PRETENSIONED CARBON FIBER WIRES FOR A ROAD BRIDGE, EXPERIENCES IN BELGIUM

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ABSTACT

The paper reports the design, fabrication and assessment of two pre-cast girders for a road bridge crossing the Belgian high speed track, loaded according to the Eurocode load models and pre-tensioned by carbon fiber wire tendons. One girder in each side span of the bridge is pretensioned with carbon fiber tendons and is designed to have equivalent strength as the other beams, prestressed with conventional steel strands. The experiment has enabled to detect several difficulties in the use of these non-metallic tendons, especially concerning the pre-tensioning process and its effectiveness. The possible backgrounds and reasons for these difficulties are explained, taking into account the fact that a qualified precast concrete manufacturer fabricates the girders without any laboratory intervention. This experiment has proven that although the eventual performance of carbon fiber prestress is satisfactory, the difficulties during construction tend to temper the optimism concerning the use of these non metallic tendons.

Keywords: Pre Cast Bridge Girders, Carbon Fiber Wires, Road Bridges, Strain Gauge Measurements

INTRODUCTION

Although the application of fiber reinforced polymer (FRP) in general and carbon fiber reinforced polymer have (CFPR) have been used often in a wide variety of engineering applications, the use of this material in civil engineering is still rather limited. Its well-known abilities of low weight, high strength and durability make this material very interesting for several applications in the above-mentioned field. The most important difference between applications in e.g. aerospace industry, sports equipment,… is the conditions of manufacturing. Whereas the first applications are fabricated in practically laboratory conditions with sensitive equipment in a clean environment, the working conditions for civil engineering projects are not that favorable. The environment is either a construction workshop or an outdoor construction site. Both environments suffer form large changes in temperature and humidity and are generally not very clean. In addition the personnel is familiar with the heavy construction equipment such as cranes, drills and shovels rather than with the fine equipment necessary for the application of FRP materials. For these reasons, although the application of RFP for construction is promising in laboratory conditions, the results can be disappointing in the field.
One application which can be tested is the use of CFRP non-metallic tendons for prestressing a precast bridge girder. The prefabrication workshop conditions are at least far better than on any construction site. In addition the normal economic time pressure (excluded in the laboratory) is present. In this first application in Belgium of this material in the field, two girders of a multiple precast girder and slab road bridge are chosen. They are part of a 3 span bridge totaling 205' (62.5 m) crossing the high-speed train track between Brussels and Cologne. The girders are designed to have equivalent strength as their traditional counterparts, equipped with normal steel pre-stressing tendons. The design is not entirely corresponding since the CFRP prestressing requires a strictly horizontal alignment whereas the steel prestressing tendons are bent upwards near the supports. In order to monitor the behavior of both girders with respect to the conventional ones, strain gauge measurements are carried out during the pre-tensioning operation, during the wire cutting and during field testing with a truck. The results indicated not only that all girders perform equally but also reveal critical points in the construction phases and possible reasons for failure.

THE CFRP WIRES

PROPERTIES OF THE CFRP WIRES

The 2" (5 mm) CFRP wires are composed of carbon fibers embedded in an epoxy resin matrix. The fibers take about 60 to 70 % of the volume of the wires. Whereas the tensile stress of the individual fibers is about 710000 psi (4900 MPa), the tensile strength of the composite wires is 355000 psi (2450 MPa). While the fibers take practically all the tensile stress, the resin matrix regulates the stress transfer between the CFRP wires and the surrounding concrete. In this experiment the girders are prestressed by pretensioning the wires used as adhesive tendons. For this reason the bond strength at the wire – concrete interface needs to be enhanced substantially. This is done by an adhesive sand coating on the wire surface installed directly after the pultrusion of the wire itself. As will be explained later, this sand coating is essential for a good anchoring capacity and therefore critical in the design. The sand coating should cover the whole perimeter of the wire, which is not easily achieved. A good quality control and inspection upon acceptance of the wires is therefore mandatory. When manufactured properly, the CFRP wires with sand finishing indicate a transfer length of only 3” to 5” (8 to 10 cm), in laboratory tests as well as in practice. Of course this is only an indicative value, since the transfer length is also determined by several other factors, such as the stress level, the concrete grade and the overall and detail geometry.

The most important characteristics of the CFRP wires are given in table 1 juxtaposed to these of conventional pre-stressing wires of equal diameter. Clearly the unit weight and the ultimate strength indicate favorable values. Furthermore, the wires show far better fatigue behavior, very low stress relaxation and negligible creep.

Table 1: Characteristics of CFRP wires juxtaposed to conventional steel wires

	CFRP	**Steel**
diameter [inch (mm)]	0.2 (5)	0.2 (5)
specific mass [pcf (kg/m³)]	96.7 (1550)	483.8 (7750)
tensile strength [psi (MPa)]	355000 (2450)	270000 (1860)
breaking load [kips (kN)]	10.8 (48)	8.2 (36.5)
Young’s modulus [psi (MPa)]	23.10^6 (160.10^3)	30.10^6 (210.10^3)
elongation at point of rupture [%]	1.5	3.5
Poisson’s ratio [-]	0.3	0.3

In addition to these properties, other advantages of CFRP as compared to conventional steel wires are a very good resistance to electrolytic, atmospheric and chemical attack in both alkaline and acidic environment and the ability to remain unaffected by electromagnetic interference. Especially the corrosion problem for steel seems to be solved by applying this CFRP material.

However it must not be forgotten that due to the unidirectional natural of the fiber arrangement within the composite structure, the CFRP wires have little or no transverse strength. Special arrangements will have to be made in order to be able to achieve full longitudinal tensile strength without laterally crushing the wires in the anchoring device or losing grip due to inefficient anchoring. Certainly conventional wedge shaped anchoring devices cannot be used. A possible solution for this problem will be proposed in the following paragraph.

Without addressing economic considerations – at present the price of the CFRP wires is still several times higher than the price of conventional prestressing steel wires – this material has certainly the potential of replacing conventional steel prestressing wires, at least in certain applications. As stated furtheron, the experiment has revealed some difficulties concerning the application of the material in real-time conditions, which will have to be dealt before the application of the material will expand beyond the laboratory phase.

