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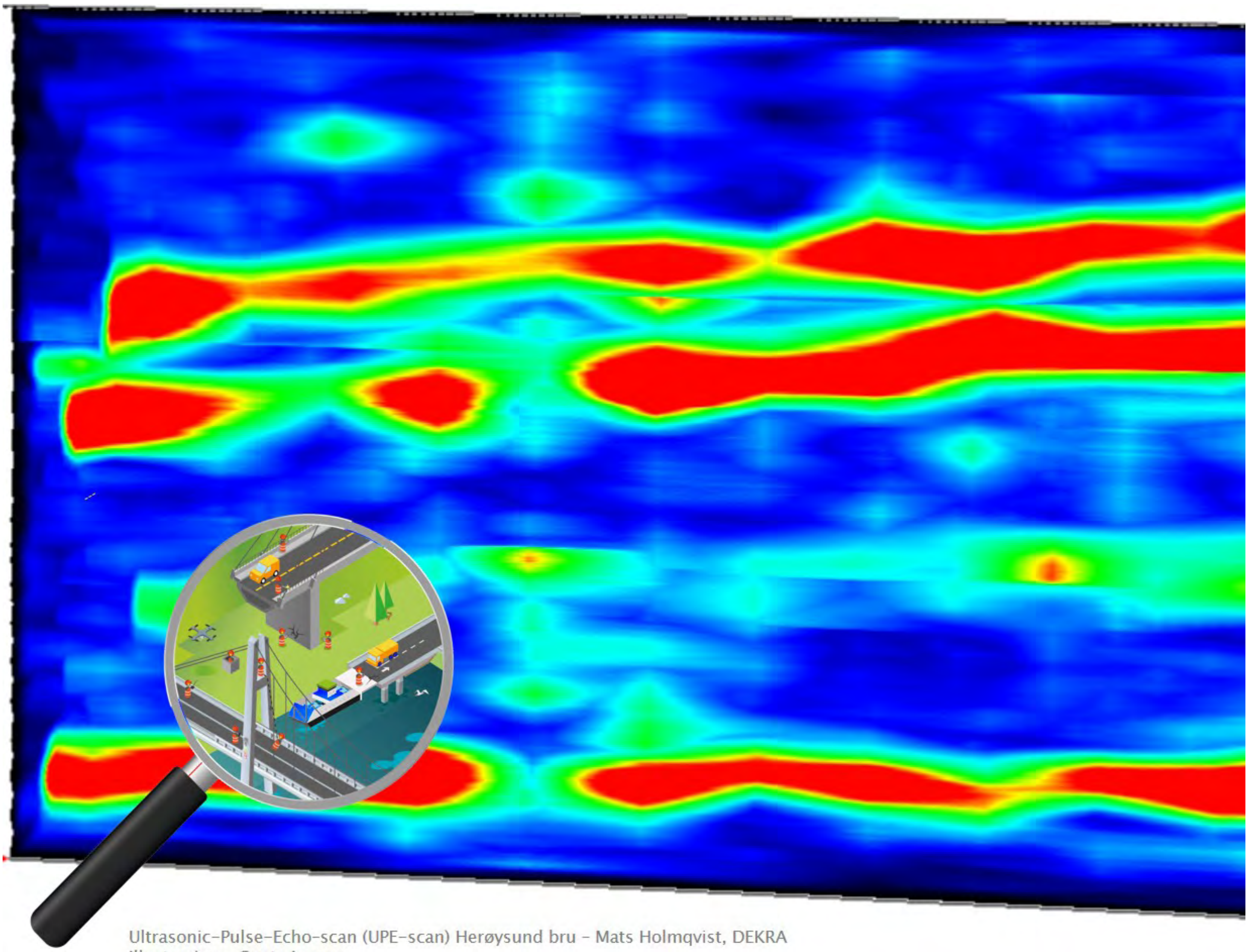
Statens vegvesen

Inspeksjon av etterspent armering i betongbruer

FoU-programmet Bedre Bruvedlikehold
2017-2021

STATENS VEGVESENS RAPPORTER Nr. 699



Ultrasonic-Pulse-Echo-scan (UPE-scan) Herøysund bru - Mats Holmqvist, DEKRA
Illustrasjon - Gaute Larsen

Statens vegvesens rapporter

NPRA reports
Norwegian Public Roads Administration

Tittel
Utredning av metoder for inspeksjon av etterspent armering i betongbruer

Undertittel
FoU-programmet Bedre Bruvedlikehold 2017-2021

Forfatter
DEKRA Industrial: A. Karlsson, P. Jilderda
Luleå University: B. Täljsten

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Prosjektleder
Bård M. Pedersen

Godkjent av
Bård M. Pedersen

Emneord
Etterspente betongbruer, Ikke-destruktiv testing (NDT), undersøkelse av inspeksjons-metoder.

Sammendrag
Statens Vegvesen, SVV, sendte i 2018 ut en forespørsel om utredning av metoder, muligheter og begrensninger for inspeksjon av etterspente betongbroer.
Spennarmert betong er historisk ansett for å være pålitelig og effektiv konstruksjons-metode. For broer er det nå rundt om i verden satt fokus på utfordringene med hensyn til å vurdere broers faktiske tilstand, spesielt de ulike komponentene i spennsystemet.
Ulike ikke-destruktive teknikker (NDT) blir studert i detalj for vurdering av tilstanden til spennsystemet i etterspente betongbruer, og muligheter og begrensninger ved bruk blir diskutert.

Title
Post-tensioned Concrete Bridges
Study of methods for inspection

Subtitle
The R&D program Better Bridge Maintenance 2017-2021

Author
DEKRA Industrial: A. Karlsson, P. Jilderda
Luleå University: B. Täljsten

Department
Structures

Section
Structural Engineering

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Project manager
Bård M. Pedersen

Approved by
Bård M. Pedersen

Key words
Post-tensioned concrete bridges, Non-Destructiv Testing (NDT), study of methods for inspection.

Summary
In 2018, the Norwegian Public Roads Administration, NRPA, issued a request for a study of methods, possibilities and restrictions for inspection of post tensioned concrete bridges.
Post tensioned concrete has historically been considered to be a reliable and efficient construction method. For bridges, there is now a focus around the world on the difficulties with respect to assessing the actual condition of bridges, in particular the tendons.
Different non-destructive techniques (NDT) are studied and their benefits and shortcomings in use of assessing bonded post tensioned concrete bridges are discussed.

Antall sider 86 + vedlegg (184 totalt)

Pages 86 + appendix (184 total)

Forord

Denne rapporten er utarbeidet av FoU-programmet Bedre bruvedlikehold (2017-2021). Bedre bruvedlikehold skal gjennom ny kunnskap bidra til at Statens vegvesen kan optimalisere ressursbruken knyttet til inspeksjon, vedlikehold og forvaltning av bruer.

Bedre bruvedlikehold består av fire prosjekter:

Prosjekt 1: Forvaltningsverktøy for bruer
Prosjekt 2: Armeringskorrosjon i betong
Prosjekt 3: Alkalireaksjoner i betong
Prosjekt 4: Vedlikehold av stålbruer

Bedre bruvedlikehold ledes av Bård Pedersen, Vegdirektoratet.

Denne rapporten tilhører Prosjekt 2: «Armeringskorrosjon i betong» som ledes av Karla Hornbostel. Prosjekt 2 er rettet mot drift og vedlikehold av betongbruer med armeringskorrosjon. Mål for prosjektet er å utarbeide anbefalinger for inspeksjonsmetoder for å utrede omfang av skader på grunn av armeringskorrosjon samt å utvikle verktøy for å kunne bedømme konsekvenser av armeringskorrosjon for bruens levetid. Prosjektet skal også utarbeide et beslutningsgrunnlag for valg av reparasjonstiltak og anbefalinger for gjennomføring av tekniske gode og økonomisk effektive reparasjonstiltak.

Rapporten er skrevet av Dekra Industrial AB i samarbeid med Luleå University of Technology og er utarbeidet i delprosjekt 2.4 «Spennarmering», som ledes av Lise Bathen. Prosjektet har søkelys på kartlegging av skader og konsekvenser av dette på både føroppspent og etteroppspent armering.

Rapporten gir oversikt over ikke-destruktive metoder (NDT-metoder) for å kartlegge plassering av etterspent spennarmering i eksisterende konstruksjoner og metoder for å kartlegge hulrom i spennsystem. Hulrom er viktig å kunne kartlegge fordi de representerer en mulig risiko for pågående korrosjon på spennarmeringskomponenter. Rapporten er en utredning av tilgjengelige metoder, muligheter og begrensninger ved bruk av disse og gir anbefalinger om forutsetninger for bruk av de enkelte metodene.

Post-tensioned Concrete Bridges

Study of methods for inspection

Review of literature and experiences from laboratory- and field tests

December 2020

DEKRA Industrial: Andreas Karlsson, Mats Holmqvist,

Pieter Jilderda, Joakim Strand, Bernt Åke Johansson

Luleå University of Technology: Björn Täljsten

Preface

In autumn 2018, the Norwegian Road Authorities (Statens Vegvesen) issued a request for a study of methods, possibilities and limitations for inspection of post-tensioned concrete bridges. DEKRA Industrial AB in Sweden was awarded the assignment, which was carried out in cooperation with Luleå University of Technology, Division of Structural Engineering. The project started in winter 2019 and will be finished in November 2020.

Post tensioned concrete has historically been considered to be reliable and an efficient construction method, demanding little maintenance in comparison to traditional reinforced concrete. For bridges however, some collapses have occurred world-wide, putting focus on the difficulties with respect to assessing the actual state of the bridges, in particular the tendons.

General methods for assessment of concrete bridges are discussed and suggestions for improved assessment regarding post-tensioned concrete bridges are given. Different NDT techniques are studied in detail and their benefits and shortcomings in the use of assessing bonded post-tensioned concrete bridges are discussed.

Cover Photo: The Kåkern Bridge, Flakstad, Norway

Summary

Post-tensioned 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 post-stressing strands, wires and bars, remaining post-stressing forces 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.

In the literature studied no NDT (Non-Destructive Testing) method has been identified that can directly detect corrosion or possible breakage in the tendon wires. This investigation has mostly focus on methods that directly identify voids (since a void or improper filling can be an indication of corrosion) in the ducts and methods that indirectly can identify corrosion or tendon breakage.

The criteria underlying the selection of methods are the following:

- Handling - Refers to how easy the equipment is to use in field, how easy data can be processed, and effort needed for training personnel.
- Cost - This can be related to cost for buying the equipment, cost for software and time for site-investigation and post processing of data.
- Accuracy - Data shall be reliable and results trustworthy.

Before an NDT investigation is carried out it is recommended that a strict assessment procedure is followed. This procedure should cover investigation of existing documents and previous rehabilitation procedures. It should clarify the objectives with the assessment in general and for post-tensioned structures identify critical sections, in particular those sections where voids are most likely to appear.

In the presented work we have suggested and applied an assessment procedure for post-tensioned concrete structures. Preceding this, laboratory tests and field tests on mock-ups were carried out. Based on this experience and knowledge gain, the next step was to investigate methods and methodologies in the field. This was first investigated on the Herøysund bridge on the north coast of Norway. In this case, main focus of the Non-Destructive Testing (NDT) was to determine voids in the cable ducts, especially at the critical areas that had been identified by the designer.

In the project we followed a strict procedure, as presented in section 5. It is important first to identify the problems to be investigated, which consequently form the basis for the methods to be used in the investigation. Based on this we decided to use a combination of different NDT methods listed below:

- Visual inspection to mark out and check critical areas
- Cover meter
- Ground Penetrating Radar
- Ultrasonic Pulse Echo
- Impact Echo
- Visual inspection/drilling and the use of endoscope

Experience is needed when these methods are combined to investigate voids and defects in tendon ducts. It is difficult to determine the degree of grouting in the duct and the NDT methods often need to be combined with partly destructive testing, i.e. a hole needs to be drilled into the duct and closer investigation with for example endoscope might be needed to confirm the findings.

www.dekra-industrial.se
tel. +46 10 455 10 00
Gamlestadsvägen 2
SE-402 51 Gothenburg

The results from the NDT, presented in this report, give a good overview of the voids in the investigated areas. We were able to identify where the voids are located and define the size of the voids.

However, even though the methodology has been successful in these projects, it is recommended that further field projects and laboratory tests are carried out to obtain more experience and also to find limitations and new possibilities with the use of NDT in assessment of existing post-tensioned concrete structures.

Keywords: Post-tensioned concrete bridges, Pre-tensioning, Post-tensioning, Remaining post-stressing forces. Non-Destructive Testing (NDT), Monitoring and Inspection, Condition Assessment, Life Cycle Assessment, Ultimate and Serviceability Limit States (ULS, SLS), Maintenance Strategies.

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 pre- and post-stressing 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 is the capability of a strained body to recover its size and shape after deformation.

Robustness is the capacity of a structure to function also with accidental or exceptional events.

Redundancy indicates that a structure can carry loads also if one part is removed.

Abbreviations

CM	Cover Meter
EIS	Electrochemical Impedance Spectroscopy
ES	Endoscope
GPR	Ground Penetrating Radar, Geo radar, Impulse radar
IE	Impact Echo
IT	Infrared Tomography
MFL	Magnetic Flux Leakage
MMFMP	Magnetic Main Flux Method - Permanent Magnet
MMFMS	Magnetic Main Flux Method - 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

Concrete civil infrastructure and concrete structures are in general 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 money and decreasing the overall environmental impact of the structure.

Communication between people and transportation of goods constitute for exchange of ideas and growth in society. Without a well-functioning 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, more precise reinforced concrete and for our large bridges post-tensioned concrete. During the last two to three decades more and more focus has been placed on evaluating the life span of our bridges and ongoing deterioration processes. During this time span, a considerable development regarding assessment and strengthening methods has also taken place. In addition, today we also have stronger calculation tools and a better understanding of our existing bridges and their behavior – at least for reinforced concrete (RC) bridges where existing assessment and repair/strengthening methods are quite well understood.

However, this is not the case with our existing post-tensioned concrete bridges, despite the fact that these bridges are critical for transportation and communication in our modern society. One large challenge with post-tensioned bridges is the lack of possibility to assess the inner parts, i.e. ducts, anchorage and tendons, with non-destructive methods. In literature, challenges and problems with post-tensioned concrete bridges have been shown.

Post-tensioned concrete has historically been considered to be a reliable and an efficient construction method. Due to the high strength of post-stressed steel constructions, they should normally require less maintenance than traditional reinforced concrete bridges. After the collapses of a few post-tensioned bridges in Europe, the system has rather been considered as a risk construction, unless constructed and built in a proper way and inspected regularly.

Bonded ducts are especially a high-risk construction due to the lack of inspection possibilities of the grouted tendons. In Norway, there are over 2000 bridges built with prestressed and post-tensioned concrete. Due to bridge collapses worldwide caused by corrosion on tendons, the regulations in Europe were updated during the 1990s and early 2000s.

Requirements for building methods of post-tensioned bridges in Norwegian norms and standards have changed through the years. Inspections made by the Norwegian Public Road Administration (NPRA) have revealed defects related to systematic deficiencies when building post-tensioned bridges. In general, inspections and studies of the earlier methodology led to the conclusion that post-tensioned bridges built before 1982 most likely have deviancies and damages such as air voids in ducts, entrapped water, frost blasting and corrosion.

In 1982 a series of improvements were carried out with respect to insufficient mixing methods. Previously, the injection mortar was often mixed with inferior results (water separation, lumps and high water-cement-ratio). In 1982 began the use of colloid mills when mixing the mortar, which led to a more satisfactory mixing. The same year a new manual (NB 13) was established by Norsk Betongforening. During the following years, further improvements were set in motion. Better materials were applied, but the most significant improvement was the creation of inspection manuals for installation of post-tensioning, defining requirements which evolved out of experiences from previous mistakes. The quality and stability of injection mortar improved, and manual NB 14 (developed from NB13) became the reference and requirement to follow for bridges as it came out in 1986. Later it has been used as a complement to the current norm NS-EN 13670. The actual manual NB 14 is dated 2016 including a number of revisions.

After 1982 there has been a decrease in problems with the mixing of injection mortar, although segregation and separation of the mortar still occurred, leaving voids on high points of the ducts. The European standards EN 445, EN 446 and EN 447 came in 1997 and put requirements on materials, equipment and test methods to ensure good quality for grouting.

SVV´s manual R762 (latest revision is dated 2018) implemented EN 445, EN 446 and EN 447, as well as NB14. In 2005 companies in Norway together with NPRA developed a new stable thixotropic grout which was a real achievement to secure sufficient grouting of the ducts. The results and development are applied in NB14. In manual R762, also further stricter regulations are applied, e.g. regarding maximum allowable chloride content (CaCl2) as a chemical accelerator for the concrete during cold temperatures, strength classes and concrete cover for reinforcement.

International experience shows that today there might be problems with post-tensioned concrete structures, especially of older constructions. The extent of the problems is to a large degree unknown since the bars, strands and wires are protected by grout and cast into the structure and therefore not accessible for visible inspection.

In this report we have carried out studies of state-of-the-art report regarding non-destructive testing (NDT) methods and in particular studied and tested several NDT techniques for validation of damages in post-tensioned concrete bridges.

1.2 Objectives

The objective of the project is initially to map available non-destructive test methods to discover damage to bonded tendons and anchors. A limited number of most promising methods are then selected, based on the following criteria: coverage (type of damages), technical maturity, complexity, accuracy, availability (in Norway), costs, user friendliness and user requirements (knowledge/training of inspectors).

The selected methods are investigated in depth and tested in practice, both in mock-ups and in the field. The abovenamed factors are evaluated for each of the selected methods.

Based on this, a strategy will be developed for bridge inspectors working in practice to ensure a comprehensible, technically and economically optimized selection of methods for the investigation of individual projects.

The result is presented in a report describing how selected methods can be used for discovering different types of damage, either directly or by using a combination of different methods.

1.3 Limitations

The project comprises of studying existing literature followed by performing laboratory tests and field applications. Due to the broad coverage, choices were made in order to concentrate resources on the most promising methods/techniques. A thorough survey of printed works on inspection of concrete was performed, but we do not claim it to be exhaustive. The lab tests were carried out on three mock-ups where the size and the number of faults were limited for practical reasons. Field tests presented in this report cover the most promising inspection techniques in several relevant applications. Naturally, they are not exhaustive, i.e. more field tests could be performed.

1.4 Report structure

Chapter 1 points out why assessment of concrete structures is important in general and for post-tensioned concrete bridges in particular. Chapter 2 discusses safety and condition assessment of post-tensioned bridges. In Chapter 3 an extensive overview of monitoring and inspection techniques for post-tensioned bridges is presented. Out of the most promising inspection techniques, a few methods have been selected and described in more detail, as presented in Chapter 4.

In Chapter 5, a suitable assessment procedure for post-tensioned concrete structures using NDT is presented, which consequently is being used during mock-up and field tests. In Chapter 6, tests on mock-ups are briefly presented and in Chapter 7 the results from the field tests are shown. Finally, chapter 8 contains a discussion and final conclusions.

In the Appendixes, tests on mock-ups and field tests are described in detail.

2 Safety and condition assessment of post-tensioned bridges

2.1 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 for maintenance, repair, strengthening or replacement are required over its lifespan. To make this most efficient from a client and cost perspective, guiding principles should be followed.

In figure 2.1 general principles for maintenance, repair and strengthening of concrete structures is presented (ISO 16311-1, 2014). It consists of four important parts. Part 1: General principles which relates to the performance of the structure, where performance refers to structural safety (load carrying capacity), serviceability, appearance (aesthetics) or mitigation of for example falling debris due to lack of maintenance. In figure 2.2 the performance of a structure over time is presented in more detail.

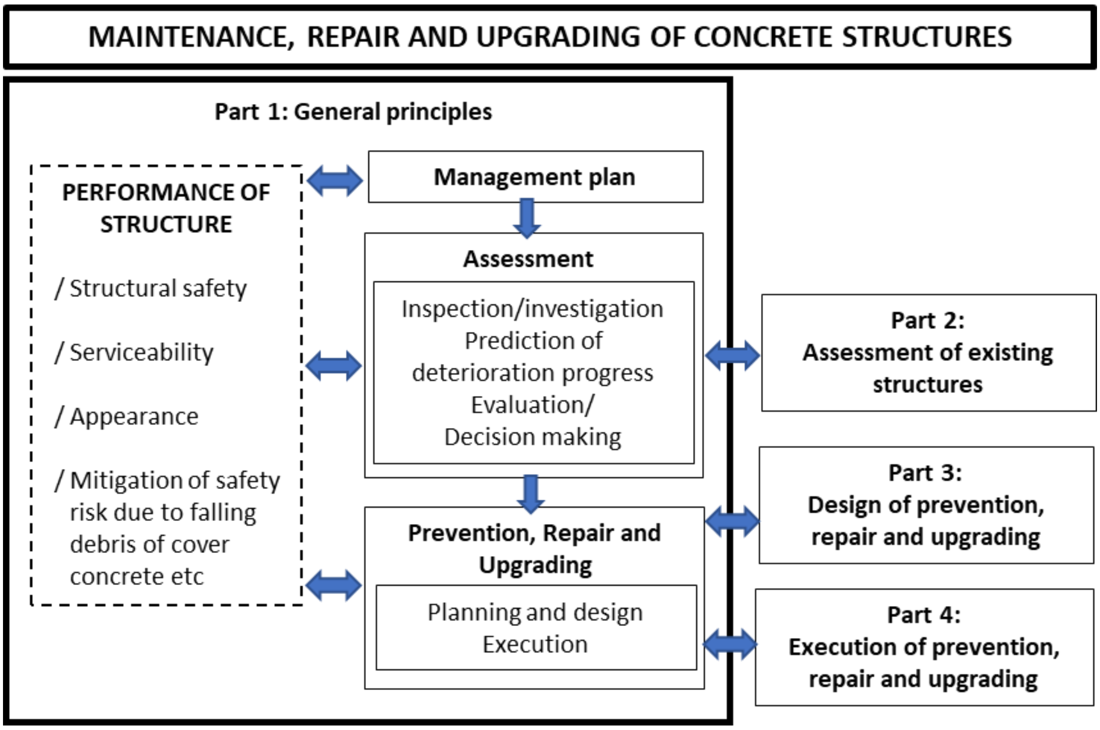


Figure 2.1 General principles for maintenance, repair and upgrading, based on (ISO 16311-1, 2014)

The structure always has an initial performance and often safety sets the lowest performance/requirement on the structure. In figure 2.2 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.

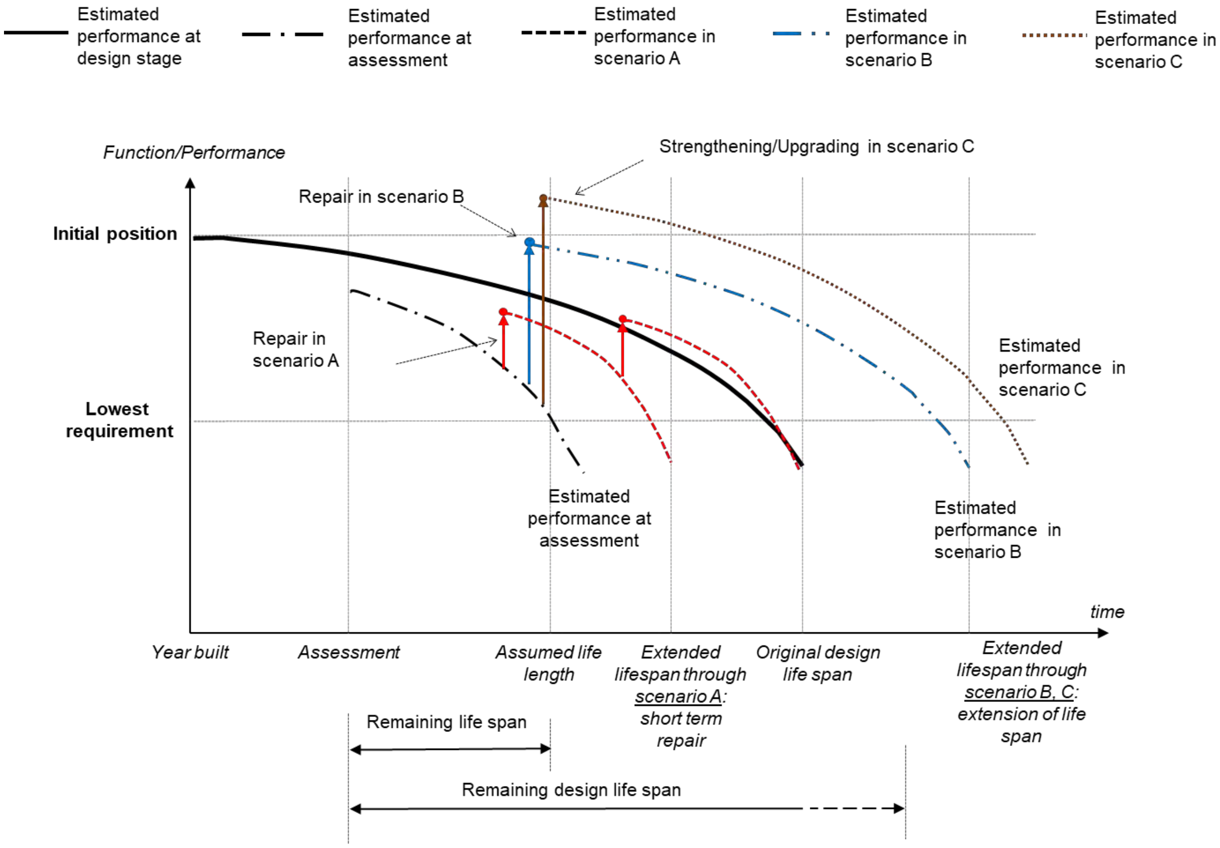


Figure 2.2 Performance of structure, based on (ISO 16311-1, 2014)

Part 2 in figure 2.1 covers assessment of existing structures, see figure 2.3 (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.

One possible 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 purpose of 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 are reported and from the findings, decisions regarding future measures should be taken.

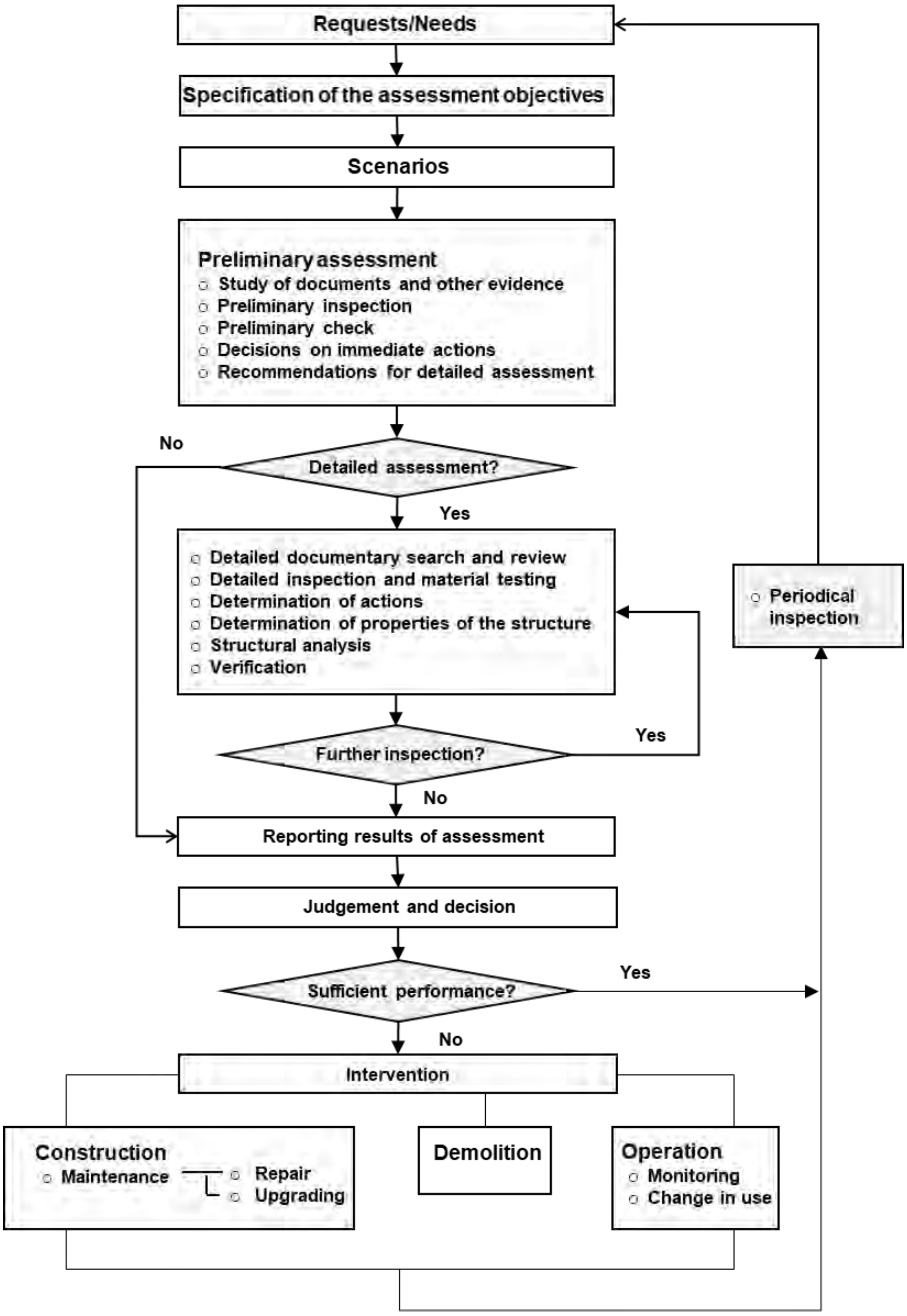


Figure 2.3 General flowchart for assessment of existing structures, based on (ISO 16311-2, 2014, ISO 13822)

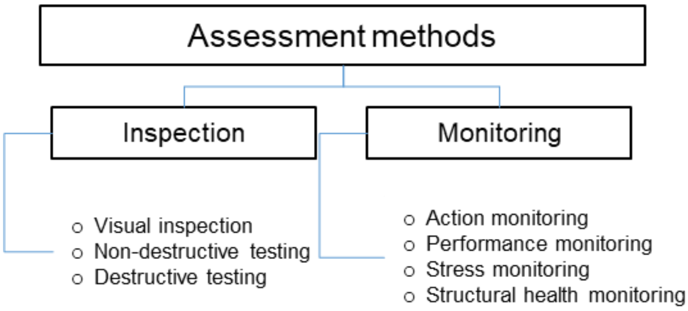


Figure 2.4 General classification of assessment methods based on (Honfi et. al, 2018)

In general, the assessment methods can be classified into inspection and monitoring, see figure 2.4. Inspection covers visual inspection, non-destructive testing with appropriate systems and methods for the concrete structure studied, see chapter 3. Also, destructive testing is included, which may vary depending on 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 whether 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, to 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.

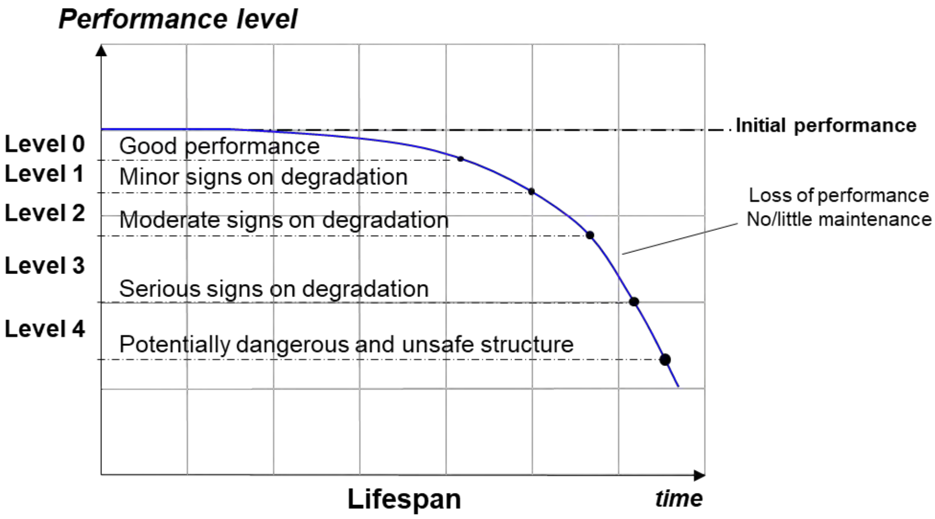


Figure 2.5 Principle illustration of the condition levels, depending on time, based on (ISO 16311-2, 2014)

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 2.6.

In order to assess the performance of the structure and possible consequences, five performance levels and five consequence levels are suggested (ISO 16311-2, 2014). The different levels of performance are explained in table 2.1.

Table 2.1 Performance levels related to figure 2.5, 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 may already have started.
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 signs 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 recommended that the following consequence levels are used, see table 2.2.

Table 2.2 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 are not acceptable, interventions need to be made. 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 situations the structure must be demolished and replaced. The intervention depends on the cause for the defect or deterioration process. Common causes are presented in figure 2.6.

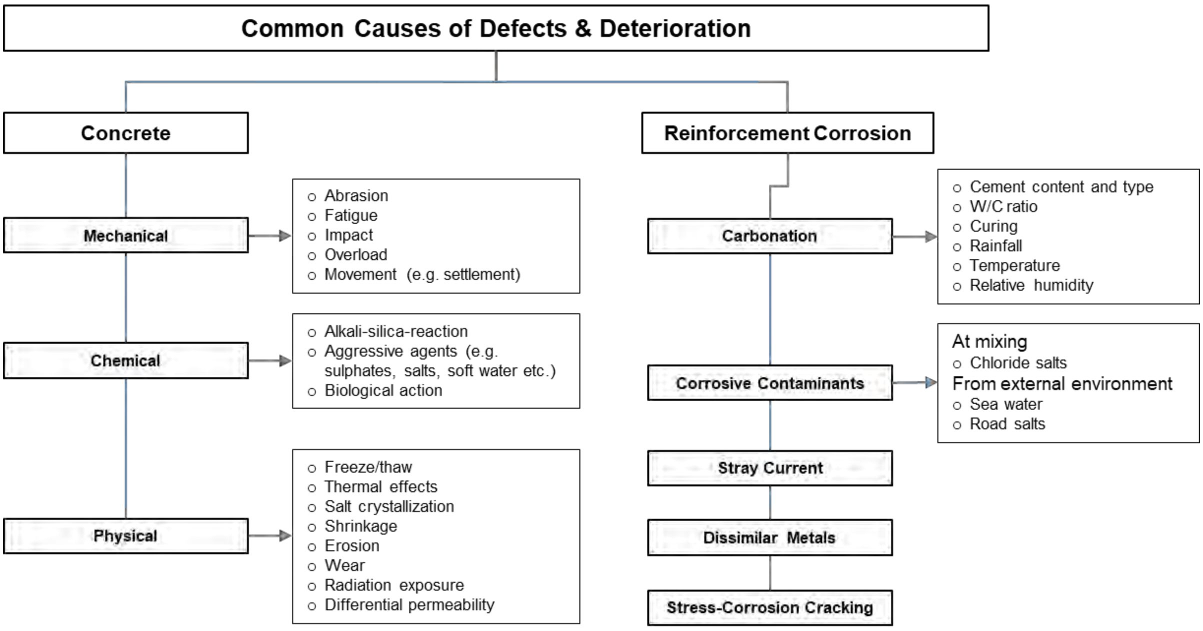


Figure 2.6 Common causes of defects and deterioration of concrete structures (ISO 16311-3, 2014)

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 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).

2.2 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 1990s, 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 existed to do this in a controlled manner. Preliminary assessments showed that the concrete fatigue capacity of many of the bridges would be jeopardized. However, it was concluded that the codes were conservative in this respect and consequently the decommissioned bridge was tested. 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, 2018a, 2018b)

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. 2017), 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. 2.7. 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).

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).

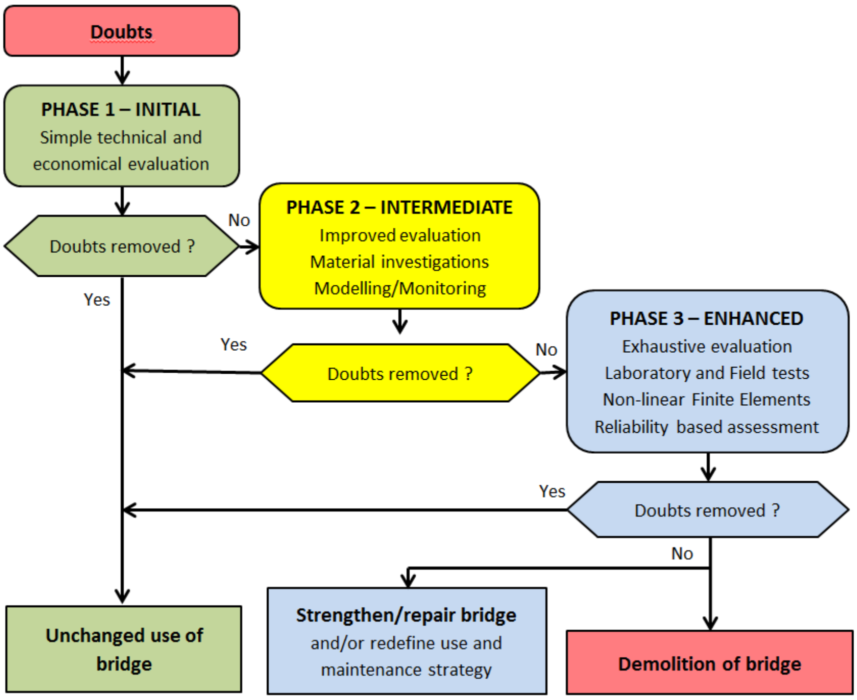


Figure 2.7 Flow chart for the assessment procedure of a bridge with three phases. Based on (Schneider 1994, 2017, Sustainable Bridges 2007, UIC Code 778-4, 2009, ISO 16311-2,2014 and Paulsson et al.2016).

2.3 Current Norwegian practice

As in most parts of Europe, Norwegian bridges are inspected regularly and systematically in order to meet the safety and accessibility requirements of road users. In general, the assessment and inspection procedures follow the ones presented in chapter 2.1.

Inspection procedures have developed through the years, partly based on knowledge and experiences from earlier shortcomings and errors, regarding various critical work steps during construction of post-tensioned bridges. Interviews with staff having experience of both building post-tensioned bridges and performing inspections since the 1970s have clarified, as well as raised a number of issues that need to be addressed. According to the interviewees bridges in more recent years are constructed with emphasis on safety and inspection but that things were different in the past. Inadequate control, documentation and building actions were, according to the interviews, common denominators during construction of older post-tensioned bridges.

A summary of the issues stated below:

- Tensioning system; inspection and control of third party were not always carried out on tendons and ducts regarding e.g. mounting of ducts, damage control of ducts and tendons, correct amount of cables and the clamping force.
- Injection of mortar; due to wrongly placed ducts, drain- pumping- and filling tubes, workability problems of the mortar, plugging of ducts by ice, concrete and debris before injection, inadequate composition of mortar etc. it is expected to be problems with air filled voids and trapped water.

The inspection shall clarify the physical and functional condition of the bridge. All bridges must have a main inspection every fifth year and less extensive visual checks annually. All inspections are then registered in their own bridge management system BRUTUS. In this system, all available data is collected from the bridges including pictures and drawings.

However, there is no standard or recommendations how to investigate a concrete structure with NDT. There are assessment strategies where three different levels of assessment are addressed;

- Level 1 – Simple analysis: Cover visual inspection with possible carbonation measurements and control of concrete cover. The visual inspections are carried out from ground level, by boat or from easily accessible places.

- Level 2 - Extended analysis: An extended analysis includes detailed visual inspection of all concrete surfaces and several measurements and material tests. It will normally include:
  - Measurement of carbonation depth
  - Thickness of concrete cover
  - Measurement of chloride content/profile
  - Measurement of crack widths
  - Detection of voids/cavities
  - Uncoverage of steel reinforcement

In addition, it may be needed at Level 2 to carry out:

  - Potential measurements
  - Pull-out tests
  - Testing of reinforcement continuity
  - Measurement of deflections/settlements or other damages

- Level 3 – Extensive analysis: At Level 3 are normally carried out after Level 2 and all analyses at Level 2 are also carried out at Level 3. In general, more material samples are taken out from the structure and tested at Level 3

Inspections of post-tensioned bridges are critical, and these inspections do not include examination of the condition of the tendons. There is currently no method for investigating tendons and inspections that are carried out are therefore limited to detecting damages such as cracks, caused by impaired load capacity of the bridge.

2.4 Assessment of post-tensioned concrete bridges

For post-tensioned 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. A thorough desk study and planning is crucial. It is impractical and uneconomical to uncover all hidden components for inspection and many competing factors must be considered before undertaking any investigation.

Questions to be asked:

- Is the component critical for the safety of the structure?
- What are the consequences of failure of the component?
- Can the component be exposed safely?
    - May this result in damage to the structure or the component itself?
    - Could this lead to long-term durability issues with the structure?
    - Can it be justified economically?
- 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 during an assessment. Investigations aiming to find hidden defects can either be undertaken as special investigations or be integrated in normal inspection routines. The former should be considered where the risk of failure of a hidden component with significant consequences is likely to occur before the next planned inspection.

Where it is not possible to inspect all hidden components within the required cycle of normal inspections, those of greatest risk should be prioritised. In figure 2.8 hazard scenarios for post-stressing steel for a typical box girder bridge are shown. Possible defects in the figure are indicated by a number which is explained below (Matt, 2000)

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
- honeycombing
- concrete deterioration by freeze-thaw
- joint leakage
- etc.

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

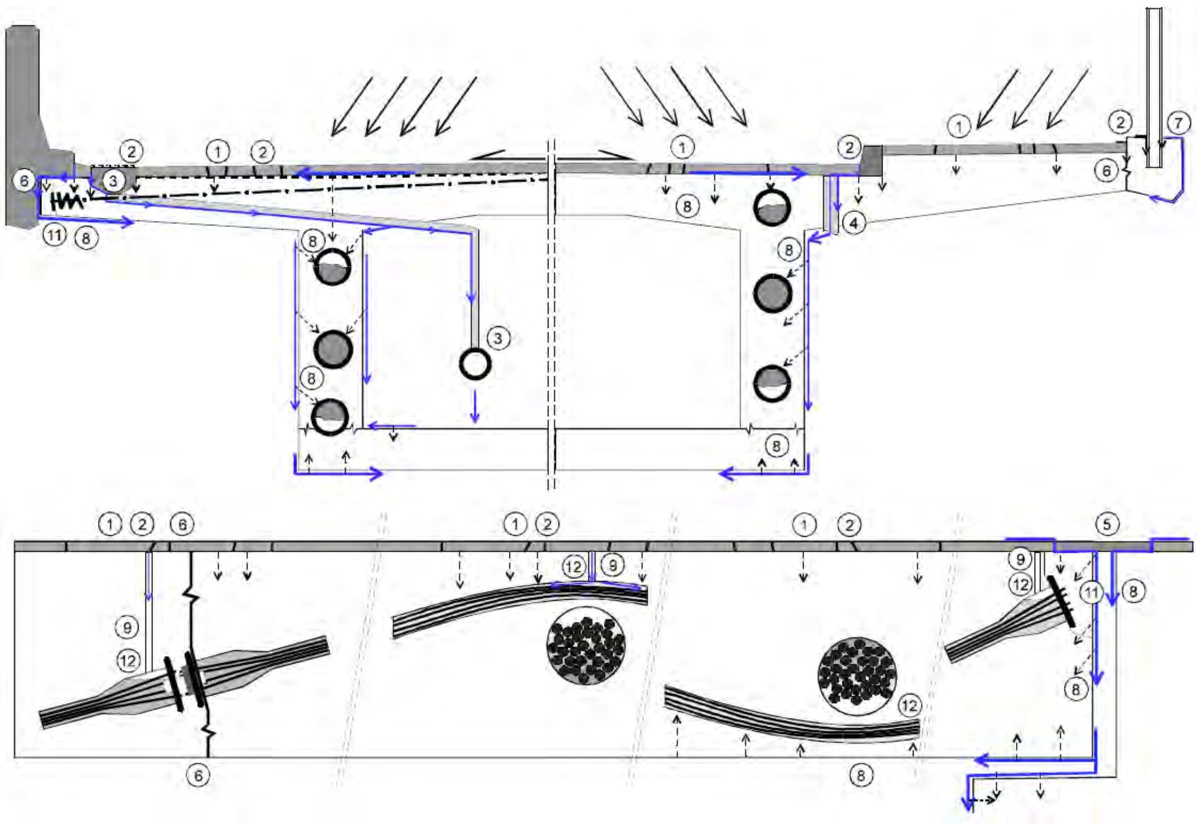


Figure 2.8 Hazard scenarios for post-tensioned steel in a typical box girder bridge. Indication of potentially weak points where water (possible with chlorides) can gain access to the tendons and cause corrosion (Matt, 2000).

2.5 Tendon corrosion

In this paragraph, a scenario for corrosion of the tensioning strands is drawn up, in order to show the mechanisms behind it and both define and quantify the circumstances required for an onward going corrosion process. The evaluation below has deliberately been limited to the most usual corrosion processes; it shall be noted that also corrosion under low oxygen circumstances may occur.

It is well established that the tensioning strands corrode if they are exposed to an environment unsuitable for them.

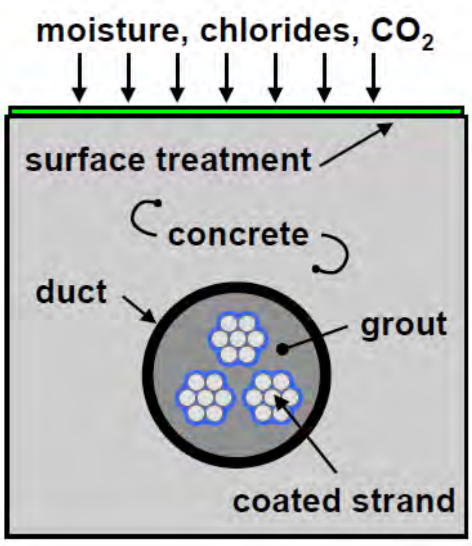


Figure 2.9 Typical configuration of tendons and their environmental exposure (West J.S. (1999))

The corrosion primarily appears in three different forms:

- General corrosion
- Pitting / local corrosion
- Stress corrosion cracking (SCC) and hydrogen related embrittlement

General- and local corrosion may follow the most common fundamental corrosion reactions, a chemical reaction between iron, water and oxygen: 2Fe + 3H2O + 3/2 O2 → 2Fe(OH)3. However, corrosion may also occur in a low-oxygen environment.

Stress corrosion cracking and hydrogen related embrittlement cause local attacks and require specific circumstances to develop. SCC only develop if the stresses are above a certain threshold and there is a media in contact with the steel that the steel is susceptible to. Hydrogen embrittlement can occur when atomic hydrogen is produced locally. The hydrogen for example originates from corrosion of iron in the presence of chloride (Fe+Cl-+H2O = FeOH + Cl- +H) or from a reaction between alkaline concrete and galvanization (Ca(OH)2 + Zn + 2H2O = Ca(Zn(OH)4) + 2H). In general, high strength steels are more vulnerable to these effects.

Pitting corrosion in the presence of water and oxygen is schematically described in figure 2.10. These attacks are normally local in nature. The presence of chloride in the pit will catalyse the process, aggravating the situation.

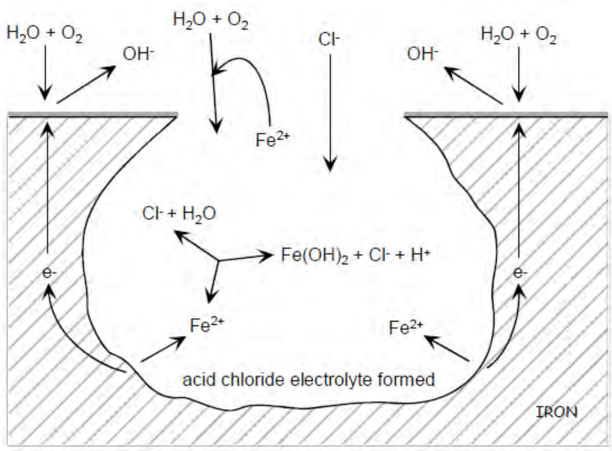


Figure 2.10 Pitting corrosion (Schokker, A. J. (1999))

It may be noted that corrosion products (iron-hydroxides) occupy a significantly larger volume than the iron (x10), therefore expansive forces may arise. For the grout this may cause serious problems.

Regardless of mechanism, the corrosion of the tendons will be dependent of several factors, making the identification of the local conditions difficult. Material quality, utilisation grade, pH, chloride content, exposed metal area, the availability of water and oxygen and presence of chemicals are but a few of the factors affecting the process.

The most common form of corrosion will occur in the presence of water and air, therefore the search for voids within ducts is deemed to be meaningful, where large voids logically maintain more air. Also, the intactness of ducts may be of importance, as it will hinder water and fresh air to penetrate.

2.6 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 2.11 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). In Figure 2.11 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 σ_M that separate the mean value μ_M from m = 0, that is $\beta = \mu_M / \sigma_M = 6 / 3,61 = 1,66$.

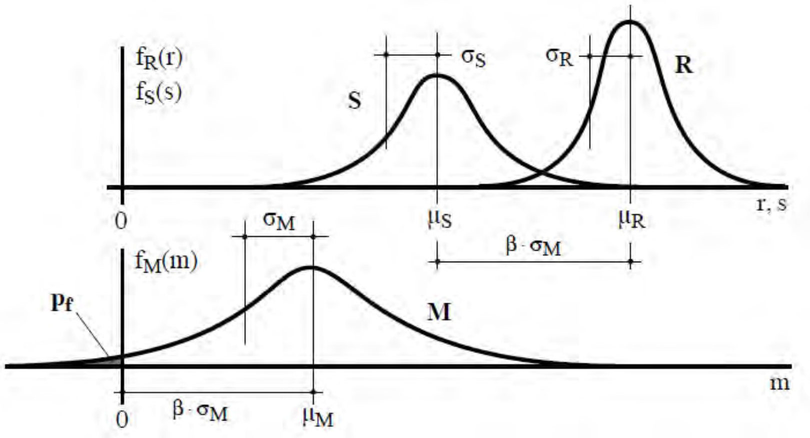


Figure 2.11. Frequency functions $f_R(r)$, $f_S(s)$ and $f_M(m)$ for resistance (**R**), load effect (French: sollicitation) (**S**) and safety margin (**M** = **R** – **S**) as functions of the resistance (r), the load (s) and the margin (m). For the bending capacity of a bridge slab we may assume the 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 μ_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 2.3.

