Chapter 1
Non Finito: Challenges in Rehabilitation

Urs Meier

Abstract "Non Finito" is 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. The idea of post-strengthening civil structures with carbon fiber reinforced polymer (CFRP) tapes was for the first time unveiled in an oral presentation at ETH Zurich in 1985. An appropriate feasibility study was published 2 years later. The reactions of the listening and reading audience were mixed. Most found the concept very crazy and absolutely not practicable. The method is meanwhile state-of-the-art. The goal of this paper is to discuss the very promising system of CFRP straps as active external reinforcement in present time and to list three examples of new "crazy" ideas which might be interesting for future R&D work and might show successful applications in 10, 20 or 30 years. (i) More and more lacy and very slender structural components of historic structures like in the matter of the dome of Milan are suffering cracking. Internal, post-tensioning along three-dimensional boreholes with thin CFRP wires could be powerful tool to close the cracks. (ii) After all the severe earthquakes within the last few years seismic retrofitting gained dramatically relevance. CFRP post-strengthening is a successful mean in many cases however not always applicable. For example in the case of a mosque with golden mosaic ceilings there is no possibility to adhere black CFRP strips to the surface. Why should we not work with huge airbags, corresponding gas generators and sensors? How could we avoid that such airbags kill people? (iii) In modern architecture it is more and more fashionable 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

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made of electro-active polymers might resolve this problem. “Non Finito” in this paper means only to sculpt ideas and leave the realization to the next generation of researchers and engineers.

1.1 An Example of a Challenge in the Past

The challenge of post-strengthening civil structures with carbon fiber reinforced polymer (CFRP) strips was for the first time addressed in an oral presentation at ETH, the Swiss Federal Institute of Technology in Zurich in 1985 by the author. An appropriate feasibility study was published 2 years later (Meier 1987). The reactions of the listening and reading audiences were mixed. CFRP strips were first applied in 1991 to strengthen the Ibach Bridge near Lucerne. Consumption of the material at that time totaled a mere 6 kg per annum. Today about one quart of the worldwide carbon fiber production is used in construction, mostly for post-strengthening purposes, i.e. approximately 7,000 ton per annum. Considering the tonnages of steel used in construction, this seems extremely little. However we have to keep in mind that the strength of such strips is above 3,000 MPa and the density is only 1.5 t/m^3. 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 and the Americas. In most cases, 1 kg of CFRP can match 30–35 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 some 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 resin cures. The overcoming of this challenge brought construction industry a modern, meaningful method for post-strengthening of structures.

1.2 A Selected “Non Finito” Challenge in the Present Time

The focus of the research by Lees, Winistörfer and Meier (1999, 2002) was the development of post-tensioned non-laminated CFRP straps as active external reinforcement. At the beginning the emphasis was on shear-strengthening for concrete. The use of an active system has several advantages compared with a passive reinforcement system. In particular, the post-tensioned CFRP straps provide confinement and enhance the performance of the concrete.

Fig. 1.1 Conceptual design of pin-loaded strap elements. On the *left side* there is a laminated and on the *right side* a non-laminated strap

The current section seeks to discuss reliability aspects of this tendon system which is based on a brittle material and therefore theoretically subject to a large variability of strength properties. Experimental results of static short-term tensile tests and creep tests under severe loading conditions prove its high reliability.

A carbon fiber reinforced laminated pin-loaded strap, as shown in Fig. 1.1 on the left side, might provide a practical tension element for the shear enhancement of concrete. Such an element can consist of a unidirectional fiber reinforced plastic, having a fiber volume fraction of about 60%, wound around two cylindrical pins in a racetrack shape. However the maximum number of layers is limited by technical considerations in the manufacturing process (Winistoerfer 1999). No machining of holes is required to make a strap. The two pins transfer tensile load between the structure, and the fully consolidated strap.

