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Try the impossible to achieve the possible

Urs Meier

Empa, Swiss Federal Laboratories for Materials Science and Technology, Dubendorf, Switzerland

ABSTRACT: "Non Finito" is in fine art a sculpting technique literally meaning that the work is unfinished. This is in a similar way the case for this paper. It only sculpts ideas and is leaving most of the needed research and development work as well as the realization the next generations of researchers, engineers, and builders. Topics are: materials by combination for lightweight girders, post-strengthening with pre-tensioned carbon fiber reinforced polymers (CFRP) including automated application, airbags that might prevent structural collapses due to earthquakes, adaptive wind fairings, hybrid cable arrangements for cable stayed bridges, and CFRP for super long-span bridge design.

1 INTRODUCTION

Hesse (1960), a world famous German-Swiss poet, novelist, and painter, best-known for his works "Steppenwolf", "Siddhartha", and "The Glass Bead Game", who received in 1946 the Nobel Prize in Literature said: "You have to try the impossible to achieve the possible". History of engineering is in many cases giving evidence to this quote.

The following sections represent first an example of a challenge in the past that has become a successful application in practice and afterwards provocations that have not yet been successful in civil engineering.

"Non Finito" is in fine art a sculpting technique literally meaning that the work is unfinished. Non finito sculptures appear unfinished because the artist only sculpts part of the block, leaving the figure appearing to be stuck within the block of material. There is some similarity with this paper. Its goal is to sculpt "crazy" ideas within the scope of advanced materials and systems in civil engineering that might be interesting for future R&D work and might show successful applications in ten, twenty or thirty years from now.

2 A PROVOCATION IN THE PAST HAS BECOME A SUCCESSFUL APPLICATION

The "provocation" of post-strengthening civil structures with carbon fiber reinforced polymer (CFRP) strips was for the first time unfilled in an oral presentation at ETH, the Swiss Federal Institute of Technology in Zurich in 1985 by the author. An appropriate short feasibility study was published two years later, Meier (1987/1). The reactions of the listening and reading audiences were extremely mixed from "absolutely impossible and crazy" to "why not?"

CFRP strips were first applied in real life 1991 to strengthen the Ibach Bridge near Lucerne. Consumption of CFRP at that time totaled a mere six kilogram per annum. Today about one quart of the worldwide carbon fiber production is used in construction, mostly for post-strengthening purposes, i.e. approximately 12'000 tons per annum. Considering the tonnages of steel used in construction, this seems nothing but we have to keep in mind that the strength of such strips is above 3000 MPa and the density is only 1.5 t/m^3. Beside that it is a niche application. The sentence “Never before has a post-strengthening method done so much with so little“ coined by the writer during a lecture tour trough the Unites States in 1997 for the promotion of CFRP rehabilitation systems, symbolizes the situation. The material's excellent corrosion resistance, extremely high strength, high stiffness, good fatigue performance and low bulk density have already enabled it to supplant steel for these applications in much of Europe, Asia, the Americas, and Australia. In most cases, 1 kg of CFRP can match 30 to 35 kg of steel in terms of strength. The material's low density makes its application so straightforward compared to steel that the additional cost (CFRP composites are about ten times more expensive per unit volume) is more than recouped by labor savings due to the extreme ease of handling. The first-rate material properties are effectively thrown in as an added value. CFRP strip or wet lay-up is best applied as “structural wallpaper" using a rubber roller; unlike externally bonded steel plate, it does not require support or contact pressure while the adhesive resin cures. The overcoming of this challenge brought construction industry a modern, meaningful method for post-strengthening of structures.

3 MATERIALS BY COMBINATION: LIGHTWEIGHT NON CORROSIVE BEAMS

Since decades face parking garages, in places where deicing salt has been used, severe corrosion problems with the steel reinforcement of the concrete decks. In many cases replacement was and still is needed. This was the background for the development of a lightweight non corrosive girder system within an Empa/MIT Collaborative Research Program. The girder is based on the three materials GFRP (Glass Fiber Reinforced Polymer), CFRP and concrete, Deskovic et al. (1995/1).

Let’s consider the case of a GFRP pultruded beam section. Thin-walled box sections are the most efficient for beams, and are, in fact, very commonly used in structural applications of pultruded profiles. However, they suffer from some disadvantages, including the following: (i) The compressive flange is considerably weaker than the tensile flange, because GFRP has a compressive strength of about half of its tensile strength and because of local buckling phenomena due to the low modulus of elasticity; (ii) failure is usually catastrophic without warning, because composite materials are linear elastic to failure; and (iii) the design is usually governed by stiffness (because of the relatively low stiffness of GFRP), resulting in a need for excessive use of composite material to satisfy certain displacement requirements.

In view of this, a novel and more efficient design of composite box sections is possible, driven by the following considerations: First, the compression stresses in the section should be carried by a material with the highest compressive strength and stiffness to cost ratio, and therefore the GFRP flange could be eliminated and substituted by a layer of concrete. Second, another composite material with a failure strain less than that of GFRP could be added to the section's tension zone, so that it will be the first element to fail, thus giving some warning of an imminent collapse (“pseudo ductility”). Because this element will be part of a flange, it should preferably also possess a high stiffness to increase the section's rigidity. A thin layer of externally epoxy-bonded high modulus CFRP appears to be the best candidate material for this purpose. Use of this material will also enhance the member's creep and fatigue behavior, given that CFRP is practically creep-free and has an excellent response to fatigue loading.

Three large-scale beams were fabricated and tested in flexure to verify experimentally the short time strength, stiffness, and ductility characteristics of the hybrid FRP-concrete members. Additionally a total of four similar hybrid beams were fabricated and tested in flexure under sustained and alternating loading.

The fabrication procedure, which included filament winding of GFRP for the box, casting of concrete, bonding of CFRP, and installation of shear connectors is described in detail in Deskovic et al. (1995). All beams were 3.2 m long with 300 mm deep and 180 mm-wide GFRP box cross sections (outside dimensions), and 53-mm-thick concrete layers. The cross sectional area of each of the two CFRP strips at the bottom was only 10 mm^2 (Figure 1).

