The 3rd International Conference on Structural Health Monitoring of Intelligent Infrastructure
Vancouver, British Columbia, Canada
November 13-16, 2007

USE OF SHM IN DESTRUCTIVE TESTING OF BRIDGES

Prof. Björn Täljsten
Denmark Technical University, Denmark
and
Luleå University of Technology, Sweden

Professor Lennart Elfgren
Luleå University of Technology, Sweden

PhD Student Ola Enochsson
Luleå University of Technology, Sweden

Abstract

This paper present a full-scale test on a strengthen railway concrete trough bridge. A unique opportunity came up. The existing railway line was going to be replaced with a new one and the bridge became obsolete. The purpose of the project was o investigate the shear capacity of the bridge. To avoid an uninteresting bending failure, the bottom beams were strengthened with Near Surface Mounted Reinforcement (NSMR) consisting of Carbon Fibre Reinforced Polymers (CFRP). The project was a part of the European funded research project Sustainable Bridges (www.sustainblebridges.net). The bridge tested is a 50 year old two-span concrete bridge located in Örnsköldsvik in northern Sweden. The bridge was tested to failure to demonstrate and test new and refined methods developed in the project regarding procedures for condition assessment and inspection, load carrying capacity, measurement and strengthening. The bridge was built in 1955.It has two spans of 12 m. In one of the spans a loading beam made of steel was placed in the centre of the span. The loading beam was then pulled down with cables injected to the bedrock beneath the bridge. Usually models for the load carrying capacity are tested in the laboratory in reduced scale. There are very few other concrete bridges which have been tested to failure in order to check their ultimate behaviour and this project demonstrate the use of SHM in full scale testing. The strengthening was successful and the results from the tests shows that the used design and FE-models underestimate the shear capacity considerably.

.

INTRODUCTION

State evaluation in existing structures is normally done by visual inspection, however they provide rather uncertain and limited information and even though more sophisticated inspections methods are available Structural Health Monitoring (SHM) of structures is still a relatively unfamiliar term for civil engineering applications. Monitoring is the act of acquiring, processing and communicating information about a structure over a period of time with a *high level of automation*.

The objective of SHM is to monitor the in-situ behaviour of a structure accurately and efficiently, to assess its performance under various service conditions, to detect damage or deterioration, and to determine the health or condition of the structure [1], [2]. 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 used to improve existing guidelines or codes and form a base for bridge maintenance and management strategies.
Structural health monitoring may also be a very interesting tool to investigate the real behaviour of a structure, its design and its ultimate load carrying capacity. In addition if the existing design codes and guidelines should be improved or if a structures real behaviour and performance should be understood filed tests in the ultimate limit stage is necessary. However, this is only possible in cases when we can apply destructive testing, i.e. testing up to failure, of structural components or the real structure. It has often been shown, see for example [3] that the capacity of existing concrete structures often exceeds the design capacity considearably. In this paper a full-scale test of a railway bridge is presented where the used of SHM is essential for a proper understanding and modelling of the structural behaviour. In this project the Örnsköldsviks bridge was obsolete due to a new railway line and it was decided to test the bridge with consideration to its shear capacity. However, for all reasonable placement of the load a bending failure would arise, therefore a decision about strengthening was taken.

BACKGROUND

In the European funded project Sustainable Bridges the main objective are to increase the transport capacity and service life of existing railway bridges in Europe. In order to demonstrate new and refined methods developed in the program, a field tests of an existing bridges were carried out during June 2006. The bridge presented here is a two-span concrete trough bridge located in Örnsköldsvik in northern Sweden. A photo of the bridge before testing is shown in figure 1.

Figure 1 Photo of the railway bridge in Örnsköldsvik before testing, photo L. Elfgren.