Laminated straps have desirable characteristics of relative high tensile strength, low weight, low thermal conductivity, and, crucial, high durability. The US-Army has therefore investigated laminated CFRP tendons for temporary bridge deployment (Bauersfeld 1984). However, laboratory experiments and analytical modeling by Winistoerfer (1999) have shown that there are severe stress concentrations in the region where the strap and the pin meet. The tensile resistance of the strap is therefore limited to about 60% or less of the material's expected unidirectional strength. This is attributed to stress concentrations, which lead to premature failure. Furthermore, the production process for multi-layered laminated straps is not straightforward, if fiber misalignment is to be eliminated where the stress concentrations occur.

An alternative option to reduce the undesirable stress concentrations, overcome the manufacturing difficulties and reduce the cost is the use of a non-laminated strap. The concept which has been patented by Meier and Winistoerfer (1998) is shown on the right side of Fig. 1.1.

The CFRP strap now comprises a number of unidirectional reinforced layers, formed from a single, continuous, thermoplastic tape of about 0.13 mm thickness. The tape is wound around the two pins and only the end of the outermost layer is fusion bonded to the next outermost layer to form a closed loop.

Table 1.1 Summary of experimental investigation on non-laminated pin-loaded straps

(a)					
Number of layers, N	1	10	20	30	40
Specimens tested, n	6	6	6	6	7
Pin diameter, D	30	30	30	30	50
Length, L	700	700	700	700	700
Ultimate load F_{ult} [kN]	6.2	50.0	101.2	157.8	207.4
Standard deviation s [kN]	0.3	1.2	2.8	3.4	3.8
Coefficient of variation [%]	5.5	2.5	2.8	2.1	1.8
Weibull modulus, m	16.0	33.6	32.0	40.9	49.2
(b)					
Number of layers, N	40	40	50	60	70
Specimens tested, n	6	3	7	4	2
Pin diameter, D	50	50	50	50	50
Length, L	350	1,500	700	700	700
Ultimate load F_{ult} [kN]	173.8	204.0	249.0	269.8	278.0
Standard deviation s [kN]	12.0	3.6	4.9	10.2	14.1
Coefficient of variation [%]	6.9	1.8	2.0	3.8	5.1
Weibull modulus, m	12.5	43.7	47.2	21.6	13.8

The non-laminated strap element enables the individual layers to move relative to each other which allow an equalization of forces in the layers as the strap is tensioned (Winistoerfer 1999). The stress concentrations are reduced since the new structural form is more compliant than the laminated equivalent. Control of the initial tensioning process reduces interlaminar shear stresses so that a more uniform strain distribution in all layers can be achieved. The approach allows greater flexibility in terms of the geometry of the tendon, and it can be manufactured on site. Moreover, the concept is going to be less expensive because there is no consolidation process required.

An important aspect of non-laminated pin-loaded straps is the brittle nature of the carbon fibers. The stress at which brittle fibers fail usually depends on the presence of flaws, which may occur randomly along the length of a fiber. A statistical approach of this situation involves conceptually dividing a length of fiber into a number of incremental lengths. The fiber fractures when it has at least one incremental element containing a flaw sufficient to cause failure under a given stress. This analysis is known as the Weakest Link Theory (WLT) (Hull and Clyne 1996).

The Weibull modulus m is an important parameter for characterizing the strength distribution of brittle solids. A low value of m (e.g. <10) implies considerable uncertainty about the strength of a specimen. In practice, many ceramic materials exhibit Weibull moduli in the range 2–15. Sommer et al. (1996) reported values of $m = 5.25$ for strength properties of single carbon fiber filaments of the type Sigrafil C.

Fifty-three straps were produced from 12 mm wide carbon fiber tape with a polyamide 12 matrix and Toray T700 fibers. Various configurations in terms of number of layers, N, length, L, and pin diameter, D, were investigated. The tensile tests for the smaller pin diameters, summarized in the Table 1.1a and b, were performed on an Instron 1251 universal testing machine. The larger pin diameter straps were