ANCHORING AND PRE TENSIONING

In this experiment the key issue has proven to be how to harness the available tensile strength of the wires. Conventional wedge shape anchoring devices for steel wires cannot be used with composite wires due to the material's low lateral properties, including the shear strength. The manufacturer of the CFRP wires has proposed the use of the steel sleeve anchoring displayed in figure 1 for the active (tensioning) side. The sleeve with a diameter of 0.8" (20 mm) and a length of 9.8" (250 mm) has given maximum allowable tensile stress values of up to 90% of the ultimate strength of the cable, at least in laboratory conditions.

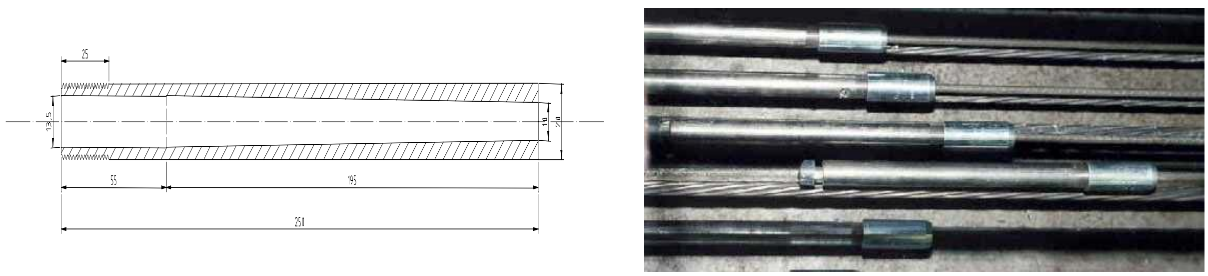


Fig. 1 Active Side Anchoring

The sleeve has a conical cavity, covered with Teflon on the inside. The CFRP wire is placed inside this sleeve and the remaining cavity is filled with a two component epoxy resin. At the wide end of the sleeve, a thread is used to couple this device with a conventional steel prestressing cable on which the actual jacking operation takes place. This is done by a conventional three piece wedge system. A factor of major importance is the use of steel wires in stead of steel strands for this purpose, since these last tend to twist during tensioning. This twist may cause severe damage including rupture of the CFRP wires, as, due to the low lateral strength, there is practically no torsional resistance. A second important feature of this type of anchoring is the Teflon layer on the inside of the sleeve. It causes the epoxy wedge to slide a little through the cavity while tensioning, thus applying rather small lateral force on the wire. This force has a positive effect on the slip of the wire, but is small enough not to crush it laterally.

Laboratory tests provided by the manufacturer of these sleeves indicated on average a failure in the anchoring at a tensile load close to the theoretical ultimate load of the wires. These values are however a result of laboratory tests under conditions not comparable to the prestress workshop conditions of the experiment. The workshop results are in fact quite inferior to those obtained in the lab, as will be pointed out later in this document.

On the passive side a 1 m long concrete block provide the necessary anchoring length. All the tendons are embedded in this block. Of course this length is taken several times the theoretical transfer length, for practical and supplementary safety reasons.

DESIGN OF THE EXPERIMENTAL PRECAST BRIDGE GIRDERS

SITUATING THE PROJECT

The experimental CFRP girders are part of a 205‘ (62.5 m) precast girder and slab road bridge crossing the high speed track from Brussels to Cologne and a local line at Kortenberg near the Brussels airport. The bridge has 3 consecutive spans of 62’ (18.9 m), 85’4”(26 m) and 57’9” (17.6 m) and the total width is 18’10” (5.75 m). This includes a 9’10” (3 m) carriageway, 2 1’10” (0,55 m) bicycle/foot paths and heavy safety barriers. It consists of 4 pre-cast inverted T beams in each span. One of both central beams in each of the side spans is prestressed with CFRP wires. All other beams are conventionally prestressed with steel strands. By choosing this configuration, any possible damage or need for repair has effect on the road traffic over the bridge only. All four train tracks remain unaffected. The precast beams are 4’1” (1.25 m) to 4”3” (1.3 m) high and are finished with a 9” (23 cm) concrete deckplate and an asphalt wearing course. Figure 2 displays the bridge.

Fig. 2 Project Situation

GIRDER DESIGN

Essentially both CFRP prestressed girders are designed to have equivalent strength as their conventional counterparts. The design load is taken from the European standard design code : “Eurocode 1.3 – Loads and Actions on Bridges” [1]. For this type of bridge the Load Model 1 is determining. This model consists of distributed load of 1.3 psi (9 kN/m²) over the 3 m lane and 2 axle loads of 67 kips (300 kN). As compared to many other codes including the Belgian national code the model is quite severe. The transverse distribution of the concentrated loads over the different girders is determined using the Guyon Massonet method [2]. This is an analytical method based on the general plate theory, used widely for this purpose in Europe. The method gives fast and sufficiently reliable results for this type of bridge and depends only on the relative characteristics of the longitudinal and transverse stiffnesses of the bridge and not on the absolute values. The method is therefore especially useful in the preliminary design since the final stiffness values do not have to be known. Using this method, the prestress values of table 2 are found. The final girder design for both conventional and experimental girders is given in figure 3.

Table 2 : Prestress design values

	CFRP	**Steel**
type of wires / strands [-]	wires ϕ 0.2”	T12 (ϕ ½”)
number of tendons [-]	126	40
total prestress force [kips (kN)]	663 (2948)	1079 (4801)
eccentricity [” (mm)]	13.7 (348.5)	9.6 (244)

Figure 3 indicates clearly the density of the CFRP wires within the lower flange of the primary girder. The necessary prestress force requires 126 CFRP wires and this leads to a narrow spacing of approximately 0.9” (23 mm) between individual wires, horizontally as well as vertically. In addition, 8 heavy steel reinforcement bars of ϕ 1.25” (32 mm) are provided along the full length of the girder. This passive reinforcement serves as an additional safety for this experimental design. They can carry the full proper weight of the bridge, even if all CFRP wires fail.