The results of a short time loading test of such a hybrid beam are presented in Figure 1. There was a typical CFRP tensile fracture at mid-span followed by concrete crushing at a higher load. CFRP failure occurred so suddenly that the laminate debonded almost completely from the bottom flange, which remained unharmed. Experimental, analytical and finite-element results are found in good agreement, Deskovic et al. (1995/1).

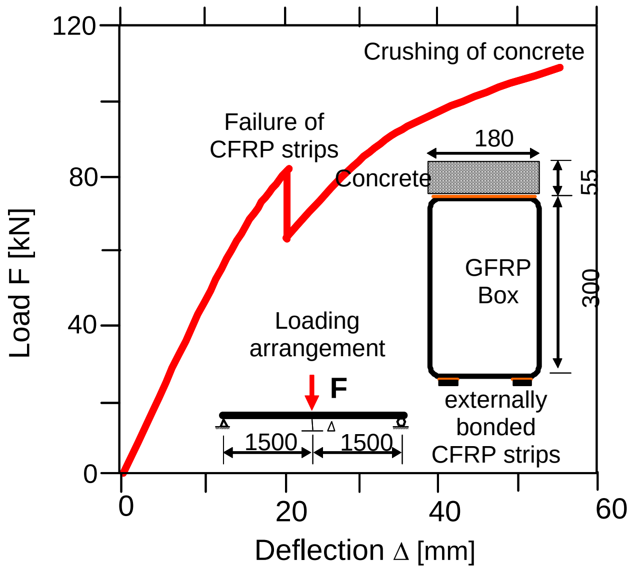

Figure 1. Short time loading test of hybrid GFRP/CFRP/concrete beam; all measurements in [mm].

The combination of different fibre reinforced polymer materials with concrete appears to be a feasible way of producing efficient and cost-effective hybrid members. These members possess many desirable mechanical behaviour characteristics, such as pseudo ductility and high strength and stiffness, while maintaining a low weight. The main features of the proposed concept for hybrid sections can be summarized as follows: a concrete layer substitutes the GFRP compressive flange of traditional pultruded box sections, thus reducing the materials cost and in-creasing the stiffness; and the bottom flange is made by a combination of two composites (GFRP and CFRP), one failing in tension earlier than the other (and, possibly, in a gradual manner), serving the role of a sensor that indicates an imminent collapse. The design of single-span hybrid FRP-concrete components based on short-term behaviour is a relatively straight-forward task once all the possible failure mechanisms have been identified and the design requirements and constraints have been set up by Deskovic et al. (1995/1).

The experimental results emphasized the important role of bond at the GFRP-concrete interface; the ideal bond should be provided by the combination of adhesives and mechanical shear connectors.

Fatigue testing involved two beams, which were subjected to four-point alternating loading. The constant amplitude loads were applied in a sinusoidal pattern at a frequency of 4.2 Hz. The loading range was 20 to 60 kN for beam No. 4 and 40 to 80 kN for beam No. 5. Fatigue testing was interrupted after about 3’300’000 cycles for beam No. 4 due to failure of one of the supports, and after about 4’400’000 cycles for beam 5 due to fracture of the high modulus CFRP strips at a strain equal to about 0.84% , Deskovic et al. (1995/2).

Overall, the hybrid FRP-concrete beams, described in this section, possess good response characteristics to long-term loading, and the models described in Deskovic et al. (1995/2), along with those given in Deskovic et al. (1995/1), can be used as valuable tools for the analysis and optimum design of the members under both short-term and long-term loading actions.

What is missing for a successful commercialization for such hybrid beams? Answer: a highly automated fabrication process.

4 ACTIVE SHEAR STRENGTHENING

Extensive work has been carried out within the last thirty years investigating the use of FRPs for flexural strengthening of reinforced concrete structures. In comparison relative little has been done for the shear strength enhancement and especially for the active (post-tensioned) shear strength enhancement of reinforced concrete structures. Researcher at Empa in Switzerland (Lees et al. 1999, Meier et al. 2001, Winistoerfer et al. 2001) and the University of Cambridge in England (Lees et al. 2002 and 2011) developed a system applying post-tensioned, non-laminated, carbon fiber-reinforced polymer (CFRP) straps as external shear reinforcement for concrete structures. Experiments were carried out on not post-strengthened control beams and beams post-strengthened with external post-tensioned CFRP straps. It was found that the ultimate load capacity of the strengthened beams was significantly higher than that of the control specimens. One example is presented in Figure 2. These results were and still are very promising. With little CFRP a lot of structural improvement has been reached. Control beams failed in general in a brittle shear failure. The beams post-strengthened with post-tensioned CFRP straps failed after successive failure of CFRP straps and yielding of the internal steel rebars in a quasi ductile manner in flexure.

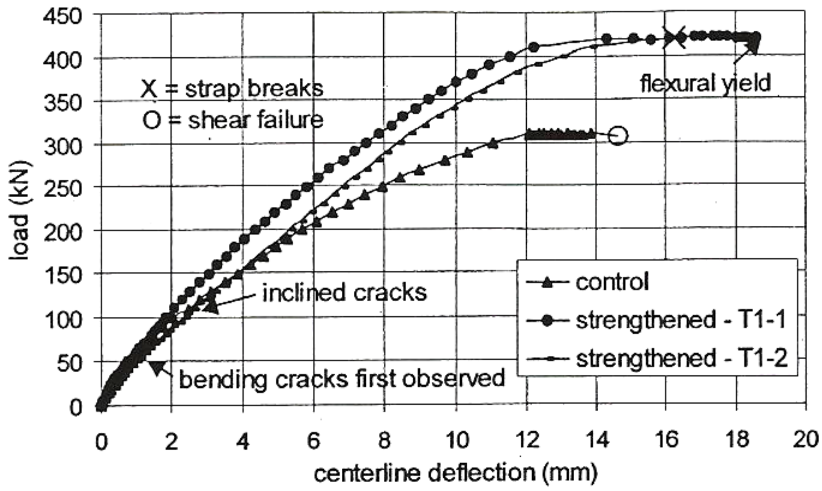

Figure 2. Load deflection curves (taken from Lees et al. 2002).