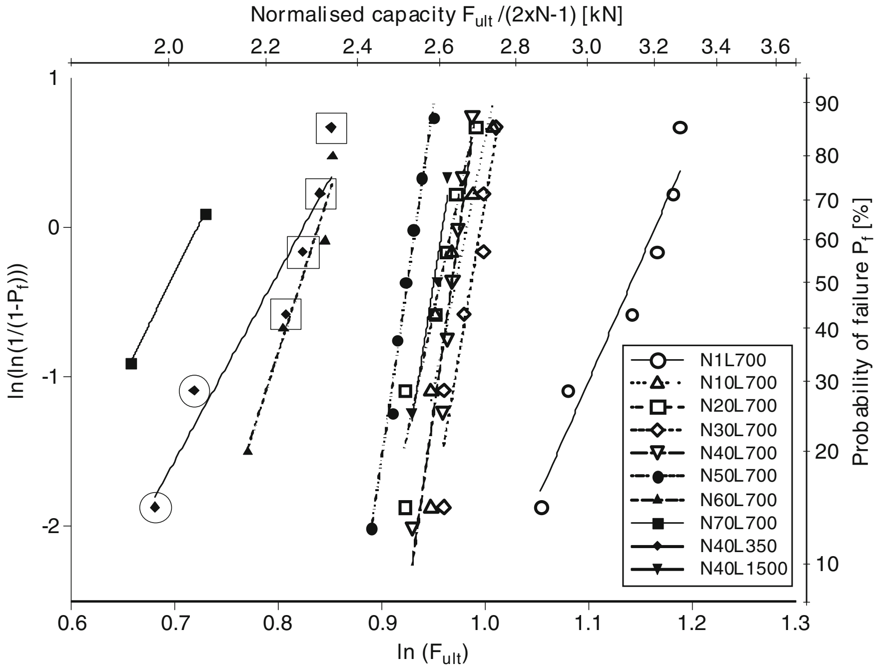

Fig. 1.2 Weibull plots of load carrying capacities of non-laminated pin-loaded straps with different numbers of layers and lengths (Winistoerfer 1999)

tested on an Instron 1346. The pin diameter had to be increased for the straps consisting of 40 or more layers to prevent failure of the steel pin due to extensive bending and shear stresses. Load carrying capacities, F_{ult}, are given instead of stresses because of variations in the tape dimensions, hence inaccurate measurements of the cross-sectional area.

The Weibull plots of the experimentally determined F_{ult} and the corresponding linear curve fits to resolve the gradients (Weibull moduli m) are presented in Fig. 1.2. The capacities are normalized with the total number of tapes present in each strap.

The highest mean strength was attained with a single layer containing a fusion bonded joint. Failure occurred in the joint, which may be the reason for the large variability. Only minor differences in the capacity per tape were observed for straps consisting of 10 up to 50 layers. Furthermore, the variability of these sets was considerably reduced. Specimens consisting of less than 30 layers failed in a brittle manner, whereas those with 30 or more layers started to fail with localized fractures in the pin region starting with the innermost layer. Substantial, clearly visible, damage was accumulated in the pin region before the final failure occurred. This experimental observation is also reflected by the increase of the Weibull moduli m, given in the Table 1.1a and b. A considerable reduction of the capacity per tape was observed in specimens consisting of 60 and more layers. It suggests that failure of such straps is dominated by excessive through-thickness stress components.

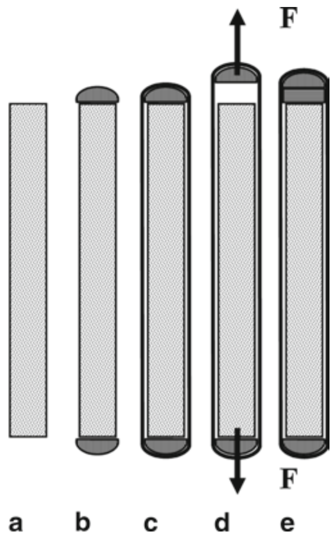

Fig. 1.3 Conception for post-tensioning with CFRP straps shown on the example of a steel rebar reinforced concrete shear wall: (**a**) cross section of shear wall, (**b**) addition of semi-elliptical interface pad elements that are placed on the *bottom* and *top* faces of the shear wall, (**c**) the layers of the CFRP tapes are wound around the pads and shear wall, (**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 interface pad and the shear wall and then releasing the tensioning force

Similarly, the short specimens (L = 350 mm) exhibited a considerable reduction of the load carrying capacity (Table 1.1b). This is attributed to an uneven strain distribution between individual layers because a reduced length requires increased relative displacement to attain an even strain distribution. The large variability in the set N = 40 and L = 350 mm indicates a difference in the tape qualities since the specimens marked in Fig. 1.2 with large circles and square boxes are made from different production batches.

