FLEXURAL-SHEAR FAILURE OF A FULL SCALE TESTED RC BRIDGE STRENGTHENED WITH NSM CFRP

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This paper present a full-scale test on a strengthen 50 year old 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 was extensive monitored with both traditional monitoring systems and with more advanced systems such as fibre optics and photographic monitoring tools. In order to investigate the shear-bending interaction and to avoid a pure bending failure, the bridge was strengthened with rectangular bars of near surface mounted (NSM) Carbon Fibre Reinforced Polymers (CFRP). The longitudinal load carrying capacity was increased with 28 %. The maximum vertical load the bridge could carry was 11,7 MN and bond failure of the CFRP bars initiated rupture of the stirrups and failure of the bridge. The capacity was assessed with codes from the United States (ACI), from Canada (CSA), and Europe (EC2). The European truss model with a variable compressive strut angle gave the most conservative result, the Canadian simplified compression field theory a somewhat less conservative result, and the American addition theory gave the most realistic result. The very conservative results are partly due to the fact that the bridge had a comparably low shear reinforcement percentage of 2%. The conclusions are that (1) NSM CFRP worked excellently and increased the flexural and shear capacity considerably, (2) the present codes are quite conservative, and (3) more accurate models are necessary to accurately assess the capacity of existing bridges.

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ABSTRACT: A test to failure has been carried out on a 50-year-old RC railway trough bridge. In order to investigate the shear-bending interaction and to avoid a pure bending failure, the bridge was strengthened with rectangular near-surface mounted (NSM) Carbon Fibre Reinforced Polymers (CFRP) bars. The longitudinal reinforcement capacity was hence increased by about 30 %. The bridge was heavily monitored al the way up to failue. The failure occurred at 11.7 MN load, by a flexural shear failure mechanism. At failure intermediate crack (IC) debonding of the CFRP bars was noticed, and at the same moment the major shear crack opened. The capacity of the bridge was assessed using codes from the United States (ACI), Canada (CSA) and Europe (Eurocode). The European standard with variable angle truss theory (VAT) gave the most conservative results, while the Canadian simplified modified compression field theory (MCFT) and the US fixed truss model (45° TM) gave less conservative estimates. The conclusions are that NSM CFRP worked excellently and increased both the flexural and shear capacity of the bridge. Additionally the present codes provided conservative estimates of shear force capacity, and more accurate models are required for detailed capacity assessments of existing concrete through bridges.

1 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, Structural Health Monitoring (SHM) methods are available this term is still a relatively unfamiliar 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.

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 field tests in the ultimate limit state is necessary.

European railway bridges are older than 50 years (Bell (2004), hence most of these bridges require specific monitoring, strength assessment and structural rehabilitation or strengthening as a result of increases in traffic loads, frequencies and speeds. Some of the aspects have been addressed, to varying degrees, in recent updates of three of the major design codes: ACI (ACI-318, 2008), Eurocode (CEN, 2005a) and CSA (CSA-A23.3, 2009). However, validating the models applied in the codes has been difficult since few results of full-scale failure load tests have been published (Scordelis et al., 1977; Scordelis et al., 1979; Plos, 1990; Täljsten, 1994; Plos, 1995), possibly because of time, legal and cost constraints. Such data are extremely valuable for elucidating the complex structural behaviour of bridges and their components. Clearly, there is a need for refining current design methods, and/or developing new techniques, for monitoring and assessing bridges in service, and for testing possible solutions for repairing and strengthening them. These were major aims of the Integrated Research Project ‘‘Sustainable Bridges’’, part of the European Framework Program 6, which incorporated field tests of existing bridges. This paper describes results of the tests on one of the selected bridges; the two-span, concrete railway trough bridge located in Örnsköldsvik, Sweden, see Figure 1. It was considered to be more scientifically challenging to investigate the shear failure mechanism rather than the flexural failure. Hence, this bridge was strengthened in flexure with NSM CFRP bars in order to ensure a shear failure following loading in the test. The NSM reinforcement strengthening increased the flexural capacity of the bridge, thus resulting in the desired shear failure. However, the complexity of the failure mechanism also increased; the shear failure occurred simultaneously with flexural failure, and intermediary crack (IC) debonding of the NSM reinforcement from the concrete surface was noticed.

