CFRP Strengthening and monitoring of the Gröndals Bridge in Sweden

B. Täljsten[1,2] and A. Hejll[1,3]

[1] Department of Civil Engineering
Luleå University of Technology
971 87 Luleå, SWEDEN
arvid.hejll@ce.ltu.se
bjorn.taljsten@ce.ltu.se

[2] Skanska Teknik AB
169 83 Solna, SWEDEN
bjorn.taljsten@skanska.se

[3] EMPA
Structural Engineering
Research Laboratory
8600 Dubendorf, Switzerland
arvid.hejll@empa.ch

ABSTRACT: The Gröndal Bridge, is a large freivorbau bridge (pre-stressed concrete box bridge), approximately 400 meters in length with a free span of 120 m. It was opened to tram traffic in 2000. Just after opening cracks were noticed in the webs, these cracks have then increased, the size of the largest cracks exceeded 0.5 mm, and at the end of 2001 the bridge was temporarily strengthened. This was carried out with externally placed prestressed steel stays. The reason for cracking is still debated and will be further discussed in this paper. Nevertheless, it was clear that the bridge needed to be strengthened. The strengthening methods used were CFRP laminates in the Service Limit State (SLS) and prestressed dywidag stays in the Ultimate Limit State (ULS). The strengthening was carried out during 2002. At the same time monitoring of the bridge commenced, using LVDT crack gauges as well as optical fibre sensors.

To date, a large amount of data has been collected and the data is still under evaluation. Primary results show that the largest stress can be referred to the temperature load and that the contribution from the live load is minor. The results from the monitoring show that the CFRP laminates work as intended and that the cracks are not propagating.

1 INTRODUCTION

1.1 *Background*

Over the past decade, the issue of deteriorating infrastructure has become a topic of critical importance in Europe, and to an equal extent in North America and Japan. FRP (Fibre Reinforced Polymers) are today used for various applications, such as reinforcement in RC and PC structures, stay cables and newly built structures. However, by far the most extensive application is in repair and strengthening of existing structures. This strengthening technique may be defined as one in which composite sheets or plates of relatively small thickness are bonded with an epoxy adhesive to, in most cases, a concrete structure to improve its structural behaviour and strength. The sheets or plates do not require much space and give a composite action between the adherents. The adhesive that is used to bond the fabric or the laminate to the concrete surface is normally a two-component epoxy adhesive. The old structure and the newly adhered material create a new structural element that has a higher strength and stiffness than the original.

The motivation for research and development into repairing, strengthening and restoration of existing structures, particularly concrete systems, is increasing. If consideration is given to the capital that has been invested in the existing infrastructure, it is not always economically viable to demolish and rebuild a deficient structure.

The challenge must be to develop relatively simple measures such as restoration, reparation and strengthening that can be used to prolong the life of structures. This challenge places a great demand on both consultants and contractors. Also, there could be difficulties in assessing the most suitable method for an actual repair; for example, two identical columns within the same structure can have totally different lifespan depending on their individual microclimate.

The use of epoxy as the bonding medium for the adherent has proven to give excellent force transfer. Not only does epoxy bond to concrete, steel and composites, it has also shown to be durable and resistant to most environments.

1.2 *History*

The method of strengthening existing concrete structures with the use of epoxy adhesives originates in France in the nineteen-sixties (L´Hermite, 1967), (Bresson, 1971), where tests on concrete beams with epoxy bonded steel plates were conducted. Even though the method was used widely, it was not considered very successful.

The drawbacks such as corrosion, the need for overlap joints, the heavy working loads during installation and the need for pressure on the adhesive during hardening could not be overcome. In the last decade the plate bonding method has gone through a revival. The reason for this is mainly the increased need for retrofitting of our existing buildings and bridges. However, another very important factor is the introduction of advanced composites to the civil engineering arena. Fibre composites and reinforced plastics offer unique advantages in applications where conventional materials cannot supply a satisfactory service life (Agarwal & Broutman, 1990).

The high strength to weight ratio and the excellent resistance to electrochemical corrosion of composites make them attractive materials for structural applications. In addition, composites are formable and can be shaped to almost any desired form and surface texture. One interesting application of currently available advanced composite materials is the retrofitting of damaged or structurally inadequate building and bridges. In Switzerland, (Meier, 1987), one of the first applications with the use of carbon FRP (CFRP) was carried out at the end of the 1980´s, and since then several thousand applications have been carried out worldwide.

Clearly there is a great potential for, and considerable economic advantages in FRP strengthening. However, if the technique is to be used effectively, it requires a sound understanding of both the short-term and long-term behaviour of the bonding system. It also requires reliable information concerning the adhesion to concrete and composite. The execution of the bonding work is also of great importance in order to achieve a composite action between the adherents. Of the utmost importance is to know the practical limits of any proposed strengthening method.