Motivated by this first success a PhD-thesis about the post-strengthening of shear walls was initiated. Stenger (2001) demonstrated in a comprehensive study the efficiency of shear strengthening with post-tensioned CFRP straps for shear walls. His thesis covers five large scale experiments with steel reinforced concrete shear walls. All of them had the same dimensions and the same internal steel reinforcement. The total length of a wall was 3840 mm, the depth 1200 mm and the thickness in the central part 150 mm and at the clamps 800 mm. The dimensions of the observed shear zone were 2240 mm in length, 1200 mm in depth and 150 mm in thickness. The flexural internal steel reinforcement consisted of twice 6 rebars diameter 26 mm symmetrically on bottom and on top. The steel stirrups of diameter 6 mm had a distance of 375 mm in the shear zone.

Four of these five shear walls were post-strengthened each with four vertically applied external CFRP straps of the same CFRP tape and with the same cross section. Those straps were build up of tapes with Toray T700S carbon fibers and a thermoplastic matrix of Polyamide PA12-. The tapes had the following properties: elastic modulus E 130 GPa, strain at failure 1.2 %, width 12 mm, thickness 0.16 mm, cross section: 1.92 mm^2. The load carrying capacity of each strap wound of 25 layers was 125 kN and the cross section (twice 25 layers) 96.0 mm^2.

The CFRP tapes were wrapped around semi-elliptical interface pad elements that were placed on the bottom and top faces of the shear walls to be post-strengthened. They were tensioned by lifting the top or bottom interface pad using a jacking system. The transverse post-tensioning force was applied to the concrete by overstressing the strap, inserting a spacer between the interface pad and the beam and then releasing the tensioning force (see Figure 3).

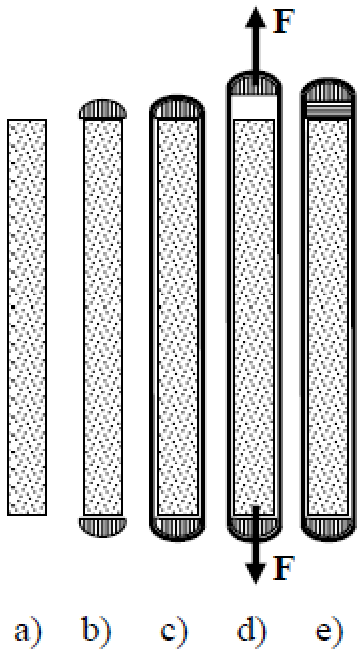

Figure 3. Conception for post-tensioning with CFRP straps shown on the example of a steel rebar reinforced concrete shear wall: a) cross section of shear wall, b) addition of semi-elliptical inter-face pad elements that are placed on the bottom and top faces of the shear wall, c) the layers of the CFRP tapes are wound around the pads and shear wall and the end of outermost layer is welded to the second outermost layer, d) the straps are tensioned by lifting the top interface pad using a jacking system, e) the post-tensioning force is applied to the concrete by overstressing the straps, inserting spacers between the inter-face pad and the shear wall and then releasing the tensioning force.

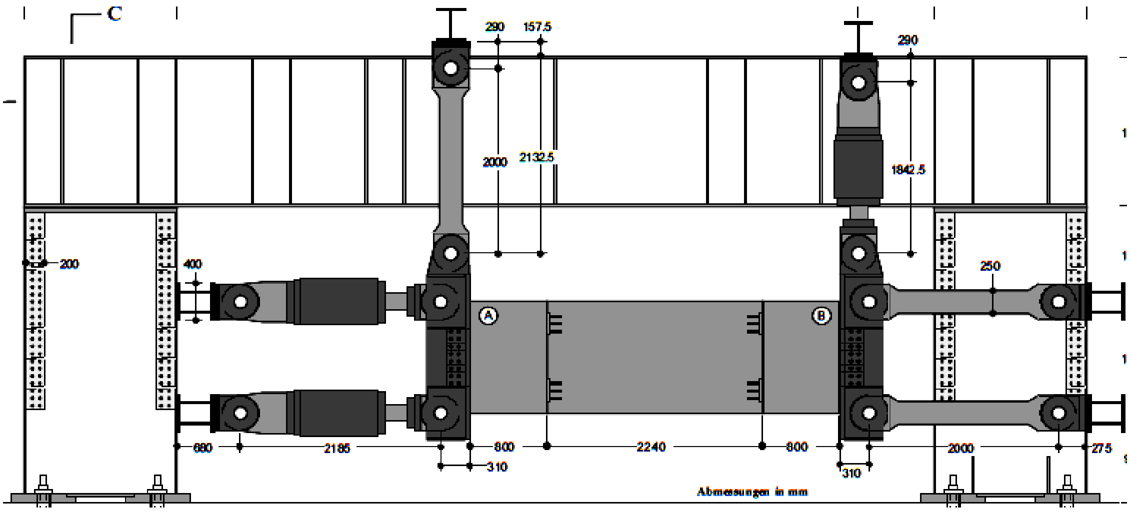

Figure 4. “Beam-Element-Tester” developed by Professor Peter Marti at ETH Zurich.

