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USE OF FRP IN CONSTRUCTION IN SCANDINAVIA – EXPERIENCES AND A VERIFICATION TEST

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Abstract

In this paper a general overview of the use of FRP in Scandinavia and in particular Sweden is presented. A brief discussion abut guidelines in Europe open up the paper. Where different guidelines and where they are used is presented. However, focus in placed on FRP strengthening since the number of structures where FRP bars or fully FRP structures in Scandinavia is more or less zero and are not yet accepted by the authorities in new built structures – at least not for bridge building.

Nevertheless the acceptance for retrofitting and strengthening is large, event though scepticism regarding the function of the behaviour of strengthening structures still exists. Therefore the paper ends with a full scale example where a bridge has been strengthened with FRP plates and its strengthening effect has been verified by loading and monitoring.

Introduction

The structures may have to carry larger loads at a later date or fulfil new standards. In extreme cases, a structure may need repair due to an accident, or due to errors made during the design or construction phase, such that the structure needs to be strengthened before it can be used. Over the past decade, the issue of deteriorating infrastructure has become a topic of critical importance in Europe, and to an equal extent in North America and Japan.

The advantages of FRP-strengthening have been shown time and again during the last decade. All over the world several thousand structures retrofitted with FRPs exist. There are various reasons why the retrofit is needed, but since buildings and civil structures usually have a very long life, it is not uncommon that the demands on the structure change with time. There are several reasons for this; a structure may have to carry larger loads in the future or to comply with new standards than it was designed for. In extreme cases, a structure may have to be repaired or strengthened due to accidents. There are several methods for repairing or strengthening a concrete structure with FRPs; wrapping columns with sheets, bonding laminates to the outer face of a structure or even to place it in sawed grooves in the concrete cover as Near Surface Bonded Reinforcement.

Mostly do the strengthening systems use epoxy as the bond medium, but also systems with cementitious bonding agents exist. If the technique is to be used effectively, it requires a sound understanding of both the short-term and long-term behavior of the bonding system and the materials used. The execution of the bonding work is also of great importance in order to achieve a composite action between the adherents. Maybe the most important factor is a proper understanding of the design process. In this overview a short summary of the some guidelines developed in Europe will be discussed.

Guidelines in Europe

In this paper only guidelines that can be considered national or European documents will be covered. Work with guidelines for external strengthening of structures in Europe has been going on for quite a time. In 1996 a collaboration work within the framework of FIB (Fédération Internationale du Béton) started. The aim with this work was to compile a guideline in various fields for FRP in construction. In March 2001 the guideline [1] "Externally bonded FRP reinforcement for RC structures was printed". The aim with the guideline was to present a general description of materials and techniques related to the application of composites as external reinforcement of concrete elements. The guideline contains several chapters with each of them devoted to one particular aspect of strengthening, for example strengthening for flexural, shear, torsion and confinement as well as practical execution and quality control. This guideline is now updated and a new completely revised version will be printed in early 2008. In Sweden a guideline for external strengthening was first made for steel plate bonding in the end of the 80-ties and during the end of the 90-ties a design guideline for FRP plate bonding was written and incorporated in the Swedish Bridge Code: BRO 94. However, this guideline has since then been extended and improved. The guideline covers flexural, shear, torsion, fatigue, confinement and execution and quality control. The guideline also gives design examples [2]. A large part of this guideline has also been incorporated into the Norwegian guideline, with national adjustments, [3] for external strengthening, with FRPs. Also in the UK a design guideline for external strengthening exists [4]. This guideline covers design in flexural, shear and for confinement. It also discusses installation procedures and workmanship. However, this guideline is not as detailed as [1] and [2], but give a good understanding of external strengthening of concrete structures. In addition to this guideline a guideline for external strengthening of steel members using FRPs exists in the UK [5]. This guideline is very comprehensive and deals with many aspects regarding strengthening of steel structures. There also exists other national design guidelines of codes for external strengthening with FRPs in Europe, for example the Swiss [6], which partly covers strengthening with FRPs, but cover the use of structural adhesive extensively. Recently Italy also published a design manual for the use of FRP for retrofitting, this guideline has recently been published and adopt new findings of FRP retrofitting. In addition to the above mentioned guidelines, there also exists several company specific design manual, such as for SIKA, M.Brace and Sto. However, they are not considered as national guidelines and are supplier specific. In general it is the apprehension from the author that company specific manual should be avoided in design. However, guidelines is not further discussed in this paper instead focus on the use of FRP in Scandinavia and in particular in Sweden.