2 BACKGROUND

2.1 The Bridge

The bridge is a reinforced concrete railway trough bridge with two spans 12+12 m, see Figure 2, where the 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

Figure 1 The railway bridge in Örnsköldsvik before testing, (Photo L. Elfgren).

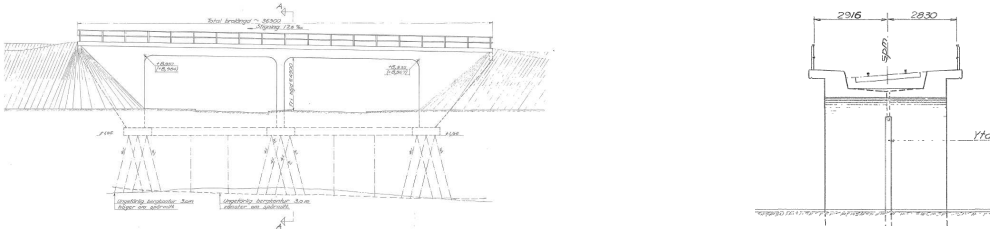


Figure 2 The evaluation of the bridge

2.2 Material properties

The bridge was designed using a K 400 (40 MPa) concrete class and Ks 40 (400 MPa) quality steel reinforcement. According to the current European Standard Eurocode (CEN, 2005a), the concrete had an equivalent strength class of C28/35. The steel reinforcement consisted of hot rolled steel bars with diameters (Ø) of 10, 16 and 25 mm. Prior to the test concrete cylinder core samples were collected and tested. The mean value of the concrete cylinder compressive strength was determined to be 68.5 MPa, with a standard deviation of 8 MPa. According to the Eurocode (CEN, 2006) standard this value corresponded to a characteristic compressive strength of 57 MPa. The steel reinforcement properties were tested according to (CEN, 2005b) after the bridge has been demolished. Tests of the steel reinforcement properties according to (CEN, 2005b), after the bridge has been demolished, indicated that the Ø 16 and 25 mm bars had tensile strengths of 441 and 411 MPa at yielding. No Ø 10 mm bars were tested. The rectangular 10 x 10 mm CFRP bars had a modulus and strain at failure corresponding to 250 GPa and 0.8 % respectively. The adhesive used for bonding was a cold cured two component epoxy adhesive with a modulus of 6.5 GPa and a bond strength of approximately 22 MPa.

2.3 Test setup

The bridge was tested with a vertical point load in the northern mid span, see Figure 3. In total three tests were carried out. In the first test, the through slab was loaded through the ballast, to check the distribution of loads and the cracking load of the slab. In the second test, the two main beams were tested before strengthening up to the cracking load and in final test 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. Before testing a thorough investigation was undertaken.