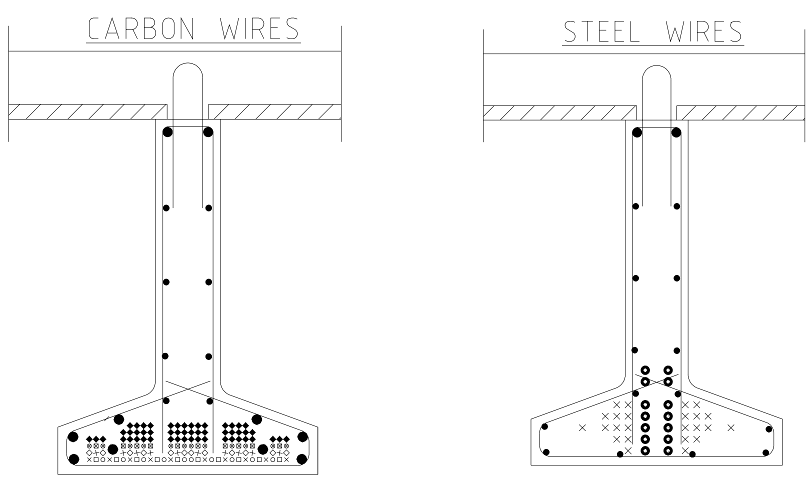


Fig. 3 Primary girder prestress and passive reinforcement location

The total prestress force of 663 kips in the CFRP girders results from a 5.3 kips (23.4 kN) force in each of the 126 horizontally aligned wires. This value is created by a 5.9 kips (26 kN) tension force applied by the jacking device, taking into account an instanteneous loss of only 10%. This instanteneous loss value is considerably lower then normal but is justified by the excellent creep and relaxation values given for this material and the considerable length of the wires, reducing the loss by slip in the anchoring devices. The total length of each cable is about 196’10” (60 m) since both girders are in line on one prestressing bench in the workshop. Both the high density of the sand coated CFRP wires and the heavy passive reinforcements are displayed in figure 4.

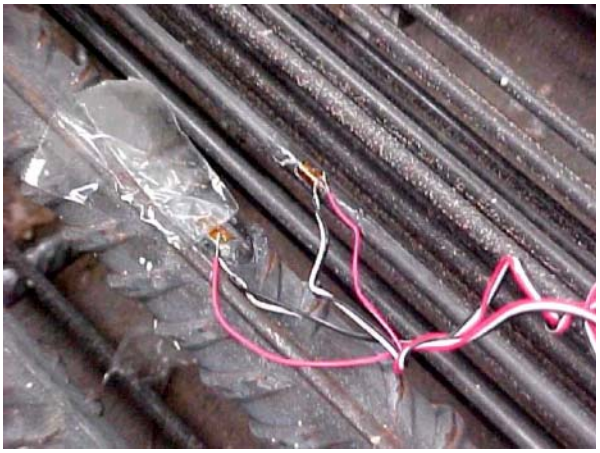


Fig. 4 CFRP wires and passive reinforcement

Whereas a part of the conventional prestressing strands are bent up near the supports, where the induced upward bending moment is unnecessary and even not acceptable, this cannot be done with the CFRP wires. Indeed the low transverse strength of the unidirectional fibers would cause lateral crushing of the cable at the bending devices due to the immediate change of slope resulting in a large local upward shear force. Due to this unfortunate property, no shear favorable alignment can be obtained with pretensioned CFRP wires. To avoid the unfavorable upward bending moment near the supports due to the prestress 74 out of the 124 wires need to be encapsulated in hollow plastic tubes. As the number of individual wires is large and this encapsulation has to be executed manually, this is a very costly solution, which needs to be improved.

GIRDER PREFABRICATION

PRELIMINARY TESTS

Based on laboratory tests provided by the manufacturer of the wires, the advised design load of the carbon fiber wires is 80 % of the breaking load of 10.8 kips (48 kN). This would imply a design load of 8.7 kips (38,5 kN) per wire. As stated in the previous paragraph the applied load was only 26 kN. This was the result of 12 tests performed in the prefabrication workshop on the wires and the anchoring system, prior to the actual manufacturing of the girders. The test was carried out, using the previously mentioned anchoring procedure, in accordance with the manufacturers guidelines. In 9 cases, the breaking force exceeded 10.1 kips (45 kN). The other wires showed an ultimate force at rupture of 6.4 kips (28.6 kN), 7.6 kips (34.4 kN) and 8.5 kips (38 kN) respectively. In each case the carbon wire anchoring system had failed pulling the wire out of the sleeve. After the initial slip in the anchoring the wire is pulled out of the sleeve causing a shock wave in the wire. It is this shock waves that causes the final rupture of the wire as seen in figure 5. This is most probably caused by 2 reasons. Firstly the sand coating of the wires was in some cases not equally distributed along the perimeter of the wire. This may have caused a reduction in adhesive surface. Secondly the conditions of application, and the experience of the personnel for the sealing up of the anchoring sleeve did not match the laboratory conditions of the manufacturers tests. Temperature, humidity and general surface preparation are essential to the application of an epoxy based sealing in order to create full adhesion to the wire and sleeve surfaces. At the passive side no failure occurred, neither in the contact area, neither in the concrete.