Table 2.3. 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 (γ), special load factors for railways (α) and reliability indices (β). The partial coefficients are safety factors used for loads and material properties, the α-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 (α-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 α-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 β = 2.3, a probability $p_f = 10^{-5}$ corresponds to β= 4.2, a probability $p_f = 10^{-6}$ corresponds to β= 4.7 and a probability $p_f = 10^{-7}$ corresponds to β = 5.2 see Table 2.3, 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 ***T* = 1** year the recommended reliability index is **β = 5.2** corresponding to a failure probability of $P_f = 10^{-7}$ (1 in 10 million). If the period is increased to ***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 ***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 for 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 γ, α-factors or reliability indices β, see e.g. SB-LRA (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).

3 Inspection techniques for post-tensioned bridges

3.1 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) in combination with NDT (Non-Destructive Testing) 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.

Authors' note: The goal of the study of publications on NDT techniques was not to investigate them in depth. Focus has been on selecting the most promising methods in an effective way. Thereafter, the selected methods were studied in depth and applied in practise.

3.2 Inspection techniques

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, corrosion and corrosion rates.

In this section, focus is placed on inspection techniques for post-tensioned concrete bridges and methods to detect faults within the post-tensioning system itself. Naturally, there are more parts of a post-tensioned bridge that can deteriorate and need to be inspected.

Post-tensioned tendons in post-tensioned 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 post-stressing force should preferably be done by non-destructive testing or at least low-destructive techniques and with minimum 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. Moreover, with NDT internal damages might be discovered before they grow and reach the surface, thus enabling corrective measures in due time.

Due to the nature of reinforced and post-tensioned concrete, several 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 post-stressing 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 are also possible via endoscope and monitoring techniques may be used to measure the residual force. For intrusive methods, great care is needed to avoid damage to post-stressing elements and to re-sealing. In general, there are five factors that needs to be investigated in post-tensioned concrete structures, either by inspection or monitoring; these are;

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

There are several inspection 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)
- Intrusive methods
  - Endoscope (ES)
  - Residual Prestressing Force (RPF)
  - Potential Mapping (PM)

As we are focussing on existing bridges, the abovenamed residual prestressing force (RPF) is defined as an intrusive method, assuming that access to the tendons must be obtained by drilling/grinding. For new constructions, it may be possible to measure forces continuously by a monitoring system.

As we investigated different NDT techniques for concrete, one publication was found to be very informative, covering many different inspection techniques with focus on pre-stressed tendon systems. Therefore, many references are made to this particular source: NCHRP Inspection Guidelines for

Bridge Post-Tensioning and Stay Cable Systems using NDE Methods (2017) (http://nap.edu/24779), especially for more uncommon methods such as AE, IT, EIS and magnetic methods.

Table 3.1 Summary of some NDT and intrusive methods in relation to their use for post-tensioned concrete bridges (based on Sederholm, 2006)

Method	**Localisation of voids in concrete**	**Detecting post-tensioning system**	**Detecting corrosion** [5]	**Quantifying damages in tendons**
Manual Methods				
Visual Inspection	N.A	N.A	N.A	N.A
Acoustic Methods				
Impact Echo	X	X	N.A	X[1]
Ultrasonic Pulse Echo	X	X	N.A	X[1]
Spectral Analysis of Surface Waves	X	X	N.A	X[1]
Acoustic Emission	N.A	X	N.A	X[2]
Radiographic Methods				
Radiography	X	X	X[1]	X[1]
Electromagnetic Methods				
Ground Penetrating Radar	X	X	X[1]	X
Infrared Tomography	X	N.A	N.A	N.A
Cover Meter	N.A	X[4]	N.A	N.A
Magnetic Methods				
Magnetic Flux Leakage	N.A	N.A[3]	X	X
MMFM-Permanent magnet	N.A	N.A[3]	X	N.A
MMFM-Solenoid	N.A	N.A[3]	X	N.A
Electrochemical Methods				
Electrochemical Impedance Spectroscopy	N.A	N.A[3]	X[3]	N.A
Intrusive Methods				
Endoscope	N.A	N.A	X	X
Residual Prestressing Force	N.A	N.A	N.A	X
Potential Mapping	N.A	N.A	X	N.A

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

3.2.1 Manual Methods

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 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 result from the inspection is also dependent on the accessibility to the structure and weather and light conditions. Nevertheless, in general a visual inspection is fast and economical and to our experience all assessments shall start with a visual inspection before more advanced methods are introduced.

3.2.2 Acoustic methods

Impact Echo is an acoustic method based on sending out a wave by an impact on the concrete surface and recording the response (wave energy) with a transducer. The signal will reflect on essential boundaries, such as enclosed air or objects with a distinct difference in density. Due to a relatively low attenuation, the method is particularly suitable for thick concrete elements. Internal defects such as voids or objects with a high density such as tendons can be identified.

In figure 3.1, 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.

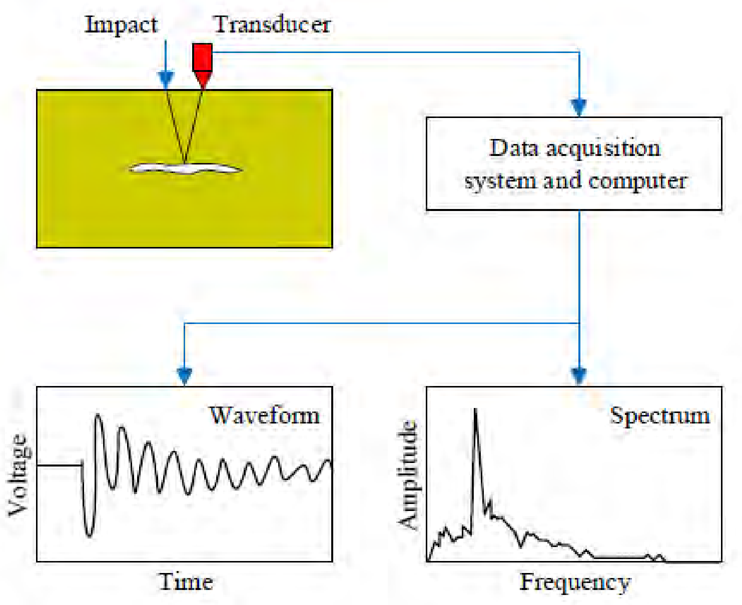


Figure 3.1 Simplified diagram of the IE-method (from www.ndt.net)

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 3.2 capabilities and limitations of the IE technique are summarised.

Table 3.2 Capabilities and limitations of the IE technique (partly from http://nap.edu/24779)

Defects	IE can locate voids and grout in internal ducts with moderate accuracy. It can also identify tendons. IE cannot locate strand defects in ducts nor defects of the anchoring.
Duct location	Internal and external ducts
Duct type	Metal and nonmetal ducts
Effect of concrete cover	Good penetration, however, thick concrete attenuates the impact echo signal and may lead to difficulties during measurements
Effect of layered ducts	Layered ducts obstruct measurements
Effect of reinforcement	Steel reinforcements obstruct measurements
Effect on corrosion	No possibility to detect corrosion
Accessibility requirement	IE demands only a small accessible area.

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, see a principal sketch in Figure 3.2. 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 amounts 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 post-tensioned concrete, rebar and tendon locations, compaction faults and voids. Corrosion can normally not be detected. With 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 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 3.3 capabilities and limitations of the UPE technique are summarised.

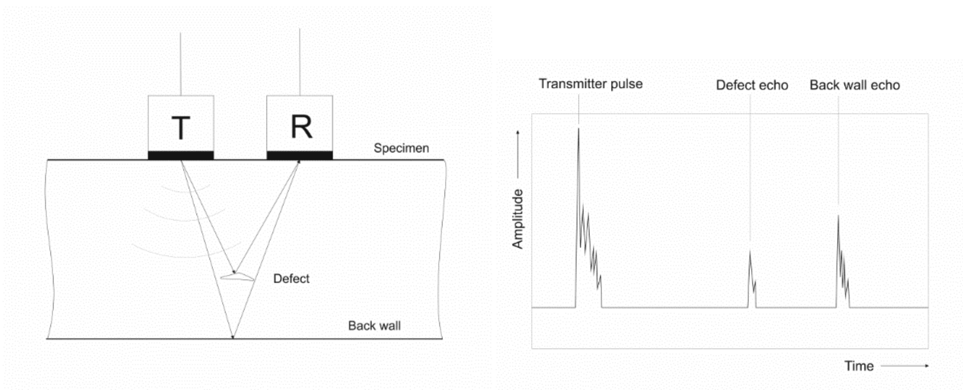


Figure 3.2 Simplified diagram of the UPE-method (from www.ndt.net, zfp.cbm.bgu.tum.de)

Table 3.3 Capabilities and limitations of the UPE technique (partly from http://nap.edu/24779)

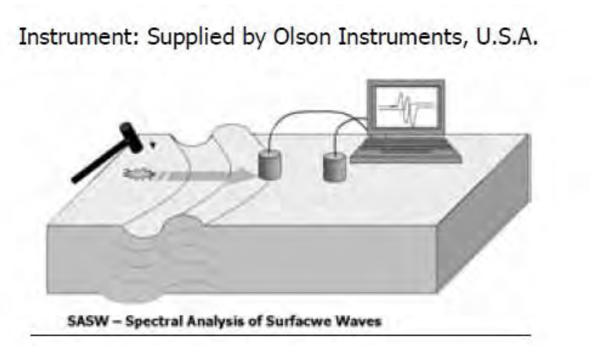
Defects	UPE can locate voids in concrete and grout defects (enclosed air) in internal ducts with moderate accuracy. UPE cannot be used for detecting defects of the anchoring.
Duct location	Internal ducts
Duct type	Metal and non-metal ducts
Effect of concrete cover	Typical concrete covers do not obstruct UPE inspection, though signals are attenuated with increasing depth. Depending on the concrete composition and required accuracy of results, the maximum possible thickness to inspect is up to 1….1,5 m.
Effect of layered ducts	Ducts behind other ducts can in certain cases be distinguished using UPE. The position of the two transducers can be varied, which aids in mapping out the volume of the defect.
Effect of reinforcement	A considerable amount of steel reinforcement bars will hinder measurements, as they reflect acoustic waves. Objects underneath the reinforcement will be hard to distinguish.
Effect on corrosion	Not possible to detect corrosion
Accessibility requirement	For UPE devices, the width required for scanning is about 300 mm. The surface must be relatively smooth to enable sound to penetrate into the material.

Spectral Analysis of Surface Waves (SASW)
From ‘A Practical Guide to Non-Destructive Examination of Concrete (2004) – Nordic Innovation Centre / Force Technology’:

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 extensive. 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.



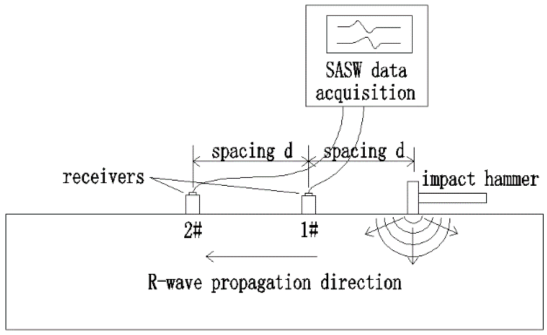


Figure 3.3 Simplified diagram of the SASW-method (from www.mdpi.com)

Table 3.4 Capabilities and limitations of the SASW technique (http://nap.edu/24779)

Defects	SASW can locate grout defects with low to moderate accuracy. It does not detect strand defects. SASW cannot be used for detecting defects of the anchoring
Duct location	Internal ducts
Duct type	Metal and nonmetal ducts
Effect of concrete cover	SASW can be used with concrete covers between 50 – 300 mm. Deeper cover may be acceptable provided there is not too much reinforcement.
Effect of layered ducts	Layered ducts obstruct measurements
Effect of reinforcement	A considerable amount of steel reinforcement bars will hinder measurements. Objects underneath the reinforcement will be hard to distinguish.
Effect on corrosion	Not possible to detect corrosion
Accessibility requirement	SASW demands only a small accessible area

Acoustic Emission (AE)

From www.nde-ed.org/EducationResources/CommunityCollege/Other%20Methods/AE/AE_Intro.php

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 3.4

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.

Acoustic Emission is unlike most other non-destructive 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 per-

formed 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.

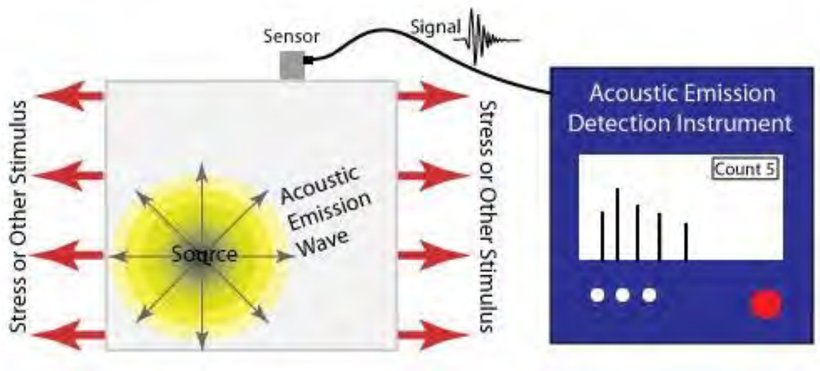


Figure 3.4 Simplified function of the AE-method (from www.nde-ed.org)

Table 3.5 Capabilities and limitations of the AE technique (http://nap.edu/24779)

Defects	AE cannot discern static defects, 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	N.A
Effect on corrosion	Cannot detect corrosion, but should be able to detect breakage due to corrosion over time
Accessibility requirement	Need of accessibility for the sensors, for large structures many sensors might be needed

3.2.3 Radiographic methods

Radiography

From 'A Practical Guide to Non-Destructive Examination of Concrete (2004) – Nordic Innovation Centre / Force Technology':

> *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 dependent 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.*

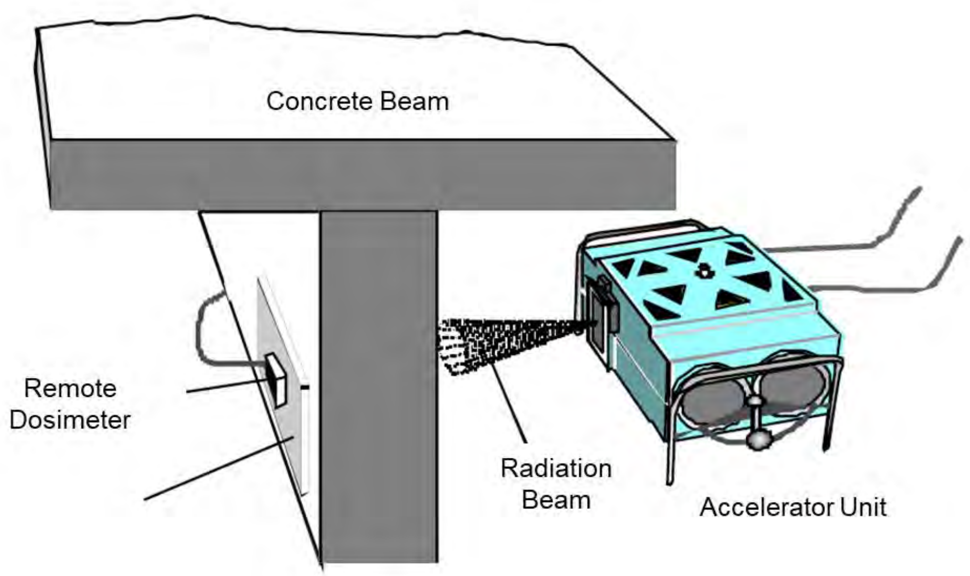
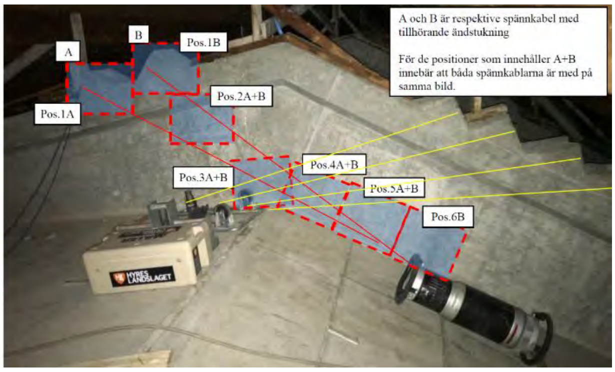


Figure 3.5 Typical set-up for radiography with a Betatron

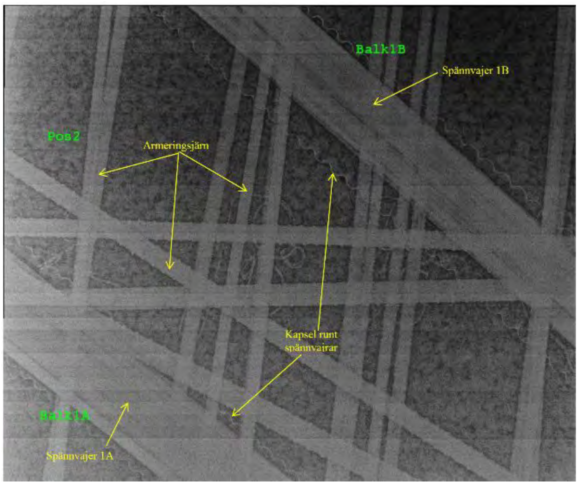
In order to define the optimum procedure for an inspection using radiography on concrete structures, several parameters must be determined:

- 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 required exposure time

In figure 3.6 typical results from using the radiography method are shown. The ducts and the tendons can clearly be seen and in table 3.6 the capabilities and limitations of the method are presented.



Equipment and different positions



Results. The duct and the tendons together with steel reinforcement can clearly be noticed in the photo

Figure 3.6 Example of results from the radiography method.

Table 3.6 Capabilities and limitations of the Radiography technique (partly from http://nap.edu/24779)

Defects	Radiography can detect defects in concrete, but high moisture content increases the scatter and the exposure time. It is easier to detect a void of a certain size in homogeneous concrete with small aggregates, than in a non-homogeneous concrete with large aggregates.
Duct location	Internal ducts
Duct type	Metallic ducts can clearly be seen, but plastics will not be discerned clearly on the screen.
Effect of concrete cover	The method is affected by the concrete cover. Conventional X-ray system may inspect concrete with thickness of roughly 200 mm. Using high power radiation sources (Betatron equipment) may increase the maximum thickness to be inspected to above 0,5 m of concrete.
Effect of layered ducts	Ducts behind other ducts can be discerned using radiography. The position of the radiation source with respect to the object or internal structure can be varied such that any area that is shadowed may be inspected.
Effect of reinforcement	Reinforcing can make it difficult to distinguish a void in an image 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 in extensive corroded areas. Can give notice for possible corrosion at areas of voids etc.
Accessibility requirement	Access is necessary to both sides of the object to be inspected. A detector or film is needed on one side of the object while a radiation source is placed on the other side of the object. The equipment is heavy, quite complicated and needs access to high voltage. Special permits and protection measures are required due to X-rays.

3.2.4 Electromagnetic methods

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, 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 3.7 an example of the use of GPR is shown. The evaluation of the results is very dependent on the existing algorithms and user interface.

Table 3.7 Capabilities and limitations of the GPR technique (partly from http://nap.edu/24779)

Defects	GPR can detect defects in concrete such as e.g. air voids as well as cast-in objects such as, steel, plastic, honeycombing. Similar defects can be detected in external plastic ducts (HDPE), however with moderate accuracy. It is also possible to locate and distinguish between different materials, e.g. a plastic duct from a metal duct. The GPR technique not very suitable for detection of defects on strands.
Duct location	Internal and external ducts
Duct type	For locating ducts: both metal and nonmetal ducts For identifying defects: only 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 but at the expense of reliable signals of smaller objects in greater depths. Measurements made with high or low frequencies effect the receiving signal. With higher frequencies comes an enhanced signal for smaller objects at smaller depths whilst measurements with low frequencies enable detection of larger objects at greater depths, however with a low resolution.
Effect of layered ducts	Layered ducts obstruct measurements
Effect of reinforcement	A considerable amount of steel reinforcement bars will hinder measurements. Objects underneath the reinforcement will be hard to distinguish.
Effect on corrosion	Can possibly detect corrosion but needs larger areas. Results are difficult to evaluate and show a large uncertainty.
Accessibility requirement	Ground coupled GPR: For GPR inspection, it is required that the wheels of the device are 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. Air coupled GPR: Is almost exclusively used on the bridge deck and therefore requires a bridge closed from traffic.

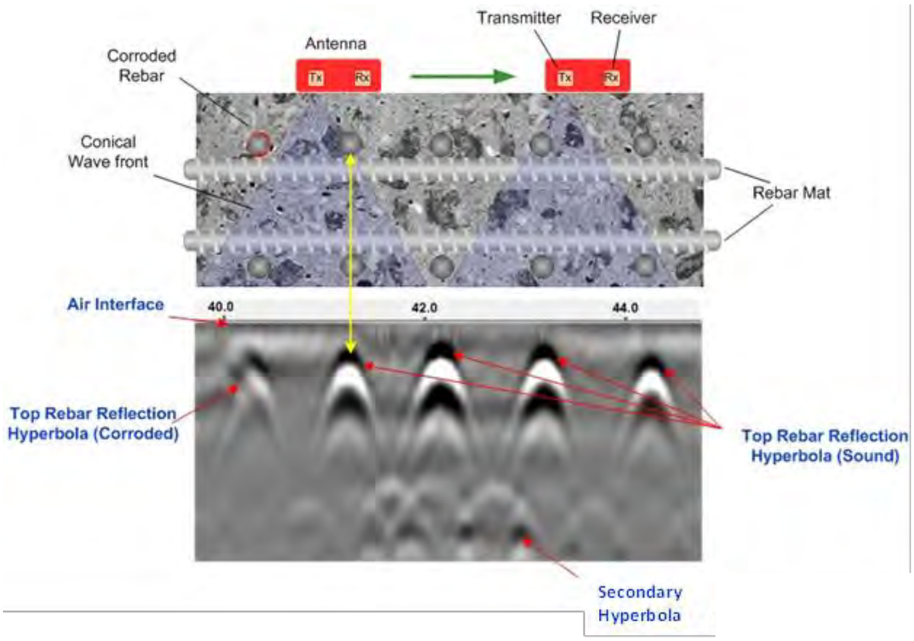


Figure 3.7 Ground penetrating radar, from (https://fhwaapps.fhwa.dot.gov/ndep/DisplayTechnology.aspx?tech_id=25, 2019)

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.

From NCHRP Inspection Guidelines for Bridge Post-Tensioning and Stay Cable Systems using NDE Methods (2017) (http://nap.edu/24779):

> *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 3.8.*

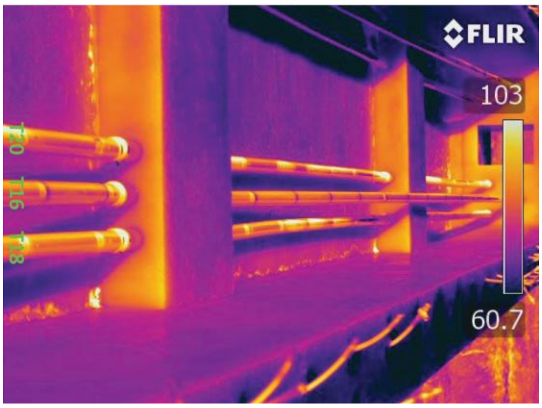


Figure 3.8 IRT Technique using FLIR T640 Infrared Camera and sample of infrared image of external tendons, from (http://nap.edu/24779)

Table 3.8 Capabilities and limitations of the IRT technique (http://nap.edu/24779)

Defects	IRT can locate void and water infiltration defects in nonmetal ducts, although the differentiation between these defects cannot be made. It cannot locate strand defects and hardly detect compromised grout. IRT cannot be used when ducts are placed deep within the concrete. It may be able to detect voids and water infiltration in the end caps of the anchorage regions.
Duct location	External ducts. Internal ducts under certain favorable circumstances.
Duct type	Nonmetallic ducts.
Effect of concrete cover	Significant effect
Effect of layered ducts	Not possible
Effect of reinforcement	Significant effect
Effect on corrosion	N.A
Accessibility requirement	IRT demands only a small accessible area. Temperature differences should be avoided.

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. Cover meter surveys form part of most concrete condition surveys of buildings or structures. Cover meter surveys can also locate main and secondary reinforcement, determine bar sizes and spacing and determine minimum cover thickness and cover variability across an element. The position of reinforcing steel and pre-stressing strands is sometimes also required, in order to circumvent them during core sampling or other tests which may be affected by their presence (concrete resistivity).


Figure 3.9 Use of cover meter, from (http://www.proceq.com)

Table 3.9 Capabilities and limitations of the cover meter technique (partly from http://nap.edu/24779)

Defects	CM cannot detect defects in concrete
Duct location	Internal ducts
Duct type	Metallic ducts
Effect of concrete cover	The concrete cover affects the accuracy of the method
Effect of layered ducts	N/A
Effect of reinforcement	In largely reinforced areas, the size and no. of reinforcement can be difficult to evaluate.
Effect on corrosion	N/A
Accessibility requirement	The equipment is normally handheld and will easily access most areas. However, for large areas the method can be time consuming.

3.2.5 Magnetic methods

MMFM-Permanent magnet (MMFMP)

From NCHRP Inspection Guidelines for Bridge Post-Tensioning and Stay Cable Systems using NDE Methods (2017) (http://nap.edu/24779):

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 (magnetic flux leakage) that supplements the detection of defects.

Table 3.10 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	External ducts
Duct type	Metal and nonmetal ducts
Effect of concrete cover	N.A.
Effect of layered ducts	N.A.
Effect of reinforcement	N.A.
Effect on corrosion	The method locates metal loss, but cannot differentiate between corrosion, section loss, and breakage of single wires
Accessibility requirement	For the investigation of external ducts, a clearance of approximately 300 mm radius is required from the center of the duct.

Magnetic flux leakage (MFL)

From NCHRP Inspection Guidelines for Bridge Post-Tensioning and Stay Cable Systems using NDE Methods (2017) (http://nap.edu/24779):

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 Hall-effect sensors) between the poles of the magnet is sensitive to this change in magnetic field and indicates the leak.

Table 3.11 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	External ducts and internal ducts at a maximum depth of approximately 150 mm.
Duct type	Metal and nonmetal ducts.
Effect of concrete cover	The concrete cover itself does not interfere with the inspection. The thickness of the cover will though limit the inspection depth.

Effect of layered ducts	Other ferrous materials on top of or very close to the ducts makes the interpretation of data difficult and may lead to difficulties in detecting
Effect of reinforcement	Other ferrous materials (i.e. rebars) on top of or very close to the ducts makes the interpretation of data difficult and may lead to difficulties in detecting strand defects.
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. For the investigation of internal ducts, everything including concrete cover leading to a larger distance between the strand to inspect and the sensors "lift-off" will reduce the possibility inspection depth. Inspection of internal ducts is limited to a "lift-off" of the sensors to approximately 150 mm. For the investigation of internal ducts, everything including concrete cover leading to a larger distance between the strand to inspect and the sensors "lift-off" will reduce the possibility inspection depth. Inspection of internal ducts is limited to a "lift-off" of the sensors to approximately 150 mm.

MMFM-Solenoid (MMFMS)

From NCHRP Inspection Guidelines for Bridge Post-Tensioning and Stay Cable Systems using NDE Methods (2017) (http://nap.edu/24779):

> *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.*

Table 3.12 Capabilities and limitations of the MMFMS technique, (http://nap.edu/24779)

Defects	MMFM-solenoid can locate the strand defects in both metal and nonmetal 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. MMFM solenoid does not detect grout defects in ducts.
Duct location	External ducts.
Duct type	Metal and nonmetal ducts.
Effect of concrete cover	N.A.

Effect of layered ducts	N.A.
Effect of reinforcement	N.A.
Effect on corrosion	MMFM-solenoid consistently locates corrosion, section loss, and breakage 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.

3.2.6 Electrochemical methods

Electrochemical impedance Spectroscopy (EIS)
From NCHRP Inspection Guidelines for Bridge Post-Tensioning and Stay Cable Systems using NDE Methods (2017) (http://nap.edu/24779):

> *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.*

Table 3.13 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	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 ability to drill small holes into the external duct. The holes must be sealed after the EIS testning.

Potential Mapping (PM)
This electrochemical method can be used for the determination of defects related to corrosion of reinforcement. 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 3.10. Limitations of PM are the conductive cover between reinforcement and surface, membranes, asphalt or other sealing parts.

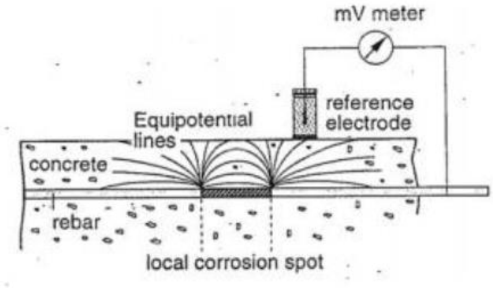


Figure 3.10 Principle and main components of half-cell potential measurements. Reference electrode, high impedance voltmeter, connection to the rebar. Adopted from RILEM TC 154-EMC, (Elsner, 2003)

Table 3.14 Capabilities and limitations of the PM technique

Defects	The PM method can detect corrosion. It cannot 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	N.A.
Effect on corrosion	Can detect corrosion
Accessibility requirement	This is an intrusive method.

3.2.7 Intrusive methods

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.

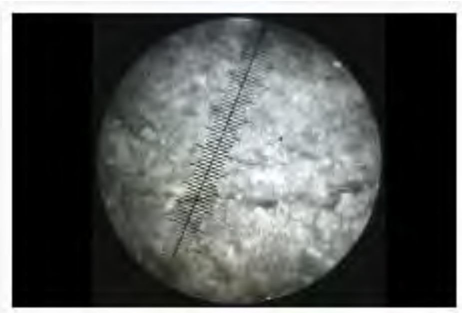


Figure 3.11 Use of endoscope identifying a crack in concrete, from (http:// www.nishimatsu.co.jp)

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. A variant of this system is the endoscopic video made through a miniaturised camera directly connected to a device recording images. In table 3.15 capabilities and limitations of the ES technology is presented.

Table 3.15 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	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, it can often be used as a complementary to other methods.

Residual Prestressing Force (RPF)

A partly destructive method that might be used to investigate the prestressing force in the tendons could be valuable for evaluation. However, as this method is expensive and complicated to use, it is recommended that the structure is thoroughly assessed, and re-calculation of the capacity is made before the RPF method will be considered.

When assessing post-tensioned 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. Therefore, it can be useful to calibrate theoretical 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 ε_{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 φ giving the increase of the elastic strain $\varepsilon_{el} = \sigma/E_c$ calculated as the stress σ 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 ε_{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 $\chi = 1 - \sigma/\sigma_0$, σ is the stress after a certain time and σ_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 post-stressing cable is below.

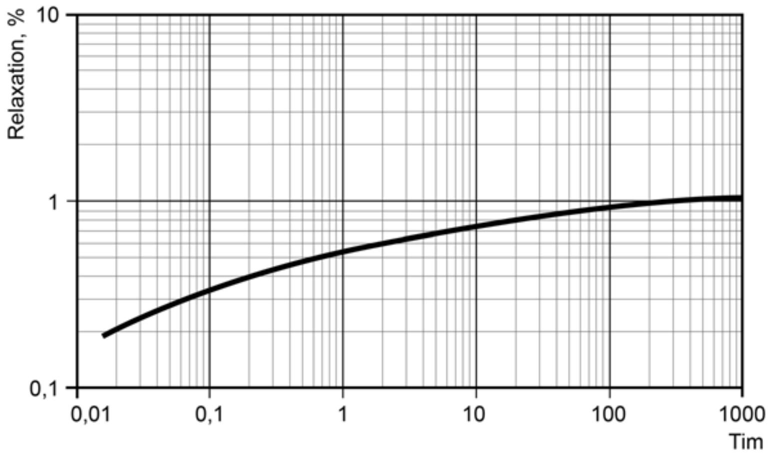


Figure 3.12. Relaxation of a ϕ12,9 mm cable post-tensioned at 20ºC to 70 % of the ultimate strength (here 1860 MPa). BHM-I (2017).

Several methods can be used to determine the remaining force. Examples of non-destructive and destructive methods are given in figure 3.13, 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.

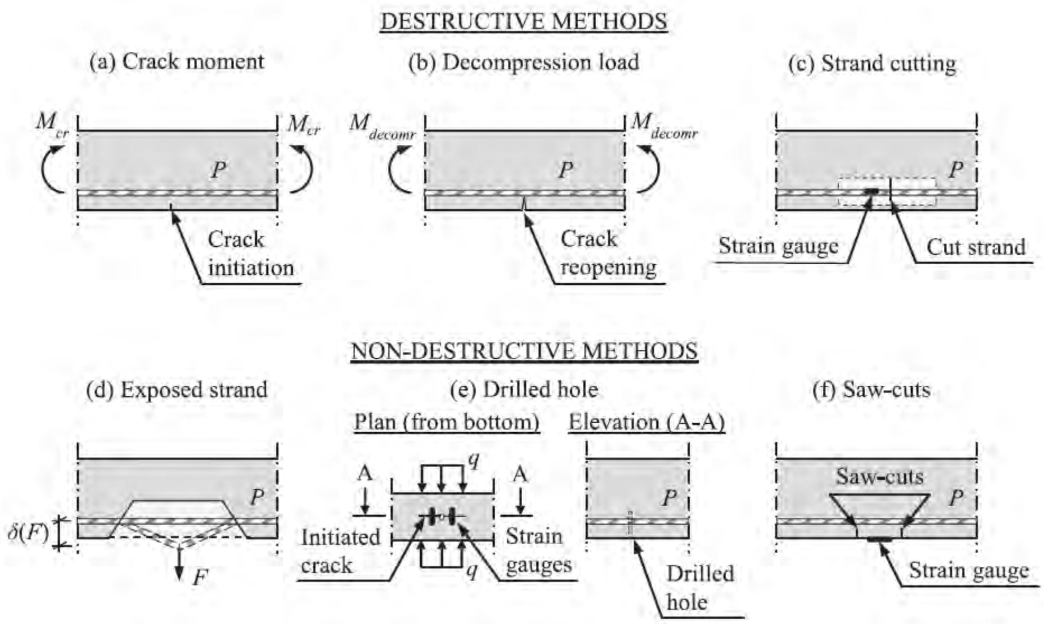


Figure 3.13. Methods to determine residual prestress force P in concrete members experimentally: (a) crack moment method, (b) decompression-load method, (c) strand cutting method; (d) exposed strand method, (e) drilled hole method and (f) saw-cut method, Bagge et al.(2017).

- **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 post-tensioned 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 to be preferred. 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 (3:1):

$$\sigma = \frac{P}{A} + \frac{Pe_P y}{I} + \frac{M_R y}{I} + \frac{M_G y}{I} + \frac{M_Q y}{I} \tag{3:1}$$

where σ 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-post-tensioned 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 3.14. Therefore, using this method, it is possible to avoid damage to the structure which might be difficult to repair.

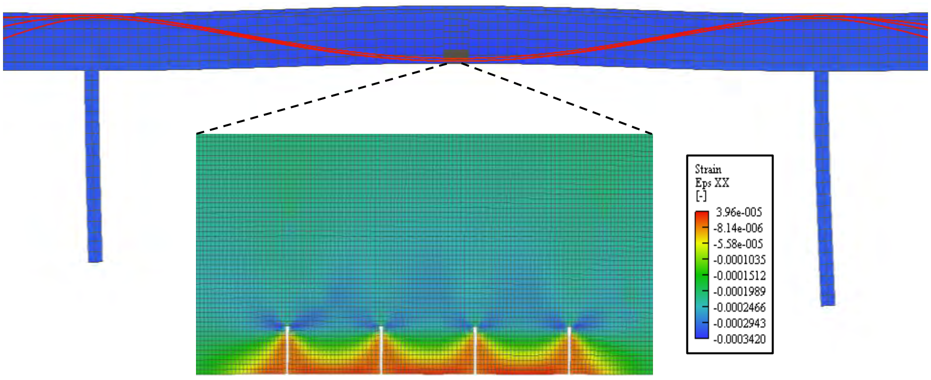


Figure 3.14 A part of an FE model for simulation of the strain distribution as saw-cuts are introduced transversally at the base of a concrete beam (Bagge et al. 2017).

<u>Decompression-load method</u>
As with the saw-cut method, Navier's formula as shown in Equation (3:1) 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). Results from the test on the Kiruna Bridge show that the remaining force varies between some 60 to 90 %, Bagge (2017). Comparisons between field results and FE calculations are given in Figure 3.15.

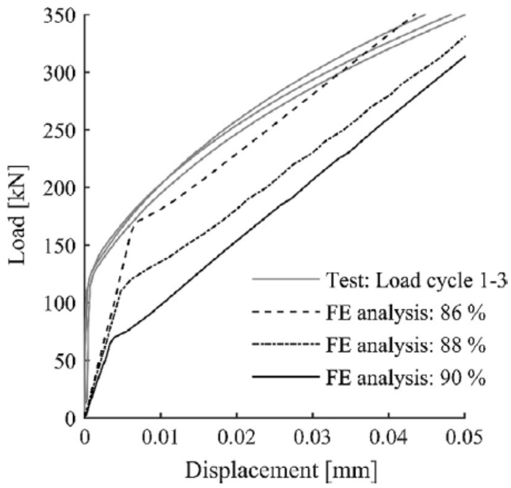


Figure 3.15. Load-displacement behaviour at crack opening according to test at the Kiruna bridge and FE analyses at indicated level of prestress losses, Bagge et al. (2017).

The influence of freezing and thawing on the post-stressing 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.

3.3 Monitoring techniques

Five categories developed by (Webb et. al., 2015), are shown in table 3.16 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 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 3.16 Categories of SHM systems, (Webb et. 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 difficulty is that structural responses are relatively 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/and implementing an SHM strategy the following 7 questions should be asked, Why?, Where?, Who?, What?, When?, How? and Which?, see also figure 3.16, (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 a new or existing structure but also answers to what system is going to be used to get most out of the investigation.

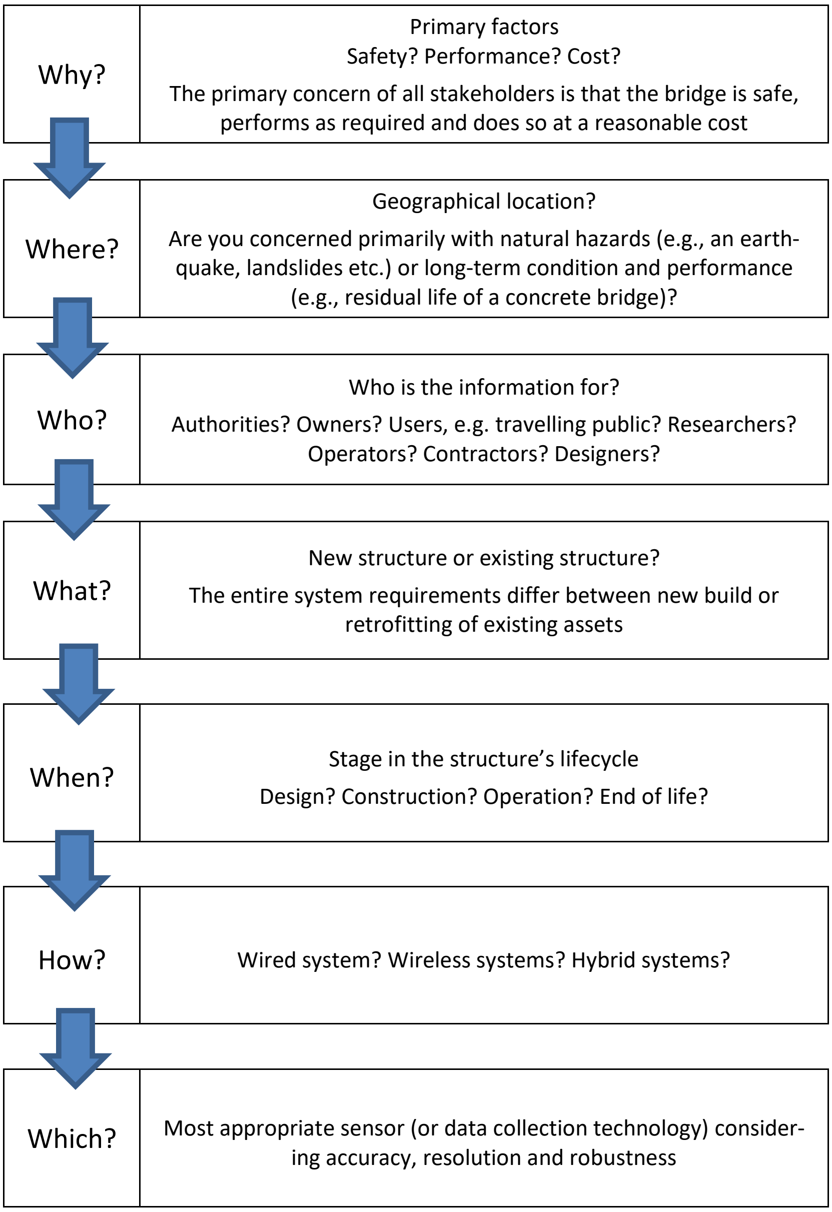


Figure 3.16 Main considerations when examining whether or not to employ a structural health monitoring system, from (Vardanega et. al., 2018)

Some monitoring techniques which are not treated in this report may 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, these technologies can so far only detect global and surface behaviour, not the status of tendons cast into concrete. Despite this, these technologies in combination with the ones discussed below can be a way forward to increase the knowledge about our existing structures.

4 Selection of methods

4.1 Findings of pre-study

The pre-study of reports and publications led to a list of suitable methods and techniques summarized in table 4.1. It was noted that many attempts have been made to locate corrosion in ducts and tendons, but in studied literature no method or application was found that could be reliably used for this purpose. In literature only methods to identify corrosion on external ducts were found, and since this was not the purpose of the current project, they were discarded in our selection of methods.

Table 4.1 NDT methods and assessment of their usefulness. Red marked methods were discarded.

Method	Localisation of voids in concrete	Detecting post-tensioning system	Detecting corrosion	Quantifying damages in tendons	Used in the evaluation
Manual Methods					
Visual Inspection	N.A	N.A	N.A	N.A	X
Acoustic Methods					
Impact Echo	X	X	N.A	X	X
Ultrasonic Pulse Echo	X	X	N.A	X	X
Spectral Analysis of Surface Waves	X	X	N.A	X	N.A
Acoustic Emission	N.A	X	N.A	X	N.A
Radiographic Methods					
Radiography	X	X	X	X	N.A
Electromagnetic Methods					
Ground Penetrating Radar	X	X	X	X	X
Infrared Tomography	X	N.A	N.A	N.A	N.A
Cover Meter	N.A	X	X	N.A	X
Magnetic Methods					
Magnetic Flux Leakage	N.A	N.A	X	X	N.A
MMFM-Permanent magnet	N.A	N.A	X	N.A	N.A
MMFM-Solenoid	N.A	N.A	X	N.A	N.A
Electrochemical Methods					
Electrochemical Impedance Spectroscopy	N.A	N.A	X	N.A	N.A
Intrusive Methods					
Endoscope	N.A	N.A	X	X	X
Residual Prestressing Force	N.A	N.A	N.A	X	N.A
Potential Mapping	N.A	N.A	X	N.A	N.A

X = applicable N.A = not applicable

Some comments to various studied methods:

- Magnetic methods like Magnetic Flux Leakage, MMFM Permanent Magnet and MMFM Solenoid can only be used to find corrosion on external tendon ducts. Corrosion cannot be located on internal tendon ducts.
- Spectral Analysis of surface waves is similar to Impact Echo (IE) with the difference that the surface wave is the point of interest. The method is good to measure concrete thickness and to estimate crack depth. To locate voids in ducts the method is not as accurate as IE, therefore the method was discarded during the selection.
- Acoustic emission is used to “listen” to the concrete, and unlike IE no sound waves are supplied from an impactor. Instead, the cracks emit soundwaves which are analyzed and interpreted. The sensors need to be installed over a long time and in large structures many sensors might be needed. When interpreting the results, the inspector needs to filter the noise from traffic, existing cracks etc. This method is more a surveying system and in order to obtain quantitative results about size, depth, and overall acceptability of a part, other NDT methods (often ultrasonic testing) are necessary.
- Radiographic methods can detect defects in concrete and voids in the tendon ducts. However, the method is expensive and time-consuming, and its utilization requires knowledge of the exact location of potential damages. In large structures, it will be necessary to use other NDT equipment to define the areas to be inspected. For testing, both sides of the object to be inspected must be accessible. In order to avoid exposure to radiation, the use of radiography demands a shutdown of the object to be investigated. It is therefore not feasible to carry out an entire bridge inspection with radiography, but in special occasions it can be of great use.
- Electrochemical impedance spectroscopy is only applicable to external ducts and requires physical access to the duct that is being inspected and the ability to drill small holes into the ducts.
- Infrared thermography is best applied to external ducts with large voided areas or internal ducts with cover less than 5 cm. Therefore, it is not applicable in most of the bridge structures for locating voids in internal ducts.

4.2 Selected methods

Results from the pre-study were combined with the professional experience of the authors in order to define which methods to select for further investigation (laboratory- and field tests). These are listed in the following paragraphs.

The actual review on previous work in NDT to detect ducts, tendons and corrosion in post-tensioned systems for large concrete structures has concluded that there is no method that alone can trace damages in the post-stressing system. A combination of methods is required, and the work must be carefully planned and executed. The post processing of the data is important. Another important factor for selecting methods was the capability to perform the inspection from only one side.

The test methodology that we decided to use in the initial work includes the below listed methods.

4.2.1 Visual inspection

Visual inspection is essential for assessment of concrete structures. During visual inspection, not only the structure itself is investigated but also the pre-conditions like equipment and scaffolding needed to access all areas of the structure, access to electricity and water, etc. The visual inspection should also clarify the congruence with drawings, earlier detected defects and repairs and any visual defects that have not been detected before.

A visual inspection consists of two parts, both of which are important in order to be able to conduct the inspection of the tendons in a time-efficient manner. The first part is the preparation, which consists of verification of drawings and documentation, is done well in advance of the actual inspection on site. The main goal of the verification of drawings is to plan the work and select the number of interesting test areas. This must be carried out accurately in order to prepare for the difficulties and limitations that may arise during the test even before the inspection.

The next part is the actual visual inspection, trying to detect damages that may have a negative impact on the structure, including investigation of cracks originating from impaired functioning of the tension reinforcement. The intended test areas should be verified and possibly moved, depending on accessibility.

4.2.2 Cover meter

The cover meter is an electromagnetic method used to locate the placement of the steel reinforcement in plane (x-y-directions) and depth (z-direction) and to define the thickness of the cover layer. The cover meter is easy to use, cost effective and accurate. Therefore, it is a useful equipment for the inspector.

It has a limit depth of about 80 mm which in most cases makes it difficult to locate ducts for post-stressing tendons. Although a cover meter returns clear signals of parts cast into the concrete, it is not possible to define whether rebars, tendon ducts or debris are recorded. Nevertheless, the concrete cover meter is an important part of the inspection as it is a very accurate method for localization, with an accuracy of ± 1–4 mm, depending on the thickness of the cover layer. In figure 4.1 reading from a cover meter is shown.

Figure 4.1 Cover depth determination with cover meter

The cover meter is usually used before application of other methods such as ground penetrating radar (GPR). The use of GPR requires a calibration against the actual cover layer, taking account of the signal speed through the concrete. With a known value of the reinforcement depth, the dielectric constant can be calculated and used in the analysis of the GPR results. Therefore, an investigation of the post-tensioned tendons always starts with the cover meter before using a GPR for location of ducts.

4.2.3 Ground penetrating radar

The GPR method involves emitting electromagnetic pulses from an antenna and receiving the reflected pulses from internal reflectors. GPR is a fast and reliable method for locating tendon ducts. The limits in depth are about 250 mm and it needs to be calibrated to a known depth. However, no calibration is required if the ducts are only to be localized in a horizontal plane. Software programs can process the results, enabling a closer study of the scans. With a GPR, a good overview of the location of the reinforcement is obtained in a relatively easy manner.

To be able to determine more precisely the depth of the reinforcement and the cable ducts, more advanced knowledge of the equipment and its use is required. Knowledge of design and building technology is recommended to be able to distinguish the reinforcement from the tendon ducts. Successful utilization of the equipment is dependent on the knowledge of the operator, both with respect to the handling of the equipment and the goal of the scanning (e.g. localize tendon bars). For example, if the horizontal reinforcement is going to be located with GPR, the scanning direction must be vertical.

GPR is an important part of the inspection, mainly for locating the ducts, which would be quite time-consuming with the UPE (Ultrasonic Pulse Echo). In figure 4.2 a typical result from a GPR measurement is shown.

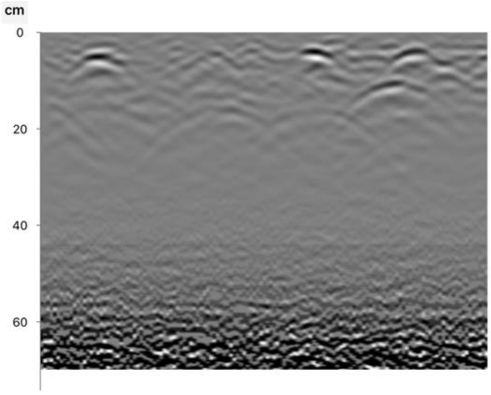


Figure 4.2 Example scan with a GPR equipment.

4.2.4 Ultrasonic Pulse Echo

The ultrasonic method uses a dry-point-contact transducer that generates shear waves. A group of transducers emits a stress pulse into the specimen. As the waves propagate, areas with changes of impedance reflect portions of the wave, and these reflections are captured by a sensor. Ultrasonic Pulse Echo is used to locate air voids in the concrete and tendon ducts. When the duct is correctly grouted the signal travels through the duct, resulting in a weak signal. If there is a void, the reflecting signal is stronger. Unfortunately, even small voids result in a strong reflecting signal, resulting in similar signals for ducts filled with air as for ducts filled to a high degree (but not completely) with grouting.