It was of interest to investigate the shear capacity in the beams of the bridge. However, to avoid bending failure, it was decided to strengthen the main girders with NSMR (Near Surface Mounted Reinforcement) CFRP (Carbon Fibre Reinforced Polymer) rods. The procedure for assessment, strengthening and testing is described in the test below. The bridge is a reinforced concrete railway trough bridge with two spans 12+12 m, see figure 2, where the elevation and a plan of the bridge is shown. The bridge was built in 1955 and has now been taken out of service due to the building of a new high-speed railway, the Botnia Line.

The bridge was planned to be demolished in 2006 and the idea was to load it to failure before that in order to test its remaining ultimate load carrying capacity after a service period of 50 years. The concrete design quality is K40 which corresponds to a compressive strength of 40 MPa measured on 200 mm cubes. The dominant steel reinforcement is ϕ 16 and ϕ 25 mm, of quality Ks400 with a nominal yield strength of 400 MPa, the real material data was measured and are recorded below.

The maximum design bending moments and shear forces according to the original calculations from 1954 was an axle load of 250 kN and a shear force of 2.3 MN whereof 0.7 MN due to the dead load. The maximum mid span moment of M = 3.6 MNm, whereof 0.8 MNm from dead load. The maximum bending moment and shear force capacity before strengthening in mid span due to the Swedish concrete codes can roughly be estimated to 10.8 MNm and 4.9 MN respectively. A FE-analys carried out gave an estimated shear force capacity of 7.8 MN.