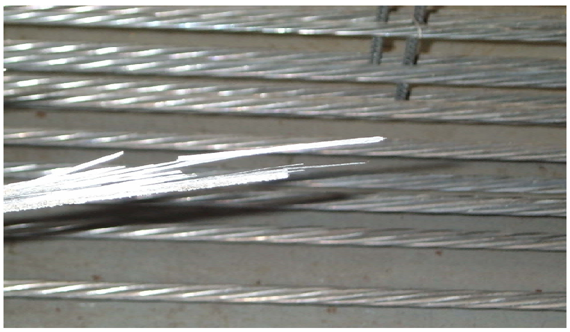
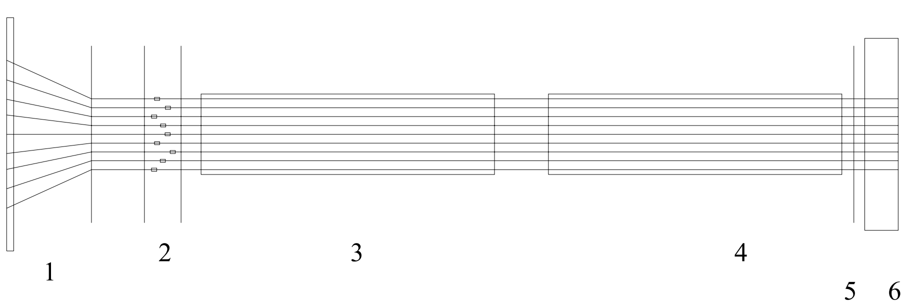


Fig. 5 CFRP wire rupture due to shock wave after initial slip

After these poor results, the applied tensioning force was reduced to 5.8 kips (26 kN), only about 55 % of the ultimate tensile strength of the wires. One can hardly call this an efficient use of the CFRP. Certainly more attention needs to be given in order to transfer the laboratory results into standard workshop conditions or even maybe construction site conditions. It's important to point out that this problem not only persists, but also grows stronger when using larger wire diameters. As the relation of wire surface to wire circumference enlarges, the shear force along the circumference increases, hence increasing the bond problem.

FABRICATION PROCEDURE

As stated above both primary CFRP girders are manufactured in line on one prestressing bench. Figure 6 gives an impression of the fabrication setup.



1. Jacking device
2. Coupling Sleeves
3. Girder 2
4. Girder 10
5. Wooden endplates
6. Passive anchoring

Fig. 6 Girder Fabrication set-up

This has proven to be an economic solution with a considerable decrease in actions, man-hours and accessories and results in a relative decrease in pre-stress loss due to creep. The fabrication of the girders required a specific sequence for the different actions. The consecutive steps are described briefly in the following paragraphs.

1) Installation of assistant strands and shear force reinforcement: 4 steel assistant strands are slightly stressed, and the shear force reinforcement stirrups are attached to them.

2) Installation of CFRP wires: the ϕ 0.2" wires are delivered on rolls of 8'2" (2.5 m) diameter. They are cut and put through holes in the wooden endplates. The endplates, usually made of steel, are in wood with rounded edges, to minimize the risk of damage while tensioning. Since the alignment cannot be made perfect, any lateral pressure at these plates might cause risk of wire crushing when metal would be used. The relatively soft wooden plates create no risk for this type of failure. During this operation the necessary hollow tubes near the support are put in place as well.

3) Installation of the active anchoring: individual attachment of the CFRP wires to the steel coupling sleeves. Each of the 126 CFRP wires is fastened into the hollow steel sleeves. For this purpose, the sleeves are positioned vertically, allowing the air to get out, thus improving the strength of the connection. As stated previously, this is the key moment in the fabrication of the girders and requires predominant attention. After curing, the sleeves are meticulously arranged horizontally and the steel wires are connected. While prestressing, the wires and sleeves should not become entangled and, in ideal circumstances, they shouldn't even touch one another. As the number of individual wires is extremely large and the spacing is very narrow this also needs attention. A well-prepared prestress procedure needs to be followed to achieve no entanglement.

4) Casting of the passive anchoring system: at the passive side, the CFRP wires are cast into a small beam using a high strength, shrink-free mortar. The dimensions of this beam are determined by transfer length and spalling forces.

5) Pre-tensioning the CFRP wires to 20 % of the final value: by means of a traditional pre-tensioning jack a value of 1.2 kips (5.2 kN) per individual wire is applied.

6) Positioning of the passive reinforcement safety bars: the wires having been placed in their final position, the passive bars can be positioned. At this time, strain gauges are attached to both the passive bars and to the CFRP wires in order make an assessment of the beam performance.

7) Pre-tensioning to 100 % of the calculated tensile force: for safety reasons, no personnel was allowed near the two girders during this operation.

8) Installation of a protection for the coupling sleeves: using a shrink-free mortar the coupling devices are secured preventing possible further slip due to time or concrete vibration. As the workshop manufacturer was still not convinced the connections would remain intact during casting, this supplementary precaution measure was taken.

9) Concrete pouring: An adequate concrete composition is used, to allow for a good result even with the very high density of the wires. The concrete is of category C50/60 that requires characteristic compressive cylinder strength of 7250 psi (50 MPa) after 28 days, which can be considered HPC. The concrete composition is given in table 3.

Table 3 : Concrete Composition

Cement [pcf (kg/m³)]	26.5	(425)
Water [pcf (kg/m³)]	9.4	(150)
Aggregrate 4/7 [pcf (kg/m³)]	62.7	(1005)
Aggregrate 0/5 [pcf (kg/m³)]	57.7	(925)
Superplastifier [pcf (kg/m³)]	0.11	(1.8)

10) Cutting of the wires: the tensile force in the CFRP wires is transferred to the concrete. This cutting process is a time consuming and hazardous work since the carbon wires tend to split into their composing fibers during cutting. In addition traditional equipment used for this purpose such as fire cutting or disk grinding cannot be used due to the risk of fire. Once freed from the epoxy matrix, the carbon fibers are easily lit to fire. Care should be taken during the cutting operation.

PRIMARY GIRDER TESTING AND COMPLETED BRIDGE ASSESSMENT

MEASUREMENTS DURING PRESTRESS OPERATION

Monitoring of the behavior of the primary girder and the completed bridge is made possible by strain gauge measurements, both internally and externally. Before prestressing the CFRP wires, 3 gauges are positions on different locations at midspan of both girders. On both girders, 2 gauges have been placed on different CFRP wires and 1 on the passive reinforcement bars. The gauges and the gauge wiring are protected for water intrusion and against the impact of the concrete pouring and a thermo-couple is installed near the gauge locations to compensate for temperature effects. After establishing the zero measurement and leadwire compensation, the wires are stressed. Figure 7 gives the results for all 4 tested CFRP wires, the passive reinforcement renders of course zero values at this point. The legend refers to the number of the girder (2 and 10) and the position of the gauge (M(iddle) and L(eft)).
The strain results from figure 7 can be transformed into stress results using the simple linear stress-strain relation for CFRP wires, using the adequate Young’s Modulus. For a tension force of 5.9 kips (26 kN) or 5.3 kips (23.4 kN) after immediate losses, these values should range between 7457 µS (microstrain) and 8285 µS. The results from 3 out of 4 of these values are satisfying at 7314,7306 and 8269 µS. The slightly lower values can be explained by differences in Young’s modulus or applied tensile jacking force. The result of the last gauge is fundamentally different. It reaches a lower maximum of 6043 µS, rapidly decreasing to 5020 µS after only 45 minutes. Apparently, in this wire, there is a considerable slip in the anchoring sleeve.