The dramatic increase of the Weibull moduli presented in the Table 1.1a and b compared to the values mentioned above for single fibers is attributed to the large number of filaments present in the composite and the ability of the matrix to transfer load into the fibers across the interface, thus resulting in load redistribution effects in the vicinity of cracks. The consequence is a much more reliable material than expected from tensile strength properties of single fibers.

A key issue in determining the effectiveness of an active shear strengthening system for steel reinforced concrete is the capability of the system to enhance the shear resistance of a reinforced concrete beam or a shear wall. This shear resistance is generally thought to be comprised of a combination of aggregate interlock, dowel action, the concrete compressive zone and the internal shear reinforcement (where present).

The strengthening system consists of external pre-stressed non-laminated CFRP straps as outlined above. The straps are made up of 5–40 layers of 0.13 mm thick tapes which are wrapped around semi-elliptical interface pad elements that are placed on the bottom and top faces of the beam, membrane or wall to be strengthened (see Fig. 1.3).

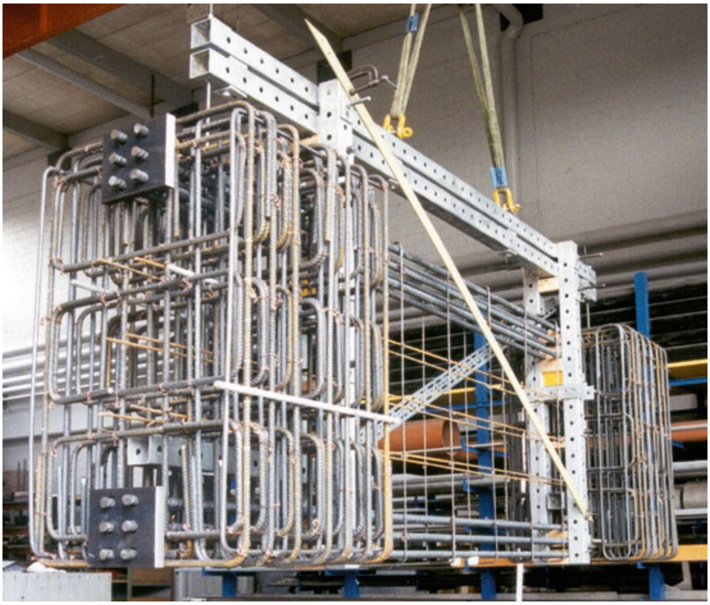

Fig. 1.4 Steel reinforcement of shear wall. The total length of a wall is 3,840 mm, the depth 1,200 mm and the thickness in the central part 150 mm and at the clamps 800 mm. The dimensions of the observed shear zone are 2,240 mm in length, 1,200 in depth and 150 mm in thickness

The strap is tensioned by lifting the top interface pad using a jacking system. The transverse post-tensioning force is applied to the concrete by overstressing the strap, inserting a spacer between the interface pad and the beam and then releasing the tensioning force.

The straps connect the concrete compression and tension zones, and such connection is felt to be important for the concrete shear resistance. Another benefit of the closed loop configuration is that the unexpected reversal of load paths can be accommodated. However, perhaps the most important advantage of the system is the ability to provide active confinement to the concrete and therefore increase the shear capacity of a beam, membrane or shear wall. The CFRP straps proved to be an extremely effective means to strengthen beams. A load carrying capacity enhancement of approximately 50% was obtained and the addition of the external straps resulted for certain beam configurations in a change from a brittle shear failure to a ductile flexural failure due to yielding of the longitudinal steel reinforcement (Lees et al. 2002).

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. All five steel reinforced concrete walls had the same dimensions and the same internal reinforcement as shown in Fig. 1.4. Four of these five walls have been post-strengthened each with four vertically applied external CFRP straps as described above. The walls were on both sides rigidly clamped and loaded in a large "Beam-Element-Tester" developed by Prof. Peter Marti at ETH Zurich (Stenger 2001). The CFRP straps used for the shear wall experiments were built up

Table 1.2 Results of shear experiments on reinforced concrete elements

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

of tapes with a thermoplastic PA12-matrix. 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 the each strap wound of 25 layers was 125 kN and the cross section (twice 25 layers) 96.0 mm^2.