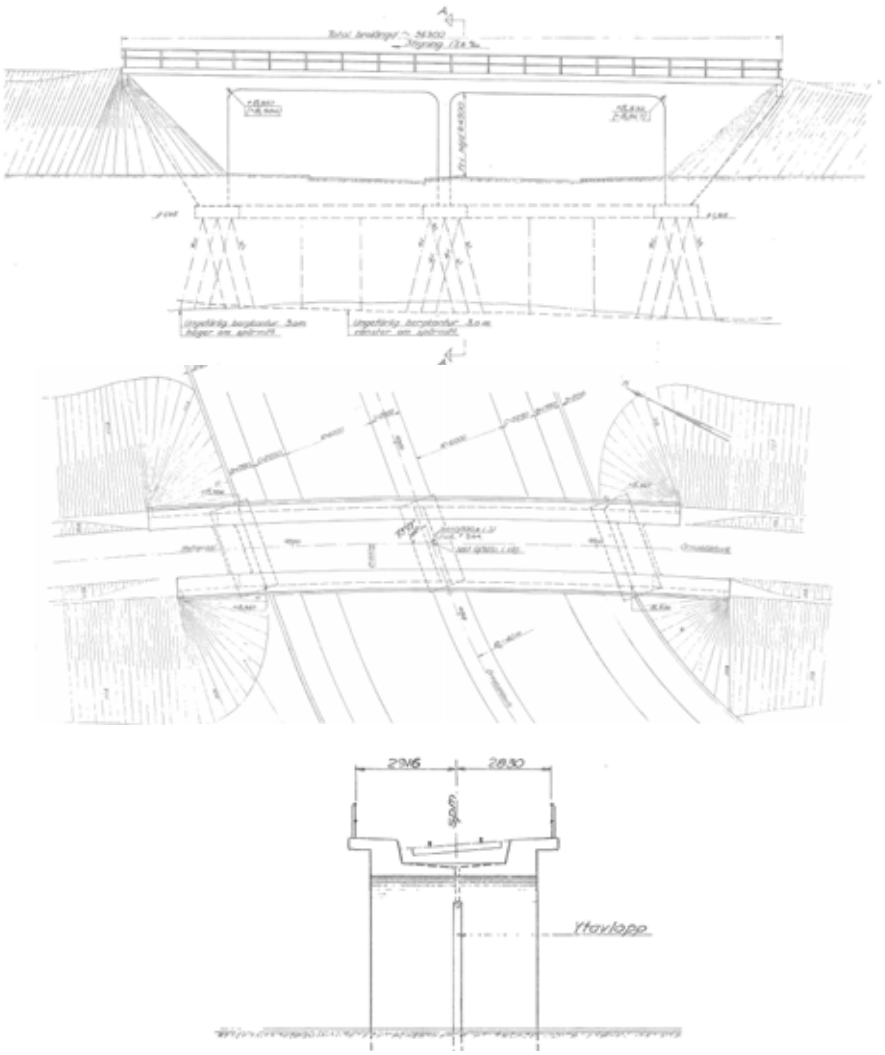


Figure 2 Elevation and plan of the bridge

TEST SET UP

The bridge was tested with a vertical point load in the northern mid span, see figure 3, where the steel beam across the bridge is where the load is applied. Of greatest interest was to investigate the shear capacity of the bridge beams. To obtain this it was necessary to strengthen the bridge for flexure, this is described more in detail below. Three tests were carried out. In test 1, the through slab was loaded through the ballast to check the distribution of loads through the ballast and the load-carrying capacity of the slab. In test 2 the two main beams were tested before strengthening up to the cracking load and in test 3 the two main beams were tested after strengthening up to failure. The two first tests were carried out in the service limit state (SLS) and the last test in the ultimate limit state (ULS). The load was statically applied by pulling the bridge down with large hydraulic jacks mounted on stay cables anchoraged 9 meters into the bedrock beneath the bridge.

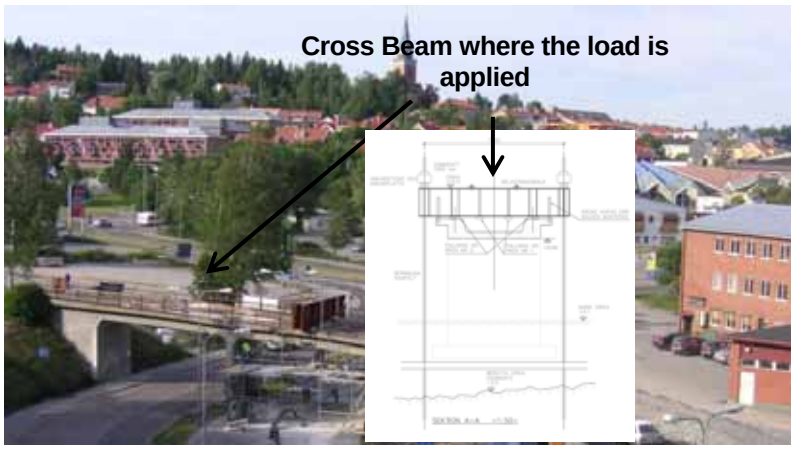


Figure 3 Test site and test set up

Before testing a thorough investigation was undertaken. The investigation included non-destructive testing of the concrete and the placement of the reinforcement, measurements of actual material properties of the steel in the reinforcement bars and of the concrete etc. Also a very detailed planning for both the strengthening work and the monitoring was carried out, for example deflections and strains in concrete, reinforcement bars the carbon fibre composite was to be followed during the loading process.