The UPE is considerably more powerful than GPR and can penetrate depths of approximately 1 meter, which is sufficient for most applications. Difficulties in finding the second layer of tendon ducts is often related to shading from the first layer. If the tendon layers have been placed with a certain offset, they may be scanned successfully. The advantage of UPE in finding voids is that scanning can be executed along with the tendon ducts, thus covering large areas and enabling to define size and length of the voids. In figure 4.3 an example scan with UPE after post processing is shown.

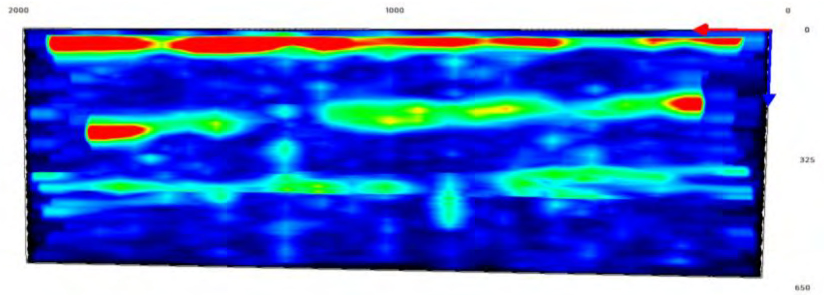


Figure 4.3 Example of scan with UPE

4.2.5 Impact Echo

In order to confirm the results from the ultrasonic testing, the impact echo (IE) method is used. The method is based on arrival of reflected stress waves and can locate the depth of internal delamination and voids in the duct. This is important, because depending on the reflection wave, we can ascertain if grouting in the ducts is truly missing.

IE can spot the difference between a grouted and an empty duct and – to a certain extent, also in rare cases identify partly grouted ducts. However, IE requires considerable experience and training to be able to interpret the results. Several signals read from the IE are irrelevant and shall be ignored.

The equipment is normally calibrated based on the thickness of the structural part to be investigated. Also, an alternative method for calibration may be used, based on several preparatory tests to find the correct thickness frequency. In figure 4.4 a reading from the IE is shown.

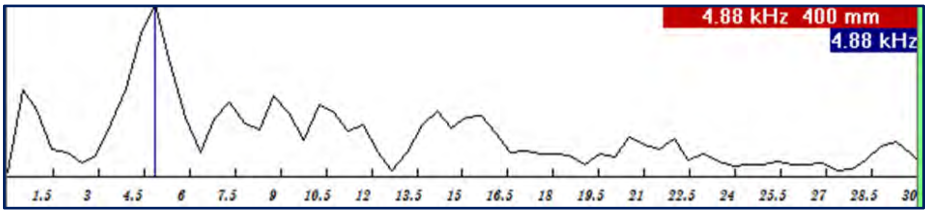


Figure 4.4 Thickness determination with IE

IE is a method that gives a result in one single test point, which makes it unsuitable for testing larger areas. It should therefore only be used for bridge inspections to verify the results obtained by UPE. For this purpose, IE is very accurate.

4.2.6 Drilling and use of endoscope

After detecting voids in the duct, a small hole is drilled through the concrete and into the duct. Then an endoscope is used to investigate possible damages in the tendon, i.e. is there corrosion or is the tendon broken? This is a critical step of the inspection, but it is a necessary step and needs to be done. When drilling, the depth of the duct needs to be known and it must be accurate.

Drilling can also be made with a core drill (Ø100-150 mm). The advantage is direct accessibility to the duct wall, enabling a more thorough inspection of the damage mechanisms on the wall and tendons. Moreover, samples of the grout can be taken, e.g. for measurement of chloride content.

5 General framework for inspection

5.1 Objectives

The purpose of inspections and resulting assessment shall be defined in advance. The objective of the assessment of an existing structure in terms of its required future structural performance shall be specified in consultation with the client (the owner, the authority, insurance companies, etc.) based on the following performance levels:

a) safety performance level, which provides adequate safety for the users of the structure;
b) continued function performance level which provides continued function for special structures such as hospitals, communication buildings or key bridges;
c) special performance requirements of the client related to property protection (economic loss) or serviceability. The level of this performance is generally based on life cycle cost and special functional requirements.

Authors' note: Usability of different inspection methods is mainly dependent on construction, configuration and accessibility of single bridge elements and not of the type of bridge.

5.2 Procedure

The procedure depends on the assessment objectives (see 5.1), and on specific circumstances (e.g. the availability of the design documents, the observation of damage, the use of the structure). A site visit is recommended prior to initiating the procedure.

The assessment is carried out taking the actual conditions of the structures into account (see the flowchart in figure 5.1) and is composed in general of steps a) to f):

a) specification of the assessment objectives;
b) scenarios;
c) preliminary assessment:
   1) study of documents and other evidence,
   2) preliminary inspection,
   3) preliminary checks,
   4) decisions on immediate actions,
   5) recommendation for detailed assessment;
d) detailed assessment:
   1) detailed documentary search and review,
   2) detailed inspection and material testing,
   3) determination of actions,
   4) determination of properties of the structures,
   5) structural analysis,
   6) verification;
e) results of assessment:
   1) report,
   2) conceptual design of construction interventions,
   3) control of risk;
f) repetition of the sequence if necessary.

The procedure outlined above may be applied to both the assessment of one specific structure and the assessment of a group of structures. A flowchart for the procedure is presented in figure 5.1.

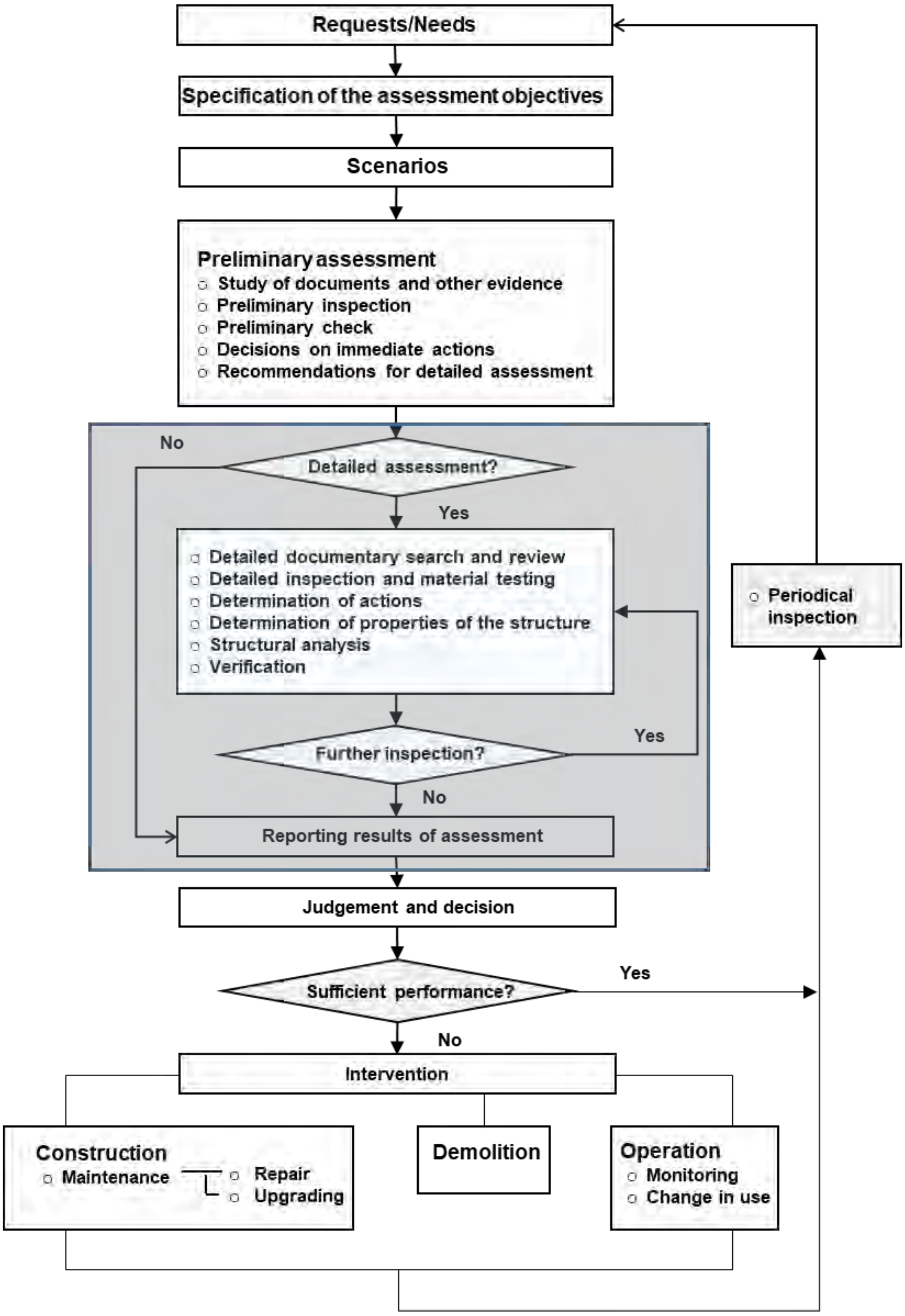


Figure 5.1 Assessment of concrete structures in general (after ISO 13822:2012).

In figure 5.1 the detailed assessment including reporting results of the assessment has been marked. For this part, with focus on assessment of post-tensioned concrete structures, a more detailed assessment procedure is suggested, see figure 5.2.

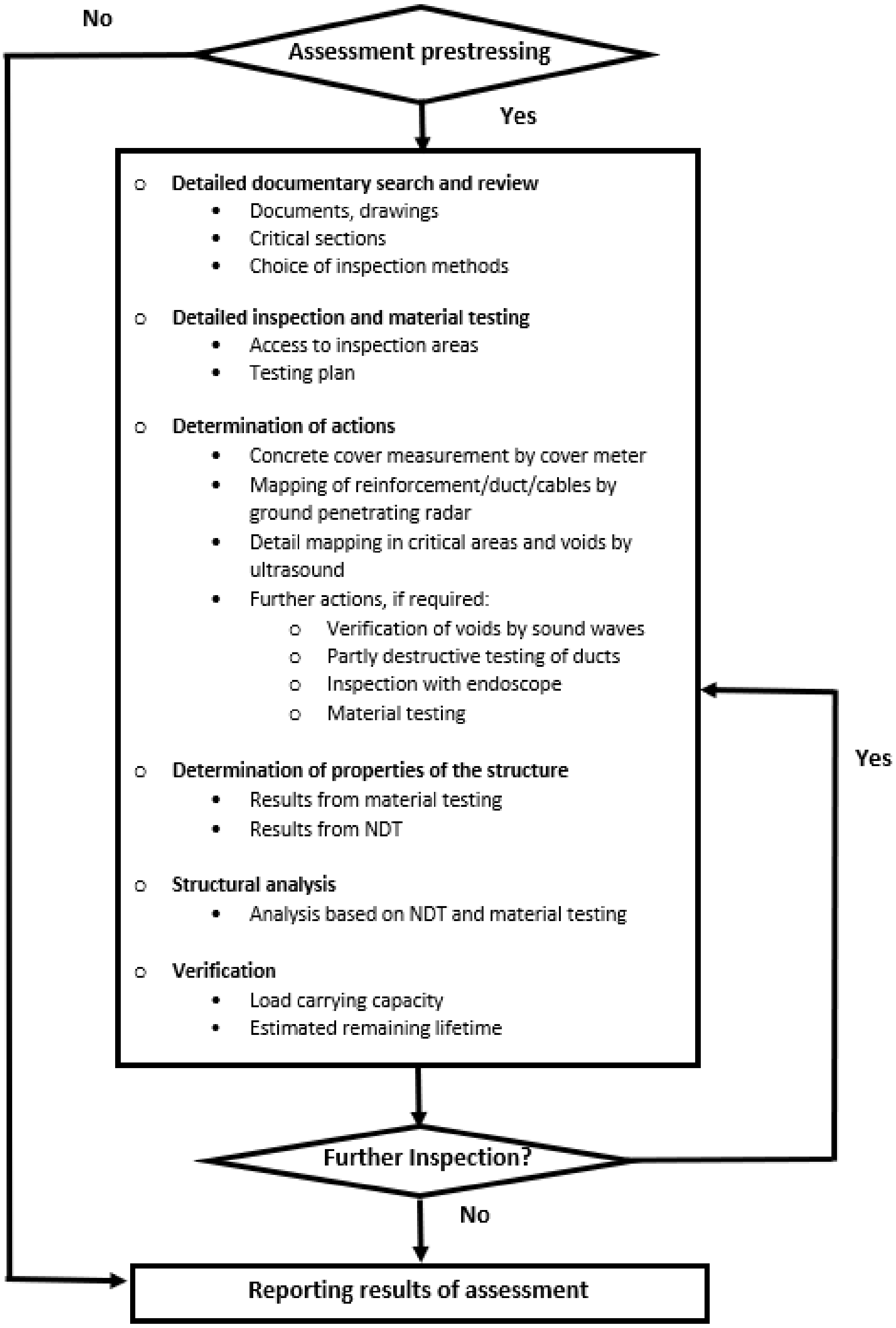


Figure 5.2 Assessment of post-stressing systems in concrete structures, developed from figure 5.1.

When planning the site assessment of a post-tensioned concrete structure, it is important to study existing documents, repairs, drawings etc. and if possible, identify critical sections where voids, corrosion or other damages may occur. At this stage, the inspection methods to be used should be chosen. It is also suggested that some material samples will be taken.

From our experience in field projects the placement of rebars, ducts and tendons can clearly be identified with GPR in combination with a cover meter to distinguish ducts and rebars and providing a clear view of the concrete cover.

It is important at this stage to mark the placement of the tendons along the bridge, both in plane and vertically (all three directions). The ducts/tendons are then mapped with ultrasound and the verification of detected voids is made by sound waves (Impact Echo). These areas are then clearly marked on the structure. Before any repairs are carried out it is suggested to inspect the voids with an endoscope and describe the type of damage clearly. Finally, the results from the inspection are compiled in a report.

In some cases, a structural analysis has preceded the assessment with assumptions regarding the quality of the structure. After assessment the structural analysis should be updated. The analysis may also be verified with the aid of a proof test. Based on the results of the assessment, it should be possible to carry out a life cycle cost analysis (LCCA).

6 Mock-up tests

6.1 General

In order to further evaluate different NDT methods, tests have been carried out on two mock-ups. One mock-up was manufactured and tested at Luleå University of Technology (LTU), the other was manufactured and tested in Hamar. Detailed reports of these tests are attached in appendices A and B, while a summary of the results is given below.

6.2 Mock-up at LTU, Luleå

The results from Luleå were satisfactory although the configuration of the test specimen was not fully optimized for the equipment used. Important lessons learnt were related to utilization and limitations of the various equipment and possible difficulties to be expected ahead, especially during the upcoming field tests.

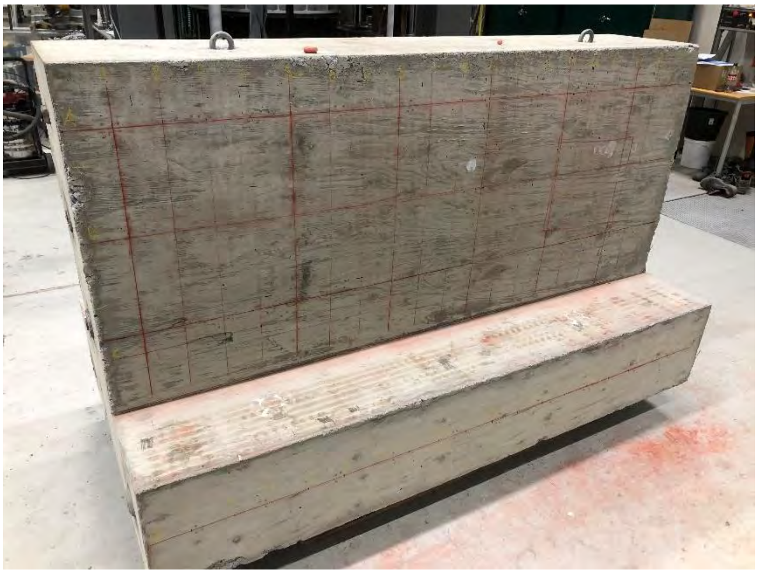


Figure 6.1 Picture of the mock-up specimen in Luleå

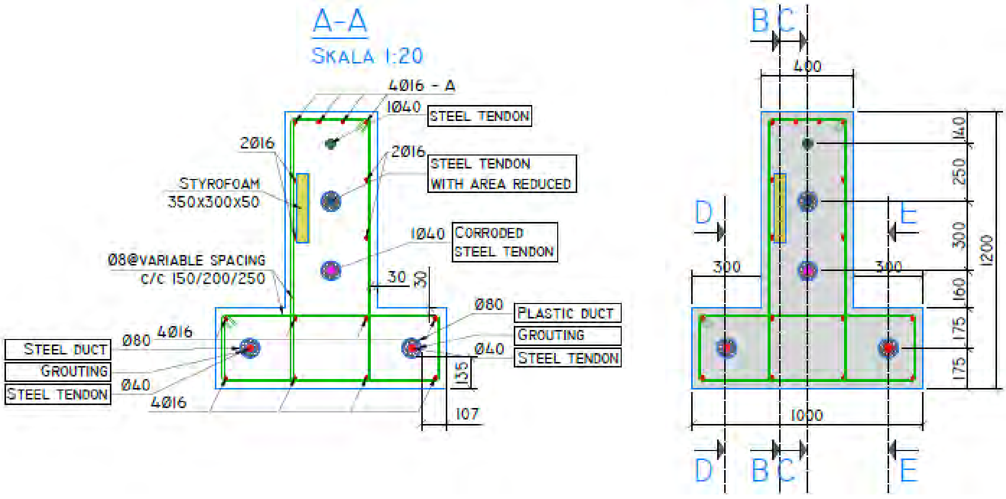


Figure 6.2 Drawing from mock-up specimen in Luleå

The results from UPE showed that we were able to map the test body well, but it was difficult to find the voids. Probably the voids are too small to be located clearly. Moreover, the ducts were located in sections that were partly difficult to access with the UPE scanner.

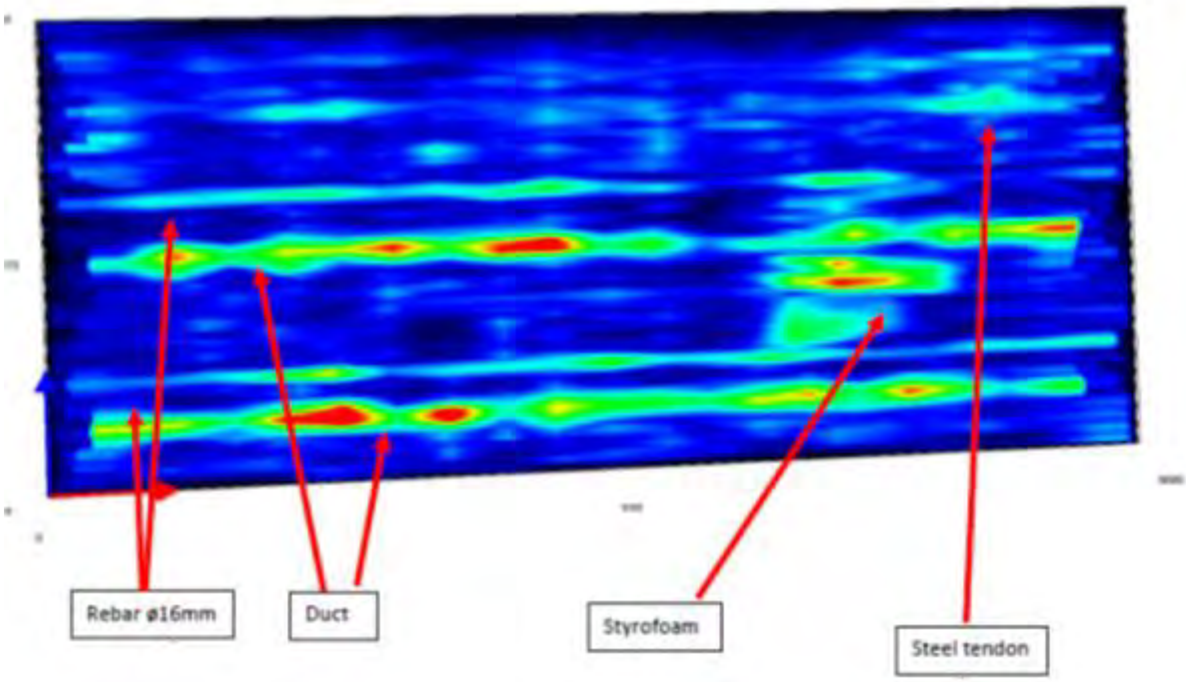


Figure 6.3 Results from UPE with reinforcement, ducts and delamination

A good understanding of the UPE´s functionality was obtained, even though we were unable to locate the voids in the mock-up.

The results with Impact Echo were interesting and here we managed to locate the voids. It was concluded that Impact Echo is a difficult method that requires training and experience in order to read and interpret the results.

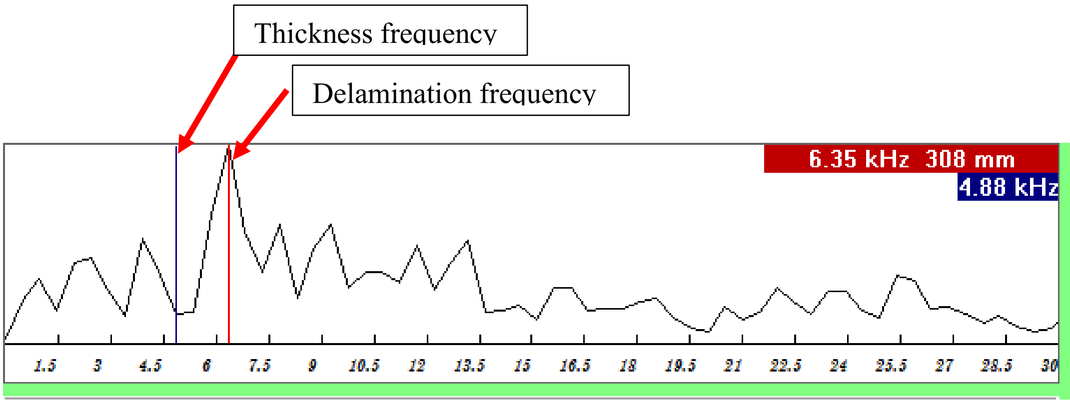


Figure 6.4 Results from IE with a delamination at a depth of 308 mm.

With GPE and UPE we can locate and find out where we need a deeper control with Impact Echo. Since the results from IE are difficult to interpret, we need to know at what frequency we should look for different results. In those cases where thickness is unknown, velocity from the UPE measurements may be used.

6.3 Mockup in Hamar

The mock-up had been casted by a manufacturer of equipment with UPE technique (UPE 1). Two metal ducts had been casted in the concrete, one duct was partially grouted (3/4) half of its length and the other half was empty. The second duct was fully grouted.

Tests were carried out with two devices (UPE 1 and UPE 2) from different manufacturers, both with UPE technique, and additional testing with one IE device.

The purpose was partly to further evaluate the methods (UPE and IE) on a different test block and also to compare different UPE devices in order to evaluate differences between the two.

Figure 6.5 Picture of the mock-up specimen in Hamar.

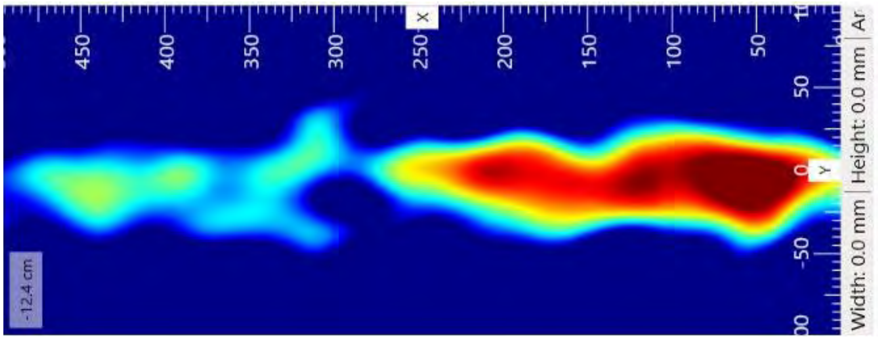


Figure 6.6 Image scan from UPE 1 on partially grouted duct. Fully grouted duct to the left (green) and void (1/4) in the duct to the right (red).

The scan with UPE 1 showed an image with a higher resolution of the empty section of the partially grouted duct compared with the image of UPE 2. This does not necessarily mean that UPE 1 is better than UPE 2 but can be explained by that the mock-up had been tested frequently with the UPE 1 prior to the test and the proper settings had probably been fine-tuned over the many occasions this occurred. With the UPE 2 the images were a bit vague in comparison but will probably improve with other settings.

The main advantage of UPE technique versus IE is that the UPE devices allow for faster measurements over larger areas. Also, results are clearer and easier to understand.

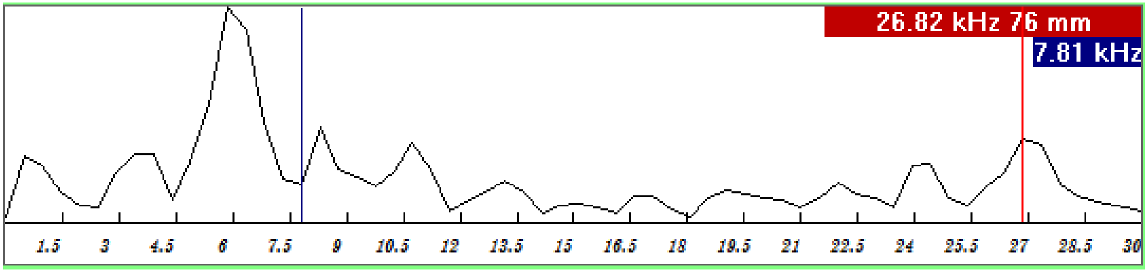


Figure 6.7 Results from IE with the empty duct placed at a depth of 76 mm

In order to make a visual inspection of the duct and tendons, a hole must be drilled through the duct wall. Because of the fragile tendons, damages on the tendons must be avoided at all costs. IE is a suitable method for verifying the drill depth obtained with UPE to minimize the risk for drilling too deep.

6.4 Mockup in Copenhagen

Tests with the Impact Echo method (ASTM C 1383) were made on a test body that had been casted by a manufacturer of instruments used for the method. The test body consisted of a 360 mm thick slab where two metal ducts (Ø100 mm) were casted in. One of the ducts was empty and the other one was filled with injection mortar and 10 pcs of strands (Ø16 mm). Tests were carried out mainly in order to further learn the method and instrument by manual tests.

The method´s strengths and limitations were also discussed with the engineers at the manufacturer. It was concluded that the method in many ways provides similar information as received with ultrasound (UPE) but that it sometimes may be useful to have both. For instance, when there is a lot of reinforcement bars, IE may be preferred due to the limited testing area it requires. Measurements with UPE require a sparser placement of the reinforcement bars because of the relatively large contact surface of the instruments on the market.

Also, IE is useful when drilling into ducts for visible inspection with endoscope or by core drilling. Identifying the exact placement of a duct is essential when drilling into a post-tensioned concrete element in order to avoid damaging the strands. Sometimes the received information obtained with UPE can be ambiguous and therefore it could be necessary to make a second test regarding depth and location with IE before drilling depth is determined.

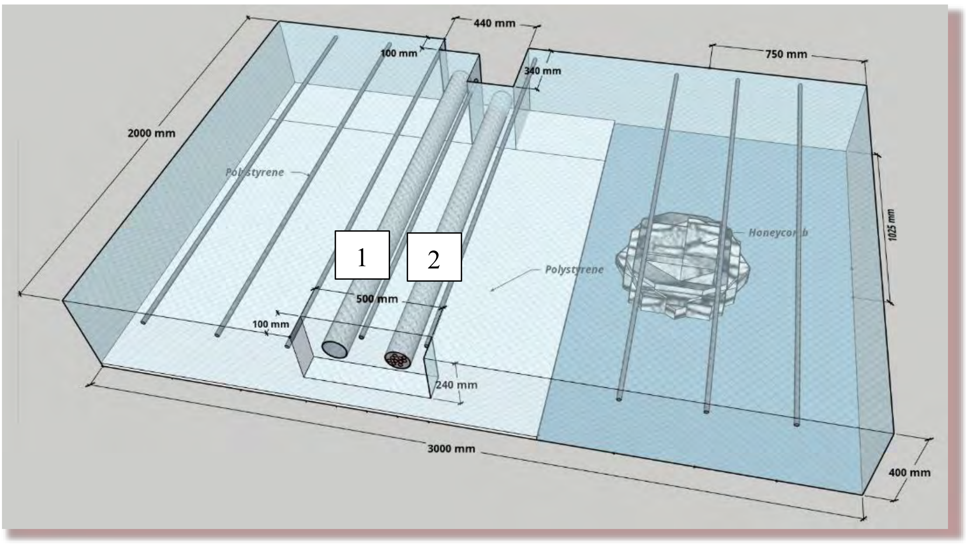


Figure 6.8 Geometry and dimension of test slab. 1) Empty duct. 2) Fully grouted duct.

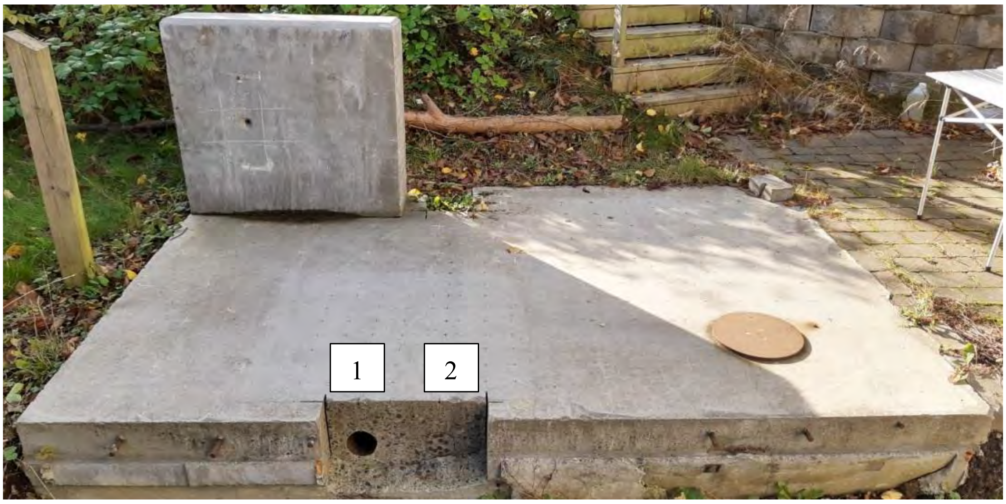
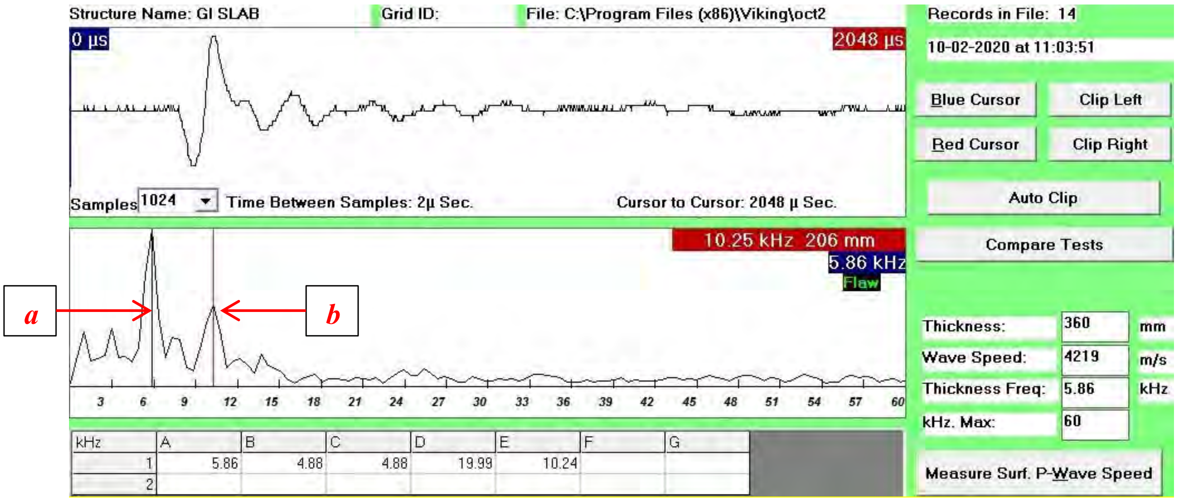
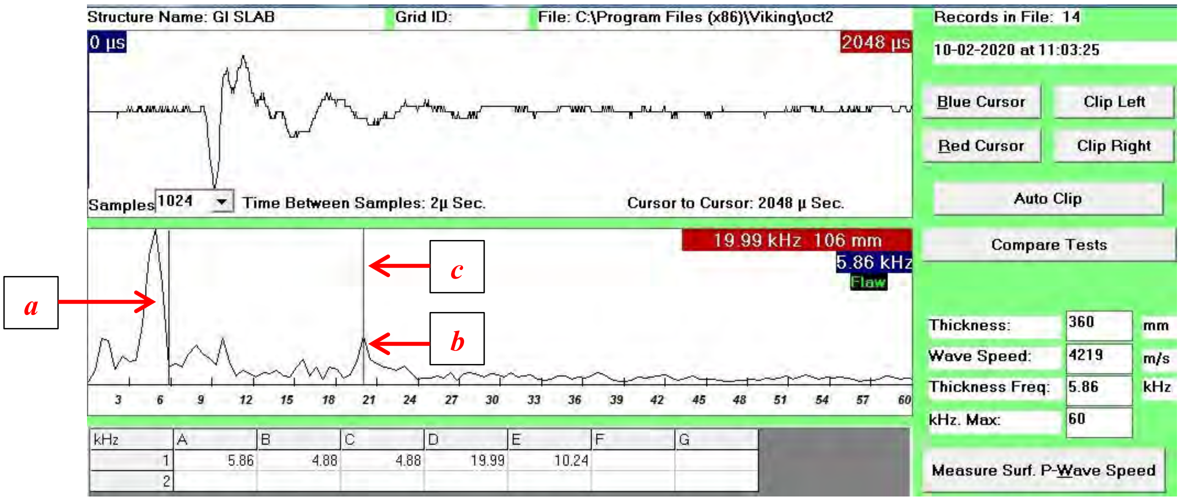


Figure 6.9 Slab. 1) Empty duct. 2) Fully grouted duct



kHz	A	B	C	D	E	F	G
1	5.86	4.88	4.88	19.99	10.24		
2							

Figure 6.10 Results from measurements on fully grouted duct with 12 mm impactor. Note that the thickness frequency peak (**a**) is not displaced related to the black cursor→ indicates no entrapped air. (**b**) Reflection from one of the strands in the duct.



kHz	A	B	C	D	E	F	G
1	5.86	4.88	4.88	19.99	10.24		
2							

Figure 6.11. Successful reading of an empty duct with 8 mm impactor. The thickness frequency peak (***a***) is shifted and there is a reflection peak (***b***) in the correct depth which clearly shows that there is a void and in which depth. Information on depth can be obtained by moving the red cursor (***c***) to the tip of the reflection peak.

7 Field tests

In addition to the mock-up tests, several field tests were performed. They are summarized below and presented in detail in the Appendices.

7.1 Farris bru, Larvik

In Larvik, the Norwegian road administration had saved two bridge segments from a demolished bridge, Farris bru. There were no documented damages on the bridge and the ducts were well grouted. This field test therefore focused on the limitations of the equipment.

The results of the inspection showed some limitations of certain equipment, especially in segment 2, where the tendon ducts were at a depth of about 450 mm.

Figure 7.1 The two bridge segments in Larvik, Norway

The concrete cover meter has a limit of about 80 mm which in most cases makes it difficult to locate the ducts. Moreover, it is impossible to differentiate between rebars and tendon ducts.

To be able to determine with radar (GPR) how deep the tendon ducts are placed in the concrete, it is necessary to calibrate against the actual cover layer. The reason for this is that the speed of the signals in the concrete depends on the quality of the concrete. With a known value of the reinforcement depth, the dielectric constant can be calculated and used in the analysis of the GPR results.

The results from GPR showed that the ducts in segment 1 could be located without major difficulties. We also obtained good pictures of the reinforcement which was located above the ducts. The ducts were located at a depth of approximately 150 mm and could be marked out on the test body. In segment 2 the result was poorer, here the GPR scans could not find the ducts, except for the parts that were placed shallower than 250 mm.

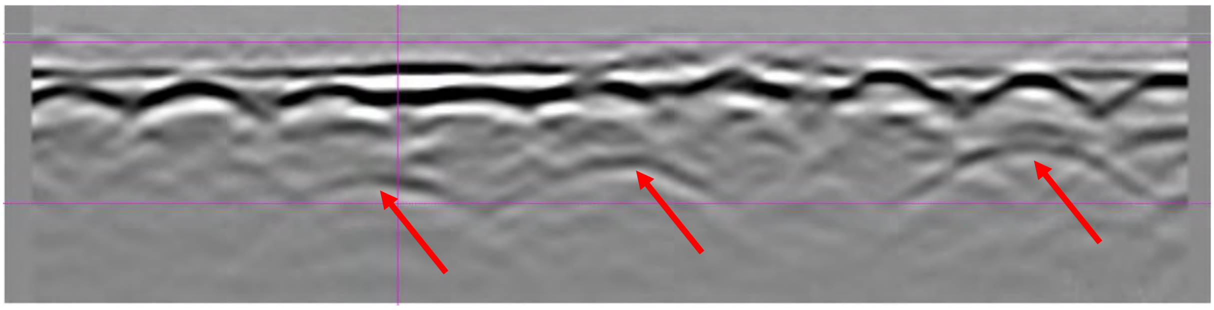


Figure 7.2 Located tendon ducts with GPR

The ultrasound measurements were performed on two ducts at each segment. In segment 1, the ducts were clearly visible during the scans and there were no signs of any voids. In segment 2, it was more difficult to locate the ducts, because they were placed much deeper, extensive data processing and examination of the results in the computer were required.

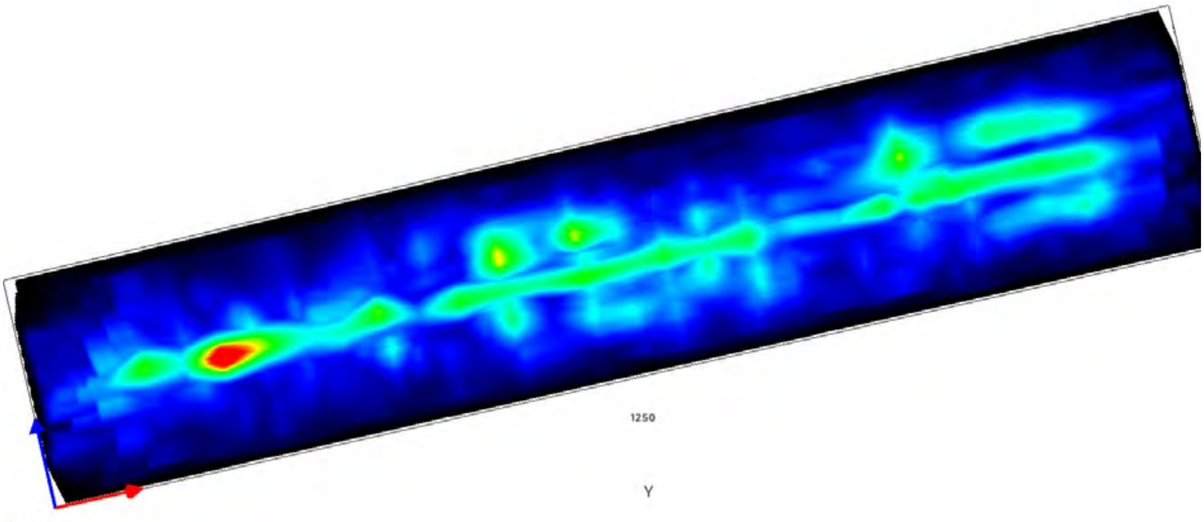


Figure 7.3 Tendon duct scanned with UPE

The signals were sometimes very weak which is partly because the ducts are completely filled with grout, but also due to the depth between 200-450 mm.

7.2 Ölandsbron, Kalmar

At Ölandsbron on the east coast of Sweden, repair works were ongoing. Testing was required to locate existing ducts and tendons in order not to damage them during the works. This was successfully done by using GPR.

GPR is a good choice of equipment for locating the tendon ducts. It is relatively easy to use and it is a quick method. The disadvantage is that it has a limit of about 250 mm in depth, which means that it cannot always find ducts that are placed further into the concrete. It is also difficult to distinguish between reinforcement and tendon ducts; in the present case the reinforcement has the dimension Ø10 mm and the tendon ducts Ø32 mm. Still there is no clear difference in the signals. Therefore, a proper drawing review is required to distinguish the two types of reinforcement.

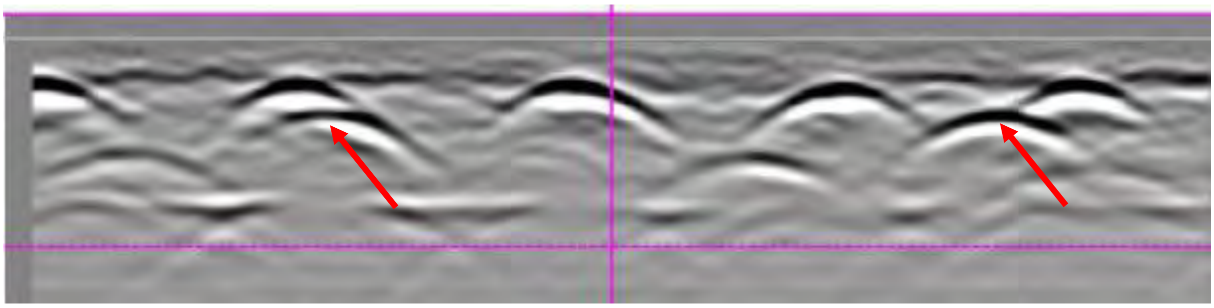


Figure 7.4 Tendon ducts located with GPR

7.3 Herøysundsbru, Herøyholmen

During some repair work at the bridge in Heröysund on the west coast of Norway, voids were found inside the ducts. In order to enable detailed strength calculations of the bridge, testing was required to define the extend of the damages. Aim was to identify additional voids, areas where there was no grout and where corrosion or even breakage could be expected. With GPR, UPE and IE four places were pinpointed for further inspection with drilling and endoscope.

The following methods were recommended by DEKRA to execute the inspection:

- Cover meter
- Ground Penetrating Radar
- Ultrasonic
- Impact Echo
- Visual inspection/drilling

The results showed that all 8 ducts in the girders, at some point, were missing grout. This was confirmed in four ducts by drilling and visual inspection. In one of the drilling holes three wire breakages were found.

www.dekra-industrial.se
tel. +46 10 455 10 00
Gamlestadsvägen 2
SE-402 51 Gothenburg

Figure 7.5 Heröysund bridge

Following our assessment procedure (see figure 5.2), we first determined the placement of the ducts and tendons by GPR. We also checked the distance to ordinary steel reinforcement with the aid of a cover meter. In critical areas, we then proceeded with the ultrasonic-pulse-echo. Verification of the findings of the UPE measurements was done with the aid of impact echo. Finally, four identified damage areas were opened up in order to verify our findings.

The inspection showed that grout was missing in large parts of the tendon ducts. In general, the largest problems were located at the head span towards axis 5. In this area 6 out of 8 tendon ducts had missing grout in all the inspected areas. Furthermore, some local cable ducts (for example: cable 3, south girder) had missing grouting near the anchors both in axis 6 and axis 3.

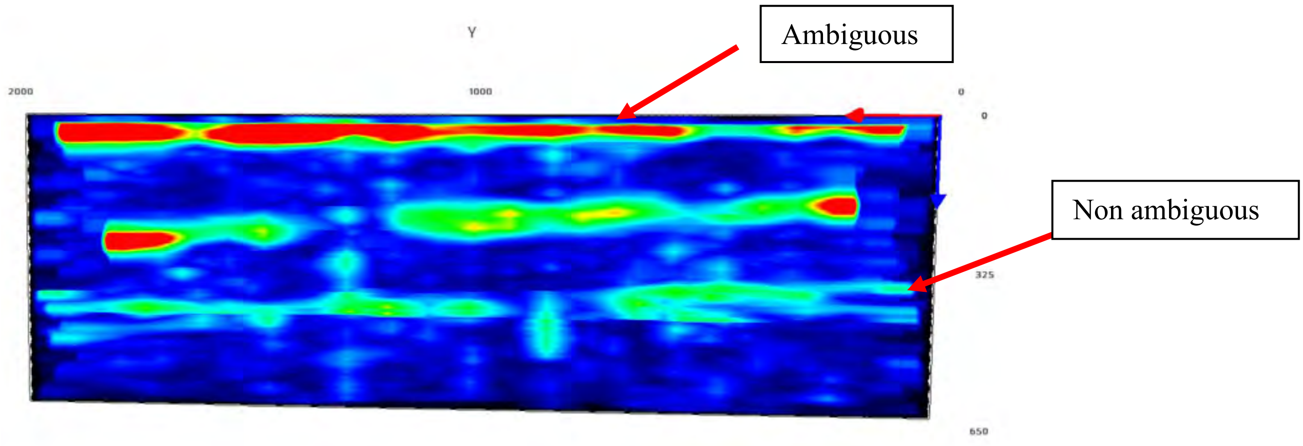


Figure 7.6 Results from UPE with ambiguous and non-ambiguous ducts

The cables in the middle of the bridge are partly missing grout, having about 1 m of continuous voids, they should be considered as a high-risk area. The amount of grouting in the ducts is difficult to determine, but taking the impact echo confirmation and following visual inspection into consideration, the results are assumed to be reliable and accurate.

Figure 7.7 Visual inspection of tendons with missing grout

Visual inspection after opening up showed that cable 3SC has at least 3 wire breakages. For the same tendon, missing grout is pointed out at a length of 6 meters from the anchor in axis 3. Because of the large sections with missing grout, it is hard to find the location of the wire breakages.

Figure 7.8 Missing grout and corrosion confirmed with visual inspection

Worth mentioning is also that there were several load cracks in the girders in the middle of the bridge. The cracks were located on the underside of the beams, which indicates a loss of load capacity.

Also, taking into consideration that the tendons were placed at the underside of the girders and that grout was partly missing, the cracks may have affected the ducts. If the ducts are damaged, a continuous provision of air and moisture may occur, which might lead to an ongoing corrosion on the tendons.

7.4 Railroad bridge, Abisko

A railroad bridge in Abisko, in the very north of Sweden, was tested in order to investigate possibilities and limitations of different equipment in localizing tendon ducts. The placement of the ducts in two rows created at special challenge, as testing was only carried out from the inside of the bridge.

The results of the inspection were a great help in further learning the equipment and its limitations. Neither with GPR nor with UPE we could locate the second layer of tension reinforcement. There were some differences in the two GPR equipments used, however both were able to locate the ducts.

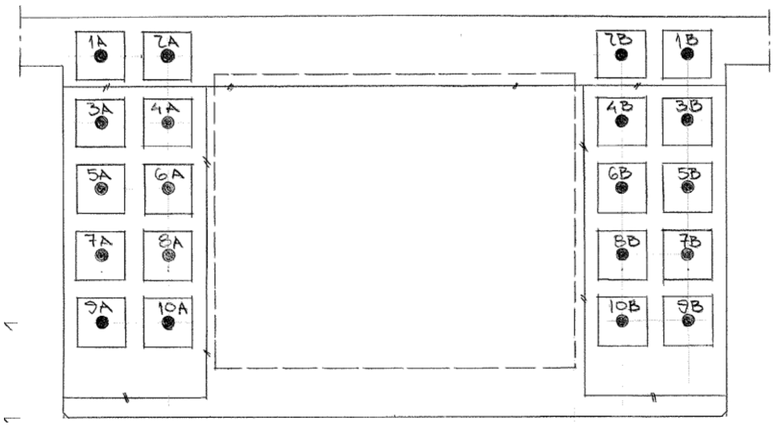


Figure 7.9 Placements of ducts in the railroad bridge

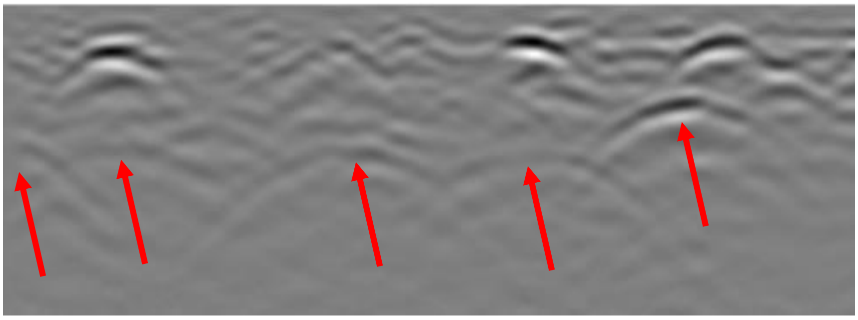


Figure 7.10 Ducts located with GPR

The reason that the second layer of tension reinforcement could not be located was that GPR simply is not powerful enough at this depth. In general, the limit in depth is about 250 mm.

The UPE is considerably more powerful and can penetrate a depth of up to about 1 meter. The reason why the second layer of tendon ducts could not be found was because they were shaded by the first layer. With the UPE we could measure the thickness to 550 mm which corresponds to the drawings. Thus, it is not the depth of the ducts that is the limit but the location of them.

