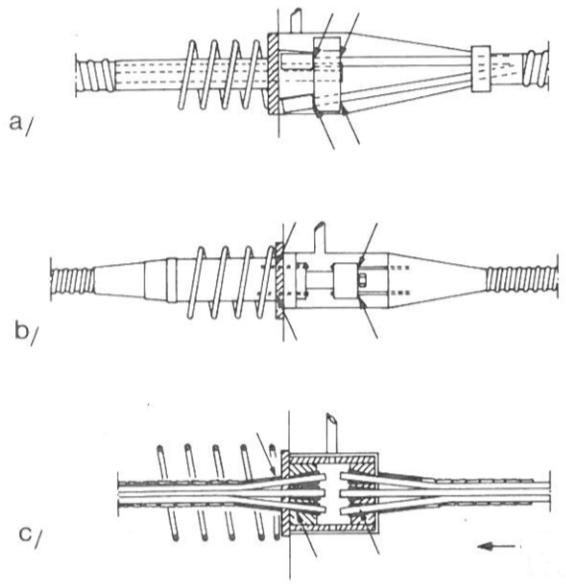



Figure 3.4. Cable couplers of standard type tested in fatigue: (a) VSL 3ϕ13 mm; (b) BBRV 6ϕ12 mm; (c) Freyssinet 6ϕ13 mm (here used with only 3 strands). The arrows indicate fatigue failure. From Emborg (1988).

### 3.3 Corrosion failure of precast prestressed bridges in 1985 and 2014

A 32-year-old bridge collapsed 1985 in Ynys-y-Gwas, Wales, due to corrosion in the pretensioned cables. The bridge consisted of nine 18,3 m long prefab beams each one consisting of 8 elements (2,41m), see Figures 3.5 and 3.6. Prestressed concrete bridges were then prevented in the UK until 1992, Woodward & Williams (1988), Woodward (1989).

Another failure took place in Sicily in 2014, when a 30 years old bridge with a single span of 30 m carried by prefabricated prestressed I-beams collapsed due to chloride induced corrosion of the strands, Collapse was determined by the breakage of the tendons and the wires inside the tendons appeared completely rusted, Figure 3.7., Anania et al (2018).

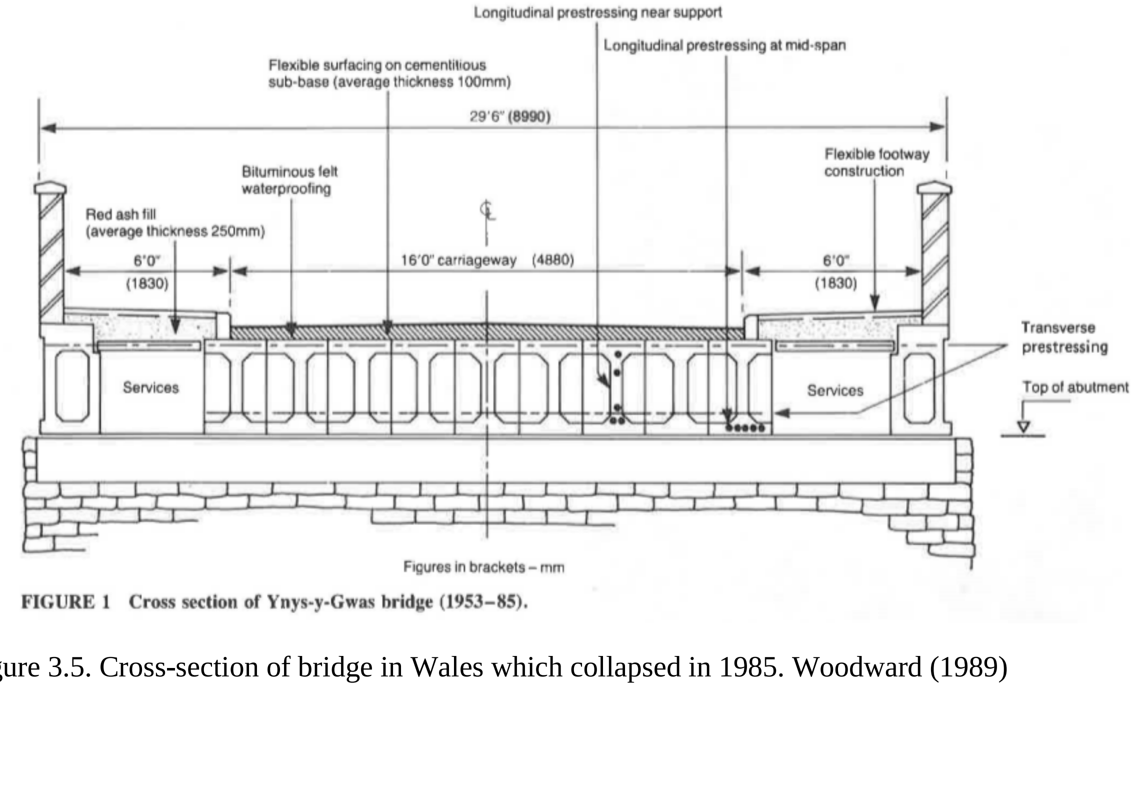



**FIGURE 1 Cross section of Ynys-y-Gwas bridge (1953–85).**

Figure 3.5. Cross-section of bridge in Wales which collapsed in 1985. Woodward (1989)

Figure 3.6. Collapse of bridge in Wales 1985. Woodward (1989).

Figure 3.7. Mid-span fracture of prefabricated prestressed beams in Sicily in 2014, Anania et al (2018).

### 3.4 Collapse of cables encased in prestressed concrete in 2018

The failure of one of the three towers of the 40-year-old bridge over Polcevera River in Genoa, Italy, in August 2018 has been much discussed, Figure 3.8. The cause seems to be corrosion combined with fatigue of a bearing cable stay supposed to be protected by a cover of prestressed concrete, Figure 3.9. However, the concrete had cracked and the cable stays in the other two towers had been strengthened already in the 1990ies, Camomilla (1995), Malerba (2010), Calvi et al. (2018).

Figure 3.8. The Polcevera bridge in Genoa, Italy, from 1967, designed by Ricardo Morandi, partly collapsed on August 14, 2018. A cable stay broke in the tower to the left in the picture, probably due to corrosion and fatigue caused by cracks in the prestressed concrete supposed to protect it, see Figure 3.9. Photo by Davide Papalini, https://commons.wikimedia.org/w/index.php?curid=11542499

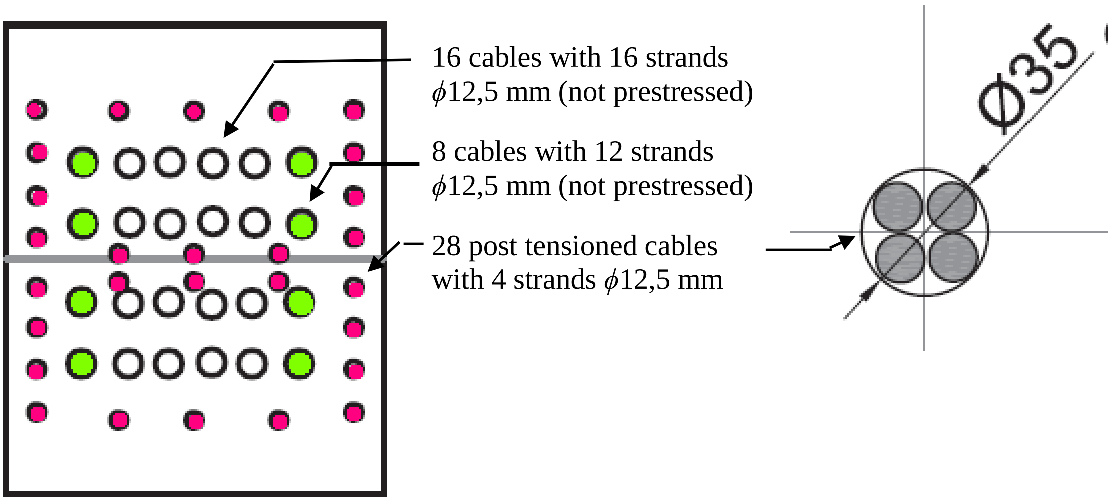



Figure 3.9. Detail of the concrete encased cable stays (0,98 x 1,22 m) in the Morandi (Polcevera) bridge in Italy. The outer small red circles mark 28 post-tensioned cables consisting of 4 strands $\phi$12,5 mm in $\phi$35 mm ducts. Each strand consists of 7 wires $\phi$4 mm. The cables were used to apply a compressive stress of originally $\sigma_c \approx$ - 8.7 MPa on the concrete. However, at maximum live load it turned into a tensile stress of $\sigma_c \approx$ 1.6 MPa, enough to crack the concrete and start corrosion in the cables. Creep and shrinkage of the concrete widened the cracks during the years. The $\phi$35 mm ducts are quite narrow making it hard to inject them to protect the cables. This may have resulted in voids facilitating the corrosion. Modified from Calvi et al. (2018).

## 3.5 General deterioration of concrete

General information on concrete and concrete deterioration is presented in e.g. BHB-K (1980), BHB-M (1994, 2018)) and *fib* Model Code (2010).

Cracks in concrete can be caused by drying shrinkage, thermal expansion, freeze thaw cycling and chemical reactions. It can also be caused by mechanical processes such as fatigue or overloading. A surface crack can be a significant indication of defects in the structure that might lead to failure or loss of structural integrity. Related to cracks, delamination can be discussed, which often are corrosion-induced and appears as horizontally cracked layers often 5-15 cm below the concrete surface. Eventually delamination can cause spalling severely reducing the durability of the structure. In this case detection of corrosion would be helpful to prolong the service life. (Al-Neshawy, et al., 2016)

One common defect for post-tensioned concrete bridges is lack of grout in the post-tensioned ducts. If the tendon duct lacks grout it might initiate corrosion, leading to a “brittle” failure mode. It is therefore crucial to identify possible un-grouted sections. By finding the tendon ducts and possible voids, for example by an appropriate NDT (Non-Destructive Testing) method, and then drilling down, an endoscope can be used to find possible corrosion in these voids (International atomic energy agency, 2002), see Chapter 6.

The process with time can be divided in three phases, an initial phase, a more or less steadily progressing phase and a final accelerating failure phase, see e.g. SB-D3.4 (2007), SB-ICA (2007), SB-LRA (2007).

In Al-Neshawy, et al. (2016) three major deterioration phases during the service life of concrete structures are established. Figure 3.10 shows these phases with some additional information.

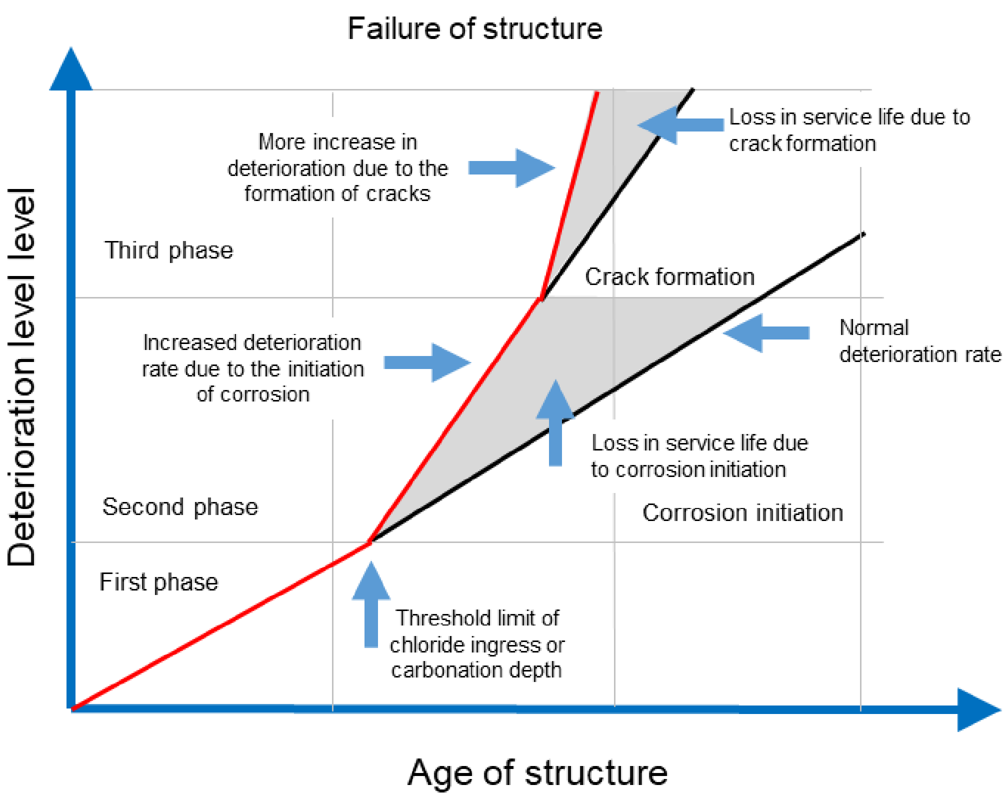



Figure 3.10 General deterioration phases for a concrete structure, based on Al-Neshawy et. al.,(2016)

The first phase occurs after construction, the structure already starts to deteriorate, but before any corrosion initiation. The second phase is the period between corrosion initiation and crack formation and the third phase is given by the period after crack formation and the final structural failure of the structure.

During the three phases, in general, different methods of non-destructive testing is used. For the first phase, the methods should be able to evaluate the execution details of the construction, such as its quality and homogeneity. If defects are discovered during this phase, it might be possible to avoid large repair costs during the latter part of a structures service life. NDT-methods used for this phase should detect honeycombs, measuring carbonation depth, chloride content, concrete cover and concrete quality.

The second phase is about monitoring the structure during operation. Visual inspections can only detect defects visible from the surface and the inside of the structure is difficult to assess. NDT-methods should hence give information about the inside of the structure, such as detection of cavities and air voids.

The third phase requires methods that can assess the cracking of the structure, delamination, corrosion and reinforcement placing. (Al-Neshawy, et al., 2016)

# 4 Prestressed bridges in Sweden

## 4.1 Behaviour and problems with prestressed bridges in general

The global behavior of post-stressed bridges is an important question. This is also pointed out in the concept in chapter 1. In the SLS post-stressed bridges acts linear. This is shown when real bridges have been tested and is an important input for future work with post-stressed bridges. Examples of this is presented in appendix A. Also real tests show that most bridges types have a quite large hidden capacity. The question to be answered is what happens with the behaviour when corrosion causes loss of compressive forces? Therefore to check the behavior is one way to take control over the situation.

The behavior also varies with different types of bridges. Therefore, this study looks at the most common and important bridge types in Sweden. Only a few types have been looked at due to the limited material we have had available.

Important construction improvements were introduced in the end of the seventies. This was also the situation in Sweden and reflected in Swedish codes and documents from the time. Example of this is that chlorides were abandoned, clearer rules regarding grouting and better ducts were introduced. However, negligence and mistakes in production fade out after some years and not directly. Due to this we recommend to investigate bridges built before 1980 in a first step.

Only four post-tensioning systems was allowed in Sweden at this time. These allowed post-tensioning systems were also the most common in Europe. Thus, international reports and papers have been a very relevant input to this study.

## 4.2 Prestressed bridges in Sweden

Generally Swedish bridges are built with good quality. They are also well maintained. All bridges are documented in a bridge management system named BaTMan. Due to this it is quite easy to get a rough picture of the situation.

Trafikverket only pays attention to prestressed bridges when visible cracks or damages have been seen during normal inspections. Very few in-depth inspections have been carried out.

Since it is a huge work to go through all documents in BaTMan, a first step was to look at all prestressed bridges and sort out the most common bridge types. From these main types, representative examples were chosen. It was also important to choose bridges with different post-tensioning systems.

These bridges was than studied more in detail see appendix C. See also limitations in previous chapter.

## 4.3 Bridge statistics from Trafikverket

The Swedish prestressed bridge stock is reported by Trafikverket, in the bridge management system BaTMan, see Table 4.1 and 4.2. Thera are totally 1686 prestressed bridges. These bridges represent the majority of prestressed bridges in Sweden. There are still more prestressed bridges managed by other owners, for example Stockholm municipality and other cities. So totally a rough estimation gives approximately 2000.

Table 4.1 Prestressed bridges managed by Trafikverket (BaTMan)

| | **Number** | **Total length, median [m]** | **Total length, average, [m]** | **Length total [km]** |
|---|---|---|---|---|
| **Road Bridges** | 1364 | 62 | 96 | 131 |
| **Rail Bridges** | 308 | 59 | 111 | 34 |
| **Pedestrian Bridges** | 14 | 45 | 50 | 0.7 |

Table 4.2 Age distribution of prestressed bridges managed by Trafikverket, (BaTMan)

| **Opening year** | **-1929** | **30-39** | **40-49** | **50-59** | **60-69** | **70-79** | **80-89** | **90-99** | **2000-** |
|---|---|---|---|---|---|---|---|---|---|
| **Road Bridges** | 4 | 3 | 3 | 23 | 131 | 173 | 143 | 279 | 605 |
| **Rail Bridges** | 28 | 7 | 1 | 5 | 26 | 39 | 31 | 93 | 78 |
| **Pedestrian Bridges** | | | | | 1 | 2 | | 5 | 6 |
| **Total** | 32 | 10 | 4 | 28 | 158 | 214 | 174 | 377 | 689 |

Table 4.2 maybe misleading since the post-tensioning technique was developed after the Second World War. The Swedish bridge management system is organized based on the substructure and not the superstructure. This means that old substructures have a new superstructure built on the old abutments. This is especially common for railway bridges since old stone abutments often are of very good quality and the replacement can be carried out in a short timeslot. By doing this you also carry out bridge replacement more cost efficient since you don't have to construct a new line. In Figure 4.1 the data from Table 4.2 is shown for clarity in a bar chart.

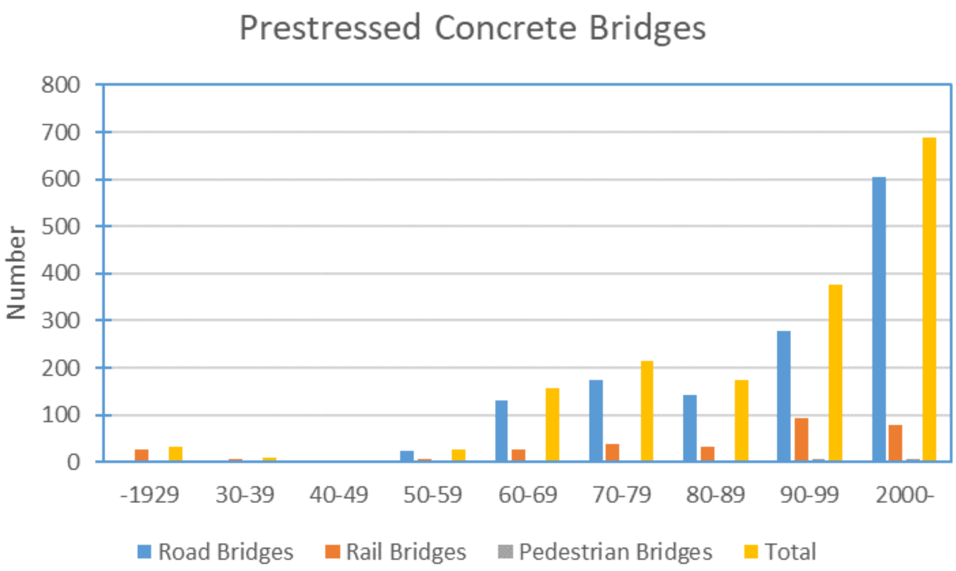



Figure 4.1 Age distribution and number of prestressed concrete bridges in the bridge stock of Trafikverket (BaTMan)

Age is also important since the codes and the equipment in the beginning (50-70) had teething troubles. For example, chlorides in grouting was forbidden from 1968 and the old ducts was replaced by thicker ducts about 1978. Another important factor is that negligence and mistakes in production sometimes start to occur after some years and not directly. This means that there is a not negligible risk that quite a large number of bridges can have corrosion problems.

A first step was to look at all prestressed bridges and sort out the most common post-stressed bridge types. From these main types, some representative examples were chosen, see appendix C. These bridges were than studied in more detail with focus on used post-stressing system. All chosen bridges were built before 1980. See limitations in previous chapter.

## 4.4 Bridge Types

The most common prestressed bridge types in Sweden are given in in Table 4.3 below.

Table 4.3 Number of different bridge types (BaTMan)

| **Bridge type** | **Number** |
|---|---|
| **Beam Bridges** | 1050 |
| **Box girder Bridges** | 350 |
| **Slab Bridges** | 200 |
| **Trough Bridges** | 80 |

In BaTMan, a lot of bridges are not correctly defined, so the numbers above are somewhat approximate but they give a rough estimation, which is sufficient for this report. In appendix A some examples of bridge types except slab bridges have been looked at.

### 4.4.1 Beam bridges:

The most important result from the failure tests in Kiruna is that a normal prestressed beam bridge showed a ductile failure and a linear behaviour far over the design load. The test also showed good conformity with used FEM-programs. The Kiruna Bridge represents a normal, well-maintained Swedish bridge.

The bridge in Marma does not show any sign of problems caused by post-tensioning from the last inspection done October 2015.

Furtherer investigations have to be carried out in order to see how general this statement is for beam bridges.

### 4.4.2 Box girder bridges:

Three brides were examined. The first was the Abisko Bridge, a railway bridge on the Iron Ore Line Luleå – Gällivare - Kiruna - Narvik. Here cracks were found. These cracks were not severe but an understanding of their causes would be desirable

Cracks in post-tensioned bridges must be better understood. Assessment calculations parallel to monitoring campaigns are recommended.

The second bridge was the Ölands Bridge. Here we looked at the result from the test-load in 1972 which showed a clear linear behavior in the Serviceability Limit Stage, SLS.

Also a special cable inspection has been carried out using endoscope and a thermograph-camera. The value of this should be studied.

The third bridge was Angeredsbron in Gothenburg. Here two different post-tensioning systems were used. The bridge was test-loaded in 1978 and it showed a clearly linear behavior for loads within 60% of the design load. There were some problems with the post-tensioning during construction but only some minor remarks were made in the inspection documents, so a more in-depth inspection should be carried

### 4.4.3 Trough bridges:

Two bridges were examined. The first was a railway bridge over E18 in Enköping. According to the inspection and other material, there are several cracks that can be related to the post-tensioning system.

The second bridge was in Örnsköldsvik and was demolished in 2005. A report, Sederhoim (2006), also summarized in SB D7-3 (2008), showed corrosion problems on wires and a high content of chlorides. This shows that more tests on post-tensioned cables should be carried out.

## 4.5 Used post-tensioning systems in Sweden

Table 4.4 below shows the allowed post-tensioning systems in Bronorm (1978). They correspond quite well with Brobyggnadsanvisningar (1968).

There are just the four systems: BBRV, Freyssinet, VSL and Dywidag that were allowed at this time. However, the size of the units must fit to the dimension of the bridge. This meant in praxis that only few units were used. These most common prestressing units are marked in red in the table. This fact will facilitate the coming work. In the beginning bars and strands was dominating with systems like Dywidag, BBRV and Freyssinet. Today wires are dominating with a market share of about 75%.

Table 4.4 Data for different prestressing systems

| System | Prestressing unit | Steel quality $f_{0,2}/f_t$ | Load at failure | Dim. for anchor plate or block | Minimum distance for anchorage[1] | Minimum distance from edge to center of anchorage[2] | Movement at anchorage | Inner diameter for duct | Distance between support in form | Concrete compressive strength at prestressing, minimum |
|---|---|---|---|---|---|---|---|---|---|---|
| | | [MPa] | [kN] | [mm] | [mm] | [mm] | [mm] | [mm] | [m] | [MPa] |
| BBRV | Wires 12 $\phi$ 6 | 1520/1770 | 600 | 140x140 | 190 (160) | 100 | 1,0 | 30 | < 0,8 | 28 |
| | Wires 22 $\phi$ 6 | 1520/1770 | 1100 | 200x200 | 250 (220) | 135 | 1,0 | 40 | < 1,0 | 28 |
| | Wires 32 $\phi$ 6 | 1520/1770 | 1600 | 220x220 | 305 (270) | 165 | 1,0 | 50 | < 1,2 | 28 |
| | Wires 44 $\phi$ 6 | 1520/1770 | 2200 | 260x260 | 350 | 190 | 1,0 | 60 | 1,2-1,5 | 28 |
| Freyssinet | Wires 12 $\phi$ 5 | 1470/1670 | 390 | $\phi$ 100 | 155 (120) | 90 | 3,0 | 30 | < 0,8 | 28 |
| | Wires 12 $\phi$ 5 | 1470/1670 | 745 | $\phi$ 120 | 215 (150) | 110 | 5,0 | 40 | < 1,0 | 32 |
| | Wires 12 $\phi$ 5 | 1320/1570 | 940 | $\phi$ 150 | 240 (180) | 125 | 6,0 | 42 | < 1,0 | 32 |
| | Tendon 12 $\phi$ 12.7[3] | 1670/1860 | 2200 | 260x270 | 350 | 190 | 8,0 | 60 | 1,2-1,5 | 28 |
| VSL | Tendon 3 $\phi$ 13 | 1560/1830 | 550 | 140x140 | 190 (160) | 100 | 4,0 | 41 | < 1,0 | 28 |
| | Tendon 7 $\phi$ 13 | 1560/1830 | 1285 | 210x210 | 290 (250) | 155 | 4,0 | 50 | < 1,2 | 28 |
| | Tendon 12 $\phi$ 13 | 1560/1830 | 2200 | 240x270 | 350 | 190 | 4,0 | 60 | 1,2-1,5 | 28 |
| | Tendon 3 $\phi$ ½''$D_y$[4] | 1670/1860 | 625 | 140x140 | 190 (160) | 100 | 4,0 | 41 | < 1,0 | 28 |
| | Tendon 7 $\phi$ ½''$D_y$[4] | 1670/1860 | 1460 | 210x210 | 300 (260) | 160 | 4,0 | 50 | < 1,2 | 28 |
| Dywidag | Bar 1 $\phi$ 26[5] | 830/1030 | 520 | 130x130 | 180 (160) | 90 | 0,5 | 32 | 1,8-2,5 | 28 |
| | Bar 1 $\phi$ 32 | 830/1030 | 785 | $\phi$ 170 | 220 (210) | 110 | 0,5 | 38 | 1,8-2,5 | 32 |

*1. The value in ( ) is valid for anchorage in one layer or most two anchorage in several layers*
*2. Assume that for respectively system demands are fulfilled*
*3. Due to ASTM-A 416-68 Grade 270*
*4. Dyform due to British Ropes Limited internal standard*
*5. This data is also valid for the known as the SH-system $\phi$ 26*

In the literature from Germany (Haveresch 2000) and France (Godart 2014) it is shown that some parts are more critical. This means that these parts shall be looked at in more in detail. Examples on this are marked with red in figure 4.2. Components of the systems have earlier been discussed in Chapter 3.

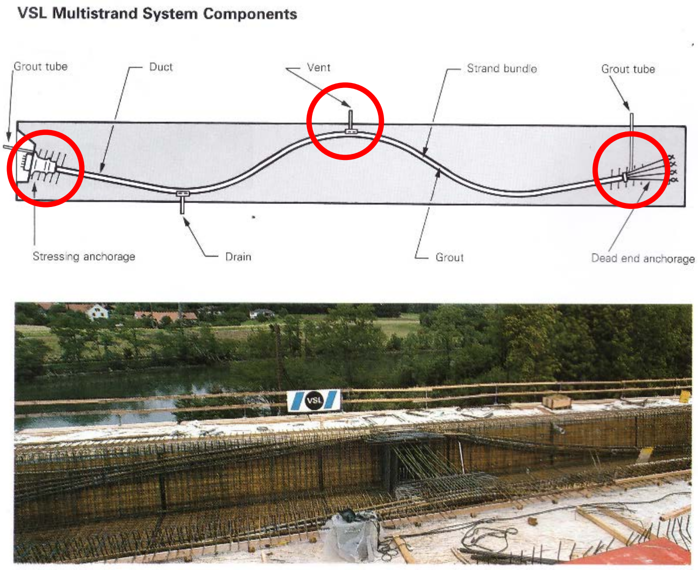



Figure 4.2. Example of critical points in wires in a post-stressed beam with anchors, ducts, drains and ventilation (VSL 1975)

### 4.6 Investigation approach

In order to answer the question how serious and extended the problem is, a structured approach is a necessity, e.g. in the following way:

- Sort the bridges into bridge types since the different types act in a similarl way in SLS and ULS. Only quite a few types are used.
- From literature and codes it is clear that the bridges built before 1980 are the most critical ones.. As a first step these bridges should be looked into
- There are three (or four) post-tensioning systems being used in Sweden and among them, there are just a few different varieties . This means that the work can be concentrated to the critical parts of these few post-tensioning components.
- In order to enhance the knowledge about the status of prestressed bridges, monitoring and inspection techniques ought to be tested on such bridges that are planned to be decommissioned, replaced or destroyed.

# 5 Safety and condition assessment of prestressed bridges

## 5.1 Safety

Today's methods for safety estimates are often based on codes, which are written for the design of new structures, e.g. the Eurocodes, EC0 – EC9 (2000-2009). Now work is going on to implement, revise and harmonize them, see e.g. EC Handbook 1-4 (2004, 2005) and JRC Bridge Examples (2012).

The philosophy for safety considerations are illustrated in Figure 5.1 where frequency functions are given for resistance $f_R(r)$, load action $f_S(s)$ (from French *sollicitation*) and the safety margin $f_M(m)$, being the difference (M) between the resistance (R) and the load action (S). Notation and Figure 3.1 are from an excellent introduction to the safety concept by Schneider & Vrouwenvelder (2017). In Figure 3.1 an example is given of e.g. a bridge deck with a mean value of the bending moment capacity $\mu_R = 15$ MNm, a mean value of the load action $\mu_S = 9$ MNm, giving the mean value of the safety margin to $\mu_M = \mu_R - \mu_S = 15 - 9 = 6$ MNm. If the frequency functions are assumed to be normally distributed with an area under the functions to be 1 (MNm) we may calculate the probability of failure. We need to know the standard variations σ of the resistance and the load. If we assume them to be $\sigma_R = 2$ for the resistance and $\sigma_S = 3$ for the load action we will obtain that the standard variation for the margin will be $\sigma_M = (\sigma_R^2 + \sigma_S^2)^{0,5} = (2^2 + 3^2)^{0,5} = (13)^{0,5} = 3{,}61$. We may then calculate how many (β) standard variations $\sigma_M$ that separate the mean value $\mu_M$ from m = 0, that is $\beta = \mu_M / \sigma_M = 6 / 3{,}61 = 1{,}66$.

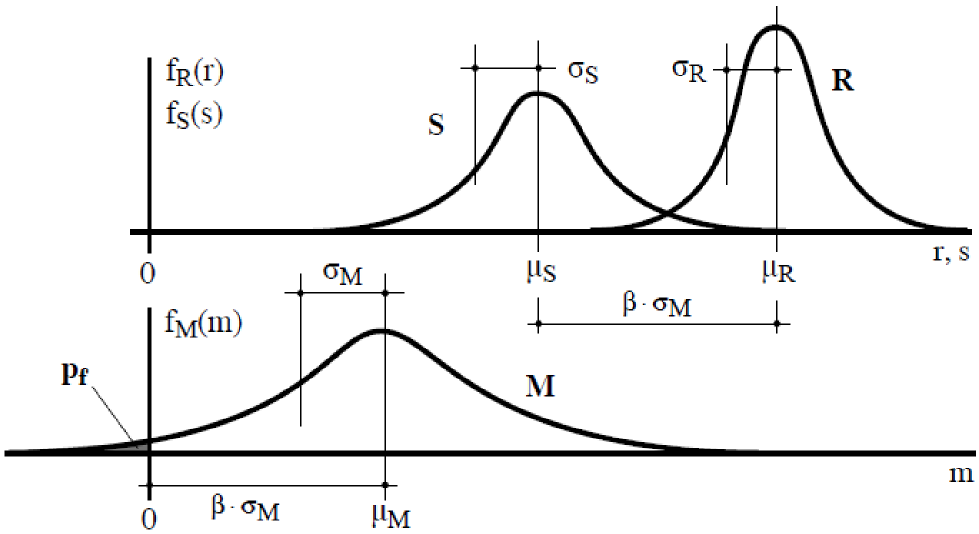



Figure 5.1. Frequency functions $f_R(r)$, $f_S(s)$ and $f_M(m)$ for resistance (**R**), load effect (French: sollicitation) (**S**) and safety margin (**M** = **R** – **S**) as functions of the resistance (r), the load (s) and the margin (m). For the bending capacity of a bridge slab we may assume the mean value of the resistance to be $\mu_R = 15$ (MNm) , of the load to be $\mu_S = 9$ and then the margin will be $\mu_M = \mu_R - \mu_S = 15 - 9 = 6$. We further assume the standard variations to be $\sigma_R = 2$, $\sigma_S = 3$ and then, if the frequencies are supposed to be normally distributed with the areas under the curves to be 1, then $\sigma_M = (\sigma_R^2 + \sigma_S^2)^{0,5} = (2^2 + 3^2)^{0,5} = (13)^{0,5} = 3{,}61$. The safety margin can be expressed as the number β of standard variations from $\mu_M$ to or m = 0, that is $\mu_M = \beta \cdot \sigma_M$. In our case we will have the reliability factor $\beta = \mu_M / \sigma_M = 6 / 3{,}61 = 1{,}66$. For a normal distribution this means that the probability of failure (the area under the $f_M(m)$-curve to the left of m = 0) will be $\mathbf{p_f} = 0{,}049$ or 4,9 % . Modified from Schneider & Vrouwenvelder (2017).

From a table for normal distributions we will find that for a reliability factor β = 1,66, the safety will be $\mathbf{p_f} = 0{,}049$ or 4,9 % which is about one failure out of 20 bridge decks, which is mostly too low a safety to be tolerated. The probability as a function of β is given in Table 5.1

Table 5.1. Reliability factors β and corresponding probabilities $p_f$

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **β** | 0 | 1 | 1,3 | 2 | 2,3 | 3 | 3,1 | 3,7 |
| **$p_f$** | 0,5 | 0,1587 | 0,1 | 0,02275 | 0,01 | 0,00135 | 0,001 | 0,0001 |
| **β** | 4 | 4,2 | 4,7 | 5 | **5,2** | 6 | 7 | 8 |
| **$p_f$** | 3,167E-5 | E-5 | E-6 | 2,867E-7 | **E-7** | 9,866E-10 | 1,280E-12 | 6,221E-16 |

When building a new structure there are uncertainties regarding geometry, material properties, construction quality and loads. These uncertainties can be balanced by a reasonable safety margin in partial coefficients ($\gamma$), special load factors for railways ($\alpha$) and reliability indices ($\beta$). The partial coefficients are safety factors used for loads and material properties, the $\alpha$-factor adjusts for rail traffic that is heavier or lighter than normal traffic and the reliability indices indicate the internationally accepted probability of failure. The ($\alpha$-factor) is used to meet todays planned situation and future needs. Typical partial coefficients in the Eurocodes for permanent loads are $\gamma_G = 1.35$ and for live loads $\gamma_Q = 1.50$. For structural material properties, typical values of the partial coefficients are $\gamma_c = 1.50$ for concrete and $\gamma_s = 1.10$ for structural steel, see e.g. EC Handbook 1 (2004). The rail traffic $\alpha$-factors may vary between 0.75 and 1.46 with $\alpha = 1.33$ recommended on lines for freight traffic and international lines, UIC 702 (2003). A probability of failure of 1 in 100 hundred, $p_f = 0.01$, corresponds to a reliability index $\beta = 2.3$, a probability $p_f = 10^{-5}$ corresponds to $\beta = 4.2$, a probability $p_f = 10^{-6}$ corresponds to $\beta = 4.7$ and a probability $p_f = 10^{-7}$ corresponds to $\beta = 5.2$ see Table 3.1, EC Handbook 2 (2005) and Schneider & Vrouwenvelder (2017).

The Eurocodes indicate target reliability indices in relation to three classes of consequences (high, normal and low) and two reference periods $T$ (1 and 50 years). For a failure with low consequences (e.g. a greenhouse) during a period of $T = 1$ year the recommended reliability index is β = 4,2 corresponding to a failure probability of $P_f = 0.00001$ (1 in 100 000) and for $T = 50$ years the recommended reliability index is β = 3.3 corresponding to a failure probability of $P_f = 0.0005$ (5 in 10 000). For a failure with high consequences (e.g. a **bridge**) during a period of $\boldsymbol{T = 1}$ year the recommended reliability index is **β = 5.2** corresponding to a failure probability of $P_f = 10^{-7}$ (1 in 10 millions). If the period is increased to $\boldsymbol{T = 50}$ years, this increases the failure probability to $P_f = 5 \cdot 10^{-6}$ (5 in 1 000 000) with **β = 4.3** or for $\boldsymbol{T = 100}$ years, this increases the failure probability to $P_f = 10^{-5}$ (1 in 100 000) with **β = 4.2**.

As a comparison, the risk of death for a person during one year varies with where the person is living and what he/she is occupied with and it changes during the lifetime. In Sweden the death probability during one year ($T$=1) for a 1 year old child is $P_f = 0.00012$ (1.2 in 10000); for a 40 year old person it is $P_f = 0,001$ (1 in 1000); for a 60 year old person it is $P_f = 0.01$ (I in 100); and for a 100 year old person it is $P_f = 0.36$ (1 in 3). The death rate due to accidents and suicide is about $P_f = 0.001$, SCB (2007). The acceptable individual risk of death during a year due to an accident caused by a structural failure is in JRC Assessment (2015) proposed to be $P_f = 10^{-5}$ which is about 1 % of the general risk for death due to accident and suicide given above for Sweden. An assessment of existing structures may be necessary when the reliability of a structure is questioned, alterations to the structural system are needed, or by requirements from authorities. This assessment can be performed by applying the same rules as for design of new structures. This will, however, in many cases show insufficient reliability. As pointed out in Vrouwenvelder and Scholten (2010), the safety assessment of an existing structure differs from that of a new one in several aspects. The main differences are:

- Increasing the safety level is usually more costly for an existing structure than during the design phase of a new.
- The remaining lifetime of an existing structure is typically less than the expected lifetime of new structures, thus the exceedance probability of certain load levels might be different.
- For an existing structure, inspections and measurements may be used to reduce uncertainties.

When assessing the capacity of an existing structure many of the uncertainties that are present when building a new structure can be resolved. The codes for assessment therefore do not need to have the same high partial coefficients $\gamma$ , $\alpha$-factors or reliability indices β, see e.g. SB-LRF (2007), ML-D1.2 (2013) and JRC Assessment (2015). In addition, standard dynamic amplification factors for the influence of dynamic loads may be reduced after a study of a structure. Thus, there is a need for special codes for assessment and upgrading of existing structures.

