

DESIGN AND REHABILITATION OF CONCRETE STRUCTURES USING ADVANCED COMPOSITE MATERIALS



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ABSTRACT

This paper seeks to demonstrate how advanced polymer matrix composite materials developed for high-performance aircraft can offer major advantages for civil structures. It focuses on the development and applications of strengthening and field monitoring technologies for bridges based on combinations of corrosion-resistant fiber-reinforced polymers and integrated fiber optic structural sensing. Since 1986, FRP laminates have been used to post-strengthen bridges (Rostasy 1992, Meier 1993). The method now represents the state-of-the-art and is a commercial success. Although cables made from advanced composite materials (Head 1992, Meier 1992) has so far failed to achieve an economic breakthrough, several new cable-stayed and suspension bridges in Europe have made use of FRP cables, some even incorporating optical fiber sensors.

Key Words: advanced composites, structural repair, CFRP cables, optical fiber sensors

1. STATE OF SELECTED PROBLEMS

1.1 Externally bonded steel reinforcement

It is frequently necessary to strengthen existing bridges or parts of them. The reasons that make this sort of reinforcement necessary can be summarised as follows: First, a change in the use of a bridge may produce internal forces in individual structural parts that exceed the existing cross-sectional strengths. These increased internal forces may be a result of higher loading or less favourable configuration of existing loading. Bridges may also need reinforcement because damage due to external factors has reduced the cross-sectional resistance. The object of repairing such damage is to restore the original cross-sectional strength. Another possibility is misdesign of a bridge or parts of it. This includes all cases where the cross-sectional strength at crucial points is too low, so that either the cross-sectional safety or the overall safety of the respective structure or structural element fails to comply with existing codes. Poor construction workmanship may mean that the cross-sectional

strengths originally calculated are not achieved. For instance, the as-built cross-sectional dimensions may be smaller than those planned. Or it can happen that individual rebars or tensioning cables are incorrectly set, interchanged or even missing, which reduces the cross-sectional strength substantially. Another severe problem is stress corrosion of pre-stressing steel. Even today it is not possible to rule out damage of this kind notwithstanding the great improvements made in the properties of pre-stressing steel. Many hundreds of thousands of bridges worldwide will need to be repaired within the next few years for one or another of these reasons.

There are a number of ways to repair a particular bridge or parts of one, depending on the type of construction and the given situation. Adhesively bonded external steel reinforcement is one possible way of achieving structural strengthening. This method was originally invented in France (L'Hermite 1971) in the mid sixties; in the early seventies it was further developed in Switzerland (Ladner 1974), Germany (Rostasy 1980) and England (James 1980) and is nowadays state-of-the-art in Europe.

In previous papers (Kaiser 1989, Meier 1987, 1991, 1992, 1997/2) the advantages and disadvantages of post-strengthening by means of steel plates have been discussed as well as the reasons for replacing steel plates with advanced composites especially with carbon-fiber reinforced plastics (CFRP), above all in bridge construction.

- The most important reason is clear from the following observations: Within the framework of long term creep tests on beams strengthened with steel plates (Ladner 1990) at the EMPA (EMPA is the German acronym for Swiss Federal Laboratories for Materials Testing and Research), the residual strength was determined after 15 years of exposure to weathering without de-icing salts. In several cases, spots of corrosion (diameter up to 15 mm) were discovered at the joint between the steel and the adhesive. These spots are growing and the strengthening system will finally fail due to corrosion.
- Steel plates are heavy. Therefore their handling on construction sites e.g. inside a box girder is not convenient. Often expensive scaffolding is required to bond the steel plates to the structure.
- Due to the weight of the steel plates their length is in general restricted to 6, maximum 10 meters. If a greater length is required joints are necessary. Such joints cannot be welded since this would destroy the adhesive bonding.
- If bonded steel plates are compression loaded they will fall off in a buckling mode in an early stage of loading.

1.2 Steel cables for external post-tensioning, suspension and cable stayed bridges

Another important method for rehabilitation is external post-tensioning. This method whereby the steel tendons are not embedded in the concrete, but are placed instead "externally" to the structural elements, offers the advantage that the tendons can be inspected and replaced. However, this normally rules out grouting of the steel tendons, thus making them susceptible to corrosion. Stress corrosion is another problem for the highly tensioned steel cables.

Bridge suspender and stay cables are exposed to severe fatigue loads, often combined with corrosion action. The performance of steel cable is often inadequate in this respect (Watson 1988, Hamilton 1995). Long familiar with this problem, designers of suspension bridges have devised systems to allow the relatively simple replacement of suspender cables. Although similar systems have not always been employed with cable-stayed bridges the excessive cost makes repair or replacement of cables every 20 to 30 years an unsatisfactory solution in either case. This dilemma has led to investigations in recent years into the suitability of non-metallic materials for the manufacture of suspender and stay cables. On the basis of a portfolio analysis, the author proposed in the early 1980s that carbon fiber reinforced polymers (CFRP) could be used as a cable material.

2. COMPOSITE MATERIALS OFFER A PROMISING ALTERNATIVE TO STEEL

Advanced fibrous composites offer the engineer in the construction industry an outstanding combination of properties not available from other materials. Fibers such as glass, aramid, and carbon can be introduced in a certain position, volume fraction, and direction in the matrix to obtain maximum efficiency. Other advantages offered by advanced composites are lightness and resistance to corrosion and stress corrosion. Some also offer outstanding fatigue performance and greater efficiency in construction compared with more conventional materials.

The most suitable manufacturing process for strips for post-strengthening and for wires of cables is the pultrusion process. The question which fiber is most suitable is still the subject of lengthy discussions. A careful evaluation (Meier 1993) showed that for bridge repair carbon fiber is the material best suited in most cases. This fiber is alkaline-resistant and does not suffer stress corrosion. These are very important arguments for such applications.

A similar study showed that the carbon fiber is also an excellent choice for bridge cables especially due to the outstanding fatigue properties.

Composites with aligned fibers perform best when the directions of the fibers coincide with those of the main loads. They are ideal for use with elements subject to tensile loads along one axis, where the load direction corresponds to that of the fibers. The extremely favourable mechanical properties exhibited by fibrous composites in such situations are superior to those of any other material. Bridge cables and laminates exposed to loads along one axis for the post-strengthening of structural members would thus appear to be the most promising applications for advanced fibrous composites in the field of structural engineering.

3. MANUFACTURING PROCESS

The pultrusion process (Fig. 1) is the most suitable method for manufacturing composite strip and composite wire for cables. Rovings (strands of reinforcement) are drawn through an impregnating tank with polymer resin, a forming die, and finally a curing area. The fibers have good parallel alignment and are continuous. The fiber volume content of the wire or strip used in structural engineering is in the range of 65 to 70%. The axial properties of CFRP wire (modulus, strength) are simple to calculate using the rule of mixtures.