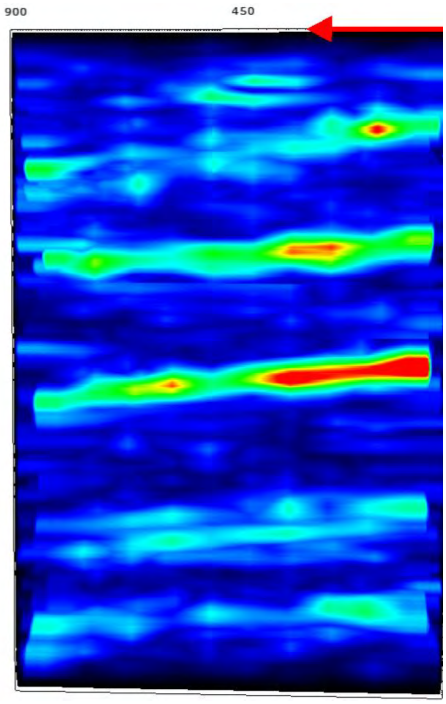


Figure 7.11 Results from the UPE scanning.

8 Discussion and conclusions

This chapter is a summary of what has been found to be the most effective approach to find voids inside tendon ducts. Issues such as cost- and time effectiveness, accessibility, user friendliness and limitations regarding different techniques have been taken into account. The same approach may be used on different types of bridges, the governing factor is the construction and accessibility of the concrete-element to be inspected, not the type of bridge.

8.1 Objective

In this study, non-destructive test methods for detecting damage to bonded tendons were investigated and evaluated. A structured approach has been defined to investigate tendons in concrete elements with the most appropriate methods. This approach has been verified in lab conditions and in the field and will result in an inspection manual, defining how to carry out future inspections on post-tensioned concrete bridges.

8.2 Techniques

The study presented in this report has focused on the usage of NDT (Non-Destructive Testing) to identify voids/damages in post-stressing systems consisting of normal tendons and tendon ducts. This has been carried out by studying national and international literature and by carrying out testing in practice. From this study it was found that no existing NDT method can solve this problem by itself.

Several promising techniques were identified and tested on mock-ups in laboratory, in the field and on existing bridges. From these tests the conclusion could be drawn that a combination of VI (Visual inspection), CM (Cover meter), GPR (Ground Penetrating Radar), UPE (Ultrasonic Pulse Echo) and possibly IE (Impact Echo) together with partly intrusive investigation using ES (Endoscope) or core drilling is the most appropriate approach in order to identify voids in post-stressing ducts. When a void has been found, visual inspection by ES or core drilling may be used to ensure accurate information regarding corrosion on tendons. All methods used allow scanning from one side of the member only.

The methodology described in this report was introduced successfully on several field bridge projects. The pros and cons from the investigation are summarized in table 8.1

Table 8.1 Application scope for selected methods

Application scope	**Methods**					
	VI	**CM**	**GPR**	**UPE**	**IE**[4]	**ES**
Surface deviancies	X	-	-	-	-	-
Locating reinforcement (L=location, Ø = diameter)	-	X (L/Ø)	X (L)	X (L)	-	-
Locating ducts (P= plastic, S=steel)	-	X (S)	X (P/S)	X (P/S)	-	-
Voids in concrete [6)]	-	-	X	X	X	-
Voids in ducts [6)] (M=medium accuracy, H=high accuracy P=partially grouted ducts)	-	-	-	X (M/H, P[3])	X (M/H, P)	X[1] (H, P)
Corrosion tendon/rebar	-	-	-	-	-	X[1]
Layered ducts	-	No	No	No	No	-
Measuring depth	-	≤80 mm	≤250 mm	≤1500 mm	[5]	-
Complexity / user friendliness [2] (E) =easy (M) = medium (A) = advanced	A	E	M	M/A	A	E
Speed (area coverage)	High	High	High	Medium	Low	Low
Accuracy	-	High	High	High	High	-
Approx. purchase cost per unit (NOK)	-	30 000	60 000 - 300 000	100 000 - 300 000	ca. 100 000	ca. 5000

[1)] by intrusive drilling
[2)] (E) =easy, no prior experience needed, (M) = medium difficulty, some prior training and experience needed, (A) = advanced, extensive prior training and experience needed.
[3)] When prior similar measurements have been confirmed through IE and ES.
[4)] Only as supplement to UPE.
[5)] For *d/T* < *1/4,* the flaw cannot be detected. For *1/4* < *d/T* < *1/3,* the flaw can be detected, but it`s depth cannot be determined. For *d/T* > *1/3*, flaw depth can be determined; and for *d/T* > *1.5*, the behaviour is that of a plate with thickness equal to the flaw depth. d = flaw size, T = flaw depth. (Impact-Echo - The Fundamentals, Carino 2015)
[6)] Sizing of voids at least 0,1….0,2 meter

8.3 Evaluation of suitable methods and testing sequence

8.3.1 Visual inspection (VI)

Step 1: Overview

All assessments start with a visual inspection, first from a distance to get a full view of a bridge and to detect for example structural deviancies. A visual inspection is then carried out section by section from a closer distance where minor damages or deviations such as cracks, delamination, corrosion and spalling can be noticed.

8.3.2 Cover meter (CM)

Step 2: Prior to locating ducts

The primary method to find tendon ducts is GPR but sometimes it can be difficult to separate e.g. slack reinforcement from metal ducts from the received signals. A quick scanning with a cover meter is a good way to exclude slack reinforcement from metal- and plastic tendon ducts. CM has limitations regarding the rather low maximum measuring depth (≤80 mm) but it is a fast, accurate method and practically no major prior knowledge is required to use it. The cover meter is not dependent on surfaces being smooth and due to its small size, it is possible to scan in areas where there are limitations in accessibility e.g. corners or tight spaces.

8.3.3 Ground Penetrating Radar (GPR)

Step 3: Locating ducts

GPR has a similar function as the cover meter, though more powerful. For structures with pre-stressed tendon bars, it is used to locate both metal- and plastic tendon ducts. The instrument is an important part of the inspection to quickly locate the ducts. The same answers can be given from the results of a scan with UPE but the GPR is more time efficient as it covers larger areas faster. The GPR enables an accurate measurement of the position in plane and in depth of the tendon ducts.

With GPR it is not always possible to distinguish between slack reinforcement and ducts, the signals are many times similar. However, careful examination of construction drawings combined with good knowledge of structure engineering will in most cases enable the inspector to identify tendon ducts. As earlier mentioned, it is in some cases also possible to use a cover meter (CM) to differentiate rebars from ducts provided rather small concrete covers. GPR has a maximum measurement depth limit of about 250 mm and it is therefore difficult to locate a second layer of ducts.

8.3.4 Ultrasonic Pulse Echo (UPE)

Step 4: Locating voids in ducts

Ultrasonic Pulse Echo (UPE) is the primary method to find voids in metal and plastic ducts. The instrument allows for a relatively fast mapping of potential voids in cable ducts over longer distances and/or larger areas.

Since ultrasonic signals are not transported in air, enclosed air in the concrete or in ducts is clearly visible. Many factors may influence the signals and consequently the quality of the results. Among them, most importantly the quality of the concrete (enclosure of air, density, homogeneity), presence of delamination, presence of reinforcement bars above the ducts, etc. Behind any objects/defects, the signal is shadowed an no results can be obtained.

Although scanning with UPE is considered the fastest method to detect voids in ducts it is a time-consuming method that requires an experienced inspector to obtain and interpret the results. Images are analyzed afterwards with the aid of computer software. A scan with UPE consists of several different segments called B scans. The results from each individual scan are reported as a slice of the concrete. When this is processed in the computer, several B scans are put together into a 3D image. Therefore, in order to find a void in the duct, scanning must be performed along the duct in a systematic manner.

One disadvantage of UPE is that a tendon duct without voids can be difficult to locate during processing, as only a minor part of the signal is reflected on the duct. This can be dealt with in place by adapting the settings or in retrospect by comparison with the results from GPR. Failure to locate the duct during data processing may cause doubts as to whether the scan was performed at the correct location. Therefore, it is of great help to be able to compare afterwards with a scan from GPR.

8.3.5 Impact Echo (IE)

Step 5: Further evaluation of voids

IE enables inspection of only one specific point at a time and is therefore only used for verification of defects located with the aid of other instruments. Defect size must at least be 1/3 of the penetration depth, in order to be identified. Signals will bounce on the first air or water defect encountered. Therefore, it is not possible to scan for layered defects i.e. defects that are located behind encountered air or water entrapment.

When a decision is made for intrusive inspection, IE may be used to confirm the exact drilling depth in order to avoid damaging the tendons.

Impact echo is based on monitoring the periodic arrival of reflected stress waves and is able to give information on the depth of internal reflecting interfaces. It is an advanced method that requires training and experience in order to be used in the field. Several factors may affect the result, such as concrete quality, repairs, castings, spacers, reinforcement, honeycombing, etc.

8.3.6 Endoscope (ES)

Step 6: Inspection of tendons (and sampling of grout mortar)

With previously mentioned NDT-methods, voids inside tendon ducts can be found with acceptable accuracy, though questions about for example tendon corrosion, water- or air filling of a duct cannot be answered. To clarify such issues, inspection by intrusive methods and visual inspection with ES are needed. The risk of damaging tendons when drilling must be taken into account, but if drilling is made with caution to a predefined depth, the method is both valuable and reliable.

If pre-conditions exist where chloride intrusion may occur, sampling of the grouting mortar may be desired. This can be achieved by using a core drill when exposing the tendons, enabling collection of chloride samples.

8.4 Assessment methodology

The assessment methodology, as described above, is summarized in figure 8.1 (copy of figure 5.2).

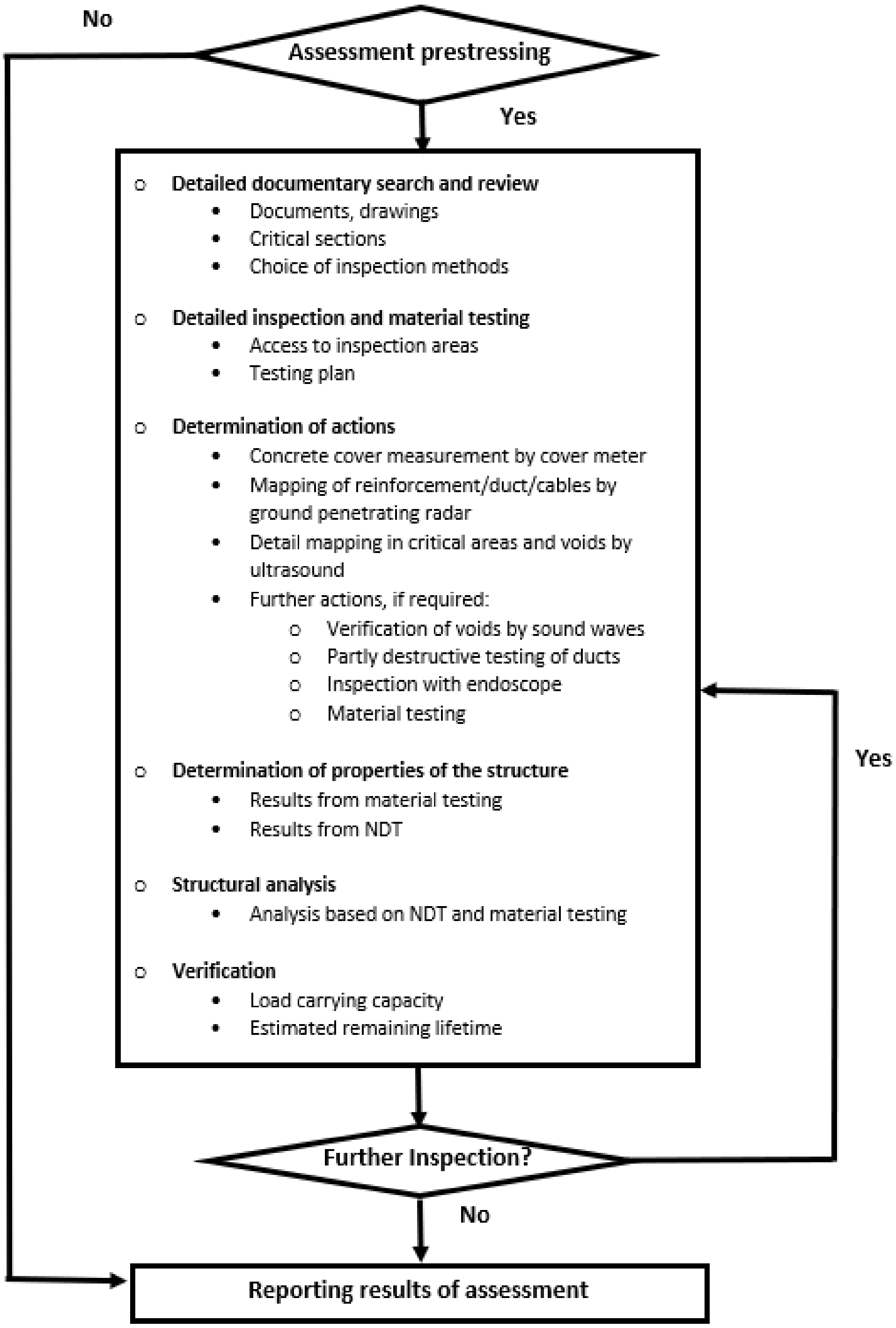


Figure 8.1 Assessment of post-stressing systems in concrete structures

8.5 Conclusions

Voids inside tendon ducts can be detected with the aid of a combination of different non-destructive testing methods. The voids indicate likely places for damages to the tendons.

The methods to be used are the following:

- Visual inspection to mark out and check critical areas
- Cover meter
- Ground Penetrating Radar
- Ultrasonic Pulse Echo
- Impact Echo
- Visual inspection/drilling and the use of endoscope

A combined use of the above methods is necessary, where the exact needs depend on the particular concrete element to be investigated. This will be explained into more detail in the forthcoming inspection manual.

From our investigation it can be concluded that:

- Identifying voids in ducts for post-stressing tendons is complicated
    - Good skills in assessment of concrete structures in general and knowledge about deterioration processes are required.
    - Skills in how to handle NDT techniques, software and equipment are needed.
- Equipment is expensive
    - The total set-up for the equipment needed may amount to about 300 000 to 750 000 NOK. In addition to this, yearly software license costs arise. Different price-models exist for GPR and UPE equipment, where purchase costs are lower while yearly license costs are higher.
- It is possible to find voids and damages.
    - Despite the challenges, this project has proven that it is possible with a reasonably high accuracy to find voids in cable tendon ducts.

A structured assessment methodology should always be used, as presented in this report.

Authors

Name	Title	Expertise	Main contribution to this project
Andreas Karlsson, DEKRA Industrial AB	Concrete inspection engineer	Concrete expert, NDT operator (UPE, IE, CM and GPR)	Project management Concrete inspections Concrete testing
Mats Holmqvist, DEKRA Industrial AB	Concrete inspection engineer	Concrete expert, NDT operator (UPE, IE, CM and GPR)	Project management Concrete inspections Concrete testing
Prof. Dr. Björn Täljsten Luleå University of Technology	Professor	Structural engineering, concrete, concrete inspection and testing	Concrete expert Concrete inspections
Pieter Jilderda, DEKRA Industrial AB	Manager materials, failure and concrete inspections	Mechanical engineer, Bachelor of science	Management, reviewing, coordination
Joakim Strand, DEKRA Industrial AB	NDT specialist level 3	NDT expert	Evaluation of NDT-methods
Bernt Åke Johansson, DEKRA Industrial AB	Senior specialist, metallurgy and failure investigations	Senior materials expert	Materials and corrosion expert

www.dekra-industrial.se
tel. +46 10 455 10 00
Gamlestadsvägen 2
SE-402 51 Gothenburg

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Appendix A – Mock-up at LTU

INSPECTION REPORT

Handläggare/Please refer to
Andreas Karlsson
Tel. +46 10 455 12 07
Mob. +46 70 285 25 74
Name.lastname@dekra.com

Datum/Date	Ref.	Ver.	Sida/Page
2020-05-01	7204-R-297038	1	1(10)

Action	Luleå mockup
Date	2020-01-14 – 2020-01-17
Location	Luleå Tekniska Universitet, Sweden
Present	Mats Holmqvist (DEKRA Industrial AB) Andreas Karlsson (DEKRA Industrial AB)

1 Background

DEKRA Industrial AB was commissioned to carry out an NDT inspection of a mock-up specimen in Luleå University as a part of the project "Study of methods, possibilities and limitations for inspection of post stressed reinforcement in concrete bridges". The purpose of the inspection was to learn how to use the equipment and interpret the results. This was a pre-study of the methods before NDT inspection on the bridge in Herøysund, Norway, managed by Statens Vegvesen.

1.1 Scope

The inspection was performed with different NDT methods listed below

- Cover meter
- Ground Penetrating Radar
- Ultrasonic pulse-echo
- Impact Echo

2 Object

The object was cast in Luleå according to the drawing in ***figure 2.1***. The defects that were created in the mockup were defects that we hoped to be able to detect with different methods.

2.1 Drawing

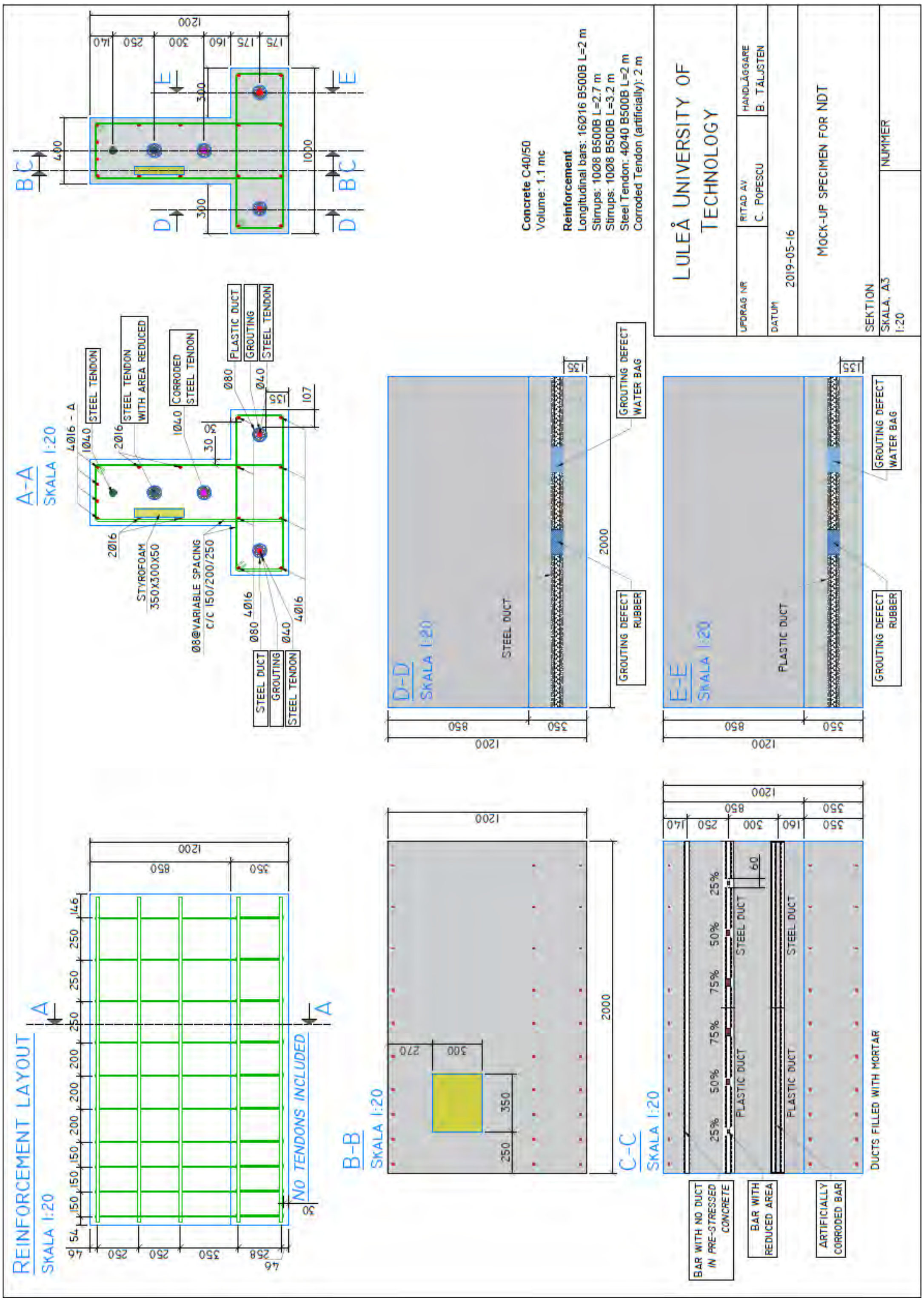


Fig 2.1 Drawing of the Mock-up

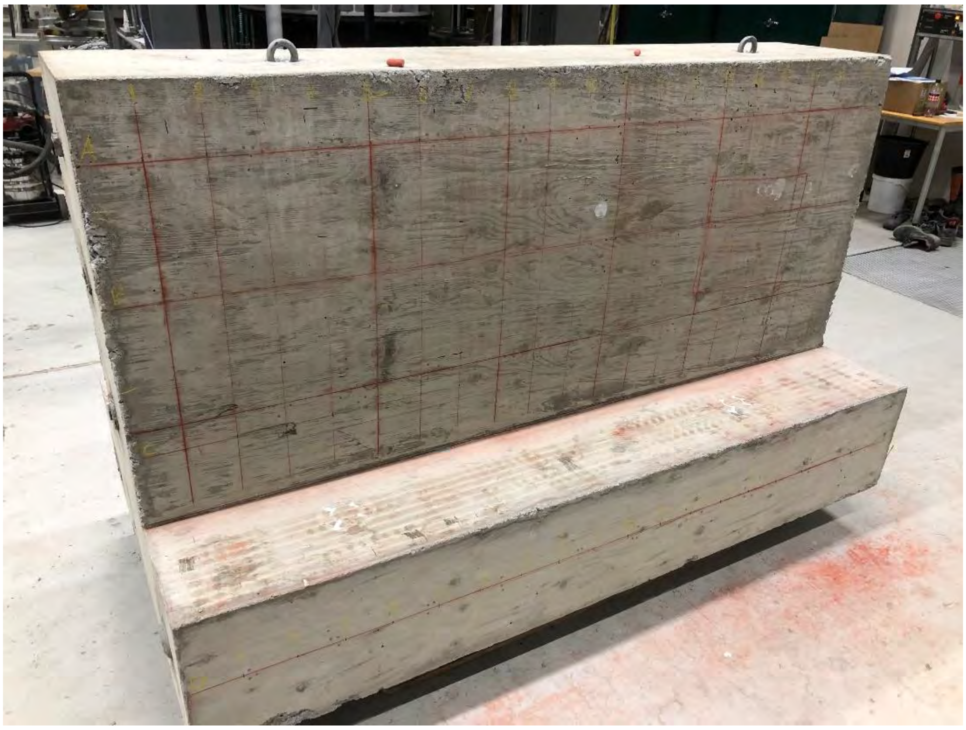


Photo 2.1 Mock-up specimen in Luleå

3 Test results

3.1 GPR

A GPR device from a manufacturer which had not been tested prior to the test failed to function. It was difficult to use and the results were difficult to interpret. Therefore, it was decided to exclude this method from the tests.

3.2 Ultrasonic pulse echo

During the test in Luleå, most effort was directed on learning the equipment: usage, presentation and interpretation of results, limitations, etc. We performed a large number of scans with UPE on the test body.

The tests consisted of the steps listed below.

- Use of the equipment
- How should we set up the scans
- How do we interpret the results
- Data processing

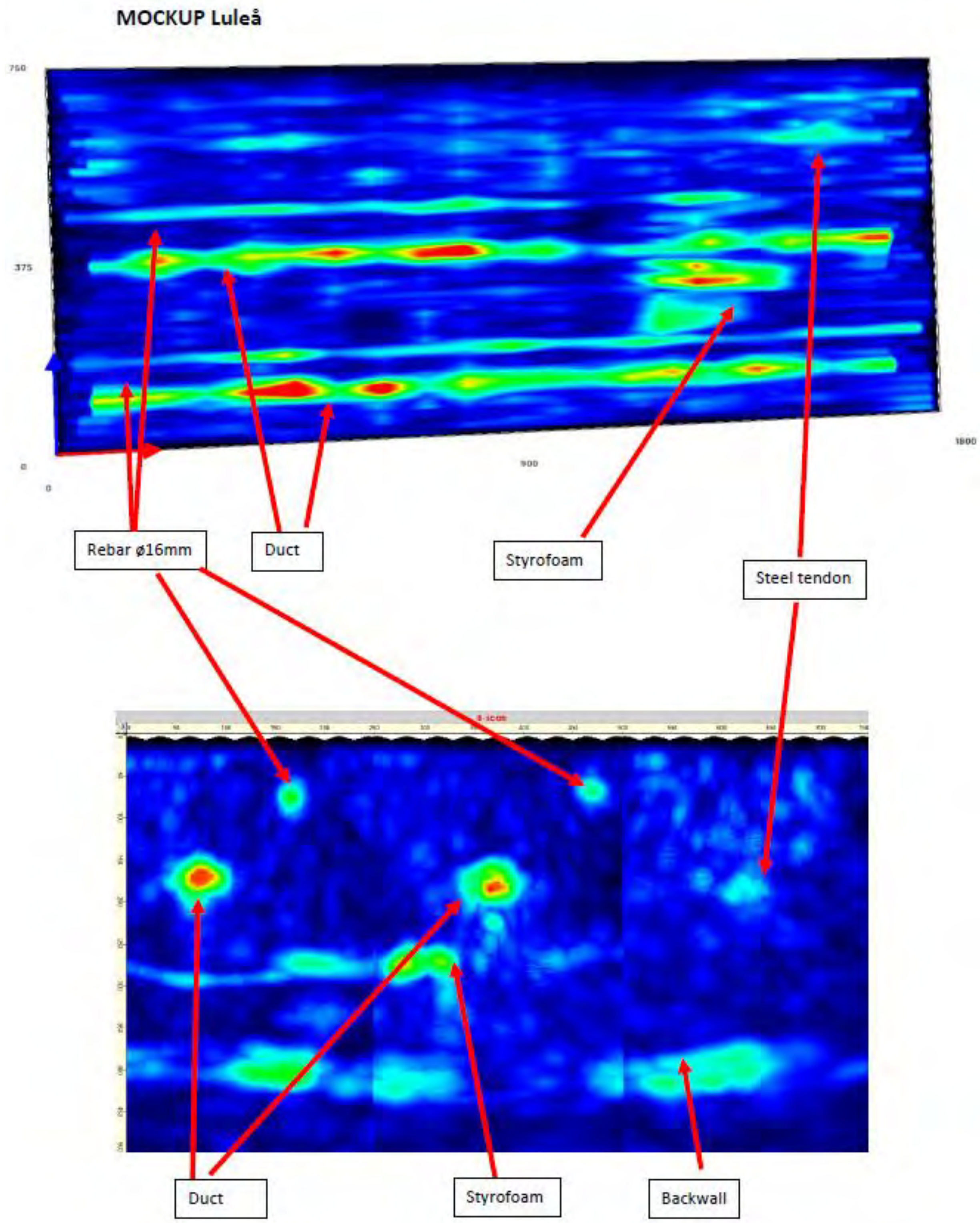


Fig 3.1 Different findings in the mockup

Figure 3.1 shows results from scanning in a horizontal direction, enabling us to find all the reinforcement and tension cables that were placed horizontally. We also got a background echo and we were able to find the styrofoam that simulated a delamination.

The scans are performed as line scans and in figure 3.1, three line scans are merged into a larger image.

In ***figure 3.2*** we see a difference between the two tendons placed inside ducts and one without. However, it was not possible to determine whether the reinforcement had a reduced area or whether it was corroded.

We also got a clear echo from the styrofoam that was casted in the concrete to simulate delamination.

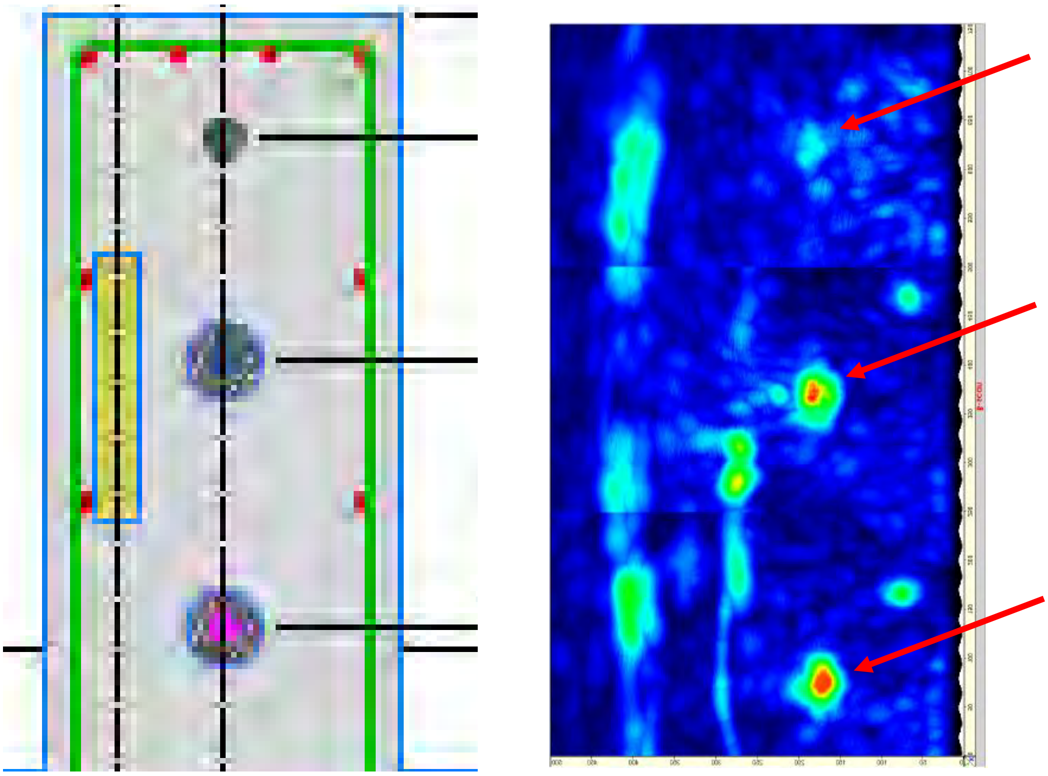


Fig 3.2 Upper part of mock-up with 3 tendons and styrofoam.

In ***figure 3.3*** we have the same image viewed from the side. Also here, tendon ducts are clearly visible. We get a relatively weak signal from the top tendon, but the Ø16 mm reinforcement which is closer to the surface gives a significantly better echo.

Also from this view, the steering foot is clearly visible and we can pinpoint exactly where it is. In this image, the styrofoam is placed behind the ducts.

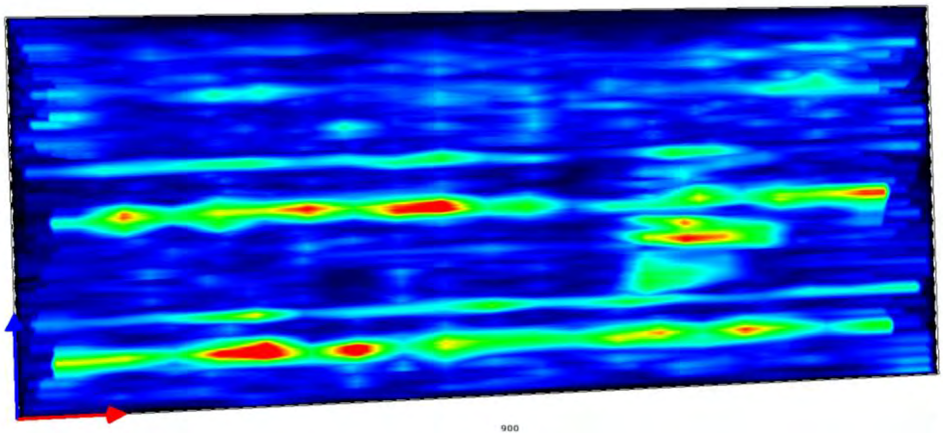


Fig 3.3 Mock-up from the side

On the two tendon ducts where we had simulated voids, the results were not entirely satisfactory. We had trouble pinpointing any voids in the ducts. However, we saw that there was no difference in the result between plastic and steel ducts.

Figure 3.4 shows a plastic duct scanned from the top, here we received a strong signal throughout the length except in two areas. These are caused by the plastic supports that were placed in the test body.

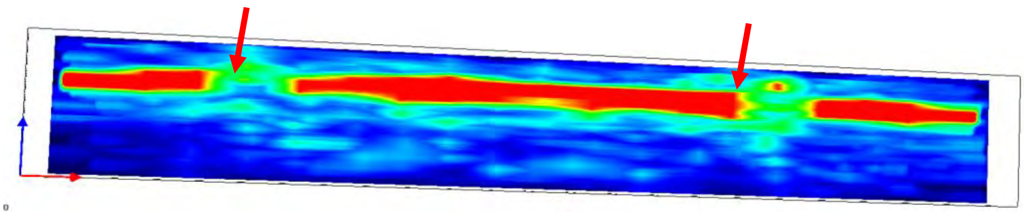


Fig 3.4 Plastic duct from the top with UPE

A scan with a different UPE equipment resulted in similar results, ***see figure 3.5.***

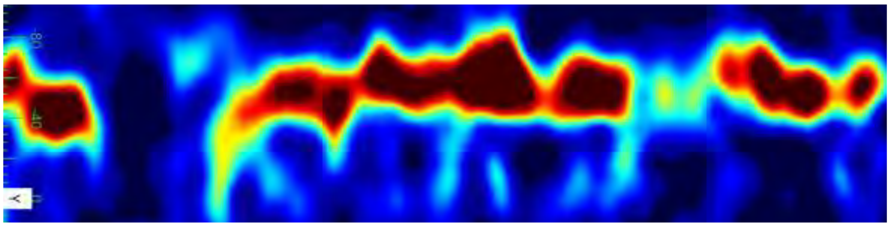


Fig 3.5 Plastic duct scanned with UPE (device from a different manufacturer)

Finally, in the steel duct, we obtained the same pattern, ***see figure 3.6.*** We cannot distinguish any voids, but we see disturbances in the signal from the plastic supports.

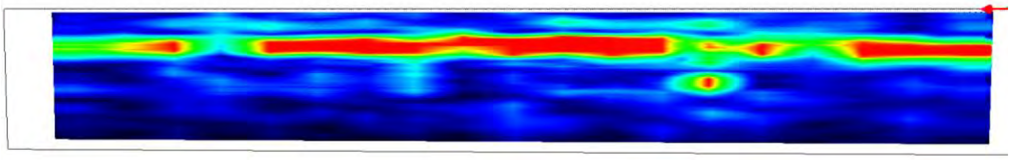


Fig 3.6 Steel duct

3.3 Impact echo

In order to use the impact echo, it is necessary to determine the speed through the concrete. For this mockup, thickness is known, enabling us to calibrate the equipment.

By doing several random tests on the test body, the thickness frequency was defined as 4.88 kHz in the upper part of the test body. With a known thickness of 400 mm, the speed could then be calculated to 3904 m/s.

$C_p=2f_TT$
$C_p=2*4,88*400 = 3904$ m/s

C_p = P-wave speed (m/s)
f_T = Thickness Frequency (kHz)
T = Thickness (mm)

In ***figure 3.7*** we see the signal from an area without any defects or reinforcement. We get a peak of 4.88 kHz which corresponds to the thickness of 400 mm

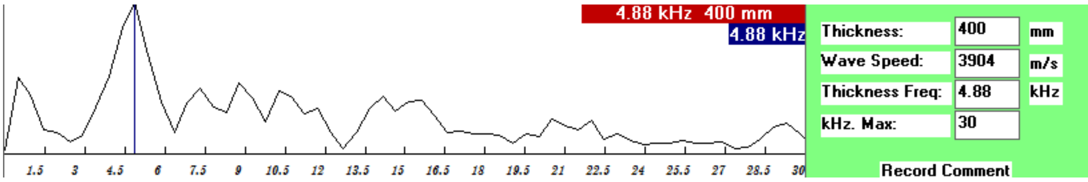


Fig 3.7 Solid 400 mm

To confirm the setting values, a test was made in the area where the styrofoam was placed. The frequency peak was moved to 6,35 kHz which equals the depth of 308 mm, namely the depth of the styrofoam, ***see figure 3.8***

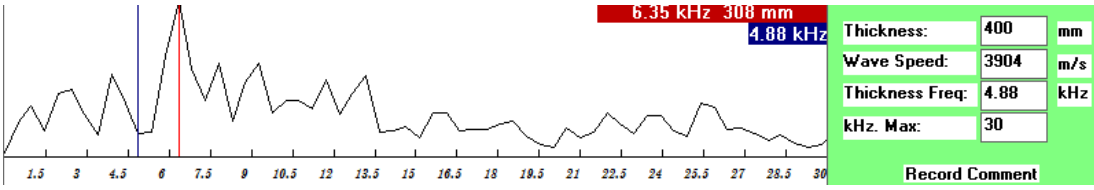


Fig 3.8 Delamination 300 mm

The same tests were done on the lower tendon ducts, here the thickness was 350 mm. In an area with only injected duct, we get the thickness echo basically unchanged, and we also see a peak at the same depth as the tendon duct. ***See figure 3.9***

C_p=2*5,58*350 = 3904 m/s

C_p = P-wave speed (m/s)
f_T = Thickness Frequency (kHz)
T = Thickness (mm)

The point of interest when looking at the tendon ducts is the cover layer of about 135 mm.

When we calculate the frequency that gives us
3904/ (2*135) = 14,4 kHz

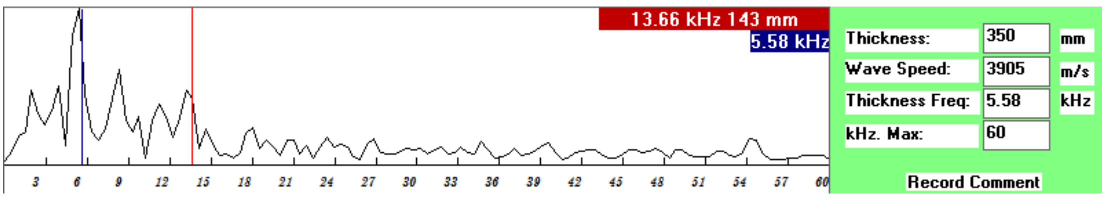


Fig 3.9 grouted tendon duct 143 mm

In ***figure 3.10*** we get a shift in the thickness frequency and here we also see a larger peak at a depth of 133 mm. This test was done above the area with a simulated void.

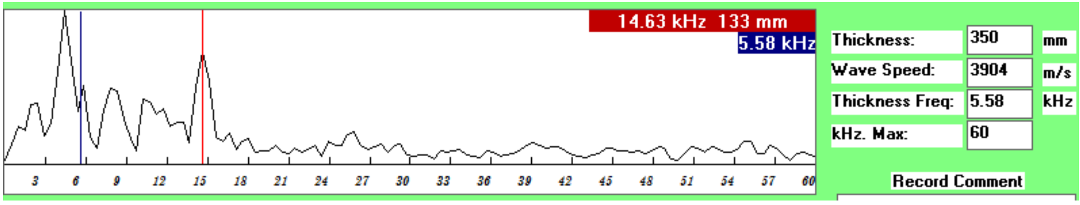


Fig 3.10 duct with void 133 mm

We see a clear difference between the two different curves where the thickness peak in ***figure 3.10*** is offset and we also get a larger peak at the frequency 14.63 kHz which corresponds to a depth of 133mm.

4 Comments

The results from Luleå were satisfactory, but with all facts on hand, the planning of the test body could have been done differently in order to make the best use of the equipment.

The results from UPE showed that we were able to map the test body quite well, though it was difficult to find the voids. Probably the voids are too small to be able to locate them. Moreover, the ducts were situated in a part of the structure which was difficult to access.

The results with the Impact Echo were interesting and here we managed to locate the voids.

With GPE and UPE we can locate and find out where we need a deeper control with Impact Echo. Since the results from IE are difficult to interpret, we need to know at what frequency we should look for different results. In those cases where thickness is unknown, velocity from the UPE measurements may be used.

5 Conclusions

- GPR can locate reinforcement and tendon ducts. Certain demands are put on the knowledge and experience of the inspector in order to correctly evaluate the results.

- UPE was successfully used to map the construction, but unfortunately, localization of voids in the ducts failed. It is assumed that the main reason is the limited size of the voids combined with difficulties in accessibility.

  It was noticed that access to a computer during the inspection is valuable, in order to analyze preliminary results on site. Some scans may have to be repeated.

- IE was successfully used to find the voids in the ducts, as their localization was known in advance. However, there are many factors in the test that influence the result, such as:
  - Impact time
  - The distance between receiver and impactor
  - Placement of the receiver (right above the defect)

  In addition, the relevant frequencies must be defined.

Appendix B – Mock-up in Hamar

www.dekra-industrial.se
tel. +46 10 455 10 00

Gamlestadsvägen 2
SE-402 51 Gothenburg

Handläggare/Please refer to
Mats Holmqvist

Rev. 2: modification of conclusions

Action	NDT, Hamar
Date och time	2020-04-17 & 18
Location	Hamar, Norway
Present	Mats Holmqvist (DEKRA Industrial AB)

1 Background

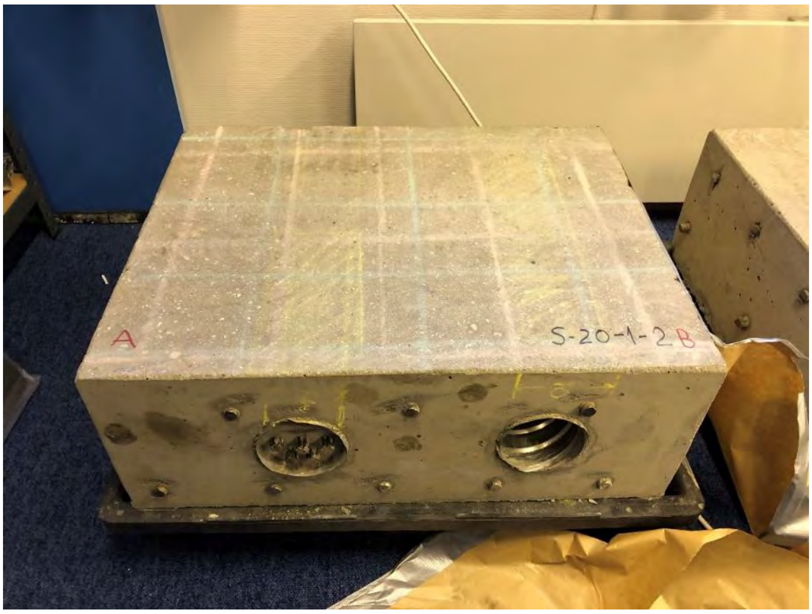
DEKRA Industrial AB was, as a part of the project "Study of methods, possibilities and limitations for inspection of post stressed reinforcement in concrete bridges", commissioned to carry out an NDT inspection of a test body in Hamar, Norway. The purpose of the inspection was to learn how to interpret the results from UPE and IE.

1.1 Scope

The inspection was performed with the NDT methods listed below
- Ultrasonic pulse-echo (2 different devices UPE 1 and UPE 2)
- Impact Echo method

2 Object

The test block at Hamar was produced by a UPE manufacturer to try out their scanner. The test body was cast with two ducts were one of them was empty and the other one was grouted to 75 % half its length and 100 % the other half, ***see picture 2.1 – 2.3 and figure 2.1.***



Picture 2.1 Mockup specimen

Picture 2.2 Duct A, Partially grouted (75%) half of the length and fully grouted the other half.

Picture 2.3 Empty duct B.

2.1 Drawing

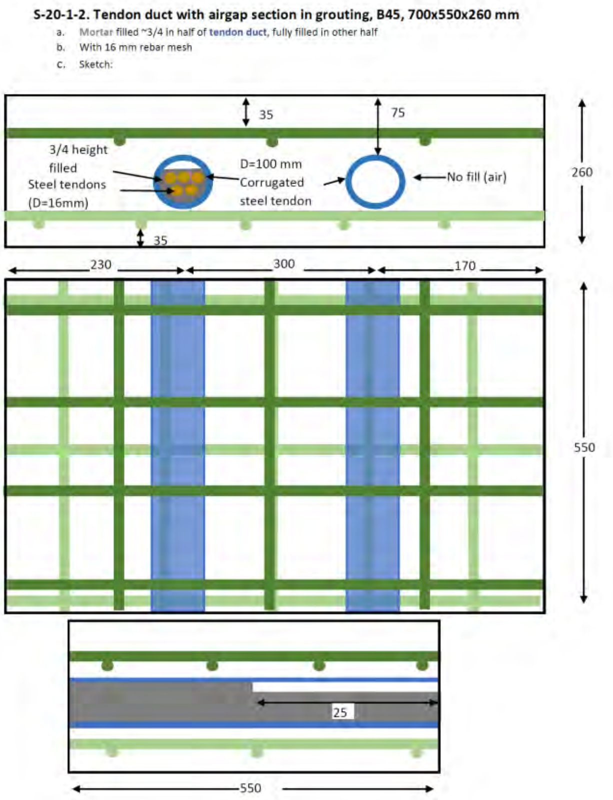


Figure 2.1 Drawing of the mock-up specimen

3 Test results

3.1 Ultrasonic Pulse Echo

It was possible to see the difference between the filled and the empty duct with both instruments. ***In figure 3.1*** is a scan with UPE 2 on cable duct A where half was injected and the other half was partially injected to ¾ of the height of the duct. The image shows strong signal in the right part of the duct, which indicates that there is a void.
However, the void extended further than the image from UPE 2 shows after processing in the software.
A similar scan was carried out with the UPE 1 and the by software processing a image with a signal equal to the actual void was obtained. By comparing the images from ***figure 3.1 and figure 3.2*** one can see obvious differences in resolution and the length of the void.

Both following scans was performed along the duct

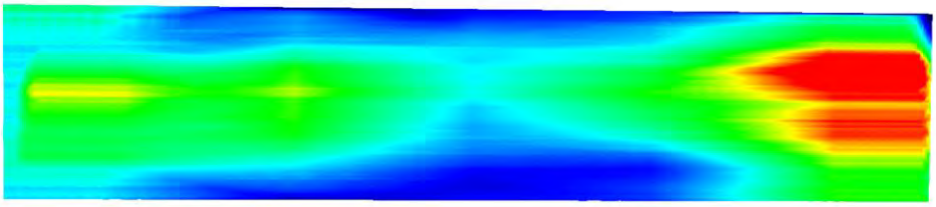


Fig 3.1: UPE 2 fully grouted (100 %) to the left and partially grouted (75 %) to the right.

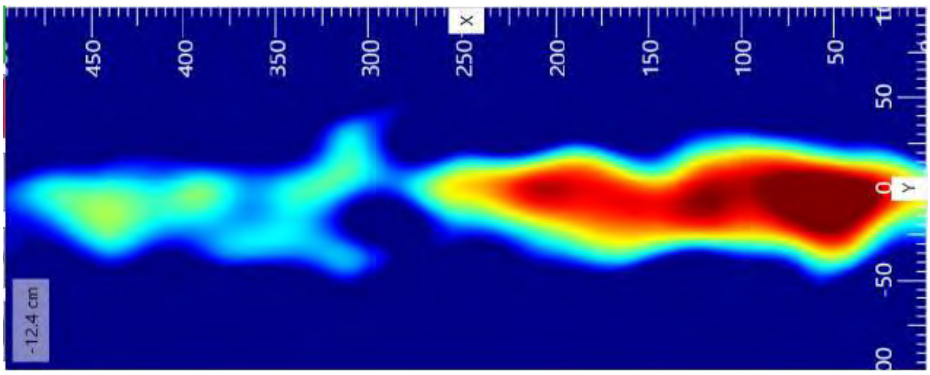


Fig 3.2: UPE 1, fully grouted (100 %) to the left and partially grouted (75 %) to the right.

In ***figure 3.3***, the scan is done longitudinal to the duct that is completely empty. A strong signal was received over the entire duct, which indicates that it is filled with air.