An early example of such codes was the Swedish BV Bärighet (1996). Example of applications of the reliability methods are given in Nilsson et al. (1999), Enochsson et al. (2002) The code has been updated several times and is presently divided in two parts, one with requirements TRV Capacity Rules (2017) and one with recommendations TRV Capacity Advice (2017). According to TRV Capacity Rules (2017) for assessment of bridges the β-value for $T$ = 1 year may be reduced from 5,2 (1 in 10 000 000) to **4,7**. (1 in 1 000 000).

## 5.2 Condition assessment in general

Concrete structures constitute a great value of today´s society. They form a great part of the infrastructure, underpinning the social, industrial and economic well-being for the community. However, concrete structures are deteriorating due to aggressive actions such as corrosion, frost, abrasion, chemical action etc. Decisions how to maintenance, repair, strengthen of replace are required over its lifespan. To make this most efficient from a client and cost perspective guiding principles should be followed. In figure 5.1 general principles for maintenance, repair and strengthening of concrete structures is presented (ISO 16311-1, 2014). Figure 5.2 consist of four important parts. Part 1: General principles which relates to the performance of the structure, where performance refer to structural safety (load carrying capacity), serviceability, appearance (aesthetics) or mitigation of for example falling debris due to lack of maintenance. In figure 5.3 the performance of a structure over time is presented in more detail.

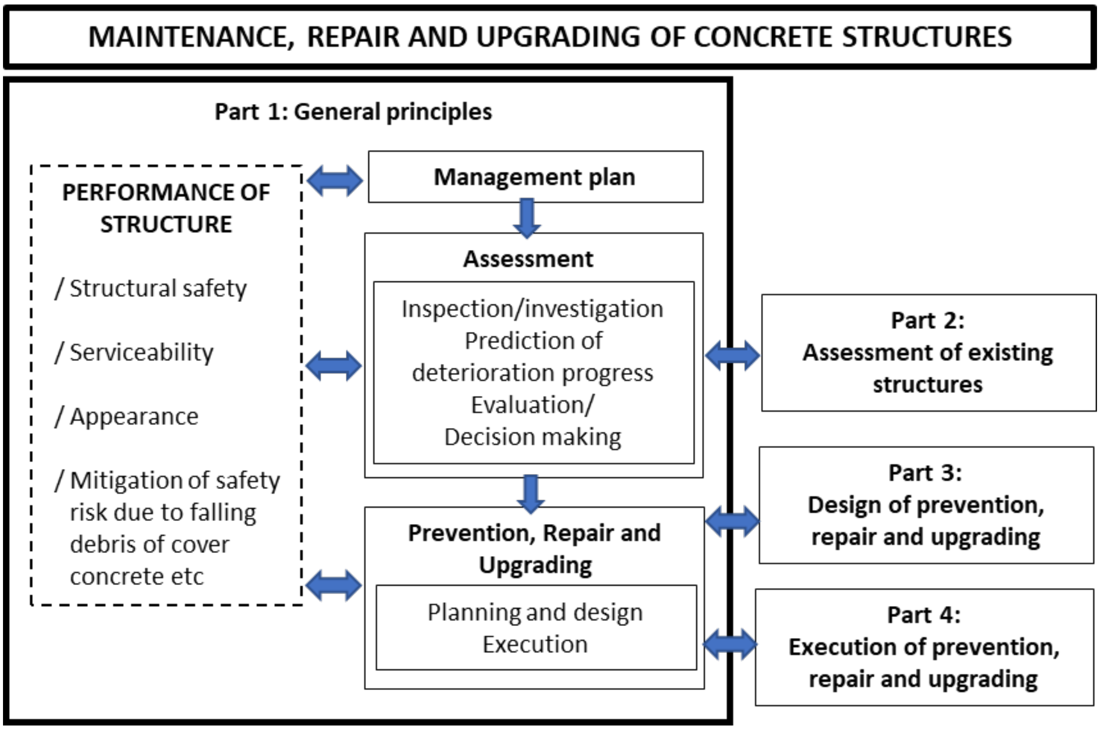



Figure 5.2 General principles for maintenance, repair and upgrading, based on (ISO 16311-1, 2014)

The structure always has an initial performance and often safety sets the lowest performance/requirement on the structure. In figure 5.2 it has been assumed that an assessment is carried out and that this assessment shows a faster degradation curve than anticipated at the design stage. There could then be different strategies for repair.

In scenario **A,** a minor repair is carried out, either as a short term repair or at two different intervals prolonging the structures life to its original planned life span. In scenario **B,** a large repair is carried out, lifting the performance to its original level and giving the structure a longer life span and finally in scenario **C,** the performance is improved as well as the lift of the structure.

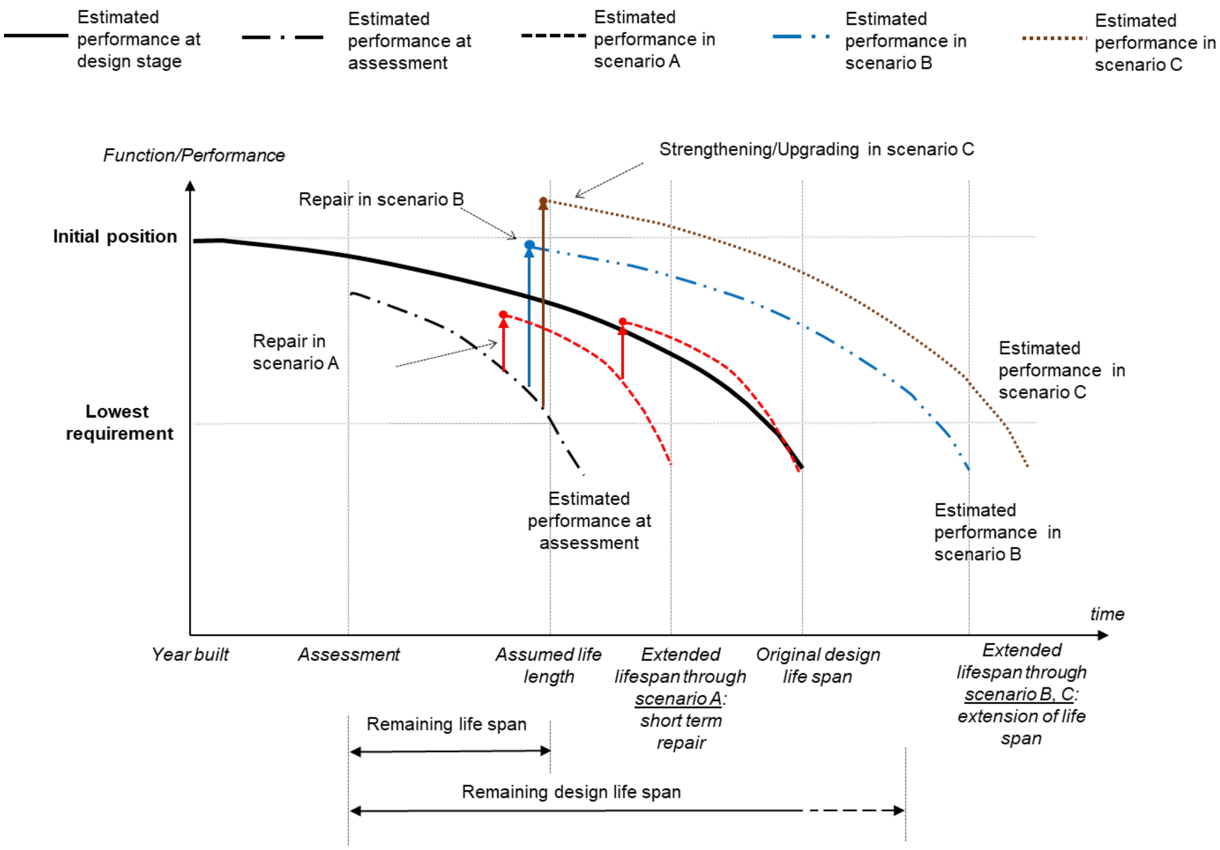



Figure 5.3 Performance of structure, based on (ISO 16311-1, 2014)

Part 2 in figure 5.2 cover assessment of existing structures, see figure 5.4 (ISO 16311-2, 2014). Assessment of a concrete structure identifies and defines areas of distress, and verifies structural performance based on the evaluated condition of the structure. The objectives of the assessment of an existing structure regarding operation and future performance shall be specified together with the client. The objectives should be based on e.g.; risk and safety aspects and chosen safety level; continued function, performance and capacity level and the client individual requirements on performance. A scenario is a change in the structural performance. This should be specified before the assessment begins based on lifespan (actual and expected). A scenario can be; corrosion, mechanical damage, chemical or physical actions. These scenarios are then verified or excluded during the assessment. The scenarios should be continuously controlled to meet; critical situations for the structure considering structural integrity and performance; amendments in the primary assessment that could not have been foreseen but arose during the actual assessment; the client individual requirements on performance. The goal with the preliminary assessment is to provide information about the condition of the structure and the causes and consequences of the degradation; find out the foundation of the overall condition; provide possible consequences with regard to future safety and performance; if necessary, provide immediate measures and suggestions for a detailed assessment. The goal with the detailed assessment is in principal the same as for the preliminary assessment, but should comprise a more comprehensive study of the background information about the condition of the structure, for example non-destructive testing, and clarify the causes and consequences of the degradation (process). The investigation should be more detailed and should also contain material testing and an extensive structural analysis and verification of load effect and load carrying capacity. The results of the assessment is the reported and from the findings, decisions regarding future measures should be taken.

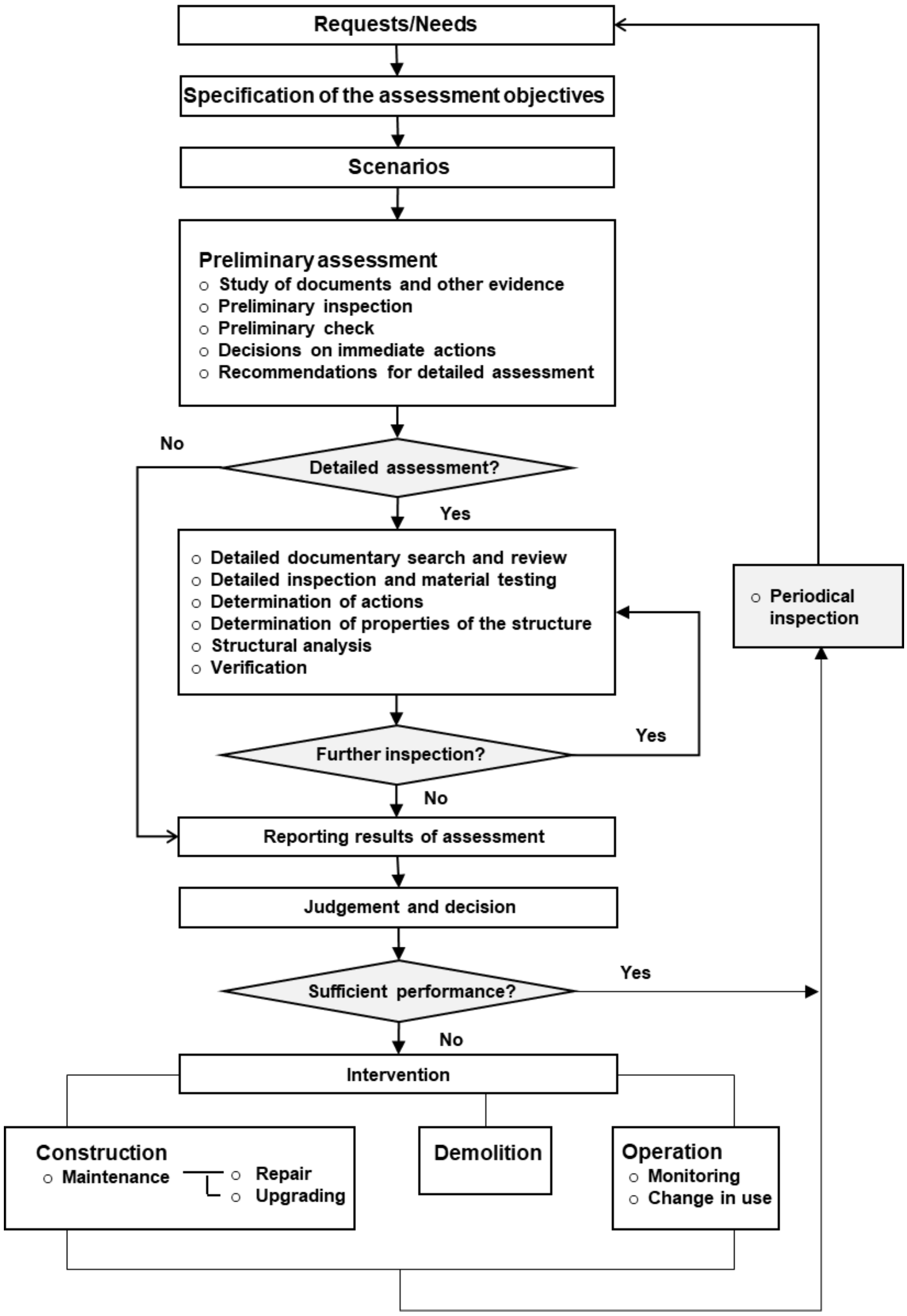



Figure 5.4 General flowchart for assessment of existing structures, based on (ISO 16311-2, 2014, ISO 13822)

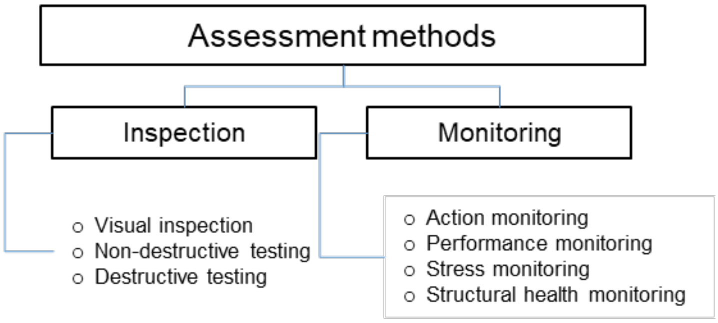



Figure 5.5 General classification of assessment methods, based on (Honfi et. al, 2018)

In general, the assessment methods can be classified into inspection and monitoring, see figure 5.5. Inspection cover visual inspection, non-destructive testing with appropriate systems and methods for the concrete structure studied, see chapter 6, and also destructive testing, which depends in the depth of the investigation. Action monitoring is described as the assessment of a structure´s response in time and space due to a known load and/or studying the load itself. Performance monitoring, on the other hand, allows an assessment of wheather a structural component meets the performance requirements under a known or any load (Honfi etl al, 2018). Stress monitoring is typically used directly in a structural assessment without any intermediate interpretation using a structural model. A SHM (Structural health monitoring) systems objective is to monitor the in-situ behaviour of a structure accurately and efficiently over time, ta assess its performance under various service loads, to detect damage or deterioration, and to determine the health or condition of the structure. The SHM system should be able to provide, on demand, reliable information pertaining to the safety and integrity of a structure. The information can then be incorporated into bridge maintenance and management strategies, and to improve design guidelines.

Judgement and decision should be based on all accessible information gathered from existing documentations, inspections, tests and structural analysis. The condition of the structure is assessed and classified according to the condition level and corresponding consequence level. The probability for the consequences should be determined and the conclusion from this evaluation will then give an estimate of the risk associated with the consequence from the damage/degradation, see also figure 5.6. In order to assess the performance of the structure and possible consequences fiver performance levels and five consequence levels are suggested (ISO 16311-2, 2014). The different levels of performance is explained in table 5.2.

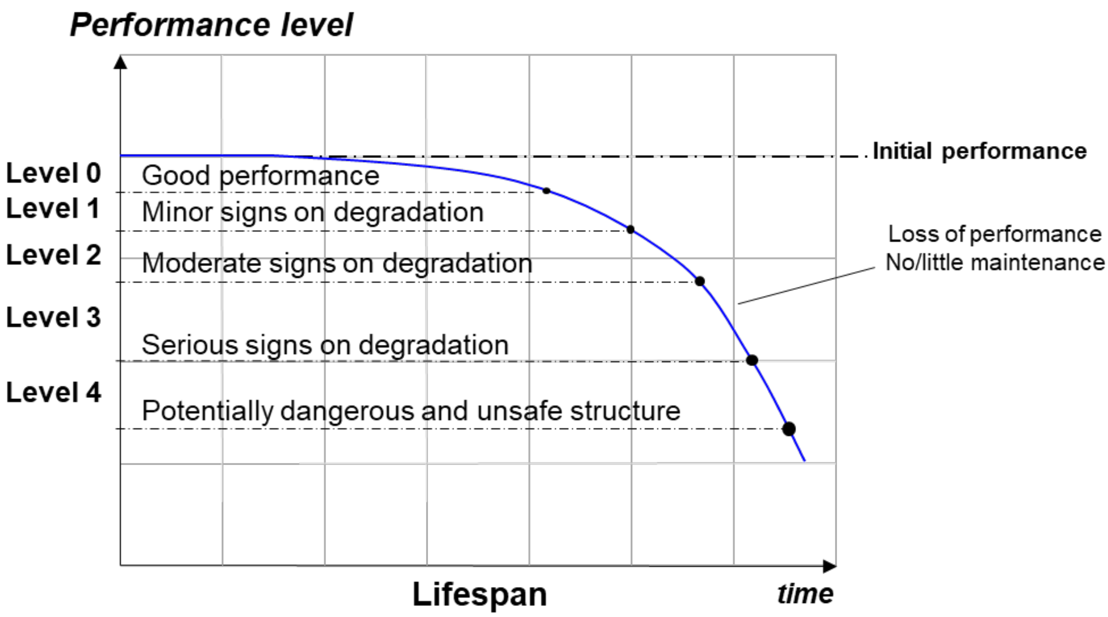



Figure 5.6 Principle illustration of the condition levels, depending on time, based on (ISO 16311-2, 2014)

Table 5.2 Performance levels related to figure 5.4, based on (ISO 16311-2, 2014)

| **Performance level** | **Explanation** |
|---|---|
| Level 0 - No signs of degradation | No visual signs on degradation. But, a larger part of the initiation phase, e.g. depending on carbonation or chloride intrusion, have already begun. |
| Level 1 - Minor signs of degradation | Minor visual signs on degradation. This level only gives the condition at the time for inspection and says nothing about the rate of degradation |
| Level 2 – Moderate sigh of degradation | Moderate visual signs on degradation. This level provides only the condition at the time for inspection and says nothing about the rate of degradation. |
| Level 3 – Serious signs of degradation | Visible signs on serious degradation. Falling parts can be dangerous, but the loss of service or safety for the structure is minimal |
| Level 4 – Potential dangerous and unsafe | Clear signs on degradation, the consequences significantly reduce the safety. Immediate action needs to be taken |

To classify the consequences of the observed condition for a structure, or a structural part, in a uniform way, the concept of consequence levels is introduced. It is defined as the expression of the seriousness of the consequences of an object related to a defined reference level. The following types of consequences might be evaluated:

- **Safety** (e.g. fire, traffic, load carrying capacity, person, falling parts)
- **Cost** (e.g. investments, labour costs during suspension, accessibility during shutdown, maintenance costs etc.)
- **Aesthetics** (e.g. colour, surface structure, cracks, discoloration etc.)
- **Health and environment** (e.g. noise, vibrations, pollution, dust etc.)

It is suggested that the following consequence levels are introduced, se table 5.3.

Table 5.3 Consequence levels, based on (ISO 16311-2, 2014)

| **Consequence level** | **Explanation** |
|---|---|
| Level 0 - No consequences | Performed evaluation shows that there are no consequences. |
| Level 1 – Small consequences | The evaluation shows that the consequences are small |
| Level 2 – Medium consequences | The evaluation of the situation shows that the consequences are moderate |
| Level 3 – Large consequences | The evaluation of the situation shows that large consequences are found |
| Level 4 – Hazardous consequences | The evaluation shows that the structure is unsafe and that the consequences are potentially dangerous. |

If the performance or consequences is not acceptable interventions has to be taken. This could for example be maintenance, repair or upgrading. In some situations monitoring to follow and control the degradation process or actual loads is suggested. In extreme situation the structure has to be demolished and replaced. The intervention depends on the cause for the defect or deterioration process. Common causes are presented in figure 5.6, (ISO 16311-3, 2014).

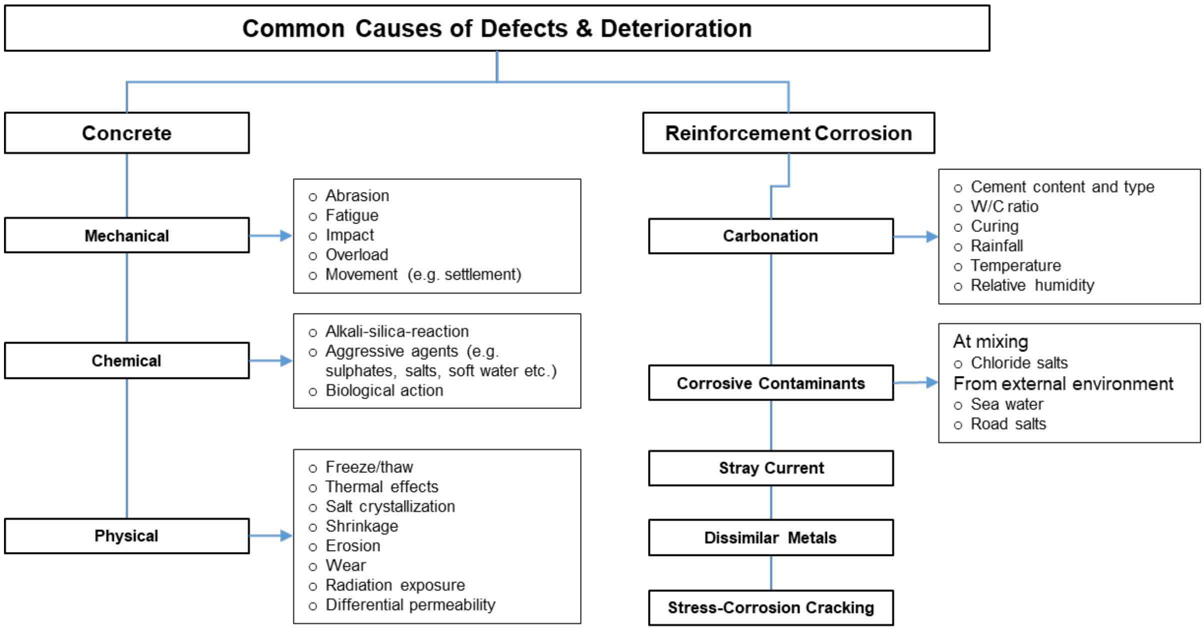



Figure 5.6 Common causes of defects and deterioration of concrete structures, based on (ISO 16311-3, 2014)

Determining the suitability of different remedies and methods for a particular condition can only be assessed after a thorough evaluation of the component or structure and reconciling maintenance, repair and upgrading design principles that include, but are not limited to:

a) Do not harm the structure of member
b) Adopt proven techniques and products with a documented record of success in similar projects
c) Harmonize prevention, maintenance, repair and upgrading strategies with budgets and planning

Finally execution should follow routines and codes and standards for different methods chosen, see (ISO-16311-4, 2014)

## 5.3 Assessment of concrete bridges in general

Additional work is needed regarding recommendations for load testing, proof load levels, test set up and calibration of numerical models. Above all, more tests to failure of different bridge types are suggested to give a better base for reliable assessment of existing bridges in order to improve quality control, a cost efficient bridge management and a sustainable usage of the existing bridge stock.

Assessment of the load-carrying capacity of existing bridges is an important task, though it can be complicated. Assessment can be used to check if a bridge is still fit or if the allowable loads must be reduced or if the bridge has to be strengthened. In the 1990ies of the Swedish railway authorities wanted to increase the allowable axle load from 25 ton to 30 ton on the 500 km long iron ore line in northern Sweden. The goal was to be able to carry heavier wagons with more iron ore and thus reducing transportation costs. At this point not enough knowledge exited to do this in a controlled manner. Preliminary assessments showed that the concrete fatigue capacity of many of the bridges would be jeopardized. However, the senior authors of this report were convinced that the codes were conservative in this respect, so they organized the testing of a decommissioned bridge. The results were very positive and the allowable axle load could be raised after minor improvements (Paulsson et al.1996, 1997). The experience initiated a European Research Project "Sustainable Bridges" 2003 –2007 (Olofsson et al. 2005, Sustainable Bridges 2007). And since then several bridges have been investigated and some tested to failure, see (Bagge, 2018)

The results from the tests may help to make more accurate assessments of similar existing bridges. Full scale tests may bring up necessary information on the real structural behaviour, detect weak points in the structure and provide knowledge on how to model the bridge in a correct way.

Load testing of interesting bridges before they are demolished have been done to some extent in the past, see e.g. Bolle (2010), Lantsoght et al. (2017), Bagge et al. (2018a) and Elfgren et al. (2018). It has also been successfully used in some European projects as Sustainable Bridges (2007) for a two-span concrete trough bridge, SB-D7.3 (2008), Puurula (2012), Puurula et al. (2014, 2015) and in MAINLINE (2014) for a steel truss bridge, ML-D1.3 (2015), Häggström (2016), Häggström et al. (2017)

Load tests are a relatively easy way to get precise information about the behaviour of a bridge and also to provide useful information about different bridge types and their typical behaviour. Tests need to be designed carefully to achieve useful results and the results need be analysed and published in order to get a full insight of its implications.

A survey of 30 concrete bridges tested to failure worldwide, (Bagge et al. 2017b), come to the conclusion that the final failure often was hard to predict; it was due to shear instead of flexure in ¼ of the cases; boundary conditions were not always understood correctly; and bridges usually had a higher capacity than what was predicted, (Plos et al. 1990, 1995, Täljsten, 1994).

The assessment procedure of a bridge can effectively be carried out in three levels according to Figure. 5.8. The procedure is based on (Schneider 1964, 2017, Sustainable Bridges 2007, UIC Code 778-4 2009, ISO 16311-2 2014 and Paulsson et al. 2016). Further refinements in Phase 3 have been proposed by (Plos et al. 2016) and (Bagge 2017) and examples are given in e.g. (Wang et al. 2016).

One example of a new set of standards for assessment of existing structures is the Swiss code SIA 269 (2011). Brühwiler (2014, 2015) gives a description of the positive experiences with it. He highlights major principles and approaches, in particular those related to a stepwise procedure, and by updating of action effects through monitoring, updating of structural resistance and novel technologies of intervention, such as strengthening. It is often beneficial to use site-specific live loads and dynamic amplification factors, see e.g. ML-D1.4 (2012) and Casas (2015). In the Swiss codes the recommended reliability index for a reference period of $T = 1$ year varies from $\beta = 3.1$ for a minor consequence of a failure to $\beta = 4.7$ for a serious consequence, JRC Assessment (2015). This is lower than the factor $\beta = 5.2$ given above for new bridges. Work has also started on a Eurocode for assessment of existing structures; see JRC Assessment (2015).

**Redundancy** and **Robustness** should also be considered in an assessment. Structures are usually designed with redundant parts, ensuring that if one part fails, the entire structure will not collapse. A structure without redundancy is called fracture-critical, meaning that a single broken component can cause the collapse of the entire structure. A bridge that failed due to lack of redundancy is the Morandi Bridge in Genoa in section 3.4. The robustness of a system is its capacity to function also with accidental or exceptional events. Recommendations are given in fib Model Code (2010), section 7.9 and in EC1, Part 1-7(2006)

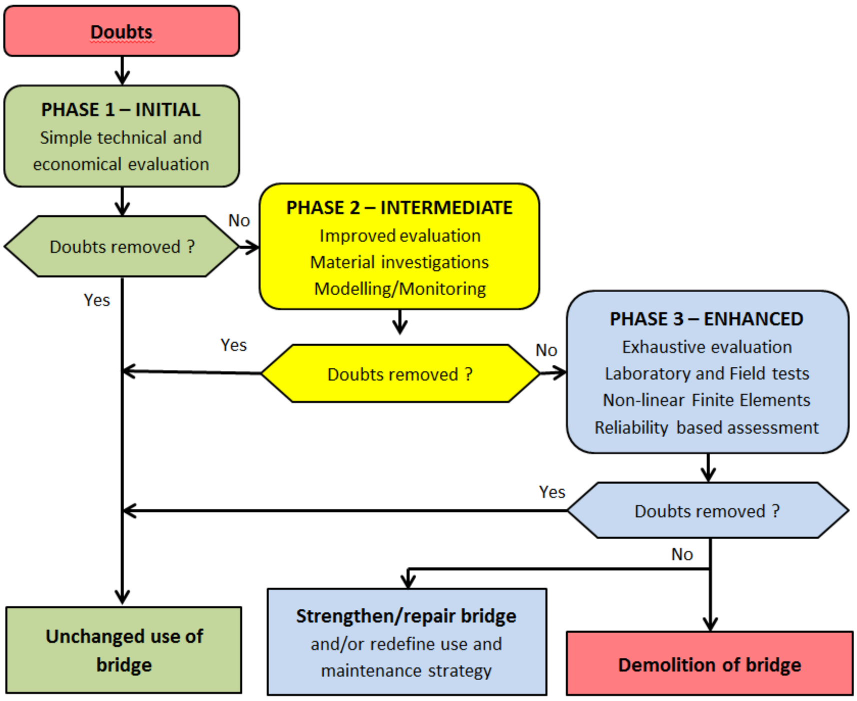



Figure 5.8 Flow chart for the assessment procedure of a bridge with three phases. Based on (Schneider 1994, 2017, Sustainable Bridges 2007, UIC Code 778-4, 2009, ISO 16311-2,2014 and Paulsson et al.2016).

## 5.4 Current Swedish practice

Trafikverket uses their own system, called BaTMan, to manage their stock of bridges. They collect and store various data about the bridges required to make decision about their maintenance at operational, tactical and strategic levels (Hallberg and Racutanu, 2007).

Trafikverket have a own bridge management system, BaTMan. The management data in BaTMan includes administrative data, photos and technical data of the object, load capacity data, and all inspection records. For some objects, construction drawings can also be found. Based on the bridge inspection the physical and functional condition of both the structural elements and the entire bridge is determined and a condition class (CC) is assigned by the bridge inspector. The CC spans from 0 to 3, see Table 5.4, and describes to what extent structural members fulfil the functional requirements at the time of inspection.

Table 5.4 Condition classes (CC), system used in Trafikverkets management system BaTMan

| **CC-class** | **Assessment** | **Follow-up** |
|---|---|---|
| **3** | Defective function | Immediate action is needed |
| **2** | Defective function expected within 3 years | Action is needed within 3 years |
| **1** | Defective function expected within 10 years | Action is needed within 10 years |
| **0** | Defective function expected beyond 10 years | No action is needed within 10 years |

The parts related to inspection and monitoring in the Swedish regulation for assessment of existing bridges (Trafikverket, 2016) are briefly summarized below (Honfi et. al., 2018).

- The regulation supports destructive sample testing for material properties of concrete, steel and aluminum. Methods are prescribed for evaluation of characteristic values. For some structures as, e.g. unreinforced concrete, material testing must be performed. For steel bridges build before 1970 the chemical composition must be determined to evaluate the material toughness.
- Testing of geotechnical properties is supported. It should be noted though, that assessments are often limited to the superstructure.
- Dimensions related to loads such as the thickness of the paving, can be measured rendering a reduction of the partial safety factor for that load.
- Fatigue assessment based on measured response is supported. Strain measurements are prescribed and a minimum duration of one week is recommended.

The verifications build on a deterministic (semi-probabilistic) format using partial safety factors. Reliability-based assessments are allowed but not supported by any guidance. Therefore, internationally recognised documents for the probabilistic assessment of existing structures might be applied, such as ISO 2394 General principles on reliability of structures (ISO, 2015) and ISO 13822 Bases for design of structures – Assessment of existing structures (ISO, 2010)

## 5.5 Assessment of prestressed concrete bridges

For prestressed concrete bridges, defects are not always visible and it is important to start investigation methodically from the bridges with the highest priority rankings for traffic but also for hidden defects. A thorough desk study and planning is crucial. It is impractical and uneconomic to expose all hidden components for inspection and many competing factors must be considered before undertaking any investigation. The questions that must be asked are:

- Is the component critical to the safety of the structure?
- What are the consequences of failure of the component?
- Can the component be exposed safely?
- Will exposing the component result in damage to the structure?
- Will exposing the component result in damage to it?
- Will exposing the component lead to long-term durability issues with the structure?
- It is economic to expose the component?
- What impact would the investigation have on the operation of the structure?

In addition, a bridge owner might have specific constraints that are not listed here and must be considered at assessment. Hidden defects investigation could either be undertaken as special investigation or worked into the normal inspection regime. The former should be considered where the risk of failure of a hidden component with significant consequences is likely to occur before the next inspection (normal). Where it is not possible to inspect all hidden components within the required cycle or normal inspections those of greatest risk should be prioritised. In figure 5.9 hazard scenarios for prestressing steel for a typical box girder bridge is shown. Possible defects in the figure is given by a number which is explained below (Matt, 2000)

**Failure of external barriers**

1 Defective wearing course (e.g.cracks)

2 Missing or defect waterproofing membrane

3 Defective drainage intakes and pipes

4 Wrongly placed outlets for the drainage of wearing course and waterproofing

5 Leaking expansion joints

6 Cracked and leaking construction or element joints

7 Inserts (e.g. for electricity)

8 Defective concrete cover

**Failure of tendon corrosion protection system**

9 Partly or fully open grouting in- and outlets

10 Leaking, damaged metallic ducts mechanically or by corrosion

11 Cracked and porous pocket concrete

12 Grout voids at tendon high, couplings and anchorage.

Possible no voids in low point as indicated in the sketch

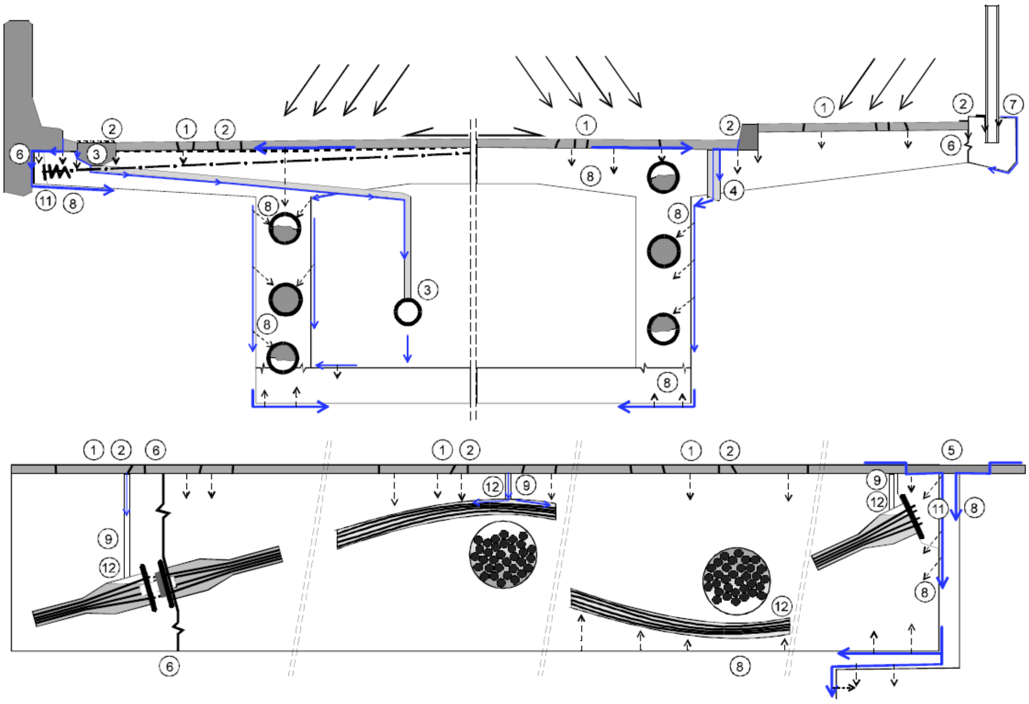


Figure 5.9 Hazard scenarios for prestressed steel in a typical box girder bridge. Indication of potentially weak points where water (possible with chlorides) can gain access to the tendons and cause corrosion (Matt, 2000).

However, for other bridge types there might also be other hazard scenarios. This has not been investigated in this report but are suggested in future studies. In general for each type of prestressing structure with its particular protection concept, water, possible chloride-contaminated, can potentially reach the prestressing steel. The key-question is: where does aggressive water get in contact with the structure and how does it flow off?

In addition, a thorough visual inspection of the concrete surfaces provides information on potential locations of damage of the unstressed steel and stressed reinforcement. The visual indicators might include:

- water flow
- discoloration (e.g. rust strains)
- spalling, delamination
- cracks
- honey-combing
- concrete deterioration by freeze-thaw
- joint leakage
- etc.

## 5.6 Life Cycle Cost Analysis (LCCA)

Life Cycle Cost Analysis (LCCA) for bridges is exemplified in the publications from the EC project MAINLINE (2014). A Life Cycle Assessment Tool (LCAT) was there developed for metallic bridges, ML-D5.7 (2014). For concrete bridges tools have been developed in ETSI (2012). It would be useful to develop a similar tool for prestressed concrete bridges. Some discussion is also given in Bagge et al. (2016) and regarding key performance indicators for bridges in COST TU 1406 (2016-2018).

The cost for society of the failure of a major prestressed concrete bridge is rather high. There is not only the cost of a new bridge which probably will be in the range of 50 - 500 MSEK (approximately 5 – 50 MEuros). To this comes the cost for leading traffic on a different route. This may be of the same order. The cost for the replacement of the Morandi bridge in Italy is estimated to be in the range of 150 – 200 MEuros.

# 6 Monitoring and inspection techniques for prestressed bridges

## 6.1 General

Tendons or ducts to be inspected in post-tensioned systems are typically embedded in concrete making it difficult to evaluate the true condition of the system by visual inspection alone. SHM (Structural Health Monitoring) and in combination with NDE (Non Destructive Methods) can serve to identify detrimental conditions at an earlier stage in order to mitigate reduced bridge safety and load capacity, costly rehabilitation actions and traffic disruptions.

The deterioration conditions that are of interest are strand defects that results in a loss of metallic area of the tendon which may be due to corrosion, section loss, or breakage of the strands, compromised grout such as unhydrated, over-hydrated or gassed grout, voids in the grout and water infiltrations into the tendons/ducts.