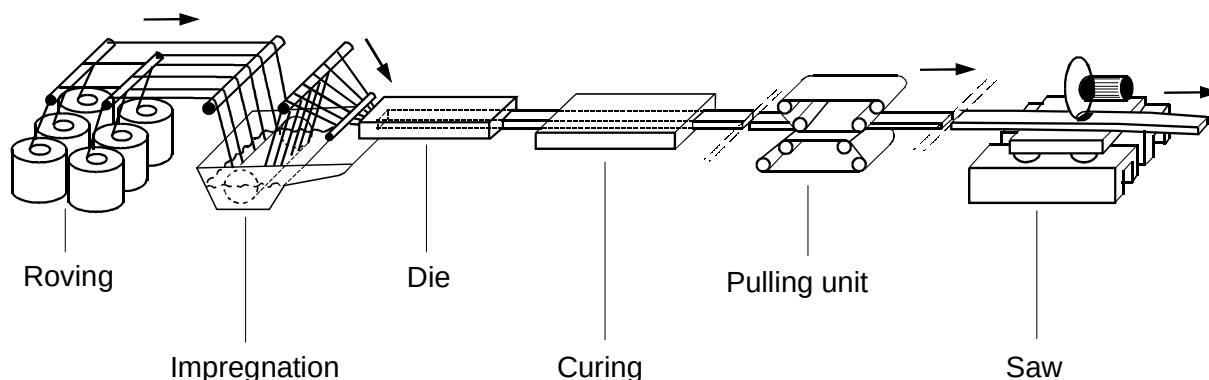


Figure 1 - Pultrusion unit for the production of CFRP strips and wires

4. ADHESIVELY BONDED EXTERNAL CARBON FIBER REINFORCED PLASTIC (CFRP) REINFORCEMENT

4.1 Specific advantages

Advanced composite strips (Figure 2) or sheets can replace steel plates for post-strengthening with overall cost savings deriving from the simplicity of the strengthening method because:

- They do not corrode.
- They are easy to handle on the construction site and can be lifted onto the structure with a scissors-lift or similar device without expensive scaffolding. They can be carried inside narrow box girders easily by hand. To give an example, 1 kg of carbon fiber reinforced polymer (CFRP) strips can replace at least 30 kg of steel plates for strength.
- It is not necessary to keep them pressed against the structure during the adhesive curing time as required with steel plates. They are simply rolled on like "wallpaper".
- The strips are available on endless reels, no joints are necessary.
- Unlike adhesively bonded steel plates CFRP strips subjected to compressive stresses do not fall off on low load level. They increase flexural and shear strength and reduce deflections and cracking.
- They cause minimal disruption to the bridge function.
- They do not reduce the overhead clearance of a bridge.
- They require less time and labour to install.
- Costs are lower than for other methods such as external post-tensioning.
- Cost efficiency is already good in terms of initial cost and excellent in terms of life-cycle costs.

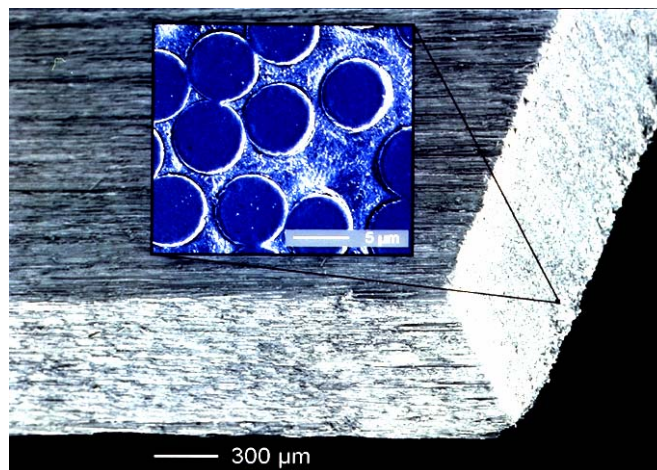


Figure 21 - Pultrusion unit for the production of CFRP strips and wires

4.2 Research and development

From 1982, carbon fiber reinforced epoxy resin composites have been successfully employed at the EMPA for the post-strengthening of reinforced concrete beams. Loading tests were performed on more than 150 flexural beams having spans of between 2 and 7 metres. Typical CFRP strips used within this research work are described in Table 1.

The research work shows the validity of the strain compatibility method in the analysis of various cross-sections (Deuring 1993, Kaiser 1989, Meier 1987, 1991, 1992, 1993, 1997/2) This implies that the calculation of flexure in reinforced concrete elements which are post-strengthened with carbon fiber reinforced epoxy resin composites can be performed in a similar way to that for conventional reinforced concrete elements. The work also shows that the possible occurrence of shear cracks may lead to peeling of the strengthening composite. Thus, the shear crack development represents a design criterion. Flexural cracks are spanned by the CFRP strip and do not influence the loading capacity. In comparison to the unstrengthened beams, the strengthening strips lead to a much finer cracking distribution. A calculation model (Kaiser 1989) developed from the CFRP composite agrees well with the experimental results.

Table 1: Properties of CFRP strips

Property	Strip type no.				
	1	2	3	4	5
Fiber type	66% T 300 34% E-glass	Carbon T 300	Carbon	Carbon T 700	Carbon M 46 J
Fiber volume fraction [%]	-	70	51	50	70
Longitudinal strength [MPa]	960	2000	1900	2100	2600
Longitudinal elastic modulus [GPa]	80	147.5	129	130	305
Strain at failure [%]	1.15	1.36	1.47	1.62	0.85
Density [g/ccm]	-	1.58	1.46	-	-

When a change of temperature takes place the differences in the coefficient of thermal expansion of concrete and the carbon fiber reinforced epoxy resin composites result in thermal stresses at the joints between the two components. After 100 frost cycles ranging from +20-deg C to -25-deg C, no negative influence on the loading capacity of the three post-strengthened beams was found (Kaiser 1989). These results were confirmed by even more severe tests in Canada (Soudki 1997, Green 1999).

For the post-strengthened beams the following failure modes were observed in the 4-point bending load tests:

- The CFRP strips failed during the loading with a sharp explosive snap, the impending failure was preceded well in advance of the failure by cracking sounds.
- Classical concrete failure in the compressive zone of the beam.
- Continuous peeling-off of the CFRP strips due to an uneven concrete surface or due to shear cracks. For thin strips of thickness less than 0.7 an even bonding surface is required. If the surface is too uneven, the strip will slowly peel off during the loading.
- Shearing of the concrete in the tensile zone (it can also be observed as a secondary failure).
- Interlaminar shear within the CFRP strip (observed as secondary failure).
- Failure of the reinforcing steel in the tensile zone (this failure mode was only observed during fatigue tests).

The following failure modes were not observed but are theoretically possible;

- Cohesive failure within the adhesive.
- Adhesive failure at the interface between the CFRP strip and the adhesive.
- Adhesive failure at the interface between the concrete and the adhesive.

For post-strengthening with CFRP composites it is recommended that the design rule for the CFRP composite is such that it should break (tensile failure) during yielding of the internal steel reinforcing bars before a compressive failure of the concrete. Yielding of the steel bars should not occur before reaching the ultimate service loads.