The use of FRP in Scandinavia

Even though the use of inernal FRP bars or fully FRP composite structures now is commodity in many parts of the world there has only been a very few applications in Scandinavia. The reason for this is not well known, but it might be explained with strong traditions in traditional building materials and that the demand for FRP structures is low. Another explanation could be that our decision makers are not very familiar with the possibilities that FRP materials offer. Of that reason it has been decided to focus this paper on the use of strengthening and retrofitting of concrete structures. Here FRP is today a full accepted method to repair and upgrade existing structures. In addition to this, since the author is Swedish, and since he has been involved in many of the carried out projects, the emphaizis is placed on FRP strengthening of structures in Sweden. It also need to be mentioned that Sweden is the country in Scandinavia where FRP is most mature and most used.

The first large application of FRP for strengthening in Scandinavia was the strengthening of a chimney in Halmstad, Sweden in 1997, see figure 1. In this particular case a CFRP sheet system was wrapped around the column, in total 3 000 m of sheets was used. An inspection 10 years after strengthening shows that the retrofitting works as intended and no deterioration effects could be noticed on the strengthening.

Three retrofitting systems is commonly used in Sweden and a forth is under development. All systems are based on CFRP. The first two, plates and sheets, are familiar to most of us. However, the third system is in general named NSMR or NSM which then stands for Near Surface Mounted Reinforcement. Compared to traditional external bonded FRP strengthening, NSMR systems have a number of advantages: the amount of site installation work may be reduced. Full-scale tests and field applications have shown that the pre-treatment when using plate bonding can be work intensive and therefore often costly. In traditional plate bonding the laitance layer must be removed and the aggregates shall be exposed before adhesive and the FRP can be applied. This to ensure satisfactory bond between the FRP and the structure.

In most cases this may be done by sandblasting and is neither complicated nor expensive. However, if the surface has irregularities, for instance from formwork, or if the grinding or more powerful surface may be necessary, which then may drift up the costs. For NSMR no surface treatment is needed, except sawing and cleaning the slots. Compared to laminates or sheets the NSMR bars can also easily be prestressed. In addition fire, vandalism and environmental loads may also harm the FRP. A damage of the external bonded reinforcement can give serious problems and eventual failure of the strengthening component. The use of NSMR can avoid these situations and the FRP will be more protected from outer damages. However, the largest advantage of NSMR can probably be found in increased force transfer compared to external bonded FRP. In figure 2 an example where NSMR has been used on a road bridge is presented. In Sweden the NSMR technique was introduced already in 1996 with the first field application in 1999. The field application presented here was carried out during the fall of 1999, see figure 2.

The reason for strengthening was a mistake at the construction site. The amount of steel reinforcement needed in a joint between a pre-cast concrete element and in-situ cast concrete was not sufficient and strengthening was demanded. The reason for choosing NSMR for this application was the resistance to corrosion of CFRP laminates and comparable stiffness and strength to that of steel. The cross section of the laminates used was 5 x 35 mm, with a Young´s modulus of 160 GPa and an ultimate strain at failure of 1.6 %. Furthermore, the main advantage of the NSMR was that the laminates could be placed in the concrete cover and that no great work effort was needed.

Common for the three above mentioned systems is that they all use epoxy as the bonding agent.

The forth system, that currently is under development is the use of a special developed grid systems with a cementitious bonding agent, polymer mortar, to replace the epoxy. Here the mortar is first placed to the base concrete, the CFRP grid is placed in the mortar and finally additional layer of mortar is applied. The systems is named MBC (Mineral Based Composites). In figure 3 a beams strengthened in shear loaded to failure with this system is shown. In comparisons the same load carrying capacity as corresponding CFRP sheets bonded with epoxy designed for the same ultimate load, can be obtained, [7].

However, it Sweden it has raised concern regarding the function of the CFRP systems in the field. Of that reason several full scale tests have been performed. This has also lead to well developed monitoring systems to follow up the structure over time. In the next chapter a verification test of a concrete road bridge strengthened with CFRP plates will be discussed. The tests are carried out both before and after strengthening. Here we also have tried to use the local and global curvature of the structure to estimate the stiffness change.

