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Prefabricated Thin-walled Structural Elements Made from High Performance Concrete Prestressed with CFRP Wires

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Abstract

The innovative combination of prestressed carbon fibre reinforced plastic (CFRP) wires and high performance concrete (HPC) opens up promising possibilities in the design of structural elements and load-bearing structures. This enables manufacturing of thin-walled, lightweight, filigree, fatigue resistant and very durable concrete elements with very low raw-material consumption for use in several market niches of the construction industry. Two or more of these advantages should simultaneously apply to the intended application for justifying the higher material costs of prestressing and passive CFRP reinforcements in comparison to prestressing and reinforcing steel. Significant economic benefits are to be found in the areas of maintenance of the load-bearing elements as well as their transport and installation. Recently, a first commercial breakthrough of this novel technique was achieved in the structural and architectural field with the realisation of two large building façades in Zurich, Switzerland using a total of 3000 m prestressed self compacting concrete profiles. In this paper design and experimental validation details as well as several field projects are described.

Keywords: carbon, fiber reinforced plastics, prestressing, prefabrication, high strength concretes, case reports

Notation:

C100 (C90)	=	strength class denomination after EN 206-1:2002 for a concrete with a minimum 150mm-cube strength after 28 d of 100 MPa (respectively 90 MPa);
F_{ser}	=	test load inducing the service bending moment according to EN40-3-1/2 (wind effect, CEN/TC 50 2000) along the considered axis (max. service load) [kN];
F_u^{min}	=	minimum required failure load according to EN40-3-1/2 (CEN/TC 50 2000) with respect to bending along the considered axis [kN];
$M_R^{y+/-}$	=	moment of resistance with respect to the positive/negative y-axis [kN•m];
$M_R^{z+/-}$	=	moment of resistance with respect to the positive/negative z-axis [kN•m];
$M_i^{y+/-}$	=	bending moment (index i) with respect to the positive/negative y-axis [kN•m];
$M_i^{z+/-}$	=	bending moment (index i) with respect to the positive/negative z-axis [kN•m];
$M_{crack}^{y+/-}$	=	minimum cracking moment with respect to the positive/negative y-axis calculated under consideration of f_{ct} = 5.0 MPa [kN•m];
$M_{crack}^{z+/-}$	=	minimum cracking moment with respect to the positive/negative z-axis calculated under consideration of f_{ct} = 5.0 MPa [kN•m];
f_{ct}	=	minimum design bending-tensile strength of the concrete [MPa];
f_{pk}	=	design tensile strength of the CFRP prestressing wires (tendons);
a_1, a_2	=	cantilever [m];
l	=	length [m];
i	=	index as given in text;
sd	=	standard deviation;

σ_{ci} = central prestress in the concrete at t = i (after prestress losses) [MPa];

σ_{pi} = prestress in the tendons at t = i (after prestress losses) [MPa];

1. Introduction and Materials: Why use CFRP Prestressed HPC Elements for Civil Engineering Structures?

Why should the new combination of materials 'CFRP prestressed HPC' be beneficial for the prefabrication of concrete elements? The following discussion will deal with slender beams which are primarily subjected to bending and are manufactured in an adapted pre-tensioning method. In this context it should be noted that an eminent reference (Burgoyne, 1997) discussing the fundamental issue of making rational use of advanced composite reinforcements in concrete comes to the conclusion that to be economic, advanced composites will have to be used for prestressing tendons (pre-tensioning resin matrix bar or wire systems are a viable option), but not for passive reinforcement.

The CFRP prestressing reinforcement used in this work consists of fine, unidirectional carbon fibre-reinforced wires with a diameter of 3 to 6 mm, typically produced using the pultrusion method with an epoxy resin matrix. Other manufacturing methods such as rolltrusion (Winistörfer, 1999) were tested, in this case with a thermoplastic matrix, and would also be feasible. Unidirectional reinforced CFRP wires have high tensile strengths around 2200 - 2500 MPa. Typical geometric and relevant thermal and mechanical properties are listed in Table 1: The tensile properties of the wires are most probably higher than the experimental results from 10 and 45 specimens, respectively, because of premature anchorage failure in the tensile tests. The difficulty of effectively measuring the theoretical tensile capacity increases with the wire diameter (Al-Mayah et al., 2006).