Kaiser (1989) investigated a 2 m span beam under fatigue loading. The cross-section was 300 mm wide and 250 mm deep. The existing steel reinforcement consisted of 2 rebars of 8 mm diameter in the tension and in the compression zones. This beam was post-strengthened with a glass/carbon fiber hybrid composite (Table 1, type 1) having the dimension 0.3 by 200 mm. The fatigue loading was sinusoidal at a frequency of 4 Hz; the test set up corresponded to a four point flexure test with loading at the one third points. The calculated stresses in the hybrid strip and the steel reinforcement are listed in Table 2. After 480000 cycles the first fatigue failure occurred in one of the two reinforcing rods in the tension zone; after 560000 cycles the second reinforcing rod failed at another cross-section; after 610000 cycles a further break was observed in the first reinforced rod and after 720000 cycles a second break in the second rod was observed. The first damage to the composite appeared after 750000 cycles and it was in the form of fractures of individual rovings of the strip; the beam exhibited gaping cracks, which were bridged by the hybrid strip. The relatively sharp concrete edges rubbed against the hybrid strips at every cycle and after 805000 cycles the composite finally failed, however, the test was executed with unrealistically high steel stresses. The aim of the test was to gain insight into the failure mechanism after a complete failure of the steel reinforcement; it was surprising to observe how much the hybrid strip could withstand after failure of the reinforcement.

Table 2: Fatigue loading and stresses

Loads	Stresses [MPa]	
	Rebars	Strip
Minimum 1 [kN]	21	11
Maximum 19 [kN]	407	205

EMPA performed further fatigue tests on a T-beam with a span of 6 m under realistic loading conditions. The cross-section was 900 mm wide and 500 mm deep. The existing steel reinforcement consisted of 4 rebars of 26 mm diameter in the tension zone. The total load carrying capacity of this beam amounted to 610 kN for each of the 2 loading points without post-strengthening. When the beam was strengthened by bonding a CFRP strip, which had dimensions of 200 x 1 mm (strip type number 2, Table 1), its load carrying capacity was increased by 32% to 815 kN per loading point. The fatigue loading was sinusoidal at a frequency of 4 Hz; the test set up corresponded to a four point flexure test with loading at the one third points. The calculated stresses in the CFRP strip and the steel reinforcement are given in Table 3.

Table 3: Fatigue loading and stresses

Loads	Stresses [MPa]	
	Rebars	Strip
Minimum 125.8 [kN]	131	102
Maximum 283.4 [kN]	262	210

The beam was subjected to this loading for 10.7 million cycles and crack development was noted after every 2 million cycles (Deuring 1993). After 10.7 million cycles the tests were continued in an environmental condition where the temperature was raised from room temperature to 40-deg C and the relative humidity to a value of 95%. The aim of this test was to verify that the bonded CFRP strip could withstand very high humidity under fatigue loading. Initially the CFRP strip was soaked with water nearly to saturation. After a total of 12 million cycles the first steel rebar failed due to fretting fatigue. The joint between the CFRP strip and the concrete did not present any damage. In the next phase of the test programme, the external loads were held constant. After 14.09 million cycles the second steel rebar failed, also due to fretting fatigue. The cracks which were bridged by the CFRP strip rapidly grew and after failure of the third rebar, due to yielding of the remaining steel, the CFRP strip was sheared from the concrete. This fatigue test was a success exceeding all expectations.

In some cases it can be very advantageous to provide pre-stressing to the flexural-strengthening strips (pre-stressing of sheets is nearly impossible). In this way, the serviceability of the bridge structure can be improved and the shearing of the strips due to shear failure of the concrete in the tension zone can be avoided. When the pretensioning force is too high, failure of the beam due to pretension release, will occur at the two ends because high shear stresses develop in the concrete layer just above the CFRP strip. Therefore the design and construction of the end regions require careful attention. Tests and calculations have shown that without special end anchoring CFRP strips shear off from the end zones immediately with a pre-stress of only 5% of their failure strength (Deuring 1993). In order to achieve a technically and economically rational pre-stress, considerably higher degrees of pre-stressing of about 50% are necessary. End anchorings for flexure beams were developed and successfully tested at EMPA (Deuring 1993). Design recommendations have been available since the early nineties. Unlike the non-tensioned CFRP strips the tensioned strips have not (yet) been a commercial success. The application of the systems described by Triantafillou 1992, Deuring 1993 and Meier 1997 is still too costly. In contrast to pure shear strengthening, the advanced composites which wrap around the flexural strips must most definitely be pre-stressed. This will build up the maximum multi axial stress condition in the concrete and also interlock cracks. In this way failure at the two ends of the CFRP strips can be avoided.

4.3 Research and development

The CFRP strips must be well ground on the bonding side. The outermost layer, normally matrix-rich, has to be removed to expose the fibers. Immediately prior to bonding, the CFRP surface must be cleaned with acetone. This must be repeated until the cleaning cloth no longer becomes „dirty“ with CFRP dust. The concrete surface must be treated by sandblasting, high-pressure water jets, stoking or grinding. The concrete skin should be removed and the aggregates exposed, providing a roughness of 0.5-1.0 mm. In case of unevenness of the surface (e.g. due to cavities and large cracks) the application of epoxy mortar might be necessary. To ensure good bonding of the epoxy adhesive to the concrete, the moisture content of the latter should be low, say below 4-5%. Also, it is recommended that the tensile strength of concrete in the anchorage zones be tested by means of pull-off testing. The

associated tensile bond strength should be in the order of 2 MPa. Shortly before bonding the concrete must be cleaned with a vacuum cleaner.

Highly filled epoxy resin is the classic adhesive for bonding. The adhesive must be applied to the CFRP strips in a roof shape so that the extra adhesive is squeezed out when the strip is pressed to the concrete structure. For a strip width of 140 mm, for example, the peak height in the middle of the strips is about 7 mm. Uniform pressing of the CFRP strips and evacuation of entrapped air can be achieved through the use of a hard-rubber roller. Excess adhesive can then be removed and the CFRP cleaned, if necessary. If required for aesthetic reasons, the outer face of the CFRP strips can be coated with epoxy paint. Cement mortars can also be applied after the reinforcement has been primed with a suitable bonding agent.

Post-strengthening with strips is best suited for more or less flat girders and slabs. For a 1-mm thick strip a minimum radius of curvature of approx. 300 mm is required. This method can therefore not be used for wrapping of columns with a rectangular cross-section.

Application of sheets (flexible fabrics) would normally require the following steps: Surface treatment: remove dirt from concrete surface, round off sharp corners (minimum radius should be in the order of 25-30 mm), blow or sweep dust off the concrete surface after sandblasting or grinding, dry the concrete (if wet). Primer coating, putty application (optional), apply putty after the primer becomes tack free. After putty application, allowable unevenness must be in the order of 1 mm. Fabric application: Mix the epoxy resin and apply it on the concrete surface (undercoating), adhere the fabric, press using de-bulk roller, allow for complete fabric impregnation and apply another coating. Finishing: Remove residual resin using a rubber scraper. Curing: Protect reinforcement from rain, sand and dust, apply paint (if needed) once the resin is tack free.