Five categories developed by (Webb et. al., 2015), are shown in table 6.1 and describe different ways in which monitoring data can be used to provide different types of information. The table also gives a description of what each type of deployment seeks to achieve. As was noted by (Webb et. al., 2015) many SHM (Structural Health Monitoring) deployments have aspects that can fit into more than one of the categories, namely: sensor deployment studies, anomaly detection model validation, threshold check and damage detection

Table 6.1 Categories of SHM systems, (Webb etl al., 2015)

| Category | System | Description |
|---|---|---|
| 1 | Sensor deployment studies | Demonstration of the ability of a sensor to measure a parameter of interest and of a communication system to transmit the data to the operators. |
| 2 | Anomaly detection | Detection of change in a parameter with time, for example many systems strive to detect natural frequency of vibration and notice changes that may represent a change in stiffness. However, changes can be due to many causes, such as condition, temperature, humidity and live loading. |
| 3 | Model validation | A system that aims to compare measured values with predictions from a structural model to quantify whether or not the structure is behaving as expected. The caveat is that structural responses are relative easy to detect, whereas input loads are difficult to quantify. |
| 4 | Threshold check | Comparison of measurements with a threshold level, derived at least in part, from a model of structural behavior, beyond which action should be taken. |
| 5 | Damage detection | Detailed investigation to detect the type, location, extent and rate of damage or deterioration at one or more locations. |

When installing a monitoring system or/ad implementing an SHM strategy the following 7 questions should be asked, Why?, Where?, Who?, What?, When?, How? and Which?, see also figure 6.1, (Vardanega et. al., 2018). These questions are aiming to give answers to safety, performance, cost, who is going to use the information and if the system is going to be used on an new or existing structure but also answers to what system is going to be used to get most out of the investigation.

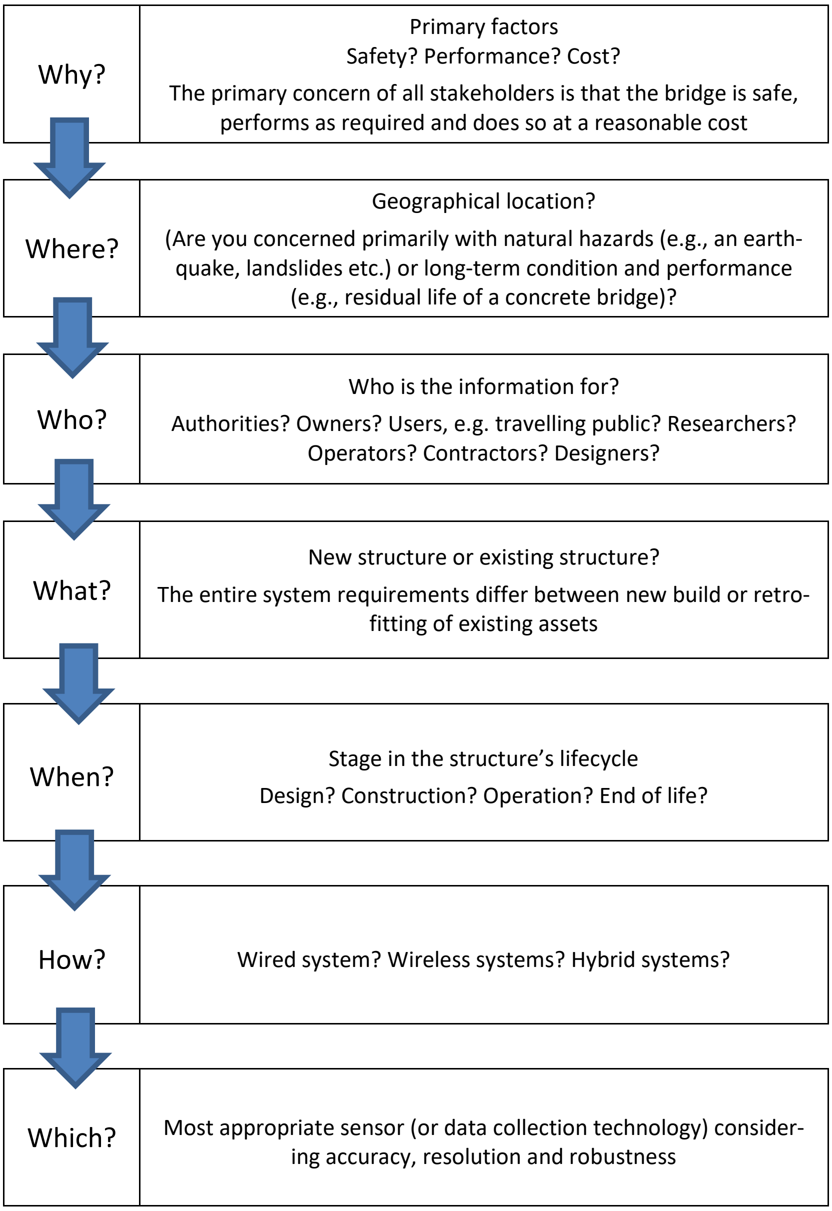



Figure 6.1 Main considerations when examining whether or not to employ a structural health monitoring system, from (Vardanega et. al., 2018)

## 6.2 Monitoring and inspection

When evaluating existing concrete structures, knowledge about inner structure is essential. Particularly, the diagnosis of concrete comprises several parameters and factors, estimations of thickness, location of reinforcing bars and metallic ducts, estimation of bar size, location of voids, effects of water, chloride content and delamination or cracking. And also corrosion and corrosion rates. In this section, the focus is placed on monitoring and inspection techniques for prestressed concrete bridges and methods to detect faults within the post-tensioning system itself. The authors are aware of that this is a simplified approach since there are many parts of a prestressed bridge that can deteriorate and that can be in need of inspection. The authors are also aware of other monitoring techniques that can be useful for assessing building structures, such as FOS (Fibre Optic Sensors), DIC (Digital Image Correlation) and also the use of advance drone technology. However, to our current knowledge none of these technologies can so far detect tendons cast into concrete only global and surface behaviour. Despite this, these technologies in combination with the ones discussed below can be a way forward to increase the knowledge about our existing structures.

Post-tensioned tendons, in prestressed concrete bridges, are structural elements essential for the safety, serviceability and durability of these structures. Consequently, it would be desirable to assess their behaviour in existing structures. Such checks to detect possible defects or damages, such as grout voids or tendon corrosion, or even residual prestressing force, should preferably be done by non-destructive testing or at least low-destructive techniques and with minimum of disturbance to the service.

During the last decades, there has been a continuous increase in the use of non-destructive testing (NDT) applied to many aspects related to the civil engineering field. This is principally due to the fact that most NDT methods work remotely, that is, without direct contact, and provide a primary image of the object under study. Some others, even those requiring direct contact with the structure, improve the models built remotely by adding information on non-visible areas. Particularly, NDT, has significantly benefitted the procedures for inspection and also, successfully solved some of the limitations of traditional methods such as lack of objectiveness, loss of safety during infrastructure inspection and also low rates of productivity. Due to the nature of reinforced and prestressed concrete, a number of components critical to structural performance are contained within the mass of the concrete, and so are hidden. Typical components such as reinforcement bars or internal post-tensioning cannot be readily inspected. In this instance, the structure's external condition and/or behaviour may indicate the need for intrusive investigation, although this is invariable expensive and can also be disruptive. Any problems with prestressing tendons do not necessarily display visual signs in the same way as reinforcement, such as rust stains, delamination or/and spalling. Internal inspections are difficult and intrusive, and potentially damaging. For these reasons appropriate types of NDT methods may be useful. Intrusive methods is also possible via endoscope, or techniques to measure the residual force. Although great care is needed to avoid damage to prestressing elements and to re-sealing. In general, there are five factors that needs to be investigated in prestressed concrete structures by the monitoring and inspection techniques; these are;

- Position of reinforcement, strands and cable ducts
- The thickness of the concrete structure
- Voids and honeycombing
- Voids in prestressing cables
- Corrosion in reinforcement and in post-tensioned tendons and anchorage

There is a number of monitoring techniques that can be used to assess/inspect post-tensioned tendons in concrete, these can be divided into:

- Manual methods
    - Visual inspection (VI)
- Acoustic methods
    - Impact Echo (IE)
    - Ultrasonic Pulse Echo (UPE)
    - Spectral Analysis of Surface Waves (SASW)
    - Acoustic Emission (AE)
- Radiographic methods
    - Radiography
- Electromagnetic methods
    - Ground Penetrating Radar (GPR)
    - Radar Tomography (RT)
    - Infrared Tomography (IT)
    - Cover Meter (CM)
- Magnetic Methods
    - Magnetic Flux Leakage (MFL)
    - MMFM-Permanent magnet
    - MMFM-Solenoid
- Electrochemical methods
    - Electrochemical Impedance Spectroscopy (EIS)

And also partly

- Intrusive methods
    - Endoscope (ES)
    - Residual Prestressing Force (RPF)
    - Potential Mapping (PM)

In appendix C a comprehensive summary of the different methods described above is presented. Table 6.2 summarises the different methods and their applicability to post-tensioned systems. Here also pro and cons are discussed.

Table 6.2 Summary of some NDT and Intrusive methods in relation to their use for prestressed concrete bridges (based on Sederholm, 2006)

| **Method** | **Localisation of voids in concrete** | **Detecting post-tensioning system** | **Detecting corrosion** | **Quantifying damages in tendons** |
|---|---|---|---|---|
| **Manual Methods** | | | | |
| Visual Inspection | No | No | No | No |
| **Acoustic Methods** | | | | |
| Impact Echo | Yes | Yes | No | Yes[1] |
| Ultrasonic Pulse Echo | Yes | Yes | No | Yes[1] |
| Spectral Analysis of Surface Waves | Yes | Yes | No | Yes[1] |
| Acoustic Emission | No | Yes | No | Yes[2] |
| **Radiographic Methods** | | | | |
| Radiography | Yes | Yes | Yes[1] | Yes[1] |
| **Electromagnetic Methods** | | | | |
| Ground Penetrating Radar | Yes | Yes | Yes[1] | Yes |
| Infrared Tomography | Yes | No | No | No |
| Cover Meter | No | Yes[4] | Yes[5] | No |
| **Magnetic Methods** | | | | |
| Magnetic Flux Leakage | No | No[3] | Yes | Yes |
| MMFM-Permanent magnet | No | No[3] | Yes | No |
| MMFM-Solenoid | No | No[3] | Yes | No |
| **Electrochemical Methods** | | | | |
| Electrochemical Impedance Spectroscopy | No | No[3] | Yes[3] | No |
| **Intrusive Methods** | | | | |
| Endoscope | No | No | Yes | Yes |
| Residual Prestressing Force | No | No | No | Yes |
| Potential Mapping | No | No | Yes | No |

[1]Large damages – large uncertainty, [2]Over time, [3]Not internal ducts. [4]Depending on depth, [5]With add on

**6.3**

## 6.4 Remaining Prestressing Force (RPF)

### 6.4.1 General – Shrinkage, Creep and Relaxation

When assessing prestressed concrete bridges, it is essential to take the current condition of the prestressing system into account. For instance, the quality of reinforcement protection (e.g. grout), steel corrosion and residual prestress force are all aspects that are crucial and require special attention (SB-LRA 2007). The residual prestress force influences the structural response both at the service-load and ultimate-load levels. By preventing cracks or limiting their formation, prestressing also reduces environmental exposure and, consequently, has a favourable impact on structures in harsh environments. However, there are often many uncertainties associated with the residual prestress force, especially after a longer time in service and, therefore, it can be useful to calibrate theoretically-based methods using experimental data from the assessed structure Bagge( 2017), (Habel et al. (2002)

The main causes for the reduction of the prestressing force with time is shrinkage and creep in the concrete and relaxation in the steel. These phenomena are presented at depth in e.g. BHB-M (1994, 2017) and fib Model Code (2010), The shrinkage of the concrete is caused by water drying out of the concrete during the hardening process. Outdoors at about 75% relative humidity the shrinkage strain $\varepsilon_{sh}$ is assumed to be below $\varepsilon_{sh}$ = 0,025 %. The creep is a long-time deformation caused by load. It is usually determined by a creep factor $\varphi$ giving the increase of the elastic strain $\varepsilon_{el} = \sigma/E_c$ calculated as the stress $\sigma$ divided by the modulus of elasticity of the concrete $E_c$. The creep factor outdoors at 75 % relative humidity is usually assumed to be of the order of $\varphi = 2$, i. e. the deformation of a specimen will double due to creep. The values of the shrinkage strain $\varepsilon_{sh}$ and the creep factor φ depend on the strength of the concrete, how old it is when the load is applied, temperature and humidity. Recommended values for design are given in e.g. EC2 (2005) and fib Model Code (2010).

The relaxation for steel is defined as $\chi = 1 - \sigma/\sigma_0$ , $\sigma$ is the stress after a certain time and $\sigma_0$ is the initial stress. If tests results are lacking, a value of 0,12 after 50 years and a value of 0,15 after 100 years are recommended in some design rules. More detailed values are given in EC2 (2005) and fib Model Code (2010). An example of a tested relaxation for a prestressing cable is given in Figure 6.2 BHM-I (2017).

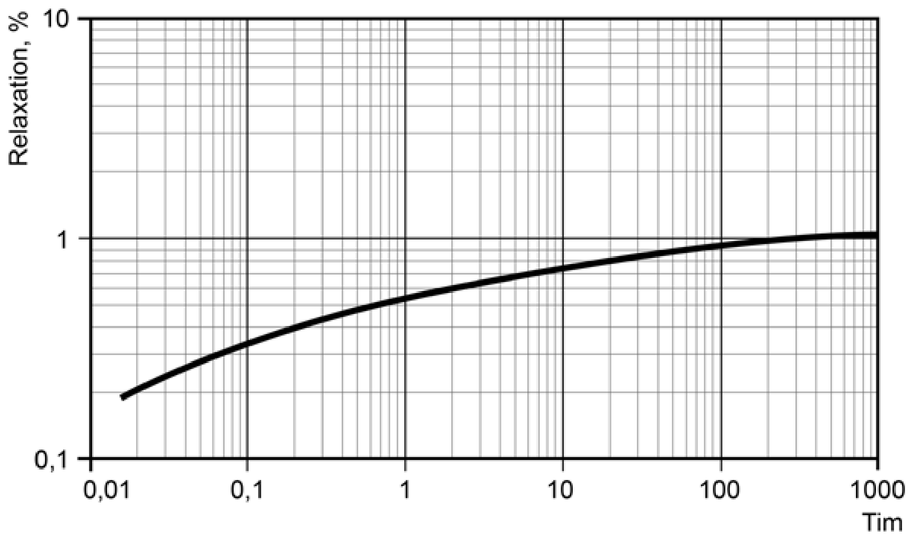



Figure 6.2. Relaxation of a $\phi$12,9 mm cable prestressed at 20ºC to 70 % of the ultimate strength (here 1860 MPa). BHM-I (2017).

### 6.4.2 Long-time deformations – Failure of Koror bridge 1996

In 1996 a 249 m long span failed of the 19-year-old Koror prestressed cantilever box-girder arch bridge in Palau in Micronesia, see Figure 6.4. The span was at the time the longest in the world of its type. The bridge had shown large mid-span deflections of about 1,5 m - three times of what was expected. The bridge was strengthened by making it continuous in the mid-point, by installing extra prestressing cables inside the box and by exchanging the concrete deck cover to black asphalt, Tang (2014). Three months later it collapsed.

Figure 6.4 The 249 m span of the prestressed Koror bridge (and the ferry it replaced) before failure in 1996 due to creep and relaxation, Tang (2014).

The failure began in the top slab near the Babeldaob main pier, where the slab might have buckled or been crushed under high compression, see Figure 6.5. When the top slab failed, the compression originally carried by the top slab concrete was transferred to the upper portion of the web and caused the webs to either buckle or crush in a longitudinal direction. Failure of the web plates caused the two parts of the webs to move against each other. This released the prestressing force in all internal and external longitudinal tendons at this location. This destroyed some deviators and increased shear force in the bottom slab.

After the webs failed, the entire shear force had to be carried by the bottom slab alone which was not possible. Thus, the bottom slab failed under shear. The girder here was then only supported by the bottom slab, thus changing from a monolithic connection to a simply supported condition with very little bending moment. The other end of the Babeldaob cantilever was supported by the mid-span monolithic connection. Thus, the Babeldaob cantilever became a simply supported beam. A large portion of the weight of this half of the bridge was now shifted to the Koror cantilever through the monolithic connection. at the mid-span. Because the bending moment of the girder near Babeldaob was fully released, the bending moment at the Koror side increased by about 100%. By adding the dynamic effect of this sudden failure, the actual bending moment in the girder at the Koror main pier became much higher and also the Koror side failed, Tang (2014).

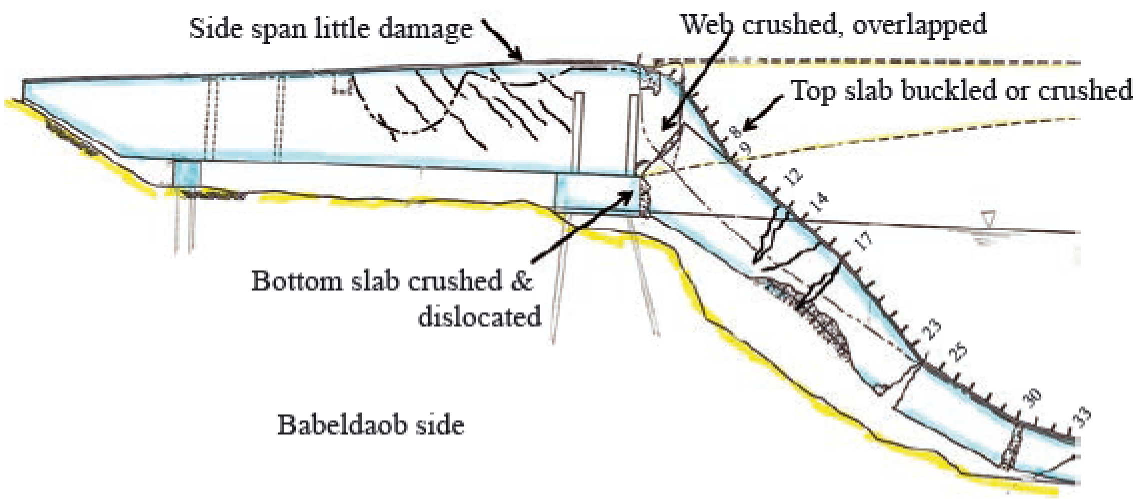



Figure 6.5. Failure of the Babeldaob side of the Koror bridge in Figure 6.4, Tang (2014). The failure was initiated by buckling or crushing of the top slab due to too high compression stresses partly caused by a high temperature.in the black asphalt.

The failure stirred up much interest and started discussions about long-time deformations, see e.g. Bazant et al. (2010, 2011), Tang (2014) and Wan-Wandereer et al. (2014). The models by the American Concrete Institute, ACI 209 (1982), Bazant's model B3, Bazant et al. (1995), and the fib Model Code (2010) were scrutinized and a new model for long-time deformations was proposed, Bazant et el. (2015).

Based on the investigations the following recommendations were made (Tang, 2014):

*1.Increase the amount of prestress:* Increasing prestressing can have two beneficial effects: firstly, it will reduce the elastic deflection, and consequently, the long-term creep deflection no matter what the creep coefficient may be; and secondly, it reduces the possibility of high-tensile stress caused by uncertainty in the estimation of prestress loss. However, note that excessive prestressing is also not advisable because this may cause congestion and a higher compressive stress and thus higher creep deformation, which, in turn, also increases prestress loss.

2. *Use higher-strength steel for the tendons:* In the Koror bridge Dywidag bars were used with 0,6 $f_u$ = 630 MPa as design stress after initial losses. Before repair the prestress was reduced by approximately 50%.  Because many new bridges are using seven wire strands for post-tensioning nowadays, this suggestion may not be relevant. Still, it may be useful to keep in mind as an option.

3. *Provide an additional camber to the bridge:* In addition to the theoretically calculated value, providing an additional upward camber is more visually appealing. A bridge that bends upward looks better than a bridge that droops downwards.

4. *Assure the applicability of the existing structural details:* When we perform a repair of an existing bridge, we are usually changing the stress in the structure. Therefore, we must carefully review the original details to understand whether they are suitable for the modified stress conditions. It would be most ideal to redesign the bridge and compare the new design to the old design to identify discrepancies or issues.

5. *Consider the history of the original bridge:* A repair design must carefully consider the history of the existing structure. Obviously, any bridge would only require repair if it has been distressed in some way.

### 6.4.3 Residual forces in nuclear reactor containments

The question of the remaining prestressing force is also important for nuclear reactor containments. Here the prestress is a requirement for upholding the function of the structure, that is to prevent dangerous nuclear radiation from leaking out of a containment. The question has been studied by e.g. Andersson (2007, 2016) and Lundqvist (2012). Examples of long-term measurements on cables are shown in Figure 6.6. After 20 to 25 years the remaining force is often 90 to 95 % of the original one. However, there are also forces as low as 80% of the original force.

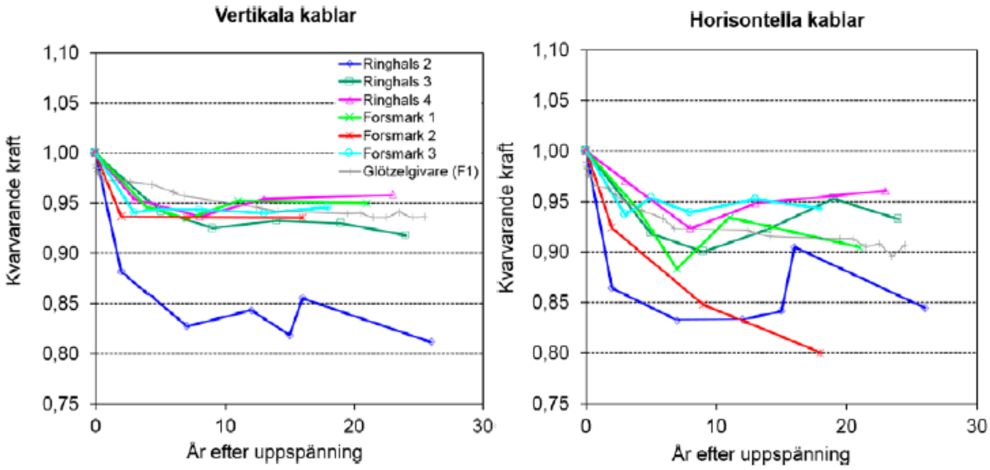



Figure 6.6. Remaining prestressing in relation to the original force for Swedish nuclear reactor containments. The left figure refers to vertical cables and the right to horizontal ones. The vertical axes give the relative remaining force and the vertical axes the time in years. From Andersson (2017).

### 6.4.4 Test methods

Several methods can be used to determine the remaining force:both non-destructive and destructive tests. Examples of non-destructive and destructive methods are given in figure 6.7, Bagge et al. (2017). In fact, the non-destructive methods described often cause minor, local damage and the impact on the structure can be negligible if repaired properly. The principles of these are briefly summarised below.

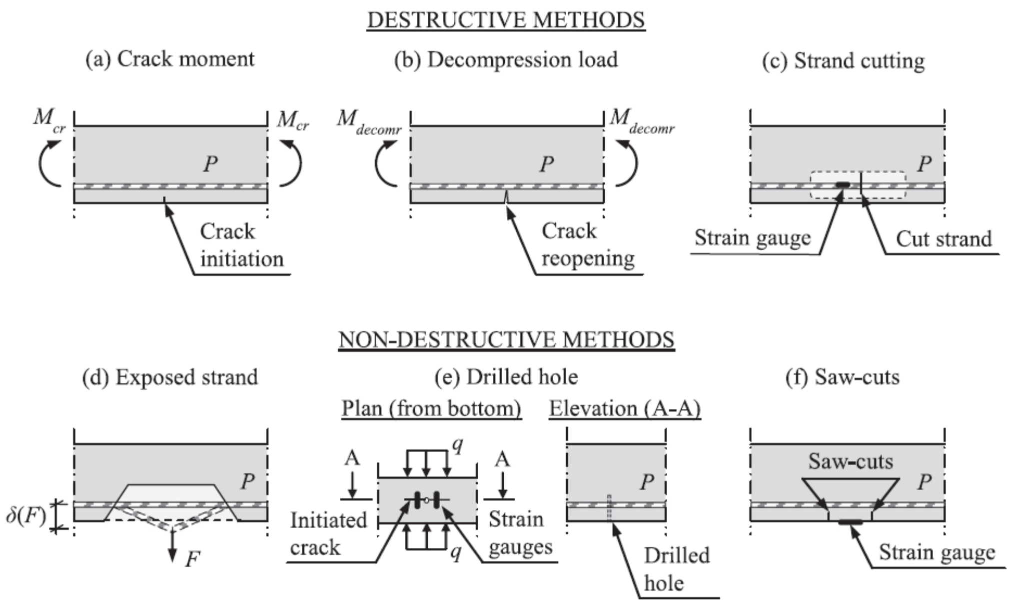



Figure 6.7. Methods to determine residual prestress force P in concrete members experimentally: (a) crack moment method, (b) decompression-load method, (c) strand cutting method; (d) exposed strand method, (e) drilled hole method and (f) saw-cut method, Bagge et al.(2017).

- **Crack moment method:** The prestress force is calculated based on the external load required to cause the first crack and the tensile properties of the concrete.
- **Decompression-load method:** The prestress force is calculated from the external load leading to reopening of existing cracks and, thus, eliminating the uncertainties associated with the tensile properties which are present in the crack moment method.
- **Strand-cutting method:** The prestress force is calculated from the development of strains measured when an exposed strand is cut.
- **Exposed strand method:** The prestress force is derived based on calibrated data and the response of an exposed and laterally loaded strand.
- **Drilled hole method:** The prestress force is calculated from the development of strains measured around a hole drilled adjacent to the prestressed reinforcement.
- **Saw-cut method:** The prestress force is calculated from the response of a concrete block when introducing saw-cuts adjacent to the prestress reinforcement to isolate the block from the acting forces.

When assessing existing bridges, the destructive methods are not always suitable and non-destructive methods are preferable. However, destructive methods can be valuable for examination of other methods when permanent damage to the structure is acceptable.

Saw-cut method

Due to its expected applicability to full-scale bridge members reinforced with post-tensioned tendons, the saw-cut method was further investigated.. The method was developed by Kukay (2008) and experimental studies examining such methods showed acceptably accurate estimates of the residual prestress force. However, the experiments were carried out on simply supported beams with pre-tensioned strands with constant eccentricity, both rectangular beams produced in the laboratory and I-shaped girders taken from a bridge under reconstruction (Kukay et al. 2010; Kukay 2008), and there have been no additional studies on more complex structural members.

The principle of the method is to measure the development of longitudinal strain at the surface (top or bottom) of a member when a block of concrete is isolated from the loads acting on it. The isolation is carried out gradually by introducing transverse saw-cuts on each side of the position of measured strains and the concrete block is regarded as isolated when increasing the depth of saw-cuts does not cause further changes in the strains at the measured surface. Based on the current action effects (i.e. associated with prestress force, restraint force, dead load and external applied load), Navier’s formula can be used to quantify the residual prestress force at the position of the isolated concrete block. The axial stresses in the section is given by Equation (6:1):

$$\sigma = \frac{P}{A} + \frac{Pe_P y}{I} + \frac{M_R y}{I} + \frac{M_G y}{I} + \frac{M_Q y}{I} \tag{6:1)}$$

where $\sigma$ is the longitudinal concrete stress at the surface, $P$ is the prestress force, $A$ is the cross-sectional area, $e_P$ is the eccentricity of the prestress force, $y$ is the distance to the neutral axis from the monitored surface, $I$ is the cross-sectional second moment of inertia, $M_R$ is the secondary moments due to restraint forces, $M_G$ is the moment due to permanent loads and $M_Q$ is the moment due to variable loads. In the case of a statically indeterminate structure and the presence of a secondary moment, the procedure for determining the prestress force is iterative, in contrast to the original formulation of the method assuming a statically determinate structure (i.e. $M_R$ excluded).

The original version of the saw-cut method requires full isolation of the concrete block, which is not always possible when assessing existing structures. For instance, non-prestressed reinforcement can be located too close to the concrete surface, thus limiting the possible depth of the saw-cuts to avoid permanent damage. In such situations the experimental test can be simulated using FE analysis. Thus, the residual prestress force can be quantified based on the response observed in the test, rather than the strain measured at full isolation. The development of the strains, as a function of the saw-cut depth, is followed and compared between the experiment and the simulation, while the modelled prestress force is iteratively updated until there is a consistent response. The saw-cutting can be simulated in a FE model by gradually removing FE elements corresponding to the saw-cuts in the experiments, see figure 6.8. Therefore, using this method, it is possible to avoid damage to the structure which might be difficult to repair.

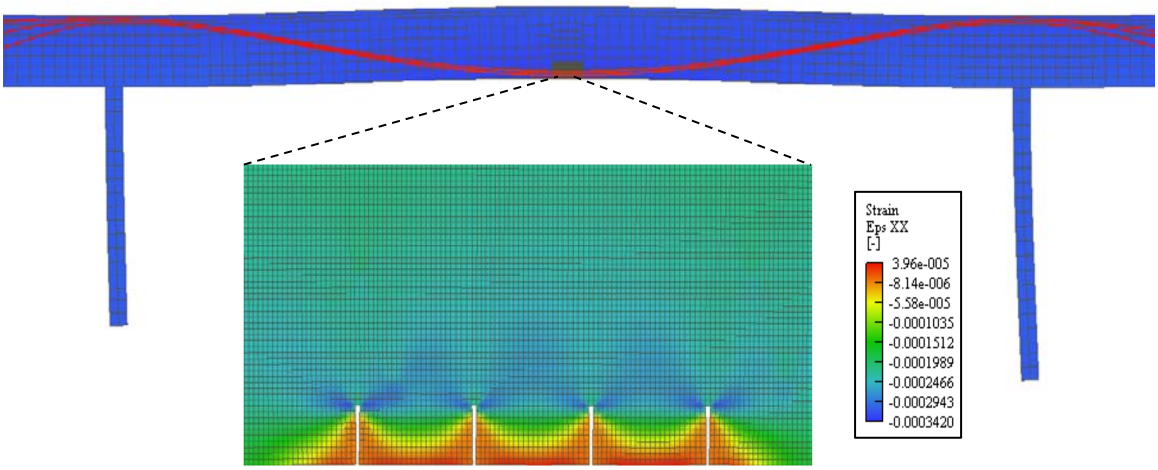



Figure 6.8 A part of an FE model for simulation of the strain distribution as saw-cuts are introduced transversally at the base of a concrete beam (Bagge et al. 2017).

<u>Decompression-load method</u>
As with the saw-cut method, Navier's formula as shown in Equation (6:1) can be used to quantify the residual prestress force. By measuring either the opening of an existing crack or the concrete strains beside it, the load at which the crack reopens (i.e. no normal stress in the crack) can be determined. Initially, for loading when the crack remains closed, a linear and rather stiff load-displacement (or load-strain) response is present. As the crack reopens, the behaviour drastically changes and the stiffness reduces. This change in the response is used to identify the decompression-load for calculating the corresponding prestress force.

When using the decompression-load method, the depth of the compression zone of the cracked concrete is one parameter that has a significant impact on the outcome. At the same time, the compression depth of the actual section can be uncertain due to nonlinear strain distributions and the lack of reliable measurements, for example see (Nilimaa et. al., 2015)

### 6.4.5 Remaining prestressing forces in bridges

Results from the test on the Kiruna Bridge show that the remaining force varies between some 60 to 90 %, Bagge (2017). Comparisons between results from and FE calculations  are given in Figure 6.9.

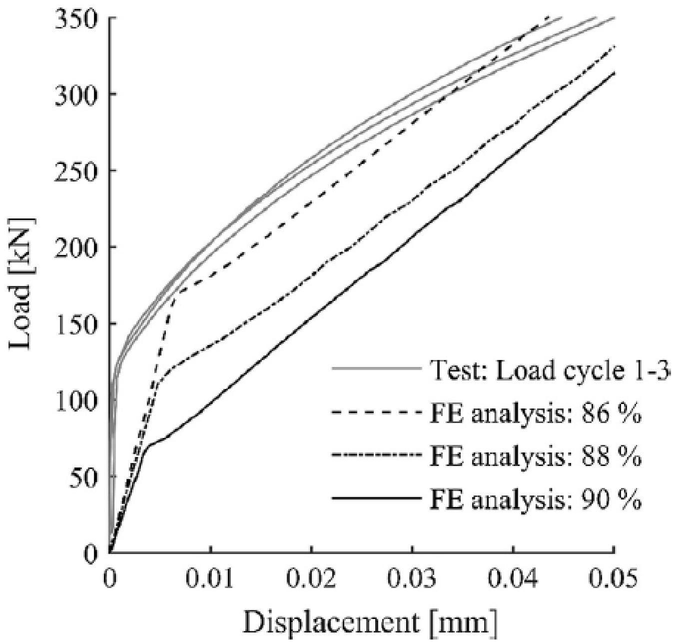



Figure 6.9. Load-displacement behaviour at crack opening according to test at the Kiruna bridge and FE analyses at indicated level of prestress losses, Bagge et al. (2017).

The influence of freezing and thawing on the prestressing force has been studied in laboratory tests by Chao et al (2015) and Qin et al (2017. When using mathematical models to predict the prestress losses due to freeze-thaw-cycles, it was found that they were relatively small when the concrete was slightly damaged. However, they increased rapidly when the freeze-thaw-cycles were repeated. Losses of about 55 were obtained.

# 7 Repair and strengthening techniques

## 7.1 General

Concrete is a building material with a high compressive strength and a poor tensile strength. A structure without any form of reinforcement will crack and fail when subjected to a relatively small tensile load. The failure occurs in most cases suddenly and in a brittle manner. To increase a structure load carrying capacity and ductility it needs to be reinforced. This is mostly done by reinforcing with steel bars that are placed in the structure before the concrete is cast. Since a concrete structure usually has a very long life the demands on the structure will normally change over time. The structures may have to carry larger future loads or meet new standards. In extreme cases, a structure may need to be repaired due to accidents. Another more reason includes errors made during the design or construction phase so that the structure needs to be strengthened before it can be used.

Over the past decade, the issue of deteriorating infrastructure has become a topic of critical importance in Europe, and to an equal extent in the United States and Japan. For example, the deterioration of decks, superstructure elements and columns of bridges can be traced to reasons ranging from ageing and environmentally induced degradation to poor initial construction and lack of maintenance. As an overall result, a significant portion of our infrastructure is currently either structurally or functionally deficient. Beyond the costs and visible consequences associated with continuous retrofit and repair of such structural components are the real consequences related to losses in production and overall economies related to time and resources caused by delays and detours. As we move into the twenty-first century, the renewal of our lifelines becomes a critical issue. As discussed in Chapter 6 to keep a structure at the same performance level it needs to be maintained at predestined time intervals. If lack of maintenance has lowered the performance level of the structure, need for repair up to the original performance level may be required. In cases when higher performance levels are needed, upgrading can be necessary. Performance level means load carrying capacity, durability, function or aesthetic appearance. Upgrading refers to strengthening, improved durability, and change of function all related to safety. In this chapter we will discuss strengthening of structures in general but with focus on methods for prestressed concrete bridges.

## 7.2 Repair and strengthening of structures

### 7.2.1 General

When a concrete structure or member exhibits inadequate strength, behaviour or stability, it may be feasible to modify the structure using various stabilisation and strengthening techniques. The different between stabilisation and strengthening are somewhat clouded and, in some cases, are used synonymously. Stabilisation can be defined as the process of halting a particular unwanted situation from progressing. For example, settlement of structure can be stabilized by grouting to halt further movement.

Strengthening is normally referred to as the process of adding capacity to a member or structure, for example adding extra reinforcement of FRP laminates to the soffit of a beam structure. In some cases, the process involves a combination of halting an unwanted situations and, at the same time, adding capacity. A structure may have to be strengthening for many reasons for example to enhance its capacity in:

- tension
- shear
- flexure/bending
- compression

Or its stability. Sometimes also strengthening in torsion is needed, this is covered here by the combination of flexure and shear.

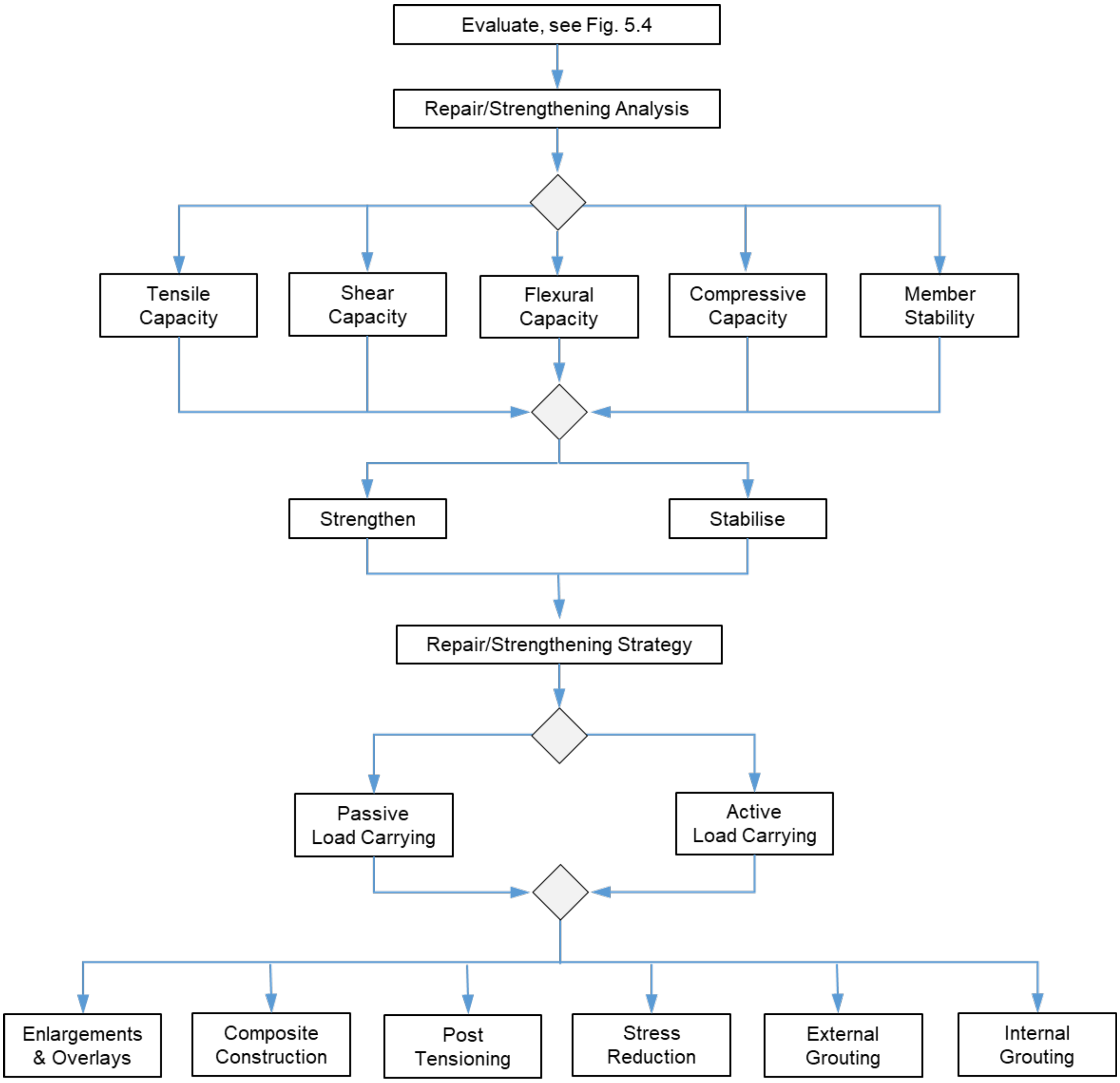



Figure 7.1 General repair and strengthen strategy for concrete structures, based on (Emmons, 1993)

### 7.2.2 Enlargement

Enlargement is the placement of additional concrete and most often reinforcing steel on an existing structural member. Beams, slabs, columns and walls. If necessary can be enlarged to add stiffness or load carrying capacity. In most cases the enlargement is attached mechanically or by bonding to the existing concrete to create a monolithic member.