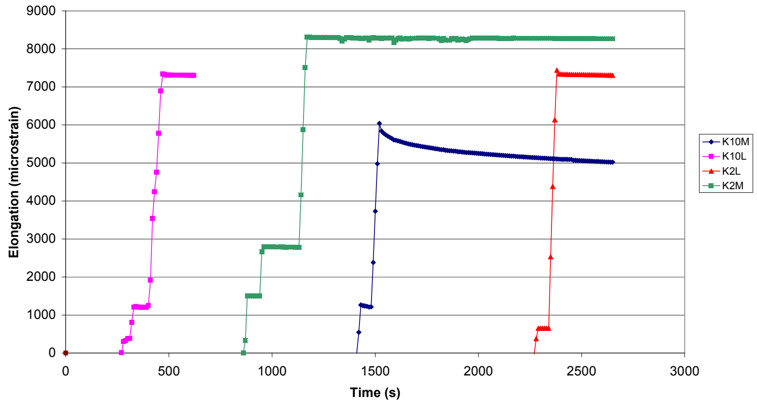


Fig. 7 Strain measurements during pretensioning operation.

During the next hours, this phenomenon continued and by the next day the strain level has dropped to a level rendering it practically useless. The slip in the anchoring sleeve is visible in figure 1.

MEASUREMENTS DURING WIRE CUTTING

After the concrete has sufficiently hardened, the wires are cut. During this operation, the progressive building up of compressive stresses in the concrete is measured by a strain loss of the CFRP girders and a compressive strain built up in the passive reinforcement. This operation lead to an average supplementary strain of –225 µS (compression) at most upper CFRP location, where the gauges are installed. This compressive strain build up is partially compensated by the influence of the proper weight of the girder due to the upward curvature of the girder. The strain value corresponds to a concrete compressive stress value of 1200 psi (8.28 MPa). The average measured value corresponds excellent to the calculated value of 1280 (8.82 MPa) taking into account the proper weight of the girder.

BRIDGE ASSESSMENT DURING LIVE LOAD TESTING

The girders are transported to the construction site. After installation of the formwork the deckplate is cast in situ. After installation of the safety barriers and the waterproofing, the bridge is assessed using additional strain gauges attached to the concrete surface of the lower flange of the CFRP prestressed beams, as well as on the other beams. These tests firstly run on both outer spans during a first test. The results of this test are described in [3]. During these tests it became clear that although the results were consistent, they did not quite match the results from the theoretical calculations. In fact the recorded strain values were substantially smaller than calculated. After examination of the results, a second test phase was performed, of which the results are presented in this contribution. In this second phase only one of both outer spans was tested, as the results from phase 1 indicated that results for both spans were fully comparable. In addition, supplementary strain gauges were attached to the top of the deckplate, and the safety barriers as they apparently contribute to the bridge stiffness.

Truck load

The bridge is loaded by a 5 axle fully loaded truck weighing a total of 93.2 kips (415 kN). Prior to the start of the test, all 5 axles have been weighed independently to provide the approximate axle load. The truck configuration and the axle loads are presented in figure 8. Measurements are carried out during the crossing of the truck at different transverse positions presented in figure 9. As is visible, the truck is positioned in the middle of the road and 4 eccentric positions left and right of the centerline, but symmetrical with respect to this centerline. As the bridge is fully symmetric, the strain values obtained from the strain gauges on the 3rd (CFRP) girder should correspond to the values obtained from the gauges on the 2nd (steel) girder for corresponding loadcases.

In order to exclude dynamic effects from the measurements, all strain data is measured statically by stopping the truck along the axle of the bridge at 1 m intervals. In fact the geometry itself of the bridge does not allow for high speeds justifying the use of static or quasi-static results.

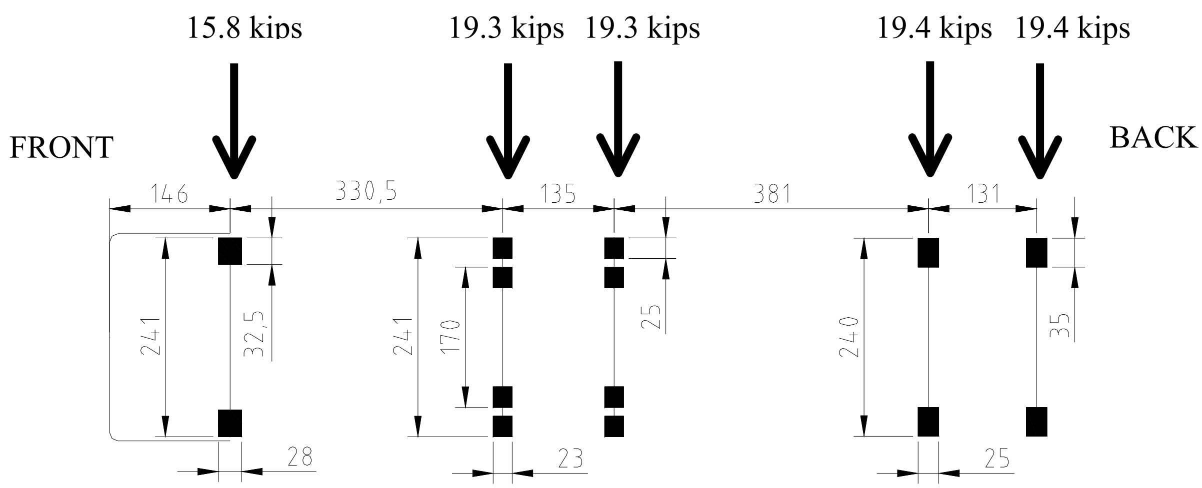


Fig. 8 Truck load configuration

Strain gauge measurements

This contribution will focus on the results obtained from strain gauges installed in the middle of the 57’9” (17.6 m) span on all four girders in longitudinal direction. This allows to check if:

- The overall strain results comply with the simple beam theory. Indeed the sum of the 4 measured values should be independent from the transverse position of the truckload. The resulting graph is then compared with a calculated graph obtained from combining the load geometry with a simple influence line for the moment at midspan of a simply supported beam
- the CFRP prestressed girders perform equally as their symmetrical conventional counterpart as transversely symmetrical load positions should give equal strain results in symmetrical girders

- the assumed Guyon Massonet distribution is accurate or at least a reliable calculation method
- the assumed stress strain relation for the concrete is accurate, as this was not found for the first series of measurements.

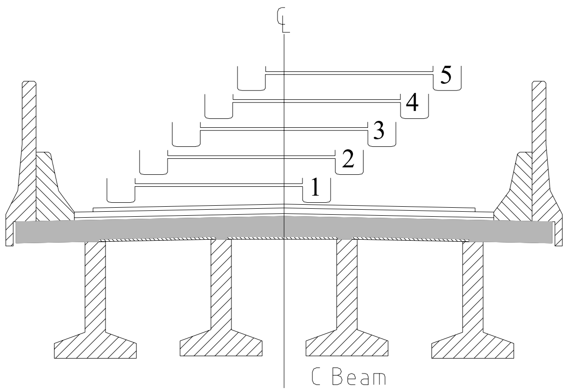


Fig. 8 Test arrangement

Summary of results

Figures 10 to 14 present the results of all five transverse load positions at the 11 measured locations. In all of these locations, only the longitudinal strain results are measured since these are the largerst strain values, although they reach only relatively low values. These values do not reach more than 30 µS. Measuring transverse strains or full planar stresses using a rectangular or delta rosette configuration would be inaccurate since these strains are too small to measure accurately, taking into account possible variations in temperature and electromagnetic interference. Although all necessary arrangements have been taken to minimize these influences, taking into account the construction site conditions, these even smaller strains cannot be measured accurately. In the transformation to stress results, it is assumed that only a longitudinal stress component exists. Although this is not an exact representation of the bridge behavior, this is a normal and generally accepted assumption. A description of the strain gauge locations is presented in table 4.

Table 4 : Strain gauge positions

A	At midspan on top of the waterproofing adjacent to the New Jersey safety barrier
B	At midspan attached to the side of the New Jersey barrier, approx. 1” from bottom
C	At midspan attached on the top of the New Jersey barrier
D	At midspan attached on the top of the additional safety barrier
7	At midspan on the center line of the outer girder not adjacent to the CFRP girder
8	At midspan on the center line of the middle girder adjacent to the CFRP girder
9	At ¼ of the span (abutment side) on the center line of the CFRP girder
10	At midspan on the center line of the CFRP girder
11	At ¼ of the span (pier side) on the center line of the CFRP girder
12	At midspan on the center line of the outer girder not adjacent to the CFRP girder
S	At midspan on the passive reinforcement in the CFRP girder

For the 5 transverse loadcases presented in figure 9, the results are given for the different longitudinal positions in the figures 10 tot 14. Unfortunately obstacles on both sides of the bridge prevented the truck from positioning very close to one side of the bridge. This caused a loss of values for the load positions 4 and 5. All other measured values for both these loadcases are however fully useable. From these graphs, it can easily be concluded that the New Jersey safety barrier does play a role in the overall strength of the bridge as the lower side strain values at the bottom reach up to 50 % of those measured at the lower side of the primary girders at midspan at 15 µS.