Post-strengthening with sheets is best suited for wrapping of columns with a rectangular cross-section. A minimum radius of curvature of approx. 25 mm is required

4.4 Examples of externally bonded CFRP reinforcement

The Ibach Bridge: This bridge located in the Canton of Lucerne was completed in 1969. In 1991 it needed repairing. The bridge is designed as a continuous, multispan box beam with a total length of 228 meters. The damaged span of the bridge has a length of 39 meters. The box section is 16 meters wide, with a central, longitudinal web. During core borings performed to install new traffic signals a post-tensioning tendon in the outer web was accidentally damaged, with several of its wires completely severed by an oxygen lance. As a result, the granting of authorizations for special, heavy loads across the bridge was suspended until after completion of the repair work. Since the damaged span crosses Swiss National Highway A2 the traffic lanes in the direction of Lucerne on this highway had to be closed during the repair work. The work could therefore only be conducted at night. Carbon fiber-reinforced plastics (CFRP's) are forty to fifty times more expensive per kilogram than the steel used to this date (Fe 360) for the reinforcement of existing structures. Do the unquestionably superior properties of CFRP's justify their high price? When one considers that, for the repair of the Ibach bridge, 175 kg of steel were replaced by a mere 6.2 kg of CFRP, the high price no longer seemed so excessive. Furthermore, all the work was carried out from a mobile platform, thus eliminating the need for expensive scaffolding. The bridge was repaired in 1991 with 3 CFRP strips of 5000-mm length. The properties of these strips are given in Table 1, strip type no. 3. A loading test with an 84-tonne vehicle demonstrated that the reinstatement work with the CFRP-strips was a complete success. The experts working on the repair of the Ibach Bridge were pleasantly surprised at the simplicity of applying the 2 mm thick and 150 mm wide CFRP-strips. This was the first repair of a bridge with externally bonded CFRP strips in the world. Since 1991 this application has enjoyed success exceeding all expectations.

The Covered Wooden Bridge Near Sins: The covered wooden bridge near Sins in Switzerland was built in 1807 to the design of Josef Ritter of Lucerne. On the western side the original supporting structure is almost completely preserved to this day. The eastern side was blown up for strategic reasons on November 10, 1847 during the Civil war. In 1852, the destroyed half of the bridge was rebuilt with a modified supporting structure. On the western side, the supporting structure consists of arches strengthened with suspended and trussed members. On the eastern side, the supporting structure is made up of a combination of suspended and trussed members with interlocking tensioning transoms. Originally the bridge was designed for horse-drawn vehicles. Since the 1930-ties vehicles with a load of 20 tones have been permitted.

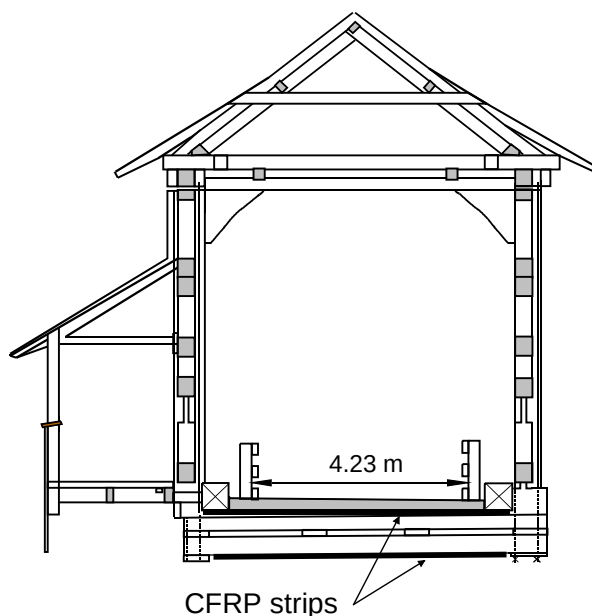


Figure 3 - Cross-section of the historic bridge in Sins

In 1992 the wooden bridge was in urgent need of repair. It was decided to replace the old wooden pavement with 20-cm thick bonded wooden planks, transversely pre-stressed. The most highly loaded crossbeams were strengthened by EMPA using carbon fiber-reinforced epoxy strips. Each of these crossbeams was constructed of two solid oak beams placed one upon the other. A cross-section of the bridge with the strengthening strips is shown in Figure 3. In order to increase the depth, wooden blocks were originally inserted between the beams. The lower beams were 37 cm deep and 30 cm wide; the upper beams 30 cm deep and 30 cm wide. The crossbeams were strengthened either with 1.0 mm thick CFRP strips made of high-modulus M46J fibers or with 1.0 mm thick CFRP strips made of high-strength T700 fibers. The M46J strips were 250 mm wide at the top and 200 mm wide at the bottom. The T700 strips were 300 mm wide at the top and 200 mm wide at the bottom. The properties of the strips with the high-modulus Toray M46J fibers are given in Table 1, strip type no. 5. Those of the high-strength Toray T700 fibers are also in Table 1, as strip type no. 4. Before bonding the strips, the bonding surface was planed with a portable system. Selected crossbeams are equipped with strain measurement devices, which allow long-time monitoring. Up to now the results are very satisfactory. After application of the CFRP strips pulse infrared thermography was applied very successfully for the first time for quality assurance of the bonding. The historic wooden bridge in Sins is a valuable structure, both from the aesthetic and the technical viewpoint. It is also of historic value and under protection as a national monument. The technique using CFRP strips is especially suited for the post-strengthening of such structures since the thin but extremely stiff and strong strips are hardly noticeable and therefore do not detract from the original design of the structure.

Since 1992 the strengthened cross-beams of the Sins bridge with CFRP-strip reinforcement have helped to provide practical experience under extremely high loading and built up confidence in this technique for preserving historic bridges. Meanwhile many similar structures have been rehabilitated in this manner in Europe and in North America.

The Oberriet Bridge: Rehabilitation of the Oberriet-Meiningen Bridge was planned in late 1996. The bridge, built in 1963, spans the border between Switzerland and Austria, linking Oberriet to Meiningen. It crosses the river Rhine in 3 spans (35 - 45 - 35 m) as a continuous steel/concrete composite girder. Due to increased traffic loads, post-strengthening of the concrete bridge deck (Figure 4) became necessary. The application of a total length of 640 m of CFRP strips has proved extremely successful. Thorough investigations have shown that besides routine maintenance the concrete bridge deck was also in need of transversal strengthening. This was obviously due to the fact that the deck was designed in 1963 for the then standard truckload of 14 tons. Today the standard truckload for this type of bridge is 28 tons. Different solution options were available to guarantee structural safety for the future under today's traffic loads:

- replacing the entire deck,
- improving the cross-section by providing additional depth and/or
- post-strengthening by bonding additional reinforcement to the existing deck.