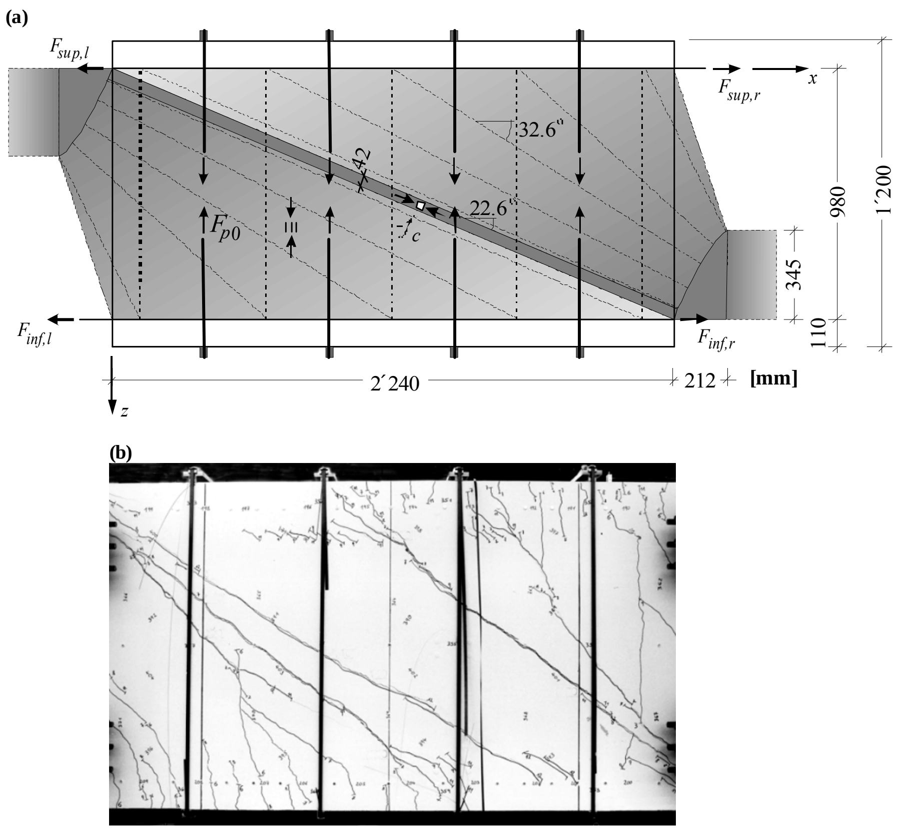

Figure 5. Shear wall ST1 loaded up to −635 kN: (a) on top: discontinuous stress field; (b) on bottom: crack distribution.

Table 1. Results of shear experiments on reinforced concrete elements.

Wall element number	cracking shear load [kN]	ultimate shear load [kN]	max. vertical displacement [mm]
ST2: Reference, not post-strengthened	-210	-465	-3.85
ST3: 5 kN-post-tensioned	-250	-479	-11.10
ST1: 70 kN-post-tensioned	-305	-703	-13.01

The shear walls were on both sides rigidly clamped and loaded in a large "Beam-Element-Tester" (Figure 4) developed by Professor Peter Marti at ETH Zurich, Stenger (2001). In Table 1 selected results of the shear experiments are presented. The wall element number ST2 was the reference and was not post-strengthened with CFRP straps. ST3 was post-strengthened with 4 CFRP straps with a distance of 500 mm. Each strap was post-tensioned with only 5 kN. ST1 (Figure 5) had the same arrangement like ST3; however each strap was post-tensioned with 70 kN.

The general failure mode was in the case of all three shear walls brittle. However in the case of the post-tensioned walls the failure of the external CFRP straps was of a progressive type, mostly starting from the innermost layer. Gradual longitudinal splitting of the CFRP tapes and the breaking of fiber bundles could easily be ob-served and heard quite a while before the abrupt and noisy collapse.

Remarkable within the results of Table 1 is the difference between the post-tensioned (70 kN each strap) and the not really post-tensioned (only 5 kN each strap) application. 70 kN-post-tensioning helped to increase the ultimate shear load for 51%. The same amount of CFRP in the same arrangement reached in the "not" post-tensioned (in reality 5 kN) shear wall only an in-crease of 3%. This demonstrates the high effectiveness of post-tensioned applications.

In the experimental phase the semi-elliptical pads were made of steel. It does not make much sense to propagate CFRP as a non-corroding material and to use steel for the anchorage of such elements. Meanwhile the semi-elliptical steel pads have been success-fully replaced by pads made of CFRP reinforced mortar, polyethylene or GFRP, Meier (2012/1).

Motavalli et al. (2011) are presenting advanced developments for pre-stressed applications of CFRP for the strengthening of reinforced concrete structures.

5 IS THERE A FUTURE FOR AUTOMATED APPLICATION OF PRE-TENSIONED CFRP STRIPS FOR POST-STRENGTHENING?

The post-strengthening method developed in the 1980s Meier (1987/1) for the refurbishment of structures using externally with epoxy bonded carbon fiber reinforced polymer (CFRP) strips is now in common use. But today in construction in most applications only about 15% of the high strength of the CFRP strips is used. The price of carbon fiber is likely not to drop still further as it happened in the last forty years. Increasing expenses for process energy may even increase the price of the fibers. The focus of future developments should therefore be on a better exploitation of the potential offered by CFRPs. A better exploitation is possible when the CFRP strips are pre-tensioned. Figure 6, Meier (2012/1), shows such a pre-tensioning device for automated ap-plication. However, this approach alone will not yet be sufficient to be able to compete success-fully in large projects against classical post-strengthening techniques.

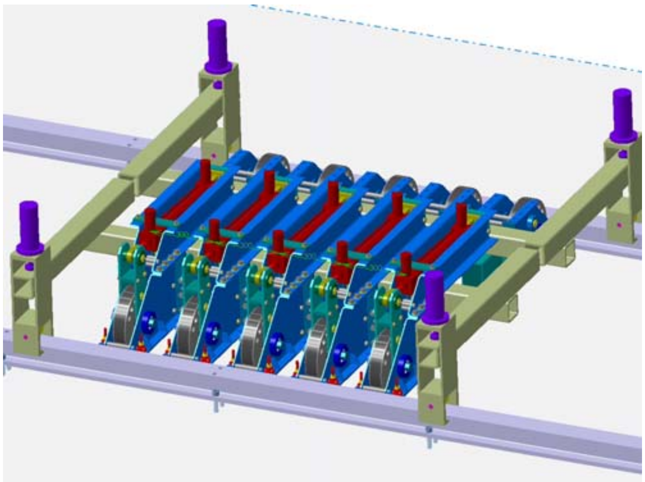

Figure 6. Automated device for simultaneous application of five pre-tensioned CFRP strips; Meier (2012/1).

Since Henri Ford introduced in 1913 the moving assembly line, tremendous progress has been made in automation of car production. This is by far not the case for construction and aircraft industry. In both industries still most of the work is done by hand. It will change in aircraft industry. “Flex Track”, an innovative manufacturing technology developed by the Division “Commercial Airplanes” of Boeing is a portable and low-cost, programmable, numerically controlled tool that creates and countersinks fastener holes in a single pass. It takes its name from the flexible rails that it rides over the component it is drilling. Assembled from interlocking track segments, these rails are secured to the component via suction cups (Figure 7). Workers do not have to place them exactly, because the system, which is small and light enough for one person to lift, knows precisely where it is in relation to the structure, Thompson et al. (2005).