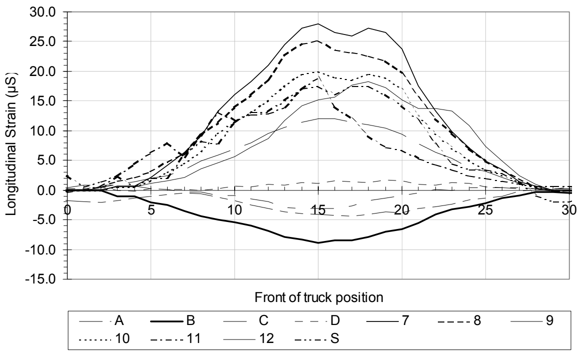


Fig. 10 Loadcase 1

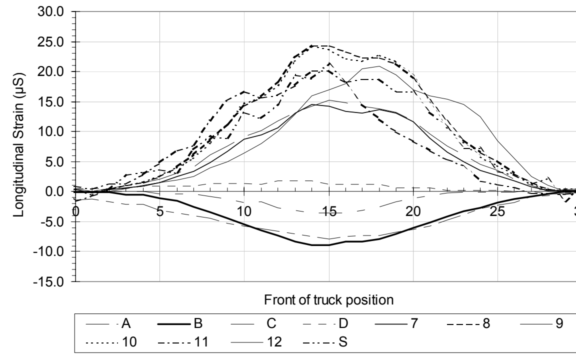


Fig. 11 Loadcase 2

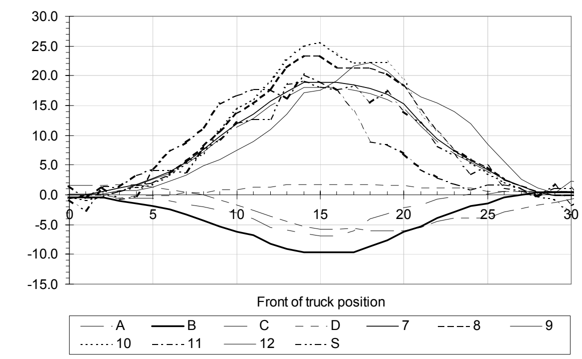


Fig. 12 Loadcase 3

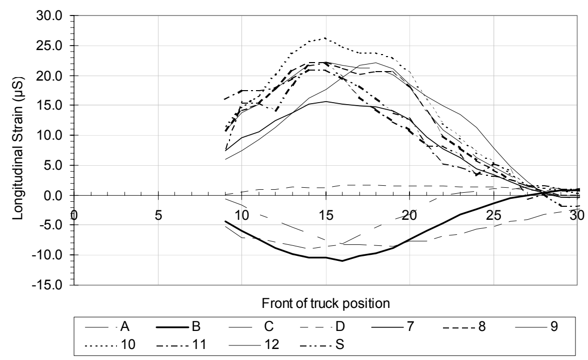


Fig. 13 Loadcase 4

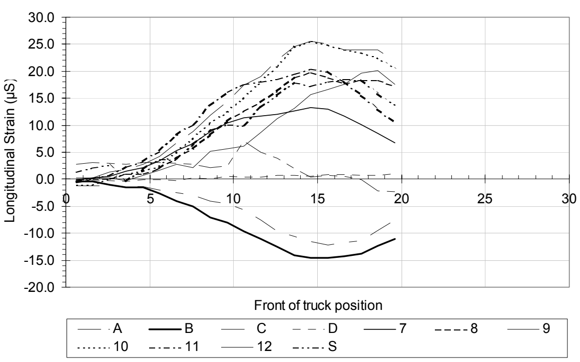


Fig. 14 Loadcase 5

On the other hand the strain distribution in the safety barriers is not linear. In fact on the top of the barrier the strain values are practically zero. This is due to the fact that additional saw cuts are made in these barrier allowing for shrinkage during concrete hardening. A calculation taking into account this effect would be too time consuming and too expensive for the purpose of this relatively simple structure. Nevertheless the safety barrier creates an additional stiffness and a reduction of longitudinal stresses in the bridge. Both, the additional high safety barrier and the waterproofing layer do not seem to interact with the rest of the structure as the strains measured in both these components is relatively low at less than 5 µS.

Bridge beam action

Apart from the transverse distribution of the wheel loads on the 4 girders, the total longitudinal bending moment is equal for the 5 loadcases. This is checked by summarizing the 4 measured strains at midspan. As the measured strains are very small, the construction acts linear elastic, as it should. Indeed the code requires remaining compression at the lower flange of the girders under normal loading conditions. As the center of gravity is equal for all girders, taking into account the secondary slab, the strain results are proportional to bending moment values. The results of this sums for the 5 loadcases is given in figure 15 along with a simple calculation of bending moments using the truck load configuration of figure 8 and the influence line of a simply supported beam. Of course the relation between the bending moments derived from the calculation and the strains requires a conversion factor including the moments of inertia and the Young's moduli of both primary and secondary concrete sections and possibly the influence of the steel reinforcement. For now the result from the calculation presented in figure 15 is given only relative to the strain gauge measurements.

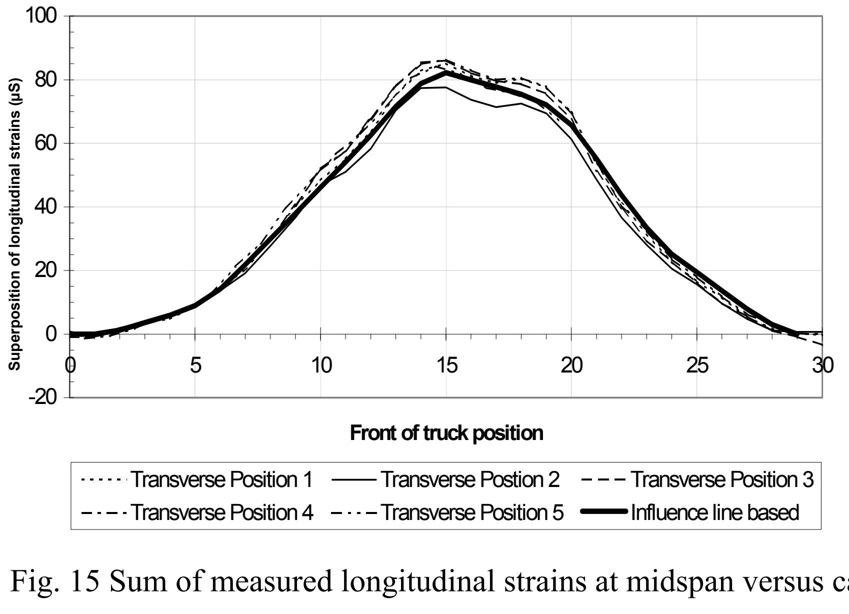


Fig. 15 Sum of measured longitudinal strains at midspan versus calculated beam results

Figure 15 indicates indeed a good match for all 5 transverse load positions proving the independence of transverse position in the longitudinal moment distribution and a good agreement with the influence line based calculated results proving the simply supported beam action.

CFRP girder action

The action of the CFRP girders can be considered equal to that of the conventional girders, at least for short-term performance if the measured strains in the CFRP girder are equal to those in its transversely symmetrical counterpart for the respective symmetrical loadcases. For this test case, this should imply that the recorded strain values on the CFRP girder for loadcases 1 and 2 should be comparable to the recorded values on the middle girder adjacent to the CFRP girder for loadcases 4 and 5. To give an example of this similarity, figures 16 and 17 provide these results. Clearly both CFRP and conventional girders possess equal stiffness and take a proportional part of the load.