1.3 *Research at Luleå University of Technology*

At Luleå University of Technology, Sweden, research has been carried out in the area of plate bonding. The research work started in 1988 with steel plate bonding and is now continuing with FRP materials. Both comprehensive experimental work and theoretical work have been carried out.

Initially steel plates were used to strengthen members. Currently, however, all research is focussed on plate bonding using fibre reinforced polymer (FRP) composite materials in which, carbon fibre composite is the favoured material.

The laboratory tests have included strengthening for bending as well as for shear (Täljsten, 2001). Full-scale tests on strengthened bridges have also been performed (Täljsten, 1994, Täljsten & Carolin, 1999 and Täljsten, 2000). In particular, the theory behind the development of peeling stresses in the adhesive layer at the end of the strengthening plate has been studied, as has the theory of fracture mechanics to explain the non-linear behaviour in the joint (Täljsten, 1994, Täljsten, 1996 and Täljsten, 1997).

In Sweden the FRP strengthening methods have been used in the field for almost 10 years now, and both laminates and wrap systems are used. Sweden is also one of the first countries in the world where a national code exists for FRP strengthening (Täljsten, 2003).

2 THE GRÖNDAL BRIDGE

2.1 *Background*

The main span of the Gröndals Bridge is 120 meters with two adjacent spans each of 70 meters (see Figure 1). The bridge carries two railway tracks which are placed symmetrically about the cross-section of the bridge. The bridge has no footpath.

Bridge inspection carried out on the newly built, 2000, Gröndals Bridge revealed extensive cracking in the webs of its concrete hollow box-girder section. The bridge is a part if a light-rail commuter line which is located in the south of Stockholm. The cause of cracking is still under investigation and has resulted in several articles in Swedish construction industry magazines, (Sundquist, 2002 and Hallbjörn, 2002).

The bridges were designed to the currently applicable Swedish codes, BRO 94 and BBK 94. On the basis of these regulations, it was possible to erect the bridges with extraordinarily slender webs. Relatively high shear stresses and principal stresses are generated by the small web widths although the webs are fully compressed considering the normal stresses caused by pre-stressing in the longitudinal direction. Furthermore, the permanent loads on the structure are dominant. As the permanently exerted principal tensile stresses reached the value of the tensile strength of the concrete, shear cracks were finally created. In addition, restraining bending moments have been superimposed in the webs due to sun radiation. Assuming a linear temperature difference of 10 to 15 K, this, together with the other transverse bending moments, additionally causes vertically directed tensile stresses in the inside of the web amounting to approximately, σ_z = 2 to 3 MPa. The positions of the cracks in longitudinal direction of the bridges correspond to the areas of the maximum principal tensile stresses, (König, 2002).

The cracks first appeared after only a few years of service and subsequent inspections showed that the number and size of the cracks were increasing, (James, 2004). The cracks widths were between 0,1 - 0,3 mm and in a few isolated cases between 0,4 - 0,5 mm in the most cracked sections.(when was this- in 2000 or 2004?)

Preliminary investigations as to the cause of the cracking suggested that they were due to inadequate shear reinforcement in the webs. The webs are slender with a thickness of 350 mm and a total height of the box girder close to the main span supports of approximately 7,5 m. In addition to that the flanges are quite thick, the bottom flange at most is about 1300 mm. The reasons for cracking can be summarised as follows:

- Due to the slender webs high tensile stresses are developed.
- The principal stresses, due to high permanent loads, are the main cause for cracking.
- The location of the cracks is in accordance with the highest principal stresses.
- The cross-sections are under-reinforced due to shear reinforcement.

To increase the safety level of the bridge strengthening was decided.

Because of the progressive nature of the cracking in combination with wariness for shear cracks, the bridge was temporarily closed for traffic towards the end of 2001.