Table 1. Properties of pultruded CFRP wires for the manufacture of CFRP prestressed HPC structural elements

Property	Diameter 4.2 mm CFRP wire	Diameter 5.4 mm CFRP wire
Effective diameter of wire	4.2 mm (coated: 5.4 mm)	5.4 mm (coated: 6.6 mm)
Scatter in diameter (sd)	0.05 mm (coated: 0.4 mm)	0.2 mm (coated: 0.5 mm)
Carbon fibre type	Tenax UTS	Tenax UTS
Pultrusion matrix	Epoxy Bakelite Rütapox	Epoxy Bakelite Rütapox
Carbon fibre tensile strength	4800 MPa	4800 MPa
Carbon fibre (filament) diameter	$6.9 \, . \, 10^{-3}$ mm	$6.9 \, . \, 10^{-3}$ mm
Number of carbon filaments in wire	19 rovings à 12 000 fibres	32 rovings à 12 000 fibres
Fibre volume content	61.5%	64.5%
Glass temperature	141 °C	135 °C
Transverse CTE of wire	$32.8 \cdot 10^{-6}$ 1/°C	$26.5 \cdot 10^{-6}$ 1/°C
Average longitudinal tensile strength	2540 MPa	2471 MPa
Scatter in tensile strength (sd)	40 MPa	168 MPa
Minimum measured tensile strength	2475 MPa	2223 MPa
Design tensile strength f_{pk}	2200 MPa	2000 MPa
Average longitudinal E-modulus	168.5 GPa	155.7 GPa
Scatter in E-Modulus (sd)	3.5 GPa	2.4 GPa
Design E-Modulus	160 GPa	150 GPa
Average fracture strain	1.64%	1.54%
Scatter in fracture strain (sd)	0.04%	0.10%
Minimum measured fracture strain	1.60%	1.33%
Number of tensile tests performed	10	45

The indicated scatter values of the tensile properties result from this anchoring difficulty, which arises even when using resin potted anchors of the type described in (Terrasi, 1998). The density of CFRP is 1.6 kg/m^3 which is just short of one fifth of the density of prestressing steel. The wires are coated with quartz sand particles (mean diameter 0.5 mm) in order to control bond to the high performance concrete (Figure 1). The quartz sand coating is bonded in-line following pultrusion using the same epoxy resin used to impregnate the carbon fibres of the wire (Table 1). The bond behaviour of the CFRP wires used in this work was studied with bond pullout tests of the wires from HPC cylinders by (Maluk et al., 2011) and compared with the bond behaviour of standard 6 mm diameter prestressing steel wires. The CFRP-HPC pullout specimens tested by (Maluk et al., 2011) failed by slipping at the bond interface between the quartz sand coating and the tendon, at a bond stress of approximately 5.3 MPa (at 20°C). After failure, pullout continued at a constant rate, and a remnant bond strength capacity of 4 MPa was measured. These bond strength values are lower than the corresponding steel wire bond strengths, but they are high enough to guarantee a stable prestress transfer (Terrasi, 2001)(Terrasi, 2012) and the full anchorage of the CFRP wires in a HPC element loaded in flexure (Terrasi, 1998)(Terrasi & Lees, 2004).