INSPECTION AND CONDITION ASSESSMENT

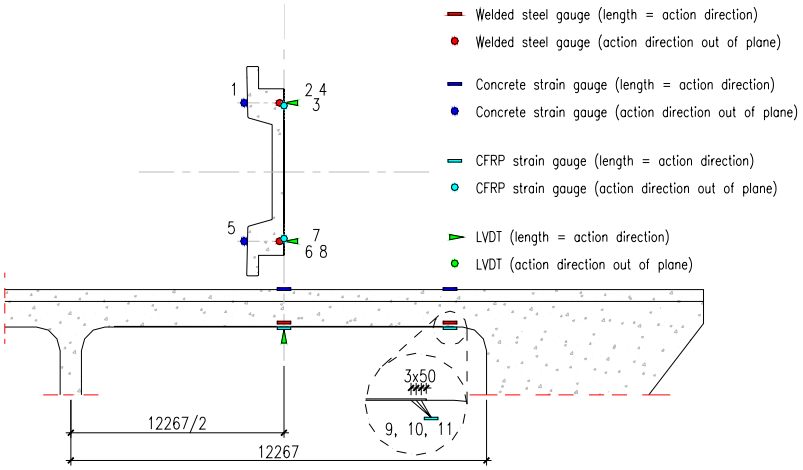
The bridge was inspected in 2005 before it was chosen. When the decision to test the bridge had been made it was inspected again, first by Luleå University of Technology and later by BAM, Germany, and COWI, Denmark. In this paper only some general comments are given to the inspection and assessment. Concrete cores with a diameter of 100 mm were drilled out of the bridge in May 2006. From the main beams four cores were drilled out. They were tested in June 2006 it was found that the mean compressive concrete strength was higher in the wing than the mean value for the two beams of the bridge. The mean value for the two beams was tested to 68.5 MPa with a standard deviation o 8.0 MPa. The mean axial tensile strength for both beams was 3.1 MPa with a standard deviation of 0.4 MPa Three of the tested reinforcement bars (diameter of 16 mm) had a mean yield stress, of 403 MPa with a standard deviation of 15 MPa. The mean value of the yield stress for the bars with the diameter 25 mm was 404 MPa with a standard deviation of 10 MPa measured from three specimens.

Defects and deterioration was tested by BAM and COWI. In general it can be said that the bridge was in a good condition although it had some damages from impact from heavy traffic.

MONITORING

In this particular project the bridge was heavily monitored with many different types of sensors, for example; electrical strain sensors on concrete, steel and CFRP bars, LVDT (Linear Voltage Displacement Transducers) for measuring the displacements at various locations and curvature, laser deflection meters for measuring the mid-displacement, accelerometers, fibre optic crack sensors and fibre optic strain (Bragg) sensors. It is not possible in this paper to report from all placements of the gauges nor the final result from the testing since data is still under evaluation. Nevertheless, the result from strengthening is evaluated and a part of this result will be presented. For the performance of the strengthening system and the effects of it were analyzed by analyzing four quantities; load, strain, curvature and stiffness. The strain distribution was established by applying strain gauges both on the compressed concrete, tensile steel reinforcement and the CFRP rod. The compatibility in the strengthened cross section could thereby be viewed upon as well as the individual strain in each material. This setup makes it also possible to compare the steel strain at a particular load level before and after strengthening. Further, the curvature is calculated from the distribution and it could be

compared to the curvature measured with the external curvature measurement from the LVDT monitoring. The sensor placement and a description of the sensors are presented in figure 4.



Sensor	Denotation	Description
1	CoT4E	Strain on concrete, mid section, east-beam
2	StB4E	Strain on steel reinforcement, mid section, east beam
3	CaB4E	Strain on NSMR rod, mid-section, east beam
4	DlB4E	Deflection, longitudinal curvature, east beam
5	CoT4W	Strain on concrete, mid-section, west beam
6	StB4W	Strain on steel reinforcement, mid-section, west beam
7	CaB4W	Strain on NSMR rod, mid-section, west beam
8	DlB4W	Deflection, longitudinal curvature, east beam
9	CaB7E1	Strain on NSMR rod at the cur off end
10	CaB7E2	Strain on NSMR rod at the cur off end
11	CaB7E3	Strain on NSMR rod at the cur off end

Figure 4. Placement and type of sensors – not all results from the sensors are reported