This versatile tool can work on highly contoured structures. It has built-in visual and vacuum systems that let it orient itself, follow cues, and clean up its own shavings. As it glides over the part being drilled, its computer program analyzes what the tool "sees" to locate and drill holes within a couple of hundredths of a millimeter of the desired location. The system is e.g. in use for the production of the Boeing 777 and 787 to help assemble this commercial airplane's wing panels and join its fuselage sections. In future it will also be equipped with multi-head CFRP tape-laying machines.

Figure 7. Application of the Flex Track system on the fuselage of a airplane equipped with a drilling machine, Thompson et al. (2005), courtesy of Boeing.

Already in 1995 a highly automated system for the preparation of the concrete surfaces and the adhesion of CFRP strips was in Meier (1995) proposed. Meanwhile nothing happened in construction industry. It is only to hope that the introduction of such automated systems into construction industry does not take so much time as for the use of carbon fibers. First applications in aircraft industry took place in the early 1960ties. Those in construction industry thirty years later!

6 AIRBAGS MAY PREVENT STRUCTURAL COLLAPSES DUE TO EARTHQUAKES?

The airbag for automotive applications traces its origin to air-filled bladders out-lined as early as 1941. Patented ideas on airbag safety devices began appearing in the early 1950s. U.S. patent 2,649,311 was granted on August 18, 1953, to John W. Hetrick for an inflated safety cushion to be used in automotive vehicles. Early air bag systems were large and bulky, primarily using tanks of compressed air, compressed nitrogen gas (N_2), freon, or carbon dioxide (CO_2).

The first use of large airbags for landing of space vehicles were Luna 9 and Luna 13, which landed on the Moon in 1966. The Mars Pathfinder lander employed an innovative airbag landing system. This prototype successfully tested the concept, and the two Mars Exploration Rover Mission landers employed similar landing systems. The US Army has incorporated airbags in its Black Hawk and Kiowa Warrior helicopter fleets. Airbags have been proven very effective in preventing motor vehicle accident injuries. Why should they not also be used in the case of earthquakes to protect lives and prevent structural collapses?

In 2002 Hans-Joachim Kuempel was awarded the US Patent 6,360,384 B1 “Earthquake proof sleeping place”. This place comprises a base frame with arcuate guiding tubes. From each guiding tube, an arcuate supporting bar may be telescoped. The supporting bars are connected by a longitudinal bar and form a protective frame therewith. Two protective frames can be raised out from the guiding tubes from opposite sides to close above the bed. The protective device requires little space since the base frame is mostly arranged under the bed. An earthquake sensor causes the protective device to be triggered, in which event the protective frames come up. This proposed system does not use airbags, however, similar to those the protection is initiated by a sensor system.

Bonjar (2005) coined the expression Earthquake Airbag (EA). Based on many scientific reports, fatality rates are lower in automobiles equipped with airbags than unequipped ones. Accordingly, it was postulated that similar devices can be adopted in buildings to protect people and lower human casualties in building crashes. The safety advantage of EAs would be that they can reduce impact injuries upon indoor people from falling debris in earth-quakes.

Why one should not try to avoid building crashes and thereby save lives and buildings at the same time? The author was in 2003 member of a peer review committee of the DFG (German Research Foundation) concerning structural investigations on the Hagia Sophia in Istanbul and visited this monument in 2004. During this visit the idea was born to think about the use of airbags to protect structures from earthquakes.

The current structure of Hagia Sophia was completed in 537. But it suffered under severe seismic loadings, e.g. in the years 553, 557, 865, 869, 986, 989, 1344, 1346, 1462, 1500, 1509, 1719, 1754, 1766, 1894 and 1999. The dome had to be reconstructed several times during the centuries. Based on recent seismic activity and the history of the North Anatolian fault south of Istanbul there is a high probability that Istanbul with the Hagia Sophia will be hit with a major earthquake over the next three decades. In Istanbul many structures have been post-strengthened against seismic loading with advanced composite materials. In the case of Hagia Sophia, e.g. it is impossible to cover the golden mosaics with black carbon fiber reinforced polymers. Seismic retrofitting of historic structures is anyway being between the poles of curators of monuments and structural engineers a difficult task.

Many wonder how Hagia Sophia will fare in the next great earthquake. The region just south of Istanbul is expected to experience tremors of equal or greater magnitude to the Izmit earthquake in the next few decades, Hughes (2006), Hubert-Ferrari (2000). In 1991, a team of Turkish and US researchers fitted Hagia Sofia with several vibration sensors. From data gathered during micro earthquakes, Çakmak created three-dimensional computer simulations that could predict how the building might move during a large earthquake, Çakmak et al. (1995). Results obtained using LUSAS finite element modeling indicate that damage will occur initially in the west and east semi-domes before proceeding to the arches and main dome. The model shows also that when hit by a magnitude 7.5 tremor; the walls of Hagia Sophia will tremble and sway dramatically back and forth. The tops of its arches will feel the most stress. But the dome will remain unscathed, and the heritage structure will stand. If the earthquake is greater than 7.5, there is a high risk for a collapse.

How can this most valuable structure be preserved with airbags in the case of such a severe earthquake? Even if it would be possible from a technical point of view to have, e.g. for the central part of Hagia Sophia one huge powerful airbag being folded on the ground this cannot work. In the case of an earthquake in presence of visitors inside the building the airbag's deployment might protect the structure; however it would kill the persons being between the airbag and the dome. Therefore a support structure is needed to carry the folded airbags. For the central dome this could be a centrically arranged horizontal circular ring with a large U-profile supported by telescope-like columns. The opening of the U-profile would be on top. Inside the U-profile there are large folded tubular airbags. The initial height of this ring would be about three meter above the ground floor. It would only little disturb the appearance of the wonderful architecture. As soon as the seismic sensors identify a strong earthquake they trigger the gas generators and deploy the airbag tubes. At the same time the telescopic columns would lift up the whole system. The system would perform as an adaptive structural support and damping the oscillations caused by the seismic activity. For the semi-domes and the arches (Figure 8) semi-circular ring U-profiles and linear U-profiles respectively would be used.