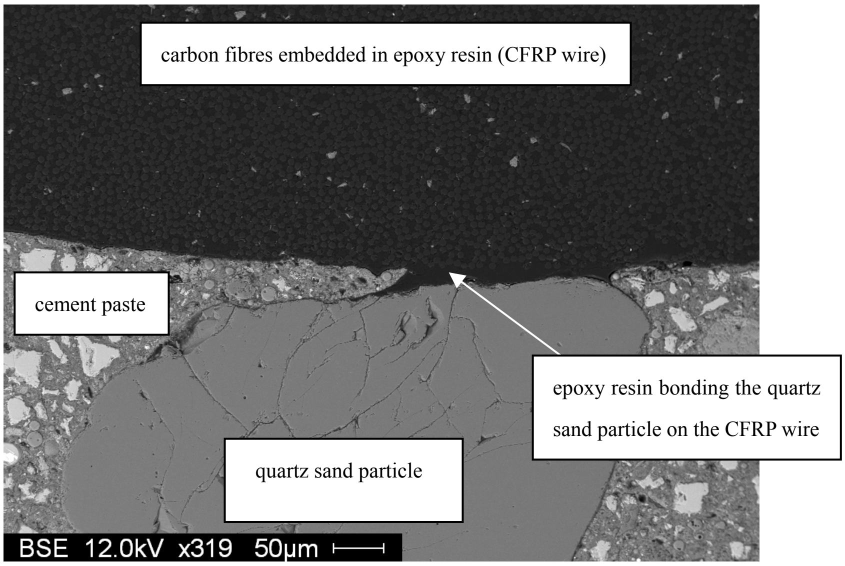

Figure 1. SEM of CFRP wire coated with quartz sand: The single carbon fibres of diameter 7 μm are embedded in epoxy resin by pultrusion to form the CFRP wire. The same epoxy is used to glue the quarz sand on the surface of the wires. The quartz sand promotes bond to the HPC and is embedded in the concrete's cement paste

Perhaps the biggest advantage of CFRP wires as a prestressing reinforcement is their total immunity to corrosion in practically all relevant media, even if subjected to high mechanical stresses. The complete absence of any tendon stress-corrosion allows the concrete cover to be reduced to well below that necessary for the protection of prestressing steel wires. The new Swiss civil engineering standard SIA 262 (SIA, 2003), e.g., requires tendon covers between 40-55 mm depending on the environmental exposure class defined when designing the structure. Hence the 5 mm diameter CFRP prestressing wires used in the first high-voltage transmission pylons manufactured in 2000 (Terrasi et al., 2001) have a concrete cover of just 18 mm. The concrete cover size is determined on the basis of static considerations (reception of the compressive stresses due to bending and of the bursting tensile stresses in the anchorage zone of prestressing reinforcements) and of the mismatch in thermal expansion coefficient between CFRP (transverse to fibre direction, see Table 1) and high-performance concrete (Terrasi, 1998).

The extremely favourable fatigue properties and the absence of time-dependent mechanical prestress losses (creep and relaxation) should not be underestimated (Uomoto, 2001). On top of the high tensile strengths, the last mentioned properties lend this material outstanding properties for prestressing of concrete elements.

It should be mentioned as well that the modest mechanical properties of unidirectional carbon-fibre reinforced wires transverse to the fibre direction (in the range of 10% of the corresponding longitudinal properties) must also be given careful consideration, e.g. in view of the anchorage of the prestressing wires (Kim & Meier, 1991). Apart from this, the coefficient of thermal expansion of the profile transverse to the fibre direction is 2-3 times higher than that of HPC (due to orthotropy, Table 1).

The high quality of quartz sand-coated CFRP wires and hence the cost which is still high (specific prices around 0.05 € per kN tensile capacity) require high quality for the concrete matrix. HPC with a strength in the range of class C100 is the appropriate matrix material for the carbon fibre reinforced polymer prestressing reinforcement. HPC can be produced today in the prefabrication element plant with suitable mixing equipment with a consistently high quality providing the production process is mastered well and the personnel are suitably qualified. Typically, a very precisely batched fine-grain concrete (mortar) is used with high strength aggregates. The cement content is around 450 kg/m^3, micro silica and high-performance plasticizer are essential components of these concretes which have a water/cement factor of 0.32-0.39 for optimum workability (Terrasi, 2012). C90 to C100 high-performance concretes are relatively inexpensive (with material costs around 125 €/m^3) but have high durability, high tensile bending strength (with fractiles over 5 MPa), a relatively high modulus of elasticity (around 40 GPa depending on the aggregates) as well as the high compressive cube strength over 90 MPa (Walraven, 1995). The compaction of HPC can be achieved by centrifugal casting, vibration or following a more advanced mix design (Persson & Terrasi, 2002) by the self compacting method (High Performance Self Compacting Concrete [HPSCC]). The treatment after casting often corresponds to a simple covering of the moulds in order to keep the young HPC humid for typically 36 hours before prestress release and demoulding.

Taking advantage of the properties of the two components CFRP and HPC, it is possible to minimise the weight of the bending element by reducing the wall thickness while guaranteeing excellent service characteristics (no susceptibility to corrosion, high bending stiffness and high fatigue strength (Agyei et al., 2003).