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.

In Table 1.2 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 had a similar 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 CFRP tapes and breaking of fiber bundles could easily be observed and heard quite a while before the abrupt and noisy collapse.

Remarkable within the results of Table 1.2 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 increase of 3%. This demonstrates the high effectivity of post-tensioned applications. Figure 1.5 shows exemplarily the test setup, the theoretical stress field and the crack distribution for the shear wall ST1.

In the experimental phase the pins and 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 very powerful pins made of CFRP are available. Also the semi-elliptical steel pads have been successfully replaced by pads made of CFRP reinforced mortar (Fig. 1.6), polyethylene or GFRP (glass fiber reinforced polymers).

In 2007 such CFRP straps were applied for the first time as post-strengthening tendons to improve the seismic resistance of a masonry wall of a three floor administration-building (Fig. 1.7). The EMPA spin-off company Carbo-Link Ltd. was responsible for the production, installation and post-tensioning of five CFRP-elements of 13 m in length and 34 mm in diameter (Fig. 1.8). The pre-tensioning force on each tendon is 360 kN and the ultimate load amounts to 1,050 kN. Due to easy installation purposes the tendons are placed on the outside of the façade. The

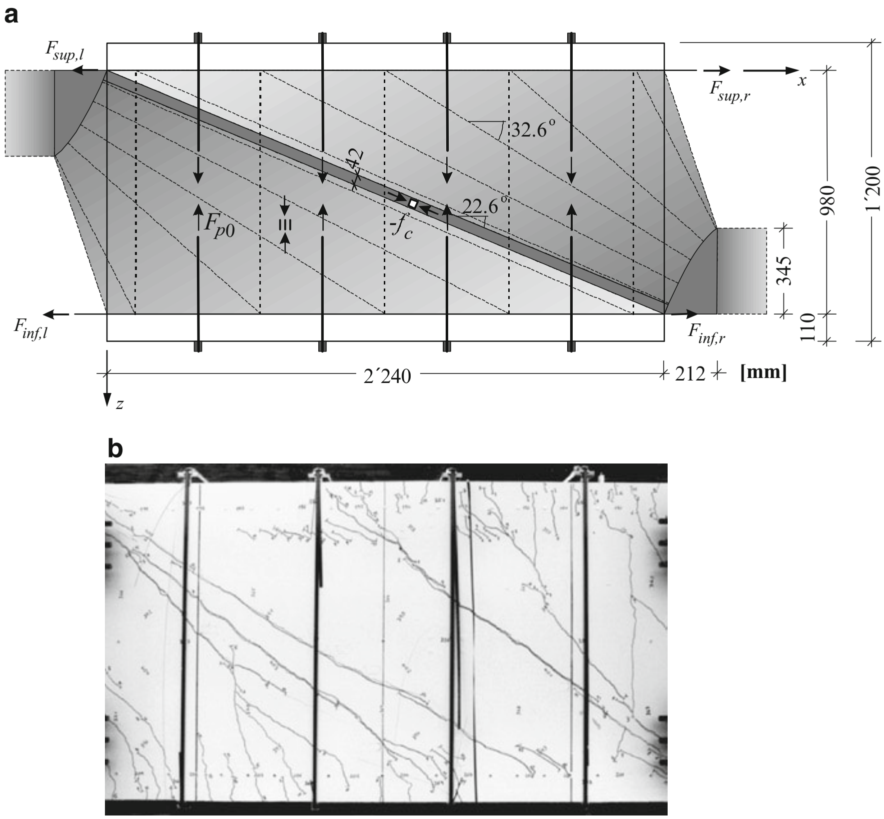

Fig. 1.5 Shear wall ST1 loaded up to −635 kN: (*top*) discontinuous stress field; (*bottom*) crack distribution

Fig. 1.6 Semi-elliptical CFRP reinforced mortar pad

advantages of these tendons are the low weight and therefore very easy handling during installation, high tensile strength and outstanding corrosion resistance. Similar elements have been used for Liebherr cranes with extreme cantilevers and for shovel dredgers of the same manufacturer. In mechanical engineering the steel cable substitution with CFRP tendons is resulting in considerable weight savings and an outperforming fatigue life.