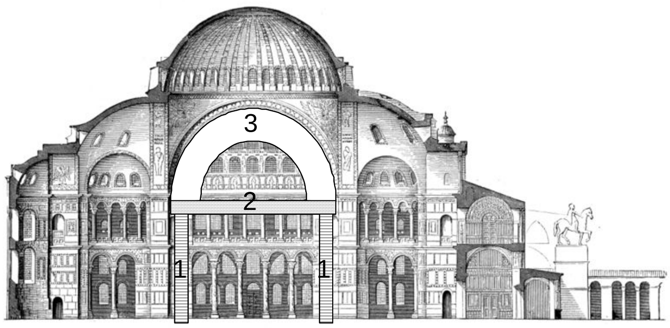

Figure 8. Cross section of Hagia Sophia: Example for the seismic protection of an arch. 1 = telescopic CFRP columns; 2 = linear CFRP U-profile, open to the top; 3 = deployed tubular earthquake airbag.

These systems could not fully prevent the falling of single stones between the tubes. The feasibility of the application of such earthquake airbags had first to be checked on the existing computer models. If the answer is positive experiments on smaller scale had to be provided. If all these investigations should give positive answers the great advantage of earthquake airbags would be a fast installation and that there is no irreversible intervention on the historic structure needed. For the first application of this kind of “rehabilitation” the cost would be high due elevated R&D expenses. However further applications should be very fair priced. A special challenge would be the design of stable telescopic columns.

7 ADAPTIVE WIND FAIRINGS?

Due to low structural damping and relatively low mass, long-span suspended bridges become susceptible to vibrations caused by winds. That was the reason for the collapse of the Tacoma Bridge in 1940. The Bronx–Whitestone Bridge in New York used the same general design as the Tacoma Narrows Bridge. In 1943 6000 tons heavy trusses were installed on the Bronx–Whitestone Bridge on both sides of the deck to weigh down and stiffen the bridge in an effort to reduce oscillations after the Tacoma Narrows Bridge disaster. These trusses detracted from the former streamlined looking span. In 2003, the Metropolitan Transit Authority restored the classic lines of the bridge by removing the stiffening trusses and installing instead glass fiber reinforced polymer (GFRP) wind fairings along both sides of the bridge deck. The lightweight GFRP fairings are triangular in shape giving it an aerodynamic profile. The removal of the trusses and other changes to the decking cut the bridge's weight by 6,000 tons, some 25% of the mass suspended by the cables, Meier (2002).

In future in similar cases an innovative wind-induced vibration mitigation strategy based on active control of the bridge’s aerodynamic profile might be applied. An array of adjustable flaps, Meier (2002), might be installed along both edges of the girder and their angular position is controlled as a function of the current dynamic state of the structure and the local wind field measurement. This information is shared with other similar units distributed over the whole length of the bridge through wireless networking, Bischoff (2009). The characteristics of the interaction between the wind field and the underlying structure with the additional degrees of freedom introduced by the flap system result in complex models and associated control strategies. The need for real-time coordination between various units, leading to an active control of the global aerodynamic profile of the full bridge model with the constraints introduced by the mechanical structure makes the problem of mitigating vibrations at the perturbation source extremely challenging. Also for this challenge the cost for the first application of this kind of “rehabilitation” would be high due soaring R&D expenses. However further applications should be very fair priced.

The Structural laboratory of Empa is under the leadership of Prof. Masoud Motavalli since more than a decade successfully focusing on the topic “vibration mitigation”, Gsell et al. (2004), Motavalli (2004, 2006), Weber et al. (2002, 2004, 2005, 2009, 20011, 2013). The R&D results look very promising and many of them are already transferred into many practical applications, e.g. Weber et al (2007).

In modern architecture it is more and more fashion to design and construct lean high raised towers. Already in the past some of them faced oscillation problems due to aerodynamic excitations. Instead of the subsequent installation of tuned mass dampers an adaptive outer skin made of electro-active polymers might resolve this problem, Jordi (2011). Such an outer skin may not only improve the aerodynamic properties but also serve a curtain wall with special thermal, acoustic, and optical properties. Such multifunctional materials will play an important role in rehabilitation and new construction in future.

8 HYBRID CABLE ARRANGEMENTS FOR CABLE STAYED BRIDGES?

CFRP cables have a very high specific strength and stiffness, do not corrode, perform outstanding under fatigue loading, do not relax, do not suffer stress corrosion and are very light weight. Light weight is on one hand a great advantage for the stiffness performance of long stays and on the other hand for extremely long span bridges (will be discussed in next section). The stiffness of a cable-stayed bridge depends largely upon the tensile stiffness of the stay cables. The displacement of the end of a free-hanging stay cable under an axial load depends not only on the cross sectional area and the modulus of elasticity of a cable but to a certain extent on the cable sag, as described by Ernst (1965). The relative equivalent modulus of elasticity E_e/E of a cable is defined as:

$$\frac{E_e}{E} = \frac{1}{1 + \frac{(\rho\, l)^2}{12\,\sigma^3} E} \tag{1}$$

where E_e is the equivalent modulus, E the modulus of elasticity, l he horizontal span of the cable, ρ the density of the cable material and σ the applied cable stress. Figure 9 gives a comparison between steel and CFRP stays.

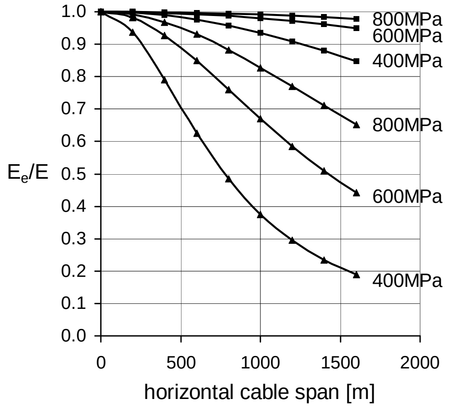

Figure 9. Relative equivalent modulus of elasticity E_e/E versus the horizontal cable span for CFRP and steel cables according to Eq. 1. The applied cable stress is given as a parameter. The constants are: E_{steel}= 210 GPa; E_{CFRP}= 165 GPa; ρ_{steel}=7.8t/m^3; ρ_{CFRP}=1.6t/m^3. The three curves at the top end (▪) represent CFRP stays and those three at the bottom end (▲) steel stays.