The outstanding durability of HPC prestressed with pultruded CFRP wires has been confirmed by several field tests, in particular by the results of long-term (four-point bending creep) outdoor tests on three scaled pylon sections subjected to extremely high bending loads which have been running at Empa for the last 15 years (Terrasi, 1998). The durability of CFRP prestressed HPSCC is currently tested by the precaster SACAC through outdoor four-point bending creep tests of load-bearing façade elements. Figure 2 shows the example of a 8.4 m long U-beam (width 400 mm, height 270 mm, wall thicknesses 60-100-120 mm) under a façade service load of 3.6 kN/m (simulated by concrete blocks). The beam is simply supported symmetrically at 1.65 m from the ends (the central span has a length of 5.1 m). The CFRP prestressing reinforcement ratio is 0.3% (9 wires with diameter 5.4 mm) for a total prestress of 200 kN at release. Deflection monitoring at the beam's ends and at midspan vs. time and vs. external temperature is plotted in Figure 3. The short-term deflection at midspan after service load was applied amounted to 1.21 mm (calculated value 1.25 mm) while the long-term deflection limit at midspan given by standard SIA 262 (SIA, 2003) is span/500 = 10.2 mm. This deflection criterion could be fulfilled for 2 years, the maximum measured midspan deflection being only 7 mm. Visual crack inspections were performed once a week and the absence of bending cracks could be confirmed.

Figure 2. Typical four-point bending creep tests of a load-bearing façade element

2. Pilot Project 'CFRP Prestressed Electricity Pylon'

In 1994, SACAC embarked on a R&D project with Empa, the Swiss Federal Laboratories for Materials Science and Technology. The aim of this project was to investigate the fundamental principles of non-stressed (passive) and prestressed (active) CFRP reinforcement of HPC for the production of tubular elements.

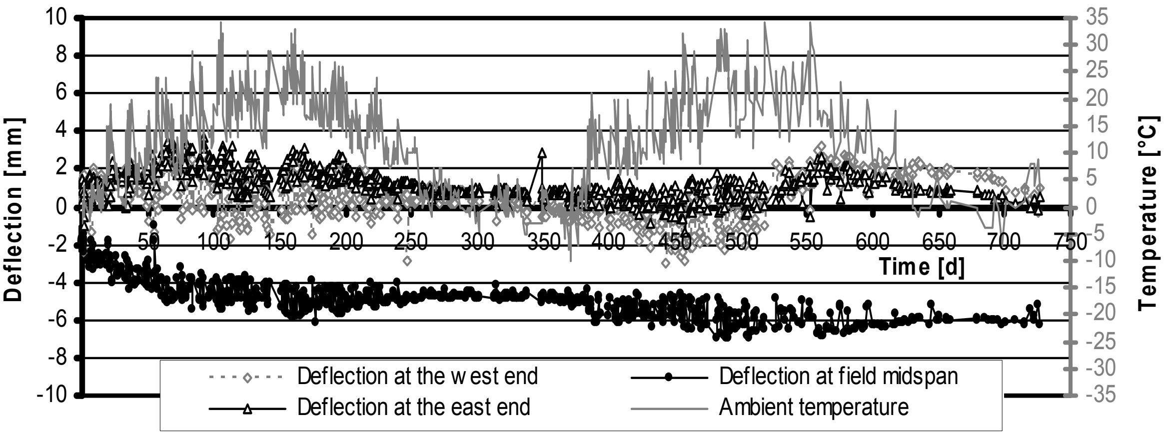

Figure 3. Deflection vs. time curves for a façade element subjected to 4-point bending creep

After the fundamental principles for the new construction method had been established (Terrasi, 1998), the Nordostschweizerische Kraftwerke (NOK, Power Companies of North-Eastern Switzerland) decided to install a pylon of this type for the first time in one overhead electricity power line of their 110 kV distribution network. SACAC produced a 27 m high pylon for this project using an adapted pre-tensioning-spinning method (Terrasi et al., 2001).

The conical pylon had an external diameter of 850 mm at the bottom and 530 mm at the tip (1.175% taper) with an average wall thickness of 48 mm and a standard deviation of 4 mm (Figure 4, the minimum required design wall thickness was 40 mm).