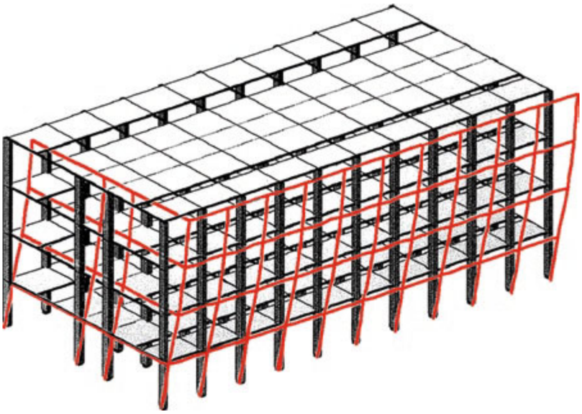

Fig. 1.7 Structure of three floor administration-building made of masonry. The wall on the *left side* had to be post-strengthened

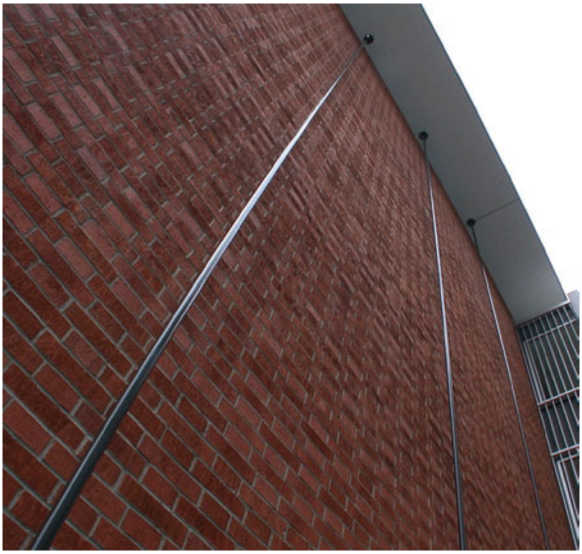

Fig. 1.8 Masonry wall post-tensioned with external CFRP tendons each of 1,050 kN ultimate load and 34 mm diameter

The described CFRP-strap-tendons were originally designed for rehabilitation of structures, especially post-strengthening in shear. Nowadays they are also used in new construction, e.g. for a bowstring arch bridge (Meier et al. 2009) realized with six longitudinal arranged CFRP bowstrings and glulam for the finally arch shaped bridge deck. This deck is stress-laminated as a flat plate. It is constructed by placing sawn lumber laminations on edge and stressing the laminations laterally together on the wide face with thin high-strength thermoplastic CFRP tapes (Fig. 1.9). It is causing the deck to act as a large orthotropic wooden plate. The bowstrings are composed of non-laminated pin loaded thermoplastic CFRP strap elements. These elements enable the individual layers to move relative to each other which allow an equalization of forces in the layers as the strap is tensioned. The approach allows excellent

Fig. 1.9 Glulam plate orthotropic post-tensioned with CFRP strap elements

use of the strength of CFRP and great flexibility in terms of the geometry of the strap elements as tendons. After prefabrication of the flat glulam plate and the CFRP straps the bow has been drawn. The bridge deck, which has the arch function, was axially loaded with the CFRP straps and elastically bent with a deflection of 1/75 of the span. The described bowstring arch concept allows an extremely slender and therefore very elegant bridge design. The first bridge of this kind with a span of 12 m was built at the end of 2006. A similar application in concrete is absolutely feasible. This kind of tendons would be predestined for a concrete stress ribbon bridge.