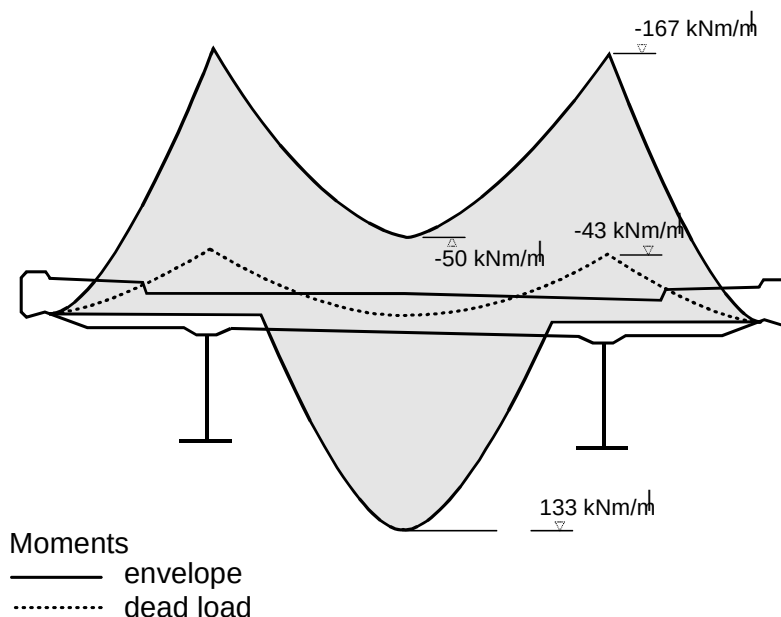


Figure 4 - Cross-section and moment envelope of Oberriet bridge

Because the existing concrete was in good condition and the chloride concentration in the concrete exceeded the critical values only in the outermost 10 mm it was decided not to replace the deck. Simply increasing the depth of the deck by adding concrete to attain the necessary transversal flexural capacity would, however, have caused inadmissible longitudinal stresses for the superstructure. Bonding of additional reinforcements therefore remained the only solution. Structural components post-strengthened with bonded plates or strips should have a total residual safety factor of 1.2 after failure of the plates or strips. The fact that the required strengthening factor was 2.15 meant that the sectional area of the deckslab still had to be increased. Bonding transversal CFRP strips on the bottom of the slab and adding 8 cm of concrete on top of the slab made it possible to meet all requirements. Adding new concrete also allowed removal of the top layer of concrete with the high chloride concentration by water blasting. CFRP strips 80 mm wide and 1.2 mm thick (70-vol% T700 fibers, strength 3000 MPa) were chosen for post-strengthening (Figure 5). A total of 160 strips, each 4 m

long, were laterally bonded to the bridge deck every 75 cm.

The Fürstenland Bridge: This 60-year-old, reinforced concrete, box girder bridge supported by an arch is located near St. Gallen. Inspection revealed a high chloride content in the concrete (due to moisture that was unable to drain out of the box girder) and significant corrosion of the reinforcing steel in parts of the box girder. When the bottom flange of the box girder was replaced, however, the torsional resistance of the girder was reduced. It has now been strengthened with CFRP strips - similar to those used on the Oberriet Bridge. Two CFRP strips were bonded to the lower portion of each of the inside walls of the webs of the box girder. This strengthening method was used during construction to provide stiffness to the girders without closing the bridge and afterwards to increase the permissible load for trucks from 28 tons to 40 tons. A total of 8000 meters of CFRP strip was used.



Figure 5 - Oberriet bridge: concrete deck post-strengthened with CFRP strips (black)

5. CABLES FOR CABLE-STAYED/SUSPENSION BRIDGES AND POST-TENSIONING

5.1 Structure of the Cables

The cables comprise bundles of pultruded parallel CFRP wire (Figure 6). The fiber volume content is in the range of 68-72%, the wire diameter 5 mm and the longitudinal strength approx. 3,300 MPa. The modulus depends on the fibers used. For a T 700 fiber it reaches 165 GPa; for a M46J fiber 300 GPa. The main purpose of bundling the wires parallel was to ensure less strength and stiffness loss than for twisted wires. Given the extreme orthotropic behavior of a CFRP wire, however, its proper anchorage in parallel wire bundles poses difficulties.

Since CFRP wires are corrosion resistant there is no corrosion inhibiting compound or grout required. However, it is still necessary to protect the wires against wind erosion and ultraviolet radiation attack because the combination of these two attacks could degrade the wires. A polyethylene (PE filled with carbon black) pipe or sheath would be adequate for shielding.

When a load is applied to a stay cable with a horizontal as well as a vertical span, the elongation consists of the material deformation augmented by a geometric deformation due to the straightening out of the cable sag. The ratio of the applied stress to the observed "strain" (elongation/original distance between the end points) is called the relative equivalent modulus. The low mass density of CFRP cables gives them an advantage in comparison to steel cables with increasing horizontal cable span. Some data are given in Table 4.

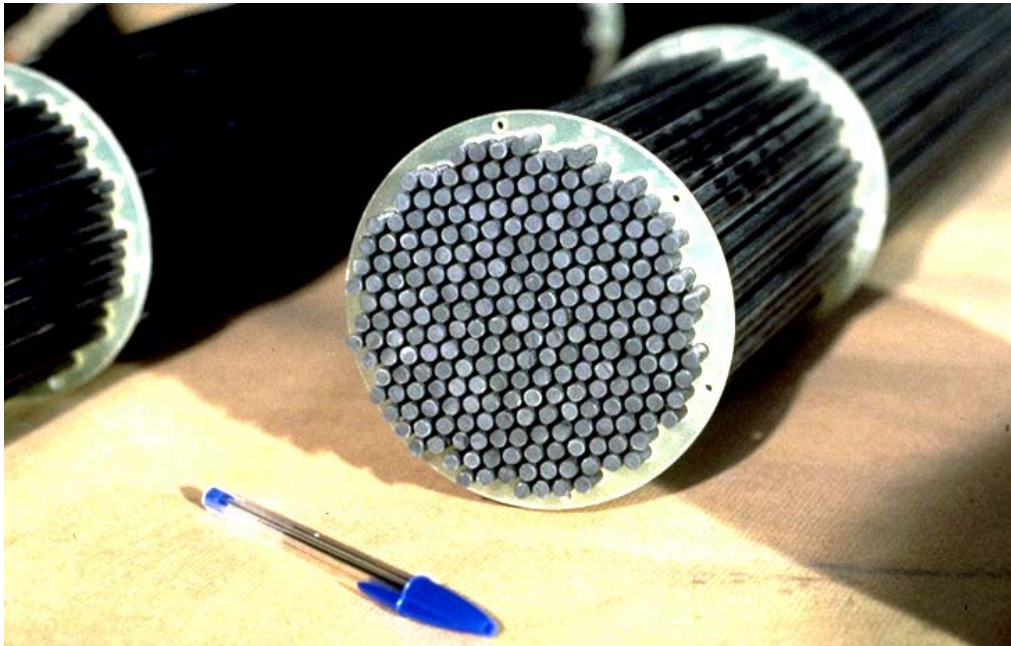


Figure 6 - Parallel wire bundle, 241 wires diameter 5 mm, 12 MN ultimate load capacity

Table 4: Relative equivalent modulus for 720 Mpa

Horizontal cable span [m]	Relative equivalent modulus [GPa]	
	Steel	CFRP
0	210	165
500	196	165
1000	163	163
1500	128	162
2000	98	159

5.2 The Anchorage of CFRP Cables

The key problem facing the application of CFRP cables and thus the impediment to their widespread use in the future is how to anchor them. The outstanding mechanical properties of CFRP wires mentioned above are only valid in a longitudinal direction. The lateral properties including

interlaminar shear are relatively poor. This makes it very difficult to anchor CFRP wire bundles and obtain the full static and fatigue strength.

The EMPA has been developing CFRP cables using a conical resin-cast termination similar. The evaluation of the casting material to fill the space between the metallic cone of the termination and the CFRP wires was the key to the problem. This casting material, also called load transfer media (LTM) has to satisfy multiple requirements:

- The load should be transferred without reduction of the high long time static and fatigue strength of the CFRP wires due to the connection.
- Galvanic corrosion between the CFRP wires and the metal cone of the termination must be avoided. It would harm the metal cone. Therefore the LTM must be an electrical insulator.