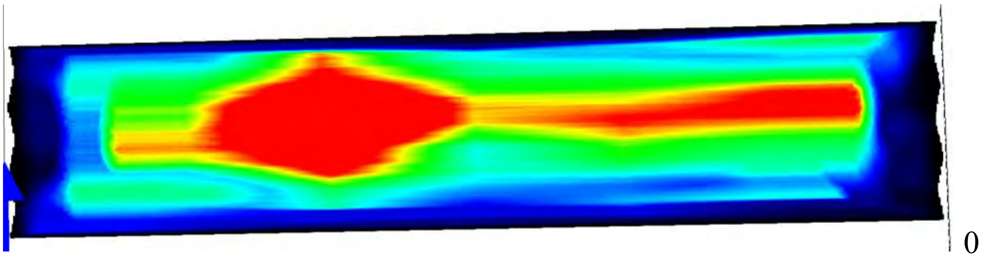


Fig 3.3: UPE 2, Empty duct B

Figure 3.4 shows the difference between the signals on one grouted- and one empty duct as opposed to the signals two ducts with void in ***figure 3.5***. Both of these scans were made with the UPE 1

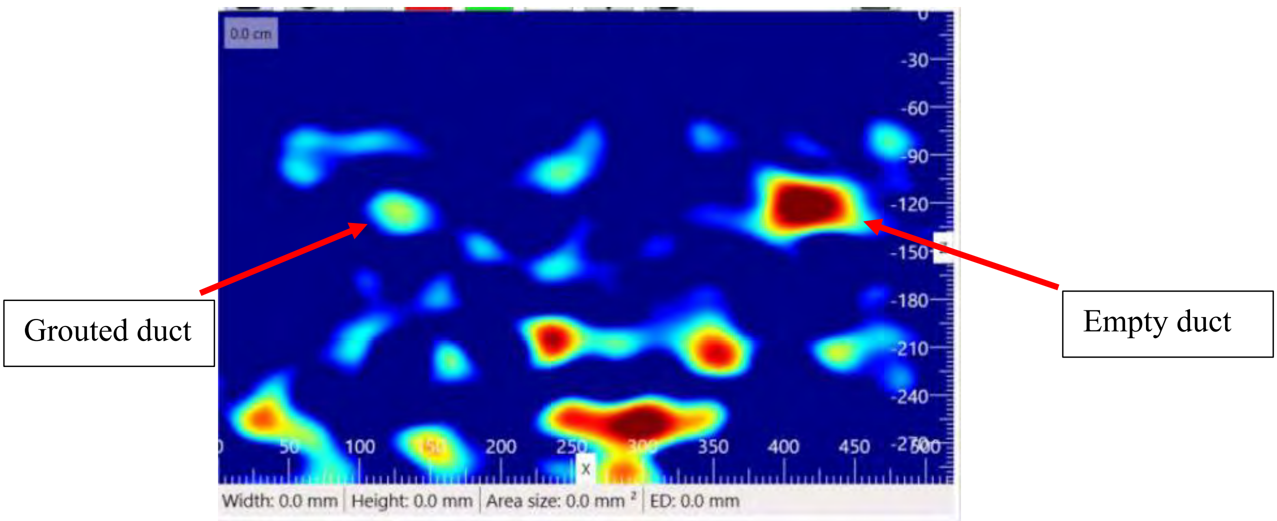


Fig 3.4: UPE 1, Grouted and empty duct

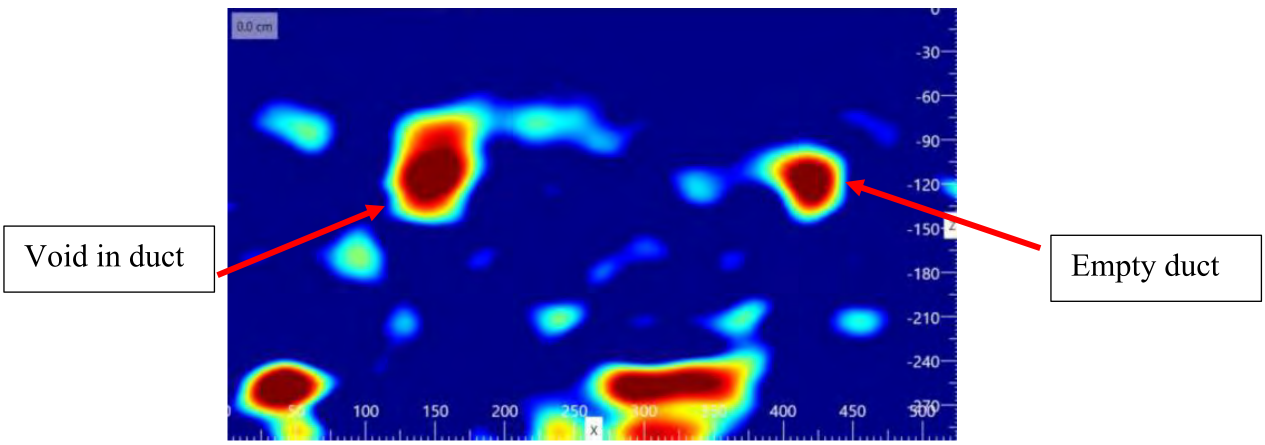


Fig 3.5: UPE 1, Duct with void and empty duct

3.2 Impact Echo

It was simulated that the thickness of the test block was unknown, therefore some calculations had to be made from a thickness determination of the test body.
With UPE 2 measurements of the concrete speed was established to 2518m / s, the wave speed value was used to calculate the thickness frequency in the test body.
Since device UPE 2 measures S-wave speed and Impact echo uses P-wave, one has to convert the speed to the correct value when using the IE. This is done by a calculation with a ratio of 0.62.

Measured S-wave speed (C_s) = 2518 m/s
Calculated P-wave speed (C_p) = 2518/0,62 = 4061 m/s

With repeated tests on the test body we were then able to measure a thickness frequency of 7.81 kHz and with the formula $C_p=2f_T T$ calculate the thickness.

Thickness Frequency (f_T) = 7,81 kHz

$$T = \frac{4061}{2 * 7{,}81}$$

Thickness = 260 mm

When the thickness and the frequency is known, we could go ahead and do the same calculations on tendon ducts to see what frequencies are interesting to look at to locate the void. For this we used the formula

$$f = \frac{\mathrm{Cp}}{2 * T}$$

According to the drawings, the tendon ducts had been placed at a depth of 75 mm and we then get the following calculation to obtain the frequency

$$fv = \frac{4061}{2 * 75}$$

Calculated void Frequency (f_V) = 27,07 kHz

After completion of the calculations, we obtained the two frequencies that we will look more closely at in the process of the test. We did tests along both cable ducts to see the differences in the results.
In picture 3.1 we see the location of four selected testing points.

1. Grouted duct
2. Partially grouted duct
3. Empty duct
4. Empty duct

Picture 3.1: Placements of test points.

3.2.1 Test point 1, fully grouted duct (100 %)

In test point 1, the duct is fully grouted, which we see in ***figure 3.6***. Here we get a clear thickness peak at the frequency 7.81 kHz which corresponds to the thickness 260 mm. This is in line with our calculations and indicates that the reflection goes straight through the injected duct and reflects on the back of the test body.

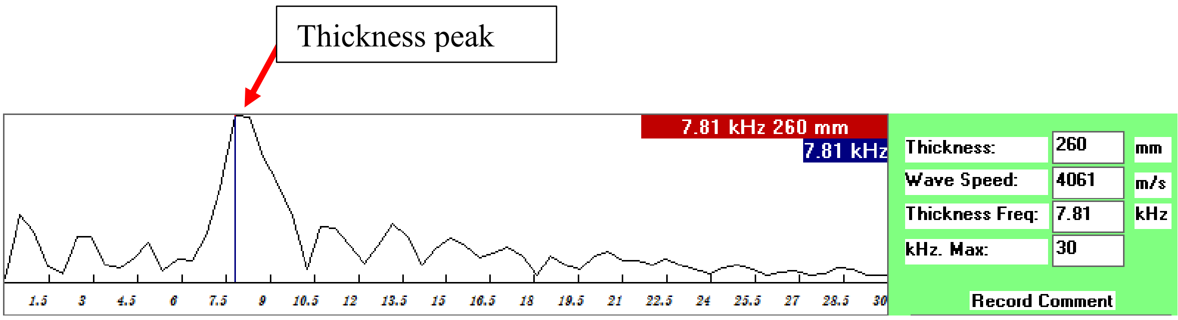


Fig 3.6: Test point 1: Grouted duct

3.2.2 Test point 2, partially filled (75 %)

Figure 3.7 shows that the thickness peak has moved to a lower frequency towards the left on the bottom scale. This means that something is in the way of the signal and it will travel a longer distance.
When this happens, we move on and look at the signal from the depth where we suspect that there is a void, in this case the calculated frequency around 27 kHz. Here we also do not see any major peak, which means that we do not get enough reflective signals from the void.

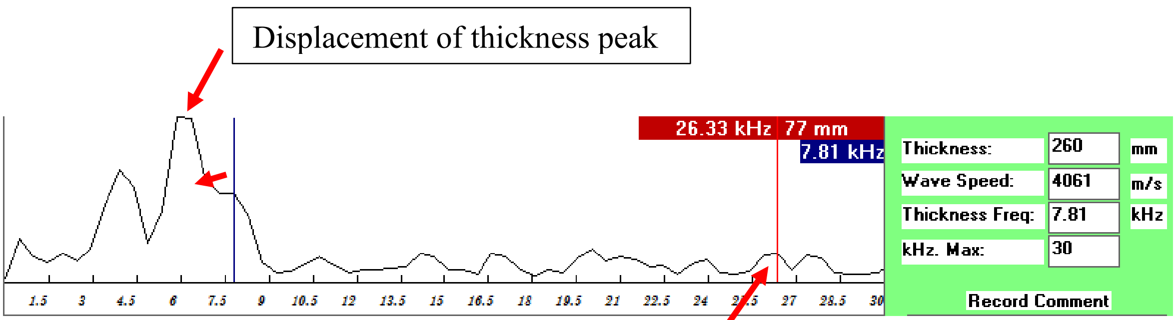


Fig 3.7: test point 2, partially grouted

To clarify the result, we can put a comparison between two different test areas in the computer. This helps us to quickly find differences in the various tests to determine if there are is any different frequencies that stand out.
In ***figure 3.8*** below we have a comparison between test point 1 (green) and test point 2 (black).
Here we can clearly see the difference in the thickness peak and we also see that we have a change around the frequency 27 kHz corresponding to the depth of the tendon duct.

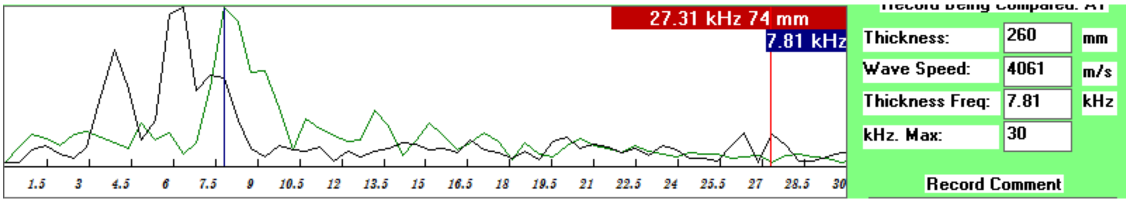


Fig 3.8: Comparison between test point 1 and 2

3.2.3 Test point 3 & 4, empty duct

In test points 3 and 4 are the results of an empty cable duct, and in the same way as in test area 2 we see a displacement of the thickness peak in both test areas.
But unlike ***figure 3.6***, we get a clear peak at the depth of the void.
In ***figure 3.9***, the frequency 26.82 corresponds to a depth of 76 mm, which is also well in line with the drawing.
In ***figure 3.10*** we get a slight change in the frequency of the void, here we get a depth down to the cavity of 83 mm.

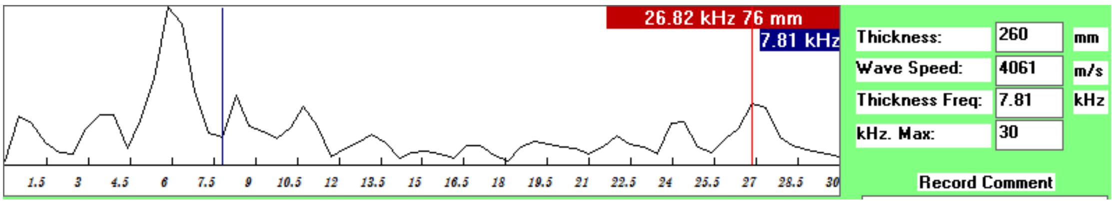


Fig 3.9: test point 3, void

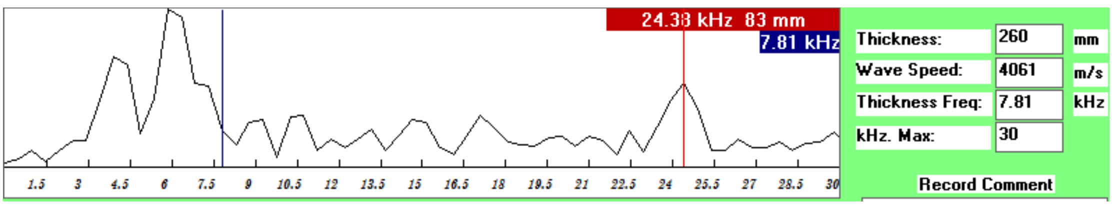


Fig 3.10: test point 4, void

Again, we made a comparison between measuring point 1 (green) and measuring point 4 (black) to clearly show the differences in frequencies and the displacement of the thickness peak.

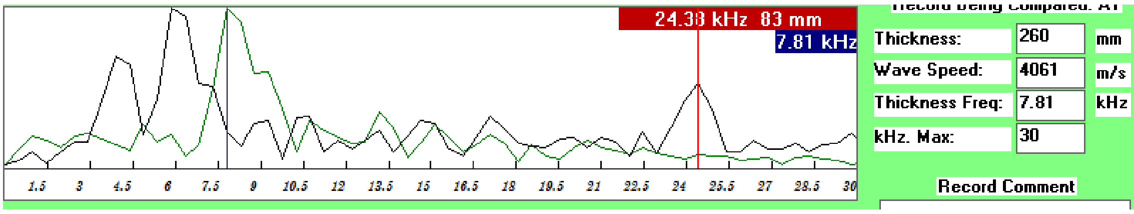


Fig 3.11: Comparison between test point 1 and 4

4 Comments

The results showed that we could see differences in the signals with UPE on ducts that were injected and ducts that were empty.
Between the two different instruments, the signals from UPE 1 were somewhat clearer than with the UPE 2. With some changes in the settings, the results would probably have become better even with UPE 2, but it was possible to locate the voids with both.

It is also important that the settings are correct to avoid unnecessary costs for a return visit to the bridge or that incorrect assessments are made.

But anyway, the great advantage of UPE is to find voids and be able to conduct the scan along the tendon ducts to cover larger areas of the inspected ducts.
The reason for this is that the voids can be of different sizes and on longer distances it is then difficult to only scan transversal in a local test point.
The risk is big that the wrong decision will be made if you carry out tests in local areas.

With IE we got good results and we succeeded in locating the voids in the test body. Several of the signals read from the IE are uninteresting for the current inspection and accurate calculations are required to be able to select this.

There are various ways to calibrate the equipment and the quickest is to know the thickness of the design.
If this is unknown, a number of preparatory tests are required to be able to find the correct thickness frequency and then the calibration is performed using this.
IE is a method that gives a result in a small test point, so it is time-consuming to use it over larger areas.
It should therefore only be used for bridge inspections to verify the results obtained by UPE. For this purpose, IE is very accurate.

If it is to drill a hole and then inspect with the Endoscope, IE is a good method for verifying the drill depth. This is not to damage the tension wires if you drill too deep.

5 Conclusions

The tests in Hamar showed us that it is possible to find voids in the ducts with both UPE and IE. Both methods require training and experience to interpret the results.
There is an uncertainty in both UPE and IE and therefore you need to use them both to make the correct assessment of the bridge.

- There is several different ultrasound equipment on the market, which of those used in the inspection does not matter. The important thing is that the inspector is well acquainted with the equipment and how to use it. Depending on the equipment used, the software is also different and there may be some differences in how advanced it is to process the results in the computer and interpret the results.

  In order to be able to find voids with UPE, the rule of thumb is that the larger voids, the greater accuracy you have in your assessment.

  It is not excluded that during the inspection you get results that do not correspond with reality, in most cases this is because the setting at the scan was incorrect or that you have had a poor contact with the concrete surface at the scan. Therefore, it is important to carefully evaluate the results already after the first scan to avoid unnecessary scans or incorrect decision making.

- Impact Echo is a method that gives the result in a single point, therefore it is not time effective to perform scans over larger surfaces. When locating voids, this should mainly be used as a complement to UPE.

  If IE is used properly, you look for divergent patterns in the frequencies and, with the help of this, confirm or dismiss your suspicions.
  To determine which of the frequencies that are different from the normal, some preliminary work and calculations are required.
  For example, your deviation in frequency may be a rebar located below your measuring point, then you have to discard the frequency from the rebar.
  We also noted that it is of the utmost importance that the test be performed directly above the tendon duct. If you do the measurement in the wrong place, the result will not be reliable or incorrect.

Appendix C – Mock-up in Copenhagen

Handläggare/Please refer to
Andreas Karlsson

Tel. +46 10 455 12 07
Mob. +46 70 211 25 74
andreas.karlsson@dekra.com

Action	Evaluation Impact Echo
Date	2020-10-01
Location	Copenhagen, Denmark
Present	Andreas Karlsson (DEKRA Industrial AB) Erik Boström (DEKRA Industrial AB)

1 Background

DEKRA Industrial AB was, as part of the project "Study of methods, possibilities and limitations for inspection of post stressed reinforcement in concrete bridges", commissioned to carry out test measurements on a test body located in Copenhagen, Denmark. The purpose of the testing was to evaluate equipment properties for locating flaws in ducts and concrete on bridges. One issue before the visit was if partially grouted ducts can be investigated with the DOCter impact echo method.

1.1 Scope

Measurements were conducted with the DOCter Impact Echo on a slab with two cast-in ducts with different flaws. The surfaces were grinded and center line of the ducts were located and marked before testing.

2 Object

The object was a 360 mm thick slab of concrete. Two steel ducts (Ø100 mm) had been casted in. One of the ducts was empty (no grouting and no strands) and the other duct had strands and was fully grouted with injection mortar, ***(Fig 2.1 and 2.2).***

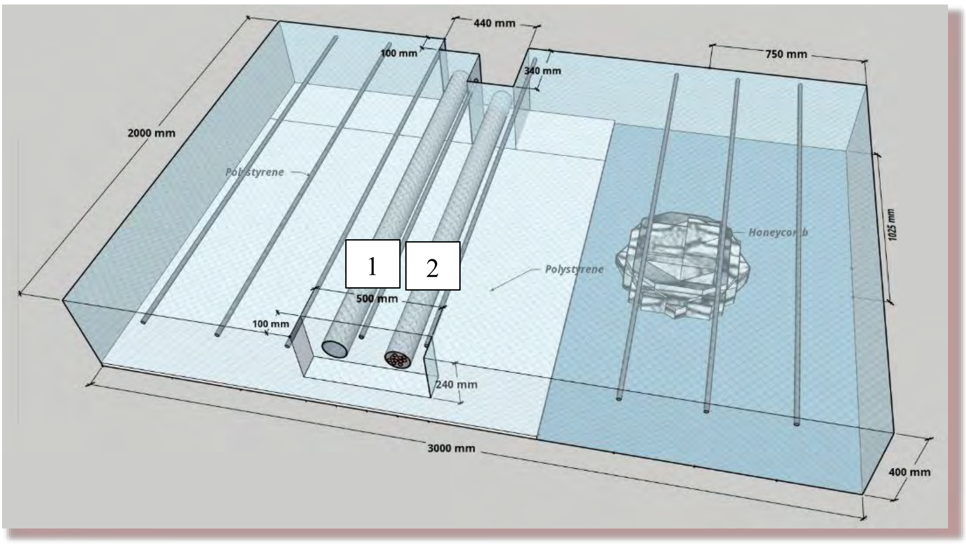


Fig 2.1: Geometry and dimension of test slab. 1) Empty duct. 2) Fully grouted duct. (Note: Thickness of slab wrongly stated to 400 mm, in reality thickness of the slab was 360 mm).

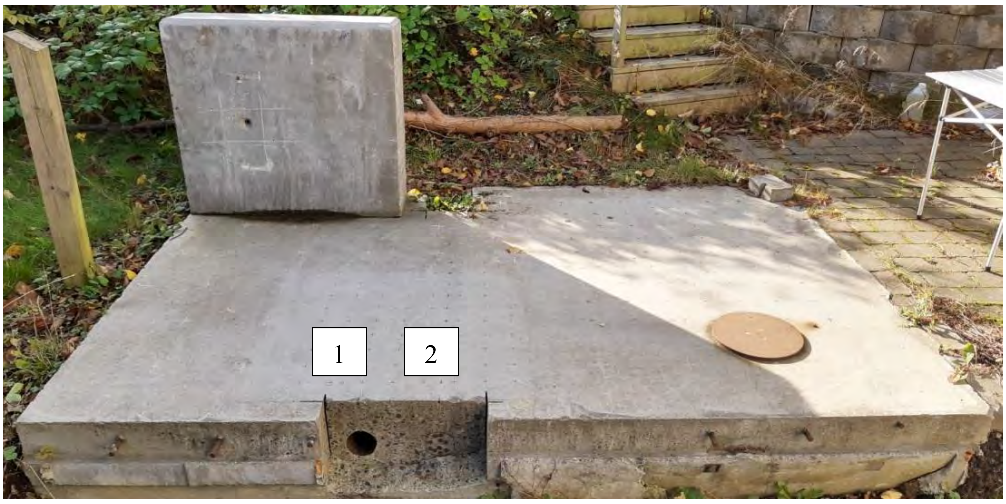


Fig 2.2: Slab. 1) Empty duct. 2) Fully grouted duct

3 Manufacturers description of the DOCter Impact Echo

The impact echo method is based on monitoring the periodic arrival of reflected stress P-wave produced by impact from steel spherical impactors of different sizes and is able to obtain information on the depth of the internal reflecting interface or the thickness of a solid member.

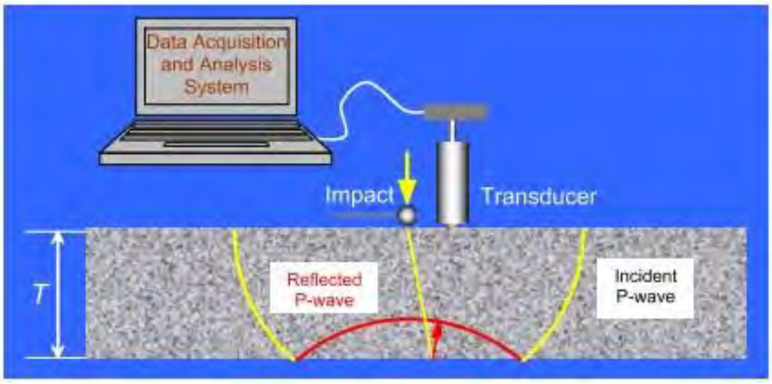


Fig 3.1: General principle of DOCter Impact Echo method.

A short duration stress pulse is introduced into the member by mechanical impact. This impact generates three types of stress waves that propagate away from the impact point. A surface wave (R-wave) travels along the top surface, as well as a P-wave and an S-wave travel into the member. In impact-echo testing, the P-wave is used to obtain information about the member. When the P-wave reaches the back side of the member, it is reflected and travels back to the surface where the impact was generated. A sensitive displacement transducer next to the impact point picks up the surface displacement due to the arrivals of the P-wave. The P-wave is then reflected into the member, and the cycle begins again. Thus, the P-wave undergoes multiple reflections between the two surfaces. The recorded waveform of surface displacement has a periodic pattern that is related to the thickness of the member and the wave speed.

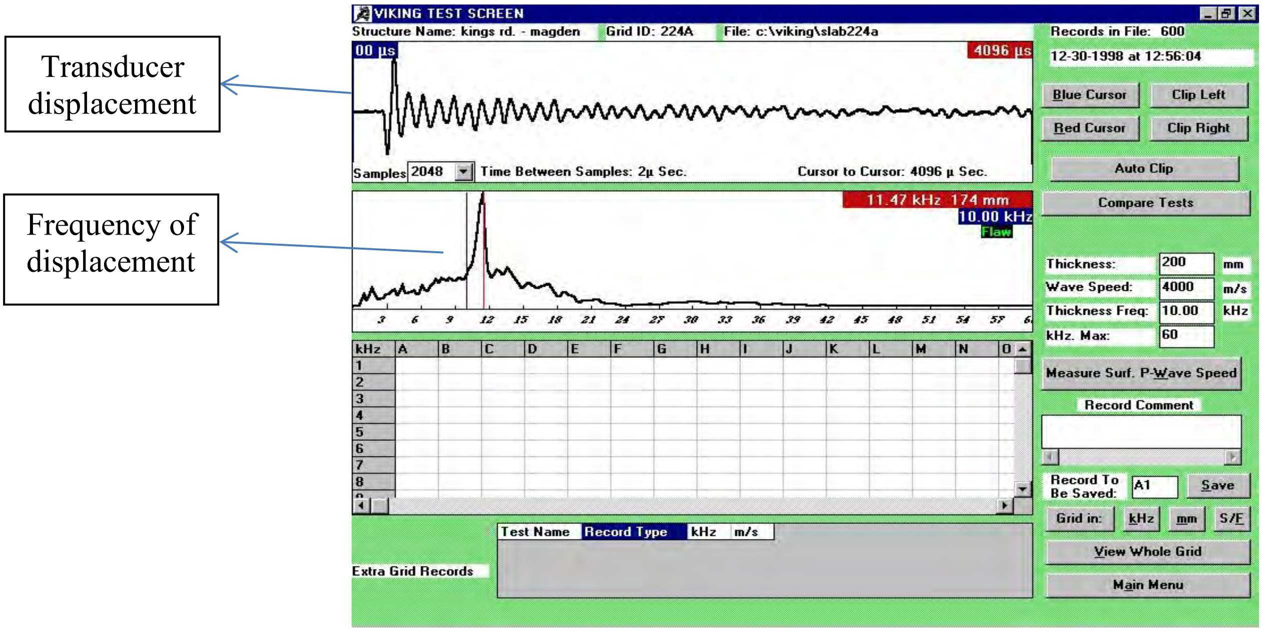


Fig 3.2: Example of test curve with the DOCter

The transducer displacement waveform (upper window in ***Fig. 3.2***), is transformed into the frequency domain to produce an amplitude spectrum, which shows the predominant frequencies in the waveform (lower window in ***Fig. 3.2***). The frequency of P-wave arrival is determined as the frequency with a high peak in the amplitude spectrum. The thickness (T) of the member is related to this thickness frequency (f) and wave speed (*Cp*) by this simple approximate equation:

$$T = \frac{C_p}{2f}$$

The same principle applies to reflection from an internal defect (delamination or void). Thus, the impact-echo method is able to determine the location of internal defects as well as measure the thickness of a solid member.

4 Preparations before testing

4.1 Calculations

At first the settings must be correctly adjusted. The wave speed (*Cp*) must be determined by the equation

$$Cp = 2f_T T$$

f (thickness frequency is determined by carrying out an impact echo reading with the DOCter in an area with solid concrete without flaws. By doing this you receive the thickness frequency (*f*, in kHz). *f* was determined to **5,85 kHz.**

T (Thickness of member) Determine the thickness of the member by measurements if accessible, if not drilling of a hole through the member can be made for measurements or by reviewing drawings etc. The test slab allowed for measurements with a ruler and was determined to **360 mm.**

The acquired information of *f* and *T* gave the following wave speed.

Cp = (2*5,85)360 mm = **4219 m/sek**

All the previously unknown variables are accounted for and measurements can be carried out.

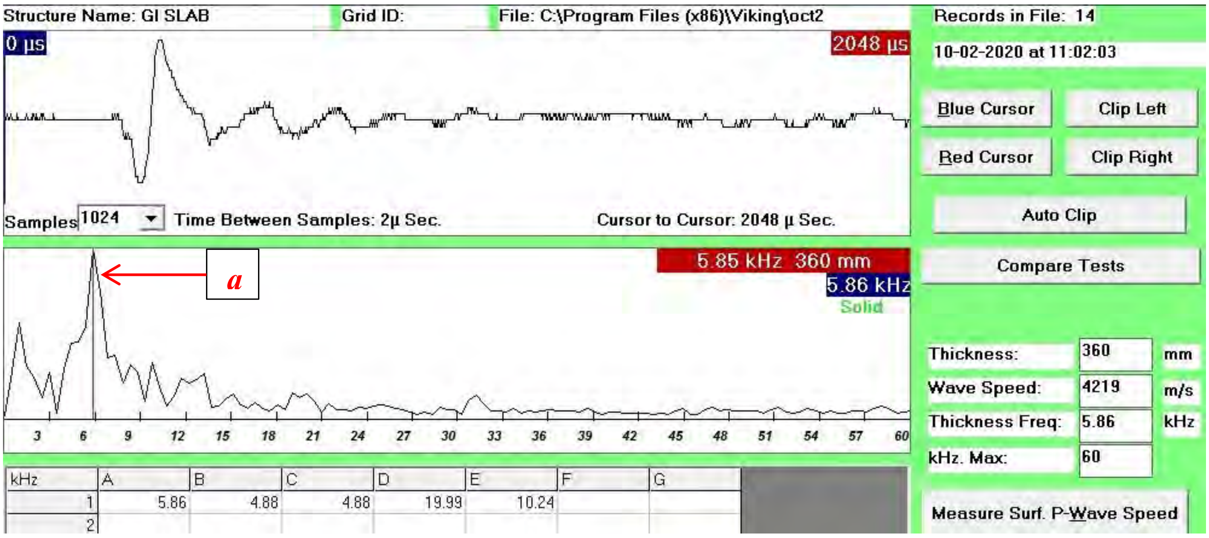


*Fig 4.1: Readings from test of slab thickness. Note that if the thickness frequency peak (**a**) is not displaced related to the red cursor→ indicates solid concrete if correct values are set.*

5 Test results

5.1 Fully grouted duct with strands

Measurements were carried out directly above the fully grouted duct. Because of the thin steel wall of the duct there is nothing in the frequency spectrum that reveal that the signal passes through metal. If the test had been carried out in an area without encapsulated steel and solid concrete the signal would have had a similar appearance.

The reading from the test shown in ***fig 5.1.***

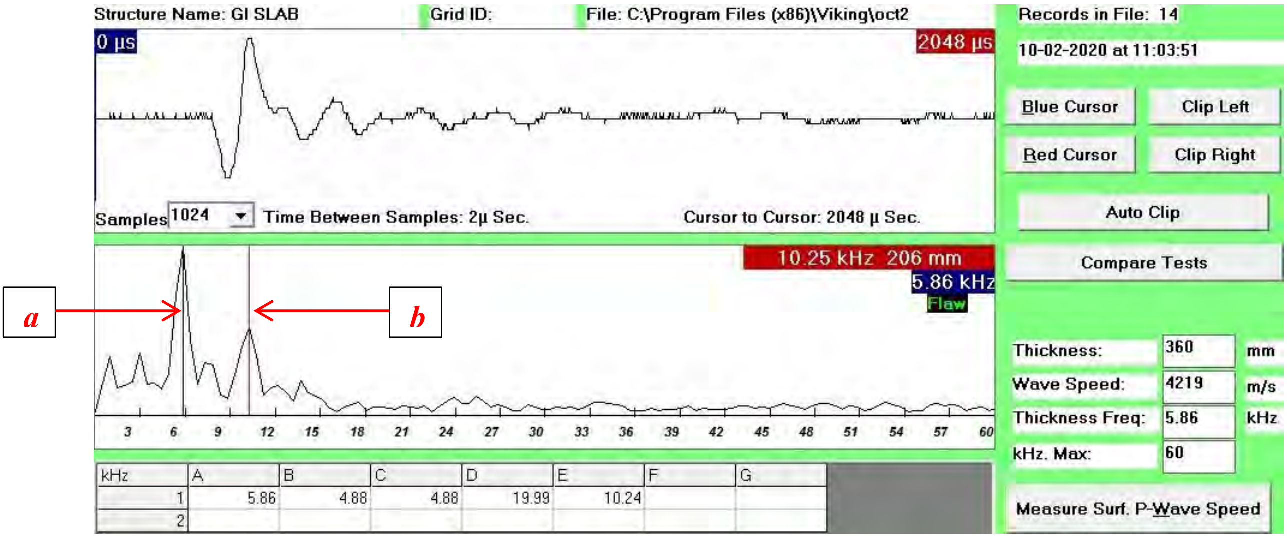


*Fig5.1: Results from measurements on fully grouted duct with 12 mm impactor. Note that the thickness frequency peak (**a**) is not displaced related to the black cursor→ indicates no entrapped air. (**b**) Reflection from one of the strands in the duct.*

5.2 Empty duct

5.2.1 12 mm impactor

Measurements were carried out directly above the empty duct. As can be seen in ***fig 5.2.1*** *(lower window)* the displaced thickness frequency peak indicates a void but the low frequency range does not reveal information of where the void is placed in the structure.

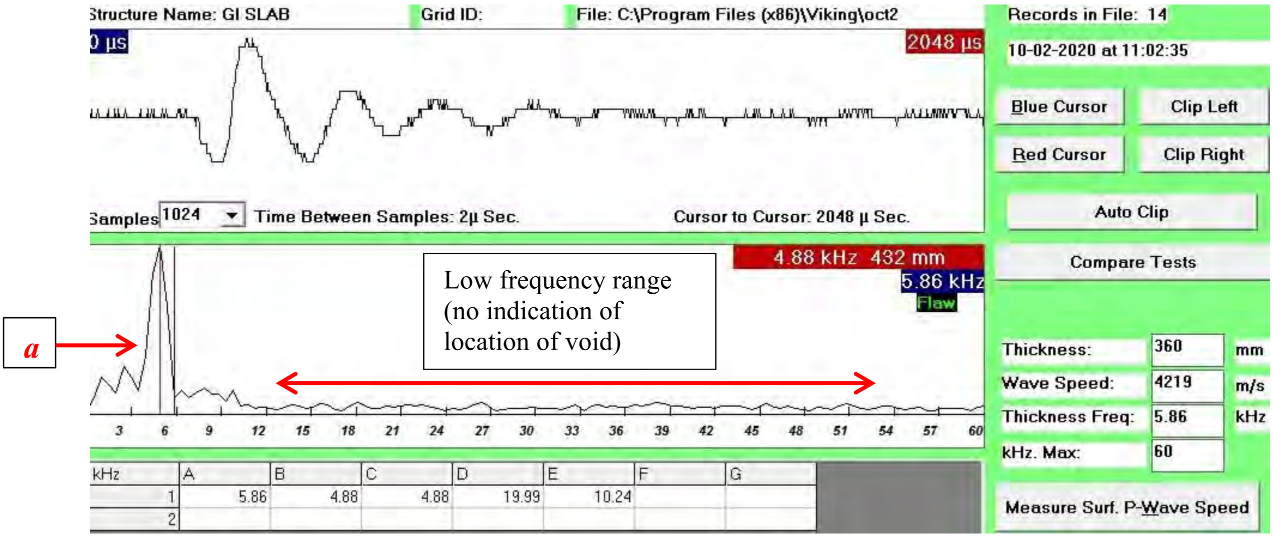


Fig 5.2.1: Results from measurements on the empty duct with no strands with the 12 mm impactor. Note the displacement of the thickness frequency peak (a) in correlation to the cursor which indicates that the signal travels a longer way due to the air entrapment. The low frequency range indicates that a smaller impactor can be used to locate the depth of the void.

5.2.2 8 mm impactor

Testing with the 8 mm impactor showed more promising readings. However multiple impacts often need to be carried out when the spectrum does not show the expected. An example of a test with questionable readings in ***fig 5.2.2*** despite testing with the appropriate size diameter impactor. Testing often require that multiple impacts are carried out.

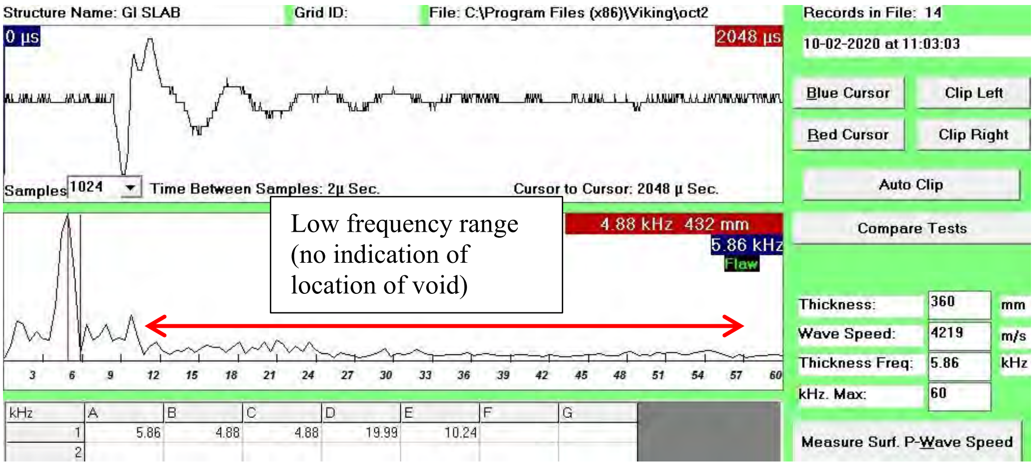


Fig 5.2.2: Partly successful reading from measurements on the empty duct with size 8mm impactor. Still no clear reading of location of the void.

Figure 5.2.3 shows reading from a successful measurement. The thickness frequency peak (*a*) is shifted and there is a reflection peak (*b*) in the correct depth which clearly shows that there is a void and in which depth. Information on depth can be obtained by moving the red cursor (*c*) to the tip of the reflection peak.

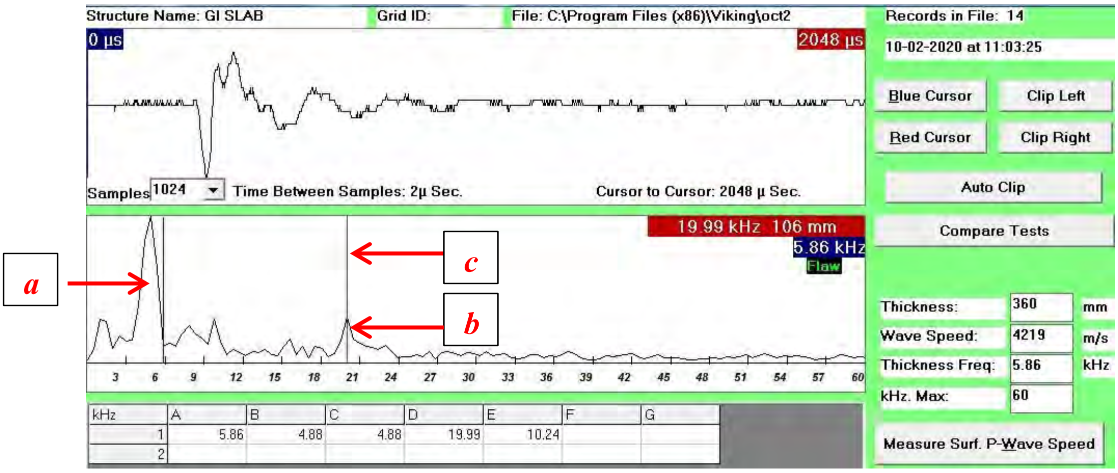
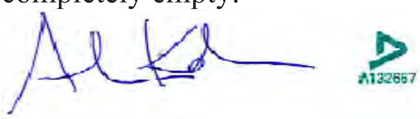


Fig 5.2.3: Successful reading of empty duct with 8 mm impactor.

6 Conclusions

The information that the DOCter provides is similar to the information that can be obtained from scanning with an ultra sound tomographer (for ex. Proceq Pundit). However there are some advantages when using only the DOCter Impact Echo as well as together with ultra sound tomographer.
When reinforcement is tightly placed in a structure it is sometimes difficult to obtain usable information from ultra sound. The DOCter is used in small areas and measurements can more easily be directed on surfaces between rebars and therefore a more suitable equipment when access to areas with less reinforcement is not possible.
It is our assessment that when critical moments such as intrusive drilling into a duct is to be carried out, the DOCter can provide a secondary “safety reading” of which depth the duct is located in. Drilling into ducts with possible damages to the strands can lead to lowering of the carrying capacity.
No testing was carried out on partially grouted ducts but according to the manufacturer it can be obtained “in theory” but in reality, it is difficult to interpret from the frequency spectrum readings if a duct is partially or completely empty.



Andreas Karlsson
/Concrete Inspection

Appendix D – Field test Farrisbru, Larvik

INSPECTION REPORT

Handläggare/Please refer to
Mats Holmqvist
Tel. +46 10 455 13 28
Mob. +46 70 285 10 61
Mats.Holmqvist@dekra.com

Datum/Date	Ref.	Ver.	Sida/Page
2020-05-01	7204-R-297038	1	1(8)

Action	NDT, Farrisedet bru 07-0436, Larvik
Date och time	2020-03-17 & 18
Location	Larvik, Norway
Present	Mats Holmqvist (DEKRA Industrial AB)

1 Background

DEKRA Industrial AB was, as a part of the project "Study of methods, possibilities and limitations for inspection of post stressed reinforcement in concrete bridges", commissioned to carry out an NDT inspection of two bridge segments in Larvik, Norway. The purpose of the inspection was to try out the limitations in the NDT equipment.

1.1 Scope

The inspection was performed with different NDT methods listed below

- (CM) Covermeter (Proseq Profoscope+)
- (GPR) Ground Penetrating Radar (Hilti PS 1000 X-Scan)
- (UPE) Ultrasonic pulse-echo (MIRA)

2 Object

The bridge was built in 1976 and it was torn down in 2018. There were no reported damages in the bridge, and the inspection was therefore focused on finding limitations in the methods.
Two different bridge segments are covered in this report, named 1 and 2.

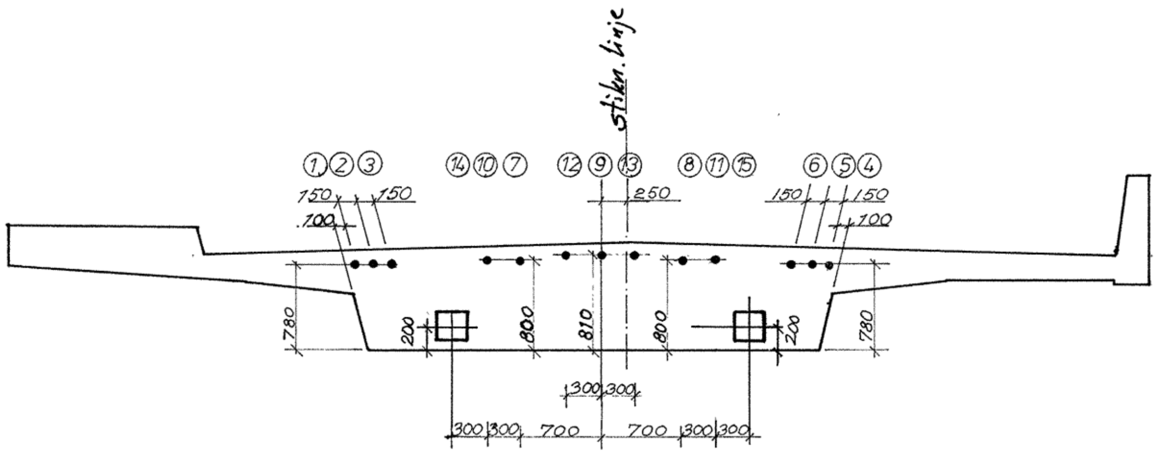
2.1 Segment 1

Segment 1 had five cables and they were placed between 130 – 200 mm down from the top. The width was 2000 mm and the length 2800 mm, the thickness of the segment was 900 mm.

2.2 Segment 2

Segment 2 also had five cables and they were placed between 130- 205 mm down from the top on one side and 450 mm down on the other side. The width 2000 mm and the length 2100 mm, the thickness of the segment was 900 mm.

2.3 Drawing



3 Test results

3.1 Covermeter

With the use of covermeter we located the first layer of rebars with a concrete cover to around 16-25 mm.

The results from the covermeter was later used to calibrate the GPR

3.2 GPR

With the use of GPR, the tendon ducts could be located in the segments.
In segment 1, the tendon ducts were placed with a cover layer of between 130–200 mm.
An area scan was done to get a picture of both tendon ducts and the mesh, ***see figure 3.1***.
In the figure, you can clearly see the tendons that were placed at a depth of about 150 mm. The ducts were marked out on the test body for further inspection with ultrasound.

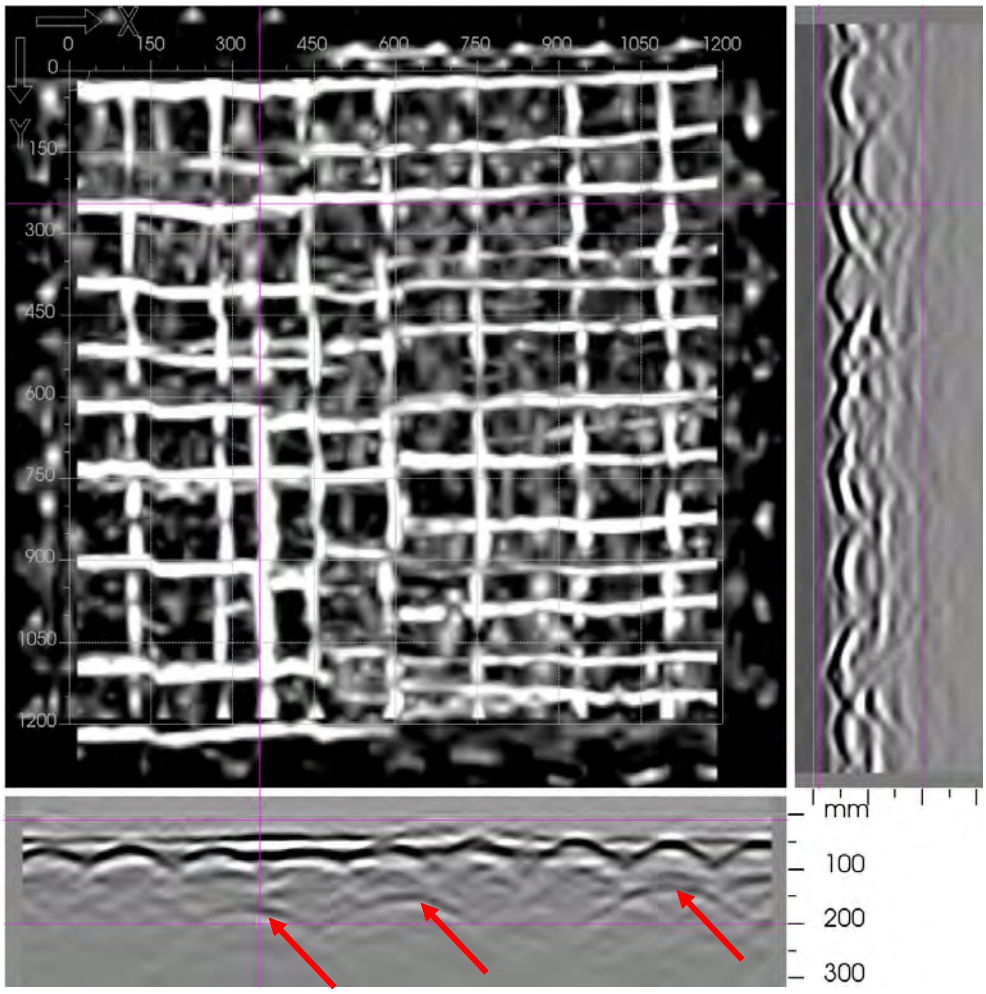


Fig 3.1: Results from segment 1 with the tendon ducts marked with arrows

On segment 2, the ducts were placed at a considerably greater depth, which made it difficult to locate. A scan was carried out on the center of the test body, in this area the ducts was placed with a depth of about 350 mm but these could not be located with radar, ***see figure 3.2***.

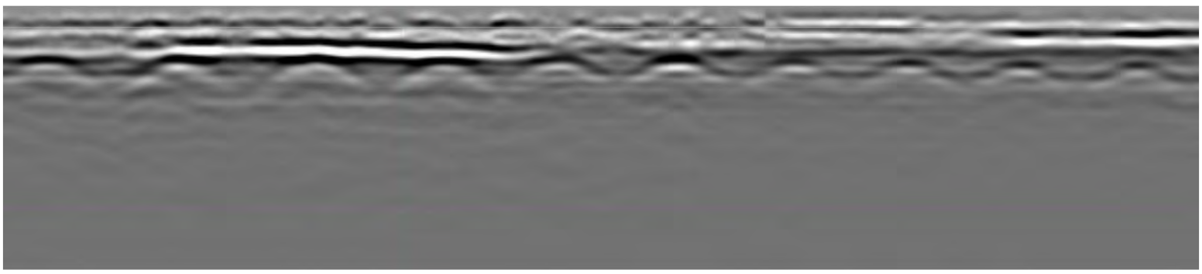


Fig 3.2: Results from segment 2, the tendon ducts couldn't be located

3.3 Ultrasonic pulse echo

On segment 1, a scan was made with the ultrasound on the entire length on two cable ducts. When scanning duct 3 and 4, the full length of the tendon duct was clearly visible, ***see figure 3.3 and 3.4.***

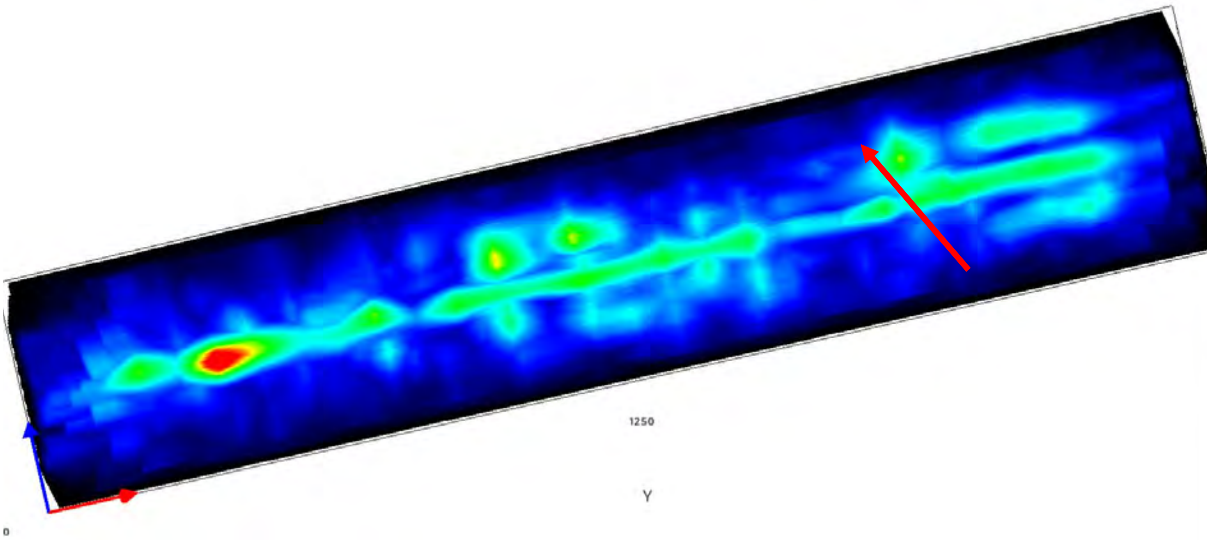


Fig 3.3. Tendon duct 3 in segment 1 at an average depth of about 150 mm.