2.2 *Strengthening*

The bridge required strengthening in several sections, strengthening was needed both in the ultimate limit state (ULS) and in the service limit state (SLS), but in different sections. Strengthening in the ultimate limit state was carried out by pre-stressed dywidag-stays and in the service limit state by CFRP laminates. Strengthening in the SLS was in this case particularly complicated since for this type of structure the portion of the dead load is considerable, approximately 85 % of the total load. It was decided to use high modulus carbon fibre laminates to strengthen the bridge. The purpose of strengthening in the SLS was to inhibit existing cracks and prevent new cracks from developing. Therefore the existing crack widths would be reduced to no more than 0.3 mm (maximum allowed crack width in the Swedish code in the SLS). The design of the CFRP strengthening in the SLS is not covered by existing codes and a fracture mechanics approach was here applied where considerations to the total energy to open up new cracks over a unit distance was taken. The design, due to limited space, is not presented in this paper but the design philosophy is discussed. The sections strengthened with CFRP laminates had no or only very small cracks, < 0,05 mm, at time of strengthening. In this particular case strengthening performed in the SLS state will also contribute in the ULS state. Carbon fibre strengthening was chosen to prevent and minimise future cracks in areas where minor or no cracking had developed. In the calculations the strain in the existing steel stirrups has been calculated due to sectional forces. In the calculation the following assumptions have been made:

- Calculation in Stadium I, non-cracked concrete.
- The crack widths are the limited factor.
- The cracks are not allowed to become larger than 0.3 mm after strengthening
- The effect of pre-stressing has in the crack calculation been neglected but is accounted for when calculating the internal forces.
- The concrete is only exposed to tensile stresses perpendicular to the crack plane.
- Only vertical reinforcement in the webs has been considered.
- The concrete starts to crack at approximately 100 μs.
- The first visible crack arises at approximately 0.05 - 0.10 mm.
- Characteristic material data has been used in the calculations.

Table 1 Material data for CFRP Strengthening

	Characteristic value		Design value	
Concrete	E_{ck}	*33.0 GPa*	E_c	*22.9 GPa*
	f_{cck}	*32.0 MPa*	f_{cc}	*17.8 MPa*
	f_{ctk}	*2.10 MPa*	f_{ct}	*1.17 MPa*
	ε_{cu}	*3.5 ‰*		
Steel	E_{sk}	*200.0 GPa*	E_s	*173 GPa*
	f_{yk}	*490.0 MPa*	f_{st}	*371 MPa*
			A_s	*2010 mm²/m*
BPE® Laminates 1412M (1.4 x 120 mm)	E_{fk}	*250 GPa*	E_f	*189 GPa*
	ε_{fk}	*11.0 ‰*	ε_{fd}	*5.0 ‰*
Dimensions	b	*1.0 m*	d	*3.8 m*

The material data for concrete, steel and CFRP laminates is given in Table 1. In the design it has been assumed that the steel reinforcement may yield in the crack tip for a crack width of 0.3 mm. However, bonding CFRP laminates to the surface of the concrete at development of cracks gives a stress (and strain) distribution between the steel and composite. The bridges are very important for the commuters going from the south parts of Stockholm into the centre of the town. For that reason it was undesirable that the bridges were closed during strengthening. However, it has been shown that strengthening with CFRP laminates can be carried out during traffic (Hejll and Norling, 2002); therefore it was decided to permit traffic during strengthening.

Before the strengthening work started the concrete surfaces were sandblasted and holes were drilled in the upper and lower flanges for anchorage of the laminates. Laminates were only placed on the inside of the bridge.

The surfaces were thoroughly cleaned with pressurised air and vacuum cleaners. The surfaces to be bonded were treated with a primer for the system to enhance the bond. The laminates were bonded to the surface, the webs of the structure with a high quality epoxy adhesive, BPE® Lim 567, specific for the strengthening system used. The Young's modulus of the adhesive is approximately 6.5 GPa at 20 °C. The average thickness of the adhesive was *2 mm*. A total of 2 500 meters of CFRP laminates was used for the bridge. The placement of the CFRP laminates in the Gröndal Bridge in section A, the east side is shown in Figure 1.

It can also be noticed in Figure 1 that the CFRP laminates have been bonded at an angle of 70° to the horizontal plane. This was in order that for the laminates to be bonded perpendicular to the direction of the cracks.

In Figure 2 the anchorage system is shown. Here, steel plates with welded steel bars that were anchored by epoxy bonding in pre-drilled holes in the top and bottom flanges. The anchor length was approximately 250 mm.

Figure 2 Anchor system for the CFRP laminates. The anchorages are bonded with an epoxy adhesive in pre-drilled holes in the flanges

The final result after strengthening is shown in Figure 3.

Figure 3 Result after strengthening - seen from inside the bridge. This photo shows strengthening approximately from section A to the column no 8.

To follow up the behaviour of the bridge over time a monitoring system was installed. The system and some results from the monitoring are explained in the next section.