The conical shape inside the socket (Figure 7) provides the necessary radial pressure to increase the interlaminar shear strength of the CFRP wires. If the LTM over the whole length of the socket is a highly filled epoxy resin there will be a high shear stress concentration at the loadside of the termination on the surface of the CFRP wire. This peak causes pullout or tensile failure far below the strength of the CFRP wire. We could avoid this shear peak by the use of unfilled resin. However this would cause creep and an early stress-rupture. The best design is a gradient material. At the loadside of the termination the modulus of elasticity is low and continuously increases until reaching a maximum. This way a shear peak can be avoided. The LTM is composed of aluminum oxide ceramic (Al_2O_3) granules with a typical diameter of 2 millimeters. All granules have the same size. To get a low modulus of the LTM, the granules are coated with a thick layer of epoxy resin and cured before application. Hence shrinkage can be avoided later in the socket. To obtain a medium modulus the granules are coated with a thin layer. To reach a high modulus, the granules are filled into the socket without any coating. With this method the modulus of the LTM can be designed tailor-made. The holes between the granules are filled by vacuum-assisted resin transfer molding with epoxy resin (Meier 1998).

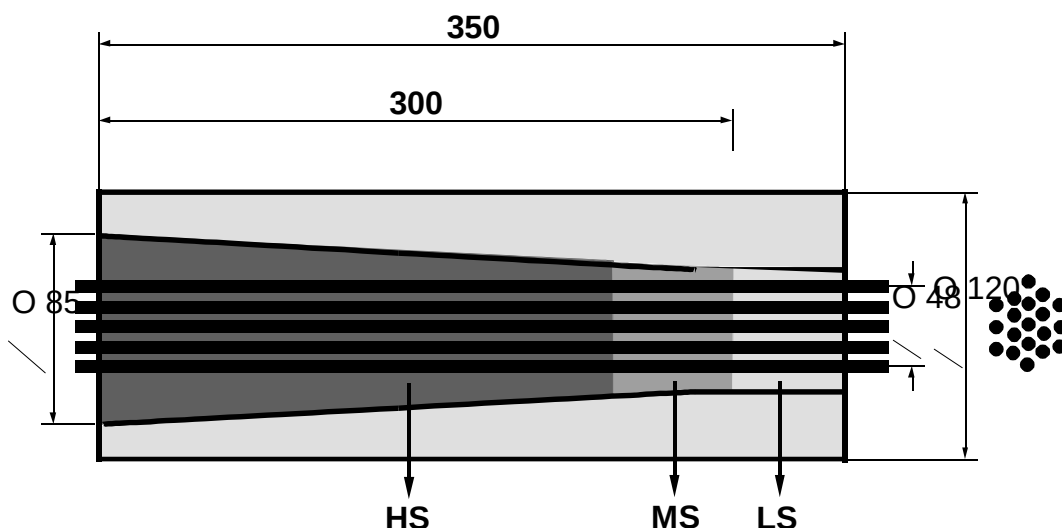


Figure 7 - Termination of a CFRP parallel wire bundle (cable). HS = high, MS = medium and LS = low stiffness of the load transfer media; right = loadside.

The termination of a 19-wire-bundle is shown in Figure 7. Many such bundles were tested at EMPA in static and fatigue loading. The results prove that the anchorage system described is very reliable. The static load carrying capacity generally reaches 92% of the sum of the single wires. This result is very close to the theoretically determined capacity of 94%. Fatigue tests performed on the above described 19-wire cables at EMPA showed the superior performance of CFRP under cyclic loads.

5.3 External post-tensioning

Cables made of composite materials can also replace steel cables for external post-tensioning. The advantages are:

- They do not corrode nor do they suffer stress corrosion.
- They are easy to handle.
- The materials exhibit outstanding fatigue behavior.
- There is a good efficiency for life-cycle cost but not for initial cost. A CFRP cable is 3 to 4 times more expensive, for example, than a comparable steel cable.

In a major R&D program Maissen (1997) analyzed the behavior of pre-stressed and post-tensioned concrete beams with CFRP wires or strands and made comparisons with beams using steel reinforcement. In an initial phase, a large number of tests were carried out on statically determinate rectangular beams, slab-strips and T-beams. After the most important parameters had been investigated and the basic feasibility of CFRP reinforced concrete confirmed, a second phase of testing was carried out on 17 m long two-span beams (Figure 8) made of post-tensioned concrete with an I-shaped cross-section. One test beam was post-tensioned using steel strands for comparison with the beams having the same geometry but post-tensioned with CFRP strands or wires.

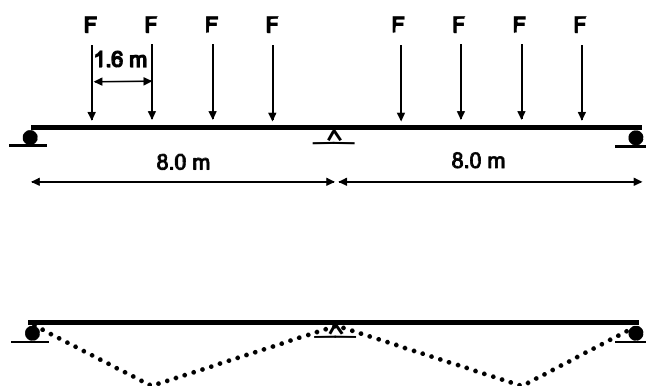


Figure 8 - Set up for loading tests with girders of I-shaped cross-sections post-tensioned with CFRP strands or parallel wire bundles.

The most important results of the program can be summarized as follows:

- The use of post-tensioned CFRP tendons is basically feasible for statically indeterminate systems.
- CFRP post-tensioned beams exhibit deformability characteristics that differ only slightly from steel reinforced concrete beams (Figure 9).
- For serviceability checks the same design methods can be used for CFRP as for steel post-tensioned concrete. However, for ultimate limit checks, the linear elastic deformation behavior of the CFRP tendons (no yielding) must be taken into account.

The external post-tensioning method is for repair more "powerful" than using external bonded FRP strips or sheets where a high degree of strengthening is needed. On the other hand this method is generally more expensive. The reason for the high cost is the relatively expensive anchorage systems.

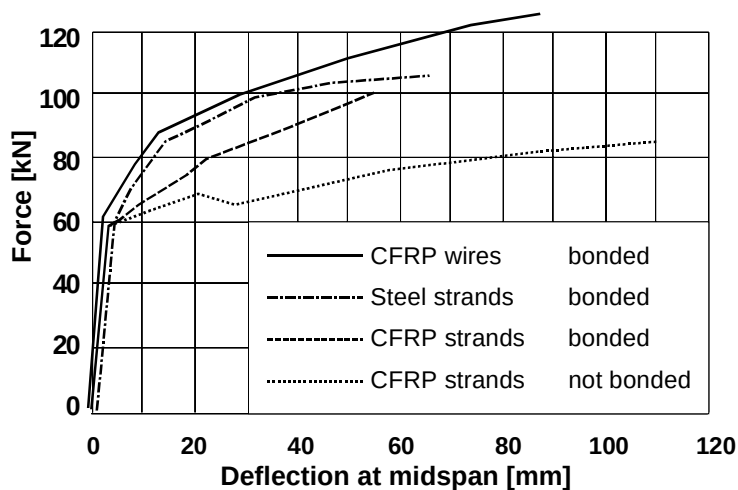


Figure 9 - Results of static loading tests according to Figure 8

5.4 Examples of bridges with CFRP cables

The cable-stayed Storchenbrücke Bridge in Winterthur: Apart from the price, the lack of extensive empirical data on CFRP cable continues to inhibit its general acceptance. Even if sponsors were to be found to finance the use of CFRP as an alternative to steel cable, no building authority would run the risk of approving large-scale use of a relatively untried material. This problem can only be solved by the incorporation of individual CFRP cable components in existing or new bridges and the monitoring of their performance in practice. Given that the linear thermal expansion coefficient of CFRP in longitudinal direction is zero, the parallel use of steel and CFRP cable is only possible in certain situations. The first suitable opportunity to test the material arose in 1996 in connection with the Storchenbrücke bridge project in Winterthur. Here, for the first time ever, two CFRP stay cables (from a total of 24) were installed (Meier 1996). As the main girders of this cable-stayed bridge (Figure 10) are very flexible, the varying thermal expansion coefficients of the cable materials did not pose insuperable problems. Both the CFRP cables (Figure 6) and the adjacent steel cables are being continually monitored by EMPA. So far results have met up with expectations and will become increasingly important in building confidence in years to come.