Case study – the Panken road bridge

Here the use of SHM to verify the strengthening effect of CFRP plates before and after strengthening is presented. A road concrete beam bridge, located in the centre of Sweden, needed to be strengthened due to severe cracking, which depended on a slight deficiency in load carrying capacity. The bridge was strengthened in the ultimate limit state (ULS) but the strengthening was also aimed to contribute to the service limit state (SLS), with less deflection and opening of existing bending/shear cracks. The bridge was strengthened in the most loaded section with approximately 150 kNm corresponding to almost a 10 % increase in flexural capacity. Moderate loading in SLS showed a 15 % decrease in strains measured on both steel reinforcement and concrete and also that the crack sizes decreased correspondingly. In addition, an idea how estimate the performance of a bridge over time is presented by introducing a performance factor. This factor considers the stiffness change of a structure over time or due to any extreme measure.

The Panken bridge is a road bridge in six spans with a total length of 117.0 m, the end spans have a length of 13.5 m and all other spans of 21.0 m, the bridge shown in figure 4, is a RC beam bridge with a nominal concrete strength of 40 MPa and a nominal steel yield strength of 600 MPa. The bridge deck has a width of 13.0 m divided on two traffic lanes, each 6.5m wide.

The bridge was built in 1977 and the first cracks in the webs was registered in 1994. To find out the reasons for cracking a detailed investigation was carried out in 2001, first mapping all cracks in every span and then a thorough design calculation was conducted. The calculation reviled that the bridge needed to be strengthened by approximately 10-15 % in flexure, which in this particular case corresponded in the most strengthened section to four CFRP plates. Here Sto FRP Plate S10C with dimension 1.4 x 50 mm and with a stiffness and elongation at failure of 170 GPa and 15 ‰ respectively was used. In total 900m of CFRP plates were bonded to the sandblasted and primed (primer Sto BPE Primer 50 Super) surface with a two component epoxy adhesive (Sto BPE Lim 567). The adhesive has a stiffness of 6.5 GPa.

The temperature during strengthening and hardening of the adhesive was at minimum 16 °C and as maximum 26 °C. Since earlier tests by [8] has shown that the strengthening effect is not affected due to traffic loads the, traffic were allowed on the bridge but the speed was decreased from 90 km/h to 50 km/h to minimize the effect of "bumps & jumps" when larger vehicles were passing the bridge. Photos of the bridge before strengthening and after strengthening are shown in figure 5

Verification of strengthening by monitoring

The monitoring of the Panken Bridge is going to be carried out during a time period of one year. Three periodic measurements are planned, where two already have been carried out. The first measurement was done before strengthening in the beginning of the summer 2006, the second, was carried out after strengthening in September 2006 and, the third is going to be carried out during 2007. The two first measurements are reported in this paper.

After the definition of the goals one cross section in one of the mid-spans was chosen. Two out of three girders where instrumented. From top to bottom the following variables where measured on the north side, compression strain in the upper steel reinforcement in the bridge deck, compression or tension at the concrete surface just below the upper flange and the tension strain in the lover steel reinforcement. Similar measurements were carried out on the south side with the exception that no steel strain was measured in the bridge deck. Due to the fact that several tension concrete cracks where found in the measured cross section in the web. Some of the most severe cracks where instrumented with crack opening displacement sensors. Together with the strain and crack measurement points two deflection points of interest where chosen, one to investigate the global deflection and one to monitor the average curvature on this span by mounting the deflection sensor on a stiff bar with a length of 4 meters. After strengthening the strain level in the CFRP is also of interest. The position of the measuring cross section is showed in figure 6. In table 1 the naming and the placement of the sensors are shown.

During the controlled loading no traffic was allowed on the bridge. A number load tests were performed when the truck was placed on the north or south side of the bridge. Also dynamic testing when the truck was travelling with different speeds was investigated. In addition to this also measurements of the traffic was carried out where the trigger was set on larger trucks weighing over 25 tons. However, in this paper only tests where the truck was placed above the north beam in the section shown in figure 7 is presented.

For the presented results the trucks where placed with the rear bogies on top of the monitored cross section, see figure 7. The loads from the truck are presented in table 2. All the results are normalised according to the heaviest truck which was used for the load test after CFRP strengthening. All the results before strengthening where hence multiplied with a factor of 1,0444. The maximum bending moment from the truck in the centre of the monitored span is calculated to 770 kNm. The monitoring results from the north girder are presented before and after the strengthening. The overall results show that the strain level in the tensile reinforcement decreased with approximately 25% and the strain the compressed steel bar with approximately 5 %. The concrete sensor was not very stable for the measurement before strengthening and a correct comparison is not possible. Even more interesting is the results from the tension field where the compressed zone increased with 130 mm, see figure 7 right.