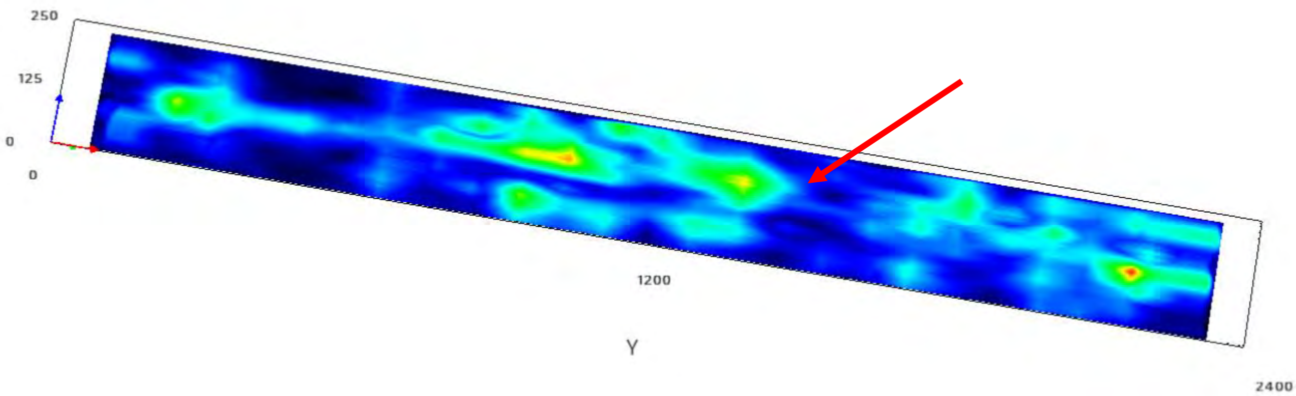


Fig 3.4 Tendon duct 4 in segment 1 at an average depth of 150 mm

Segment 2 had larger variations in the cover layers, and scans were performed on two different tendon duct. The maximum depth of the cable ducts was 450 mm, ***see figure 3.5 and 3.6***.

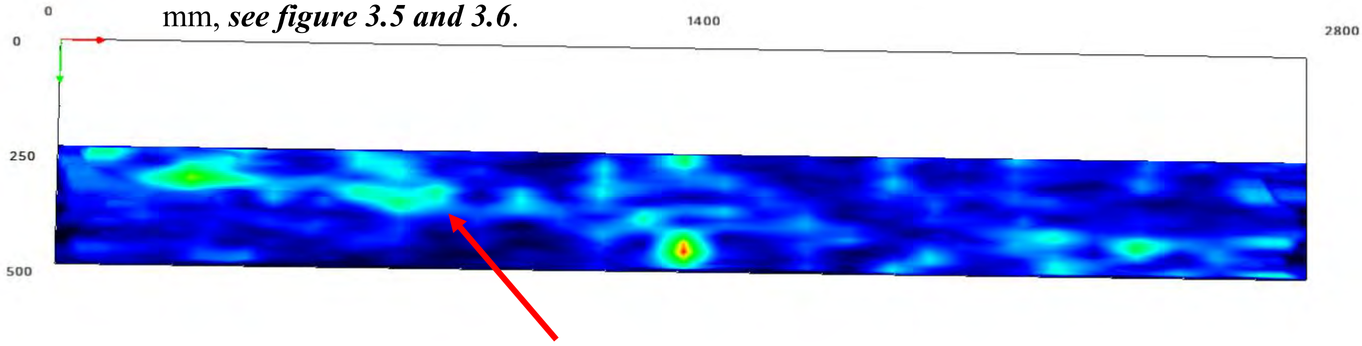


Fig 3.5 Tendon duct 2 in segment 2 at a depth between 175-450 mm

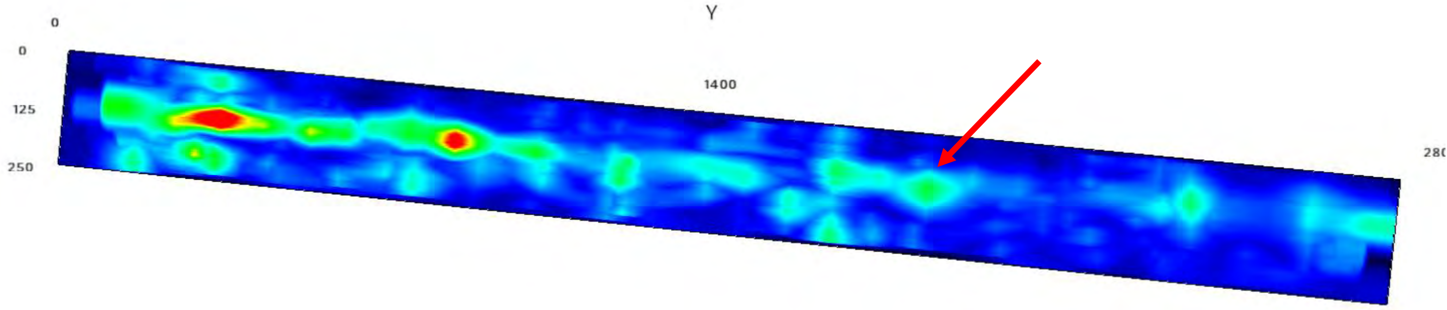


Fig 3.6 Tendon duct 3 in segment 2 at a depth between 165-450 mm

4 Comments

The results of the inspection showed some limitations in the equipment, this was especially true of segment 2, where the tendon ducts were at a depth of about 450 mm.
The concrete cover meter has a limit of about 80 mm which in most cases makes it difficult to locate the ducts. With the cover meter, you also can determine what it is that causes ~~a~~ signal, whether it is a rebar or a tendon duct. Nevertheless, the concrete cover meter is an important part of the inspection as it is a very accurate method with an accuracy of localization of ± 1–4 mm depending on the thickness of the cover layer.

To be able to determine, with radar, how deep the tendon ducts are placed in the concrete, it is necessary to be calibrated against the actual cover layer. The reason for this is that the speed of the signals in the concrete depends on the quality of the concrete. With a known value of the reinforcement depth, the dielectric constant can be calculated and used in the analysis of the GPR results.

The results from GPR showed that the ducts in segment 1 could be located without major difficulties. We also got good pictures of the reinforcement which was located above the ducts.
The ducts were located at approximately 150 mm and could be marked out on the test body, ***see fig 3.1***.
In segment 2 the result was poorer, here the GPR scans could not find the ducts, except for the parts that were placed shallower than 250 mm, ***see fig 3.2***.

The ultrasound measurements were performed on two ducts at each segment.
In segment 1, the ducts were clearly visible during the scans and there were no signs of any voids.
In segment 2, it was more difficult to locate the ducts, because they were placed much deeper, extensive data processing and examination of the results in the computer were required.
The signals were sometimes very weak which is partly because they are injected, but also that they are placed in a depth between 200-450 mm.

5 Conclusions

The tests that were carried out at Farris Bridge were of great use to better learn the instruments and their limitations.

- Cover meter can withstand a depth of max 80 mm which makes it difficult to locate the tendon ducts. With the covermeter it is also not possible to distinguish between reinforcement and ducts.
  Covermeter is a simple method that can be learned quickly, no major knowledge requirements are needed to learn how to use the equipment. However, it should be noted that the greater knowledge the user has of construction technology, the better and more accurate results are obtained.

- GPR is a fast and good method for tendon ducts. The limits in depth are about 250 mm and it needs to be calibrated to a known depth for the depth to match. However, no calibration is required if only the ducts are to be located. With the software in the computer it is possible to process the results afterwards, and thus be able to study the scans that have been carried out more closely.
  With a GPR you get a good overview of the location of the reinforcement, and this without any major knowledge requirements.
  To be able to determine more precisely the depth of the reinforcement and the cable ducts, more advanced knowledge of the equipment and its use is required.
  Knowledge of construction technology is also required to be able to distinguish the reinforcement from tendon ducts

- Of the three methods discussed in this report, ultrasound is the most advanced equipment. With the right settings, it manages to locate at a depth down to about 1 meter. To get maximum results from the ultrasound, a lot of preliminary work is required.
  Partly with drawing reviews but also with the mapping of the construction using GPR and cover meter measurements.
  With the ultrasound, the signals are strongest as there are cavities in concrete and ~~cable ducts~~, as most of the signals are reflected when it finds air.
  Therefore, it can be difficult to locate tension that are well injected because only a small number of signals are reflected.
  It is a time-consuming method as the results require imaging in the computer to be analyzed. This means that a user with extensive experience in the equipment must be in place at the inspection.

Appendix E – Field test Ölandsbron, Kalmar

Handläggare/Please refer to
Mats Holmqvist
Tel. +46 10 455 13 28
Mob. +46 70 285 10 61
Mats.Holmqvist@dekra.com

Action	Locating tendon ducts
Date och time	2019-10-14
Location	Kalmar, Sweden
Present	Mats Holmqvist (DEKRA Industrial AB)

1 Background

DEKRA Industrial AB was, as a part of the project "Study if methods, possibilities and limitations for inspection of post stressed reinforcement in concrete bridges", commissioned to carry out an NDT inspection of a bridge in Kalmar, Sweden. The purpose of the inspection was to locate the tendon ducts.

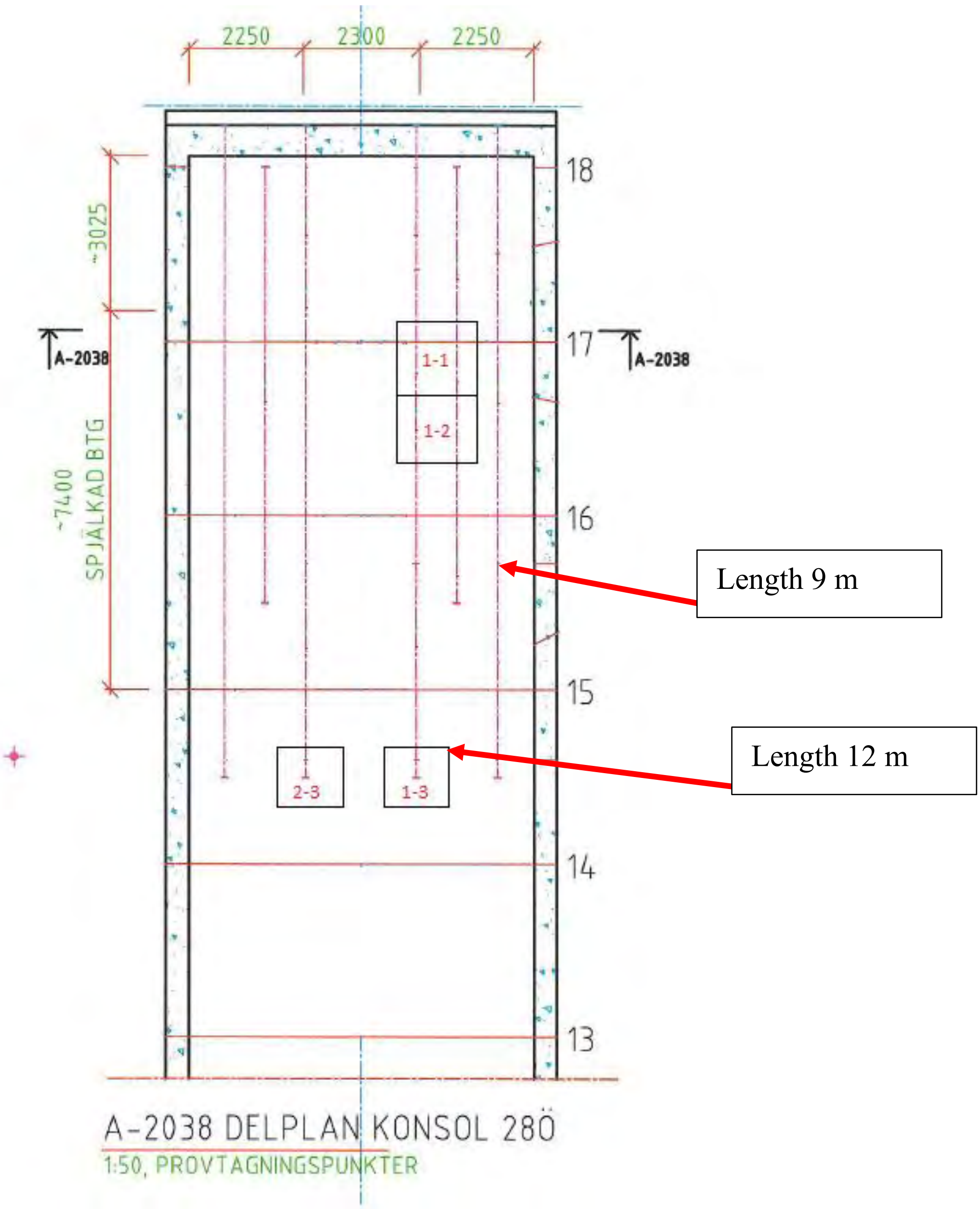
1.1 Scope

The inspection was performed with different NDT methods listed below

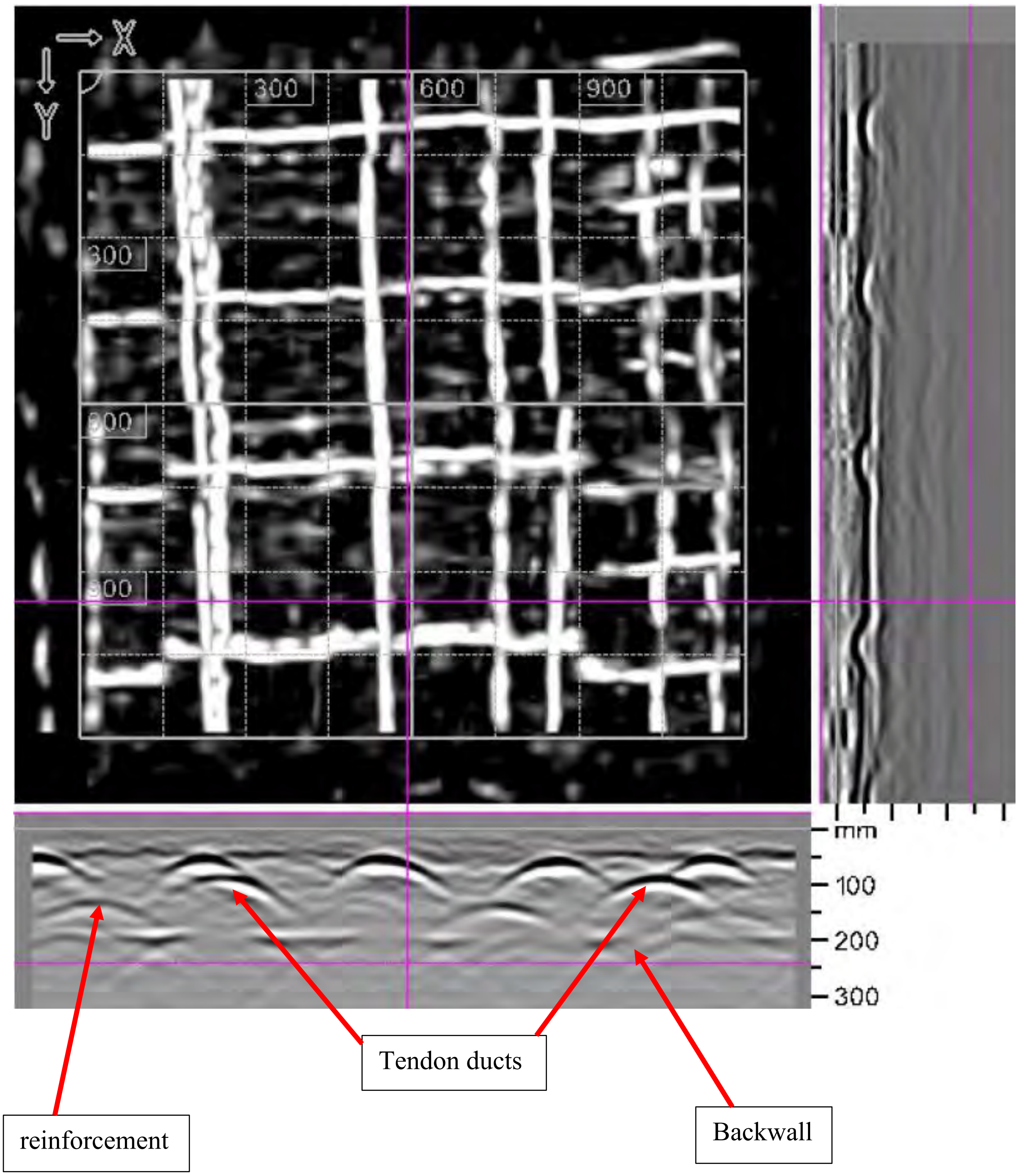
- (CM) Covermeter (Proseq Profoscope+)
- (GPR) Ground Penetrating Radar (Hilti PS 1000 X-Scan)

2 Results

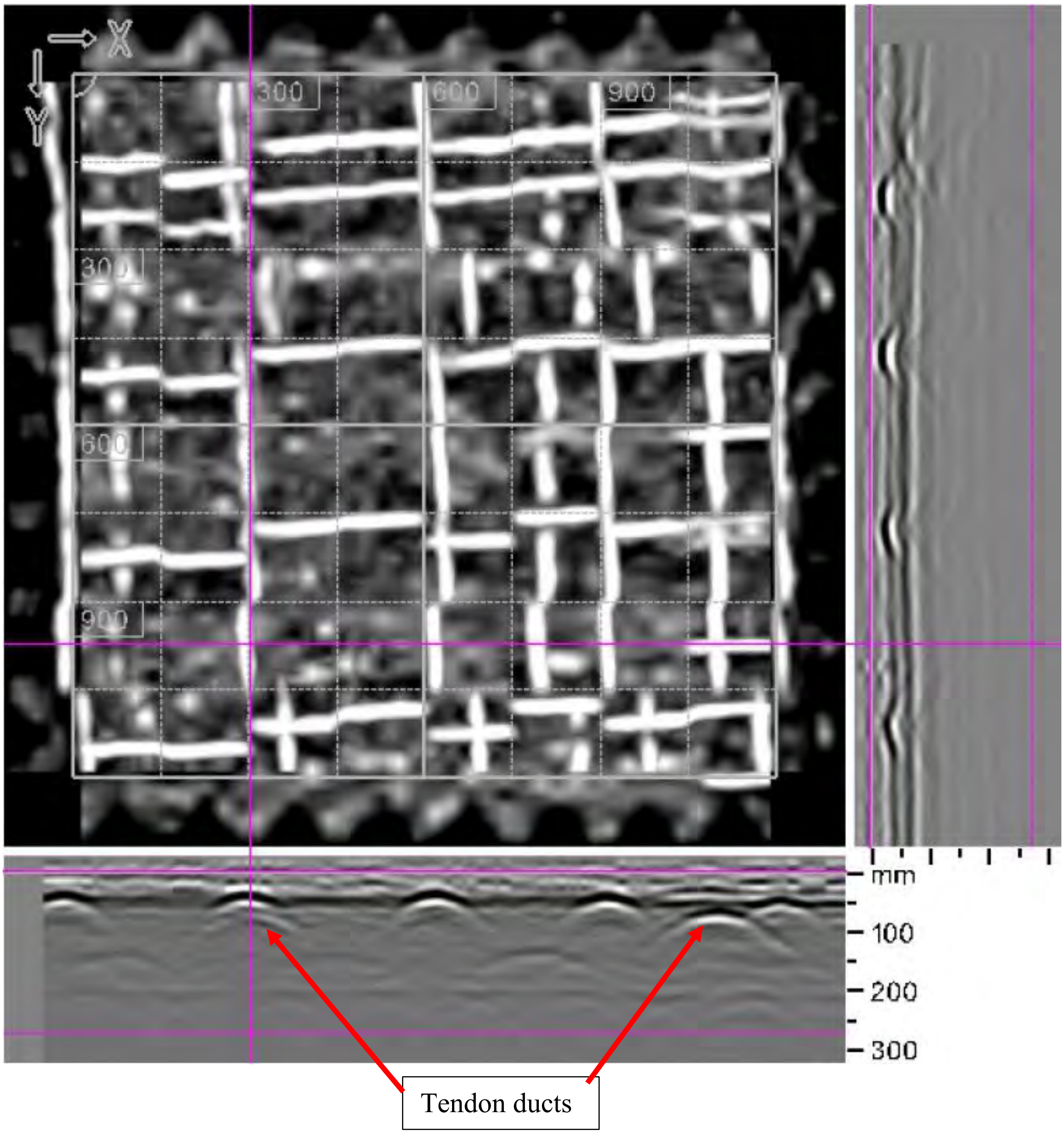
A scan of the position of the tendon ducts was performed to ensure that it was positioned according to the drawing. The tendon ducts were marked in place to facilitate the repair.
The work surface was bounded by a splash guard all around and therefore the two outermost tendon ducts could not be located.
Four scans were saved for accounting and the locations of the scans are shown in the following sketch.



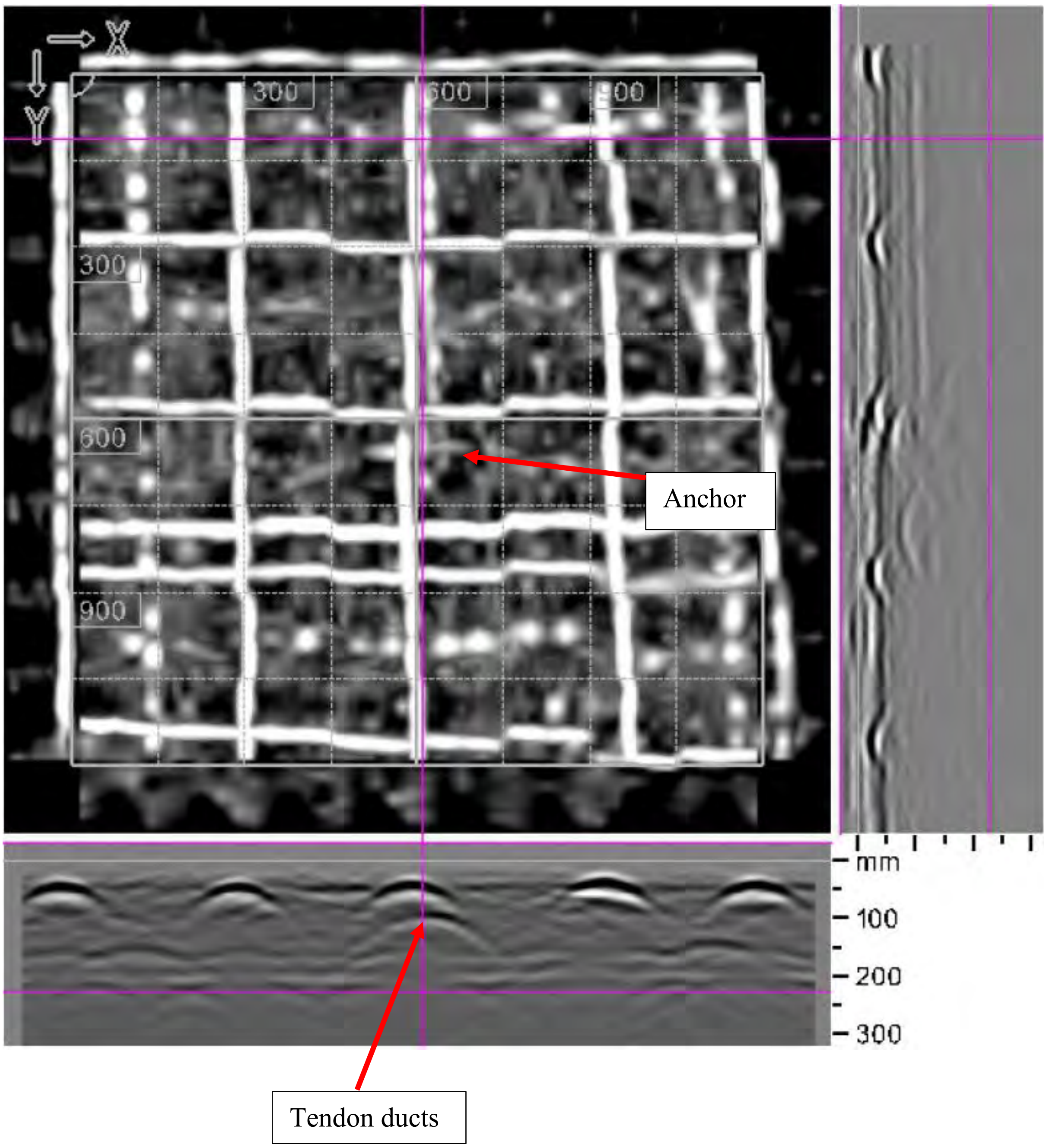
2.1 Scanning area 1–1



2.2 Scanning area 1–2

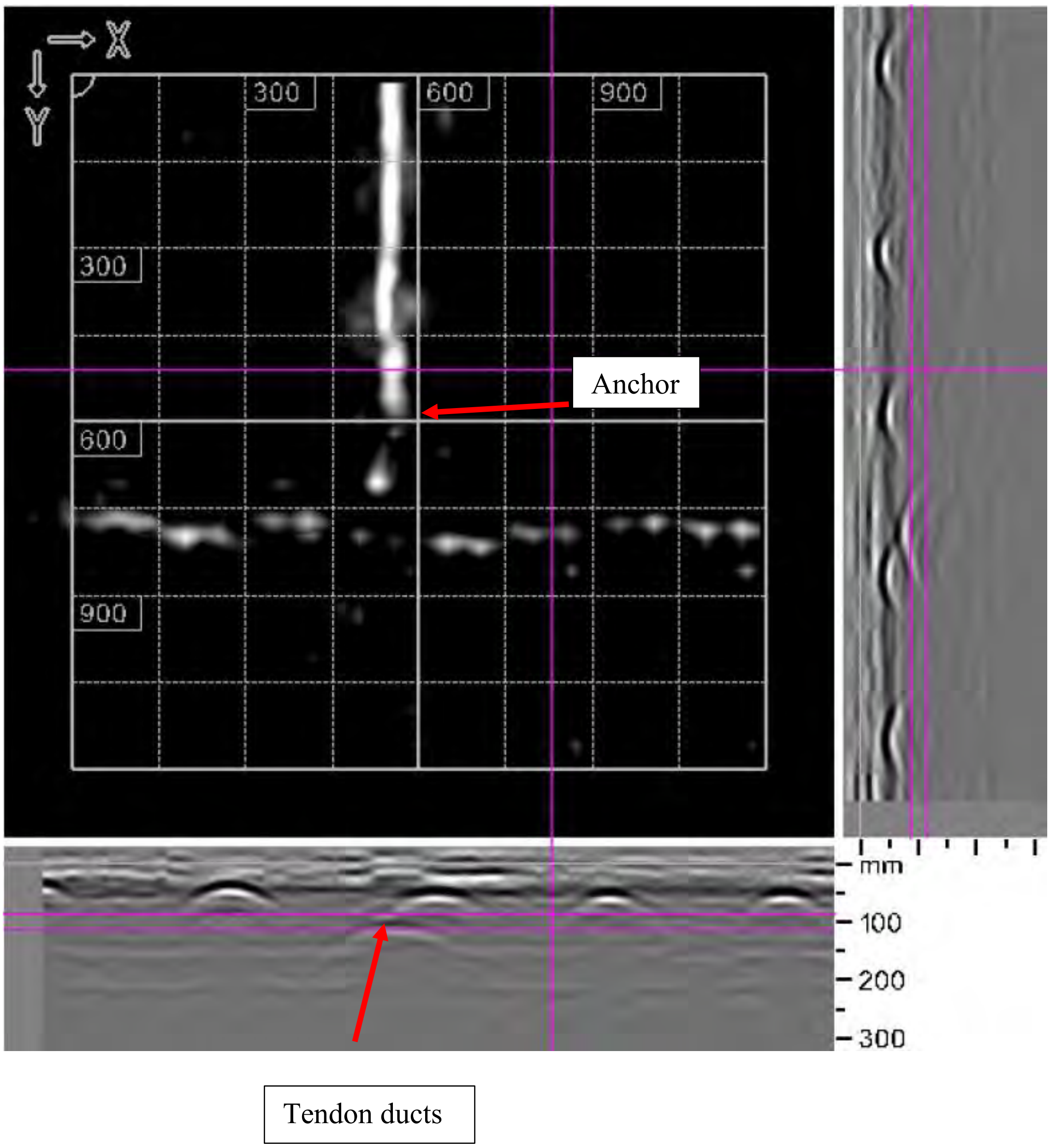


2.3 Scanning area 1–3



2.4 Scanning area 2–3

In this picture, the top edge reinforcement has been hidden and only the-tendon duct is visible.



3 Comments

GPR is a good choice of equipment for locating the tendon ducts.
It is relatively easy to use and it is a quick method. The disadvantage is that it has a limit of about 250 mm in depth, which means that it cannot always find ducts that are placed further into the concrete.

It is also difficult to distinguish between reinforcement and tendon ducts, in the present case the reinforcement has the dimension Ø10 mm and the tendon ducts Ø32 mm.
There are no difference in the signals and if the inspector only looks at the results from the scans, he cannot say with certainty which is tendon ducts. Therefore, a proper drawing review and experience is required to distinguish the two types of reinforcement.

Appendix F – Field test Herøysundsbru, Herøy

REPORT

Herøysund bridge

Locating voids in grouted tendon ducts with NDT

REPORT: Ref. 7204-R-584522-Ver.2

Author:	Mats Holmqvist (DEKRA Industrial AB)
Contact:	Roy Antonsen (Norwegian Road Administration)
Reviewed by:	Andreas Karlsson (DEKRA)/Björn Täljsten (Invator AB)
Date:	2020-02-18

TABLE OF CONTENT

SUMMARY

DEKRA Industrial got an inquiry from the Norwegian road administration to perform an NDT inspection with the goal of finding voids inside tendon ducts at Herøysund bridge in Norway.

The following methods were recommended by DEKRA to execute the inspection:

- Cover meter
- Ground Penetrating Radar
- Ultrasonic
- Impact Echo
- Visual inspection/drilling

The results showed that all 8 ducts in the girders, at some point, were missing grout. This was confirmed in four ducts with drilling and visual inspection. In one of the drilling holes we also noticed 3 wire breakages.

The results and methods are described in the following report.

A430245

Mats Holmqvist

Tel. +46 10 455 13 28
Mob. +46 70 285 10 61
mats.holmqvist@dekra.com

1 BACKGROUND

The Herøysunds bridge on the west-coast in Nordland Fylke in Norway is undergoing concrete repair works with focus on reinforcement corrosion. During these actions, it was discovered that the tendon ducts had loss of injection. Concrete cover was removed with hydrodemolition and voids was found in the duct in some of these areas.

To ensure the safety of the bridge it was decided to map voids and defects in the ducts. Several risk areas were pointed out by the constructor and a non destructive investigation was carried out. This report describes the process for investigation and the results.

The inspection was made: week 4, 2020.

2 OBJECT / SCOPE

The Herøysunds bridge is located between the islands South and North-Herøy at the Norwegian west coast in Nordland fylke.
The bridge was built in 1966 and has a total length of 154 meters, with the largest span 60 m. The two girders on the north and south side have each four post-stressed cables with the anchors placed at axis 6 in the west and axis 3 in the east. Each cable has two joints placed about 15 meters on each side of the middle of the bridge. The girders are 400 mm thick and 1000 mm high from the bottom to the underside of the bridge deck.

A view and a cross section of the bridge is presented in figure 2.1

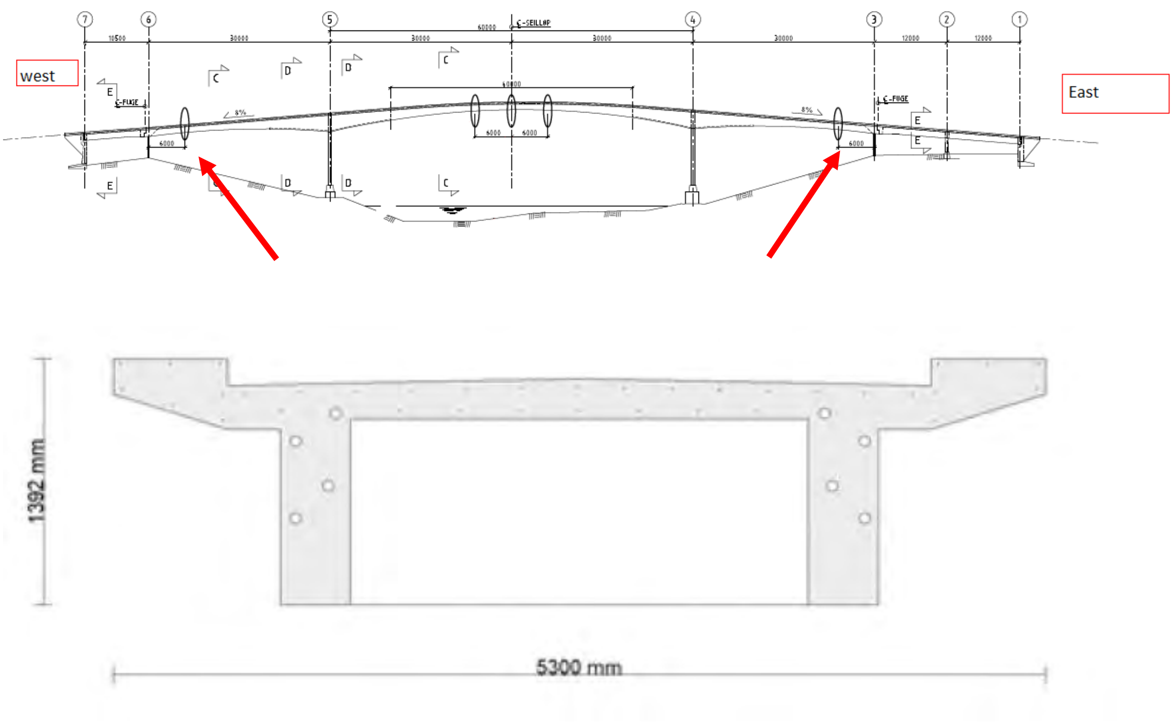


Figure 2.1 View and cross-section of Herøysunds bridge

The scope of the Non Destructive Testing (NDT) inspection was to determine voids in the cable ducts at the areas which had been pointed out by the constructor. In the project we followed a strict procedure presented in figure 2.2. Here it is important first to make a detail investigation of the problems and the problems to be investigated, this then forms a basis for the methods that should be used in the investigation. Based on this we decided to use a combination of different NDT methods listed below:

- Cover meter
- Ground Penetrating Radar
- Ultrasonic
- Impact Echo
- Visual inspection/drilling and the use of endoscope

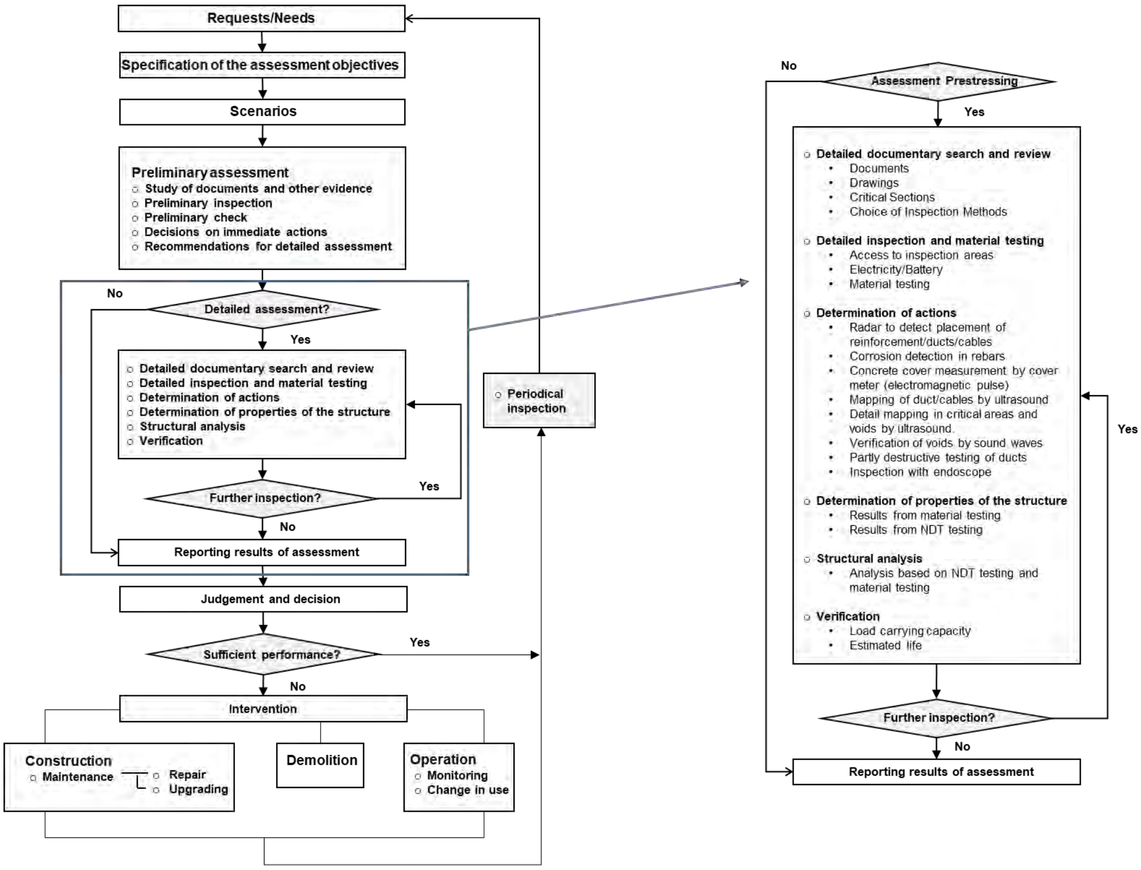


Figure 2.2 Procedure for investigating the defects in Herøysunds bridge

It should be mentioned that considerable experience is needed when these methods are combined to investigate voids and defects in tendon ducts. In addition it is difficult to determine the degree of grouting in the duct and the NDT methods often need to be combined with partly destructive testing, i.e. a hole needs to be drilled into the duct and closer investigation with for example endoscope might be needed.
The results from the NDT, presented in this report, give a good overview of the voids in the investigated areas. We were able to identify where the voids are located and also the size of the voids. In the next section, the different used NDT methods are briefly described.

3 INSTRUMENTS/ METHODS

3.1 Cover meter:

The cover meter is an electromagnetic method used to locate the reinforcement and measure the concrete cover. The method is however limited to a depth of about 80 mm and it is mostly used to identify placement of steel reinforcement. It has limitation to identify tendon ducts since they in general are place quite deep into the concrete member

3.2 Ground Penetrating Radar:

The GPR method involves emitting electromacnetic pulses from an antenna and receiving the reflected pulses from internal reflectors.
The method is used to locate rebars; the advantage with radar is that it is possible to locate tendon ducts at a depth up to about 200-250 mm.

3.3 Ultrasonic:

Ultrasonic method uses a dry-point-contact transducer that generate shear waves. A group of sensors emits a stress pulse into the specimen. As the waves propagate, areas with changes of impedance reflect potions of the wave, and these reflections are captured by another sensor.
Ultrasonic is used to locate air voids in the concrete and tendon ducts. When the duct is correctly grouted the signal travels through the duct and you will get a weak signal. If there is a void the reflecting signal is stronger. The disadvantage with the ultrasonic technique is that even if there are small voids in the duct, you will get a strong echo. This means that the tendons may be filled with grouting even though a strong signal is obtained.

3.4 Impact echo:

In order to confirm the results from the ultrasonic testing, we use the impact echo method. The method is based on arrival of reflected stress waves and can locate the depth of internal delamination and voids in the duct. This is important, because depending on the reflection wave, we can ascertain if grouting in the ducts is truly missing.

3.5 Visual inspection

In this case, with several different measurement ranges, we had to learn how to interpret the results. Therefore, we uncovered the duct with drilling in four selected areas. Two of them were made in ducts where we suspected missing grout and two where we got results from full grouting. This method can also be combined with an endoscope.

4 TEST PROCEDURE

To locate voids in the duct we use several different methods, each one of them with their own unique advantage.

The cover meter is first used to locate the rebars and measure the cover. This is necessary to calibrate the GPR and make sure that the settings are correct. The GPR is used to locate the tendon ducts and mark them in the construction to facilitate the use of ultrasonic testing.

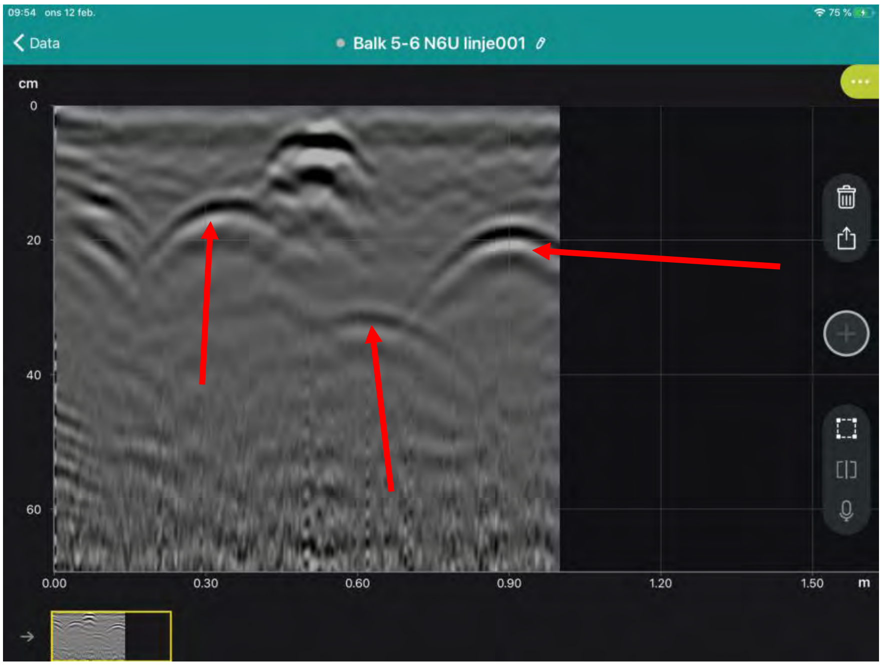


Figure 4.1: Tendon ducts located with GPR

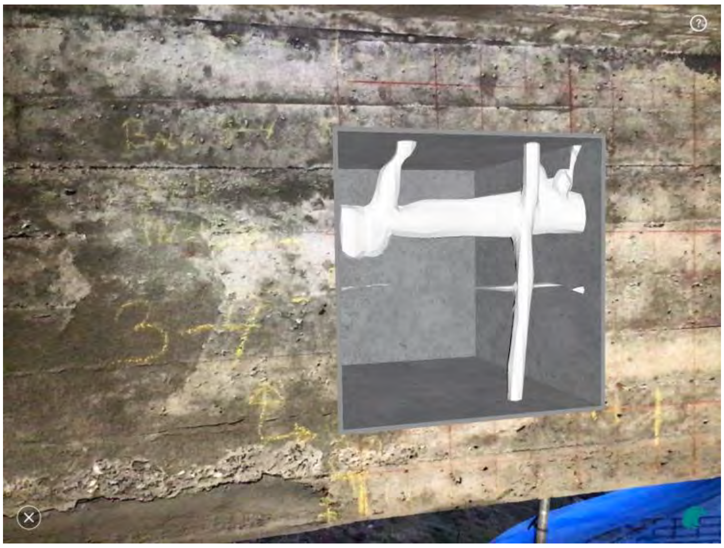


Figure 4.2: Example of 3D image of tendon duct with GPR

When the depth and location of the tendon ducts is confirmed, we know exactly where to use the ultrasonic method. This method is used to find suspicious areas inside the tendon ducts, the results need to be processed in a software and therefore it's important to make the measurements at the right spot at once to avoid unnecessary and time-consuming scans. In each test area, the scans were made from both sides of the girder. This was done because we wanted to be sure that the signals correspond to each other and that we got satisfactory results.

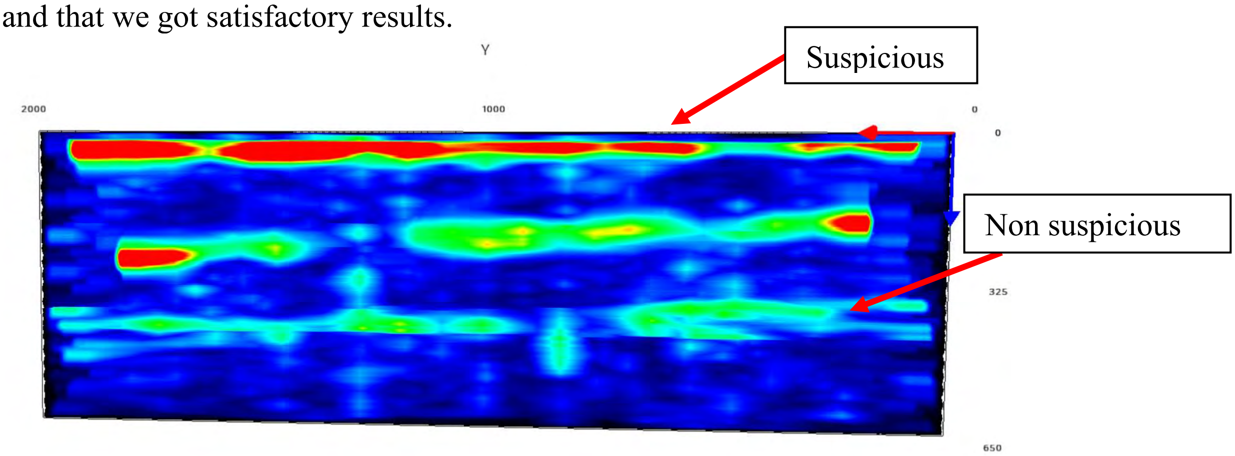


Figure 4.3: Tendon ducts located with Ultrasonic

After scanning with the ultrasonic we locate suspicious and non-suspicious areas in the ducts. To further investigate the suspicious areas, we use Impact Echo to confirm the results. With impact echo, we are looking for abnormal frequencies at the depth of the duct, and differences in the thickness peak. Figure 4.5 and 4.6 below shows examples of tests performed at a mockup specimen.

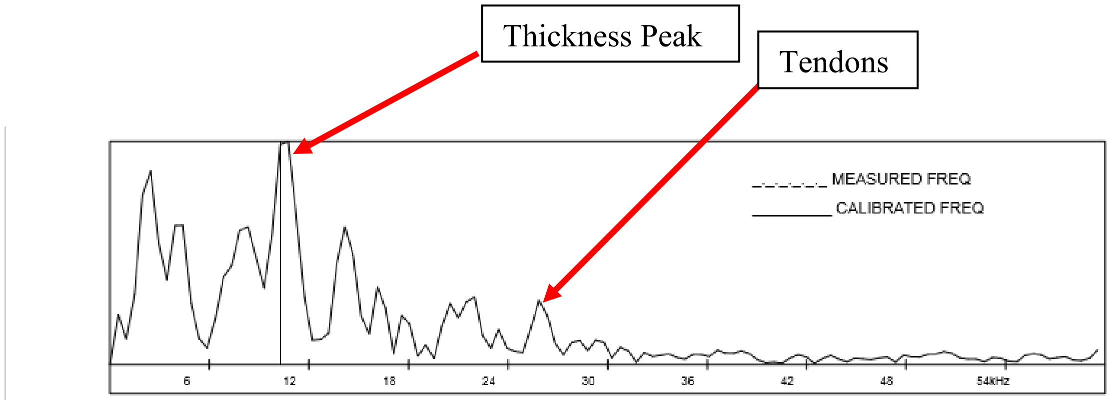


Figure 4.4: Frequency peak in a grouted duct, mockup specimen.

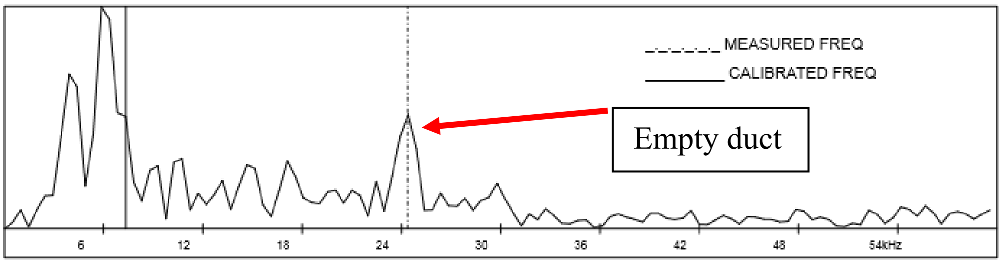


Figure 4.5: Frequency peak in an empty duct, mock up specimen

The following results in figure 4.6 and 4.7 shows the results from two different tendon ducts at Herøysund bridge.

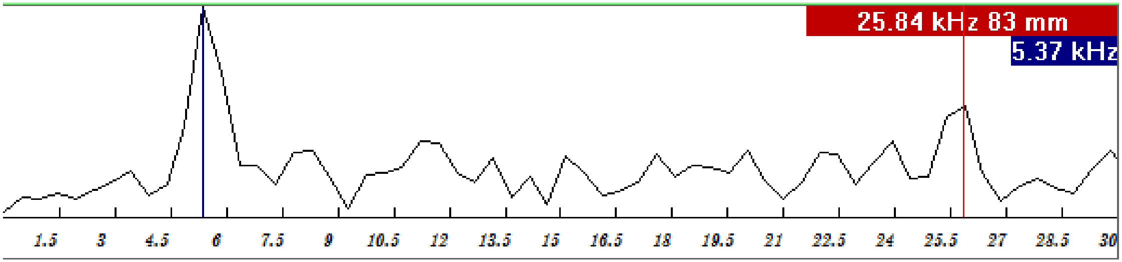


Figure 4.6: Frequency peak in a grouted duct, Tendon duct 2S.

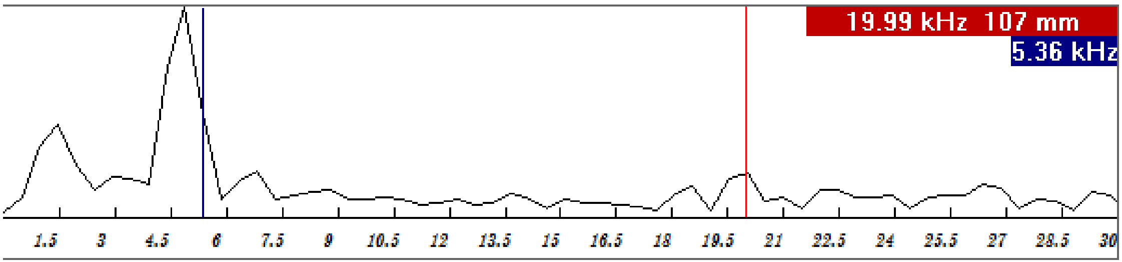


Figure 4.7: Frequency peak in an empty duct, Tendon duct 3SC.

Once the results from Impact echo confirmed the suspicion we got from ultrasonic tests, we carried out a confirmation test in four different ducts. In total, four different ducts were inspected visually thereof two with missing grout and two with full injection, ***see photo 4.1***. Once we got our results confirmed visually, we knew that our results were reliable and accurate.

Photo 4.1: Two different visual inspection areas in girder.