Figure 10 - Storchenbrücke in Winterthur

The cycle and pedestrian bridge over the Kleine Emme river near Lucerne: A further three bridge projects in Switzerland using CFRP cable followed in quick succession in late autumn 1998. Here too,

the aim was to gather experience from practical CFRP cable applications over a longer period of time in order to promote confidence in the material. Two CFRP cables produced by a BBR Systems Ltd/EMPA-team were used as chord members for the bridge over the Kleine Emme river, described in detail in (Burkhardt 1999). This project saw the first ever use of CFRP wires with integrated optical fiber sensors (Figure 11) for this kind of application (Nellen 1999). The continuous monitoring and optimization of future production processes for high-grade wire will obviate the need for time-consuming final quality checks. Where the projected application requires incorporation of sensors in the CFRP wires, these may be integrated at the production stage for process monitoring. For the bridge over the Kleine Emme, an EMPA-team used industrial production plant belonging to Stesalit to incorporate "endless" optical fiber sensors with Bragg gratings in the CFRP wires during the continuous pultrusion process. The expansion and temperature signals (Figures 12a and b) from the sensors were monitored and analyzed during production. Parallel-wire cables, each comprising 91 5-mm-thick CFRP wires, were then produced at Dübendorf, wound onto 2.5-m-diameter reels and transported to Emmen. Since October 1998 these cables have tensioned the bottom chord of the new bridge over the Kleine Emme river near Emmenbrücke. The sensors used for process monitoring during production now serve to monitor cable strain and thus the pre-stressing force in the bottom chord. These or similar techniques may be used for the continuous monitoring of main structural members in future. The bridge over the Kleine Emme is significant for the future development of CFRP cable in two respects: firstly in view of the innovative use of sensors and secondly due to the permanently high stress (1350 MPa) acting on the cables.

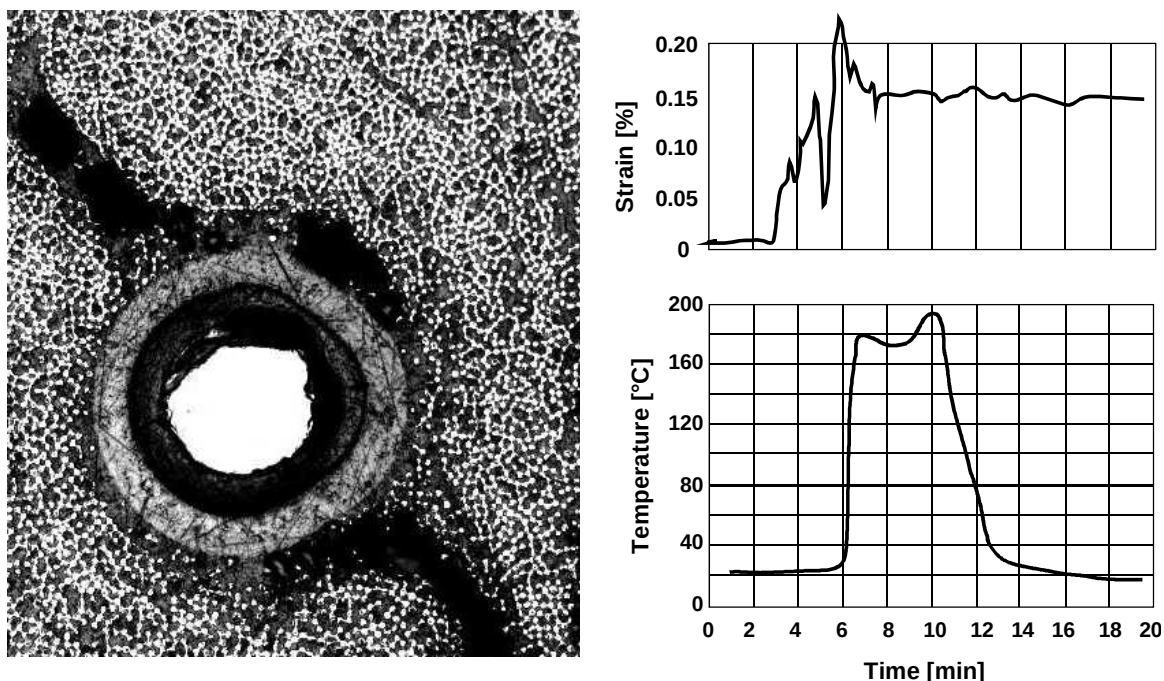


Figure 11 (left) - SEM picture of a unidirectional CFRP wire with a fiber Bragg grating sensor. The diameter of the sensor fiber is 100 µm.

Fig. 12a (top right): Strain signals from Bragg grating during pultrusion of 5 mm CFRP wires. Figure 12b (bottom right): Temperature signals during pultrusion.

Post-strengthening of the Verdasio bridge using external cables: The refurbishment of the Verdasio bridge (Figure 13) described by Guidotti (1999) represented the first attempt to make practical use of the results of extensive experiments performed at EMPA in recent years on continuous two-span beams pre-stressed with CFRP cable (Maissen 1997). Smooth wires were, however, used in place of the CFRP strands described in the relevant publication, with an anchorage system similar to that of the

Storchenbrücke bridge (Meier 1996). Four cables each comprising 19 5-mm-diameter CFRP wires were used for external post-tensioning to replace corroded steel cross-sections. The main problem in this application was to carry the cable round relatively tight bending radii of 4.5 m, as CFRP wires are sensitive to transverse pressure due to their composition. However, following a series of experiments (Figure 9) with 3.0 m radii, EMPA is confident that a satisfactory solution was found to this problem in Verdasio.