Professor Fritz Leonhardt, one of the fathers of modern cable stayed bridges, always stressed the importance of high stay cable stiffness. This was the reason he initiated the replacement of locked coil strand cables by parallel wire bundles, offering higher axial stiffness, Andrä and Saul (1974). Leonhardt's largest bridge main span in the mid 1950ties reached 260 m in Düsseldorf. For such spans was the equivalent modulus of elasticity in practice of subordinate importance. Today we face main spans of more than 1100 m and as can be seen in Figure 9 the equivalent modulus gains relevance. Beside that CFRP cables can be operated on a higher working stress than steel cables.

The cross section of stay cables is typically sized such that the maximum axial stresses in a stay cable under service conditions at SLS (Serviceability Limit States) do not exceed specified limits. In the past, typically the maximum axial stress was limited to 45% GUTS (Guaranteed Ultimate Tensile Strength). According to fib Task Group 9.2, Stay Cable Systems, 2005, higher axial stresses of up to 50% GUTS are nowadays considered permissible.

Corresponding results are given in Table 2. A maximum working stress of 1600 MPa for CFRP stays seems very high, but this value is not just a "fancy number". There is long-term data available from three bridges as shown in Table 3. The CFRP tendons in all these applications are very successful in operation since 13 to 15 years. In the case of Verdasio Bridge with the high sustained stress of 1610 MPa not any stress relaxation has been found, Meier (2012/3).

Table 2. Maximum working stresses for steel and CFRP stay cables.

Parallel wire bundle of:	Ultimate tensile stress [MPa]	Maximum working stress At 50% GUTS [MPa]
Steel	1800	900
CFRP (Toray T700S fiber)	3200	1600

Table 3. Examples of CFRP tendons with high working stresses.

Bridge	Open since:	Sustained stress:	References:
Verdasio	1998	1610	Meier (2012/2)
Kleine Emme	1998	1350	Meier (2012/2)
Dintelhaven	2000	1480	Vervuurt & Hordijk (2001)

In the described three cases the Toray T700S fiber was used for the production of the CFRP wires of the parallel wire bundles. Today fibers with higher strength are available. In the case of the T1000G fiber (UTS of fiber = 6370 MPa, tensile modulus 294 GPa) e.g. a guaranteed ultimate tensile strength for CFRP stays of 4000 MPa and therefore a maximum working stress of 2000 MPa could be reached.

Why do builders of large cable stayed bridges, where the described excellent properties would be very compelling, not yet use CFRP stays? The two main reasons are cost and confidence in the long-term reliability. In the case of the new Forth Road crossing north of Edinburg, the new cable stayed bridge is due to finish in 2016. It would be 25% more expensive with CFRP stays instead of steel stays.

Considering the bad experiences with steel stay cables, Hamilton et al. (1995) and even with suspension cables, Colford & Clark (2010), all underlying arguments are supporting the use of CFRP, especially considering the whole-life costing. However, within such discussions bridge owners always raise the question about the reliability of CFRP cables for at least 100 years. The behavior of all CFRP cables in pilot-applications is fully matching the high expectation (Table 3), but it would not be serious to extrapolate the results of 15 years positive full scale experience to a life span of 100 years. What can be done? More pilot-applications, especially in long cable stayed bridges, are needed. If in such a bridge e.g. one pair of the longest stays would be replaced by CFRP. Such a solution is a calculable risk and might even be in the range of the first investment cost for steel, if the much better performance for long CFRP cables puts in the balance, Figure 9 and Table 2.

What are possible drawbacks for such a hybrid cable arrangement? There is a difference in the coefficients of thermal expansion between CFRP and steel. The coefficient of thermal expansion for CFRP is about zero. But the cable stayed “Stork Bridge” in Winterthur has since 1996 a hybrid CFRP/steel cable arrangement. There the difference in the coefficients of thermal expansion is no problem because the composite bridge deck is relatively soft in flexure. In summertime the CFRP cables carry about 10% above average loading and in wintertime vice versa.

Another problem might be the dynamic behavior of CFRP stays. There are for any long cables a number of causes of aerodynamic excitation of stay cables, so there are several possible approaches to developing mitigating measures, FHWA (2007).

By raising the natural frequencies of the cables the wind velocity at which aerodynamic instability starts is increased. The natural frequency depends on the cable mass (Figure 10), the tension (Figure 11), and the length (Figure 10). The higher tension is an advantage for CFRP, the higher mass per meter for steel. Increasing the damping is one of the most effective ways of suppressing aerodynamic instability, or postponing it to higher wind velocity and thus making it rare enough not to be of concern. Since the damping of long cables tends to be naturally very low, the addition of relatively small amounts of damping at or near the cable ends can provide dramatic improvements in stability, Weber et al. (2002) and Weber et al. (2007). Mitigating measures developed for steel cables might have to be adapted and new measures like e.g. the controlled use of internal pressure in the cable pipes have to be considered for CFRP stays.

CFRP cable systems that proved in full scale structures between 13 and 15 years to be very reliable are available; it's only a question of decisions to use them.

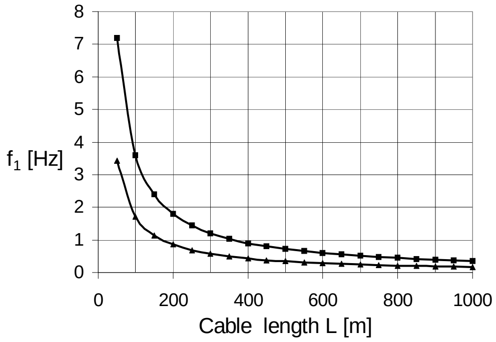

Figure 10. Frequency f_1 vs. cable length. The curve at the top (▪) represents CFRP stays with a cable tension of 1600 MPa and that at the bottom (▲) steel stays with a cable tension of 1000 MPa. Cable mass per meter: CFRP 21 kg/m, Steel 102 kg/m.