### 7.2.3 Composite construction

Composite construction is a method wherein materials other that concrete are placed in concert with an existing concrete member to add stiffness or load carrying capacity. Steel of FRP-materials are the most used materials used in this technique. These materials can be fabricated to meet almost any configuration requirement. Load transfer in the composite member is accomplished by the used of adhesives, grouts and mechanical anchorage systems.

### 7.2.4 Post-Tensioning

Post-tensioning is a technique used to prestressed reinforced concrete. The tensioning provides the member with an immediate and active load-carrying capability. Placement of the tension components can be achieved either internally within the member or externally to the member. Tension components are generally steel plates, FRP laminates, rods, tendons or strands. Tension is imparted to the components by jacking or stressing the component. Post-tensioning enhances a member ability to relieve overstressed conditions in tension, shear, bending and torsion. The post-tensioning technique can also be used to eliminate unwanted displacements in members and to turn discontinuous members into continuous members.

### 7.2.5 Stress Reduction

Stress reduction is a technique that reduces stress in a member or structure. Some of the more common methods of stress reduction include cutting new expansion joints, jacking displaced structures, and installing isolation bearings. Other more radical techniques involve the removal of parts of the structure.

### 7.2.6 Internal Grouting

Internal grouting is the placement of a flowable material into an unwanted discontinuity, such as a crack within the concrete member, the flowable material upon reaching the discontinuity will solidify and assume necessary structural properties. Internal grouting is used to repair fractured honeycombed or voided concrete placements. The most common materials used for internal courting are polymers and cement based materials.

### 7.2.7 External Grouting

External grouting is the placement of a pumpable material outside the structure, generally within the surrounding foundation or soils or at the interface between the structure and the soil. The grouting materials can be used either to provide necessary load transfer between the structure and soil, or to displace unwanted settlement. Most materials used for external grouting include cement-based mixtures.

In table 7.1 an overview of the stabilisation and strengthening techniques is given and how well they might apply to prestressed concrete bridges in general.

Table 7.1 Stabilisation and strengthening techniques, see also figure 7.1

| **Technique** | **Description** | **Applicable to prestressed concrete** |
|---|---|---|
| Enlargements | Stiffness or load carrying capacity. Passive strengthening. | In some sections, in combination with external post-tensioning |
| Composite construction | Stiffness or load carrying capacity. Passive or active strengthening. Could be difficult to obtain active strengthening. | Yes, could be used separate or in combination with other methods |
| Post-Tensioning | Active system to increase the load carrying capacity. | Yes, could be used both for flexure and shear strengthening. |
| Stress Reduction | To reveal stress. Also systems for span shortening. | Maybe in special situations and small bridges. |
| Internal Grouting | Injection with polymer or cement based material. | To increase stiffness, improve integrity. Not direct for strengthening. |
| External Grouting | Grouting for larger cavities or for stabilisation of foundation. | Not for the main structure. |

## 7.3 Passive and Active Design

Strengthening and stabilisation techniques are generally considered to be either passive or active, depending how loads act on the additional components used to strengthen or stabilise the structure. Techniques in which strengthening do not participate in stress sharing until additional loads (live and/or dead) are applied or/and until additional deformations occurs are called passive. There are many situations in which additional deformations is not acceptable: the strengthening must immediately participate in the stress sharing. These strengthening's are called active. Active systems require either prestressing the repaired elements or temporarily removing the loads (both live and dead) from the existing elements or a combination of the two. Active systems can be compared to elastic suspenders used to hold up pants. If the suspenders are placed in tension immediately (active), the pants will stay in their vertical position. On the other hand (passive) if the suspenders are placed loose or without tension, the pants will fall vertically until sufficient tension occurs in the suspenders to resist the weight of the pants.

Passive systems work well when live load changes are anticipated. For example, upgrading a bridge to sustain heavier loads may require only a passive system. However, if a member is overstressed, the only choice may be to use an active repair technique that will immediately reduce the stress by sharing the loads, thus, eliminating the overstressed condition.

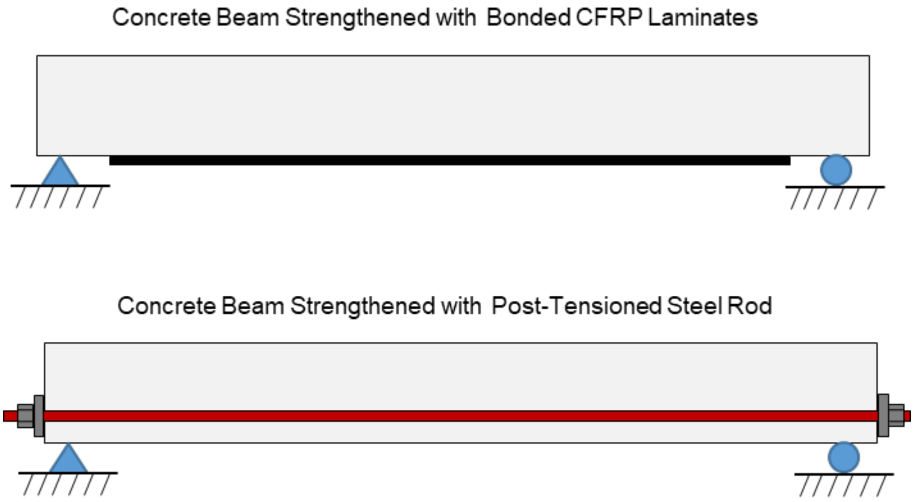



Figure 7.2 Passive and active strengthening

## 7.4 Repair and Strengthening of prestressed concrete bridges

### 7.4.1 General

Prestressed concrete bridge members can be subjected to accidental damage because of vehicle impact, mishandling, or fire. Methods currently used or potentially available for repair of such members need to be identified and evaluated for various levels of damage. In addition deterioration of concrete and steel over time can affect the overall performance severely. Furthermore, need of higher axle loads and increase traffic volumes might also imply strengthening of prestressed members or the whole structure.

Methods for repair and strengthening of prestressed concrete bridges is depends on the underlying causes; the type of deterioration process, impact, strengthening need, what component of the structure that needs to be rehabilitated and type of bridge structure etc.. In many situation a combination of measures are needed. As mentioned in Chapter 6 it is important that the cause to repair and strengthening are clear, i.e. level of damage and type of damage, for example shear problems need most likely other strengthening methods than flexural or anchorage problems. Normally repair is done to restore the original performance level and is related to the service limit state (SLS). However, strengthening/upgrading refers to improving the performance level and is most often related to load carrying capacity in the ultimate limit state (ULS). Repair and strengthening can also be carried out for fatigue and for deflections and crack widths. However, this is not discussed in this section. It is important to make the repair/strengthening measure based on a classification scheme.

A classification that might be used is the one presented in Chapter 5 and in table 5.2, here shown gain as table 7.1, where performance level is related to grade of degradation. Level 0-3 can be referred to the SLS and level 4 to the ULS.

Table 7.1 Performance levels related to figure 5.4, based on (ISO 16311-2, 2014)

| **Performance level** | **Explanation** |
|---|---|
| Level 0 - No signs of degradation | No visual signs on degradation. But, a larger part of the initiation phase, e.g. depending on carbonation or chloride intrusion, have already begun. |
| Level 1 - Minor signs of degradation | Minor visual signs on degradation. This level only gives the condition at the time for inspection and says nothing about the rate of degradation |
| Level 2 – Moderate sigh of degradation | Moderate visual signs on degradation. This level provides only the condition at the time for inspection and says nothing about the rate of degradation. |
| Level 3 – Serious signs of degradation | Visible signs on serious degradation. Falling parts can be dangerous, but the loss of service or safety for the structure is minimal |
| Level 4 – Potential dangerous and unsafe | Clear signs on degradation, the consequences significantly reduce the safety. Immediate action needs to be taken |

### 7.4.2 Repair

Minor and moderate damages can be repaired via patching and with the focus to prolong the service life of the structure. However, to place a decision-making process on a rational basis and to facilitate appropriate engineering solutions for the repair of prestressed concrete bridges is it also necessary to assemble and evaluate information on the effect of repair methods on the service life, safety performance, and maintenance of the structure. Decisions on method of repair must also consider cost, user convenience, and esthetics. The materials, quality controls and execution should also fulfil codes and standards, e.g., EN 1504. In addition the durability of structure components should be, as nearly as possible, equal to the durability of the original construction. This has to be verified by reference projects, research and/or data from material suppliers. However, in many situations the durability of a repair is strongly related to the skills of the workmanship. It is therefore important that the contractor can show experience from similar repair works. However, in this report traditional repair methods will not be any further discussed even though strengthening normally precedes strengthening measures.

In cases where it is needed to restore the original performance of a prestressed structure due to for example tendon breakage or corrosion in the post-tensioning systems this is normally considered as a repair measure, but is here discussed under the strengthening section.

### 7.4.3 Strengthening

Strengthening of structures increase the performance level. Despite this, strengthening methods discussed in this section might be used for repair as well as for strengthening measures. Strengthening methods could also be divvied in passive and active strengthening systems as earlier discussed.

From the literature, there is a number of studies where different strengthening techniques has been applied to prestressed concrete members. A very comprehensive and detailed work has been presented by (Schnellenbach et. al., 2016) where different strengthening methodologies for old prestressed bridges are discussed. Other comprehensive reports are (NCHRP, 1980,1985), (Harris et. al., 2009, 2012) and for external prestressing, (Nordin, 2005). The main strengthening methods that have been found for prestressed concrete bridges are discussed below.

External post-tensioning

External post-tensioning is affected using steel rods, strands or bars anchored by corbels or brackets (typically referred to as ‘bolsters’) which are cast or mounted onto the girder; typically on the girder’s side (although occasionally on the soffit). The steel rods, strands or bars are then tensioned by jacking against the bolster or preload. Design of external post-tensioned repair systems is relatively straight forward using a simple plane sections analysis (recognizing that the post-tensioning bar is unbonded). The attachment/interface of the bolsters, however, requires significant attention. These elements are ‘disturbed regions’ subject to large concentrated compression forces. Additionally, sufficient shear capacity along the interface between the bolster and existing beam must be provided to transfer the post-tensioning force. Effective shear transfer often requires the bolsters themselves to be post-tensioned (transversely) to the girder to affect adequate ‘friction’ forces along the interface. Finally, the design of the bolsters and interface must consider the moments induced by the eccentric post-tensioning forces.

An ideal tendon material should not only have high-strength but it also has to remain in elastic range until relatively high stresses are reached. Furthermore, it has to show sufficient ductility and good bonding properties, low relaxation and high resistance to fatigue and corrosion.

External prestressing, both for new and existing structures, has proven to be an effective technique. Picard et al. (1995) listed the following advantages:

- Concreting of new structures is improved as there are no or few tendons and bars in the section
- Dimensions of the concrete section can be reduced due to less space needed for internal reinforcement.
- Profiles of external tendons are simpler and easier to check during and after installation.
- Grouting is improved because of a better visual control of the operation.
- External tendons can be removed and replaced if the corrosion protection of the external tendons allows for the release of the prestressing force.
- Friction losses are significantly reduced because external tendons are linked to the structure only at the deviation and anchorage zones.
- The main construction operations, concreting and prestressing, are more independent of one another; therefore the influence of workmanship on the overall quality of the structure is reduced.

It is also important to understand the weaknesses of the technique. The following disadvantages should be kept in mind, (Picard et al. 1995):

- External tendons are more easily accessible than internal ones and, consequently, are more vulnerable to sabotage and fire.
- External tendons are subjected to vibrations and, therefore, their free length should be limited.
- Deviation and anchorage zones are cumbrous additions to the cross section. These elements must be designed to support large longitudinal and transverse forces.
- In the deviation zones, high transverse pressure acts on the prestressing steel. The saddles inside the deviation zones should be precisely installed to reduce friction as much as possible and to avoid damage to the prestressing steel.
- In the case of internal grouted tendons, the long-term failure of anchor heads has limited consequences because prestressing may be transferred to the structure by bond. In the case of external tendons, the behavior of anchor heads is much more critical. Therefore, anchor heads should be carefully protected against corrosion.
- At ultimate limit states, the contribution of external tendons to flexural strength is reduced compared to internal grouted tendons. The stress variation between the cracking load and ultimate load cannot be evaluated at the critical section only, as is done for internal bonded tendons.
- At ultimate limit states, failure with little warning due to insufficient ductility is a major concern for externally prestressed structures.
- The actual eccentricities of external tendons are generally smaller compared to internal tendons.

As the tendons are placed outside the structure the connection to the structure are at deviators and anchorage. Between those points the tendons are free to move relative to the section of the structure. If deviators are not used the second-order effects due to changing tendon eccentricity, see figure 7.3, lead to a lower load carrying capacity, (Tan and Ng 1997). The use of deviators along the span of the structure can effectively reduce those effects, tests conducted by Tan and Ng (1997) showed that a single deviator at the section of maximum deflection resulted in satisfactory service and ultimate load behavior.

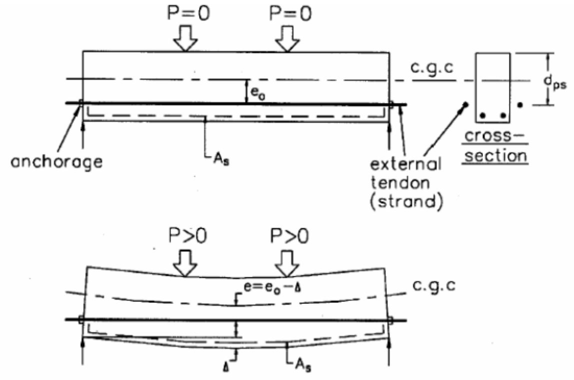



Figure 7.3 Second order effects without deviators, (Tan and Ng, 1997)

External post-tensioning can be used for new structures as well as for existing structures when strengthening is needed. The application is not restricted to concrete structures. Any material with reasonable compression capability can be strengthened with external tendons. The technique has been used for various types of structures such as:

- Bridge superstructures
- Girders in buildings
- Roof structures
- Circular structures like silos, reservoirs and chimneys

In figure 7.4 and 7.5, examples of external post-tensioning of a concrete bridge are shown.

Figure 7.4 External post-tensioning applied to concrete box-girder bridges Photos from a French bridge on high speed line to Marseilles left and a Hungarian bridge on the new freight line through Slovenia to the Mediterranean (Photos by B. Paulsson)

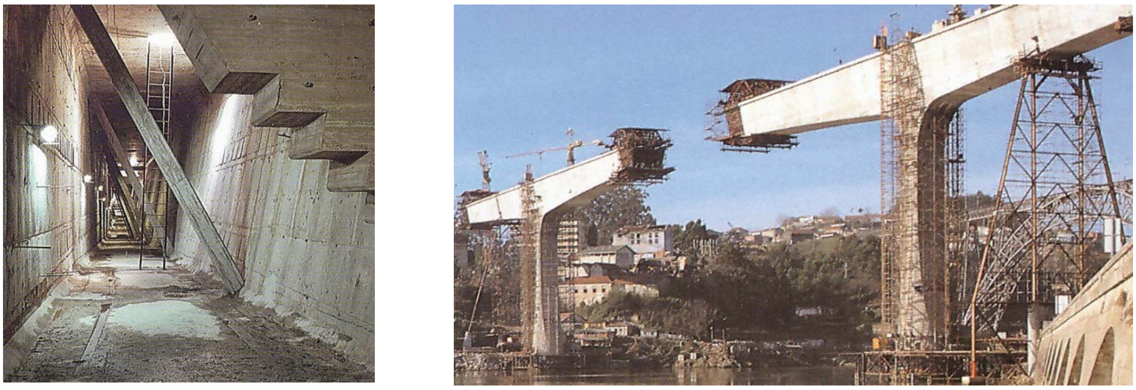


Figure 7.5 Photos from the new railway bridge in Porto shows preparation for extra external post-tensioning, (Nova ponte ferroviaria sobre o Rio Douro no Porto)

For strengthening purposes external post-tensioning for beam bridges is today well known after many years of experience, see also case studies in appendix C.

Strand splicing

Strand splices are designed to reconnect severed strands (Harris et. al., 2009). Methods of reintroducing prestress force into the spliced strand are preloading, strand heating and torqueing the splice; the latter is most common, essentially making the splice a turnbuckle of sorts. Strand heating is a method whereby the strand is heated, the strand splice is secured to the strand and as the strand is allowed to cool, it shrinks, thus introducing tension back into the strand. Strand heating of conventional high-strength prestressing strand is not believed to be a terribly rational method of affecting any reasonable prestrain: either a) a long length of strand must be heated; or b) a short length of strand must be heated to a high temperature. The former is impractical in a bridge girder and the latter will affect the material properties of the strand. Strand heating is not recommended. Commercially available strand splices have couplers connected to reverse threaded anchors; as the coupler is turned, both anchors are drawn toward each other, inducing a prestress in the attached strand, see figure 7.6. Strand splicing are normally not used for internal post tensioning systems.

Figure 7.6 Strand splice, (Harris et. al., 2009)

Steel jacketing

Steel jacketing is the use of steel plates to encase the girder to restore girder strength. With this repair technique, post-tensioning force can only be introduced by preloading. Generally, this method of repair will also require shear heads, studs or through bars to affect shear transfer between the steel jacket and substrate beam. Steel jacketing is felt to be a very cumbersome technique. In most applications, field welds will be necessary to ‘close’ the jacket (since the jacket cannot be ‘slipped over’ end of beam in most applications). Additionally, the jacket will need to be grouted in order to make up for dimensional discrepancies along the beam length. In figure 7.7 External steel jacketing for strengthening in shear is shown, here we can also notice that the steel plates are anchored in the compressive zone (Schnellenbach-Held et. al., 2016).

Figure 7.7 Shear strengthening of a concrete bridge in Germay (Rose rans Kerkeni, 2010)

<u>External non post-tensioned CFRP Technique</u>
In this method, CFRP materials are applied to the concrete member as tension reinforcement. The CFRP is applied using a structural adhesive and is not stressed in any way prior to application. Thus the CFRP is composite with the substrate concrete only in resisting loads applied following CFRP application. For this reason, non-PT retrofits are typically only used to affect the ultimate capacity of the prestressed concrete member. Typically, the critical limit state of such externally bonded retrofit measures is associated with failure of the CFRP- concrete interface. In figure 7.8 passive CFRP strengthening of a road bridge outside Karlstad in Sweden.

Figure 7.8 Passive strengthening with CFRP laminates, (Photo B. Täljsten)

<u>External prestressed or post tensioned CFRP technique</u>
Prestressed or post-tensioned (the terms are used inconsistently in the literature) CFRP repairs require affecting a tensile stress in the CFRP. Under stress, the CFRP is attached to the prestressed member. There are three approaches to installing PCFRP systems. The following terminology is adopted;

*Prestressed CFRP:* The CFRP is drawn into tension using external reaction hardware and is applied to the concrete substrate while under stress. The stress in maintained using the external reaction until the bonding adhesive is cured. The reacting stress is released and the "prestress" is transferred to the substrate concrete. This method of prestressing is potentially susceptible to large losses at stress transfer and long term losses due to creep of the adhesive system. Additionally, details (such as FRP U-wraps) must be provided to mitigate debonding at the termination of the CFRP strips or plates. Prestressed

CFRP systems are analogous to prestressed concrete systems where the stress is transferred by bond to the structural member.

*Unbonded post-tensioned CFRP:* The CFRP is drawn into tension using the member being repaired to provide the reaction. The stress is transferred to the member by mechanical anchorage. Typically a hydraulic or mechanical stressing system will be used to apply the tension after which it will be “locked off‟ at the stressing anchorage. This method of posttensioning is susceptible to losses during the “locking off‟ procedure. Depending on the anchorage method, long term losses due to creep in the anchorage is a consideration. Such systems must be designed with sufficient clearance between the CFRP and substrate concrete to mitigate the potential for fretting. Unbonded post-tensioned systems are analogous to conventional unbonded post tensioning systems.

*Bonded post-tensioned CFRP*: The CFRP is stressed and anchored in the same fashion as unbounded systems. Following anchorage, the CFRP is bonded to the concrete substrate resulting in a composite system with respect to loads applied following CFRP anchorage. Since the adhesive system is not under stress due to the post-tension force, adhesive creep is not a significant consideration with this system. The bonding of the CFRP may help to mitigate creep losses associated with the anchorage. Bonded post-tensioned systems are analogous to conventional bonded post tensioning systems.

In figure 7.9 a laboratory test of a active CFRP system is shown. The concrete beam is strengthen with external CFRP tendons (Bennitz, 2011)

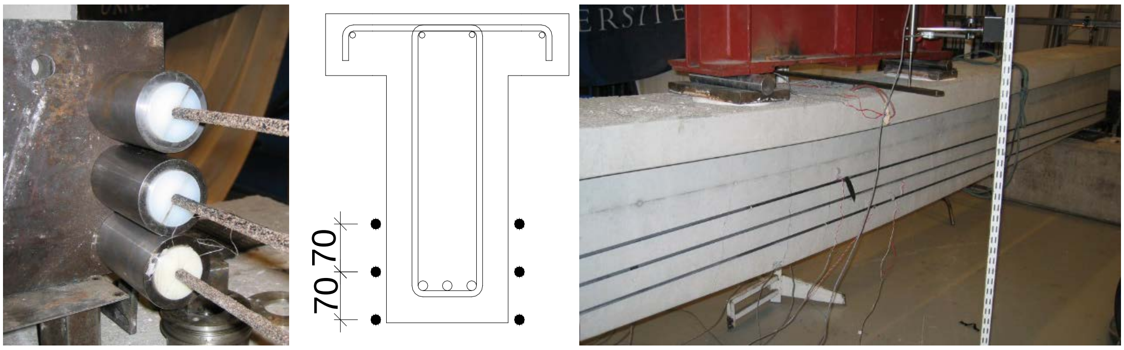



Figure 7.9 Active strengthening with CFRP rods, (Bennitz, 2011)

The result from this test show that you can obtain as high strengthening effect as for external steel tendons, however, the CFRP systems are linear elastic which means that the failure has to be controlled via the anchorage system.

In table 7.2 a summary of selection criteria’s has been made considering the strengthening methods describe above.

Table 7.2 Strengthening selection criteria’s, based on (Shanafelt and Horn, 1980).

| **Damage Assessment Factor** | **Repair/Strengthening Method** | | | |
|---|---|---|---|---|
| | **External PT** | **Steel Jacket** | **CFRP** | **CFRP PT** |
| Behavior at ultimate load | Excellent | Excellent | Excellent | Excellent |
| Overload | Excellent | Excellent | Excellent | Excellent |
| Fatigue | Excellent | Excellent | Excellent | Excellent |
| Adding strength to non-damage girders | Excellent | Excellent | Excellent | Excellent |
| Preload required | No | Probably | Probably | No |
| Restore of loss concrete | Excellent | Excellent | Excellent | Excellent |
| Speed | Good | Good | Excellent | Good |
| Durability | Excellent | Good | Good | Good |
| Cost | Low | High | Medium | Medium |
| Aesthetics | Fair | Fair | Excellent | Fair |

However, every repair/strengthen object is unique and a detailed evaluation for the specific project should always be carried out.

### 7.5 Design consideration for bridges with external tendons

In general traditional codes and standard can be used for repair and strengthening with external tendons of steel or CFRP if considerations is taken to the existing load on the structure, see (Nordin, 2005, Ng, 2003) and figure 7.10. There is a difference in the ductility behaviour between beams with prestressed CFRP tendons compared to steel tendons. In general, a conventional prestressed concrete beam with steel tendons will deform elastically until cracking, and then the member deflections will progressively increase as the tendon yields. FRP prestressed beams deform elastically until cracking and then continue to deform under increasing load until the tendons rupture or the concrete crushes.

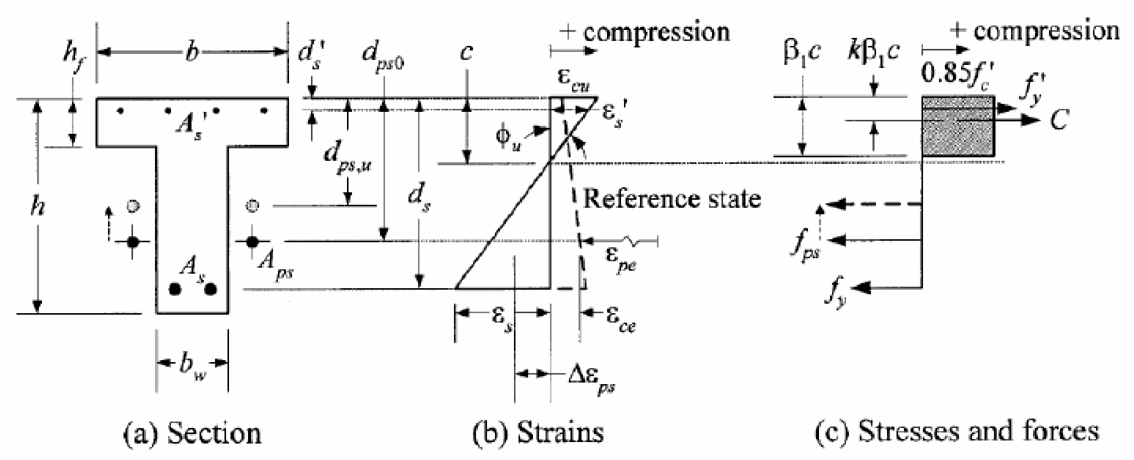



Figure 7.10 Strain and stress distribution at critical section of externally prestressed beam at ultimate limit state, (Ng, 2003)

In figure 7.10 the ultimate moment can be determined using the following equation:

$$(7:1)\quad M_u = A_{ps} f_{ps}\left(d_{ps,u} - k\beta_1 c\right) + A_s f_y\left(d_s - k\beta_1 c\right) + A_s^{'} f_y^{'}\left(d_s^{'} - k\beta_1 c\right)$$

Where $k\beta_1 c$ is the depth of centroid of the concrete compression zone. For a rectangular section or a T-section with $\beta_1 c \le h_f$ ($h_f$ is the flange thickness) *k=0.5*. For $\beta_1 c \le h_f$ then k has to be determined from the centroid of the force in the concrete compression zone. For unbonded tendons refers to e.g. (Naaman et. al., 2002). For CFRP the material parameters for the external steel is replaced with a strain based design criteria, see (Täljsten et. al., 2016).

# 8 Proposal for future actions

## 8.1 General

As has been shown earlier in this report, there has over the years been a considerable work carried out in many different parts in Europe to increase the safety and efficiency of prestressed concrete bridges. Some examples are. FIP(1980), *fib*-IABSE (2001), *fib* Bull 26 (2003), *fib* Bull 33 (2005), *fib*-Bull 62 (2012), *fib* Model Code (2010), Godart (2014), BAST (2012), BAST (2016), Harris et al. (2012), Hurlebaus (2017), HTRL 3/246 (2003),_ASTRA 12210 (2007), ASTRA 82015 (2011), BA 86/06 (2017).

In order to propose a way for Trafikverket to continue with the work regarding prestressed concrete bridges, some examples of possible future actions are described in this chapter. The proposals do not cover everything but highlights the findings from the work presented in this report, based on case studies and an extensive literature study.

In our opinion an international cooperation is the best long term way to go forward of the following reasons

- A lot of research in the area has been carried out in several countries. If we use this we will have a better platform to start from.
- The problems that have occurred up to now are rare. An international study gives a better and broader input both to risk analyses and on how to find solutions.
- The costs will be significantly lower since it will be shared among several participants.
- Implementation will go faster since different demonstrators can be done in parallel.

## 8.2 Short term

### 8.2.1 EU-project;

Support an initiative for an EU-project. This has already started by establishing contacts with ECTP (European Contractors Technology Platform) and possible partners in Europe. ECTP is probably the most suitable Technology Platform to use in order to contact EC. Next activity is proposed to be to contact EC and to create a core team.

### 8.2.2 A deeper study concerning the bridges managed by Trafikverket

In this study we have tried to answer how comprehensive the problem is and also how serious the problem is in Sweden. It has been done by some case studies and by input from reports. We can from this knowledge conclude that problems exist. In order to get a more precise knowledge, a more extensive study is recommended for post-stressed bridges built before 1980. This choice is also supported by the international literature. The study should also look at used post-tensioning systems and possible sensitive parts.

### 8.2.3 A preliminary guideline on how to do inspect Post-stressed bridges

Based on reports studied, it is possible to compile a preliminary guideline on how enhanced inspections can be performed. This can be based mainly on French, German, Swiss and UK experiences.

### 8.2.4 Investigations of post-stressed bridges

Today the knowledge is poor on the status of the post-stressed bridges managed by Trafikverket. It would be of help to have an increased knowledge on how useful and accurate different monitoring and inspection technique are. These two areas can looked into by an investigation of post-stressed bridges. Thus, we propose that post-stressed bridges that are planned be replaced, should be investigated guided by the findings in this report.

## 8.3 Long term

A proposal for long-term international studies was already outlined in Figure 1.1, also summarized in the next Chapter 9. It is preferable that projects that may start, fits into such a concept. The concept includes implementation to ensure market uptake and not only the produce of shelf warmers.

A future asset management system must take care of the aging post-stressed bridges better than what is done at present. Input can come from the investigations proposed above p. But it must be implemented and used in practice by Trafikverket.

# 9 Conclusions and further studies

## 9.1 Conclusions

Overall

- Our society is dependent on a well-functioning and safe infrastructure. Most long and important concrete bridges are prestressed bridges and they must be kept under control and in good condition in order not to mal-function or even collapse due to corrosion in the prestressing cables. This corrosion is often hard to detect as it appears inside the structure and cannot be detected with the naked eye.
- If a problem occurs it is important to have plans for how to handle the situation without unnecessary traffic disturbances.
- This report aims to identify how extensive and serious the problems are with prestressed concrete bridges and to propose actions to mitigate them

From this study:

- In the late seventieths a lot of progress was made regarding materials and construction. But bridges built before 1980 may have problems. Trafikverket in Sweden has about 400 such bridges. The seriousness of their problems should be evaluated.
- No single nondestructive monitoring or inspection techniques can alone give enough information for decisions since the overall accuracy is not sufficient.
- Today there are on the market strengthening and repair methods that are cost-effective and environmentally friendly. They are promising but have to be further evaluated.
- Better knowledge of the status of post tensioning cables is important as well as how accurate and well monitoring and inspection technique work. Investigation of bridges that are planned to be replaced is a cost-effective way to go forward.

## 9.2 Further studies

In order to meet the long term demands we propose further studies in order to:

- better understand the behavior of prestressed bridges
- protect, detect and mitigate corrosion of prestressing reinforcement
- elaborate nondestructive monitoring and inspection methods.
- augment methods for risk analyses and for the study of the reliability and the robustness of prestressed concrete bridges
- develop methods for Condition Assessment and for Life Cycle Cost Assessment (LCCA) to optimize maintenance strategies. The methods ought to be calibrated by full scale tests of bridges decided to be demolished
- advance repair and strengthening methods
- promote proactive and adaptive maintenance strategies that supports a sustainable society

Such studies could preferably be carried out in an international research project

# References

Web links are active in 2018


Abeles, P. W. & Bardhan-Roy, B. K. (1981). Prestressed Concrete Designer's Handbook. 3d ed. View-point Publications, London, 1981

ACI 209 (1982). *Prediction of creep, shrinkage and temperature effects in concrete structures*. reported by American Concrete Institute Committee 209. Subcommittee II, Report No. ACI209R-82, ACI Publication SP-76, 1982 and subsequent revisions.

ACI 222.2R-14 (2014).Report on Corrosion of Prestressing Steels. Reported by ACI Committee 222, Farmington Hills, MI, US: American Concrete Institute, 2014, 53 pp. 1st Edition 2001, 43 pp

ACI 228.2R-13 (2013). Report on Nondestructive Test Methods for Evaluation of Concrete in Structures Reported by ACI Committee 228, Farmington Hills, MI, US: American Concrete Institute, 2013, 81 p

Al-Neshawy, Fahim, Sistonen, E, Puttonen, J, Ferreira, M & Bohner, E (2016), Selection Matrix for Non-Destructive Testing of NPP Concrete Structures. In "19th World Conference on Non-Destructive Testing 2016", WCNDT 2016 (NDT.Net), Munich, Germany, 13-17 June, 9 pp.. NPP refers to Nuclear Power Plants. Available at https://research.aalto.fi/files/9890026/we2e1.pdf

Ahlberg, Sven Olof and Spade, Bengt (2001). Våra broar - en kulturskatt (Our bridges - a cultural heritage. In Swedish). Stockholm: Banverket och Vägverket, 446 pp., ISBN 91-88250-43-1

Anania, Laura; Badalà, Antonio and D'Agata, Giuseppe (2018). Damage and collapse mode of existing post tensioned precast concrete bridge: The case of Petrulla viaduct. Engineering Structures, vol 162, 2018, pp 226-244.

Andersson, Patrick (2007). Structural integrity of prestressed nuclear reactor containments, PhD Dissertation, Report TVBK-1035, Division of Structural Engineering, Lund University, 6 + 62 pp

Anderson, Patrick (2016). Spännarmerade betongkonstruktioner vid kärntekniska anläggningar. (Prestressed concrete for nuclear plants. In Swedish). Stockholm: Strålsäkerhetsmyndigheten, Rapport 2016:13, 92 sid. Available at https://www.stralsakerhetsmyndigheten.se/contentassets/c73d641152264cda986671d611e2eac4/201613-spannarmerade-betongkonstruktioner-vid-karntekniska-anlaggningar

ASTRA 12210 (2007). Massnahmen zur Gewährleistung der Dauerhaftigkeit von Spanngliedern in Kunstbauten. (Durability of Pre- and Post-stressing Reinforcement. In German). Bern: Bundesamt fúr Strassen, Guideline 12010, 50 pp.

ASTRA 82015 (2011). Schrägseile und externe Spannglieder für den Brückenbau. (Cables for Prestressed Bridges. In German). Bern: Bundesamt fúr Strassen, Guideline 82015, 118 pp.

ATB Bro-UH 2002 (2002). Vägverkets allmänna tekniska beskrivning för underhåll av broar. Brounderhåll 2002. (Rules for maintenance), Vägverket Publ 2002:48, 94 pp. https://trafikverket.ineko.se/Files/sv-SE/11038/RelatedFiles/2002_48_brounderhall_2002.pdf

B5 (1965). Bestämmelser för Betongkonstruktioner. Material och utförande, Betong. (Concrete and Concrete Materials. In Swedish). Statens Betongkommitté 1965, Svensk Byggtjänst, Stockholm, ca 120 sidor. Rev. 1973, 123 pp.

B6 (1968). Bestämmelser för Betongkonstruktioner. Material och utförande. Armering. (Reinforcement. In Swedish). Statens Betongkommitté 1969, Svensk Byggtjänst, Stockholm, 77 pp.

B7 (1968) Bestämmelser för Betongkonstruktioner. Material och utförande. Allmänna Konstruktions-bestämmelser.(Design of Concrete Structures. In Swedish). Statens Betongkommitté 1969, Svensk Byggtjänst, Stockholm, 91 pp.

BA 86/06 (2017), Advice Notes on the Non-Destructive Testing of Highway Structures. London: Highways England, Scotland, Wales and Northern Ireland. Design Manual for Roads and Bridges, Volume 3 Highway Structures – Inspection and Maintenance, Section 1 Inspection, Part 7, Incorporating Amendment No 1 dated May 2017, 300 pp.

Bagge, Niklas (2017). Structural Assessment Procedures for Existing Concrete Bridges. Experiences from failure tests of the Kiruna Bridge. *Doctoral Thesis, Luleå University of Technology*, ISBN 978-91-7583-879-3, 310 pp. http://ltu.diva-portal.org/

Bagge, N., Nilimaa, J., Blanksvärd, T., Täljsten, B., Elfgren, L., Sundquist, H. & Carolin, A. (2016): Assessment and failure test of a prestressed concrete bridge. In "Life-Cycle of Engineering Systems. Emphasis of Sustainable Civil Infrastructure", Ed. by Bakker, J., Frangopol, D.M. and van Breugel, K. CRC Press, Taylor and Francis Group, London, ISBN 978-1-138-02847-0. Abstract p 194. Full paper on attached USB card pp 1058-1063.

Bagge N, Nilimaa J & Elfgren L (2017): In-situ methods to determine residual prestress forces in concrete bridges. *Engineering Structures*, 135, 41-52

Bagge N, Popescu C & Elfgren L. (2018a). Failure tests on concrete bridges: Have we learnt the lessons? *Structure and Infrastructure Engineering*. 2018;14(3):292–319. Available from: http://urn.kb.se/resolve?urn=urn:nbn:se:ltu:diva-65060</div>

Bagge N, Plos M & Popescu C (2018b): A multi-level strategy for successively improved structural analysis of existing concrete bridges: Examination using a prestressed concrete bridge tested to failure, *Structure and Infrastructure Engineering*, DOI: 10.1080/15732479.2018.1476562

Barrile, V. & Pucinotti, R., 2005. Application of radar technology to reinforced concrete structures: a case study. *NDT&E International*, Volume 38, p. 596–604.

BAST (2012). Verstärkerung älterer Beton- und Spannbetonbrücken. Erfahrunggssammlung 2016 (Strengthening of older Prestressed Concrete Bridges. In English). Bergisch Gladbach: Bundesanstalt für Strassenwesen (BASt), Projekt FE 15.0570/2012/NRB. By Martina Schneilenbach-Held, Torasten Welsch, Silvia Fickler, Josef Hegger, Karin Reissen.288 pp.