The last step of the investigation is to a data processing of the results and to write the report. With a lot of different investigation areas, this a time-consuming work. At this point we want to exclude the signals with no point of interest to make sure we look at the expected damages. Therefore, we compare all the results from GPR, Ultrasonic and Impact echo.

5 TEST AREAS

The test areas selected by the constructor and was placed as seen in figure 5 below and marked with chapter name.

In the following chapter, the different areas are closer described.

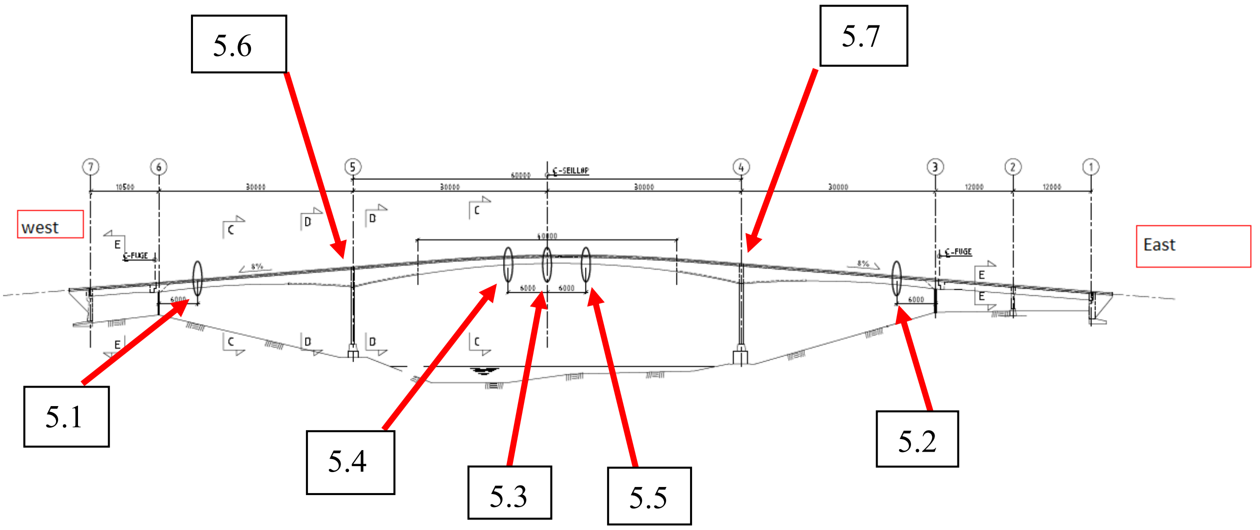


Figure 5: Overview of test areas

Photo 5: Example of test area with grid.

5.1 Axis 6, north and south girder

The investigation was carried out 5 meters from the west support. The tendons where located 400, 600 and 900 mm from the bottom. We could notice that tendon duct 4 was located in the bridge deck and wasn't available for inspection.

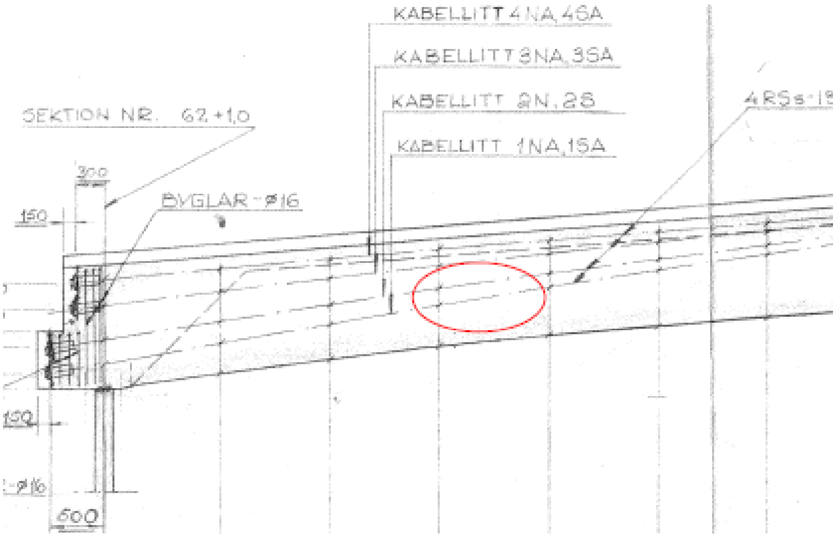


Figure 5.1.1: placement of tendon ducts

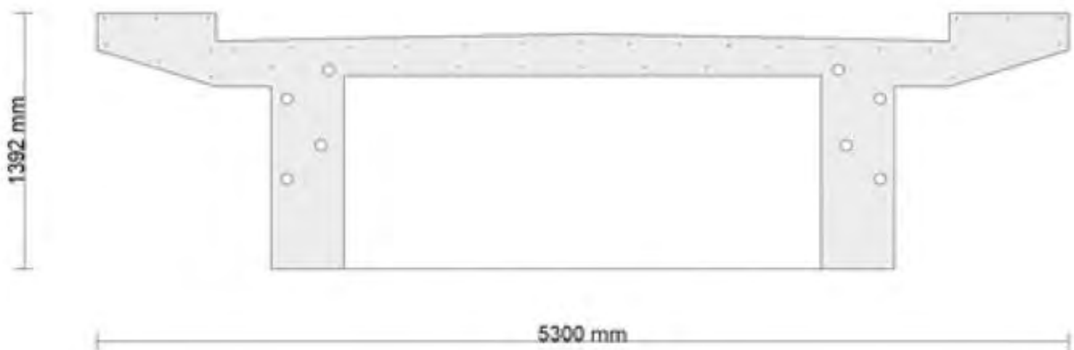


Figure 5.1.2: cross-section at inspection point

5.2 Axis 3, north and south girder

The investigation was carried out 5 meters from the east support. The tendons where located about 400, 600 and 900 mm from the bottom. We could notice that tendon duct 4 was located in the bridge deck and wasn't available for inspection.

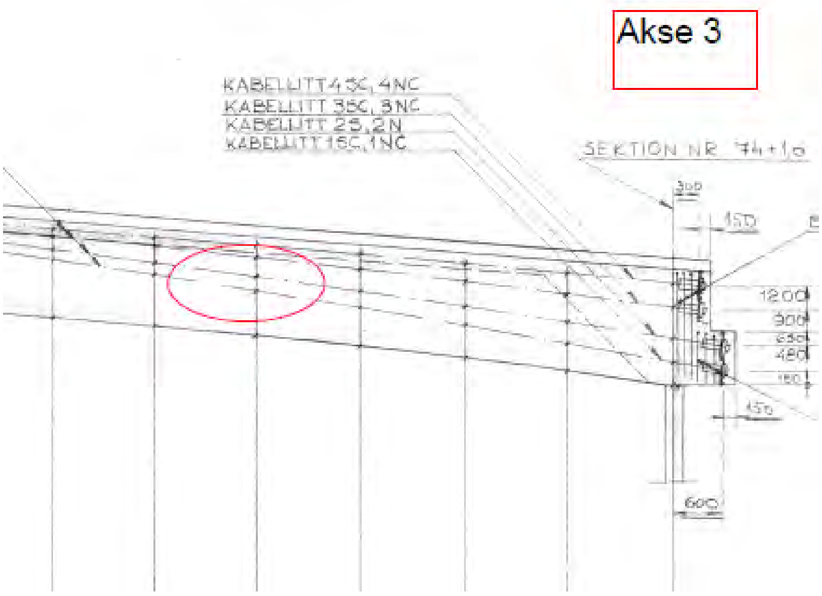


Figure 5.2.1: placement of tendon ducts

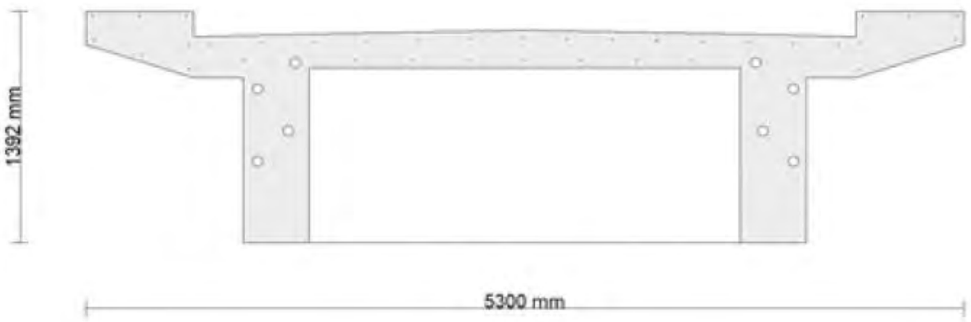


Figure 5.2.2: cross-section at inspection point

5.3 Middle span; north and south girder

The investigation was carried out in the middle of the bridge and 2 meters towards each side. The tendons where located in the bottom of the girder.
All four tendon ducts where available for inspection at this area.

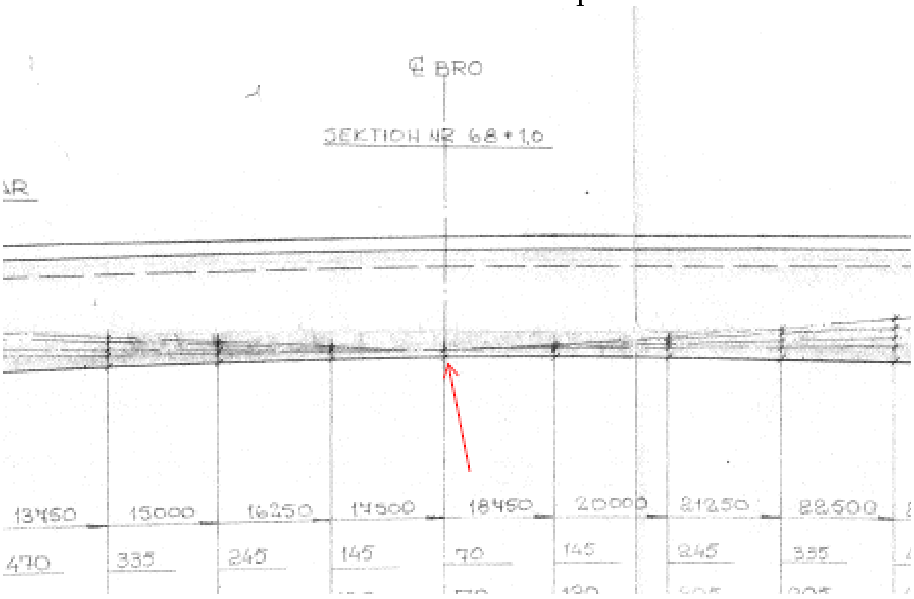


Figure 5.3.1: placement of tendon ducts

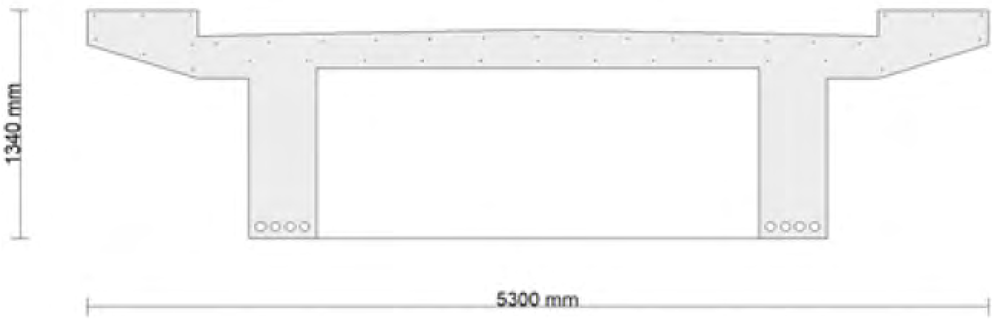


Figure 5.3.2: cross-section at inspection point

5.4 Middle span north and south girder towards axis 5

The investigation was carried out 6 meters from the middle of the bridge towards the west support. The tendons where located about 200, 300, 400 and 600 and from the bottom. All four tendon ducts where available for inspection at this area.

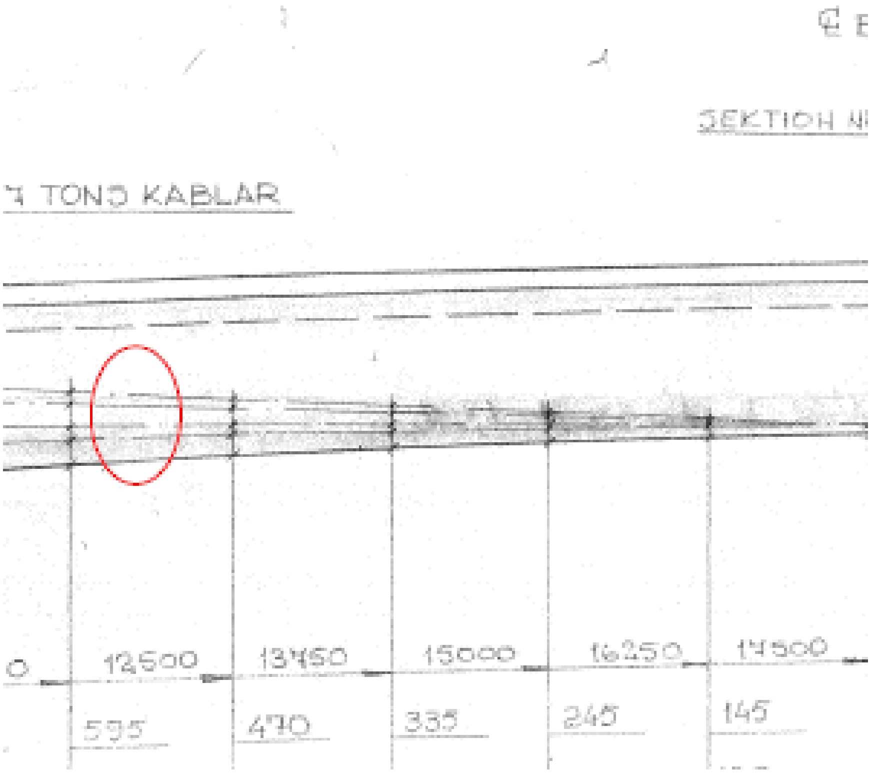


Figure 5.4.1: placement of tendon ducts

Figure 5.4.2: cross-section at inspection point

5.5 Middle span north and south girder towards axis 4

The investigation was carried out 6 meters from the middle of the bridge towards the east support. The tendons where located about 200, 300, 400 and 600 and from the bottom. All four tendon ducts where available for inspection at this area.

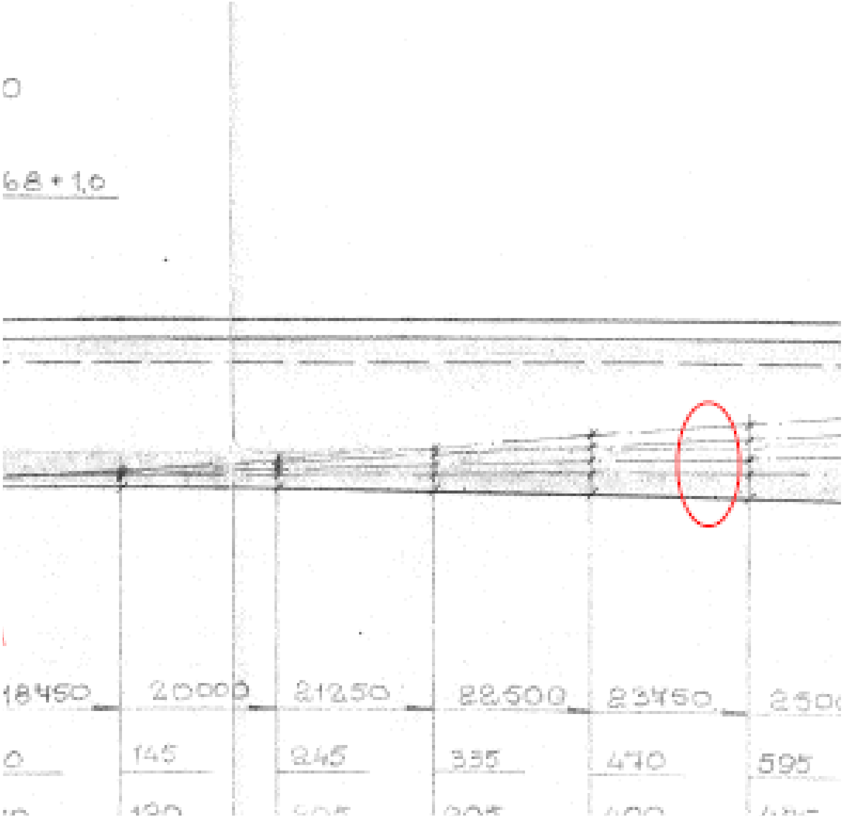


Figure 5.5.1: placement of tendon ducts

Figure 5.5.2: cross-section at inspection point

5.6 Bridge deck Axis 4

The investigation was carried out in two tendon ducts, 10 meters from the support towards each side. Tendon duct 12Ö22 was located in the south side of the deck and 7Ö17 in the north side.

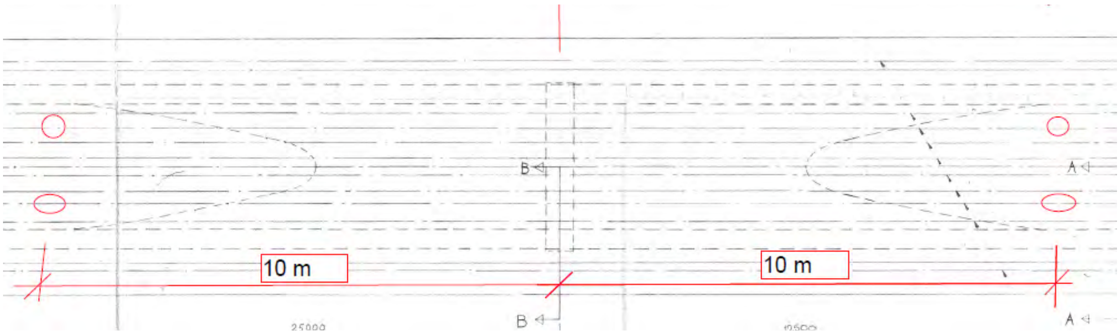


Figure 5.6.1: placement of tendon ducts

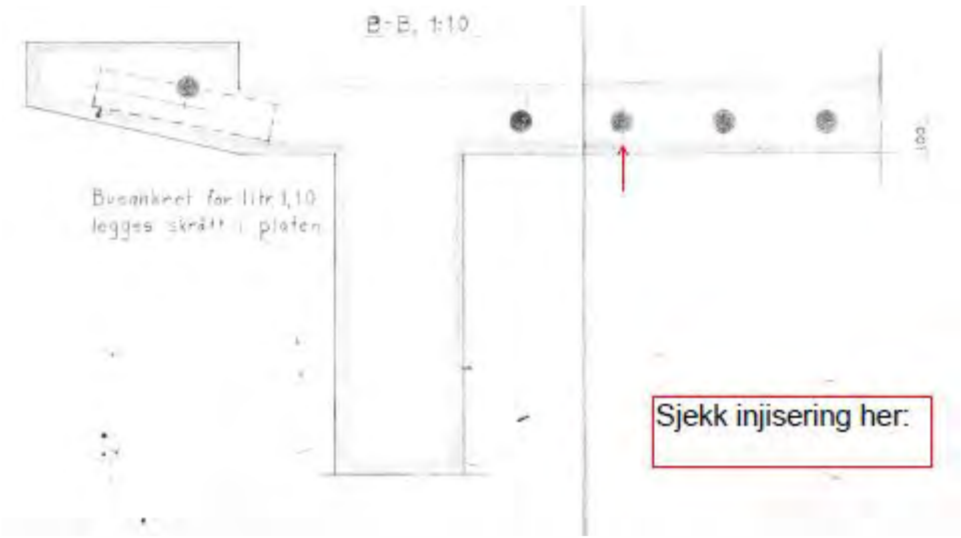


Figure 5.6.2: cross-section at inspection point

5.7 Bridge deck Axis 5

The investigation was carried out in two tendon ducts, 10 meters from the support towards each side. Tendon duct 12Ö22 was located in the north side of the deck and 7Ö17 in the south side.

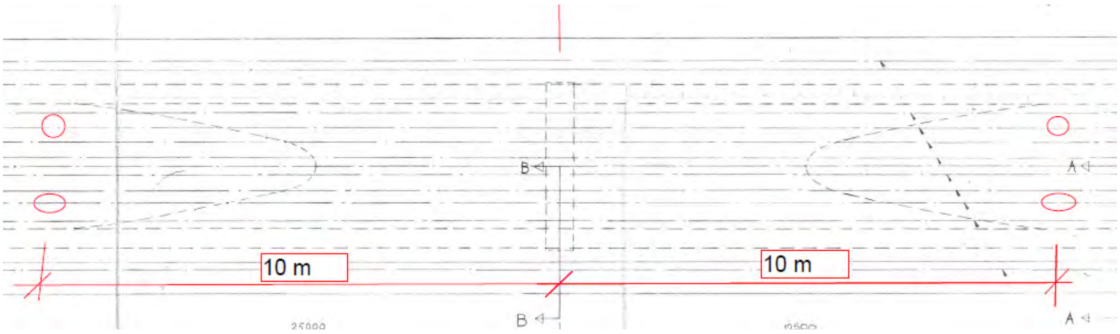


Figure 5.7.1: placement of tendon ducts

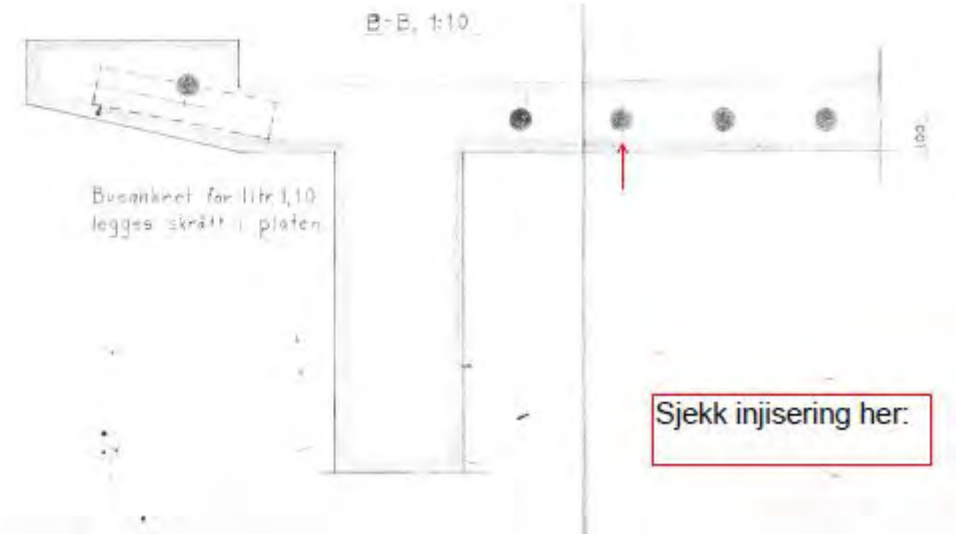


Figure 5.7.2: cross-section at inspection point

6 RESULTS

The investigation showed a successful result with good accuracy in finding voids in the ducts. In some of the areas investigated, no traces of void appeared in the ducts. Despite this, the results showed that all 8 ducts in the girders, at some point, were missing grout. This was confirmed in 4 areas with drilling holes, and in one duct (3SC) we confirmed 3 wire breakages, **see photo 6.1.**

Even in the bridge deck we found some areas with missing injection. However, the missing grout in the bridge deck is expected just minor areas with partially filled ducts.

In table 6.1 below is a compilation that gives an overview of the grouting in the ducts and in the following chapter the results from each individual test area is presented.

							Bridge middle						
	Axis 6 Anchors			Axis 5						Axis 4			Axis 3 Anchors
North girder		4NA	N/A		100%	4NB	50%	4NB	100%		4NC	N/A	
		3NA	0%		100%	3NB	50%	3NB	50%		3NC	0%	
		2N	0%		0%	2N	50%	2N	0%		2N	0%	
		1NA	0%		100%	1NB	50%	1NB	0%		1NC	25%	
South girder		4SA	N/A		100%	4SB	50%	4SB	100%		4SC	N/A	
		3SA	100%		100%	3SB	25%	3SB	0%		3SC	100%	
		2S	25%		100%	2S	25%	2S	0%		2S	0%	
		1SA	0%		0%	1SB	50%	1SB	0%		1SC	0%	

1NC	Void confirmed with drilling		0%	grouted
3SC	Void confirmed with drilling	3 confirmed wire breakages	25/50/75 %	partially grouted
2S	Grouting confirmed with drilling		100%	missing grout
1SC	Grouting confirmed with drilling			
Duct 3SC has missing grout from the anchor at axis 3 and at least 6 meters towards axis 4				

Table 6.1 A summary of the investigation of the ducts

Photo no 6.1: Tendon duct 3SC with confirmed wire breakage

6.1 Axis 6, North girder

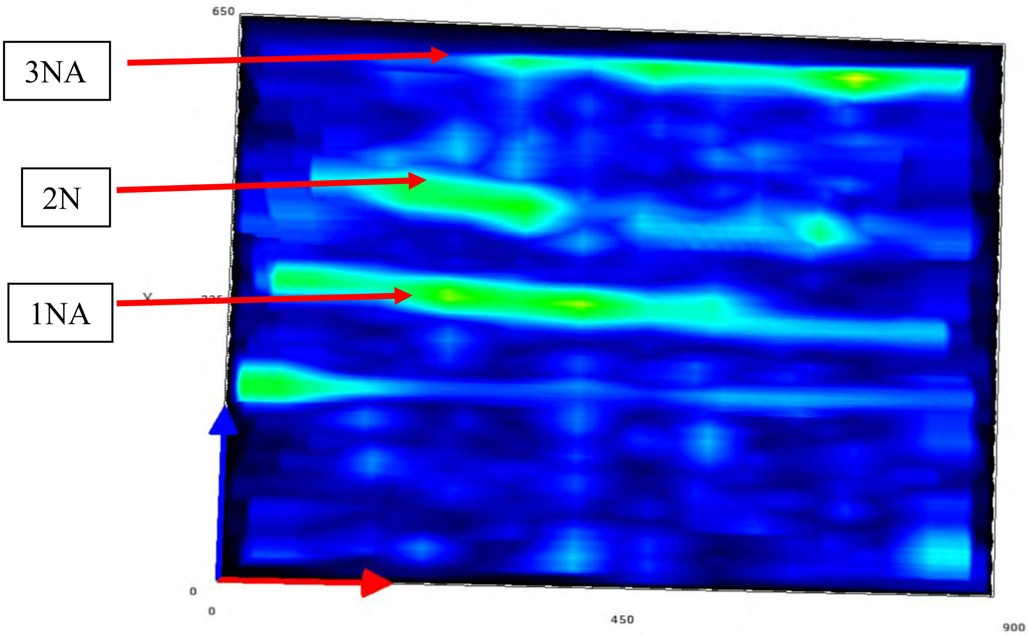


Figure 6.1.1: Results from ultrasonic

The test area was placed at the northern girder between 5-6 meters from axis 6, in this area only three tendon ducts were available for testing.
Tendon duct 4NA was situated in the bridge deck and no measurements could be done.
Tendon duct 1NA, 2 N and 3NA didn't show any signs that the grout was missing.

- 1NA: No voids in duct
- 2N: No voids in duct
- 3NA: No voids in duct

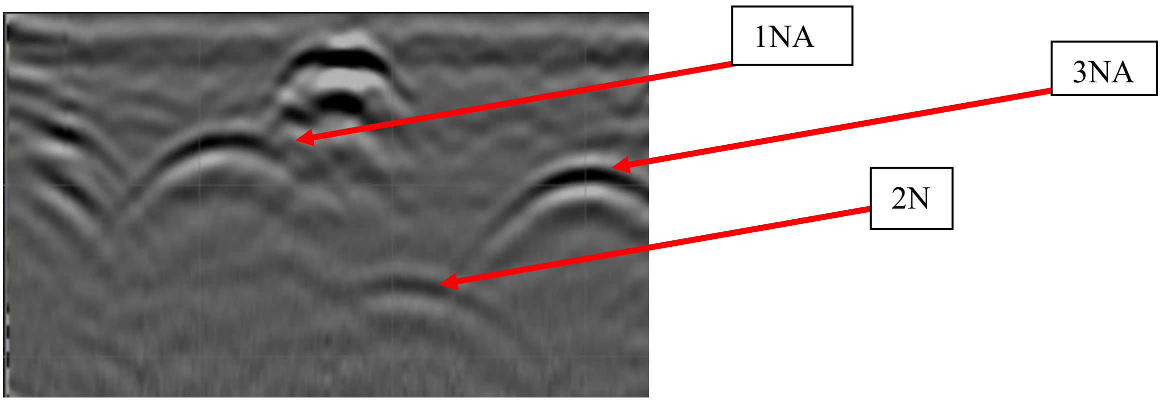


Figure 6.1.2: Results from GPR

6.2 Axis 6, South girder

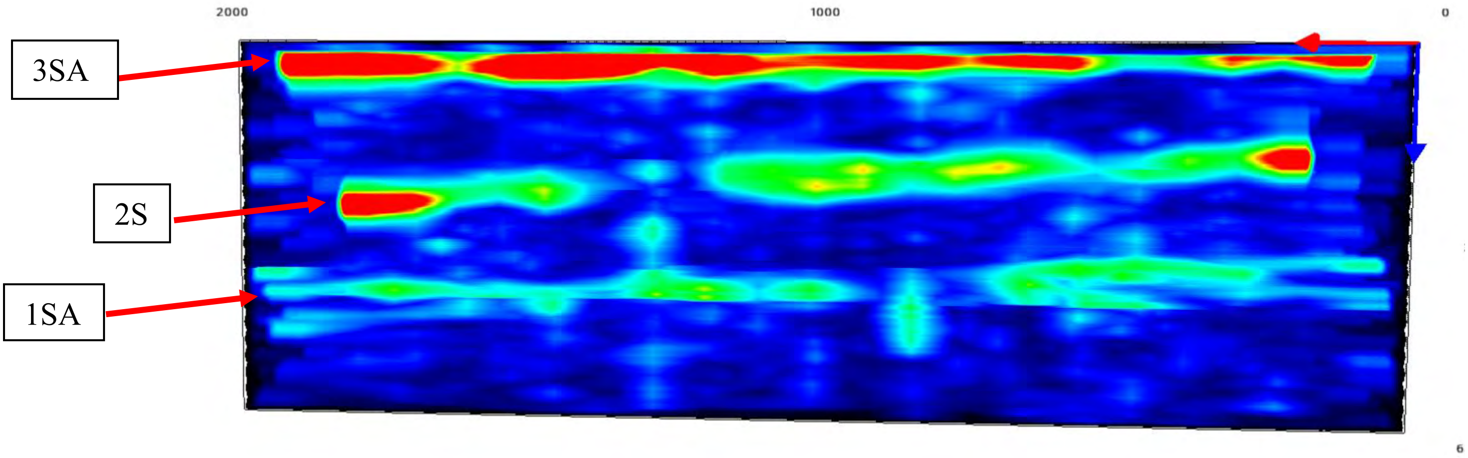


Figure 6.2.1: Results from ultrasonic

The test fields were placed at the southern girder between 5-7 meters from axis 6, in this area only three tendon ducts were available for testing.

Tendon duct 4SA is placed in the bridge deck where no measurements could be done.

Tendon duct 1NA didn't show any signs that the grout was missing.

There were some smaller voids in tendon duct 2S shown as red areas in the picture above. Voids could be larger outside of the measurement area. In tendon duct 3SA grout was missing over the whole length of the measurement area.

- 1SA: No voids in duct
- 2S: Small voids in duct
- 3SA: Missing grout, 2 meters of length.

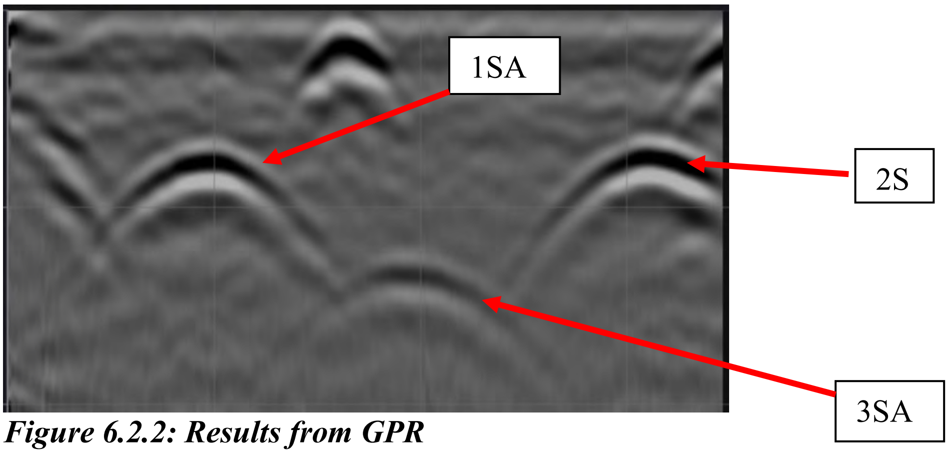


Figure 6.2.2: Results from GPR

6.3 Axis 3, North girder

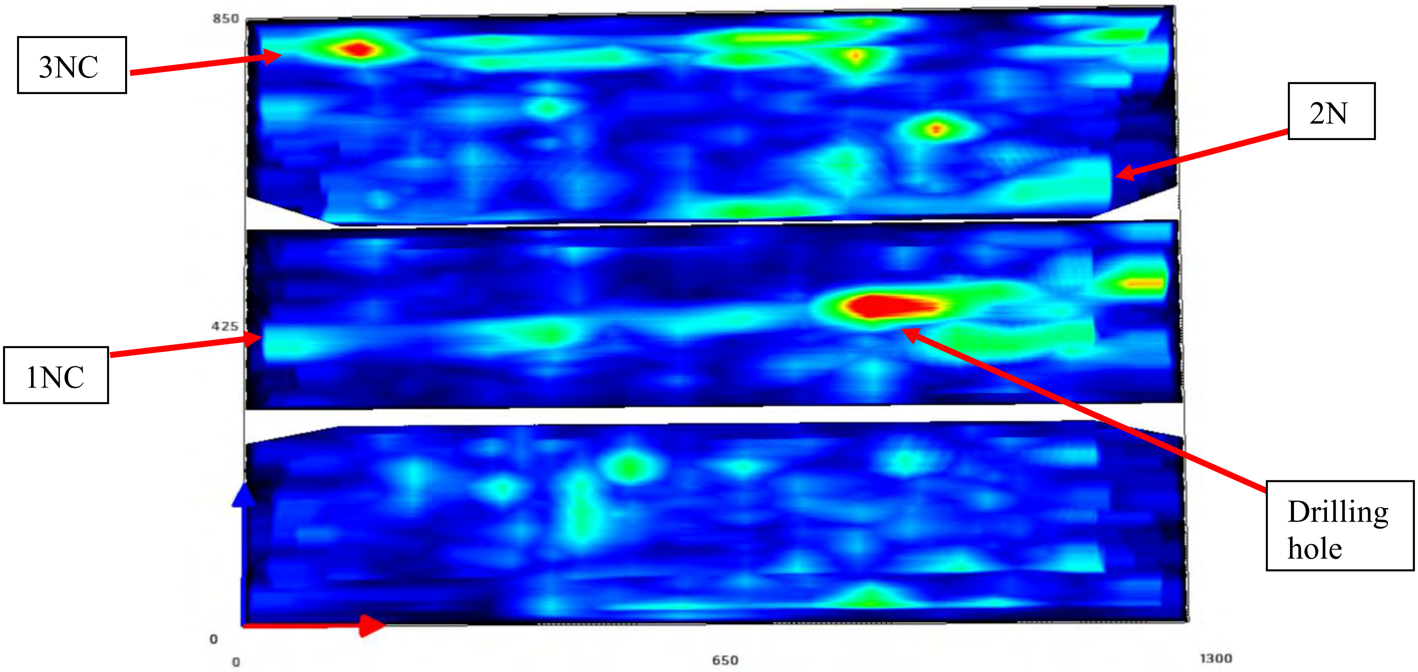


Figure 6.3.1: Results from ultrasonic

The test fields were placed at the northern girder between 5-6,5 meters from axis 3, in this area only three tendon ducts were available for testing.

Tendon duct 4NC was placed in the bridge deck and no measurements could be done. Tendon duct 1NC had a local area with a length of about 200 mm where we got suspicious measurements. In order to learn how to interpret the suspicious result, we made a drill hole to do a visual check in the duct. The drilling hole confirmed our suspicions as a local area with missing grout in the duct was found, **see photo no 6.3.**

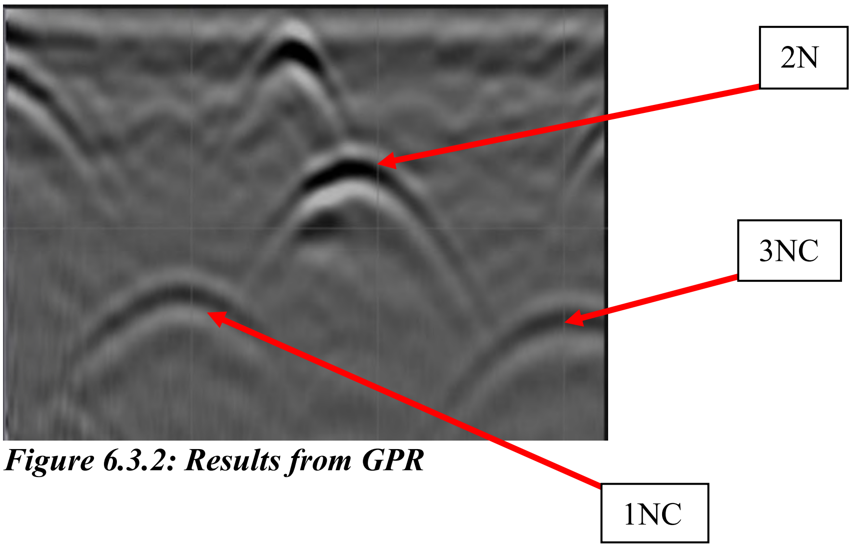


Figure 6.3.2: Results from GPR

Photo no 6.3: Missing grout and corrosion in tendon duct 1NC.

Tendon duct 2N and 3NC didn't show any signs that the grout was missing.

- 1NC: Locally missing grout, about 200 mm of length
- 2N: No voids in duct
- 3NC: No voids in duct

6.4 Axis 3, South girder

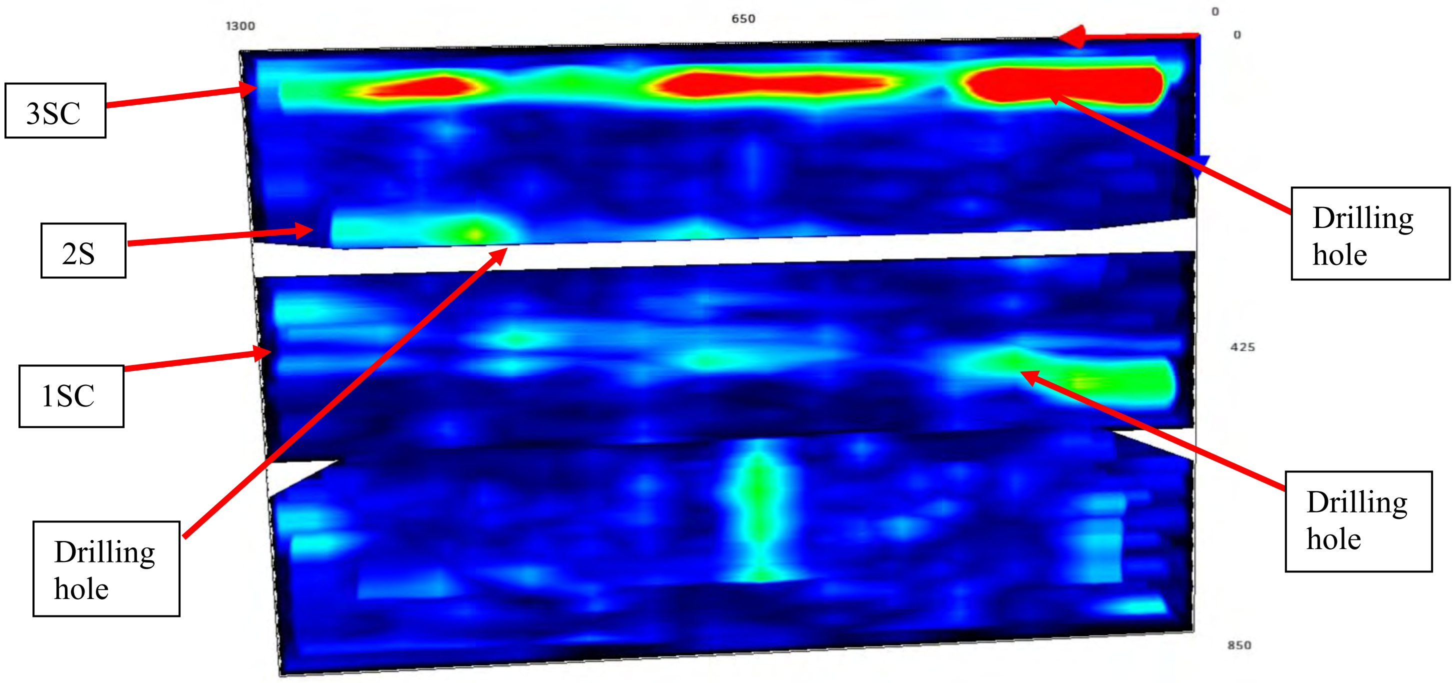


Figure 6.4.1: Results from ultrasonic

The test field were placed at the southern girder between 5-6,5 meters from axis 3, in this area only three tendon ducts were available for testing.
Tendon duct 4SC was placed in the bridge deck and no measurements could be done.

Tendon duct 1SC and 2S didn't show any signs that the grout was missing. This was confirmed with drilling and a visual check in both ducts, **see photo no 6.4.1 & 6.4.2.**

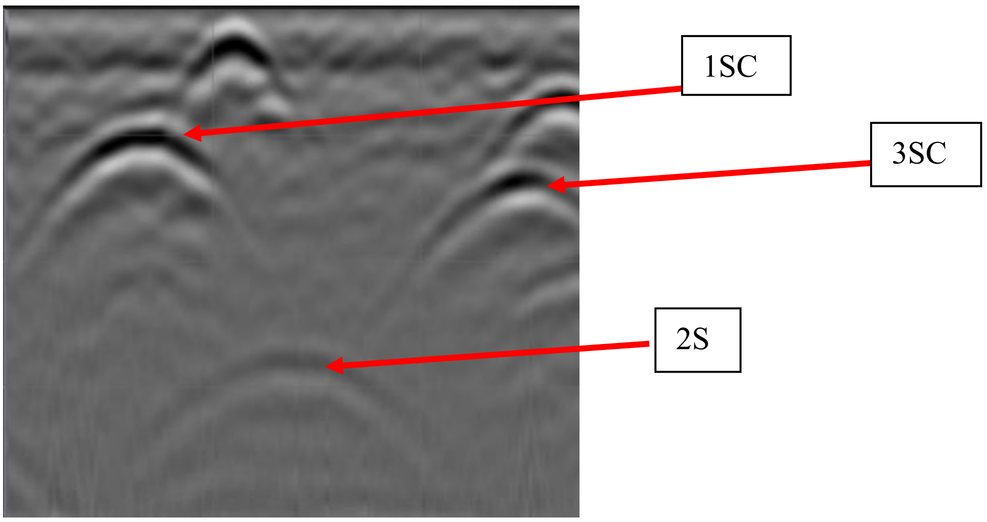


Figure 6.4.2: Results from GPR

Photo no 6.4.1: Tendon duct 1SC with confirmed grouting

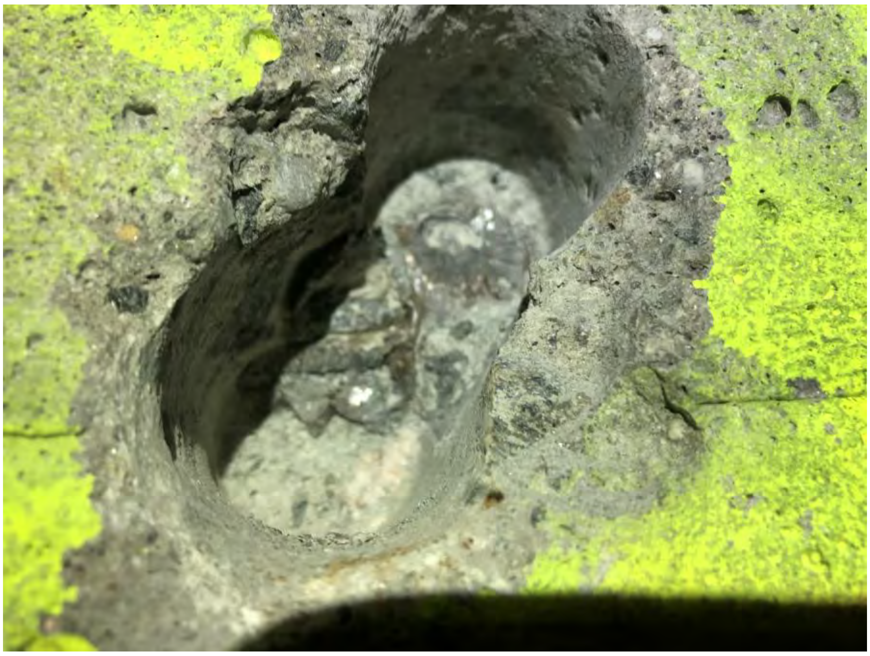


Photo no 6.4.2: Tendon duct 2S with confirmed grouting

Cable duct 3SC shows missing grouting over the whole length. To learn more about the grouting process we made a full scan at duct 3SC from the anchor and 6 meters towards axis 4. The results showed that the duct was missing grout over the whole length, **see figure no 6.4.3**. In this duct, we made a drilling hole to confirm the missing grout and we also confirmed three wire breakages in this area, **see photo no 6.4.3**.

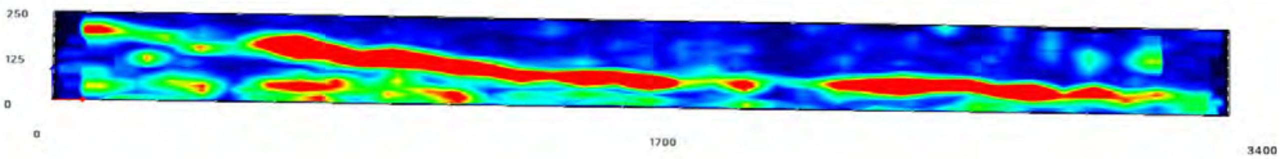


Figure no 6.4.3: Tendon duct 3SC missing grout in 6 meters, (anchors to the right)

Photo no 6.4.3: Tendon duct 3SC with confirmed void and wire breakage

- 1SC: No voids in duct
- 2S: No voids in duct
- 3SC: Missing grout 6 meters of length, 3 confirmed wire breakages.

6.5 Middle span; North girder

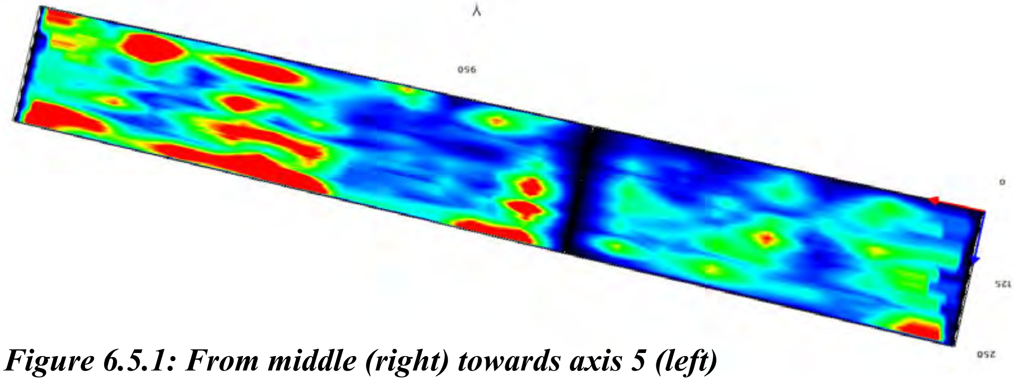


Figure 6.5.1: From middle (right) towards axis 5 (left)

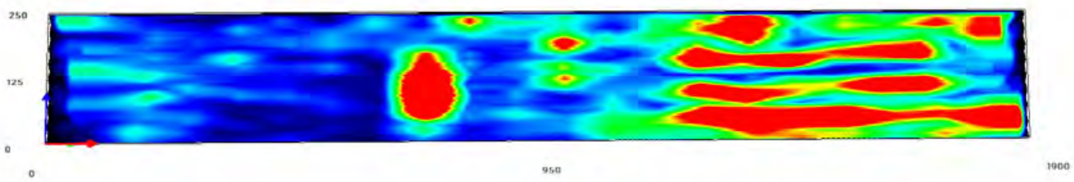


Figure 6.5.2 From middle (left) towards axis 4 (right)

In the middle of the bridge all cables were placed in the bottom of the girder. The measurements on the northern girder were made from the middle of the bridge and 2 meters to both sides.

The results showed us that the 2 meters in the middle of the bridge didn't contain any voids. The interesting part is that we found voids and missing grout in all ducts about 1 meter from the middle in both directions.