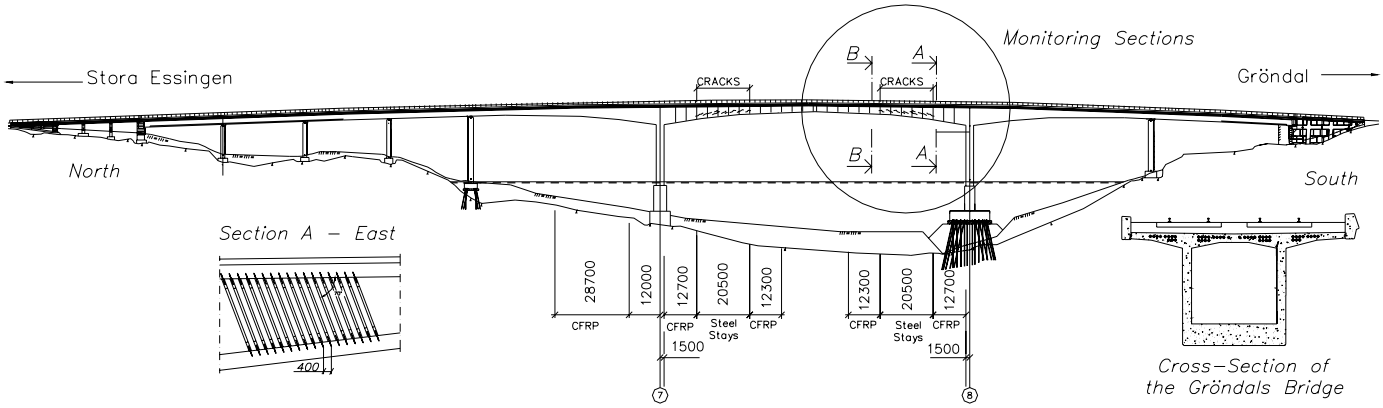

Figure 1 The Gröndals Bridge , CFRP strengthening and monitoring area

3 MONITORING THE BRIDGE

3.1 *SHD System*

When the first crack in the webs of the bridge was found it would have been preferable if measurements had been undertaken immediately. A visual inspection has been carried out regularly, but unfortunately no real measurements have been carried out. In spite of this it has been decided to measure the behavior of the bridges after strengthening, and in particular the future crack developments. Consequently a monitoring program has been put together. In this program measurements have been suggested to be taken at specific locations to measure crack development and strain on the CFRP laminates.

To obtain the most out of a measurement program it is important to carry out SHD (Structural Health Diagnostics) in a structured way and that a well-planned procedure is followed. Luleå University of Technology has worked out a method termed Structural Health Diagnostics (SHD), which in short implies that a rough diagnosis is made on the structure; this can be simple calculations, on site visual inspections or minor measurements. Next it is decided what is to be measured and the purpose with the measurement. In this phase the acquisition system is also decided. Sensors and communications systems are established as well as hard- and software. On top of this, an evaluation system is connected which connects the data from the measurement to a model of the structure. From this action plans are then suggested. In figure 4 an example on a SHD for a railway bridge is shown.

3.2 *SHD System for the Gröndals Bridge*

Two monitoring systems have been installed on the Gröndals Bridge, one traditional monitoring system using LVDTs (Linear Vertical Displacement Transducers), and one with Fibre Optic Sensors (FOS). The first is used for continuously monitoring and the second for periodic monitoring. The traditional system is installed for monitoring the long term effect of the crack development. The FOS system was installed for two purposes; first to monitor crack development and strain changes due to temperature and tram traffic, second to increase the practical experience by using FOS in field. In addition to this a comparison between the two systems have been made. The traditional system has been installed and followed up by the Royal Institute of Technology (KTH) in Stockholm, (James, 2004) and the FOS system has been installed and followed up by Luleå University of Technology in collaboration with City University in London, UK. (Täljsten & Hejll, 2004).

There are a total if six LVDT´s mounted on the Gröndals Bridge, one of which is a dummy used to verify the accuracy of the traditional monitoring system. With exception of the dummy and a sensor, which is positioned between the top flange and the web of the box girder, all the LVDT´s are positioned across and perpendicular to a crack so as to measure the opening and closing of a crack. The placement of the gauges can be seen in Figure 5. Four of the LVDT´s are mounted on the inside on the west web of the box girder, one of which is the dummy. One of the LVDT´s is mounted on the inside of the east web. The last of the LVDT´s is mounted between the top flange and the web of the box-girder and measures the relative displacement of the bridge deck to the web.