BAST (2016) Verstärkerungen älterer Beton- und Spannbetonbrücken. Erfahrungssammlung, Dokumentation 2016 (Strengthening of older concrete and prestressed concrete bridges. Experiences and documention 2016. In German). Bundesministerium für Verkehr und digitale Infrastruktur vertreten durch Bundesanstalt für Strassenwesen (BASt). BASt Projekt-Nr FE 15.0570/2012/NRB, 288 p. Available at: https://www.bast.de/BASt_2017/DE/Ingenieurbau/Publikationen/Regelwerke/Sonstiges/Verstaerkung-aelterer-Beton-und-Spannbetonbruecken.pdf?__blob=publicationFile&v=2

BAST (2018) Brücken an Bundesfernstrasse (Bridges on national highways. In German). Avaialble at https://www.bast.de/BASt_2017/DE/Statistik/Bruecken/Brueckenstatistik.pdf?__blob=publicationFile&v=10

BaTMan (2018) *Bridge and Tunnel Management in Sweden (BaTMan – Bro och Tunnel MANagement. In Swedish)* [Online]. Webpage with data on Swedish bridges and tunnels and recommendations for their management, Version 8, Trafikverket. https://batman.trafikverket.se/externportal# [Accessed 9th Sept 2018].

Bažant, Z.P., and Baweja, S. (1995)., “Creep and Shrinkage Prediction Model for Analysis and Design of Concrete Structures—Model B3 (RILEM Recommendation),” *Materials and Structures*, V. 28, No. 6, 1995, pp. 357-365, (with Errata, V. 29, No. 2, Mar. 1996, p. 126).

Bazant, Z.P., Yu, Q., Li, G.H., Klein, G.J., Kristek, V (2010). Excessive deflections of record–span prestressed box girder. *Concrete International* 2010; **32**(6): 44–52.

Bazant, Z.P., Hubler, M.H., Yu, Q. Excessive creep deflection – an awakening (2011). *Concrete International* 2011; **33**(8): 44.

Bazant, Zdenek P, & Wan-Wendner, Roman (2015). RILEM draft recommendation: TC-242-MDC multi-decade creep and shrinkage of concrete: material model and structural analysis Model B4 for creep, drying shrinkage and autogenous shrinkage of normal and high-strength concretes with multi-decade applicability. Materials and Structures 48(4),753-770. DOI: 10.1617/s11527-014-0485-2

BBK 79 (1979): Bestämmelser för betongkonstruktioner. Band 1 Konstruktion och Band 2 Material, Utförande, Kontroll. (Design of Concrete Structures. In Swedish). Utgivna av Statens Betongkommitté hos AB Svensk Byggtjänst, Stockholm 1979, 157 + 77 pp. ISBN 91-7332-046-3. Utgåva 2. Svensk Byggtjänst, Stockholm 1988, 163 och 81 pp. ISBN 91-7332-450-7 och 451-5.

BBK 94 (1995): Boverkets handbok om betongkonstruktioner, BBK 94, Band 1, Konstruktion, och Band 2, Material, Utförande, Kontroll. (Design of Concrete Structures. In Swedish). Boverket, Byggavdelningen, Stockholm 1995, 185 and 116 pp. ISBN 91-7147-235-5 och 91-7332-687-9.

BBK 04 (2004). Boverkets handbok om betongkonstruktioner, BBK 04. (Design of Concrete Structures. In Swedish). Karlskrona: Boverket. 273 pp, Web version https://www.boverket.se/globalassets/publikationer/dokument/2004/boverkets_handbok_om_betongkonstruktioner_bbk_04.pdf

Bennitz A., (2011), Externally unbonded post-tensioned CFRP tendons: a system solution, PhD thesis at Luleå University of Technology, ISBN 978-91-7439-206-7, p198.

Berrocal, Carlos G. (2017). Corrosion of Steel Bars in Fibre Reinforced Concrete: Corrosion mechanisms and structural performance. PhD Thesis. Göteborg: Chalmers University of Technology, Concrete Structures, No 4289, 70 pp + 6 appended papers and experimental data. ISBN 978-91-7397-608-2. Available at: http://publications.lib.chalmers.se/records/fulltext/251032/251032.pdf

BHB-K (1990): Betonghandbok, Konstruktion. (Handbook for Design of Concrete Structures. In Swedish). Edited by Krister Cederwall, Mogens Lorentsen & Lars Östlund. AB Svensk Byggtjänst och Cementa AB, 2nd Ed., Stockholm 1990, 791 sidor. ISBN 91-7332-533-3,

BHB-M (1994): Betonghandbok, Material. (Handbook on Concrete Marerials. In Swedish). Edited by Christer Ljungkrantz, Göran Möller & Nils Petersons. AB Svensk Byggtjänst och Cementa AB, 2nd Ed., Stockholm 1994, 1127 sidor. ISBN 91-7332-709-3. New version being published 2018-2019.

BHB-M (2017): Betonghandbok Material, Del I, Delmaterial, samt färsk och hårdnande betong (Handbook on Concrete Materials, Part I. Materials, fresh and hardening concrete. In Swedish). 3rd Edition., Stockholm 2017, 50 pp. eISBN 978-91-7333-827-1

Blanksvärd, Thomas (2017) H. H. C. P., 2017. *Icke-förstörande provning i samband med tillståndsbedömning*, Luleå: SBUF, Skanska, Luleå Tekniska Universitet.

Bolle, Guido; Schacht, Gregor; Marx, Steffen. 2010. Geschichtliche Entwicklung und aktuelle Praxis der Probebelastung. (Loading tests of existing structures – history and present practise. In German). *Bautechnik*, 87(11&12), 700-707 & 784-789.

British Standards Institution, 1988. *BS 1881, Part 204*, London: British Standards Institution,.

Brobyggnadsanvisningar (1968) Statens Vägverk P TB 103

Bronormer (1976) Statens Vägverk TB 103

Bronormer Tillägg (1987) Statens Vägverk TB 103

Bronormer (1978-11) Statens Vägverk TB126

BRO 94 (1999). BRO94 Allmän teknisk beskrivning för broar. (General Technical Description for Bridges,. In Swedish). Borlänge, Sweden: Swedish National Road Administration. Publikation 1994:2, 1994:3, 1994:4, 1994:6, 1994:7, 1994:8, 1999:18, 1999:19, 1999:20, 1999:21, 669 sid. ISSN: 1401-9612.

Brühwiler, Eugen (2014): Bridge "examineering" or how monitoring and UHPFRC improve the performance of structures. . In "Bridge Maintenance, Safety, Management and Life Extension". Edited by Chen, Frangopol&Ruan. Taylor & Francis Group, London, pp 13-26, ISBN 978-1-138-00103-9.

Brühwiler, Eugen (2015): Swiss standards for existing structures – Four years of implementation. IABSE Workshop Helsinki, Safety, Robustness and Condition Assessment of Structures February 11-12, 2015, International Association for Bridge and Structural Engineering, IABSE, Zürich, pp 2 – 11

BV BRO (1997): BV BRO, utgåva 4, Banverkets ändringar och tillägg till Vägverkets BRO 94, (1997). (Rail Bridge Design. In Swedish). Handbok BVH 583.10, Banverket, CB, Borlänge 1977, 77 pp.

BV Bärighet (1996): Bärighetsbestämning av järnvägsbroar. (Assessment of Rail Bridges. In Swedish). Handbok BVH 583.11. Justerad utgåva av förhandskopia 1996-01-17. Banverket, TB, Sektion Bansystem, Borlänge, 68 sidor. Revised 2000-03-01, 108 pp + 6 appendices.

BV Bärighet (2000): Bärighetsberäkning av järnvägsbroar. Handbok BVH 583.11. Banverket, CB, Borlänge 2000-03-01, 108 sidor + 6 bilagor.

BYGG 9 (1966). BYGG - Handbok för hus-, väg- och vattenbyggnad, Del 9 Väg- och vattenbyggnad (Handbook in Civil E(ngineering. In Swedish). Kapitel 933:7 av Uko Müllersdorf "Brokonstruktioner av spännbetong" Bridges of Prestressed Concrete), pp 373-385. Stockholm: Byggmästarens förlag 1966, 3rd Ed., 1167 pp. (1st Ed ca 1950).

Bygginnovationen (2010). Delrapport Bro, Bygginnovationen fas 1 (Bridge Building Innovations. In Swedish). VINNOVA Research and Innovation Project, Stockholm 2010-11-04, 68 pp. Available at http://www.bygginnovationen.se/library/2806/rapport-bro-170316.pdf

Calvi, Gian Michele; Moratti, Matteo; O'Reilly, Gerard J.; Scattarreggia, Nicola; Monteiro, Ricardo; Malomo, Daniele; Calvi, Paolo Martino & Pinho, Rui (2018). Once upon a Time in Italy: The Tale of the Morandi Bridge, Structural Engineering International, 20 pp. DOI: 10.1080/10168664.2018.1558033

Camomilla CG, Marioni A, Martinez C F, Pisano, F (1995): Repair of the stay cables of the Polcevera Viaduct in Genova, Italy. Presented at: Extending the Lifespan of Structures, IABSE Symposium, San Francisco, August 1995, pp. 535-540.

Cao, Da-fu; Qin, Xiao-Chuan; Meng, Shao-Ping; Tu, Yong-Ming; Elfgren, Lennart; Sabourova, Natalia; Grip, Niklas; Ohlsson, Ulf & Blanksvärd, Thomas (2015): Evaluation of prestress losses in prestressed concrete specimens subjected to freeze–thaw cycles, Structure and Infrastructure Engineering: Maintenance, Management, Life-Cycle Design and Performance, DOI: 10.1080/15732479.2014.998241

Casas JR (2015): The bridges of the future or the future of bridges? Frontiers in the Built Environment. 1:3, 8 April 2015. doi: 10.3389/fbuil.2015.00003

CEB-FIP (1993): Model Code 1990. Design Code, Comité Euro-International du Béton, Bulletin d'Information No 213/214, Thomas Telford, London 1993, 437 pp (ISBN 0- 2777-1696-4).

CEPA (2007). Stress Corrosion Cracking. Recommended Practices, 2nd Ed. , Calgary, Canada: Canadian Energy Pipeline Association, CEPA. Available at https://www.cepa.com/wp-content/uploads/2011/06/Stress-Corrosion-Cracking-Recommended-Practices-2007.pdf

Collins, M P., and Mitchell, D. (1991). *Prestressed Concrete Structures*, Prentice Hall, Englewood Cliffs, N.J., 1991, 766 pp.

Concrete Society (1996). Durable Bonded Post-Tensioned Concrete Bridges. Technical Report 47. The Concrete Society. Crowthome. UK.

Coric, Ibrahim; Täljsten, Björn; Blanksvärd, Thomas; Ohlsson, Ulf & Elfgren, Lennart (2018). Railway Bridges on the Iron Ore Line in Sweden. From Axle loads of 14 to 32,5 ton. IABSE Conference 2018. Engineering the Past, to Meet the Needs of the Future, June 25-27, 2018, Copenhagen, Denmark, 8 pp

COST TU1406 (2016-2018). Cost Action TU1406: Quality Specifications for Roadway Bridges. Standardization at a European Level. General info: https://www.tu1406.eu/
Four Technical Reports and other material available at: https://www.tu1406.eu/file-repository
WG 1. Performance indicators for Roadway Bridges, 2016, 40 pp + Data base online
WG 2. Performance Goals for Roadway Bridges, 2017, 62 pp.
WG 3. Establishment of a Quality Control Plan, 2018, 114 pp.
WG 4. Guidelines for Preparation of a Case study, 2018, 34 pp

COWI (2008). Vägverket - H200 Bro över Kalmarsund, Ölandsbron. Särskild inspektion av kablar (Inspection of cables in the Ölands-bridge at Kalmarsund. In Swedish). Written by Finn Janssen och Lars Ahlvik , 72 pp

Davis, A C (1924). A Hundred Years of Portland Cement 1824-1924. London: Concrete Publications limited, 282 pp

Davis, Joseph R., Editor (2000). Corrosion: Understanding the Basics. Materials Park, OH, US: ASM International, 563 pp, ISBN 978-0-87170-641-6.

Dowling Norman E (2012), Mechanical Behaviour of Materials. International Ed. Pearson Education, Ltd, 969 pp. ISBN 9780273764557.

Duvnjak, Ivan; Damjanovic, Domago; Sabourova, Natalia; Grip, Niklas; Ohlsson, Ulf; Elfgren, Lennart & Tu, Yongming; (2019). Damage Detection in Structures - Examples. IABSE Symposium 2019 Guimaraes, Portugal: Towards a Resilient Built Environment - Risk and Asset Management, 8pp.

EC0 (2002): *Eurocode 0: Basis of structural design, EN 1990;2002*. The Eurocodes are issued by the European Committee for Standardization or, in French: Comité Européen de Normalisation, CEN, and the corresponding national Standard Associations as e.g. the Swedish Standards Institute, SIS. There is

a lot of material developed to assist in the use of the Eurocodes see e.g. handbooks by the Leonardo da Vinci Pilot Project CZ/02/B/F/PP-134007, see below EC Handbook 1-4 (2004-2205)
There was also a dissemination workshop in Brussels in 2008 with many good presentations: http://eurocodes.jrc.ec.europa.eu/showpage.php?id=332#EN1994 (Assessed 2018-12-21).

EC1 (2002-2006) *Eurocode 1: Actions on structures*, *EN 1991*. Part 1 (2002-2006): 1-1 Self weight, 1-2 Fire Actions, 1-3 Snow, 1-4 Wind, 1-5 Thermal Actions, 1-6 Construction loads, 1-7 Accidental Actions. Part 2 (2006): Traffic on bridges, Part 3 (2006): Loads from cranes, Part 4 (2006): Silo loads.

EC2 (2004-2006): *Eurocode 2: Design of concrete structures*, *EN 1992*. Part 1: General rules; Part 2(2005): Concrete Bridges; Part 3(2006): Liquid containment structures.

EC3 (2005-2007): *Eurocode 3: Design of steel structures*, *EN 1993*. Part 1(2005-2007) Design of steel structures, 12 subparts; Part 2 (2006) Steel bridges; Part 3 (2006): Towers, masts; Part 4 (2007): Silos; Part 5 (2007): Piling; Part 6 (2007): Supporting structures.

EC4 (2004-2005): *Eurocode 4: Design of composite structures, EN 1994*. Part 1 (2004-2005): General rules; Part 2 (2005): Bridges.

EC5 (2004) *Eurocode 5: Design of timber structures, EN 1995.*

EC6 (2005-2006): *Eurocode 6: Design of masonry structures, EN 1996.*

EC7 (2004, 2007): *Eurocode 7: Geotechnical design, EN 1997.*

EC8 (2004-2005): *Eurocode 8: Design for earthquake resistance, EN 1998.*

EC9 (2007): *Eurocode 9: Design of aluminium structures, EN 1999.*

EC Handbook 1 (2004): *Basis of Structural Design. Handbook 1*. Leonardo da Vinci Project CZ/02/B/F/PP-134007. Development of Skills facilitating implementations of Eurocodes, 155 pp. Available at http://eurocodes.jrc.ec.europa.eu/showpublication.php?id=62

EC Handbook 2 (2005): *Reliability Backgrounds. Handbook 2*. Leonardo da Vinci Project CZ/02/B/F/PP-134007. Development of Skills facilitating implementations of Eurocodes, 254 pp. Available at. http://eurocodes.jrc.ec.europa.eu/showpublication.php?id=63

EC Handbook 3 (2005): *Action effects for buildings. Handbook 3*. Leonardo da Vinci Project CZ/02/B/F/PP-134007. Development of Skills facilitating implementations of Eurocodes, 158 pp. Available at: http://eurocodes.jrc.ec.europa.eu/showpublication.php?id=64

EC Maintenance (2018). State of infrastructure maintenance. Discussion Paper. European Commission, DG for Internal Market, Industry, Entrepreneurship and SMEs, Industrial Transformation and Advance Value Chains, Clean Technologies and Products. Brussels, 27 September 2018, 17 pp. . https://ec.europa.eu/docsroom/documents/31721

Elfgren, L. (2015) Fatigue Capacity of Concrete Structures: Assessment of Railway Bridges. Research Report, Structural Engineering, Luleå University of Technology, 103 pp. [Online] Available at http://ltu.diva-portal.org/ [Assessed 9th September 2018].

Elfgren L, Bagge N, Nilimaa J, Blanksvärd T, Täljsten B, Shu J, Plos M, Larsson O & Sundquist H (2015). Brottbelastning av en 55 år gammal spännbetongbro i Kiruna - Kalibrering av modeller för tillståndsbedömning: Slutrapport till BBT (Failure loading of a 55 year old prestressed bridge in Kiruna. - Calibrating of assessment models. In Swedish). Luleå: Luleå University of Technology. Department of Civil and Environmental Engineering. Division of Structural Engineering; 2015. (Forskningsrapport /

Luleå tekniska universitet). 39 + 206 pp. Available from: http://ltu.diva-portal.org/smash/get/diva2:996746/FULLTEXT01.pdf and http://ltu.diva-portal.org/smash/get/diva2:996746/FULLTEXT02.pdf

Elfgren, Lennart; Täljsten, Björn; Blanksvärd, Thomas; Sas, Gabriel; Nilimaa, Jonny; Bagge, Niklas; Tu, Yongming; Puurula, Arto; Häggström, Jens& Paulsson, Björn (2018). Load testing used for quality control of bridges. *COST TU 1406*, Wroclaw, 1-2 March, 6 pp, http://www.tu1406.eu/

Emborg M, Cederwall K. & Elfgren L, (1982). Fatigue strength of cable couplers in prestressed concrete beams. *Nordic Concrete Research* 1982;1:5.1-5.14. https://www.danskbetonforening.dk/media/ncr/publication-no-01-05.pdf

Emborg Mats (1988). Fatigue strength of cable couplers in prestessed concrete beams. *Nordisk Betong 1988;*1988(2–3):62–70. Available from: http://ltu.diva-portal.org/smash/get/diva2:978036/FULLTEXT01.pdf, EN 1990 – EN 1999 (2002-). See EC0 – EC9

Emmons P.H., (1993), Concrete Repair and Maintenance Illustrated, problem Analysis, Repair Strategy, Techniques, ISBN 0-87629-286-4, pp314

Engström, Björn (2011). Design and analysis of prestressed concrete structures. Göteborg: Chalmers University of Technology, Concrete Structures, Report 2011:7.

Enochsson, Ola; Hejll, Arvid; Nilsson, Martin; Thun, Håkan; Olofsson, Thomas & Elfgren, Lennart (2002). Bro över Luossajokk, Beräkning med säkerindexmetod, Böjdragkapacitet i överkant i mittsnittet i korta spannet. (Bridge over Luossajokk assessed with reliability method. In Swedish). Luleå: Teknisk Rapport 2002:06, 106 pp. Available at: http://ltu.diva-portal.org/smash/get/diva2:996544/FULLTEXT01.pdf

ETSI (2012) Bridge Life Cycle Optimisation. A Nordic Project. Several publications tools available at http://etsi.aalto.fi/

fib Bull 26 (2003)..Influence of material and processing on stress corrosion cracking of prestressing steel – case studies. Technical Report, Task Group 9.5, International Federation for Structural Concrete (fib), Lausanne, 44 pp. https://www.pci.org/PCI_Docs/Members_Only/Technical%20Resources/Fib%20Bulletin/fib_Bull26_NMG%20OCR.pdf

fib Bull 33 (2005) Durability of post-tensioning tendons. Recommendations by Working Party 5.4.2. , International Federation for Structural Concrete (fib), Lausanne, 79 pp. https://www.pci.org/PCI_Docs/Members_Only/Technical%20Resources/Fib%20Bulletin/fib_Bull33_NMG.pdf

fib Bull 62 (2012): *Structural Concrete. Textbook on behaviour, design and performance. 2nd Ed., Volume 5. Through-life care and management of concrete structures – assessment, protection, repair and strengthening*. International Federation for Structural Concrete (fib), Lausanne, January 2012, 462 pp, ISBN 978-2-88394-102-1.

fib-IABSE (2001). Durability of post-tensioning tendons. Proceedings of a workshop held at Ghent University on 15-16 November 2001 edited by Luc Taerwe. Technical Report, Bulletin No 15, fib – International Federation of Concrete in collaboration with IABSE – International Association for Bridge and Structural Engineering, 279 p. Available at https://www.pci.org/PCI_Docs/Members_Only/Technical%20Resources/Fib%20Bulletin/fib_Bull15_NMG%20OCR.pdf

*fib* Model Code (2010). Fib model code for concrete structures 2010. International federation for concrete (fib), Berlin, Ernst & Sohn, 2013, 402 pp

FIP (1980). Report on Prestressing Steel 5. Stress corrosion cracking resistance for prestressing tendons. Lausanne: International Federation for Structural Concrete (fib), 58 pp.

Fischer O, Müller A, Lechner T, Wild M & Kessner K (2014). Ergebnisse und Erkenntnisse zu durchgeführten Nachrechnungen von Betonbrücken in Deutschland. (Findings and insights concerning the results of re-analyzed concrete bridges in Germany. In German with English summary and captions). Beton- und Stahlbetobau 109(2014). No 2, pp 107-127.

fib Model Code (2010). Fib model code for concrete structures 2010. International federation for concrete (fib), Berlin, Ernst & Sohn, 2013, 402 pp.

Freyssinet, Eugène (1950). Lecture on "Pre-stressed Concrete: Principles and Applications" by Eugène Freyssinet, presented at a joint meeting on Thursday, 17 November, 1949, of the Institution of Civil Engineers and the Société des Ingénieures de Civils de France (British Section). *Journal of the Institution of Civil Engineers*, Vol 33, Issue 4, February 1950, pp 331-380.
https://doi.org/10.1680/ijoti.1950.12924

Germann Instruments, 2018. *http://germann.org.* [Online]
Available at: http://germann.org/products-by-application/flaw-detection/mira
[Accessed 23 05 2018].

Ghasemi, Rohollah (2011). Hydrogen-assisted stress corrosion cracking of high strength steel . M Sc Thesis, Stockholm: Royal Institute of Technology and SWEREA/KIMAB. 70 pp. http://www.diva-portal.org/smash/get/diva2:559358/FULLTEXT01.pdf

Godart, Bruno, Editor (2014). La Pérennité du Béton Prtécontrinte (Durability of prestressed concrete. In French). Association Francaise de Genie Civil, AFGC, Presses des Ponts, Paris, 443p., ISBN 978-2-85978-483-6.

Grace, N., Hanson, J. & Abdel Messsih, H., 2004. *Inspection and Deterioration of Bridge Decks Constructed Using Stay-in-Place Metal Forms and Epoxy-Coated Reinforcement,* Southfield: Lawrence Technological University.

GSSI, 2017. *Concrete Handbook - GPR Inspection of Concrete,* Nashua: Geophysical Survey Systems, Inc..

Habel W, Kohlhoff H, Knapp J & Helmerich R (2002). Monitoring system for long-term evaluation of prestressed railway bridges in the new Lehrter Bahnhof in Berlin. Third World Conference on Strucutral Control, 7-12.4.2002, Como, Italy, 7 pp

Halabe, U. B., Sotoodehnia, A., Maser, K. R. & Kausel, E. A., 1993. Modeling the Electromagnetic Properties of Concrete. *ACI MATERIALS JOURNAL,* 90(6).

Hallberg D, Racutanu G, 2007. Development of the Swedish bridge management system by introducing a LMS concept. Materials and Structures, Vol. 40, Issue 6, pp. 627–639.

Hallbjörn, Lars (2015). Betongplattor, beräkning och dimensionering. (Design of Concrete Slabs. In Swedish). TRITA-BKN Rapport 154, Bro - och Stålbyggnad, Stockholm: KTH and Report K7002B-2015-1, Konstruktionsteknik, Luleå: LTU, 281 pp.

Haegermann, Gustav; Huberti, Günter; Deinhard, Johan-Martin & Leonhardt, Adolf (1964). *Vom Caementum zum Spannbeton: Beitrage zur Geschichte des Betons*, Band I mit Teil A: "Vom Caementum zum Zement" von Gustav Haegermann, Teil B: "Die erneuerte Bauweise" von Günter Huberti und Teil C: "Der Spannbeton". - Band II: "Massivbrücken gestern und heute" von Johann-Martin Deinhard. - Band III: "Von der Cementware zum konstruktiven Stahlbetonfertigteil" von Adolf Leonhardt. - Mit insgesamt 1234 Abbildungen. Bauverlag GmbH, Wiesbaden, Berlin, 1964.

Harris K.A., Kasan J. & Aktas C., (2009), Repair Methods for Prestressed Girder Bridges, Commonwealth of Pennsylvania Department of Transportation, Contract #510601, Report No. FWHA-PA-2009-008-PIT 006, p178

Harris K. A., Kasan J., Miller R. and Brinkman R., 2012, Guide to recommended practice for the repair of impact-damaged prestressed concrete girders, p 36, http://onlinepubs.trb.org/onlinepubs/nchrp/docs/NCHRP20-07(307)_AppendixA-GUIDE.pdf (February 8, 2019)

Haveresch Karl-Heinz, 2000, Verstärkung älter Spannbetonbrücken mit koppelfugenrissen. (Strengthening of Older Prestressed Concrete Bridges with Cracks at the Stages of Construction. In German with an Abstract and Figure Captions in English). *Beton- und Stahlbetonbau*, 95, 2000, Heft 8, pp 452-460. Available at: https://rdcu.be/bdLeetonbrûcken

Halabe, U. B., Sotoodehnia, A., Maser, K. R. & Kausel, E. A., 1993. Modeling the Electromagnetic Properties of Concrete. *ACI MATERIALS JOURNAL*, 90(6).

Heinrich, Jens & Maurer, Reinhard (2018). Check of resistance to fatigue on existing prestressed concrete bridges by monitoring. Proc. of the 12th fib Int. PhD Symposium in Civil Engineering, Aug 29 to 31, 2018, Czech Technical University in Prague, Czech Republic, pp 987-994. Available from: https://www.fib-international.org/

Hendriks, M.A.N.; de Boer, A; Belletti, B (2017). Guidelines for Nonlinear FEA of Concrete Structures, Rijkswaterstaat Centre for Infrastructure, Report RTD:1016-1:2017, The Netherlands, 69 pp.

Honfi, D., Portal N. W., Leander L., Larssson Ivanov, Björnsson Í, Plos M., Zandi K., Carneriro E., Lechner T., Magnusson J., Gabrielsson H., 2018, Inspection and monitoring of bridges in Sweden,, SP Rapport 2018:25, pp 51.

HTRL 3/246 (2003). Trial of non-destructive testing advice notes. London: Highways Agency Transport Research Laboratory Contract 3/246, Project no 2562. 84 pp

Hu, Wei-Hua; Tang, D-H; Teng, J; Said, S & Rohrmann, R G (2018). Structural Health Monitoring of a Prestressed Concrete Bridge Based on a Statistical Pattern Recognition of Continuous Dynamic Measurements over 14 years. *Sensors, 18, 4117*, doi: 10.3390/s18124117, 28 pp. Available at https://www.ndt.net/article/ndt-review/papers/sensors-18-04117-v2.pdf

Hurlebaus S., Hueste M.B.D., Karthik M.M, and Terzioglu T., (2017), *"Inspection guidelines for Bridge Post-Tensioning and Stay Cable Systems Using NDE Methods"*, ISBN 978-0-309-44632-7, p 258

Häggström, Jens (2016). Evaluation of the Load Carrying Capacity of a Steel Truss Railway Bridge: Testing, Theory and Evaluation. Lic. Thesis, Luleå Univ. of Tech., 142 pp. http://ltu.diva-portal.org/

Häggström, Jens; Blanksvärd, Thomas; Täljsten, Björn (2017). Bridge over Åby River – Evaluation of full scale testing. Res. Rep, Div. of Struct. Eng., Luleå Univ. of Tech., 180 pp. http://ltu.diva-portal.org/

International atomic energy agency, 2002. *Guidebook on non-destructive testing of concrete structures*, Vienna: IAEA.

ISO 2394:2015 (2015), General principles on reliability of structures, Organisation internationale de normalisation. Geneva, Switzerland.

ISO 13822 (2010): Basis for design of structures – Assessment of existing structures, Genéve ISO, 46 pp.

(IO, 2015. ISO 2394:2015, General principles on reliability of structures, Organisation internationale de normalisation. Geneva, Switzerland.

ISO, 2010. ISO 13822:2010, Bases for design of structures – Assessment of existing structures. Geneva: International Organization for Standardization.

ISO, 2010. 17635:2010, Non-destructive testing of welds -- General rules for metallic materials. Geneva: International Organization for Standardization.

Johnson, R. P. (2009): *Eurocodes, 1970-2010: why 40 years?,* Proc. of the Inst. of Civil Engineers, Structures and Buildings, Vol. 162, Dec. 2009, Issue SB6, pp. 371-379. doi: 10.1680/stbu.2009.162.6.371

Jol, H. M., 2008. *Ground penetrating Radar: Theory and Applications.* 1st ed. s.l.:Elsevier.

JCSS (2018): Joint Committee on Structural Safety. In 1971 the Liaison Committee which co-ordinates the activities of six international associations in civil engineering, composed of CEB , CIB, fib, IABSE, and RILEM, created a Joint Committee on Structural Safety, JCSS, with the aim of improving the general knowledge in structural safety, They have published a report in 2008 on Risk Assessment in Engineering, 35 pp + Background documents and Examples. See https://www.jcss.byg.dtu.dk/

JCSS PMC (2001): Probabilistic Model Code, Issued by the Joint Committee on Structural Safety, JCSS, 12th Draft, March 2001-2013. 1. Basis of Design, 64 pp; 2. Load Models; 3. Resistance Models, 4. Examples. Available at: https://www.jcss.byg.dtu.dk/Publications/Probabilistic_Model_Code

JRC Assessment (2015): *New European Technical Rules for the Assessment and Retrofitting of Existing Structures. Policy Framework. Existing Regulations and Standards. Prospect for CEN Guidance.* Support to the implementation, harmonization and further development of the Eurocodes. JRC Science and Policy Report. Authors Paul Lüchinger et al. Editors Silvia Dimona et al. Report EUR 27128 EN, European Commission, Joint Research Centre (JRC), Institute for the Protection and Security of the Citizen. Ispra 2015, 137 pp., ISBN 978-92-79-46023-4. doi: 10.2788/052881. Available at http://publications.jrc.ec.europa.eu/repository/handle/JRC94918 or http://bookshop.europa.eu/en/new-european-technical-rules-for-the-assessment-and-retrofitting-of-existing-structures-pbLBNA27128/

JRC Bridge Examples (2012): *Bridge Design to Eurocodes. Worked Examples*. Worked examples presented at the Workshop "Bridge Design to Eurocodes", Vienna, 4-6 October 2010. Support to the implementation, harmonization and further development of the Eurocodes. JRC Scientific and Policy Report. Authors Yosra Bouassida et al., Editors A. Athanasopoulou et al. Report EUR 25193 EN, European Commission, Joint Research Centre (JRC), Ispra 2012, 438 pp., ISBN 978-92-79-22823-0. doi: 10.2788/82360. Available 2018-12-21 at http://eurocodes.jrc.ec.europa.eu/doc/1110_WS_EC2/report/Bridge_Design-Eurocodes-Worked_examples.pdf

Kolm, Rudolf (1929). Brobyggnader (Bridges. In Swedish). De allmänna Vetenskaperna, Stockholm:, Bonniers. Cited from Ahlber & Spade (2001).

Kordina, Karl (1979). Schaden an Koppelfugen, (Damage on cable couplers. In German). *Beton- und Stahlbetonbau,* No. 4, Berlin 1979, pp 95-100. Cited from Emborg et al.(1982).

Kukay, B. M. (2008). "*Bridge instrumentation and the development of non-destructive and destructive techniques used to estimate residual tendon stress in prestressed girders*." Ph.D. Thesis, Utah State University, Logan, UT, United State

Kukay, B., Barr, P. J., Halling, M. W., & Womack, K. (2010). "Determination of the residual prestress force of in-service girders using non-destructive testing." *Structures Congress 2010,* American Society of Civil Engineers, Orlando, FL, United States, 709-716.

Kühn, B., Lukic, M., Nussbaumer, A., Günther, H.-P., Helmerich, R., Kolstein, M. H., Walbridge, S., Androic, B., Dijkstra, O. and Bucak, Ö (2008): Assessment of Existing Steel Structures: Recommendations for Estimation of Remaining Fatigue Life. Luxembourg: Office for Official Publications of the European Communities. JRC Scientific and Technical Reports. ISSN 1018-5593, 108 pp. Available at http://eurocodes.jrc.ec.europa.eu/doc/background/EUR23252EN.pdf

KVVS (1961). Föreskrifter och anvisningar för Brobyggnad (Bronormer). (Rules for design and construction of bridges. In Swedish), Kungl. Väg och vattenbyggnadsstyrelsen Stockholm 1961-1965 Binder with updateable pages., cited from BYGG 9 (1966).

Lacroix, Roger (2004). Eugène Freyssinet. A Revolution in the art of construction. Preface by Roger Lacroix and authored by Bernard Fargeot, Jean Montagnon, Pierre Xercavins, Carlos de la Fuente, Frank Guyon & Pierre Jartoux. Association pour la mémoire et le rayonnement des travaux d'Eugène Freysinnet. Paris: Presse de l'école nationale des Ponts et chaussées, 197 pp.

Lantsoght, Eva O.L.; van der Veen, Cor; de Boer, Ane; Hordijk, Dick A. (2017). State-of-the-art on load testing of concrete bridges. *Engineering Structures*, 150, Nov 2017, 231-241.

Lachowicz, J. & Rucka, M., 2016. *Experiemental and Numerical Investigations for GPR Evaluations of Reinforced Concrete Footbridge.* Gdansk, IEEE.

Lahouar, S., 2003. *Development of Data Analysis Algorithms for Interpretation of Ground Penetrating Radar Data.* Blacksburg: Virginia Polytechnic Institute and State University.

Larsson, Ulf (1997). Brobyggaren. Otto Linton, byggnadskonsten och dess professioner i Norden under första halvan av 1900-talet (The Bridge Builder. Otto Linton, 1880-1938, and building in the Nordic countries during 1900-50. In Swedish).. Stockholm: Carlssons, 397 pp, ISBN 91 7203 2022.

Leonhardt, Fritz (1964) Prestressed Concrete: Design and construction, 2nd fully revised Ed. 1st German Ed 1955 “Spannbeton für die Praxis”, Ernst & Sohn, Berlin Germany.

Leonhadt, Fritz (1975). Rissebeschränkung (Crack limitation. In German). Vorträge auf den Betontag/ Deutscher Beton-Verein. Wiesbaden: Deutscher Beton-Verein e.V, 1957-1979, 1975, 0417-2876, pp 422-428, 454.

Leonhardt, Fritz (1984). Bridges. Aesthetics and Design (Bilingual; also in German: Brücken. Ästhetik und Gestaltung). The MIT Press, Cambridge, Massachusettds, 310 pp.

Lorentsen, Mogens (1963). Spännbetong (Prestressed concrete. In Swedish) SVR:s Förlags AB 1963, Lagerblads, Karlshamn .

Liao H (2014) Post-tentioned tendon corrosion evaluation and mitigation. IABMAS Shanghai

Lin, T. Y. & Burns, N. H. (1981). Design of Prestressed Concrete Structures. 3d ed. John Wiley & Sons, New York, 1981.

Lundqvist, Peter (2012). Assessment of long-term losses in prestressed concrete structures. Application to nuclear reactor containments, PhD Dissertation, Report TVBK-1043, Division of Structural Engineering, Lund University, 168 pp. Also published as Elforsk Rapport 12:

Maierhofer, C., 2003. *Nondestructive Evaluation of Concrete Infrastructure*, Berlin: Federal Institute for Materials Research and Testing.

MAINLINE (2014). *MAINtenance, renewaL and Improvement of rail transport INfrastructure to reduce Economic and environmental impacts*. A European FP7 Research. Projoject 2011-2014. Twenty reports are available at http://www.mainline-project.eu/
.
Magnel, Gustave (1948). Prestressed Concrete. London: Cement and Concrete Association, 1948.

MAINLINE (2014). *MAINtenance, renewaL and Improvement of rail transport INfrastructure to reduce Economic and environmental impacts*. A European FP7 Research. Projoject 2011-2014. Twenty reports are available at http://www.mainline-project.eu/

MALÅ Geoscience, 2013. *MALÅ Easy Locator HDR User Manual,* Malå: MALÅ Geoscience.

Malerba, P. G. (2010): Managing old bridges. Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko, Taylor & Francis Group, London 2010, pp 43-52. ISBN 978-0-415-87786-2. Available at https://books.google.se/books?id=rwvNBQAAQBAJ&lpg=PA43&ots=T9IepI2RvF&dq=polcever

Matt P., (2000), "Durability of Prestressed Concrete Bridges in Switzerland" / 16th Congress of IABSE, September 2000

Mayer T F, Harnisch J, Ebell G & Dauberschmidt C (2018). Korrosionsmonitoring von Stahlbetonbauwerken. Das DGZfP-Merkblatt B12 (Corrossion Monitoring of Prestressed Concrete Structures. DGZfP Specification B12. In German), Beton- und Stahlbetonbau, Vol 113 (2018) No 9, p 632-639

Mechbal, Z. & Khamlichi, A., 2017. Determination of concrete rebars characteristics by enhanced postprocessing. *NDT&E International,* Volume 89, p. 30–39.

Menn, Chrisitan (1990). Prestressed Concrete Bridges, Birkhäuser/Springer, 536 pp

ML-D1.2 (2015). Assessment methods for elderly rail infrastructure. Delivery D1.2 in MAINLINE, a European FP7 Project, 112 pp. Available at http://www.mainline-project.eu/Results,7.html

ML-D1.3 (2015). New Technologies to extend the life of elderly rail infrastructure, Delivery D1.3 in MAINLINE, a European FP7 Project, 194 pp. Available at http://www.mainline-project.eu/Results,7.html

ML-D1.4 (2015):*Guideline for application of new technologies to extend life of elderly rail infrastructure.* MAINLINE Deliverable D1.4, 2014-09-30, revised March 2015, 146 pp, see http://www.mainline-project.eu/

ML-D5.7 (2014). Manual for a Life Cycle Assessment Tool (LCAT) for Railway Infrastructure Metallic Bridges, Track and Soil cuttings. Delivery D5.7 in MAINLINE, a European FP7 Project, 125 pp. Available at http://www.mainline-project.eu/Results,7.html

Morandi, Riccardo (1944). Procedimento e dispositivo per la realizzazione di travi di cemento armato precompresso, Italian patent n. 411311.