STRENGTHENING

The chosen strengthen method was Near Surface Mounted Reinforcement (NSMR) rectangular bars of Carbon Fibre Reinforced Polymers (CFRP) which then were mounted by bonding in sawed out groves in the slab, the size of the grooves where 15 x 15 mm. This configuration does not interfere with the existing steel reinforcement. Research at Luleå University of Technology has shown that this method is superior compared to externally bonded FRP plates [4] and the method has also been used in several field applications in Sweden, both in buildings and bridges. In this particular case rectangular bars with a cross section of 10 x 10 mm were used. The rods chosen where provided by Sto Scandinavia AB with the brand name Sto FRP Bar M10C. The modulus of elasticity for the rods was 250 GPa with a strain at failure of 11 ‰, the material data is given by he material manufactured but has also been confirmed in lab [5]. The adhesive used for bonding was Sto BPE Lim 567 (A+B) a cold cured two component tixotropic epoxy adhesive with a modulus of adhesive of 6.5 GPa and a bond strength of approximately 20-22 MPa. The bonding process are described more in detail below.

Groows are being sawed

Final grooves

Figure 5 Making of grooves in the beams, photo B. Täljsten

The strengthening design is based on calculations regarding the bridges original capacity, which was estimated to approximately 6 MN for the actual placement of the load. To obtain a shear failure the bridge needed to be loaded up to approximately 8 MN. The strengthening design provided an additional capacity of 3 - 4 MN, i.e approximately a 25 - 35 % increase in flexure. The additional 4 MN corresponded to 18 CFRP rods, 9 per beam, with a length of 10.0 m.. These rods consisted of a high quality carbon fibre with the modulus of elasticity of 250 GPa and a strain at failure of approximately 11 ‰.

In figure 7 the placement of the CFRP rods are shown. As can be seen in this figure the rods are placed in the soffit of the bridge beams with a centric distance of 100 mm between the rods. The strengthening work was divided into three steps, before strengthening, during strengthening and after strengthening. Where planning of proper scaffolding and its placement was the first thing that was carried out. The next step was to mark for the placement of the NSMR rods and mounting the railing for the saw system. Then the grooves were sawn up and properly cleaned so all cement debris was washed out from the grooves. The next step was to divide the work site into different stations; one station for the rods, one station for mixing the adhesive, one station for construction waste etc. The process during strengthening can be explained stepwise. Since all CFRP rods were protected with peel-ply this had to be removed first. The adhesive was mixed and applied in the grooves. The next step was then to mount the CFRP rods in the grooves. The adhesive was let harden three days at 20 °C (average) before testing. In figure 6 mounting of one of the rods and the final result after strengthening are shown.

Mounting of CFRP rods

Final strengthening result

Figure 6 NSMR strengthening, photo B. Täljsten

The bonding was controlled due to voids and other cavities by tapping the bonded area with a hammer. No voids were found and the strengthening construction work was considered successful. The work site was finally cleaned from all waste and adhesive remainders.