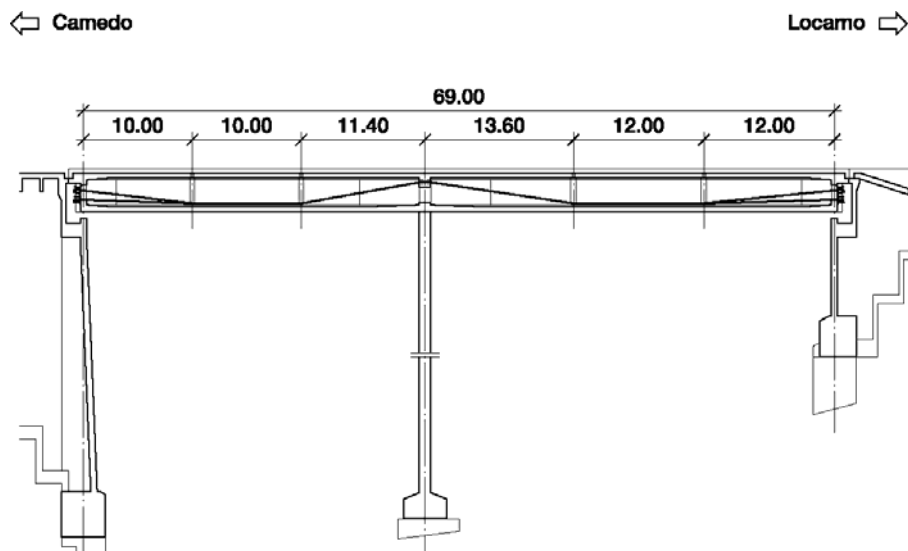


Figure 13 - Elevation of Verdasio bridge, post-strengthened using 4 external CFRP cables.

The cable radius over the center support is 4.5 m. All Measurements are in [m].

Pedestrian suspension bridge over the La Sarine river in Fribourg: The third structure to use CFRP in 1998 was the "Passerelle des Neigles" pedestrian suspension bridge in Fribourg (Veuthey 1999) spanning approx. 110 m with two CFRP cables. Each cable comprises 16 parallel CFRP strands made up of seven wires and protected by a polyethylene sheath. The anchorage devices were based on the system investigated by EMPA (Maissen 1997). Here too, the aim was to collect empirical data and build confidence in the material.

5.5 Future scenario for CFRP cable

What does the future hold? What are the prospects for CFRP cable in structural applications? Let us move forward to the year 2010. The long-term empirical data collected from the Storchenbrücke, Kleine Emme, Verdasio and Passerelle des Neigles bridges have met the requirements specified in 1998. The prices for carbon fiber have now fallen to a mere US \$ 10 per kg compared to US \$ 30 per kg in 1998. The pultrusion process for the production of CFRP wire is 50 times faster than in 1998 and production cost has sunk accordingly. Sustainable construction is now a serious issue among clients. In assessing projects, close attention is paid not only to the new-build or refurbishment costs but also to life-cycle costing. Given the above conditions, CFRP cable would probably be preferred in applications where steel cable would be prone to corrosion, stress-corrosion cracking and/or fatigue. For long spans, the low self-weight of CFRP cable and resulting substantial improvement in the equivalent modulus (Meier 1987/2) would also tip the balance in favour of this modern material.

6. NON-LAMINATED FRP STRAPS: FUTURE ELEMENTS FOR BRIDGE REPAIR?

A pin-loaded strap element as shown in Figure 14 may provide a practical means of joining different components and transferring significant forces between them. This element consists of a unidirectional FRP lamina wound around endpins in a racetrack manner. No machining of holes is required. The layers in the composite are cured to produce a solid laminate. Circular pins transfer the tensile load to the components being joined. Such straps have many desirable characteristics, including high tensile load capacity, low weight, low thermal conductivity and low thermal expansion. As a result, laminated pin-loaded straps have been used in many different structural applications, such as temporary bridges. They are ideally suited for bridge repair due to the very simple loading technique with pins. However, both experimental and theoretical studies (Winistörfer 1997) have revealed 'high' stress concentrations next to where the strap leaves the pin. The effect of these concentrations is to reduce the load at failure considerably compared to that of the straight solid laminate, as determined by a standard coupon test.

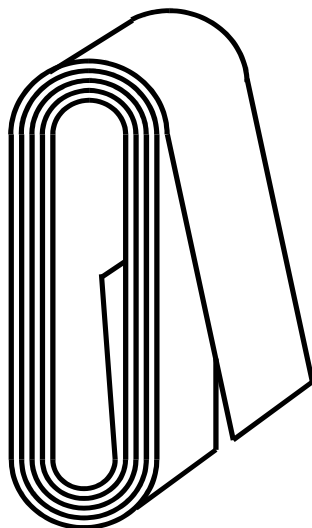


Figure 14 - A pin-loaded strap element

One means of reducing these undesirable stress concentrations is to replace the solid laminate by the non-laminated equivalent (Winistörfer 1999). In the "new" strap there are a number of non-laminated layers formed from a single, continuous, thin, thermoplastic matrix tape reinforced with unidirectional carbon fibers. In the non-laminated system the tape is wound around the pins and only the end of the outside, final layer is fixed. It can be fixed to the previous layer or to the surrounding structure using different end fixture techniques. A non-laminated (unconsolidated) strap system of this type enables the individual layers to move relative to each other. The undesirable stress concentrations are therefore reduced because this structural form has greater flexibility. Careful control of the initial tensioning process allows interlaminar shear stress concentrations to be reduced in order to achieve a uniform direct strain distribution in all layers through the thickness.

Apart from improving stress distribution, winding can easily be performed on site so the system allows greater flexibility in terms of adjusting tendon length. This flexibility allows the system to compensate for the dimensional tolerances of the components to be connected. The cost effectiveness of the non-laminated strap is also superior to the laminated strap because the consolidation process is not required and there is a low cost anchorage system available.

This system could have an excellent future in bridge repair for active shear strengthening, for the end anchorage systems of tensioned CFRP strips, for flexural strengthening and for external post-tensioning. There is a high probability that "non-laminated FRP straps" will be as strong as cables for external post-tensioning, and much cheaper.

7. CONCLUSIONS

CFRP strips and sheets are members of a growing family of non-metallic tensile elements based on high-strength fibers. These non-corrosive tensile elements composed of endless parallel carbon filaments in a polymer matrix offer a highly promising alternative to steel plates for strengthening applications where long-term durability is required. Carbon fiber composites combine the qualities of very high strength with an outstanding fatigue performance and light weight for ease of handling. The strips and sheets are resistant to practically every type of environmental attack likely in or around bridge structures. This application of CFRP strips and sheets is already almost routine for structural repair in several western European countries and in certain parts of Asia and North America.

The bending and shear strengthening method with pre-stressed CFRP strips is not so well established yet. It will take at least another year of development work before it is suitable for practical applications since the pre-stressing method described above is still complicated to use and installation techniques, both manual and automatic, have yet to be perfected. These include surface preparation, pre-stressing, placing and bonding, forming end anchorages and vacuum bonding. Automatic application methods will offer advantages in hazardous areas, where there is danger from traffic and will reduce traffic management and traffic delay costs.

From a global view the use of FRP materials in bridge repair is still in its infancy, but there are very clear indications that it will be an excellent choice for a multitude of rehabilitation projects on bridges.

CFRP cables for the "powerful" external post-tensioning are not yet a cost-effective solution, the main reason being the high cost of production of the anchorages. It is highly probable that "non-laminated CFRP straps" will play a key role in post-strengthening of bridges providing universal, easy to anchor and affordable tensile elements for bridge repair. The CFRP straps may also offer an interesting alternative for pre- and post-tensioning of new structures.

The future use of CFRP cable for the anchorage of off-shore oil platforms will of course be crucial. A single platform would probably require some 600 t of CFRP. Should such a breakthrough occur, it is conceivable in the long term that CFRP cable might be used to engineer bridges with long spans (Meier, 1987/2).

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