Morandi, R (1967) Il viadotto sul Polcevera per l'autostada Genova-Savona. L'Industria italiana del ce mento, 12, 1967

Müllersdorf, Uko (1961). Förspända konstruktioner, kapitel 314; Beräkning av förspända betongkonstrukltioner, kapitel 334 (Prestressed concrete structures; General and Design. In Swedish). BYGG 2&3, 3rd Ed., Stockholm: Byggmästarens Förlag, pp 507-512 & 673-701. (1st Ed 1947, 2nd Ed 1959)

Naaman A.E., Burns N., French C., Gamble W.L. and Mattock A.H. (2002) "Stresses in Unbonded Prestressing Tendons at Ultimate: Recommendation" ACI Structural Journal, July-August, 2002

National cooperative highway reserach program report 280, (1985), Guidelines for evaluation and repair of presressed concrete bridge members, Transport Research Board, national Research Council, Wasington D.C., ISBN 0-309-04013-2, p 93

Ng C. (2003), Tendon Stress and Flexural Strength of Externally Prestressed Beams, ACI Structural Journal, September-October 2003, pp 644-653

Nilimaa, Jonny; Sabau, Cristian; Bagge, Niklas; Puurula, Arto; Sas, Gabriel; Blanksvärd, Thomas; Täljsten, Björn; Carolin, Anders; Paulsson, Björn & Elfgren, Lennart (2018). Assessment and Loading to Failure of Three Swedish RC Bridges. In: Evaluation of Concrete Bridge Behavior through

Load Testing: International Perspectives. Ed. by Eva Lantsoght and Pinar Okumus, American Concrete Institute, Farmington Hills, MI, USA; Special Publication SP 323, Paper 323-8, p. 8.1- 8.18

NHRP 280, (1985), Guidelines for evaluation and repair of presressed concrete bridge members, Transport Research Board, National cooperative highway reserach program, National Research Council, Wasington D.C., ISBN 0-309-04013-2, p 93

Nilimaa, J. (2015). "*Concrete bridges: Improved load capacity*." Ph.D. Thesis, Luleå University of Technology, Luleå, Sweden

Nilsson, Karl Robin (2014). Getting the arch back into architecture. Master Thesis at Chalmers Architecture, 2014-05-13, MSc Design for Sustainable Development. Göteborg: Chalmers University of Technology, 92 pp. Availabale at: http://publications.lib.chalmers.se/records/fulltext/203465/203465.pdf . Video presentation at: https://vimeo.com/105850478

Nilsson, Martin; Ohlsson, Ulf & Elfgren, Lennart (1999): Partialkoefficienter för hållfasthet i betongbroar längs Malmbanan (Partial coefficients for concrete strenth in bridges on the Iron Ore line. In Swedish). Luleå: Teknisk Rapport 1999:03, **79pp**. Available at http://ltu.diva-portal.org/smash/get/diva2:994939/FULLTEXT01.pdf

Nordin H., (2005), "Strengthening structures with externally prestressed tendons - Literature review", Technical Report, Luleå university of technology, 2005:06, ISSN 1402-1536, p84.

Nordin, Håkan (2006). Strengthening structures with externally prestressed tendons. Literature review. Technical Report 2006:06, Luleå: Luleå University of Technology, Structural Engineering, 84 pp

Nürnberger, Ulf (2002). Corrosion induced failures of prestressing steel. Stuttgart: Otto-Graf-Journal, Vol 13, pp 9-25. https://www.mpa.unistuttgart.de/publikationen/ otto_graf_journal/ogj_2002/beitrag_nuernberger.pdf

Petersson, Tage and Sundquist, Håkan (2001). Spännbetong (Prestressed Concrete. In Swedish). Royal Institue of Technology, Structural Engineering, TRITA-BKN Report 46, Stockholm, 1st Ed 1997, 3rd Ed 2001, 212 pp

Pfohl, H (1979). Erfassung von Rissen im Bereich der Koppelfugen von Spannbetonbrücken. (Cracks at cable couplers in prestressed concrete bridges. In German). Mitteilungen der Bundesanstalt für Strassenwesen 3/79. *Strasse und Autobahn*, Heft 12, 1979. Cited from Emborg (1988).

Picard A., Massicotte B. and Bastien J. (1995) “Relative Efficiency of External Prestressing” Journal of Structural Engineering, Vol. 121, No. 12, December 1995

Popescu, Cosmin; Täljsten, Björn; Blanksvärd, Thomas; Sas, Gabriel; Jimenez, Alexander; Crabtree, David; Carolin, Anders & Elfgren, Lennart (2019). Optical methods and wireless sensors for monitoring of bridges. IABSE Symposium 2019 Guimaraes, Portugal: Towards a Resilient Built Environment - Risk and Asset Management, 8pp.

Popescu, Cosmin; Täljsten, Björn; Blanksvärd, Thomas & Elfgren Lennart (2019). 3D reconstruction of existing concrete bridges using optical methods. Accepted in 2018 for publication in *Structure and Infrastructure Engineering*.

Pourbaix, Marcel (1973). Lectures on Electrochemical Corrosion, (Translated from French). Springer US, Boston, MA, 1973, 336 pp.. doi:10.1007/978-1-4684-1806

Puurula Puurula, Arto (2012) Load-carrying Capacity of a Strengthened Reinforced Concrete Bridge: Non-linear..., *PhD, Luleå Univ. of Tech.*, ISBN 978-91-7439-433-7, 332 p. http://ltu.diva-portal.org/

Puurula, Arto, Enochsson, O, Sas, G, Blanksvärd, T, Ohlsson, U, Bernspång, L & Elfgren, L. (2014). Loading to failure and 3D nonlinear FE modelling of a strengthened RC bridge. *Structure & Infrastructure Engineering*, 10(12), 1606-1619.

Puurula, Arto, Enochsson, O, Sas, G, Blanksvärd, T, Ohlsson, U, Bernspång, L, Täljsten, B, Carolin, A, Paulsson, B & Elfgren, L. 2015. Assessment of the Strengthening of an RC Railway Bridge with CFRP Utilizing a Full-Scale Failure Test and Finite-Element Analysis, J. Struct. Engineering, ASCE, 2015, 141, D4014008, 11 p.

Qin, Xiao-Chuan; Meng, Shao-ping; Cao, Da-fu; Tu, Yong-ming; Sabourova, Natalia; Grip, Niklas; Ohlsson, Ulf; Blanksvärd, Thomas; Sas, Gabriel & Elfgren, Lennart (2017). Evaluation of freeze-thaw damage on concrete material and prestressed concrete specimens, Construction and Building Materials 125 (2016), 892–904

Rail Traffic (2017) Bantrafik 2017. Trafikverket, Sweden. [Online]. Available from : https://www.trafa.se/globalassets/statistik/bantrafik/bantrafik/2017/bantrafik-2017_juni.pdf [Assessed 9th September 2018]

Recio, F.J., Wu, Y., Alonso, M.C., Nürnberger, U. (2013). Hydrogen embrittlement in cold-drawn stainless steels. *Materials Science & Engineering A*, Vol 564 (2013), 57-64.

Rogers, J., 2008. *www.concreteconstruction.net.* [Online]
Available at: http://www.concreteconstruction.net/how-to/construction/post-tensioned-slabs_o
[Accessed 23 05 2018].

Roeser W and Kerkeni N., (2010), Ertüchigung der beiden Eisenbahnbrücken A13, Beton- und Stahlbetonbau 105, 2010, Heft 5, s- 324-329.

Roth, Thomas (2004). Langzeitverhalten von Spannstählen in Betonkonstruktionen (Lifetime Issues Concerning Prestressing Steel in Concrete Structures. In German). Department of Civil and Architectural Engineering, Division of Structural Design and Bridges Royal Institute of Technology SE Stockholm, Sweden TRITA-BKN. Bulletin 77, 2004 ISSN ISRN KTH/BKN/B--77--SE Licentitate Thesis, 119 pp.. http://www.diva-portal.org/smash/get/diva2:7028/FULLTEXT01.pdf

Sabourova, Natalia; Grip, Niklas; Ohlsson, Ulf; Elfgren, Lennart; Tu, Yongming; Duvnjak, Ivan & Damjanovic, Domago (2019). Detection of Sparee Damages in Structures. IABSE Symposium 2019 Guimaraes, Portugal: Towards a Resilient Built Environment - Risk and Asset Management, 8pp

Sanabra-Loewe, Marc & Capellá-Lloera, Joaqiun (2014). The four ages of early prestressed concrete structures. PCI Journal, Fall 2014, pp 93-121. Available at https://core.ac.uk/download/pdf/41775639.pdf

SB-D1.2 (2004): *European Railway Bridge Demography*. Deliverable D1.2 compiled by Brian Bell, Network Rail, in the Sustainable Bridges project, 15 pp. Available at the WP1 Section in http://www.sustainablebridges.net/

SB-D1.4 (2005): *Railway Bridge Research. An overview of current research carried out in the Sustainable Bridges project*. Deliverable D1.2 compiled by Luleå University of Technology in the Sustainable Bridges project, 44 pp. Available at the WP1 Section in http://www.sustainablebridges.net/

SB-D3.4 (2007): Condition assessment and inspection of steel railway bridges including stress measurements in riveted, bolted and welded structures. 94 pp. Background document D3.4 to SB-ICA *Inspection and Condition Assessment*. Prepared by Sustainable Bridges- a project within EU FP6. Available from: www.sustainablebridges.net.

SB-D6.2 (2007): *Repair and Strengthening of Railway Bridges. Litterature and Research Report.*, 807 pp.  Available from: www.sustainablebridges.net: Prepared by Sustainable Bridges - a project within EU FP6.

SB-D6.3 (2007): Field Tests. Örnsköldsvik Bridge, Vitmossen, Frövi Bridge, 181 pp. Available from: www.sustainablebridges.net: Prepared by Sustainable Bridges - a project within EU FP6.

SB-D6.4 (2007): Method statement guideline. *Workmanship and Quality Control of CFRP strengthened structures, 74 pp.* Available from: www.sustainablebridges.net: Prepared by Sustainable Bridges - a project within EU FP6.

SB-D7.3 (2008): *Field Test of a Concrete Bridge in Örnsköldsvik, Sweden*.  Elfgren L, Enochsson O, and Thun H. Deliverable D 7.3, Sustainable Bridges – Assessment for Future Traffic Demands and Longer Lives, see SB (2007). Available from:  www.sustainablebridges.net

SB-ICA (2007): *Inspection and Condition Assessment*. Guideline developed in the EC-FP6 Project Sustainable Bridges, 259 pp; Available at www.sustainablebridges.net

SB-LRA (2007): *Load and Resistance Assessment of Railway Bridges.* Guideline developed in the EC-FP6 Project Sustainable Bridges 428 pp. Available at www.sustainablebridges.net

SB-MON (2007): *Guideline for Monitoring of Railway Bridges*, Guideline developed in the ECFP6 Project Sustainable Bridges, 93 pp. Available at www.sustainablebridges.net

SB-STR (2007): *Guide for use of Repair and Strengthening Methods for Railway Bridges.* Guideline developed in the EC-FP6 Project Sustainable Bridges, 139 pp. Available at www.sustainablebridges.net

SCB (2007): *Livslängden I Sverige 2001-2005*. (*Life length is Sweden* .InSwedish with a summary in English), Statistiska Centralbyrån (Statistics Sweden), Stockholm 2007, 136 pp, see http://www.scb.se/statistik/_publikationer/BE0701_2001I05_BR_BE51ST0701.pdf (Accessed 2015-06-20)

Scheers, B., 2001. Chapter 2. Ground penetrating radar. In: *Ultra-Wideband Ground Penetrating Radar, with Application to the Detection of Anti Personnel Landmines.* Brussels: s.n., pp. 2-1 -2-43.
Schneider, Jörg (1994): *Sicherheit und Zuverlässigkeit im Bauwesen. Grundwissen für Ingenieure*. (Safety in Civil Engineering. Basics for Engineers. In German) Unter Mitarbeit von Hans-Peter Schlatter. vdf Hochschulverlag AG an der ETH Zürich, 188p., ISBN 978-3-7281-2167-3. DOI 10.3218/2167-7,

Schneider, Jörg & Vrouwenvelder, Ton (2017): Introduction to Safety and Reliability of Structures. 3rd reviewed and extended Ed. (1st Ed. 1997). Structural Engineering Documents No 5, International Association for Bridges and Structural Engineering (IABSE), c/o ETH Zürich, Switzerland, 164 pp. ISBN 978-3-85748-131-2.

Sederholm (2006). Korrosion av spännkablar i efterspända betongbroar (Corrosion in post-rensioned cables in prestressed concrete. In Swedish) . KIMAB-2006-121, Stockholm 30p.

Shanafelt, G.O. and Horn, W.B. 1980. Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members, NCHRP Report 226. Transportation Research Board, 66 pp.

Shaw P., Rasmussen J., Pedersen T.K., and Goodwin N., and Gudmundsson G., 2004, "A practical guide to non destructive examination of concrete, Report Nordic Innovation, 2004, p114.

Shushkewich, Kenneth W (2012). Eugène Freyssinet - Invention of Prestressed concreete and Precast Segmental Construction. *Structural Engineering International*, Vol 22, 2012, Issue 3, pp 415-420.

SIA 269 (2011): Existing structures – Bases for examination and interventions, Swiss Societyof Engineers and Architects (SIA), Zurich. (in German and French).

SJ Bro (1955). Broavdelningens konstruktionsanvisningar (Design rules for railway bridges. In Swedish. Kungl. Järnvägsstyrelsen, Statens Järnvägar (SJ), see TRV Tåglast (2010).

Schnellenbach-Held M., Welsch T. & Fickler S., 2016, "Verstärkungen älterer Beton- und Spannbetonbrücken - Erfharugssammlung Dokumentation 2016, Bundesanstalt für Straβenwesen (BASt), Bruderstraβe 53, 514 27 Bergisch Gladbach, FE 15.0570/2012/NRB, p233

SOU 1934:17 (1934). Statliga cement- och betongbestämmelser av år 1934. (Rules for Cement and Concrete. In Swedish). Kommunikationsdepartementet 1934, 61 pp. Available at: https://weburn.kb.se/metadata/328/SOU_1368328.htm

SOU 1949:64 (1949). Statliga betongbestämmelser. Del 1. Materialdelen. (Rules for Concrete Structures. In Swedish). Stockholm: Kommunikationsdepartementet 1949-12-31, 84 pp. Available at: https://weburn.kb.se/metadata/277/SOU_619277.htm

Sousa, Helder & Oliveira Santos, L (2016). Long-term performance of prestressed concrete bridges. In "8th European Workshop On Structural Health Monitoring (EWSHM 2016)", 5-8 July 2016, Spain, Bilbao. 10pp. Available at: https://www.ndt.net/events/EWSHM2016/app/content/Paper/233_Sousa.pdf

Sreeshylam P, Ravisankar K, Parivallal S, Kesavan K & Srihar S (2008). Condsition Monitoring of Prestressed Concrete Structures using Vibrating Wire Sensors. Int J of COMADEM, 11(3), July 2008, pp 46-54.

SS-EN 1992-1-1:2005 (2008). Eurokod 2: Dimensionering av betongkonstruktioner - Del 1-1 Allmänna regler och regler för byggnader (Eurocode 2 - Design of concrete structures - Part 1-1: General rules and rules for buildings. In Swedish). Stockholm: Swedish Standards Institute (SIS), 236 pp.

SS-EN 1992-2:2005 (2005). Eurocode 2 - Design of concrete structures - Part 2: Concrete bridges - Design and detailing rules, Stockholm: Swedish Standards Institute (SIS), 104 pp

Steel Construction (2019). https://www.steelconstruction.info/Corrosion_of_structural_steel

Steinbock O, Mertzsch O, Hampel T, Schmidt N & Curbach M (2018). Systematische Nachrechnung des Brückenbestands in Mecklenburg-Vorpommmern (Systematic recalculation oft he bridge inventory of Mecklenburg-West Pommern – Interaction between practise and research. In German). Beton- und Stahlbetonbau , 113(2018), Heft 4, 281-290

Sultan, A. A. & Washer, G. A., 2017. Reliability Analysis of Ground-Penetrating Radar for the Detection of Subsurface Delamination. *Journal of Bridge Engineering,* 23(2).

Sultan, A. A. & Washer, G. A., 2017. Reliability Analysis of Ground-Penetrating Radar for the Detection of Subsurface Delamination. *Journal of Bridge Engineering,* 23(2).

Sundquist, Håkan (2009). Safety, Loads and Load Distribution on Structures. Royal Institute of Technology, Structural Design & Bridges, TRITA-BKN Report 108, 1st Ed 2007, 2wnd Ed 2009, Stockholm, 133pp.

Sundquist, Håkan (2016). Historik och utveckling av ämnet brobyggnad vid KTH (History of Bridge Engineering at the Royal Institute of Technology in Sweden. In Swedish). TRITA-BKN, Report 162, Structural Design and Bridges, KTH, The Royal Institue of Technology. Stockhom 2016, 19 pp.

Sustainable Bridges (2008): *Sustainable Bridges – Assessment for Future Traffic Demands and Longer Lives.* A European Integrated Research Project during 2003-2008. Four guidelines and 35 background documents are available at www.sustainablebridges.net

Tan K. and Ng C. K. (1997) “Effects of Deviators and Tendon Configuration on Behaviour of Externally Prestressed Beams”, ACI Structural Journal, v 94, n 1, January–February 1997.

Tang, Man-Chung (2014). The Story of the Koror Bridge. Zürich: International Association for Bridge and Structural Engineering, IABSE, Bulletins, Case Studies 1, ISBN 978-3-85748-136-9, 112 pp. Available at. http://www.iabse.org/Images/Publications_PDF/Case%20Studies/Case_Studies_Koror_web.pdf

Total Materia (2011). Corrosion protection of steel. Total Materia. https://www.totalmateria.com/page.aspx?ID=CheckArticle&site=kts&LN=EN&NM=300

(Trafikverket, 2016. Bärighetsberäkning av broar, TDOK 2013:0267, version 3.0.)

(Trafikverket, 2014. Förvaltningsdata och uppgifter i BaTMan för byggnadsverk, TDOK 2013:0263.)

(Trafikverket, 2015. BaTMans handbok. https://batmanhandbok.trafikverket.se.)

Troyano, Leonardo Fernández (2003): *Bridge Engineering. A global perspective.* London: Thomas Telford 2003, (First Spanish Ed 1999), 775 pp, ISBN 0-7277-3215-3.

TRV Capacity Rules (2017) Rules for analysis of the load-carrying capacity of bridges (Krav. Bärighetsberäkning av broar. In Swedish). Swedish Transport Organisation (Trafikverket) TDOK 2013:0267, Version 4.0, 2017-02-20, 142 pp. Earlier version 2016, TDOK 2013:0267, version 3.0 [Online] Available from http://trvdokument.trafikverket.se/ [Assessed 9th September 2018.

TRV Capacity Advice (2017) Recommendations for analysis of the load-carrying capacity of bridges (Råd. Bärighetsberäkning av broar. In Swedish). Swedish Transport Organisation (Trafikverket) TDOK 2013:0273, Version 4.0, 2017-02-20, 34 pp. [Online]. Available from: http://trvdokument.trafikverket.se/ [Assessed 9th September 2018].

TRV Data (2014). Förvaltningsdata i BaTMan (Data on structures in BaTMan. In Swedish). TDOK2013:0263, see BaTMan (2018).

TRV Design Rules (2016) Rules for design and building of bridges. (Krav Brobyggande. In Swedish). Swedish Transport Organisation (Trafikverket) TDOK 2016:0203, Version 1.0, 2016-10-03, 197 pp. [Online]. Available from: http://trvdokument.trafikverket.se/ [Assessed 9th September 2018].

TRV Design Advice (2016) Recommendations for design and building of bridges. (Råd Brobyggande. In Swedish). Swedish Transport Organisation (Trafikverket) TDOK 2016:0204, Version 1.0, 2016-10-03, 353 pp. [Online]. Available from: http://trvdokument.trafikverket.se/ [Assessed 9th September 2018].

TRV Inspection Rules (2018) Rules for inspection of structures. (Krav inspektion av byggnadsverk. In Swedish). Swedish Transport Organisation (Trafikverket) TDOK 2018:0179, 2018-06-227. [Online] Available from: http://trvdokument.trafikverket.se/ [Assessed 9th September 2018].

TRV Inspection Advice (2018) Recommendations for inspection of structures. (Råd inspektion av byggnadsverk. In Swedish). Swedish Transport Organisation (Trafikverket) TDOK 2018:0180, 2018-06-227 . [Online]. Available from: http://trvdokument.trafikverket.se/ [Assessed 9th September 2018].

TRV Traffic (2016) Trafikarbetets förändring 2015-2016 (The change of Traffic Work 2015-2016. In Swedish). Publication 2017:178 by Thomas Vestman, Stockholm: Trafikverket, 52 pp. [Online]. Available from : https://trafikverket.ineko.se/Files/sv-SE/31751/Ineko.Product.Related-Files/2017_178_trafikarbetets_forandring_2015_2016.pdf [Assessed 9th September 2018].

TRV Tåglast (2010). Tåglaster genom tiderna.Trafiklaster och tillhörnade tåglaster för Järnvägsbroar (Traffic loads for rail bridges from 1886. In Swedish). Trafikverket, Version 3.3 by Valle Janssen, 8pp

Tzanis, A., 2010. matGPR Release 2: A freeware MATLAB® package for the analysis & in-terpretation of common and single offset GPR data. *FastTimes,* 15(1), pp. 17-43.

Täljsten, B., Blanksvärd, T., Sas, G. (2011): *Handbok för dimensionering i samband med förstärkning av betongkonstruktioner med pålimmade fiberkompositer* (Design Guideline for FRP Strengthening of Existing Concrete Structures. In Swedish). Division of Structural Engineering, Luleå University of Technology. ISBN 978-91-7439-146-6, 184 pp.

Täljsten B., Blanksvärd T. och Sas G., (2016), Kompositförstärkning av betong, AB Svensk Bygg-tjänst, 113 87 Stockholm, ISBN 978-91-7333-763-2, p178

UIC 702 (2003): Static Loading Diagrams to be taken into consideration for the Design of Rail carrying Structures on lines used by International services**.**UIC Code 702, Ed. 3, 2003, International Union of Railways, Paris, 16 pp.

UIC 778-3 (2014*) Recommendations for the inspection, assessment and maintenance of masonry arch bridges*. UIC Code 778-3 R, 3rd Ed., International Union of Railways, Paris, to 167 pp.

U.S. Patent 375999 (1888). Construction of artificial stone or concrete pavements No. 375,999. Pa-tented Jan. 3, 1888. P. H. Jackson

USRadar, 2018. *usradar.com.* [Online]
Available at: http://www.usradar.com/ground-penetrating-radar-products/3d-concrete-scanning-inspection-systems/quantum-mini-concrete-scanner/
[Accessed 23 05 2018].

Utsch Jens H and Jahn Thomas 1999. Die entwicklung der externen Vorspannung

Valiente, A. & Elices, M. (2001). Premature failure of prestressed steel bars. In "Failure Analysis Case Studies II" Edited by D. R. H. Jones, Elsevier, pp 147-155. Available at https://ac.els-cdn.com/B9780080439594500158/3-s2.0-B9780080439594500158-main.pdf?_tid=5ba72120-89a0-4c65-8e9c-a0eada4a591d&acdnat=1548171777_d2df5484cc1e4d1e627635020649d87f

Varela-Ortiz, W., Lugo Cintrón, C. Y. & Velázquez, G. I., 2013. *Load testing and GPR assessment for concrete bridges on military installations*, Lincoln: US Army Research.

Von Olnhausen, Werner (1991). Sveriges vägbroar. Återblick – Nuläge – Utblick (Swedish Road Bridges. Past - Present - Future. In Swedish). Stockholm: Daedalus, Tekniska museéts årsbok, 33-53

VSL (2019). External Post-tensioning Systems, Brochures available at http://www.vsl.com/bro-chures/post-tensioning-strand-systems.html

VV Klassningsberäkning (1998): Allmän teknisk beskrivning för klassningsberäkning av vägbroar. (Assessment of road bridges. In Swedish). Vägverket Publikation 1998:78, Borlänge 1978, 110 p

Wan-Wendner, Roman; Tong, T.; Strauss, Alfred; Wang, W. & Yu, Q (2014). "Effects of concrete creep and it's randomness on long-term deflections of prestressed box girders", IALCCE Life-Cycle of Structural Systems, Tokyo, November 2014 – Furuta, Frangopol & Akiyama (Eds), Taylor & Francis Group, London, 2015, 978-1-138-00120-6, 477-483

Wang, J. & Su, Y., 2013. Fast detection of GPR objects with cross correlation and hough transform. *Progress In Electromagnetics Research C,* Volume 38, pp. 229-239.

Woodward R J (1989). Collapse of a Segmental Post-Tensioned Concrete Bridge. *Transportation Research Record no 1211*, Washington DC: Transportation Research Board, pp 38-59. Available at: https://onlinepubs.trb.org/Onlinepubs/trr/1989/1211/1211-005.pdf

Woodward R J & Williams F (1988), Collapse of Ynys-y-Gwas bridge, West Glamorgan. Proc. Instn Civ. Engrs, Part 1, Vol 84. Aug 1988c pp 635-669.

Wärnfeldt, Anders, Editor (1952). Förspänd betong. (Prestressed Concrete. In Swedish).Based on a course arranged in 1952. Stockholm: SVR.s Förlag, 221 pp. A review by Gösta Lundin is published in Teknisk Tidskrift, vol 83, 1953, pp 60-61, see http://runeberg.org/tektid/1953/0076.html

Zilch, Konrad; Niedermeier, Roland and Finckh, Wolfgang (2014): Strengthening of Concrete Structures with Adhesively Bonded Reinforcement. Design and Dimensioning of CFRP Laminates and Steel Plates. Berlin: Ernst & Sohn, Wiley, 148 pp, ISBN 978-3-433-03086-8. The original German text was published in Betonkalender 2013.

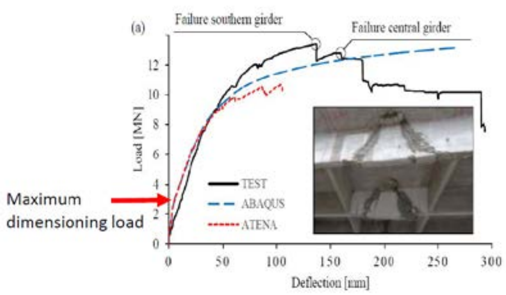
.

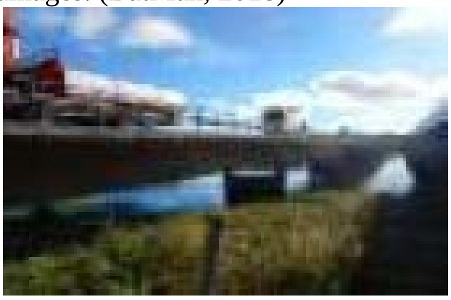
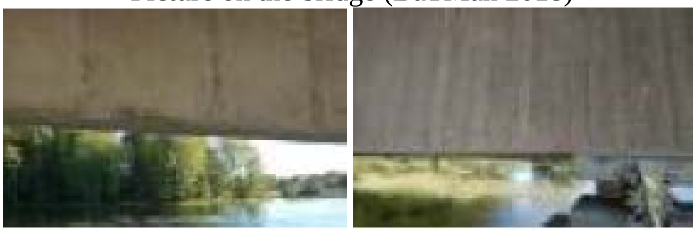
74

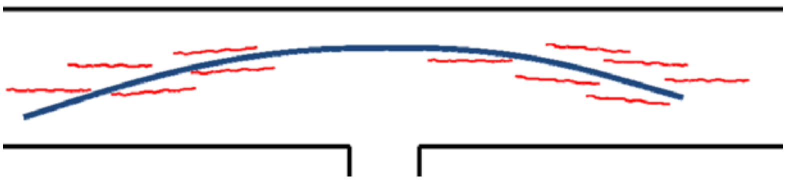
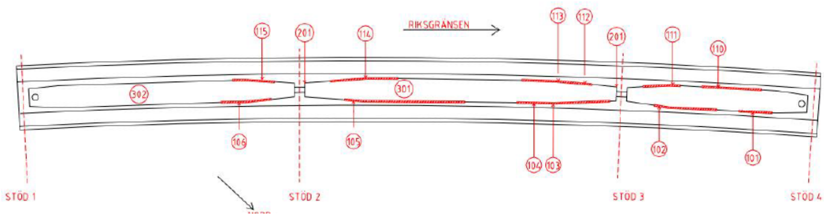
.

# Appendix A: Case Studies

This appendix is not fully structured due to varying availability of information from different bridges. Most information is from BatMan but also from old reports from Björn Paulsson. Therefore every investigated bridge starts with general information. After that the used post-tensioning system is noted followed by special information that can be of interest. Finally conclusions is drawn.

| | |
|---|---|
| **Bridge/Location** | It was a road bridge in Kiruna and opened in 1959 and demolished in2014. |
| **Type** | Beam bridge on three girders |
| **Length/Spans** | 122 m with five spans of 18.0 + 20.5 + 29.35 + 27.15 + 26.5 meters and consists of three longitudinal post-stressed girders connected with a reinforced concrete slab common way to build post-stressed continuous beam bridges.<br><br>The remarkable is that they started with the central part. It was post-stressed from each end. This was then followed by the outer segments post-stressed from the abutments. |
| **Post-Tensioned system** | BBRV 32 ø 6 probably steel quality 150/175 kp/mm$^2$. According to the drawings passive and active anchorages was used. Also couplers was used. No investigations was carried out concerning corrosion on the strands and chloride content in the grout. The ducts that was used are unknown. |
| **Designer** | Sven Hultquist Konsulterande Ingenjörsbyrå, Stockholm |
| **Contractor** | Not known |
| | The bridge war tested to failure in 2014. This is well reported in (Bagge, 2017). The test showed that this “Beam Bridge” acted linear far over maximal dimensioned load and showed a ductile behaviour in Ultimate Limit Stage (ULS). See figure AB1.1.<br><br>This behaviour is not expressed in literature since very few failure tests to collapse have taken palace on this bridge-type. It gives input to the main hypotheses in the future work that a normally maintained beam bridge acts in this way.<br><br><br><br>Figure A1 is taken from (Bagge, 2017) and shows load/deflection of the failure test. |

| | |
|---|---|
| **Bridge/Location** | A road bridge over Dalälven in Marma opened in 1977 |
| **Type** | Beam bridge on two girders |
| **Length/Spans** | 304 meters. The eight spans are 31.8 + 40 + 40 + 40 + 40 + 40 + 40 + 31.8 meters. It was traditionally built in 7 casting phases which was the common way to build continuous beam bridges.<br>It consists of two longitudinal post-stressed beams connected with a reinforced concrete slab. |
| **Dimensioned according to** | VV Bronormer 1969 |
| **Post-Tensioned system** | BBRV 44 ø 6 with steel quality 150/175 kp/mm$^2$ and with a break load of 218 Mp. According to the drawings passive and active anchorages was used. The ducts that was used are unknown. |
| **Designer** | ELU Konsult AB |
| **Contractor** | BPA Örebro |
| **Assessment information** | Notes from the last inspection done October 2015 does not show any damages that can be related to problems with post-stressing system. Below you can see to other damages. (BatMan, 2018)<br><br>Picture on the bridge (BaTMan 2018)<br><br>Pictures of damages (BaTMan 2018) |

| | |
|---|---|
| **Bridge/Location** | Abisko Bridge: A railway bridge on the Oreline in the north of Sweden going over Abiskojokka opened for traffic in 1978. |
| **Type** | Box girder bridge |
| **Length/Spans** | 94.4 meters. The tree spans are 30 + 35 + 21 meters.<br>The bridge is a traditional box girder. |
| **Dimensioned according to** | It is dimensioned with trainload typ F and VV:s Bronormer 76. |
| **Post-Tensioned system** | BBRV 44ø6 with steel quality 155/180 kp/mm$^2$. |
| **Designer** | Erik Lysedal AB |
| **Contractor** | Vägverket |
| | |
| | |
| | |
| **Assessment information** | **Bridge**<br><br><br>Picture of Abisko bridge (WSP report in BaTMan)<br>In 2011 the following cracks were found on the inside of the box above the pillar.<br><br>Figure A2 showes the cracks<br>Due to planned tests of increased axleload to 32.5 tonnes a special inspection took place in 2016. In this inspection a lot of cracks were found. See below. Most cracks were in the range of 0.1-0.25 mm. Only two were larger one 0.3 mm and one 0.4 mm.<br><br><br>Figure A3 showes all cracks and how they were documented both with locations and on photos.<br>In order to see what axleload the bridge could resist a bearing capacity calculation was performed in 2016. The calculations included SLS, ULS and fatigue. No special investigation concerning background of cracks was done.<br>In 2019 monitoring will take place in order to evaluate if cracks are propagating or if it is ordinary creep/temperature cracks.<br>Cracks in post-tensioned bridges must be better understood. Therefore parallel calculations to the monitoring campaign is recommended. |

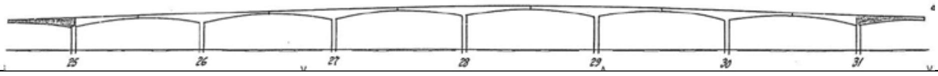
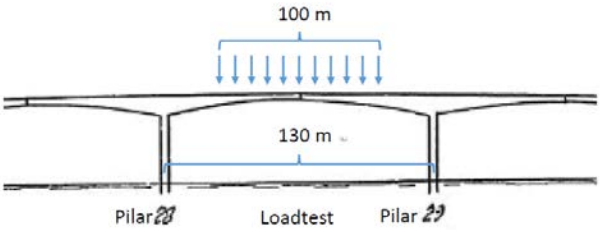
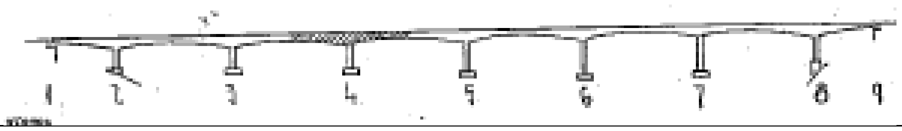
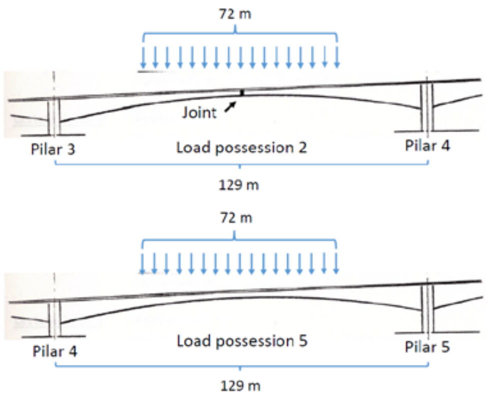
<table>
<tr><td>Bridge/Location</td><td>Ölandsbridge is a road bridge with a total length of 6072 m and opened for traffic in 1972. It connects the island Öland with Swedish mainland. The high bridge part consists of 7 cantilever spans of 130 meters. The other part is strengthened with traditional reinforcement. It is the 7 cantilever T's. See picture below.<br>

</td></tr>
<tr><td>Type</td><td>Box girder bridge</td></tr>
<tr><td>Length/Spans</td><td>The high bridge is 910 meters with six 130 meter spans.<br>The high bridge part was built with cantilever technique in seventeen phases ended with casting of a crossbeam. In all phases post-stressing bars was stressed. This was stat-of-the-art when it was built 1968-1972.</td></tr>
<tr><td>Dimensioned according to</td><td>KVVs Bronormer 1965</td></tr>
<tr><td>Post-Tensioned system</td><td>Dywidag bars was used. Bars of ø26 mm with steel quality 80/105 kp/mm$^2$ and ducts with steel thickness of 0.2 mm was used. The used post-tensioning system is well described in 4 drawings.<br>From 1968 chlorides was forbidden in grout according to Brobyggnadsanvisningar.</td></tr>
<tr><td>Designer</td><td>AB Skånska Cementgjuteriet</td></tr>
<tr><td>Contractor</td><td>AB Skånska Cementgjuteriet</td></tr>
<tr><td>Assessment information</td><td>The bridge was test-loaded in 1972 with as much loads as possible on three files. The values from the test load showed a clear linear behavior and good agreement with calculations done with small adjustment of the modulus of elasticity.<br>


Figure A4 shows the parts that was load tested in 1972, (Paulsson, 1972)<br>
Table A1 shows values from the test loading. (Paulsson, 1972)<br>
<table>
<tr><th colspan="3">Load tests on Ölandsbron</th></tr>
<tr><th colspan="3">Load possession middle of pillar 28 to 29 with joint</th></tr>
<tr><th>Section</th><th>Calculated value (mm)</th><th>Measured value (mm)</th></tr>
<tr><td>27.50</td><td>-64</td><td>-59</td></tr>
<tr><td>27.75</td><td>-43</td><td>-41</td></tr>
<tr><td>28.00</td><td>0</td><td>0</td></tr>
<tr><td>28.25</td><td>54</td><td>48</td></tr>
<tr><td>28.50</td><td>130</td><td>117</td></tr>
<tr><td>28.75</td><td>54</td><td>48</td></tr>
<tr><td>29.00</td><td>0</td><td>0</td></tr>
<tr><td>29.25</td><td>-43</td><td>-42</td></tr>
<tr><td>29.50</td><td>-64</td><td>-62</td></tr>
</table></td></tr>
</table>

Special cable inspection: Several normal inspection reports have been carried out during the years. In 2008 a special inspection of post-tensioned bars was carried out. The quality and the conclusions of this report can be questioned. The inspection was done by using endoscope and thermograph-camera.
The endoscope was done in 18 points and in 5 points the bar duct was laid bare. From this simple investigation the consultant draw the conclusion that the condition was good and that there will be no problems the next 25 years.
The investigation with the thermograph-camera was not optimal due to present temperature situation when the measuring was performed so the result from this is doubtful.


A picture above is from the special inspection showed a bar that had no grouting and some corrosion.