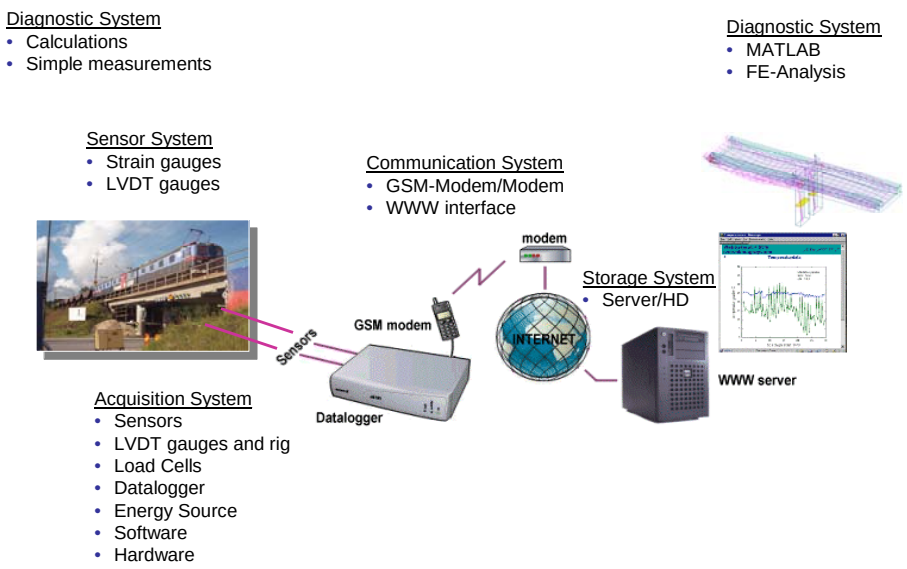

Figure 4 Structural Health Diagnostic System

In addition two temperature sensors, thermocouples, have been installed on the bridge and they are both positioned on the east web of the box girder, one on the inside and one on the outside, both belonging to the traditional system.

In total 32 FOS gauges have been installed on the Gröndals Bridge, they are all installed in section A and section B, see Figure 1 and Figure 5. In section A, 12 sensors have been installed on the west side where three were used to compensate for the temperature. Two of these sensors (including temperature compensation) are positioned on a section of CFRP laminates, the others on different locations of concrete cracks. On the east wall in the same section seven sensors are installed, five on the concrete (including temperature compensation) and two on a CFRP laminate also here including temperature compensation. In section B, west side, four sensors have been positioned on the concrete, one of which is for temperature compensation and two have been positioned on a CFRP laminate, where one is for temperature compensation. On the east side in the same section six sensors have been positioned on the concrete, one for temperature compensation and two on a CFRP laminate, of which one is for temperature compensation. The placement of all the gauges is shown in Figure 5.

As can be noticed in Figure 5 many gauges have been placed on the bridge. The filled rectangular blocks represent the LVDT´s and the open rectangular blocks represent the thermocouples for the traditional system. The FOS are represented by open circles. In table 2 the sensors are presented systematically. For the FOS system the previously mentioned sensors for temperature compensation shall also be included, however, this is not recorded in Table 2.

Table 2 Sensors for monitoring

Section A (and C)			
Sensor	*Measure*	*Sensor*	*Measure*
3372	Crack opening	GAW1-6	Crack opening
3374	Crack opening	GWR1-3	Strain Concrete
3376	Dummy	GAWC1	Strain CFRP
5/10G8	Displacement	GAE1-4	Crack opening
		GAEC1	Strain CFRP

Section B			
Sensor	*Measure*	*Sensor*	*Measure*
3379	Crack opening	GBW1-3	Crack opening
		GBWC1	Strain CFRP
		GBE1-4	Crack opening
		GBEC1	Strain CFRP