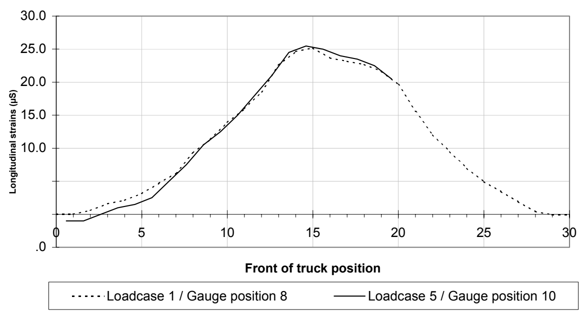


Fig. 16 CFRP – Steel Comparison 1

Fig. 17 CFRP – Steel Comparison 2

Guyon - Massonet distribution

Based on the theory of Guyon Massonet, the distribution of the total bending moment over the 4 girders can be calculated. This method applies Hübers equation for torsionally stiff orthotropic plates :

$$D_l \frac{\partial^4 v}{\partial x^4} + 2\alpha \sqrt{D_l D_t} \frac{\partial^4 v}{\partial x^2 y^2} + D_t \frac{\partial^4 v}{\partial y^4} = p(x,y) \text{ with } \alpha = \frac{K_l + K_t}{2\sqrt{D_l D_t}} \tag{2}$$

for a rectangular grill, simply supported at 2 equidistant edges. This method is widely used in Europe for this type of torsionally stiff girder and plate bridges. Using the 2 normalized transverse influence lines from figure 18 does this. Application of these influence lines gives the proportional results of table 5 for the center and most asymmetrical position.

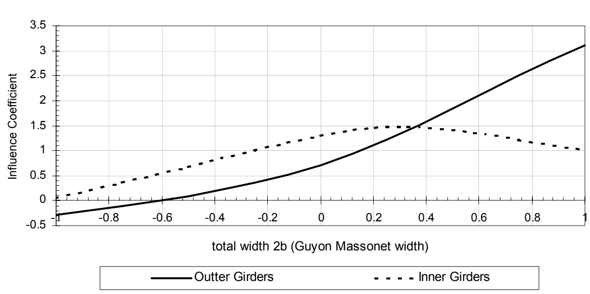


Fig. 18 Guyon Massonet based influence lines for transverse load distribution

Table 5 : Calculation and measured moment distribution

	Girder 1	Girder 2	Girder 3	Girder 4
Loadcase 1				
Theoretical	1.1	1.2	1.05	0.65
Measured	1.3	1.2	0.94	0.56
Loadcase 3				
Theoretical	0.85	1.15	1.15	0.85
Measured	0.88	1.09	1.19	0.84
Loadcase 5				
Theoretical	0.65	1.05	1.2	1.1
Measured	0.63	0.95	1.21	1.21

The values from table 5 indicate an acceptable compliance for the use of this method. It also indicates that there is no significant global stiffening caused by the cooperation with the safety barrier. This is due to the fact that this barrier is non-continuous as it is saw cut after concrete pouring for shrinkage control. On the other hand, the stiffness can locally be increased due to cooperation of primary girders, slab and New Jersey safety barriers. This would result in an overall reduction of the stresses at the lower flange of the primary girders, yet respecting the calculated Guyon Masonnet transverse distribution of load.

Bridge stiffness considerations

Up to now all calculations and comparisons have been carried out without taking into account any relation between the calculated values of bending moment and the measured values of strain. Assuming the structure behaves elastically and the concrete is always compressed, this relation is noted by equation 2:

$$\varepsilon_{long} = \frac{M.y}{E.I} \qquad (2)$$

where : ε_{long} : longitudinal strain at the lower flange of the primary girder
M : bending moment
y : ordinate at gauge location with respect to the center of gravity
E : Young's Modulus
I : moment of inertia

As figure 14 indicates that the influence line based stress history for the truckload corresponds very well to the measured strain histories for the various load positions the calculation can be carried out for only 1 longitudinal truck position. For this purpose truck position 15 is chosen as this position gives the largest absolute strain values. As also the transverse load position does not influence the sum of the measured stress values, this transverse position does not have to be taken into account if the average values in each of the girders is taken. The calculation is carried out in 2 ways, first by taking into account the effective bridge section at midpoint and second by taking into account the contribution of the New Jersey safety barrier. The results of this calculation, using formula (1) is presented in table 6. In this equation the Youngs modulus is taken at $5.33.10^6$ psi (36773 MPa), the value corresponding to the concrete grade C50/60.

Table 5 : Calculated and measured average strain values (lower flange) for load position 15.

Average measured value	21.5 µS
Average calculated value (without New Jersey cooperation)	36.6 µS
Average calculated value (with New Jersey cooperation)	23.3 µS

The results from table 5 indicate that in fact the cooperation of the New Jersey safety barrier in the stress calculation is necessary to create an acceptable stress – strain relation. The slight difference between the calculated and measured values can be explained by the fact that for the calculation, a normative Youngs modulus values is used. In practice this value is higher than the calculation value.

CONCLUSIONS

In order to evaluate the first application in Belgium of CFRP wires for prestressing pretensioned primary bridge girders, strain gauge measurements have been used. The measurements during pretensioning of the wires have enabled to determinate the bond between the CFRP wires and the epoxy resin material in the coupling devices is the critical failure location within this experiment. The conditions of application in the prefabrication workshop are, although relatively good for construction practice, not comparable to laboratory conditions. Further investigation is needed to improve the applicable loads and to possibly make this design less dependent on the workshop conditions. On the other hand, once the coupling devices have been sealing in a supplementary concrete cover, this problem no longer exists. Finally the bridge design is on the safe side since there is a cooperation between the bridge superstructure and the New Jersey safety barrier cast in place. This barrier provides an additional local stiffness to the bridge resulting in lower stress and strain values at the lower edge of the primary girders. However the cooperation is not easily incorporated in the verification process and secondary, in an accidental condition, the cooperation of the safety barrier is of course to be neglected. It merely gives an additional strength to the bridge in normal conditions and complicates interpretation of the measured values. In addition the measurements have indicated that the assumed transverse load distribution, calculated by the Guyon Massonet method is sufficiently accurate for this purpose. Finally it was found that the stiffness of the CFRP girders could be considered equal to that of the conventional girders, since they take an equal amount of the total bending moment in the bridge. Therefore this first experiment in Belgium may be qaulified as favourable. The observed problems with transferring the laboratory results into workshop conditions combined with the high amount of manual labour and the high cost of the CFRP material tend to temper this optimism. Obviously, more field research in stead of laboratory research is needed to assess accurately the practical application of the material in bridge building, especially with respect to developing a reliable and labour effective anchoring system.

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