<table>
<tr><td>Bridge/Location</td><td>Angeredsbridge is a road bridge with a total length of 902 m and opened for traffic in 1979. It goes over two main roads, a railway line and the river Göta Älv.<br>A picture of the bridge is shown on the front-page.<br></td></tr>
<tr><td>Type</td><td>Beam bridge</td></tr>
<tr><td>Length/Spans</td><td>902 metres. The bridge consists of 7 cantilever parts and 8 spans with 68 + 129 + 129 + 129 + 129 + 129 + 122 + 66 meters individual length.<br>The bridge was built with cantilever technique in fourteen phases ended with casting of a crossbeam or a connecting part. In all phases post-stressing cables was stressed. In the end the connected part was stressed together with cables. The bars in the webs was stressed when it was possible due to weather and when it was feasible.</td></tr>
<tr><td>Dimensioned according to</td><td>VV Bronormer 1976</td></tr>
<tr><td>Post-Tensioned system</td><td>On the Angeredbridge two different post-tensioning system was used.<br>For the main system VSL 12 ø13 159/187 kp/mm$^2$ was used. The ducts was 0.5 mm.<br>In order to take care of shear forces 45° Dywidag ø26 bars were used in the webs. The steel quality 80/105 kp/mm$^2$ and ducts with steel thickness of 0.2 mm was used.</td></tr>
<tr><td>Designer</td><td>AB Skånska Cementgjuteriet</td></tr>
<tr><td>Contractor</td><td>AB Skånska Cementgjuteriet</td></tr>
<tr><td>Assessment information</td><td>The bridge was test-loaded in 1978. Below you can see an example of the result.<br><br><br>Figire A5 shows the possession of the loading (Paulsson 2018)<br>Table A.2 The table is extracted from original calculations (Paulsson 2018)</td></tr>
</table>

| Load tests on Angeredsbron Gothenburg | | |
|---|---|---|
| Load posision 2 with a joint pilar 3 to 4 | | |
| Section | Calculated value (mm) | Measured value (mm) |
| 3.125 | -11 | -8 |
| 3.250 | -28 | -23 |
| 3.375 | -50 | |
| 3.500 | -76 | -66 |
| 3.625 | -52 | -39 |
| 3.750 | -30 | -26 |
| 3.875 | -13 | -10 |
| Load posision 5 with no joint pilar 4 to 5 | | |
| 4.125 | -10 | -7 |
| 4.250 | -21 | -18 |
| 4.375 | -34 | -28 |
| 4.500 | -39 | -36 |
| 4.625 | -36 | |
| 4.750 | -24 | |
| 4.875 | -10 | |

The bridge test-loaded with as much loads as possible. It meant about 60% of dimensioning load. The values from the test load showed a clear linear behavior and good agreement with calculations done with small adjustment of modulus of elasticity. Also horizontal movements as well as stresses in crucial sections was measured. All measurement showed that the bridge acted according to calculations.

From the inspections carried out there are several observation that can be related to problems with post-tensioning.

During construction there was two problems with post-tensioning. One was with cracks near cable anchorages. This is also commented in the inspection. See below.

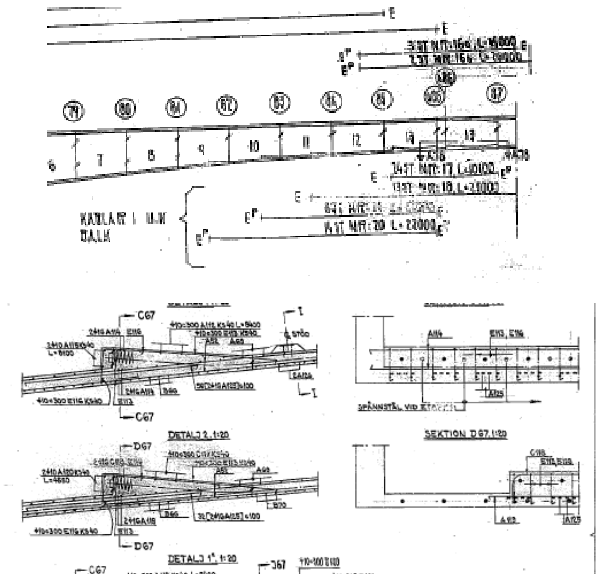
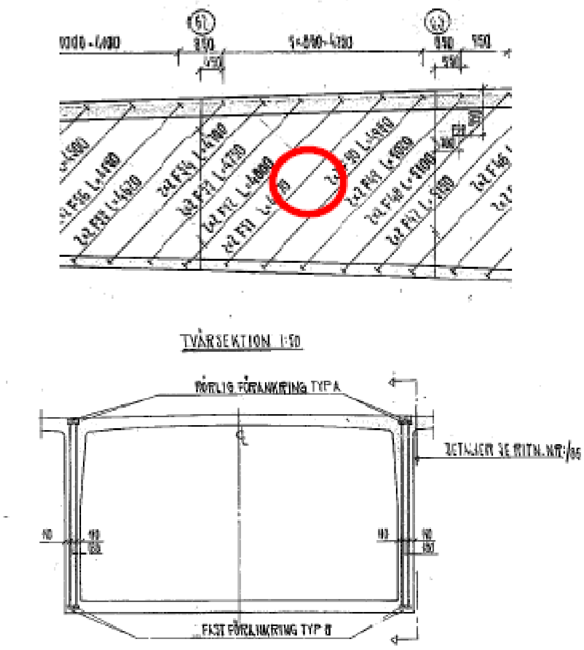
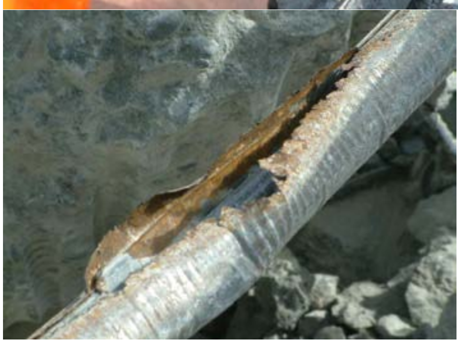


Figure A6 From drawings (BaTMan 2018)

The second problem was that grouting of the Dywidag post-tensioning using the thin ducts used in the webs could not be carried out in winter time. Therefore inleaking water expanded due to freezing and created internal cracks on several places. The cracks was repaired with epoxy and samples was bored out to prove that adhesion was sufficient. See below in red.

| | |
|---|---|
| | <br><br>Figure A7 From drawings (BaTMan 2018)<br>Next inspection is proposed to take place in 2019. On this occasion do an extended inspection also including cracks on critical parts. |

| | |
|---|---|
| **Bridge/Location** | A railway bridge over E18, Enköping opened in 1977 km 75+007 |
| **Type** | Trough bridge |
| **Length/Spans** | 73 meters. The four spans are 14 + 22.5 + 22.5 + 14 meters. All cables was stressed at the same time. |
| **Dimensioned according to** | VV Bronormer 1976 |
| **Post-Tensioned system** | BBRV 44ø6 with steel quality 155/180 kp/mm$^2$ was used. It had a break load of 218 Mp. Cable ducts 67/60 according to specification for tendons. There was severe problems with friction so some cables had to be replaced |
| **Designer** | ELU Konsult AB |
| **Contractor** | SIAB |
| **Assessment information** | According to inspection 2014 there are several cracks that can related to the post-tensioning system. |

| | |
|---|---|
| **Bridge/Location** | **A railway bridge in Örnsköldsvik opened in 1955 and demolished in 2005** |
| **Type** | **Most probably it was a trough bridge** |
| **Length/Spans** | Not known |
| **Dimensioned according to** | Not known |
| **Post-Tensioned system** | Freyssinet post-tensioning system 12ø7 vires. |
| **Designer** | Not known |
| **Contractor** | Not known |
| **Assessment information** | A special investigation was carried out by the Swedish Corrosion Institute KIMAB on 6 vires. 3 had small corrosion and tree had no corrosion.   Pictures from the bridge showing cables (KIMAB 2006) From the visual inspection the vires were found not corroded. When looking more closely by experts corrosion was found. Therefore 6 vires was chosen and tested. The figure below clearly shows that the vires with corrosion have a brittle fracture while the other have ductile fracture. This behaviour corresponds well to Mechanical Behaviour Dowling 1999. |

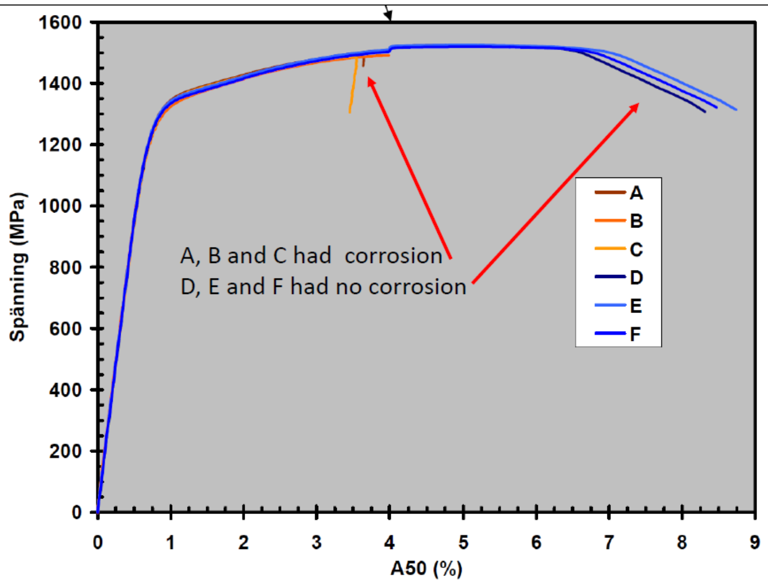



Figure A8 result of tests done by KIMAB 2006

| | |
|---|---|
| | Also content of chlorides was measured and showed high values. See more (KIMAB 2006)<br>Today too few corrosion tests on post-tensioned cables is carried out. Often non experts on corrosion take wrong decisions and underestimates the problem with only visual inspection. Experts on corrosion must therefore be more engaged. |

# Appendix B: Monitoring Techniques

## B.1 Manual Methods

### B.1.1 Visual inspection (VI)

Visual inspection is typically applied to detect contamination, material loss, deterioration, displacements and cracks. VI can be easily applied by inspectors during regular (normal) inspections and is often combined with simple NDT methods such as e.g. cover meter. This method is limited in the sense that it solely provides surface observations and a crack measurement accuracy limited to 0.1 mm. Furthermore, this method is very dependent on the experience and knowledge from the observer. The results from the inspection is also dependent on the accessibility to the structure, and the weather and light conditions. Nevertheless, in general a visual inspection can be fast and economical and to our experience all assessments always start with a visual inspection before more advanced methods is introduced.

## B.2 Acoustic methods

### B.2.1 Impact-Echo (IE)

The IE method involves hitting the surface of the area of interest with a small impactor or impulse hammer and identifying the reflected wave energy with a displacement or accelerometer receiver mounted on the surface near the impact point. The reflections of the stress wave from internal defects, material interfaces, or other anomalies are captured by transducers on the testing surface. Because the impact generates a high energy pulse and can penetrate deep into concrete, the IE method is particularly promising for identifying defects in concrete structures. It produces a better signal to noise ratio than other ultrasonic techniques because of its low attenuation in composite materials such as concrete.In figure B.1, a transducer records the surface displacements caused by multiple reflections of the waves versus time. These displacement signals are subsequently transformed into the frequency domain.

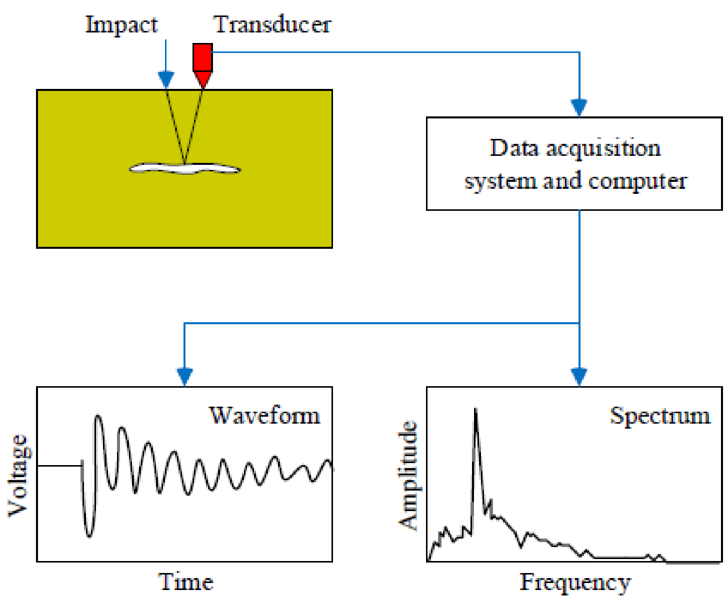



Figure B.1 Simplified diagram of the IE-method (from www.ndt.net)

Dominant frequencies are assigned to depth values by applying the so-called IE formula, whereby the wave speed must be determined for each concrete through calibration at a position of known thickness or by measuring on a core.

(6.1) $d = \frac{v_L}{2f}$

where $v_L$ is the P-wave velocity and $f$ is the measured frequency.

Minimum detectable target size varies according to the depth of the target. It is a very effective test method for a depth from 0.1 m up to about 1.2 m. This method is typically used for thickness determination, localisation of delamination, voids, inhomogeneities, as well as hollows in tendon ducts. In table B.1 capabilities and limitations of the IE technique is summarised.

Table B.1 Capabilities and limitations of the IE technique

| | |
|---|---|
| Defects | IE cannot locate strand defects in internal or external ducts. IE can locate voids and water infiltration in internal metal ducts with moderate accuracy, and compromised grout and water infiltration in internal nonmetal ducts with low to moderate accuracy. IE can also locate compromised grout, voids, and water infiltration in external HDPE (high-density polyethylene) ducts with moderate accuracy |
| Duct location | Applicable to both internal and external ducts |
| Duct type | Applicable to both metal and nonmetal ducts |
| Effect of concrete cover | The effect of concrete cover is dependent on the impact. Very thick concrete cover may prevent successful measurement. |
| Effect of layered ducts | Layered ducts do not yield meaningful results due to the large reflections from the near duct |
| Effect of reinforcement congestion | Presence of steel highly reflects acoustic waves, thereby negatively affecting investigation using IE. |
| Effect on corrosion | No possibility to detect corrosion |
| Accessibility requirement | For non-automated scanning systems, accessibility required is typically a 0.5 x 0.5  area. The area required for an automated scanning system is dependent on the system. Testing within the anchorage region is generally not possible using IE due to the physical structure of the region and the highly reflective metal used in the anchorages |

### B.2.2 Ultrasonic Pulse Echo (UPE)

This acoustical technique consists of the transmission (T) of ultrasonic-pulses into concrete which are reflected by material defects or by interfaces between regions of different densities and/or elastic moduli. A receiver (R) coupled to the surface monitors the reflected waves. Point measurements are combined to visualise the reflection. It is worth noting that the propagation of ultrasonic waves is limited by layers containing air, e.g. concrete with large amount of air pores and by very dense reinforcing bars. This method is used for the inspection of the inner structure of structural elements made of reinforced and prestressed concrete, rebar and tendon locations, compaction faults and voids, see a principal sketch in Figure B.2. In the UPE method it has to be noted that a single measurement allows no conclusion about the position of a single rebar or duct. Only measurements along a measurement grid with a constant measuring point distance allow carrying out a reconstruction calculation with a subsequent imaging of individual reinforcement bars or ducts. Compared to the radar method, the resolution here is often coarse due to the diffusion of signals at the aggregate. In table B.2 capabilities and limitations of the IE technique is summarised.

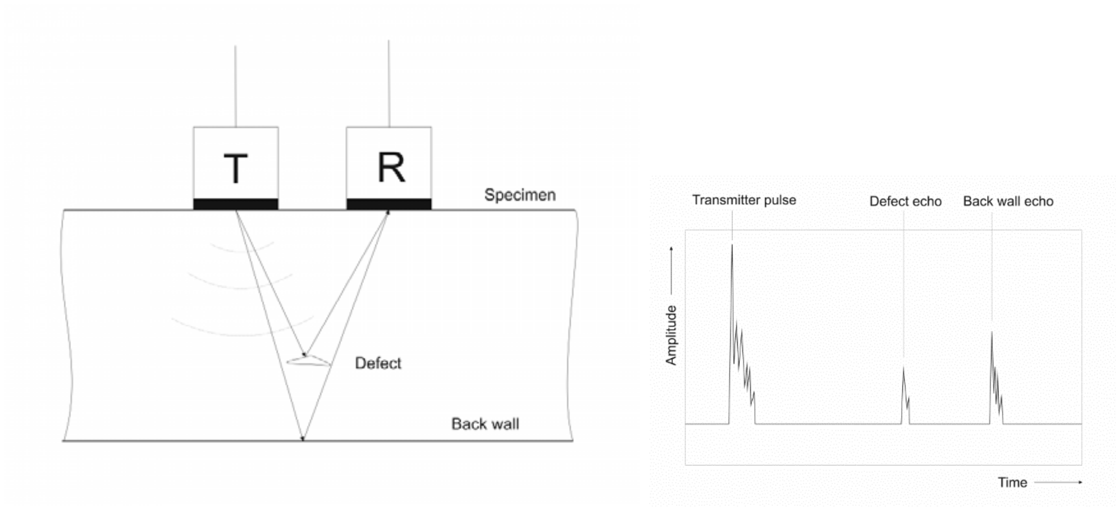



Figure B.2 Simplified diagram of the UPE-method (from www.ndt.net, zfp.cbm.bgu.tum.de)

Table B.2 Capabilities and limitations of the UPE technique

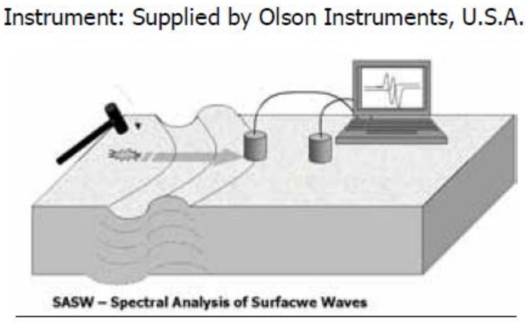
| | |
|---|---|
| Defects | UPE can locate grout defects in internal non-metal and metal ducts with low to moderate accuracy. UPE does not detect strand or grout defects within the ducts in the anchorage regions. |
| Duct location | Applicable to internal ducts. |
| Duct type | This method is applicable to both metal and non-metal internal ducts. |
| Effect of concrete cover | Typical concrete cover is not an issue for UPE inspection. |
| Effect of layered ducts | Ducts behind other ducts can be discerned using UPE. The position of the two transducers can be varied such that direct, semi-direct, and indirect tests can be performed, which aids in mapping out the volume of the defect. |
| Effect of reinforcement congestion | Surrounding reinforcement will strongly affect any investigation since the presence of steel highly reflects acoustic waves. This makes any object directly beneath the reinforcement undiscernible and areas in between reinforcement visible. Densely spaced reinforcement will therefore hide any investigation beyond the bars' location |
| Effect on corrosion | No possible to detect corrosion |
| Accessibility requirement | For UPE devices, the area required for scanning is about 300 mm. on both sides of the structure. UPE technique can be used for anchorage defects. Conventional UPE testing requires access to two surfaces, preferably two parallel surfaces such as the top and bottom surfaces of a slab or the inside and outside surfaces of a wall. However, this test can be performed using the indirect methods, which does not require access to two surfaces. |

### B. 2.3 Spectral Analysis of Surface Waves (SASW)

As with Impact Echo, the mechanical energy (in this case the Rayleigh wave) is generated by striking the concrete surface with, a small hammer or similar impactor. The propagation of the Rayleigh (surface) waves can be measured at the surface and the velocity of each wavelength component calculated as the waves pass the two sensors on the surface. In this way, it is possible to create a diagram of Rayleigh wave velocity versus wavelength. Since the waves are influenced by material at depths proportional to wavelength (short wavelength: near-surface and long wavelength: far-surface) then we can create a diagram of the variation in wave velocity with depth - a so-called dispersion curve.

The equipment and testing set-up are very similar to that used for Impact Echo. The collection of SASW data and processing is however more lengthy. Two transducers are used with a maximum separation approximately equal to the thickness of the concrete under investigation. The use of two transducers in this way does of course restrict tests to sufficiently large and accessible surfaces.

This method is suitable for relatively large planar surfaces. Also the type of investigation is usually that of layered systems or material (ground, soil, concrete) variations with depth. The ability of the method to detect and describe relatively small defects/objects is not so good as Impact Echo or Ultrasonic Pulse Echo. It has the advantage that it can quite accurately measure thickness without the need for calibration, i.e. wave velocity calibration.



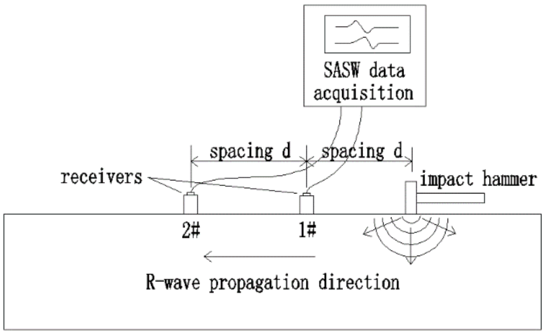




Figure B.3 Simplified diagram of the SASW-method (from www.mdpi.com)

Table B.3 Capabilities and limitations of the SASW technique

| | |
|---|---|
| Defects | SASW can locate grout defects in internal nonmetal and metal ducts with low to moderate accuracy. USE does not detect strand defects in internal ducts. This method has low accuracy in detecting voids and water infiltration in anchorage regions |
| Duct location | Applicable to internal ducts. |
| Duct type | As long as sufficient bonding between the duct lining and surrounding grout is maintained (no shrinkage cracks or air gaps present), this method is applicable to metal and nonmetal internal ducts. However, it requires calibration based on the duct material. |
| Effect of concrete cover | SASW devices perform better when the concrete cover is between 50 – 300 mm. Deeper cover may be acceptable provided there is no heavy reinforcement congestion. |
| Effect of layered ducts | Ducts behind other ducts cannot be discerned |
| Effect of reinforcement congestion | Surrounding reinforcement will strongly affect investigation since the presence of steel highly reflects acoustic waves. This makes any object directly beneath the reinforcement undiscernible and areas in between reinforcement visible. Densely spaced reinforcement will limit investigation beyond the location of the reinforcement bars. |
| Effect on corrosion | No possible to detect corrosion |
| Accessibility requirement | Automated scanners with dual probe transducers require approximately 0,5 x 0,5 m clearance around the inspected region. Testing within the anchorage regions generally does not provide useful information due to the large amount of reflective steel used in the anchorages. |

### B.2.4 Acoustic Emission (AE)

Acoustic Emission (AE) refers to the generation of transient elastic waves produced by a sudden redistribution of stress in a material. When a structure is subjected to an external stimulus (change in pressure, load, or temperature), localized sources trigger the release of energy, in the form of stress waves, which propagate to the surface and are recorded by sensors, see figure B.4.

This acoustical test method involves a series of single sensors (minimum of 4) or an array of sensors attached to the surface. Ultrasonic signals which are released by cracking are recorded. Information, such as noise amplitude, energy, duration, and crack type (cracking, delamination, spalling) can be captured. Active cracks can be identified and localised, before their effect is measurable. The ultrasonic signal changes the runtime with increasing deterioration of the concrete. There are no signals when cracks are not active. Filtering of noise due to traffic, existing cracks, etc. is necessary.

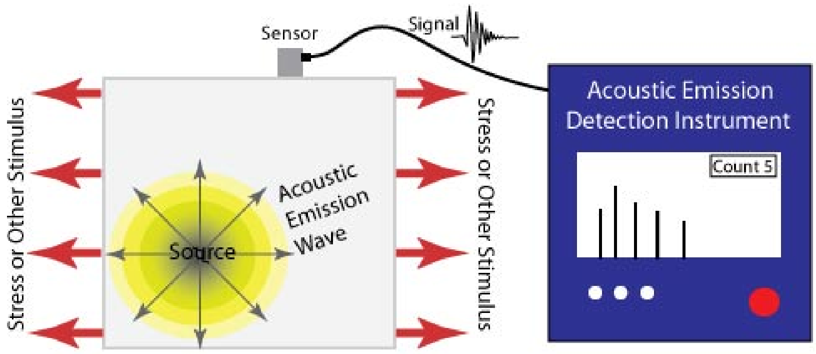



Figure B.4 Simplified function of the AE-method (from www.nde-ed.org)

Acoustic Emission is unlike most other nondestructive testing (NDT) techniques in two regards. The first difference pertains to the origin of the signal. Instead of supplying energy to the object under examination, AE simply listens for the energy released by the object. AE tests are often performed on structures while in operation, as this provides adequate loading for propagating defects and triggering acoustic emissions.

The second difference is that AE deals with dynamic processes, or changes, in a material. This is particularly meaningful because only active features (e.g. crack growth) are highlighted. The ability to discern between developing and stagnant defects is significant. However, it is possible for flaws to go undetected altogether if the loading is not high enough to cause an acoustic event. Furthermore, AE testing usually provides an immediate indication relating to the strength or risk of failure of a component. Other advantages of AET include fast and complete volumetric inspection using multiple sensors, permanent sensor mounting for process control, and no need to disassemble and clean a specimen.

Unfortunately, AE systems can only qualitatively gauge how much damage is contained in a structure. In order to obtain quantitative results about size, depth, and overall acceptability of a part, other NDT methods (often ultrasonic testing) are necessary.

Table B.4 Capabilities and limitations of the AE technique

| | |
|---|---|
| Defects | AE cannot discern static defect, but can be used during loading to detect ongoing defects |
| Duct location | Yes |
| Duct type | N.A. |
| Effect of concrete cover | N.A |
| Effect of layered ducts | N.A |
| Effect of reinforcement congestion | N.A |
| Effect on corrosion | Cannot detect corrosion, but should be able to detect breakage due to corrosion over time |
| Accessibility requirement | Need accessibility for the sensors, and many sensors might be needed for large structures. |

## B. 3 Radiographic methods

### B.3.1 Radiography

Radiography, or x-ray as it is commonly known, enables us to produce a 2-D image of the concrete and variations in the density, for example, those caused by reinforcing bars or voids. The technology used today enables us to produce extremely high-resolution digital images of reinforced concrete up to 1500 mm thick. The x-rays penetrate the concrete and are attenuated by the material to a degree that is dependant on the density and thickness of the object. The amount of radiation that penetrates the object will determine the brightness/contrast (darkness) of the image. A reinforced concrete structure will produce an image, which reflects the variations in density in the volume tested, so that rebars appear as lighter (less dense) images on a darker surface (providing that the concrete is homogeneous). If the concrete contains pores or voids then these will appear as darker spots/areas on the image.

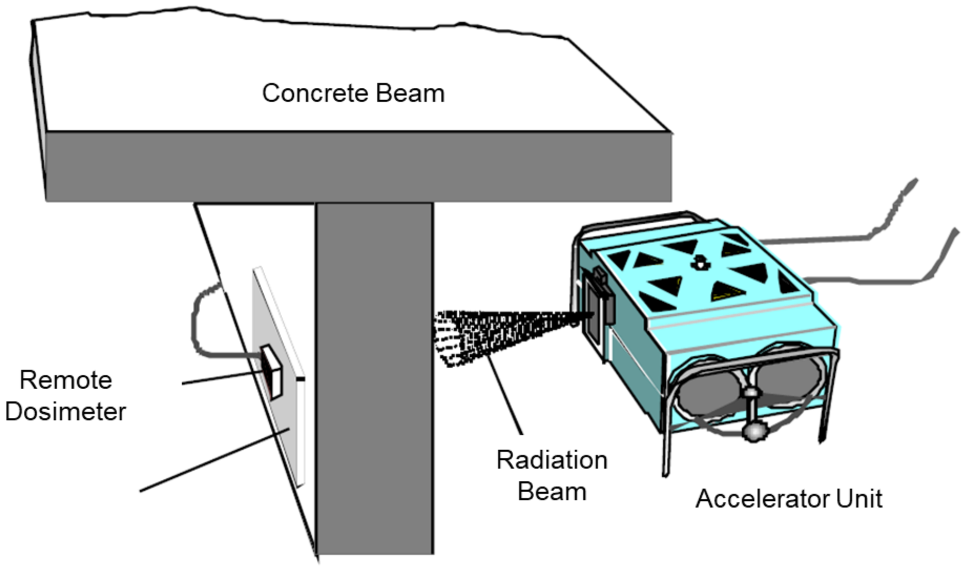
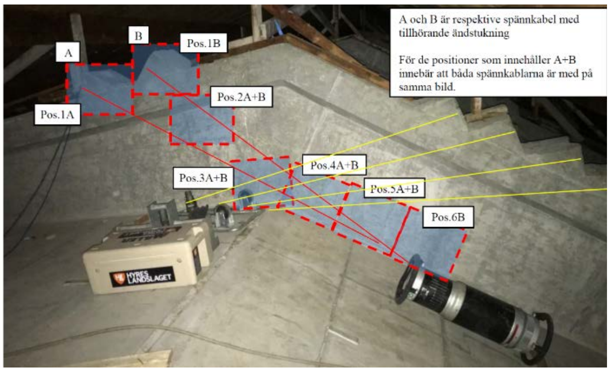



Figure B.5 Typical set up for radiography with a Betatron. The power and control units are separate. A 220 V, 15 A power supply is needed. The supply should be steady and not fluctuating, for example, due to the effects of welding or movement of heavy electrical machinery.

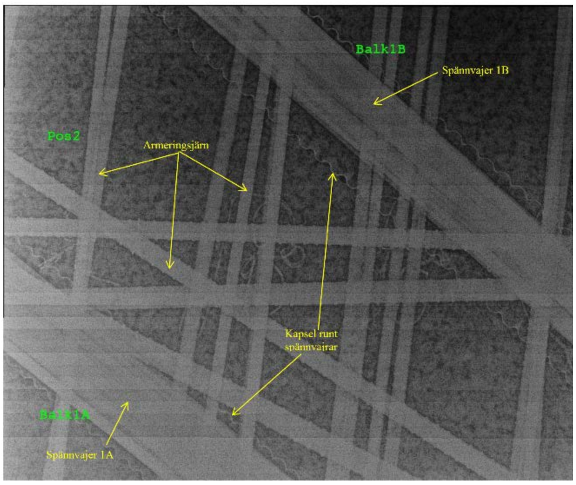
In order to define the optimum procedure for a inspection using radiography on concrete structures a number of parameters have to be decided:

- The necessary energy level or the source
- The choice of the right image plate
- The choice of the right screens for the image plate
- The necessary exposure time

In figure B.6 typical results from using the radiography method is shown. The ducts and the tendons can clearly been seen and in table B.5 the capabilities and limitations of the method is presented.




Equipment and different positions, by curtsey of DEKRA, 2019




Results. The duct and the tendons together with steel reinforcement can clearly be noticed in the photo, by curtsey of DEKRA, 2019

Figure B.6 Example of results from using the radiography method.

Table B.5 Capabilities and limitations of the Radiography technique

| | |
|---|---|
| Defects | Radiography can detect defects in concrete, but high moisture content increase the scatter and the exposure time as much as three. It is easier to detect a void of a certain size in a homogeneous concrete with small aggregates, than in a non-homogeneous concrete with large aggregates. |
| Duct location | Yes |
| Duct type | Yes, in such a way that the metallic duct can clearly been seen, but plastics will not be discern clearly on the screen. |
| Effect of concrete cover | The method is affected by the concrete cover, where thicker concrete covers demand higher energies |
| Effect of layered ducts | Ducts behind other ducts can be discerned using radiography. The position of the equipment can be varied such that direct, semi-direct, and indirect tests can be performed. |
| Effect of reinforcement congestion | Congested reinforcing can make it difficult to distinguish a void in an image. This is of course due to the higher density ratio between an air-filled void and the concrete/steel combination. A void of a certain size is easier to detect in un-reinforced concrete than in heavily reinforced concrete. However, the different layers of reinforcement can be noticed. |
| Effect on corrosion | Can detect corrosion, but has to be extensive corroded areas. Can give notice for possible corrosion at areas of voids etc. |
| Accessibility requirement | Heavy and complicated equipment, not suitable everywhere, need access to high voltage. Need special permits and protection due to X-rays. Most likely applicable for special projects. |

## B.4 Electromagnetic methods

### B.4.1 Ground Penetrating Radar (GPR)

This method applies electromagnetic waves by sliding an antenna over the concrete surface. It is important to note that if variation in the dielectric properties of the different materials is low, only a small amount of energy will be reflected. For example, electromagnetic waves cannot penetrate any metallic layer. The shape of the constructional elements (e.g. diameter of rebars) or material inhomogeneities are difficult or not at all possible to estimate. This method is often used for the inspection of the inner structure of structural elements made of reinforced or post-tensioned concrete and masonry, to detect and localise inhomogeneities (voids, metal or wood inclusion), thickness of structures which are only accessible from one side, internal structure of complex elements, as well as to determine the moisture content and distribution. In figure B.7 an example of the use of GPR is shown. The evaluation of the results is very dependent on the existing algorithms and user interface. The capabilities and limitation of the GPR inspection technique is discussed in table B.6.

Table B.6 Capabilities and limitations of the GPR technique

| | |
|---|---|
| Defects | GPR can detect defects in concrete. GPR cannot detect strand defects in external HDPE ducts. It can detect voids in external HDPE ducts with moderate accuracy, however cannot quantify the volume of the void. It can also detect compromised grout, and water infiltration defects in external HDPE ducts with low accuracy. GPR cannot detect strand or grout defects in external metal ducts, within the internal metal or plastic ducts, or the anchorage zones. However, GPR can be used to locate internal metal and plastic ducts |
| Duct location | While acceptable predominantly for internal ducts (testing on a concrete surface), with a proper setup, GPR may be used to identify voids within external ducts |
| Duct type | If it is desirable to detect conditions within the duct, GPR is only applicable to nonmetallic ducts. However, if it is desirable to locate the internal ducts, then GPR is applicable to both metal and nonmetal ducts. |
| Effect of concrete cover | The effect of concrete cover is dependent on the scanning frequency. For high frequencies (~500–3000 MHz) penetration depth can typically exceed 600 mm |
| Effect of layered ducts | GPR is unable to even locate the far duct due to the large reflections from the steel strands in the near duct. |
| Effect of reinforcement congestion | The presence of steel highly reflects the electromagnetic waves, thereby strongly affecting GPR’s capability of locating ducts, especially in the anchorage regions. |
| Effect on corrosion | Can detect corrosion, but have to have larger areas, large uncertainty |
| Accessibility requirement | The area required for GPR scanning is device dependent. For ground-coupled GPR inspection, it is required that the wheels of the device be in physical contact with the structure to ensure turning of the wheels which also acts as a distance meter. The creation of a 3D image requires either a 0,5 x 0, 5 m or 0,5 x 1,2 m manually accessible testing surface. However, these requirements could vary if an air-coupled GPR device is used. Testing within the anchorage region typically does not provide useful information due to the large volume of the highly reflective reinforcement cage present in the anchorage zones. |

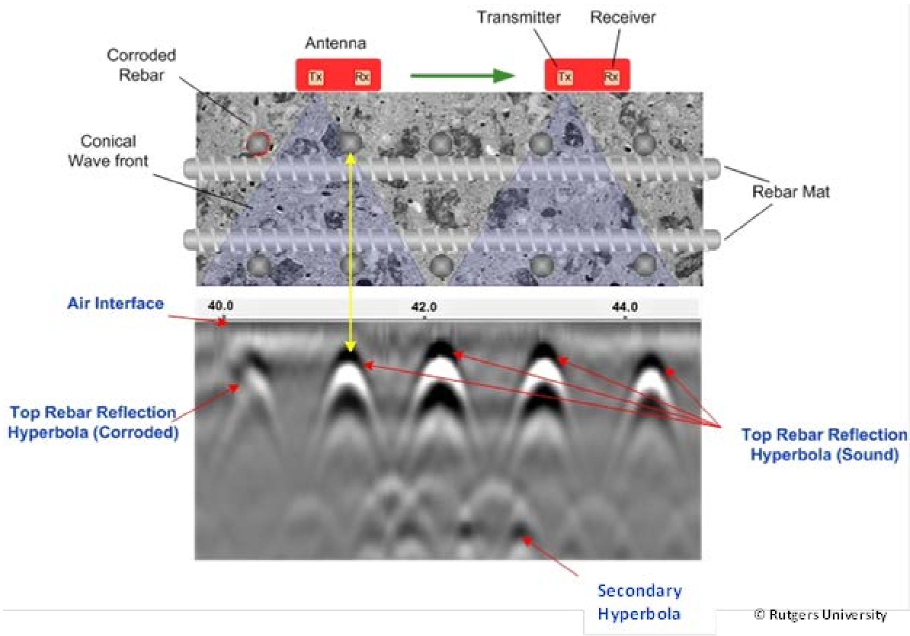



Figure B.7 Ground penetrating radar, from (https://fhwaapps.fhwa.dot.gov/ndep/DisplayTechnology.aspx?tech_id=25, 2019)

### B.4.2 Infrared Tomography (IRT)

In this approach, electromagnetic pulse is sent from one side and received on the other. Travel times and amplitude information are used to reconstruct the hidden structure and to provide velocity and attenuation distribution. The technique is used for poorly compacted concrete, high moisture and chloride content, voids greater than 100 mm. With proper calibration, IRT is used to quantify the dynamic modulus of elasticity which can be used to quantify frost damage in RC structures. The primary basis for IRT inspection lies in the emissivity of individual materials within the object being examined. Depending on the emissivity of the different materials, each material within the object may release or absorb heat at different rates, and the differential temperatures during this transition period can provide valuable information about the object. The uneven cooling or heating of the metal or nonmetal ducts, the surrounding concrete, the good grout, and the various defects, should be identifiable in a temperature profile. Therefore, it is important to perform IRT during times of the day when atmospheric temperature gradients are high, thus forcing the object being inspected to heat or cool in order to reach equilibrium with the surrounding environment. The capabilities and limitations of the IRT inspection technique are discussed in table B.7.

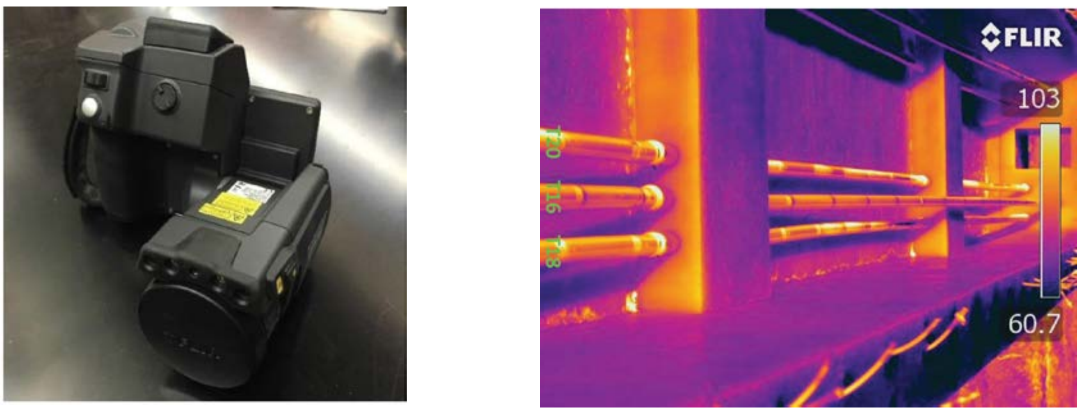



Figure B.8 IRT Technique using FLIR T640 Infrared Camera and sample of infrared image of external tendons, from (http://nap.edu/24779)

Table B.7 Capabilities and limitations of the IRT technique, from (http://nap.edu/24779)

| | |
|---|---|
| Defects | IRT cannot locate strand defects in external HDPE ducts. IRT has high accuracy in locating void and water infiltration defects and low accuracy in detecting compromised grout in external HDPE ducts. However, it cannot differentiate between these defects. It is also possible to make rough estimates on the size of the void and water infiltration defects. IRT does not detect strand or grout defects in external metal ducts, and also does not locate internal metal or plastic ducts if they are buried deep within concrete, let alone identify defects. While IRT cannot be used to locate defects within the ducts embedded in the anchorage zones, it can detect the void and water infiltration defects in the end caps of the anchorage regions with moderate to high accuracy. As in the case of external HDPE ducts, IRT cannot differentiate between void and water infiltration defects, and it is also possible to make rough estimates on the size of these defects in the end caps |
| Duct location | IRT is mainly applicable to external ducts. Applicability of IRT to internal ducts largely depends on the depth within concrete where the ducts are located. |
| Duct type | Applicable only to nonmetallic ducts. |
| Effect of concrete cover | The effect of concrete cover has a significant effect for investigating and even locating internal ducts. |
| Effect of layered ducts | Investigation of layered ducts is not possible with IRT. |
| Effect of reinforcement congestion | It is expected that surrounding reinforcement will strongly affect any investigation using IRT. |
| Effect on corrosion | N.A |
| Accessibility requirement | The only requirement for IRT is the ability for the infrared camera to have a good field of view of the subject being inspected. It is also important to avoid any uneven heating in the region under investigation. |

### B.4.3 Cover meter (CM)

A low frequency magnetic field is applied on the surface of the structure; the presence of embedded reinforcement alters this field, and a measurement of this change provides information on the reinforcement. Covermeter surveys form part of most concrete condition surveys of buildings or structures. Covermeter surveys can also locate main and secondary reinforcement to determine bar sizes, bar spacing, to determine minimum cover and cover variability across an element. The position of reinforcing steel and pre-stressing strands is sometimes also required to avoid them during core sampling or other tests which may be affected by their presence (concrete resistivity).