6.6 Middle span; South girder

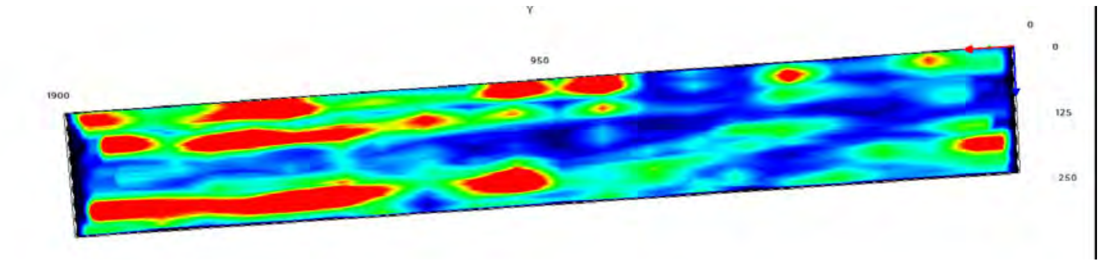


Figure 6.6.1: From middle (right) towards axis 5 (left)

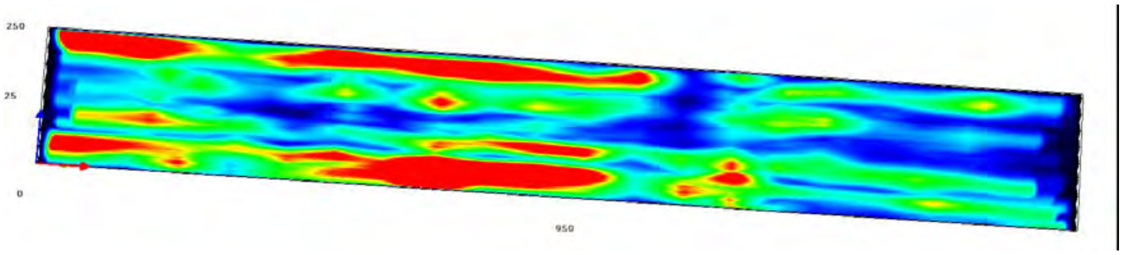


Figure 6.6.1: From middle (left) towards axis 5 (right)

In the middle of the bridge all cables were placed in the bottom of the girder. The measurements on the southern girder were made from the middle of the bridge and 2 meters in each side.

In this girder, the grouting was a little bit better than the northern. Despite that three of four cables had voids and missing grout in almost 2 out of 4 meters.

6.7 Middle span south girder towards axis 5

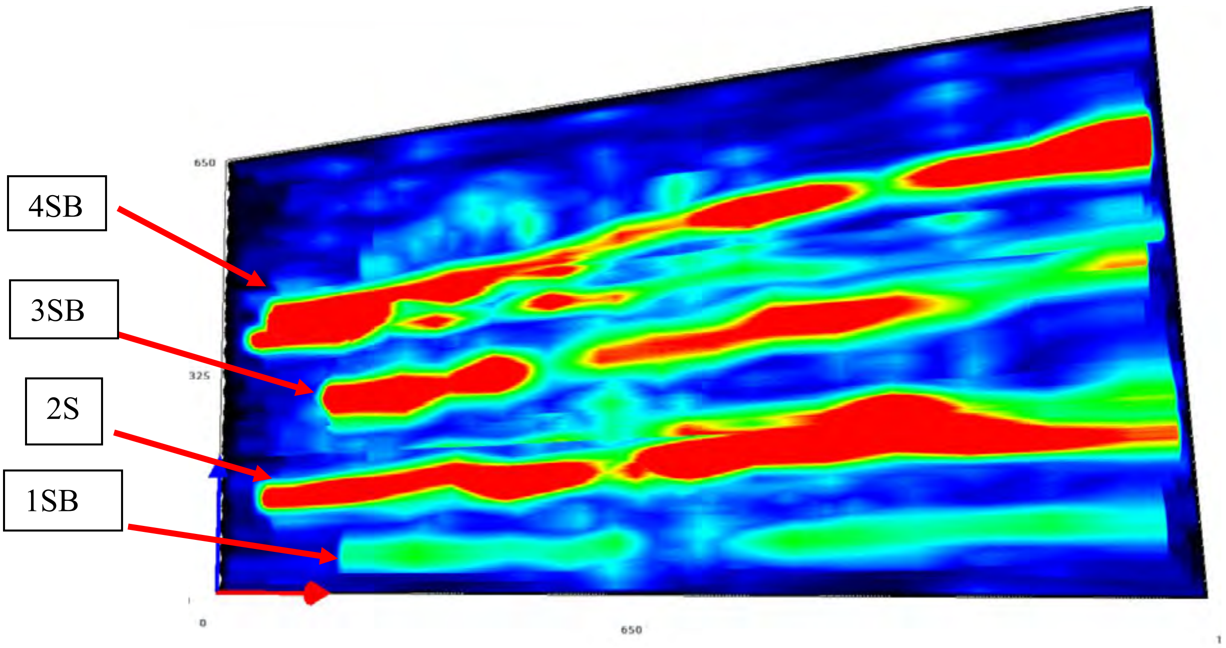


Figure 6.7: Results from ultrasonic

The test area was in the south girder, 6-7,5 meters towards axis 5 from the middle of the bridge. In this area, we could locate all four tendon ducts.

Tendon duct 1SB didn’t show any suspicious measurements.

Cable 2S, 3SA and 4SA was missing grout over the whole length of 1,5 meters.

- 1SB: No voids in duct
- 2S: Missing grout, 1,5 meters
- 3SB: Missing grout, 1,5 meters
- 4SB: Missing grout, 1,5 meters

6.8 Middle span south girder towards axis 4

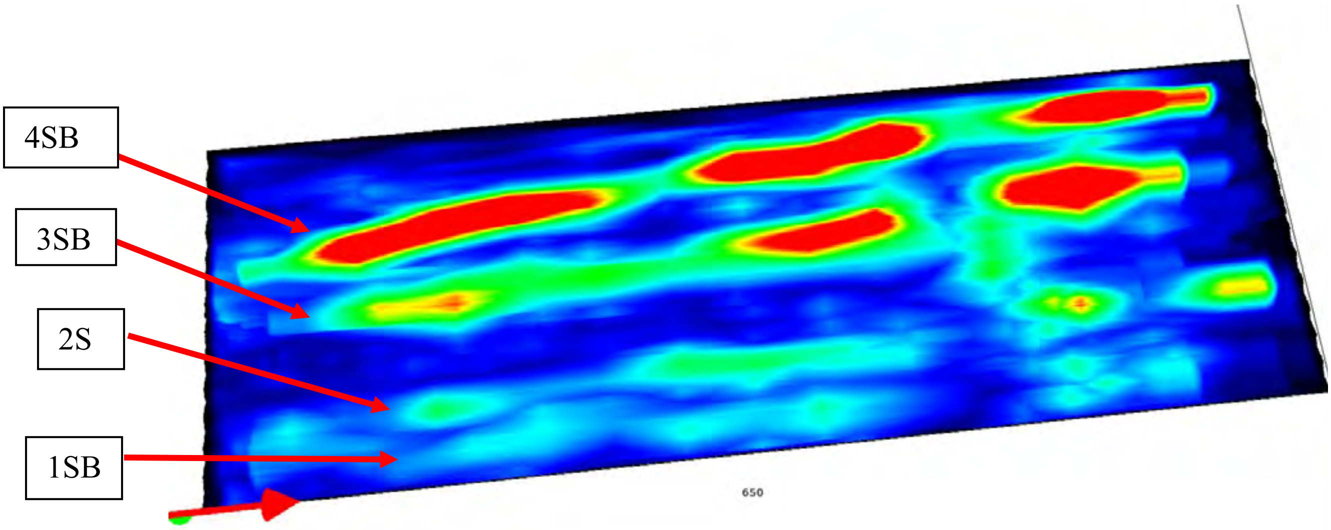


Figure 6.8: Results from ultrasonic

The test area was in the south girder, 6-7,5 meters towards axis 4 from the middle of the bridge. In this area, we could locate all four tendon ducts.

Tendon duct 1SB and 2S didn't show any suspicious measurements.

Tendon duct 4SB was missing grout over the whole length of 1,5 meters, and tendon duct 3SB had some local voids. However, it is possible that even 3SB is partially missing grout over its whole length of 1,5 m.

- 1SB: No voids in duct
- 2S: No voids in duct
- 3SB: Missing grout/partially grouted, 1,5 meters
- 4SB: Missing grout, 1,5 meters

6.9 Middle span north girder towards axis 5

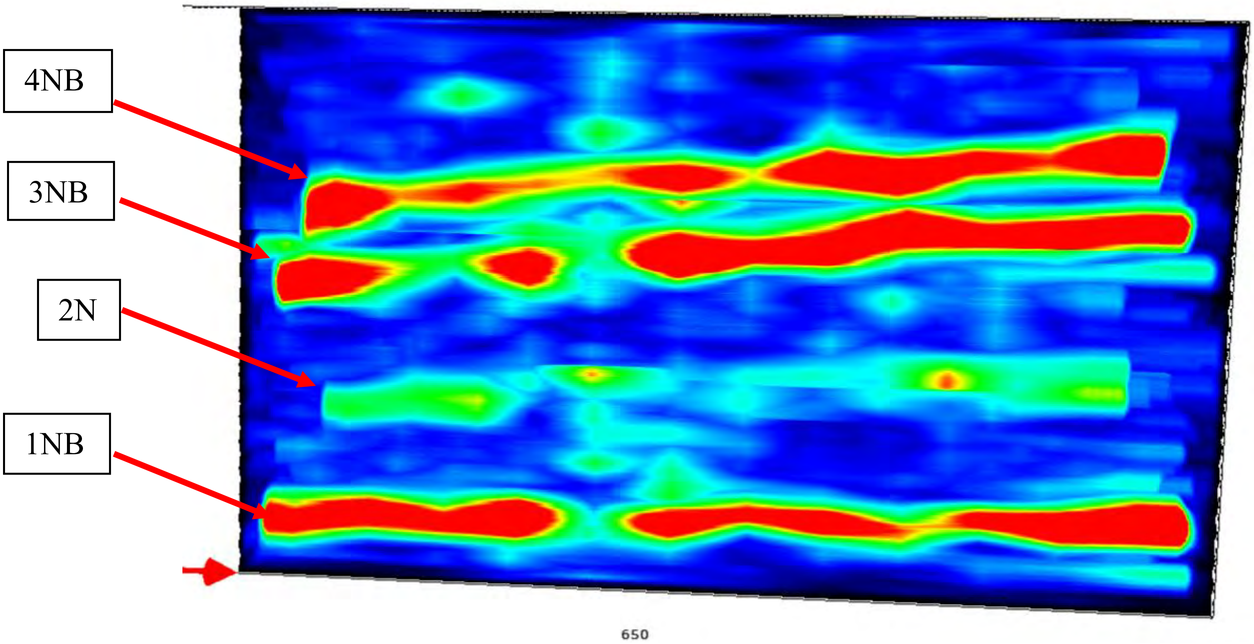


Figure 6.9: Results from ultrasonic

The test area was in the north girder, 6-7,5 meters towards axis 5 from the middle of the bridge. In this area, we could locate all four tendon ducts. Tendon duct 2N didn't show any suspicious measurements.

Tendon duct 1NB, 3NB and 4NB were all missing grout over the whole length of 1,5 meters

- 1NB: Missing grout, 1,5 meters
- 2N: No voids in duct
- 3NB: Missing grout, 1,5 meters
- 4NB: Missing grout, 1,5 meters

6.10 Middle span north girder towards axis 4

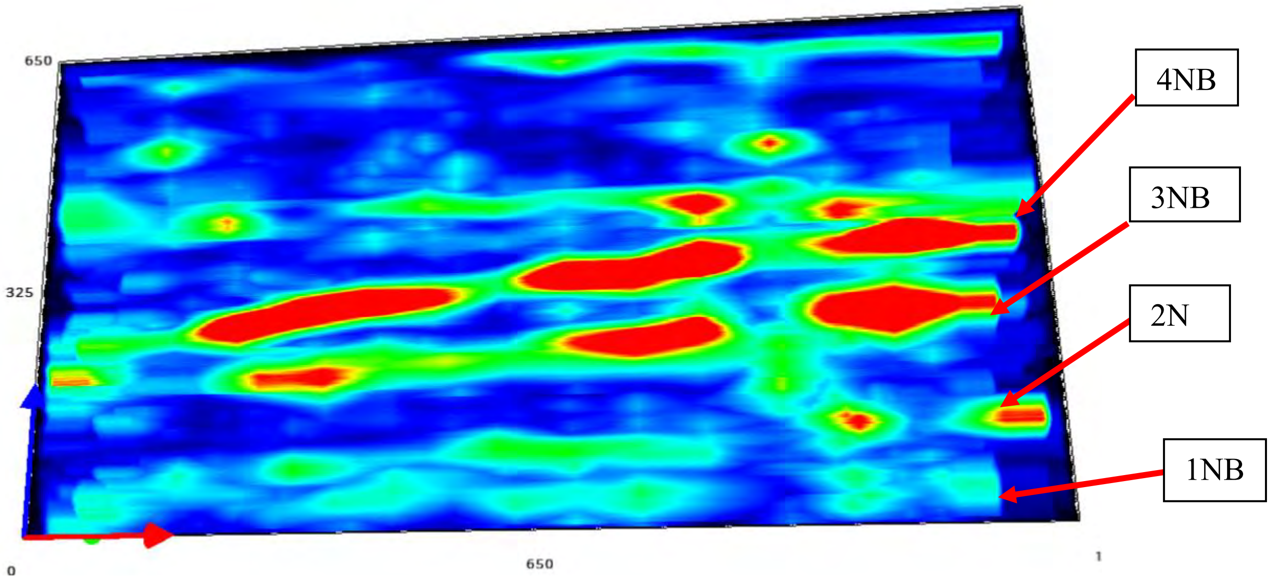


Figure 6.10: Results from ultrasonic

The test area was in the north girder, 6-7,5 meters towards axis 4 from the middle of the bridge. In this area, we could locate all four tendon ducts.

Tendon duct 1NB and 2N didn't show any suspicious measurements.

Tendon duct 4NB was missing grout over the whole length of 1,5 meters, and 3NB was missing grout or only partially grouted.

- 1NB: No voids in duct
- 2N: No voids in duct
- 3NB: Missing grout/partially grouted, 1,5 meters
- 4NB: Missing grout, 1,5 meters

6.11 Bridge deck Axis 4

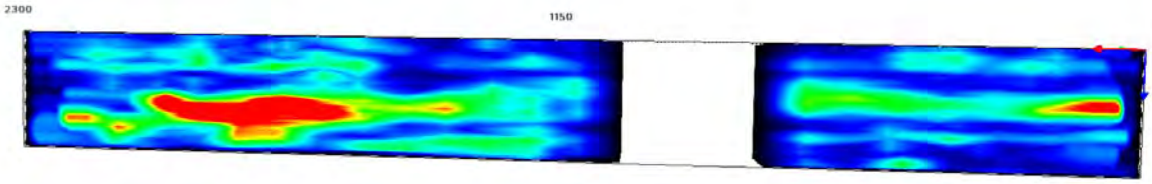


Figure 6.11.1: Head span north tendon duct 7Ö17

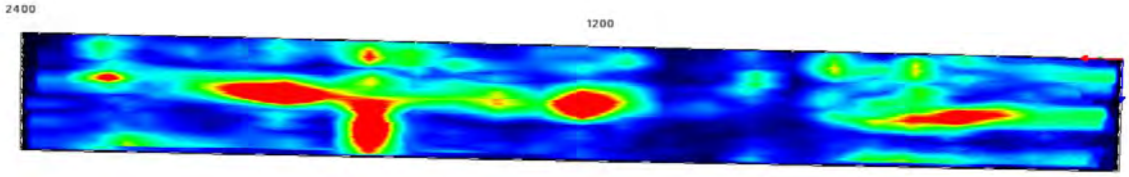


Figure 6.11.2: Side span north tendon duct 7Ö17

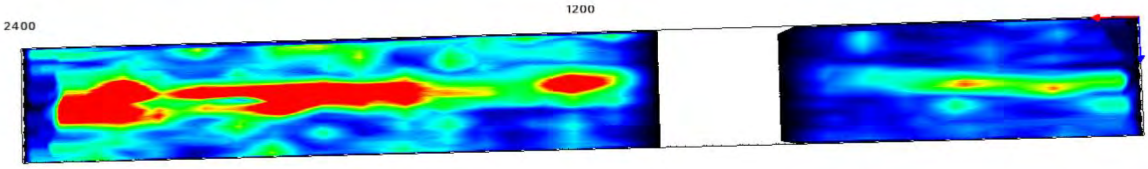


Figure 6.11.3: Head span south tendon duct 12Ö22

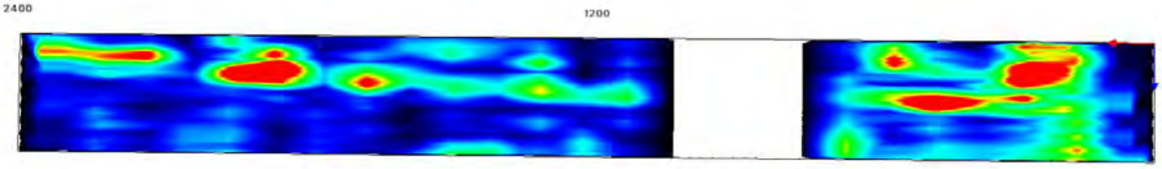


Figure 6.11.4: Side span south tendon duct 12Ö22

In the bridge deck at axis 4, the measurements were executed 10 m from the axis and a total length of 2,5 were scanned.
All four tendon ducts in the bridge deck at axis 4 had missing grout or were only partially filled. The grouting rate is estimated to be around 50% in total. However, it is possible that grout is missing over the whole length.

6.12 Bridge deck Axis 5

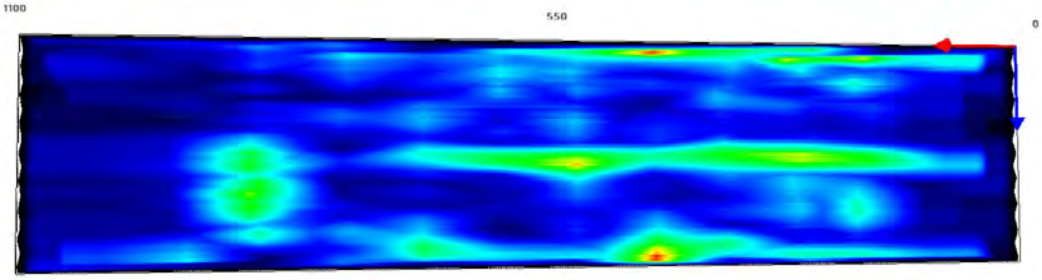


Figure 6.12.1: Head span north tendon duct 12Ö22

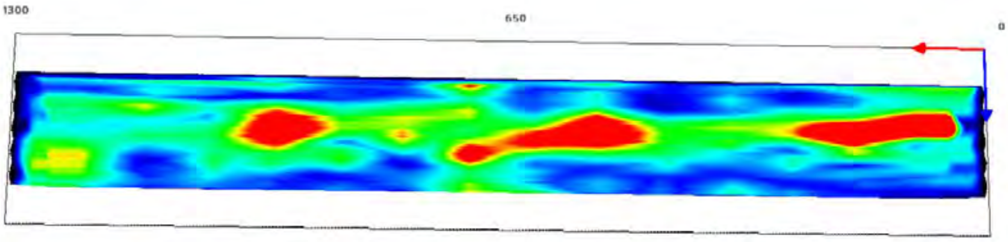


Figure 6.12.2: Side span north tendon duct 12Ö22

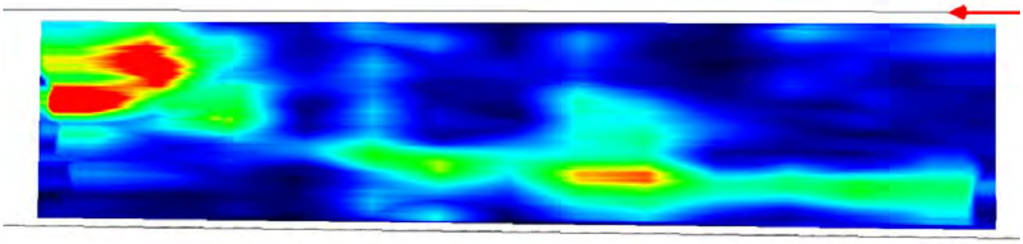


Figure 6.12.3: Head span south tendon duct 7Ö17

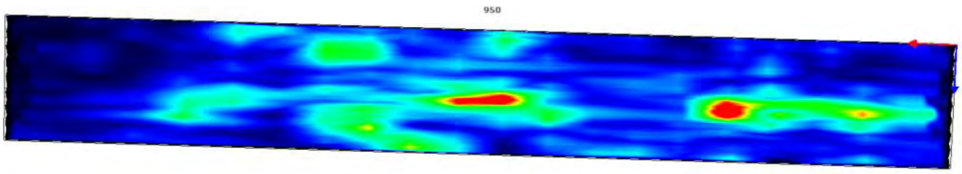


Figure 6.12.4: Side span south tendon duct 7Ö17

In the bridge deck at axis 5, the measurements were executed 10 m from the axis and a total length of 1,5 were scanned.

Head span north, head span south and side span south didn't show any signs of missing grout in the ducts. The tendon duct in side span north had missing grout in the whole length of 1,5 meters.

7 COMMENTS AND CONCLUSIONS

It is clear that a project of this size needs a detailed planning for success. It is not only the access to the structure that is important but also other practical details such as lightning and electricity. In addition to this it is of utmost importance that documents are studied before arriving to the site and that areas to be investigated are decided beforehand. In this project we used a week, 3 people, for the investigation, and still we were only able to cover a small part of the bridge in detail. Following our process in figure 2.1 we first investigated and decided the placement of the ducts and tendons by GPR. We also checked the distance and placement to the ordinary steel reinforcement with cover meter. In the critical areas, we then proceeded with the ultrasonic-pulse-echo. Then for detailed investigation we used the impact echo. In four areas, we open up the duct to verify our findings.

The inspection showed that there was a lot of missing grout in the tendon ducts. In general, the largest problems were located at the head span towards axis 5.
In this area 6 out of 8 tendon ducts had missing grout in all the inspected areas. Furthermore, some local cable ducts (for example: cable 3, south girder) had missing grouting near the anchors both in axis 6 and axis 3.

The cables in the middle of the bridge are not missing grout in 100% of the length, but they have about 1 m of continuous voids, and should therefore be considered as a high-risk area. The amount of grouting in the ducts is difficult to determine, but taking the visual inspection and Impact echo confirmation into consideration the results are expected to be reliable and accurate.

It should also be noted that cable 3SC had at least 3 wire breakages, where the wires were moving when they were poked at with a screwdriver. The same tendon had missing grout at a length of 6 meters from the anchor in axis 3. Because of the large amount of missing grout, it is hard to find the location of the wire breakages.

Worth mentioning is also that there were several load cracks in the girders in the middle of the bridge. The cracks were located on the underside of the beams, which indicates a loss of load capacity.

Also, taking into consideration that the tendons were placed at the underside of the girders and that they had missing grout, it is not impossible that the cracks has affected the ducts and the tendons has an ongoing corrosion.

8 SUGGESTION TO FURTHER STUDIE

From the investigation carried out on the Herøysunds bridge the following is suggested for further actions:

- The existing cracks and the condition of the Herøysunds bridge should be followed up by monitoring. Many of the cracks that were seen in the mid-span had developed after repair. This means that they are active and considering the risk of having tendon breakage in this section there might be a risk of severe damage.

- The results from the NDT testing were very positive and we sole like to suggest that the procedure that has been used on this bridge is also implemented on other bridges in Norway to further build up the knowledge in this area and to gain experience. Preferable this should be combined with repair methods for improving corrosion resistance in existing ducts.

Appendix G – Field test railroad bridge, Abisko

INSPECTION REPORT

Handläggare/Please refer to
Andreas Karlsson
Tel. +46 10 455 12 07
Mob. +46 70 211 25 74
Name.lastname@dekra.com

Action	Railway bridge, Abisko
Date och time	2020-04-20 & 23
Location	Abisko, Sweden
Present	Mats Holmqvist (DEKRA Industrial AB) Andreas Karlsson (DEKRA Industrial AB)

1 Background

DEKRA Industrial AB was, as a part of the project "Study if methods, possibilities and limitations for inspection of post stressed reinforcement in concrete bridges", commissioned to carry out an NDT inspection of a railway bridge in Abisko, Sweden. The purpose of the inspection was to try out the limitations in the NDT equipment.

1.1 Scope

The inspection was performed with different NDT methods listed below

- (CM) Covermeter (Proseq Profoscope+)
- (GPR) Ground Penetrating Radar (Hilti PS 1000 X-Scan & Proceq GP 8000)
- (UPE) Ultrasonic pulse-echo (MIRA)

2 Object

Bridgetype: Box Girder
Building year: 1978

2.1 Drawings

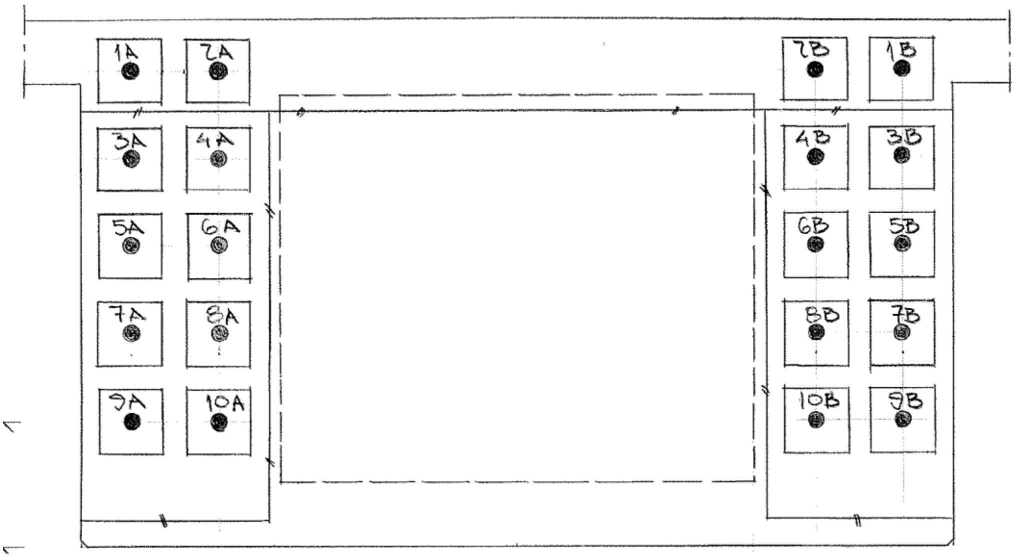


Fig 2.1: Placement of tendons

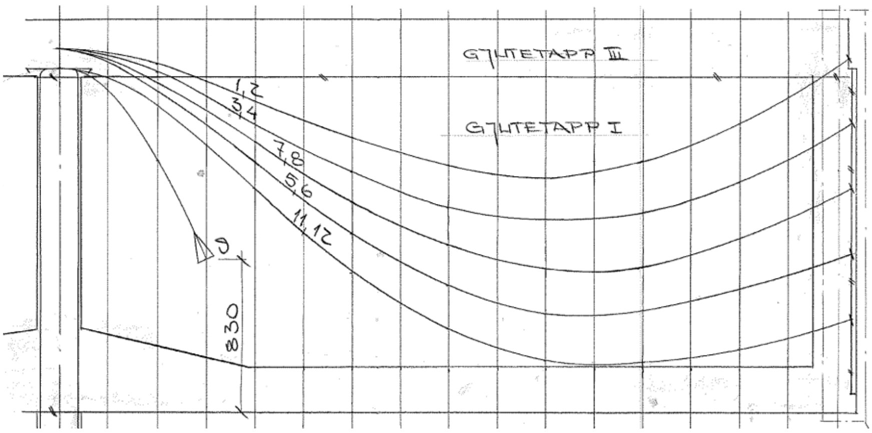


Fig 2.2: Placement of tendons

3 Results

3.1 GPR

To compare different instruments, a GPR from Hilti and Proceq was used, the results of the different scans are presented below.

The first scan was carried out on the northern girder two meters from the eastern support, where all ten ducts are placed in the girder. The bridge was scanned from the inside and there were five cables located at a depth of about 150 mm. According to the drawing, the other five cables were located on the outside of the beam with a covering concrete layer of about 400mm from the inside.

The measurement with Proceq showed that the five closest clamping cables could be found, they were at a depth of about 150 mm except to the top one which was at a depth of about 120 mm. The second layer of ducts could not be located because they were placed too deep into the concrete, ***see figure 3.1***

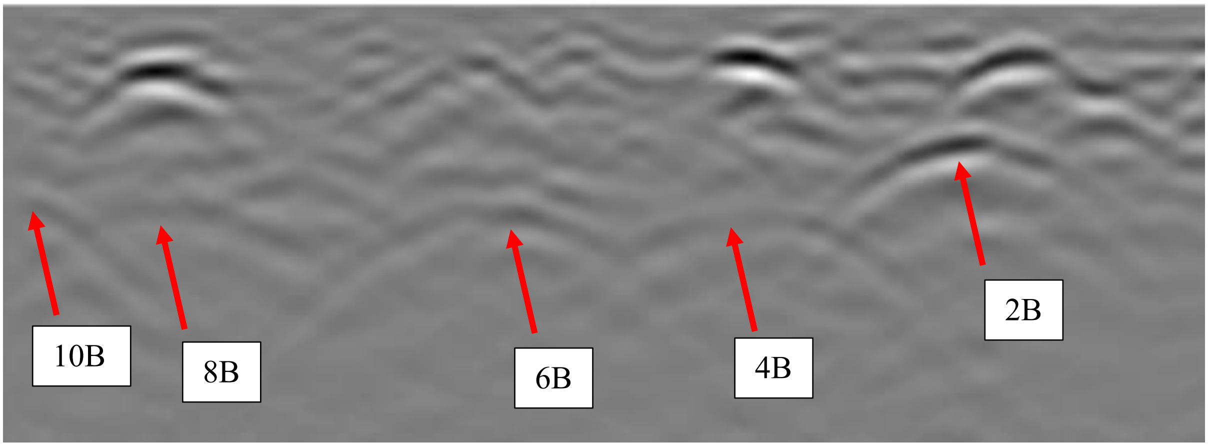


Fig 3.1: Results from Proceq GPR with the post tension cables marked with arrows

At the same place, a scan was done with the Hilti equipment, also here it was possible to locate the five ducts that were closest, but not the outer ones, see ***figure 3.2.***

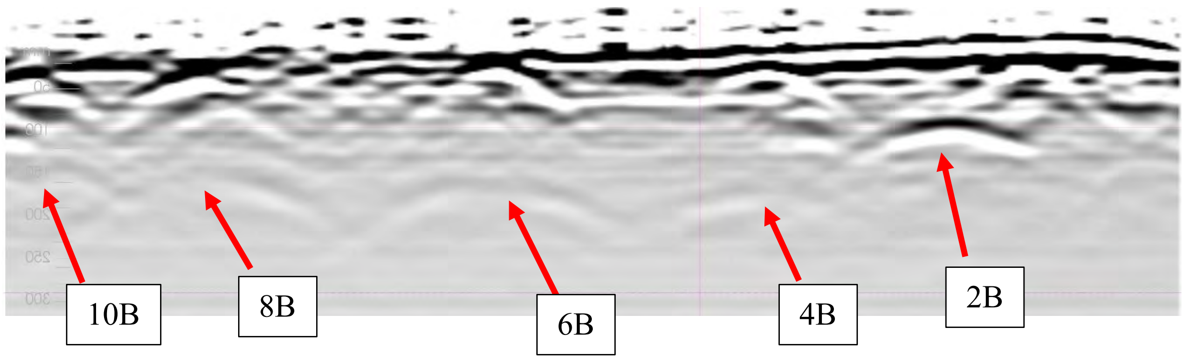


Fig 3.2: Results from Hilti GPR with the post tension cables marked with arrows

Additional scans were made 17.5 meters in from the eastern support, here the tension cables were placed in the lower part of the girder. The results from the different equipment's were equivalent.
Three ducts were located in the lower part, but it was still difficult to find the second layer of ducts, ***see figure 3.3 & 3.4.***

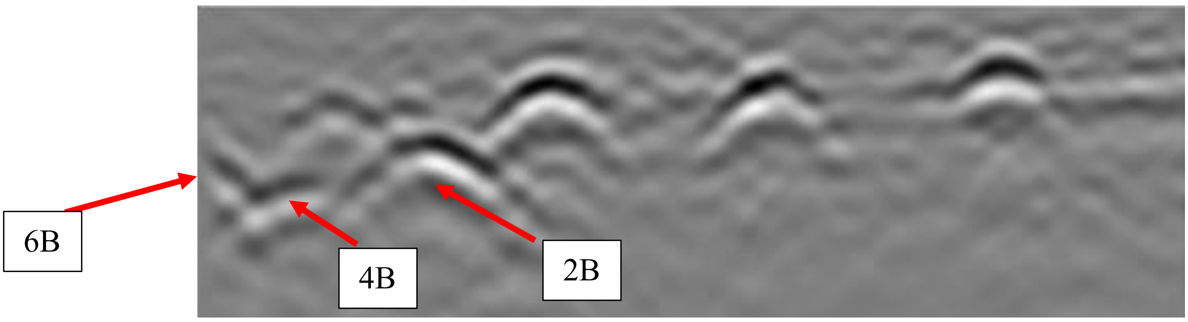


Fig 3.3: Results from Proceq GPR with the post tension cables marked with arrows

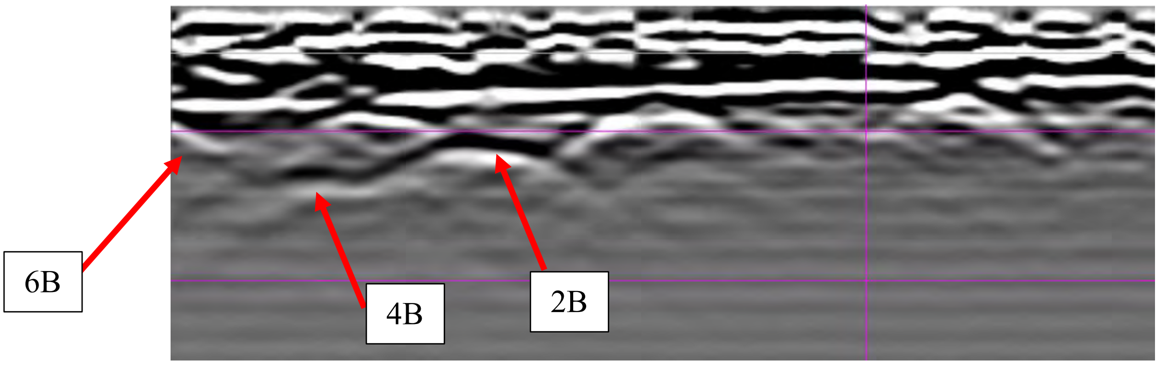


Fig 3.4: Results from Hilti GPR with the post tension cables marked with arrows

3.2 Ultrasonic Pulse Echo

In the same way, as with GPR, a scan was made with UPE on the northern girder 2 meters in from the eastern support. All five cables were located and after processing in the computer a clear image could be seen. Noteworthy is that cable 6B looked suspicious with a larger red field, ***see figure 3.4***.
As a result, a longer distance of a total of 3 meters was scanned starting at the anchor.
This scan showed that there was a suspected void in the duct that was about 1 meter long, ***see figure 3.5***.
However, at the time of inspection this could not be confirmed by any other test.

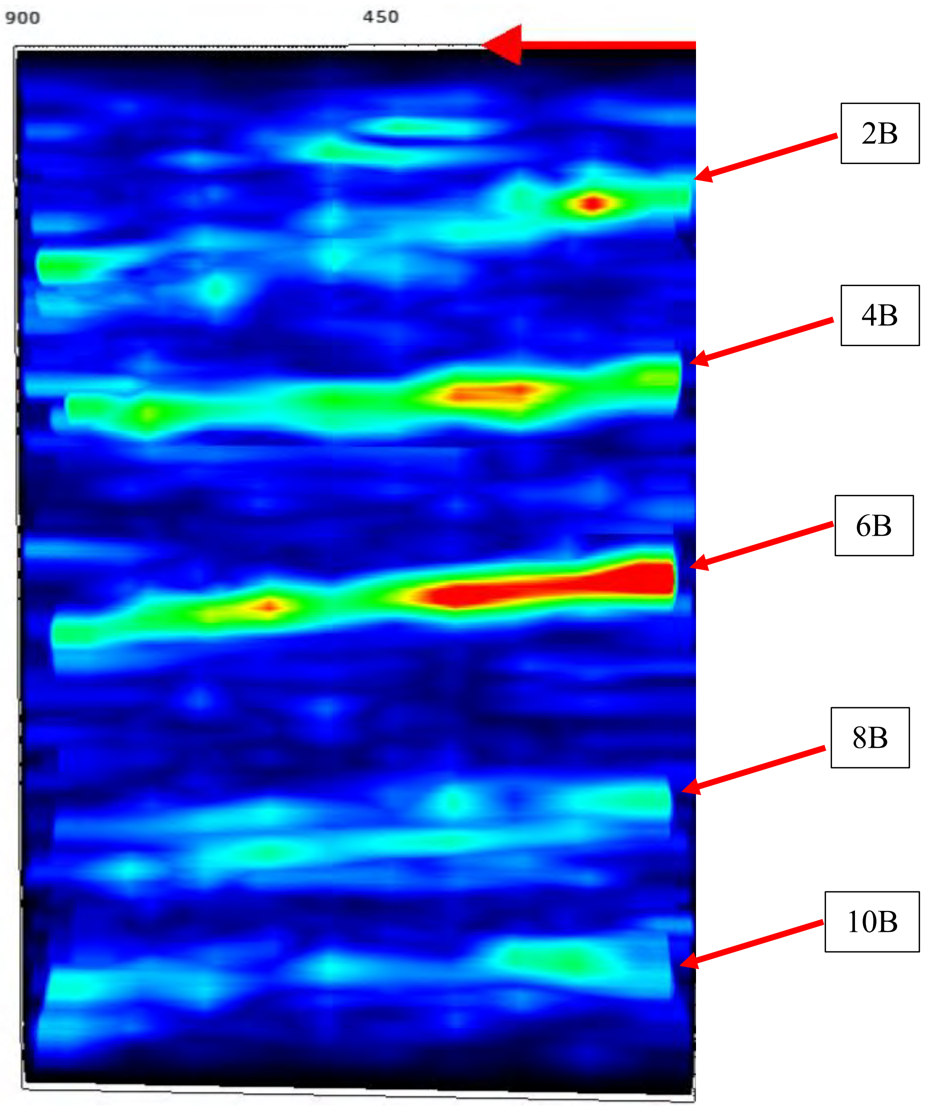


Fig 3.4 Cable ducts north girders, 2 meters in.

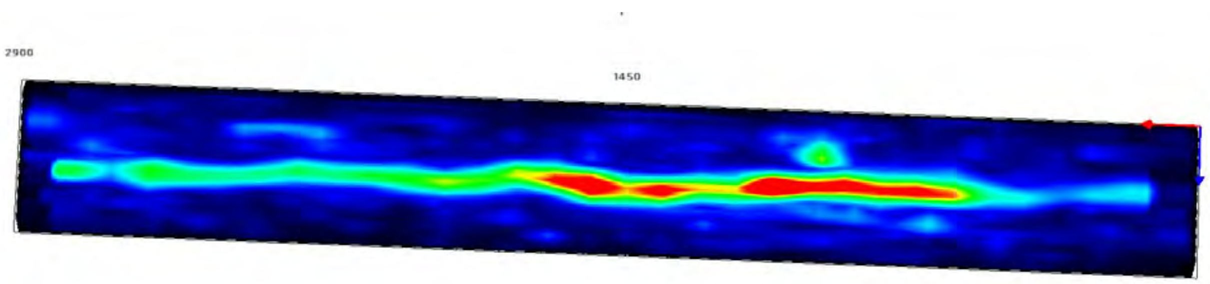


Fig 3.5 Cable duct 6B, from anchor to the right and 3 meters towards west support.

The next scan that was done was at a low point 17.5 m in, here we found all three ducts that were located at the bottom of the girder.
There were no signs of any voids in this area and the cables looked well-grouted, ***see figure 3.6.***

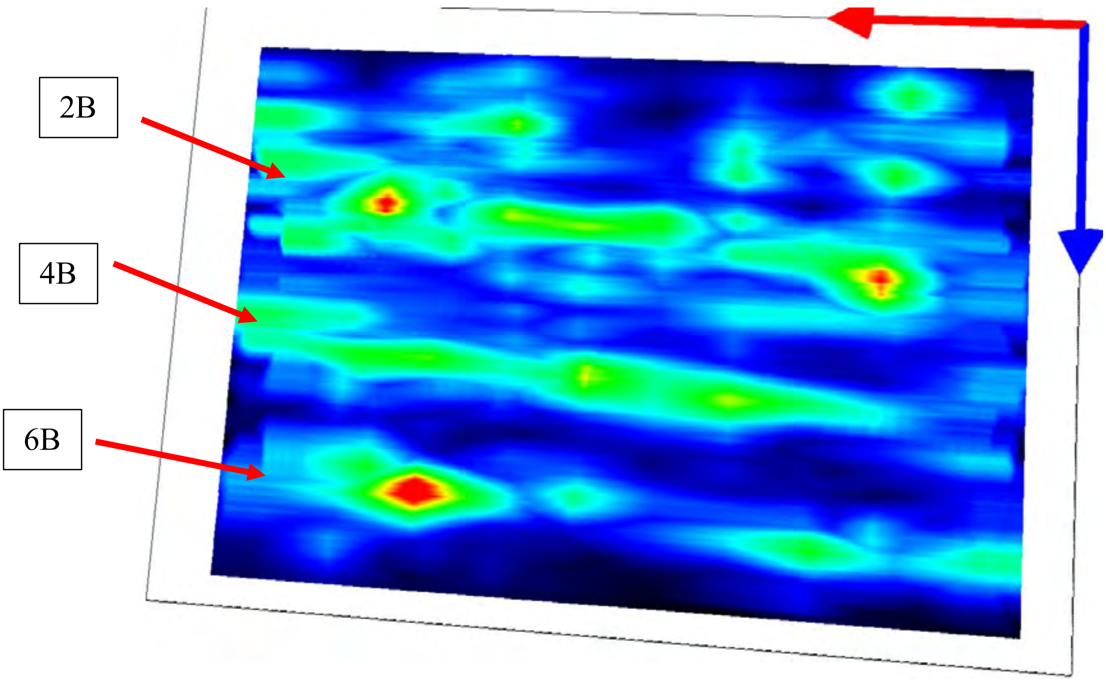
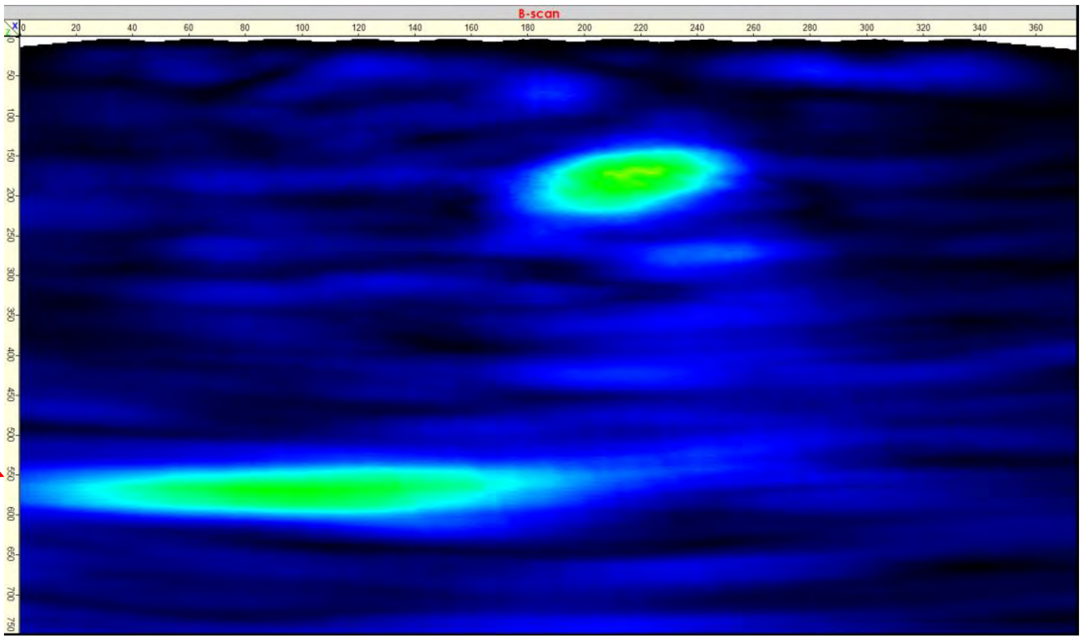


Fig 3.6 Cable ducts north girders, 17,5 meters in.

4 Comments

The results of the inspection were a great help in further learning the equipment and its limitations.
Neither with GPR nor UPE could we locate the second layer of tension reinforcement.
There were some differences in the two GPR equipment used, both of which were able to locate the ducts.
From a visual point of view, the images from Proceq are better and clearer, it is also smaller which makes it easier to perform scans near corners.
Hiltin requires a lot more experience and image processing to read the results. But, as can be seen in the result above, we got equivalent results from both equipment.
The reason that the second layer of tension reinforcement could not be located was that GPR simply is not powerful enough at this depth. In general, the limit in depth is about 250 mm.

The UPE is considerably more powerful and can withstand a depth of up to about 1 meter. The reason why the second layer of tendon ducts could not be found was because they were shaded by the first layer. With the UPE we were able to measure the thickness to 550 mm which corresponds to the drawings. Thus, it is not the depth of the clamping cables that is the limit but the location of them, **see figure 4.1.**



Backwall
550 mm

Fig 4.1 Backwall and cable duct 6B

The figures below show the difference between a scan with GPR and a scan with UPE. Both scans are made along cable 6B at a distance of 3 meters.
With GPR we can clearly see the transverse reinforcement and below it we see the longitudinal cable duct, ***see figure 4.2.***
With the UPE we do not see the transverse reinforcement, but we get a very clear picture of the duct, ***see figure 4.3.***
The reason why the transverse reinforcement shows up on GPR but not on ultrasound is because they measure two different directions.

The equipment is dependent on you as a user having knowledge of what it is you are looking for in the concrete and adapting your scans to what it is you are going to locate.
For example, if you want to locate horizontal reinforcement with GPR, the scanning direction must be vertical.
With the UPE it is different and slightly more complex, since the UPE does not have wheels, you can freely choose which scan direction you want. However, the basic rule of UPE is that the scanning direction should be the same as the object you want to locate.
If you have a horizontal duct then the scanning direction must also be horizontal. Therefore, GPR is such an important part of the inspection, without first locating the cable ducts, it is a time-consuming job to be able to scan them with the UPE.

The exception to this is shown in the figures below. With GPR we get a picture of a tendon duct that is positioned along the scanning direction. It is therefore possible to scan along individual tendon ducts and locate them.
In this way, however, you lose the advantage of GPR where you quickly and easily locate and mark the reinforcement.
The type of scans seen in ***figure 3.4*** made with ultrasound are six separate scans where each is 1 meter long. These have then been processed in the computer and integrated into each other to obtain a coherent image.

The two ***figures 4.2 and 4.3*** show a small pattern between the signals from the UPE and GPR.
The UPE shows that there was a suspected void of about 1 meter in the duct and the same signals we can see on GPR.
Whether this is a coincidence or whether it really is an echo from the void is difficult to determine and further testing is required to ensure this

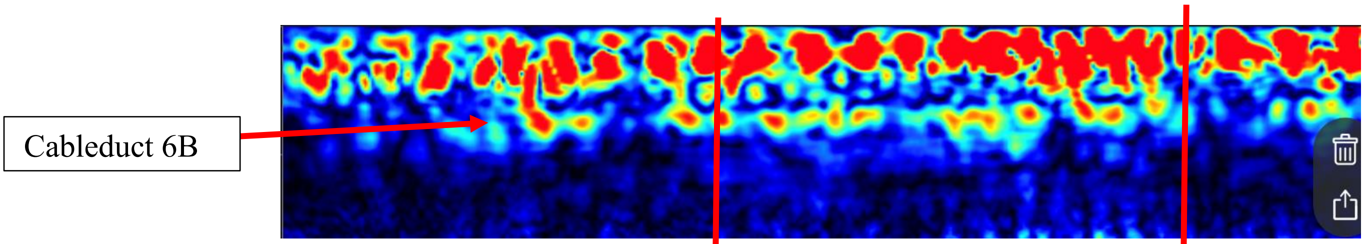


Fig 4.2 Cable ducts 6B with GPR

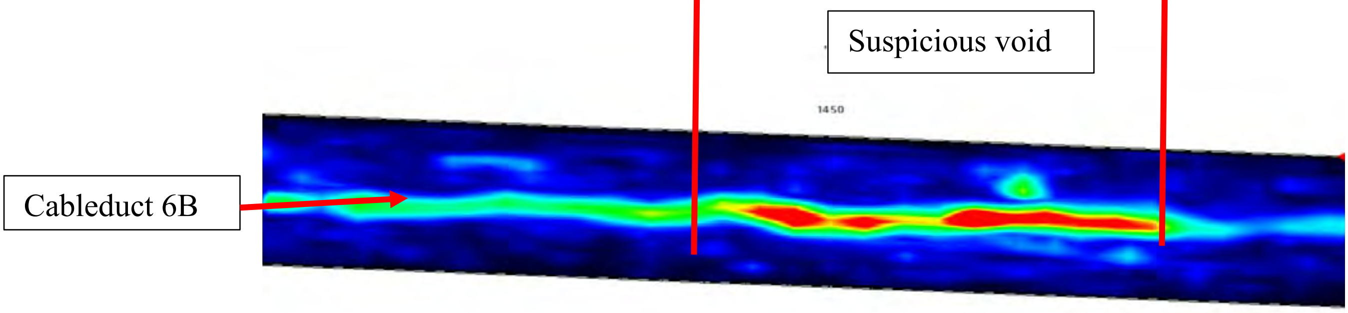


Fig 4.2 Cable duct 6B with UPE

5 Conclusions

- GPR has a limitation in depth and couldn’t locate the second layer of ducts. Between the two different instruments (Proceq and Hilti), Proceq had an advantage in use and reading the results, but both devices could locate the ducts.

- The directions of scanning are important both with GPR and US. The operator needs to know what the point of interest is and plan the scans, both with drawing reviews and experience in post stressed bridge systems.

- With the Ultrasonic you can’t locate the second layer of ducts if they are placed on a straight line behind each other, but if they are offset, it is the limitation in depth that is the deciding factor.

- It is not impossible that even GPR is able to locate voids or at least tendencies to voids, but to be certain of this a lot of tests need to be made before any conclusion can be made.

Statens vegvesen
Pb. 1010 Nordre Ål
2605 Lillehammer

Tlf:
firmapost@vegvesen.no

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vegvesen.no

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