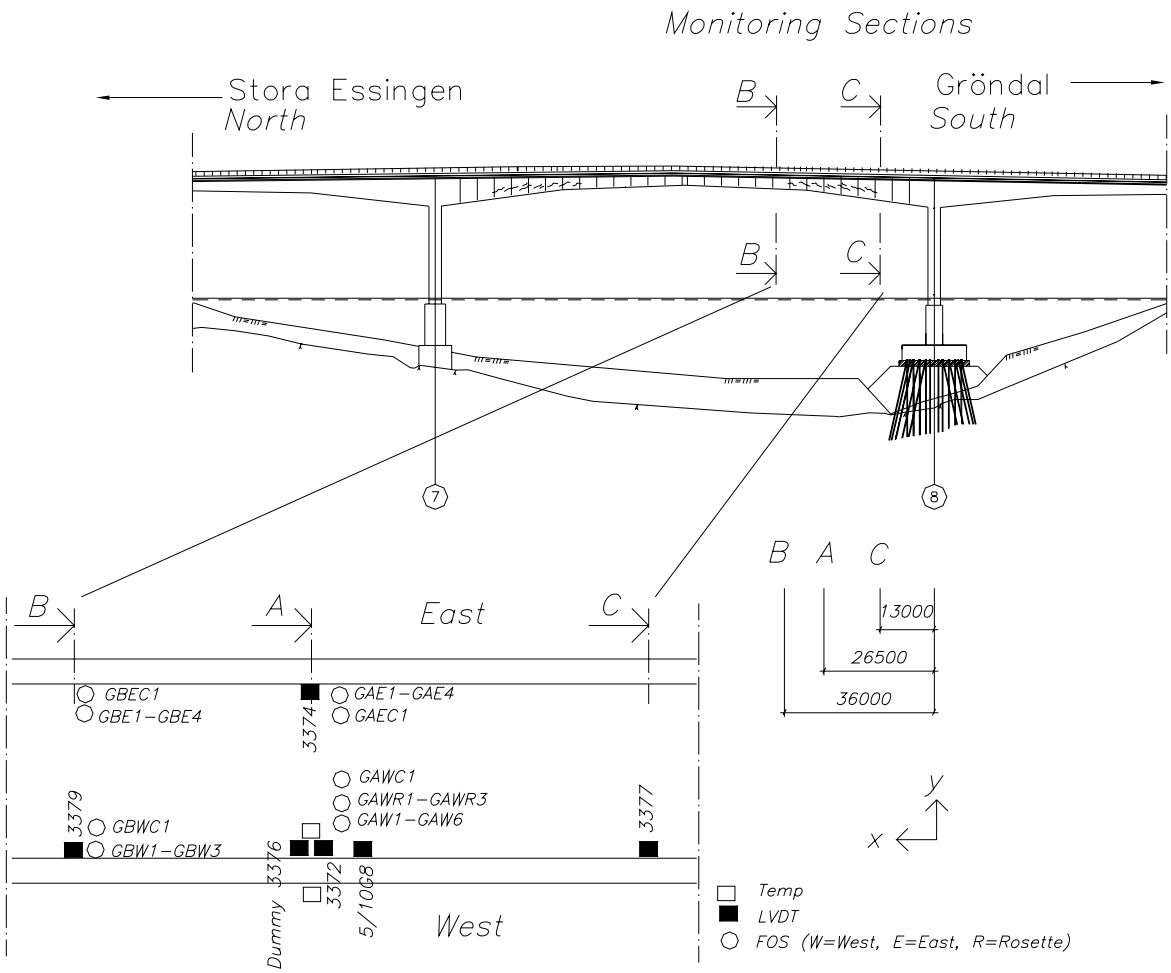

Figure 5 Position of gauges on the Gröndals Bridge

4 RESULT FROM MONITORING

It is not possible to present all the results from the measurements and therefore only the most interesting values are presented, however all data from the monitoring up to March 2004 may be found in James, 2004 and Täljsten & Hejll, 2004.

The results from the monitoring are presented for the traditional and FOS systems respectively.

4.1 *Results from the traditional monitoring system*

The result from the entire year 2003 for the Gröndals Bridge is shown in Figure 6, (James, 2004). It is not easy to extract a single value of the crack behaviour. However, from the figure it is possible to see that the cracks in the webs (indicated by sensors 3374, 3372, 3379 and 3377) do not appear to be of a progressive nature but rather that the cracks open and close depending on daily and seasonal temperature changes.

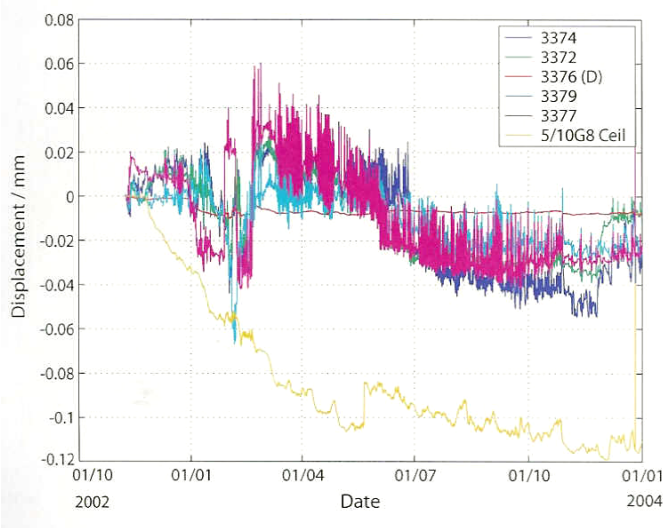

Figure 6 Result from crack-displacement with traditional monitoring system, from (James, 2004)

The crack widths of these gauges are not significantly different at the start of 2004 than at the start of 2003, with the variation in the order of 0.02-0.03 mm. The LVDT sensor (5/10G8) mounted between the underside of the top flange and the inside of the west web measures the longitudinal displacement in the top flange in relation to the web. The displacement appears from these results to be progressive even if the growth rate has decreased over the later half of the year.