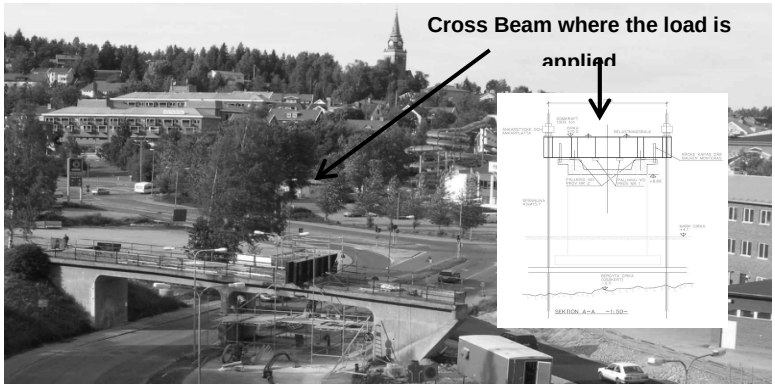


Figure 3 Test site and test set up

3 CAPACITY ASSESSMENT

Calculating the shear capacity of the bridge was not straightforward. At the time of the test two standards were used for this purpose; the Swedish national standard (BBK04, 2007) and the draft version of the European standard (CEN, 2005a). The shear capacity provisions in these two standards are different; while the Swedish standard (BBK04, 2007) applies a fixed truss model, the European norm uses the variable angle truss model. Given awareness of their conservative nature, to obtain a realistic assessment the shear force capacity was determined using a combination of these two methods. The concrete's contribution was calculated using the Swedish code and the stirrups' contribution using the European standard, considering a crack inclination angle of 30°.The shear force and bending capacity of the bridge before strengthening were determined to be 4.92 MN and 10 MNm**,** respectively. The load *P* required for shear failure was estimated to be 8.29 MN and the load required for bending failure 5.76 MN. Since a shear failure was desired the bending capacity needed to be increased, to ca. 14.4 MNm, to accommodate a minimum concentrated load of about 8.3 MN. Thus a bending capacity deficit of ca. 4.4 MNm needed to be carried by the FRP strengthening. Therefore, the bridge was strengthened before the final failure test with 18 (nine per beam) 10 m long NSM CFRP bars.

Figure 4 Making of grooves in the beams

Figure 5 Placement of the rods in the grooves.

After applying the adhesive in the grooves, see Figure 4, the rods were mounted in the soffit of the bridge beams with a centric distance of 100 mm between the rods. The adhesive was let harden three days at 20 °C (average) before testing. In figure 5 it is demonstrated how one of the rods is mounted. In Table 1 a comparison of code calculations versus the load at failure are made. It can here be noticed that calculated capacities are not very close to the test result.

Table 1 The capacities and the failure loads predicted by the compared standards for the strengthened bridge

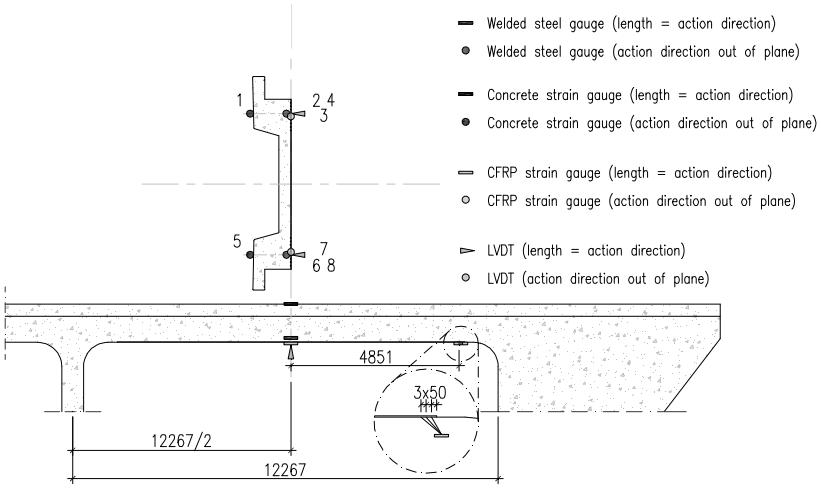
	EC2			CSA	ACI	Test
θ	21°8'	45°	33°8'	37°9'	45°	≈ 32°
V_s (MN)	2.54	1.06	1.52	1.31	1.06	-

V_c (MN)	-	-	-	0.97	1.24	-
V_{Rd} (MN)	2.54	1.02	1.52	2.28	2.29	-
M (MNm)	6.12	6.12	6.12	6.12	6.12	-
P_V (MN)	9.12	3.64	5.44	7.67	7.74	11.7
P_M (MN)	6.64	6.64	6.64	6.64	6.64	11.7

Where θ is the crack angle inclination, V_s the shear force carried by the stirrups, V_c, the shear force carried by the concrete. V_{Rd} is the shear force resistance, M the maximum bending moment, P_V the applied force that produces shear failure and P_M the applied force that produces bending failure.

4 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 rods, LVDT (Linear Variable Displacement Transformers) 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. The sensor placement and a description of the sensors related to CFRP strengthening are presented in figure 6.



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 6 Placement and type of sensors – not all results from the sensors are reported

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 deflection monitoring.

5 FIELD TESTING AND TEST RESULTS

The test carried out cannot 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. The loading carried out on the slab can consider being in the service limit state. Here the bridge was loaded up to 1.7 MN, see Figure 7. 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 8 – the failure arose almost simultaneously in both beams at a load of 11.7 MN.