Figure B.9 Use of cover meter, from (http:// www.proceq.com)

Table B.8 Capabilities and limitations of the cover meter technique

| | |
|---|---|
| Defects | CM cannot detect defects in concrete |
| Duct location | CM can detect duct location depending on distance to duct |
| Duct type | If it is desirable to detect conditions within the duct, GPR is only applicable to nonmetallic ducts. However, if it is desirable to locate the internal ducts, then GPR is applicable to both metal and nonmetal ducts. |
| Effect of concrete cover | The concrete cover affects the accuracy of the method |
| Effect of layered ducts | CM is unable to even locate the far duct due to the large reflections from the steel strands in the near duct. |
| Effect of reinforcement congestion | In heavy congested reinforcement areas, the size and no. of reinforcement can be difficult do evaluate. |
| Effect on corrosion | Can detect corrosion, with additional equipment |
| Accessibility requirement | The equipment is normally hand held and will easily access most areas. However, for large areas the method can be time consuming. |

## B.5 Magnetic methods

### B.5.1 Magnetic flux leakage (MFL)

In the MFL inspection technique, a strong permanent magnet is used to directly magnetize the ferrous material (steel) within the ducts. This induces flux paths in the material between the two poles of the magnet. Where section loss is present, the magnetic field in the material “leaks” from its typical path of least resistance. A magnetic field detector (comprised of Halleffect sensors) between the poles of the magnet is sensitive to this change in magnetic field and indicates the leak. The capabilities and limitations of the MFL inspection technique are discussed in table B.9, (http://nap.edu/24779)

Table B.9 Capabilities and limitations of the MFL technique, (http://nap.edu/24779)

| | |
|---|---|
| Defects | MFL can locate strand defects in both metal and nonmetal external ducts with moderate to high accuracy. It consistently locates corrosion, section loss, and breakage with a loss in metallic area greater than 5%. However, in some cases loss in metallic area as low as 1% may also be detected. It can be used to estimate the loss of metallic area, although these estimates may not have high accuracy. MFL cannot detect grout defects in ducts. The effects from the magnetization of the metallic end pipe embedded within the anchorage zone, which is also called “end effect,” can make the interpretation of results challenging. |
| Duct location | Applicable mostly to external ducts. |
| Duct type | Applicable to both metal and nonmetal ducts. |
| Effect of concrete cover | N.A |
| Effect of layered ducts | N.A |
| Effect of reinforcement congestion | N.A |
| Effect on corrosion | The method cannot differentiate between corrosion, section loss, and breakage |
| Accessibility requirement | For the investigation of external ducts, a clearance of approximately 300 mm radius is required from the center of the duct. Testing within the anchorage region typically does not provide useful information due to the large volume of the highly reflective reinforcement cage present in the anchorage zones. |

### B.5.2 MMFM-Permanent magnet (MMFMP)

In this method, a magnetizer installed on the sensor head is guided along the free span of the ducts. The controller unit gathers the data and this data is transmitted to a personal laptop. The permanent magnet type measurements give the signal search coil measurements (direct signal from the search coil) and the magnetic flux (integrated signal of the search coil). Since the integrated signal of the search coil correlates with cross-sectional area of the cable, any valleys in these signals indicate a loss of metallic area. The permanent magnet also gives measurements from the Hall-effect sensors, which detect the MFL that supplements the detection of defects. The capabilities and limitations of the MMFM-permanent magnet inspection technique are discussed in table B.10.

Table B.10 Capabilities and limitations of the MMFMP technique, (http://nap.edu/24779)

| | |
|---|---|
| Defects | MMFM-permanent magnet can locate the strand defects in both metal and nonmetal external ducts with moderate to high accuracy. MMFM-permanent magnet consistently locates corrosion, section loss, and breakage with a loss in metallic area greater than 5%. However, in some cases loss in metallic area as low as 1.5% may also be detected. MMFM permanent magnet inspection can be used to obtain estimates in the loss of metallic area, although these estimates may not be accurate. MMFM-permanent magnet does not detect grout defects in ducts |
| Duct location | Applicable to external ducts. |
| Duct type | Applicable to both metal and nonmetal ducts.. |
| Effect of concrete cover | N.A |
| Effect of layered ducts | N.A |
| Effect of reinforcement congestion | N.A |
| Effect on corrosion | The method cannot differentiate between corrosion, section loss, and breakage |
| Accessibility requirement | For the investigation of external ducts, a clearance of approximately 300 mm radius is required from the center of the duct. |

### B.5.2 MMFM-Solenoid (MMFMS)

The solenoid type measurements are used to identify and quantify the loss of metallic area in external ducts. Electric current is passed through the wire that is wound around a drum that encases the tendon. This is then guided along the length of the free span of the external ducts. The controller unit gathers the data and this data is transmitted to a computer. These measurements, known as the scan measurements locate the metal defects in the external ducts. In regions of interest the solenoid may be held stationary at that location and point measurements may be made to quantify the defects in the tendons.

The capabilities and limitations of the MMFM-solenoid inspection technique are discussed table B.11 ((http://nap.edu/24779)

Table B.11 Capabilities and limitations of the MMFMS technique, (http://nap.edu/24779)

| | |
|---|---|
| Defects | MMFM-solenoid can locate the strand defects in both metal and non-metal external ducts with moderate to high accuracy. However, it cannot differentiate between corrosion, section loss, and breakage.. However, in some cases loss in metallic area as low as 1.5% may also be detected. MMFM-solenoid inspection can be used to obtain estimates in the loss of metallic area, although these estimates may not be accurate. MMFMsole-noid<br>does not detect grout defects in ducts. |
| Duct location | Applicable to external ducts. |
| Duct type | Applicable to both metal and nonmetal ducts.. |
| Effect of concrete cover | N.A |
| Effect of layered ducts | N.A |
| Effect of reinforcement congestion | N.A |
| Effect on corrosion | MMFM-solenoid consistently locates corrosion, section loss, and break-age with a loss in metallic area greater than 5% |
| Accessibility requirement | For the investigation of external ducts, a clearance of approximately 300 mm radius is required from the center of the duct. |

## B.6 Electrochemical methods

### B.6.1 Electrochemical impedance Spectroscopy (EIS)

EIS is an impedance technique that applies a low-amplitude voltage (alternating current) to the steel under inspection over a wide range of frequencies. By measuring the changes in phase shift and signal amplitude, the impedance of the concrete-steel interface can be calculated. As EIS inspection generates detailed information, sophisticated approaches are required to interpret the data and extract meaningful results. The capabilities and limitations of the EIS inspection technique are discussed in table B.12.

Table B.12 Capabilities and limitations of the EIS technology, (http://nap.edu/24779)

| | |
|---|---|
| Defects | EIS cannot detect grout defects. |
| Duct location | EIS is applicable for external ducts. |
| Duct type | It can be used with external HDPE or other non-conductive ducts.. |
| Effect of concrete cover | N.A |
| Effect of layered ducts | N.A |
| Effect of reinforcement congestion | N.A |
| Effect on corrosion | EIS inspection can identify corrosion in external HDPE ducts with moderate accuracy. |
| Accessibility requirement | EIS requires physical access to the duct that is being inspected, and the abil-ity to drill small holes into the external duct. The holes must be sealed after the EIS testning. |

## B.7 Intrusive methods

### B.7.1 Endoscope (ES)

This technique, borrowed from medical applications is a slightly destructive method. Holes (approximately 25 mm in diameter) are drilled. After meticulous cleaning of the hole from dust and loose material, the endoscope is introduced into the hole. The core of the endoscope, consisting of optical fibres, allows for direct observation of the walls of the hole. In addition, pictures can be taken at any depth of the hole.

Figure B.10 Use of endoscope identifying a crack in concrete, from (http:// www.nishimatsu.co.jp)

The endoscopes can be rigid (endoscope with metallic pole of variable length) or flexible. Depending on whether the light source is placed at the target extremity (halogen lamp) or at the endoscope's tip, it is possible to define it as warm light or cold light tool. The endoscope enables the connection to video and/or photographic devices for the characterisation of the masonry mass stratigraphy the state of resistant elements and mortar. A variant of this system is the endoscopic video made through a miniaturised camera directly connected to a device recording images. In table B.13 capabilites and limitations of the ES technology is presented.

Table B.13 Capabilities and limitations of the ES technology

| | |
|---|---|
| Defects | N.A |
| Duct location | N.A |
| Duct type | N.A |
| Effect of concrete cover | N.A |
| Effect of layered ducts | N.A |
| Effect of reinforcement congestion | N.A |
| Effect on corrosion | ES can verify possible corrosion if the area for corrosion is known. |
| Accessibility requirement | ES is easy to handle and to use, and can often be used as a complementary to other methods. |

### B.7.2 Residual Prestressing Force (RPF)

When assessing prestressed concrete bridges, it is essential to take the current condition of the prestressing system into account. For instance, the quality of reinforcement protection (e.g. grout), steel corrosion and residual prestress force are all aspects that are crucial and require special attention (SB-LRA 2007). The residual prestress force influences the structural response both at the service-load and ultimate-load levels. By preventing cracks or limiting their formation, prestressing also reduces environmental exposure and, consequently, has a favourable impact on structures in harsh environments. However, there are often many uncertainties associated with the residual prestress force, especially after a longer time in service and, therefore, it can be useful to calibrate theoretically-based methods using experimental data from the assessed structure (Bagge, 2017).

For this purpose, there is a range of methods that can be used, through both non-destructive and destructive tests. Examples of non-destructive and destructive methods are given in figure B.11. In fact, the non-destructive methods described often cause minor, local damage and the impact on the structure can be negligible if then repaired properly. The principles of these are briefly summarised below:

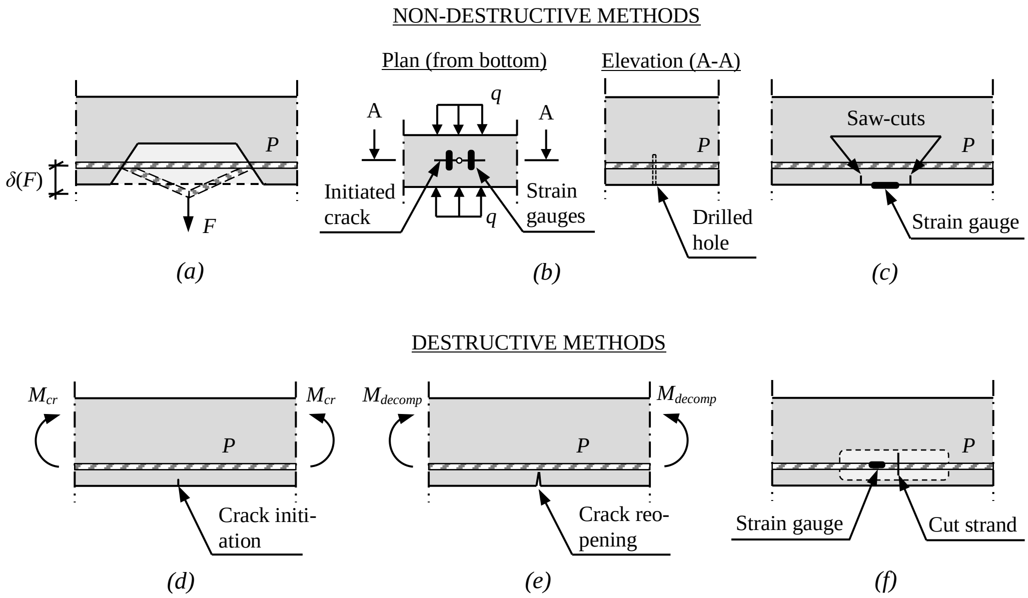
- **Exposed strand method:** The prestress force is derived based on calibrated data and the response of an exposed and laterally loaded strand.
- **Drilled hole method:** The prestress force is calculated from the development of strains measured around a hole drilled adjacent to the prestressed reinforcement.
- **Saw-cut method:** The prestress force is calculated from the response of a concrete block when introducing saw-cuts adjacent to the prestress reinforcement to isolate the block from the acting forces.
- **Crack moment method:** The prestress force is calculated based on the external load required to cause the first crack and the tensile properties of the concrete.
- **Decompression-load method:** The prestress force is calculated from the external load leading to reopening of existing cracks and, thus, eliminating the uncertainties associated with the tensile properties which are present in the crack moment method.
- **Strand-cutting method:** The prestress force is calculated from the development of strains measured when an exposed strand is cut.

When assessing existing bridges, the destructive methods are not always suitable and non-destructive methods are preferable. However, destructive methods can be valuable for examination of other methods when permanent damage to the structure is acceptable.

Saw-cut method

Due to its expected applicability to full-scale bridge members reinforced with post-tensioned tendons, the saw-cut method was further investigated in the underlying work to this thesis. The method was developed by Kukay (2008) and experimental studies examining such methods showed acceptably accurate estimates of the residual prestress force. However, the experiments were carried out on simply supported beams with pre-tensioned strands with constant eccentricity, both rectangular beams produced in the laboratory and I-shaped girders taken from a bridge under reconstruction (Kukay et al. 2010; Kukay 2008), and there have been no additional studies on more complex structural members.

The principle of the method is to measure the development of longitudinal strain at the surface (top or bottom) of a member when a block of concrete is isolated from the loads acting on it. The isolation is carried out gradually by introducing transverse saw-cuts on each side of the position of measured strains and the concrete block is regarded as isolated when increasing the depth of saw-cuts does not cause further changes in the strains at the measured surface. Based on the current action effects (i.e. associated with prestress force, restraint force, dead load and external applied load), Navier’s formula can be used to quantify the residual prestress force at the position of the isolated concrete block. The axial stresses in the section is given by Equation (B:1):




$$\sigma = \frac{P}{A} + \frac{Pe_P y}{I} + \frac{M_R y}{I} + \frac{M_G y}{I} + \frac{M_Q y}{I} \qquad \text{(B:1))}$$

Figure B.11: Methods to determine residual prestress force P in concrete members experimentally: (a) exposed strand method, (b) drilled hole method, (c) saw-cut method, (d) crack moment method, (e) decompression-load method, and (f) strand cutting method; redrawn from Bagge et al. (2017).

where $\sigma$ is the longitudinal concrete stress at the surface, $P$ is the prestress force, $A$ is the cross-sectional area, $e_P$ is the eccentricity of the prestress force, $y$ is the distance to the neutral axis from the monitored surface, $I$ is the cross-sectional second moment of inertia, $M_R$ is the secondary moments due to restraint forces, $M_G$ is the moment due to permanent loads and $M_Q$ is the moment due to variable loads. In the case of a statically indeterminate structure and the presence of a secondary moment, the procedure for determining the prestress force is iterative, in contrast to the original formulation of the method assuming a statically determinate structure (i.e. $M_R$ excluded).

The original version of the saw-cut method requires full isolation of the concrete block, which is not always possible when assessing existing structures. For instance, non-prestressed reinforcement can be located too close to the concrete surface, thus limiting the possible depth of the saw-cuts to avoid permanent damage. In such situations the experimental test can be simulated using FE analysis. Thus, the residual prestress force can be quantified based on the response observed in the test, rather than the strain measured at full isolation. The development of the strains, as a function of the saw-cut depth, is followed and compared between the experiment and the simulation, while the modelled prestress force is iteratively updated until there is a consistent response. The saw-cutting can be simulated in the FE model by gradually removing FE elements corresponding to the saw-cuts in the experiments, see figure B.12. Therefore, using this method, it is possible to avoid damage to the structure which might be difficult to repair.

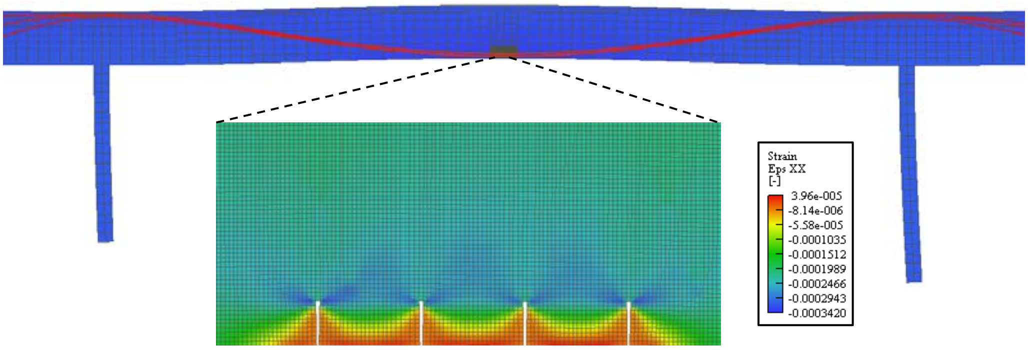



Figure B.12 A part of an FE model for simulation of the strain distribution as saw-cuts are introduced transversally at the base of a concrete beam (Bagge et al. 2017).

<u>Decompression-load method</u>

As with the saw-cut method, Navier's formula as shown in Equation (B:1) can be used to quantify the residual prestress force. By measuring either the opening of an existing crack or the concrete strains beside it, the load at which the crack reopens (i.e. no normal stress in the crack) can be determined. Initially, for loading when the crack remains closed, a linear and rather stiff load-displacement (or load-strain) response is present. As the crack reopens, the behaviour drastically changes and the stiffness reduces. This change in the response is used to identify the decompression-load for calculating the corresponding prestress force.

When using the decompression-load method, the depth of the compression zone of the cracked concrete is one parameter that has a significant impact on the outcome. At the same time, the compression depth of the actual section can be uncertain due to nonlinear strain distributions and the lack of reliable measurements, for example see (Nilimaa et. al., 2015)

### B.7.3 Potential Mapping (PM)

This electrochemical method is used for the determination of defects related to corrosion of reinforcement and chemical attack (sulphate, chloride and ASR). The corrosion potential is essentially measured as the potential difference (or voltage) against a reference electrode (half-cell). Measurements related to half-cell potentials are based on the electrical and electrolytic continuity between the rebar in concrete, reference electrode on the concrete surface and voltmeter, see figure B.13. Limitations of PM are the conductive cover between reinforcement and surface, membranes, asphalt or other sealing parts. In table B.13 capabilities and limitations for the PM technique is discussed.

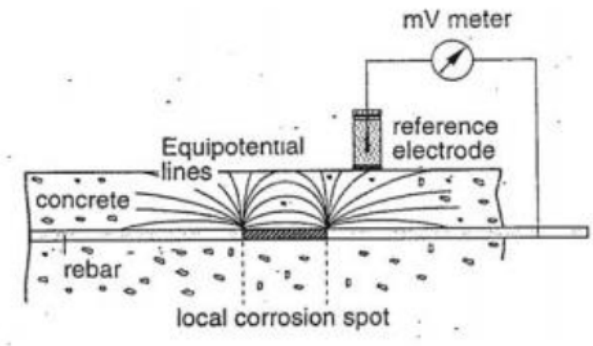



Figure B.13 Principle and main components of half-cell potential measurements. Reference electrode, high impedance voltmeter, connection to the rebar. Adopted from RILEM TC 154-EMC, (Elsner, 2003)

Table B.14 Capabilities and limitations of the GPR technique

| | |
|---|---|
| Defects | The PM method can not detect any defects in concrete or any voids in ducts. |
| Duct location | N.A |
| Duct type | N.A |
| Effect of concrete cover | N.A |
| Effect of layered ducts | N.A |
| Effect of reinforcement congestion | N:A |
| Effect on corrosion | Can detect corrosion |
| Accessibility requirement | This is a intrusive method and one has to be sure where to look for corrosion. If this is correctly identified the corrosion level could be detected. |

# Appendix C: Strengthening Techniques – Case Studies

## Box girder bridges

When designing a new bridge superstructure often internal tendons are chosen almost exclusively. There are some good examples for using both external tendon in box girder bridges where the external tendons are placed within the box. The purposes are mainly two. The first is because amount of cables and reinforcement does not allow good quality concreting. The second is extra reinforcement for future upgrading.

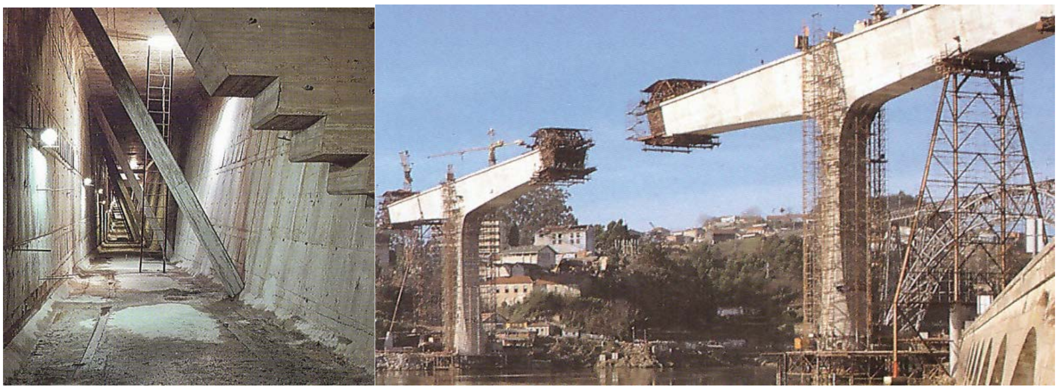


Figure C.5 Photos from the new railway bridge in Porto shows preparation for extra external post-tensioning, (Nova ponte ferroviaria sobre o Rio Douro no Porto)

Figure C.6 Photos from a French bridge on high speed line to Marseilles left and a Hungarian bridge on the new freight line through Slovenia to the Mediterranean (Photos by B. Paulsson)

From these examples in Figure C.5 and C.6 you can see that quite small efforts can make external post-tensioning feasible as a strengthening method for box girder bridges even if they are not prepared when built. Saddle at suitable points ore fastened like in figure C.4 can be done.

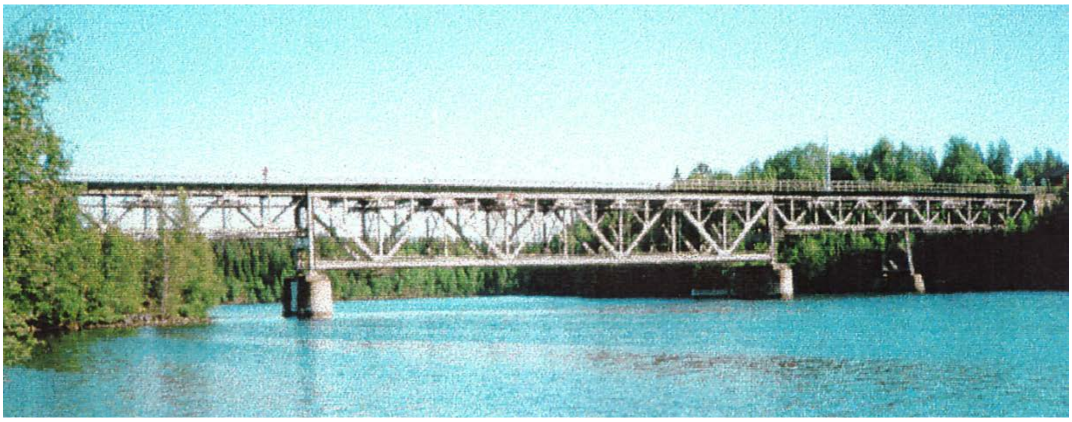
For strengthening purposes external post-tensioning for bridges is today well known after many years of experience.  There are also a great variety of tendon types so there are many possibilities to execute a strengthening of a bridge.

There are also since a long time good guidelines and comments how to use them since many years. (F Standfuss 1998)

***References***

Erläuterungen zur Richtlinie für Betongbrücken mit externen Spanngliedern. Friedrich Standfüss, Manfred Abel und Karl-Heinz Haveresch, Beton und Stahlbetonbau 1998

## Skellefte Älv



The two pictures above shows the bridge. (Björn Töyrä)

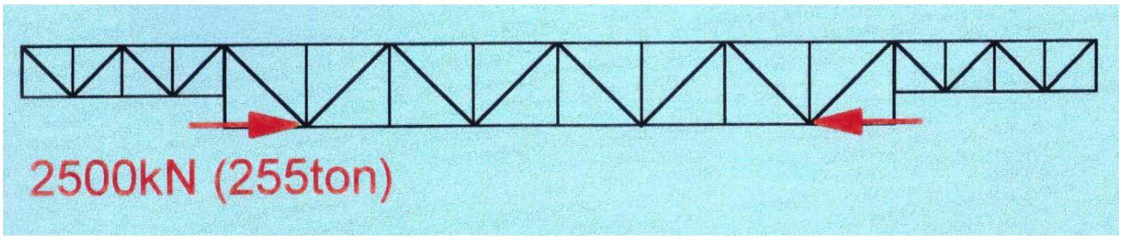



Figure AD,1 The figure shows how the forces was applied to the bridge.

In 1996 Skellefte Älv bridged was strengthened due to increase of axle load from 22.5 tonnes to 25 tonnes. The bridge is a riveted truss bridge built 1911. The bridge has tree spans 24m + 60m + 48 m and was originally dimensioned for an axleload of 8 tonnes.

The costs for a new bridge was estimated to 40 millon SEK and could not be motivated with the low traffic (7 crossing trains per day). Therefore an alternative was chosen namely strengthening. Three problems had to be solved namely;

- Martin steel, brittle and sensitive to cracks
- High compression of whole circumference
- Fatigue

In this case study only the post-tensioning will be described. In figure AD.1 you can see how the forces was applied on the structure.

Figure AD.2 shows how the compresses was applied in detail, installation of anchors on the bridge and how it looks like after injection and sealing.

By pressuring the under-frame rods, the risk for propagating cracks was reduced. The post-tensioning units was placed in the centre of the rod and stressed to between 71 to 255 tonnes depending on span. The tension force required to solve brittle fractures was only one tenth of forces mentioned above. The tensioning forces was don simultaneously on both sides in order to avoid sideway problems. In order to see the effects of the strengthening test loading was carried out with T44 locomotives. These locomotives were driven over the bridge at different speed. Also a static load was performed. Oscillation and displacements was measured at 30 chosen places. This was then analysed and shown that the strengthening performed well. Below see last inspection report. One worrying part is the 40 bolts that is missing.

**Skellefte älv, km 33+220**

Anl/Knr: **3500-2192-1 (1)** Ägare: **TRV UHj Byggnadsverk** Förvaltare: **TRV UHj Byggnadsverk Nord** Status: **Byggd / Nedsatt framkomligh**

IdNr2: **J2192** Vägnr: Bandel: **143 (Bastuträsk)-Skelleftehamns övre** Km tal: **33+220**

**Inspektionsprotokoll**

**Aktualitetsdatum 2019-01-10**

**Inspektionsförrättare** ..........................................................

**Inspektionsdatum** ..........................................................

**Inspektionstyp** ..........................................................

**Nästa inspektionsdatum** ..........................................................

**Nästa inspektionstyp** ..........................................................

| | | Skadekod | | | Mätning/Tillstånd | | | | Fiktiv åtgärd | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Nr | Konstr Element | Mtrl | Typ | Orsak | Metod | Värde | | Tk | Aktivitet | Mängd | | Apris kkr | Läge |
| 4 | 310 | 2 | 3 | 120 | 1n | 3,0 | % | 2 | 06 | 4,0 | m2 | 9,8 | Landfäste, Ö sidan. Lindrigare på väst |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 5 | 330 | 2 | 3 | 120 | | | | 2 | 06 | 30,0 | m2 | 7,7 | Bägge landfästen. |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 1 | 360 | 2 | 22 | 213 | 15b | 4,0 | mm | 3 | 45 | 15,0 | m | 3,8 | Alla |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 2 | 360 | 2 | 2 | 111 | 1n | 3,0 | % | 2 | 17 | 130,0 | m2 | 12,0 | Alla, skvalpzon |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 14 | 363 | 1 | 79 | 217 | | | | 0 | | | | | Stöd 3 |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 3 | 410 | 2 | 3 | 123 | | | | 2 | 06 | 80,0 | m2 | 7,7 | Bägge landfästen. |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 8 | 510 | 12 | 35 | 520 | 58f | 1,0 | - | 1 | 87 | 8,0 | st | 0,6 | Lager för landfäste, V sidan |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 9 | 513 | 12 | 6 | 100 | | | | 1 | 55 | 14,0 | st | 1,0 | Samtliga |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 10 | 650 | 12 | 6 | 100 | | | | 0 | 23 | 5,0 | m2 | 2,8 | Spalter och skarvar hela bron. |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 16 | 1205 | 20 | 79 | 600 | | | | 1 | 53 | 5,0 | m | 0,5 | Räcke i trä saknas på mittstöd. |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |
| 12 | 1206 | 40 | 6 | 100 | 40a | 2,0 | - | 1 | 55 | 270,0 | m2 | 1,4 | Hela räcket, ställvis. |
| ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ | ........ |

Conclusions from Skellefte Älv bridge;

- The strengthening worked well
- The cost for strengthening was just over 10% of cost for a new bridge
- Traffic disturbance was negligible since the work was done in timeslots that was acceptable for the traffic situation
- A beautiful old bridge could be kept for future generations.

## Strengthening of trough bridge in Haparanda by prestressing

The Haparanda Railway Bridge, Fig. D-x, was strengthened in the summer of 2012. The purpose of the strengthening project was to improve the load bearing capacity of the bridge to reflect an upgrade of the loads on the railway line. The maximum allowed axle load was increased from 250 to 300kN, and for this reason, the Haparanda Bridge required a higher transverse shear capacity of the slab. Assessment calculations indicated a 24% deficit in the shear capacity of the slab in the transverse direction (WSP Group 2008). The maximum shear capacity of the slab was 150kN/m, and the required shear capacity was 186kN/m. Thus a minimum shear capacity increase of 36kN/m was required. The strengthening method used to increase the capacity was internal post-tensioning and the design was performed according to Eurocode 2 (CEN 2008).

A horizontal strengthening system consisting of 8 unbonded post tensioning bars were installed internally at mid-height of the slab and post-tensioned in the transverse direction. The posttensioning not only increased the shear capacity of the slab by 25%; the flexural capacity was also increased by 13%, existing cracks were reduced in width and further cracking was postponed. The bridge was tested before and after strengthening, and the results showed that the prestressing completely counteracted the tensile strains in the main slab-reinforcement under a 215 kN/axle test-train load. The maximum tensile strains were however small, with magnitudes of only about 20 μm/m for the test-train (23 micro strain before strengthening and 20 micro strain after strengthening), which implied a higher original capacity of the bridge than calculated.


*Figure D-x The Haparanda Railway Bridge (3500-2033-1, km1309+609) was strengthened in the summer of 2012. Eight transverse prestressing bars were installed at mid-height of the slab (left insert) and posttensioned to upgrade the axle load capacity of the railway line from 25 to 30 ton.*

### *References*

Mainline D1.3 (2014). New technologies to extend the life of elderly rail infrastructure. Mainline Deliverable 1.4. 190 pp. http://www.mainline-project.eu/

Nilimaa, Jonny (2013): *Upgrading concrete bridges. Post-tensioning for higher loads*, Licentiate thesis: Luleå University of Technology. 302 pp

Nilimaa, J., Blanksvärd, T. & Täljsten, B. (2012): *Transversal post tensioning of RC trough bridges: laboratory tests*, Nordic Concrete Research. 46, 2, pp. 57-74.

Nilimaa, J., Blanksvärd, T., Täljsten, B. & Elfgren, L. (2014): *Unbonded TransversemPosttensioning of a Railway Bridge in Haparanda, Sweden*, Journal of Bridge Engineering. Vol 19, No 3, 11 pp.

Nilimaa, Jonny (2015). Concrete Bridges: Improved Load Capacity. PhD Thesis, Structural Engineering, Luleå University of Technology,180 pp, http://ltu.diva-portal.org/smash/get/diva2:999292/FULLTEXT01.pdf

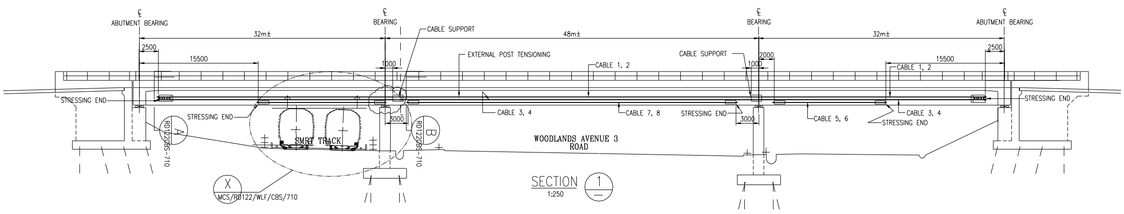
## External strengthening with post-tensioning – Singapore (SB Project)

| | | |
|---|---|---|
| Construction Period | Start: 2005 | End: 2007 |
| Client | Land Transport Authority (LTA), Singapore | |
| Designer | Maunsell/COWI | |
| Contractor | VSL Singapore | |

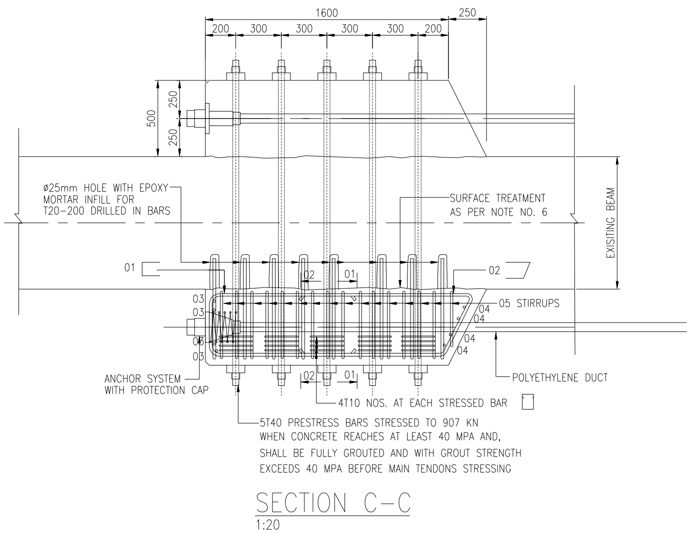
Strengthening of motorway bridge in Singapore: Woodlands  Flyover. A 3 span motorway bridge with continuous superstructure. The superstructure consists of two solid, prestressed main girders and a mild steel reinforced bridge deck. To meet the updated requirements stated by the bridge owner:

- to achieve no tensile stresses in SLS as the live load is increased by 20 % compared to the original design live load
- to achieve sufficient load bearing capacity as the live load is increased by 20 % compared to the original design live load

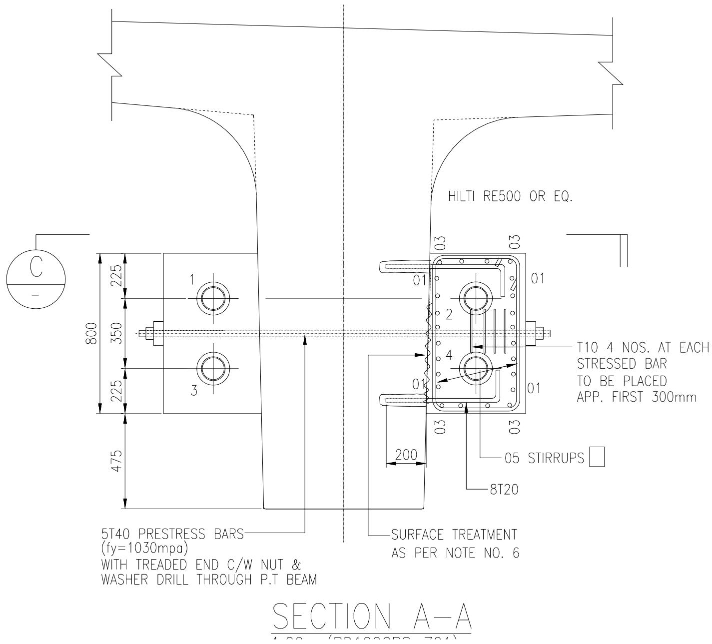
The longitudinal girders were strengthened. The stresses in SLS as well as the Load capacity in sections at mid piers as well as in mid spans were adjusted. The ordinary prestressing system VSL was used with ordinary prestressing cables, ducts, anchorages and prestressing equipment was used. For each girder up to six cables were provided and prestressed on each side of the girder. The amount of the additional prestressing was of the same magnitude  as the existing prestressing. Formal tensile stresses in the concrete of size 8 MPa was balanced by the external prestressing. The load bearing capacity in critical sections was increased by approx. 15 %. No traffic management needed as all construction works were above non traffic areas. The codes used was BS 5400 supplemented by the local (Land Transport Authority, Singapore) Bridge Design Criteria and VSL's normal guidelines for use of VSL prestressing systems. The alternatives were addition of a third main girder or using CFRP prestressed bonds at the underside of the main girders. Drawing and photos of the bridge are shown below.




Longitudinal section. 3 systems of external prestressing cables are provided




Anchor blocks. Plan section. Shear face with drilled in bars and with prestressed transverse bars




Anchor blocks. Cross section

Woodland flyover. 3 span bridge with 2 main girders and bridge deck slab.

2 continuous main girders seen from below. Before external prestressing


Anchor block for two cables.
Shear face is prestressed by 4 transverse bars

Prestressing of cable at top - "live end". "Dead end" for cable at bottom.