4.2 *Results from FOS system*

Before we discuss the result from the FOS system, a brief discussion about the installation procedure and the SHD system used will be presented. Before installation calculation of stresses in the chosen sections had been carried out (Hallbjörn, 2002), however, these calculations also considered the dead load of the bridge. The monitoring systems installed can only follow the relative changes with regard to temperature and live load. The FOS system used has been developed at City University in London and the University also took part in the installation work, calibration and monitoring of data. The FOS sensors are of type Bragg grating and at most 7 sensors were written on one fibre, the length of each senor was approximately 20 mm. The total time for installing and calibrating all the FOS sensors was approximately 10 days, this corresponds with the time it took to grind the concrete and CFRP laminate and bond and protect the fibres. Monitoring was carried out during May 2003 and over a time period of 3 days (periodic monitoring). It was not possible at that time to carry out a continuous monitoring due to the high cost of the system. Nevertheless, to store the data a portable hard disk was used, which was then transported from the bridge to the office for further evaluation of the data. Not all data has been evaluated up to date and has been planned to repeat the monitoring sequence during the next winter. However, it was found that using this FOS system was convenient and the installation procedure was quite simple.

This paper will not present results from all sensors however, in figure 7 crack displacement from sensors GAW1 to GAW6 is shown. From Figure 7 it can be seen that crack displacement for GAW3 is considerably larger than for the others, here the FOS sensor is placed in the same crack as the LVDT gauge 3372, however during this time the crack displacement for the LVDT is approximately 0,035 mm (this can not be seen in Figure 6 due to the scale). The large variation over time is due to daily variation in temperature. In Figure 7 small jumps may also be noticed, approximately 0,001 mm wide.

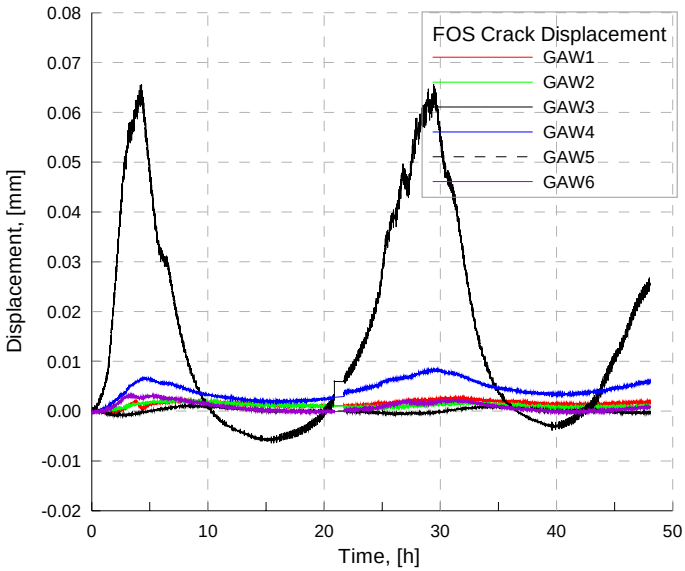

Figure 7 Result from crack-displacement with FOS system, (Täljsten & Hejll, 2004). Start date for monitoring 21-05-04

This is due to the tram traffic, it seems that the effect of the temperature on opening and closing of the cracks is tenfold to the traffic load.

Over the same crack as in Figure 7 a CFRP laminate has been bonded, in Figure 8 the strain is recorded in this laminate in the same position as the crack. A calculation of stress gives a level of approximately 30 MPa in the laminate.

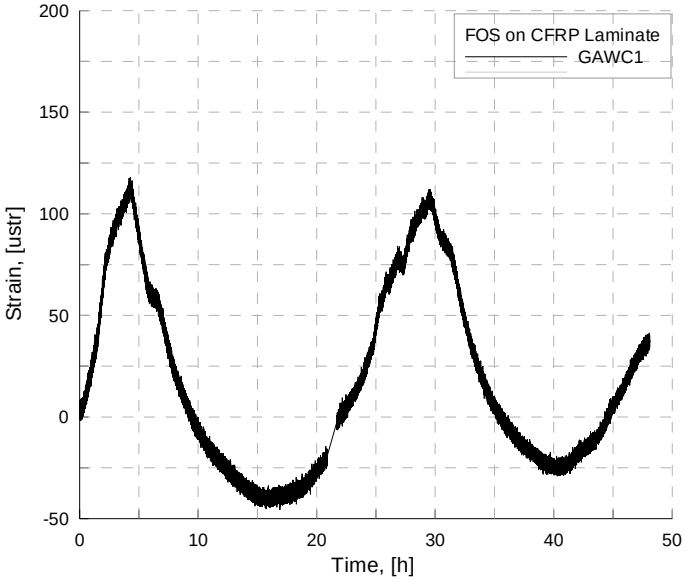

Figure 8 Result from crack-displacement with FOS system, (Täljsten & Hejll, 2004)