The non-linear behaviour tension field after the strengthening is caused by the initial slip in the adhesive and is normal for non-prestressed strengthening. The results from the deflection measurement showed a decrease in deflection with 9 %, from 1,752 mm to 1,595 mm. The most interesting results are the comparison before and after strengthening of the crack opening. The cracks showed an opening before strengthening of maximum 0,0288 mm and after the strengthening the cracks opened only 0,0195 mm, a decrease of crack opening with 32 %. Before the test was carried out an idea how to use the curvature for structural assessment was discussed. The result from the curvature measurement is not unambiguous but the results shows that with some further development this technique might work, see figure 9 to 11.

If the global curvature change of a structure that is strengthened or deteriorated, could be measured accurately over time it would be quite simple to measure stiffness changes. A simple engineering method to calculate the bending stiffness is to use the curvature by the expression:

$$\kappa = \frac{M}{EI}$$

where M is the bending moment, E the Young's modulus and I the moment of inertia respectively. By calculating κ for known moments, EI can be achieved for a certain load in time. The curvature can also be achieved from the strain measurements over the cross section. The curvature obtained from the strain measurements can then be compared with the curvature from the deflection measurements. Two deflection measurements were made during each load test, one with the total deflection and one with deflection over six meter length section of the bridge. This was made by attaching a LVDT sensor mounted to a stiff aluminium ruler fasten to the bridge. The curvature can then be calculated from the geometrical relation between the radius and the deflection. By using the deflection before and after strengthening, and the strain in the upper and lower reinforcement before and after strengthening the curvature was calculated and recorded in Table 4.

The results from the deflection and strain curvature measurements was promising, The stiffness calculation before and after differed somewhat however. The correspondence of the stiffness between the local and global curvature was better after then before strengthening.

The stiffness increase was approximately 50 % calculated from strain measurements compared to approximately 25 % for the global measurements. Theoretical calculations have given approximately 25 % stiffness increase after strengthening.

Discussion and conclusions

The use of FRP for retrofitting has gain great acceptance in Sweden, the utilisation of the strengthening systems is also taking off the neighbouring Scandinavian countries. In Sweden, Norway and Finland national guidelines exists. However, new built applications such as FRP internal reinforcement and fully composite structures are still very few. In addition to this doubtfulness regarding the strengthening effect has been raised by the autorites, of that reason demonstration projects and verification tests are very important. One such test, where a concrete highway bridge has been strengthened with externally bonded CFRP plates is presented in the paper. The purpose with the strengthening was to increase the flexural capacity and to minimise opening of existing cracks. In one section of the bridge monitoring of crack opening deflection, curvature and steel and concrete strain were measured before and after strengthening. After strengthening also the strain in the CFRP plate was measured. It was shown that the strengthening worked as interned and that the strengthening effect search for was obtained.

Acknowledgement

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Figure 1: CFRP strengthening of a chimney in Sweden.
The chimney was painted after strengthening

Figure 2: First NSMR field application in Sweden

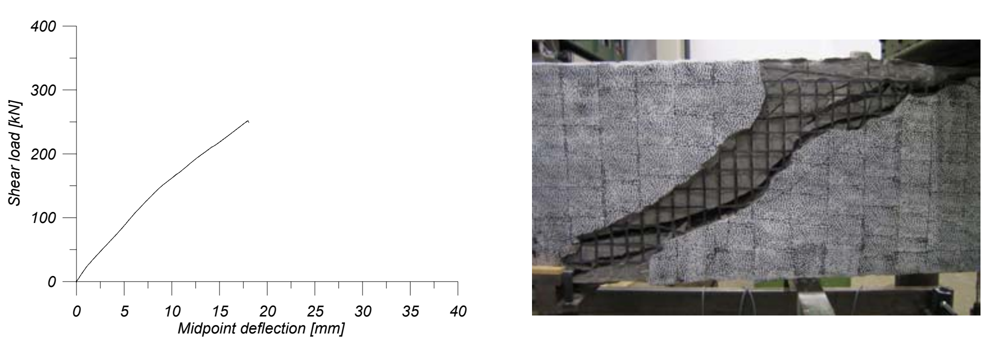

Figure 3 Concrete beams strengthen for shear with the developed MBC system. Beam loaded up to failure, see also [7]