Figure 4. CFRP prestressed pylon made from high performance spun concrete

The C100 class HPC cover on the CFRP wires was only 18 mm. The concrete cover and the wall thickness were controlled using geo-radar measurements with a 1.5 GHz antenna (Hugenschmidt, 2000) over the entire length of the pylon. The pylon weighed 6000 kg. This corresponds to a reduction in weight of 40% in comparison with a traditional steel-reinforced concrete pylon for the same application. A centric overall prestressing of 1000 kN was produced via 40 fine, 5.0 mm diameter CFRP prestressing wires. After prestress losses the respective values were: σ_{pi} = 1200 MPa, σ_{ci} = 6.5 MPa at the foot, and σ_{ci} = 14 MPa at the tip of the pylon. This corresponds to an average prestressing reinforcement ratio of 1%. A rolltruded CFRP tape spiral (0.5-1 mm thick, 13 mm wide) (Winistörfer, 1999) wound around the prestressing wires served as the shear reinforcement.

The full-scale 27 m long pylon was subjected to cantilever bending tests at Empa's Structural Engineering Laboratory before field installation. It fulfilled the requirements of the relevant regulation (Swiss Federal Government, 1994) in terms of serviceability: The top deflection under the maximum service moment of 394 kN•m (corresponding to a point load at the pylon's tip of 16 kN) was limited to 1.6 % of the cantilever length 24.7 m (Figure 5), 5% of the cantilever length are allowed according to regulations (Swiss Federal Government, 1994). A maximum fixture testing moment of 492 kN•m could be resisted with the opening of 88 fine bending cracks (of width < 0.3 mm) with an average spacing of 130 mm over a length of 11.5 m from the fixture, in which the bending rotation was concentrated. After testing, in the summer of 2001 NOK therefore installed the pylon near Würenlingen in their 110 kV overhead electricity power line 'Beznau-Baden' (Figure 4).

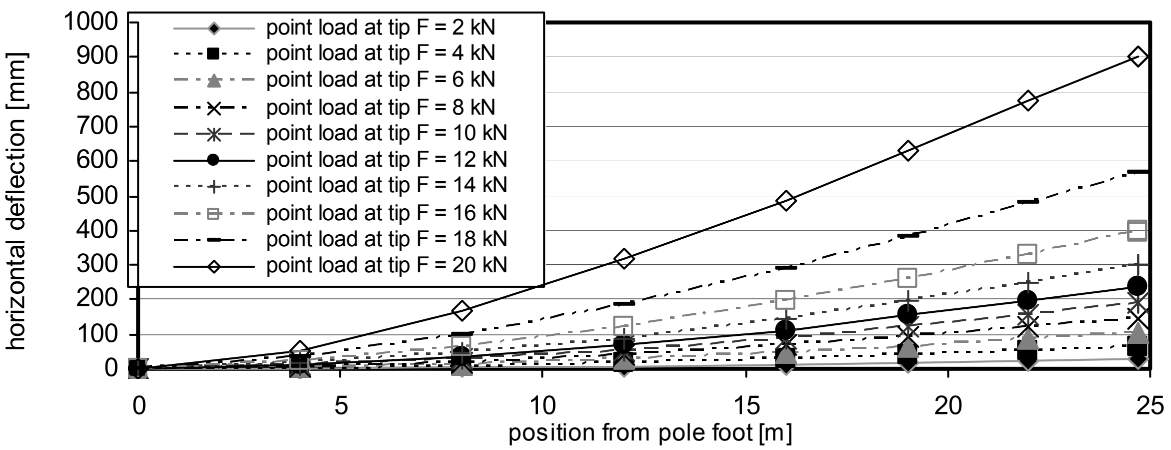

Figure 5. Deflection curves for cantilever test of a full-scale CFRP prestressed pylon

For 10 years, the pylon was remotely monitored using a novel concept from Empa's Electronics/Metrology Laboratory – a recent innovation in the construction industry which consisted of an electronic system for monitoring the CFRP prestressing level by measuring the electric resistance of the CFRP prestressing wires (Terrasi et al., 2001). This measurement method relies on using the CFRP reinforcement as a sensor itself since the longitudinal electrical resistance of the CFRP wires is dependent on their tensile elongation. A so-called master curve (calibration curve) was determined in the laboratory by measuring the electrical resistance in fibre direction during a mechanical tensile test of the prestressing wire as a function of temperature and moisture conditions. Having determined this electro-mechanical characteristic of the CFRP, it became possible to monitor the tensile force in the wires by measuring their electrical resistance over a gauge length of typically 800 mm (CFRP has linear elastic properties, i.e. the tensile elongation is directly proportional to the mechanical stress). After 5 years of monitoring, average CFRP wire prestress losses of 7% could be measured (Brönnimann & Terrasi, 2009), which are most likely caused by creep of the HPC.