5 SUMMARY AND CONCLUSIONS

In this particular project the most innovating part was not to use CFRP laminates for strengthening, it was the way the design of strengthening was carried out. By using fracture mechanics in design and in combination with CFRP laminates it was possible to (theoretically) use the energy needed to open a crack to a certain size and transfer that energy to the CFRP laminate, which then redistributes the cracks and the distance between the cracks over the length of the laminate. Therefore a cost effective solution for strengthening in the SLS (also contributing to the ULS) could be chosen and finally carried out.

Both monitoring systems installed on the bridge showed that the cracks were not propagating and that the opening for the cracks was more or less negligible The largest crack opening measured was approximately 0,06 mm. Furthermore, the temperature effect was approximately 10 times larger than the effect from the tram traffic. Monitoring will continue and it will be interesting to follow the bridge over time.

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REFERENCES

Agarwal B.D. and Broutman L.J., 1990, Analysis and Performance of Fibre Composites, John Wiley & Sons Inc. ISBN 0-471-51152-8, 1990, p 449.

Bresson J., 1971, Novelles resherches et applications concernant lútilisation des collages dans les structures. Betong plaque, Annales de l+institute technique, No. 278, 1971.

Hallbjörn L, 2002, Sprickor I Gröndalsbron och Alviksbron I Stockholm, Betong No. 2, 2002, pp 8-12. (In Swedish).

Hejll, A Norling, 2002, O “Betongbalkar förstärkta med kolfiberkomposit: Dynamisk belasting under limmets härdningsförlopp” Luleå University of Technology, Division of structural engineering, ISSN:1402-1617

L´Hermite R., 1967, Lápplication des colles et resins dans la construction. La betong coffrage portant, Annales línstitut technique, No. 239, 1967.

James G., 2004, Long term monitoring of the Alvik and Gröndal Bridges, TRITA-BKN Rapport 76, Byggkonstruktion, 2004, www.byv.kth.se, p 48.

König G. 2002, Expert Opinion about the cracks in the webs of the Gröndals bridge, Sweden. Report prepared by König und Heunisch, Planungsgesellschaft mbH, p 48.

Meier U., 1987, Repair of bridges with high performance composite materials, Material + Technik, Vol. 15, No. 4, 1987, pp 125-128.

Sundquist H., 2002, Hur mår våra Frevorbau broar egentligen? Betong No. 1, 2002, pp 4-5 (In Swedish).

Täljsten B., 1994, Plate Bonding, Strengthening of Existing Concrete Structures with Epoxy Bonded Plates of Steel or Fibre reinforced Plastics Doctoral Thesis 1994:152D, Div. of Structural Engineering, Luleå University of Technology, ISSN 0348 - 8373, 308 pp.

Täljsten B., 1996, Strengthening of concrete prisms using the plate-bonding technique, International Journal of Fracture 82: 253-266, 1996, 1996 Kluwer Academic Publishers, Printed in the Netherlands.

Täljsten B., 1997, Strengthening of Beams by Plate Bonding, Journal of Materials in Civil Engineering, November 1997, pp. 206-212.

Täljsten, B. and Carolin, A., 1999, Strengthening of a concrete railway bridge in Luleå with carbon fibre reinforced polymers - CFRP: load bearing capacity before and after strengthening, Technical Report 1999:18, Luleå: Luleå University of Technology, Structural Engineering. 61 pp

Täljsten B., 2000, Förstärkning av befintliga betongkonstruktioner med kolfiberväv eller kolfiberlaminat, Dimensionering, material och utförande, Teknisk Rapport, Luleå tekniska universitet, Avdelningen för Konstruktionsteknik, 1999:12, ISSN 1402-1536, 1999, p 122 (In Swedish).

Täljsten B., (2001), "Design guidelines - a Scandinavian approach", International Conference on FRP Composites in Civil Engineering CICE 2001 Ed. J.G. Teng, Hong Kong, ISBN: 0-08-043945-4

Täljsten, B. 2003: FRP Strengthening of Existing Concrete Structures. Design Guideline Division of Structural Engineering, Luleå University of Technology, Luleå 2003, Second edition, 228 pp, ISBN 91-89580-03-6.

Täljsten B. and Hejll A., 2004, “Evaluation of the monitoring of the Gröndal and Alvik bridges”, Report to be published.