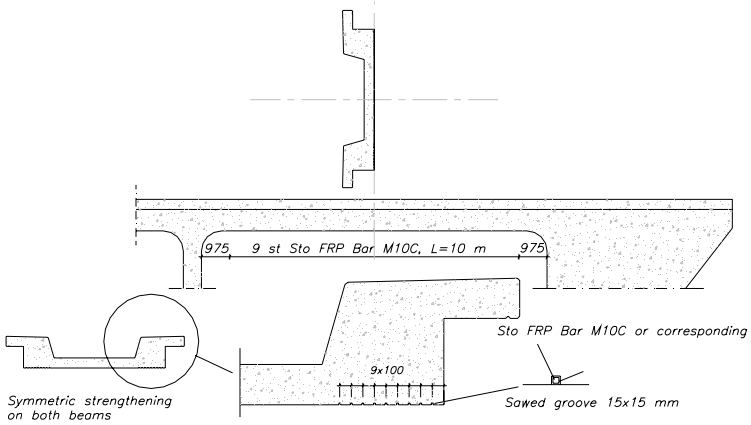
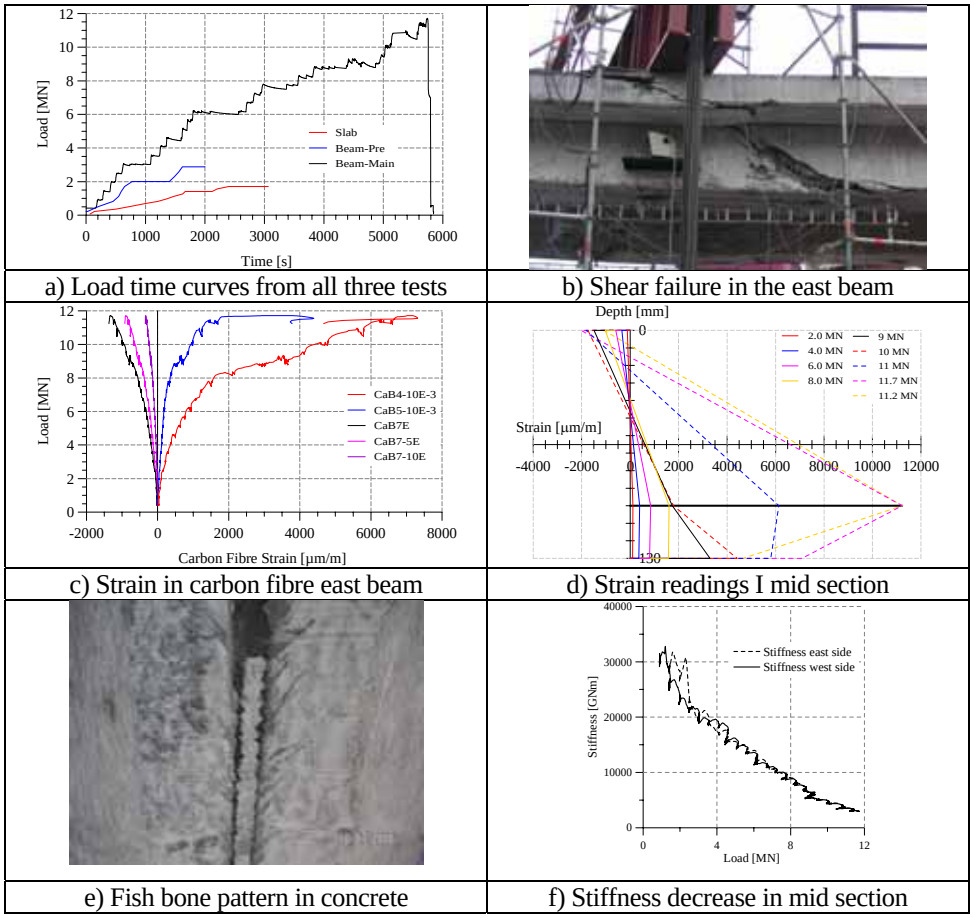


Figure 7 Placement of the rods in the soffit of the bridge beams.

FIELD TESTING AND TEST RESULTS

The test carried out can not directly be translated into train load. However, it is still interesting to make a rough comparison with the design axel load and the loading of the slab before strengthening since this loading is closes to a real train load on the bridge. A train bogie consists of two axels, that gives an approximately weight of 0.4 MN. The loading carried out on the slab can consider to be in the service limit state. Here the bridge was loaded up to 1.7 MN, see figure 8a). At this level the crack widths was measured to approximately 0.2 – 0.4 mm and the stress in the steel was measured to approximately 80 MPa. Consequently the service performance was considerably higher than the design load.