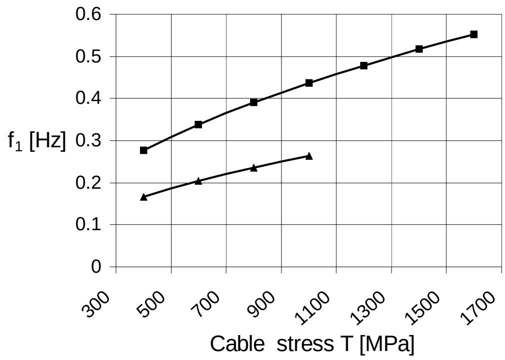

Figure 11. Frequencies f_1 vs. cable tension T for cables of 650 m length. The curve at the top (▪) represents a CFRP stay and that at the bottom (▲) a steel stays. Cable mass per meter: CFRP 21 kg/m, Steel 102 kg/m. Assumption: EI → 0 according to Girmscheid (1987).

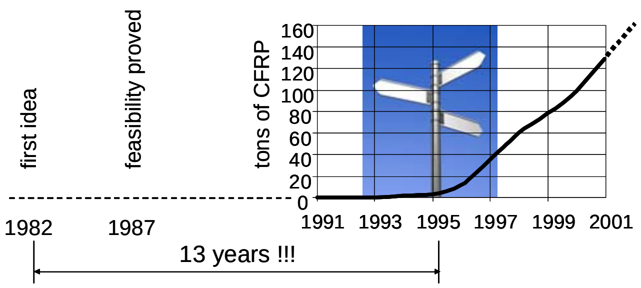

Figure 12. Historic development of the post-strengthening technique with CFRP strips: 1982 first idea, 1987 feasibility proved, 1991 application of the first 6 kg and 1995 commercial take-off. The ascending curve displays the tons of CFRP used per annum in Switzerland.

For the very simple concept of post-strengthening civil structures with carbon fiber reinforced polymer (CFRP) strips it took 13 years from the first idea to commercial take-off (Figure 12). "Charlie Chaplin" (Charles Spencer), said "There is no signpost at the crossroads of decision." Was it in 1995 something like a herd instinct, as it happens on the stock market? Who made the decision for take-off? At the beginning only the Swiss companies BBR and StahlTon were supporting this Empa R&R project. After the successful application in the case of several bridges between 1991 and 1994 a large international company launched the method in 1994 with a tremendous public relation effort and then the “herd” was following worldwide. Will it be similar with CFRP stays?

Breuer & Luchsinger (2010) work on huge kites for harvesting in the jet stream wind power using tethered airfoil, Bevirt (2010). Beside that such kites might also be used as stationary telecommunication platforms. It would not we surprising, when the above discussed CFRP stays would first be used commercially in such applications, similar as they are used since ten years as backstays from world number one super crane manufacturer Liebherr!

9 REHABILITATION AS DOOR OPENER FOR SUPER LONG-SPAN SUSPENSION BRIDGES MADE OF CFRP?

During the last decade many new long-span cable stayed and suspension bridges have been built, especially in Asia. There is a clear trend to longer spans. There are also visions for new long-span bridges across the Straits of Taiwan, Messina, Bering, Bab al-Mandeb, Hainan, Gibraltar, and others. If main spans are going to be increased, for the Strait of Taiwan e.g. it would be 3500 m, earlier or later the as called “break even span” will be reached. This is the span at which cost for a bridge made of steel or of CFRP are the same. This break even span is at about 4000 m, Meier (1987/2).

Menn & Billington (1995) proposed a concept for extremely long-span bridges. The dynamic stability of such a bridge is assured by placing on either side of the deck girder a sloping cable-stayed system carried by slender pylons which are supported by the central pylon.

Peroni & Casadei (2008) suggested a three-dimensional tensile structure with a hyperboloid shape: this consists of a 3D net with the ropes interlaced to each other to form a wicker basket containing inside the deck of the bridge. The principal net rope, beginning from two towers at the extremities of the bridge is developed around elliptic sections that gradually reduce towards the mid-span of the bridge. The particular interlaced cables conformation formed a “closed system “extremely stable with respect to the horizontal, vertical and torsion effects of the wind loads.

There are more “passive systems” for the mitigation of flutter, vibrations and oscillations under discussion. Such “passive systems” will, with a high probability, not be sufficient. Advanced “active systems” and control strategies are needed. As a contribution to fill this gap, the author initiated at the Empa laboratories in the year 2000 the program “Adaptive Material Systems”, Meier (2003).

Nevertheless today the time is by far not yet ripe for large standalone CFRP bridge projects. However there will be a need for the replacement of main cables on several large suspension bridges, Colford & Clark (2010). In such cases a stepwise procedure could be performed. The “25th of April Suspension Bridge” in Lisbon inaugurated in 1966 needed in 1999 additionally a lower train platform with two train tracks beside the existing upper platform with six car lanes. To accommodate this, the bridge underwent extensive structural reinforcements, including a second set of main cables, placed above the original set, and the main towers were increased in height. Original builder American Bridge Company was called again for the job, performing the first aerial spinning of additional main cables on a loaded, fully operational suspension bridge. Today such an operation would be much easier and faster with prefabricated, light-weight CFRP cables. This approach could be a very efficient solution to complement the loss of cross sections on existing suspension cables due to corrosion. This kind of application might be the next step in the use of CFRP cables.

Such an approach would fit seamless into the development of advanced materials in construction in the past. Rehabilitation was the driving engine for the launch of advanced materials and not new construction.

10 CONCLUSIONS

"The Non-Finito in sculpture, in which a perfect part grows out of the rough hewn surface, under certain circumstances is a source of larger magic, as if the chisel had worked to a clean end. And even where the non-finito is unwanted, it can be the pattern for future generations", von Matt (2009). In the case of this paper the non-finito is not deliberately done. Universities and R&D institutions like Empa are excellent breeding grounds for innovative ideas. But due to limited time and resources it will never be possible to chisel all of them out of the huge body of thought.

Antoine de Saint-Exupery recommended: "If you want to build a ship, don't drum up the men to gather wood, cut planks and give orders. Instead, teach them to yearn for the vast and endless sea". It is hoped that in this sense the sketched ideas and concepts, even when some of them sound crazy, might contain stimuli for future R&D.

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