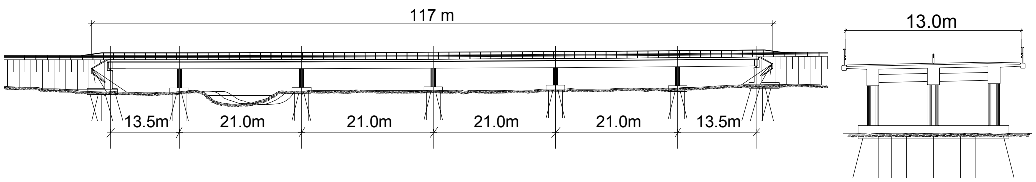

Figure 4: The Panken road bridge

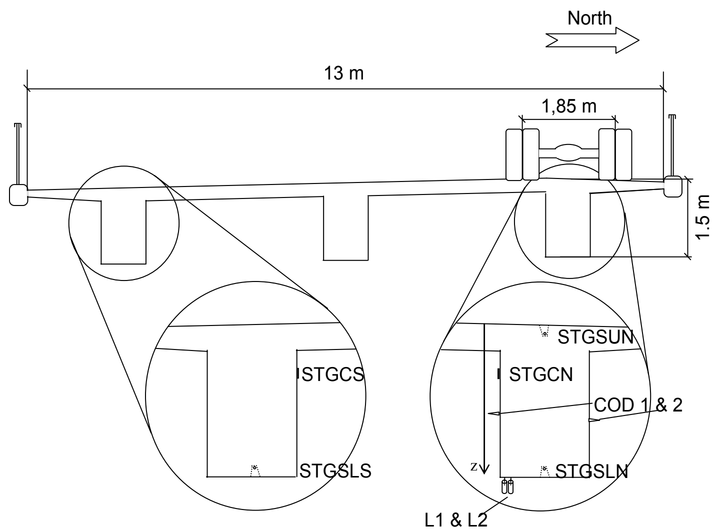

Figure 6: Placement of sensors

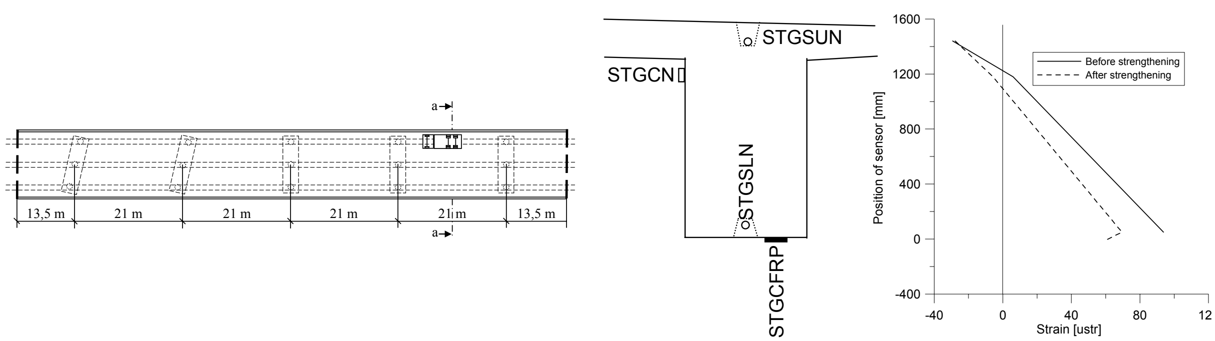

Figure 7: Placement of the truck for the static load test, left, and strain field in the north beam, right