1.3 Selected "Non Finito" Challenges for the Future

1.3.1 Three Dimensional Post-tensioning

Especially in the case of rehabilitation of historic structures it would be ideal to be able to drill three dimensional curved holes of small diameters (typically 5 mm) into components of a structure to be able to post-tension such a component with mono wires or strands of CFRP or stainless steel. This would be a very effective and efficient way for the rehabilitation of pinnacles and spires especially on gothic churches. It could also be used to repair cracked stones of monuments. It would be a very interesting technique for seismic retrofitting of masonry walls of historic buildings. Post-tensioning could be optimized according to the stress trajectories of a structure.

To the best knowledge of the author there are no such tools available today in construction industry. Oil industry is drilling curved bore holes since decades. It is only a question of miniaturizing such systems for construction. Following the US Patent 4,442,908 (Steenbock 1984) such a system could be described as subsequent: a drilling tool having a motor-driven drill bit which can be turned at an angle to the axis of the primary hole to produce lateral holes branching from the primary hole. The bit that is rotatable mounted in a series of pivotally articulated links at the end of the drill pipe string. The deviation of the drill bit from the axis is controlled by pivoting the last link with respect to the penultimate link. The last link includes a sliding surface which bears against the wall of the hole to effect the pivoting.

1.3.2 Could Airbags Prevent Structural Collapses Due to Earthquakes?

The airbag for automotive applications traces its origin to air-filled bladders outlined 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.

In 2005 Amir Hashem Shahidi 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 earthquakes.

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 et al. 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 modelling 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 steel support structure is needed to carry the folded airbags. For the central dome this could be a centrically arranged horizontal circular steel 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 3 m 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 (Fig. 1.10) semicircular ring U-profiles and linear U-profiles respectively would be used.

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

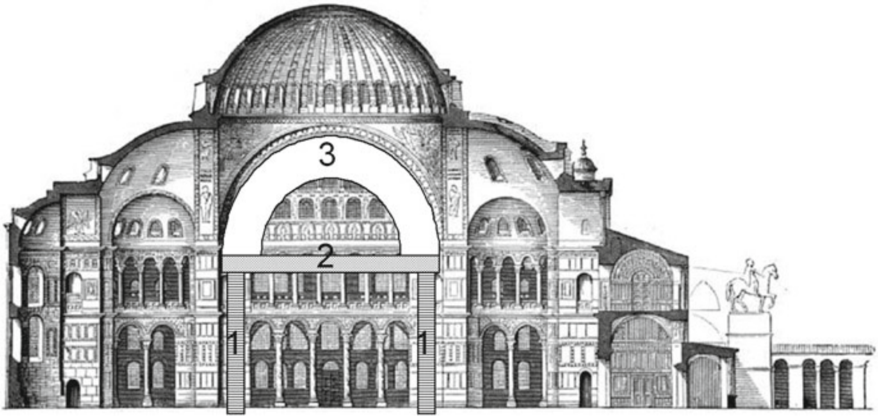

Fig. 1.10 Cross section of Hagia Sophia: Example for the seismic protection of an arch. *1* telescopic columns; *2* linear U-profile, open on the top; *3* deployed tubular earthquake airbag

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 to elevated R&D expenses. However further applications should be very fairly priced. A special challenge would be the design of stable telescopic columns.

1.3.3 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 6,000 ton of 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 ton, some 25% of the mass suspended by the cables (Meier 2003).

In future 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 might be installed along both edges of the girder and their angular position 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 et al. 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 to soaring R&D expenses. However, further applications should be very fairly priced.

In modern architecture it is more and more fashionable 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 et al. 2010). Such an outer skin may not only improve the aerodynamic properties but also serve as a curtain wall with special thermal and optical properties. Such multifunctional materials will play an important role in construction in the future.

1.4 Conclusions

Today's state-of-the-art in civil engineering is the accumulation of past innovations. As engineers, we must be innovative so tomorrow's world will be better. Innovation starts with the questions "Why?" and "Why not?". The question "Why?" gives us the opportunity to challenge the status quo. "Why not?" gives us the opportunity to introduce new ideas or overcome restrictions. The question "What if?" keeps us humble and conservative. How can we overcome these barriers? We have to accept challenges. We have to overcome them. It is what makes civil engineering meaningful. General George S. Patton said: "Accept challenges, so that you may feel the exhilaration of victory".

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