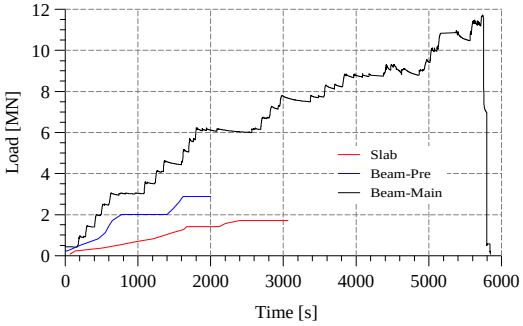


Figure 7 Load time curves from all three tests

Figure 8 Shear failure in the east beam

At this level the tensile steel was yielding in the beams and considerably strain (stress) was taken up in the CFRP rods, see figure 9. The strain in the CFRP corresponds approximately to a stress of 1950 MPa. Furthermore it was shown, see figure 10, that we had a slip in the rod at load levels near failure. This can also be seen in figure 11 where a typical fish bone pattern had developed close to the rods.

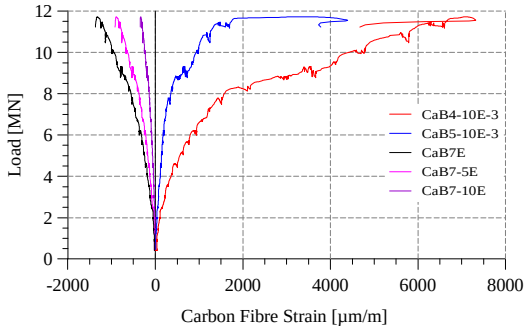


Figure 9 Strain in carbon fibre east beam

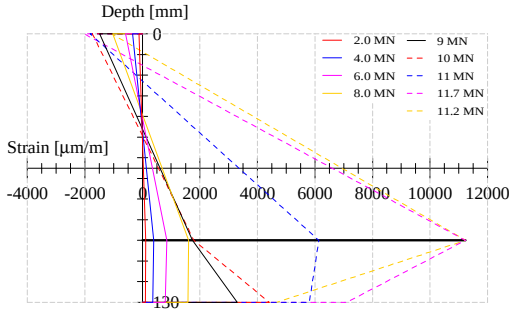


Figure 10 Strain readings in mid section

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.

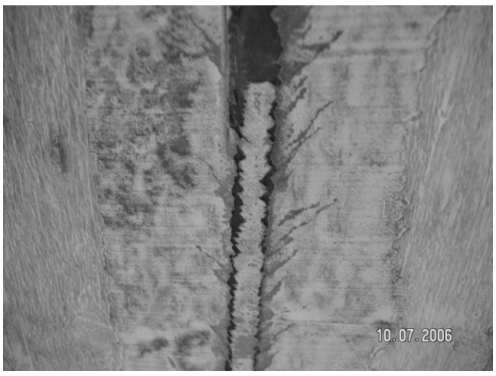


Figure 11 Fish bone pattern in concrete

6 DISCUSSION 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 %. 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.

The three codes used to predict the shear force capacity of the bridge after the strengthening capture this interaction differently, but all predict the shear force capacity in a conservative manner. One advantage of using Eurocode is that the VAT model predicts the crack inclination angle quite accurately, but it gives the most conservative estimates of the shear capacity because it does not consider the concrete contribution to the total shear capacity (which can lead to unexpected failure modes).

The ACI model is a fixed model that does not consider the flexural shear interaction. The advantages of using this model for such assessment is that it is simple, direct and provides the most realistic estimates. The CSA model captures the flexural shear interaction in a conservative manner compared to Eurocode. The shear crack angle predicted by this standard and used in the estimation of the shear force capacity leads to more conservative results for the stirrups' contribution than Eurocode, but higher estimates than the ACI model. The authors believe that this is connected to the fact that the longitudinal strain at the middle of the cross-section has been calibrated with test results obtained from steel reinforced concrete beams rather than FRP strengthened beams.

7 ACKNOWLEDGEMENTS

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