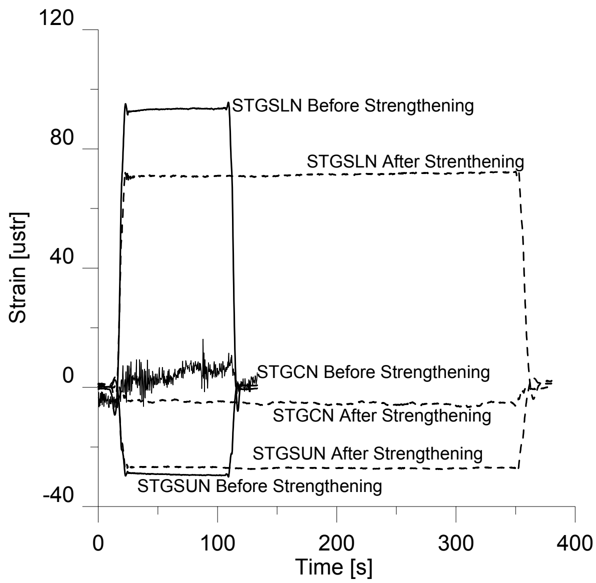

Figure 9 Strain sensors in the north girder

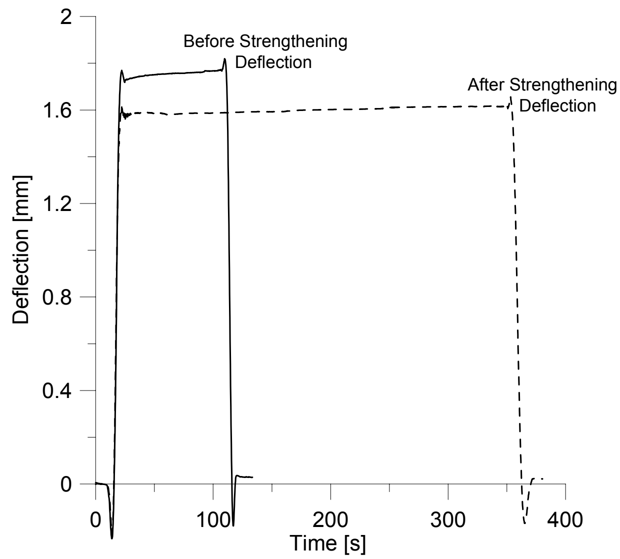

Figure 10. Deflection in the midpoint of the monitored span, before and after strengthening

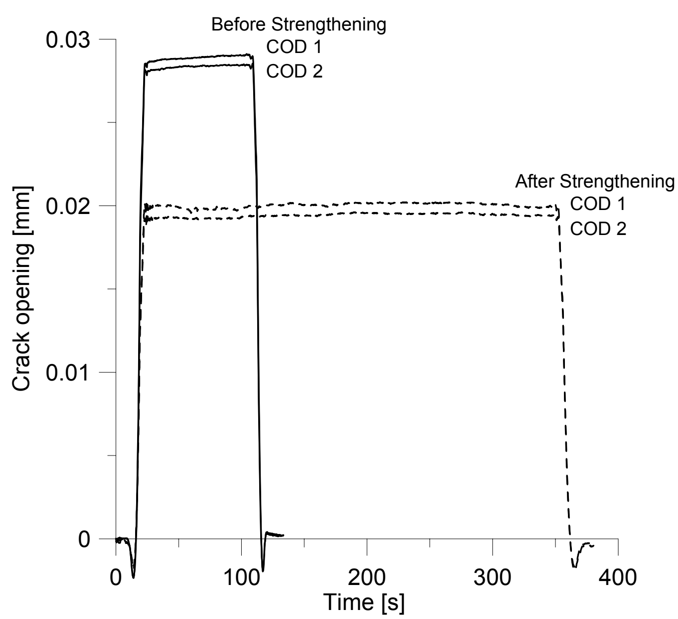

Figure 11. Crack opening on both sides of the north girder before and after strengthening.

Table 1: Sensors notification from figure 6.

Sensor	Distance	Girder	Notation
Concrete strain gauge	345 mm	South	STGCS
Lower steel strain gauge	1450 mm	South	STGSLS
Upper steel strain gauge	60 mm	North	STGSUN
Concrete strain gauge	320 mm	North	STGCN
Lower steel strain gauge	1450 mm	South	STGSLN
Crack sensor	815 mm	North inside	COD1
Crack sensor	860 mm	North outside	COD2
Deflection	0	North	L1
Curvature	0	North	L2

Table 2: Actual loads from the load tests

Before Strengthening			Date:	2006-07-03
Total Load: 25 200 kg	Front load: 7 200 kg	Rear load: 18 000 kg	Temperature: 26.2 ºC	
After Strengthening			Date:	2006-09-04
Total Load: 26 320 kg	Front load: 6 700 kg	Rear load: 19 300 kg	Temperature: 14.4 C	

Table 4: Results from curvature measurements and calculations

	Before strengthening	After strengthening		Before strengthening	After strengthening
Calculation of the curvature from the strains measurements			Calculation of EI from the bending moment and the curvature		
ε_c	-29,24 ustr	-27,8 ustr	M	770 kNm	770 kNm
ε_t	93,79 ustr	69,5 ustr	κ_δ	4,643E-05 m^{-1}	3,00E-05 m^{-1}
z_{tot}	1,39 m	1,39 m	L	21 m	21 m
κ_ε	4,643E-05 m^{-1}	3,000E-05 m^{-1}	[illegible]	0,00175	0,001596
EI	1,66E+10	2,57E+10	EI	2,43E+10	2,66E+10