The failure was an expected shear failure, shown in figure 8b) – the failure arose almost simultaneously in both beams at a load of 11.5 MN, considerably higher than the analysis for the hand calculation, 4.9 MN and from the FE-analysis 7.8 MN. At this level the tensile steel was yielding in the beams and considerably strain (stress) was taken up in the CFRP rods, see figure 8c). The strain in the CFRP corresponds approximately to a stress of 1950 MPa. Furthermore it was shown, see figure 8d), that we had a slip in the rod at load levels near failure. This can also be seen in figure 8e) where a typical fish bone pattern had developed close to the rods. A calculation of the shear stress from the measured strain at the end of the rod shows that it was possible to transfer as high shear stresses as 10 MPa into the concrete from the CFRP rods. The curvature in the mid section was calculated for different load levels from figure 8d) and the stiffness as a function of the load has been plotted in figure 8e) where the stiffness decrease in the beams clearly can be noticed. From this measurement is it obvious that the curvature can be used as a parameter to follow the performance of a structure or a structure component.



a) Load time curves from all three tests

b) Shear failure in the east beam

c) Strain in carbon fibre east beam

d) Strain readings I mid section

e) Fish bone pattern in concrete

f) Stiffness decrease in mid section

Figure 8 Results from test

DISCUSSIONS AND CONCLUSIONS

In the current paper a full scale test up to shear failure of a concrete trough bridge is presented. It was of primary interest to investigate the shear capacity of the bridge beams. However, for all reasonable placement of the load a bending failure would occur. To avoid this, the bridge beams was strengthened for flexure with NSMR CFRP rectangular rods. Extensive monitoring was carried out, both before and after strengthening. For the Örnsköldsviks bridge the analytical design model, the Swedish concrete code, underestimate the shear capacity of the bridge beams with approximately 100 %and the FE-model with approximately 50 %. The strengthening of the bridge was very successful and a stress of approximately 1950 MPa was calculated from strain readings in the CFRP rods.

Furthermore, very high shear stresses, approximately 10 MPa were transferred from the CFRP rods to the concrete in the bonded slots. At failure a very distinct fish bone pattern had developed in the concrete and the end location of the rods. It was found from the test that it is very difficult to predict the ultimate behaviour of the bridge even though it was mapped in detail before the monitoring and testing was carried out. The study stresses the importance of using SHM for evaluation of existing design models and the behaviour of real structures and it will be very interesting and challenging to evaluate the result from the bridge further.

FUTURE RESEARCH

There is a great challenge to use SHM for understanding our existing structures. A large amount of information can be obtained through monitoring in the SLS. However, it is important also to carry out SHM in the ULS. By this it is possible to obtain very detailed information about the tested structure. In the near future we will continue to evaluate the test result and to make more accurate models of the bridge.

ACKNOWLEDGEMENT

The authors first of all want to acknowledge the European Union for funding the research in the project Sustainable Bridges, without this funding the testing would not have been possible. Also the Swedish Rail Administration, Banverket, shall be acknowledged for their support during the test project. Finally all personnel taking part in the work, especially laboratory technicians should be acknowledge - without your hard work and effort there would not have been any testing at all.

REFERENCES

1. Hejll A., 2007, Civil Structural Health Monitoring – Strategise, Methods and Applications, Doctoral Thesis 2007:10, Luleå University of Technology, ISBN: 978-91-85685-08-0, 189 p
2. Mufti A. ,ISIS Canada, 2001, Guidelines for Structural Health Monitoring, Design Manual No. 2, ISBN 0-9689006-0-7. p 120.
3. Täljsten B., 1994, *Plate Bonding, Strengthening of Existing Concrete Structures with Epoxy Bonded Plates of Steel of Fibre Reinforced Plastics*, Doctoral Thesis 1994:152D, ISSN 0348 – 8373, Department of Structural Engineering, Luleå University of technology, 1994, 308 p
4. Nordin H., 2003, "Fibre Reinforced Polymers in Civil Engineering – Flexural Strengthening of Concrete Structures with Prestressed Near Surface Mounted CFRP Rods, Licentiate Thesis, Luleå University of Technology, ISBN: 91-89580-08-7, 145 p
5. Nordin H. and Täljsten B., (2006), "*Concrete Beams Strengthened with Prestressed Near Surface Mounted CFRP*", Journal of Composites for Construction, Vol 10, No. 1, February 1, 2006, pp 60-68.