3. CFRP Prestressed HPC Balcony Sills and Window Sills

In the spring of 2001, a series of several filigree window and balcony sills were manufactured in the Lenzburg SACAC plant to be used in a multiple dwelling near Zurich. The architects' aesthetic requirements of manufacturing very thin, long and jointless border elements of appropriate strength, bending stiffness and high durability led to the choice of HPC slabs prestressed with CFRP wires. The balcony sills have a trapezoidal cross-section with wall thicknesses ranging between 50 and 55 mm, a width of 350 mm and lengths between 3.6 and 5.1 m. The trapezoidal cross-section of the window sills varies between 50 and 60 mm with a width of 350 mm and lengths between 4.3 m and 5.3 m (Figure 6).

Figure 6. CFRP-prestressed HPC window sill 4.7 m long

All elements are prestressed to a total force of 112 kN with 7 pultruded CFRP prestressing wires of 5.0 mm diameter. This corresponds to 820 MPa tension for each wire at prestress release which is sufficient for all elements to be fully prestressed for the determining service load (load case: simple support at the two ends under the element's own weight when installing the slabs). By using a C90 class HPC, shear reinforcement was rendered unnecessary with the slim plates being subject primarily to bending stresses. The filigree CFRP prestressed HPC window sills and balcony sills disappear in the plain exterior of the modern façades and have now proved effective after 11-years use (Figure 7).

Figure 7. Façade of a modern townhouse with CFRP prestressed HPC window sills

4. CFRP Prestressed Spun Concrete Lighting Columns

In 2002, SACAC designed and manufactured a series of lighting columns from CFRP prestressed, C100 high performance spun concrete (spun HPC). The Type 9.2/12-21.2 columns have a nominal height of 8 m and a total length of 9.2 m and are prestressed with 6 pultruded CFRP 4.2 mm diameter wires (properties in Table 1) to a total compression of 100 kN (at prestress release). With a taper of 10 mm per m, the external diameter of the tubular profile varied from 120 (tip) to 212 mm (bottom). The spun concrete method meant that an average wall thickness of 40 mm ± 10 mm could be safely achieved. The necessity of using shear reinforcement was investigated with three different versions: Rolltruded CFRP-tape spiral with thermoplastic matrix, PVA or Aramid fibre-reinforced grid.

Five of these columns were subjected to a detailed certification procedure with quasi-static, dynamic and durability tests according to EN40 (CEN/TC 50, 2000) in collaboration with the Engineering Department (structures group) of Cambridge University, England (Terrasi & Lees, 2003)(Terrasi & Lees, 2004). In particular, five horizontal cantilever tests were carried out where a tensile load at the tip F was applied off-centre (Figure 8). The slight eccentricity (140 mm) of the tip tension F produced a small torsion moment which increased proportionally with the main bending moment.

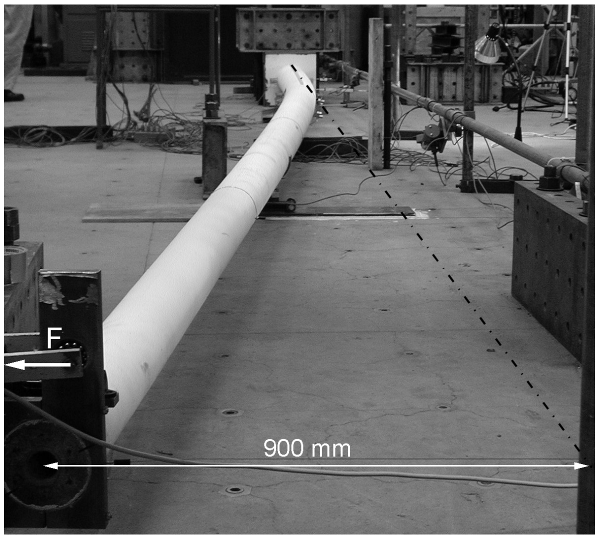

Figure 8. Deflection of a test-column (pole 3) shortly before bending failure

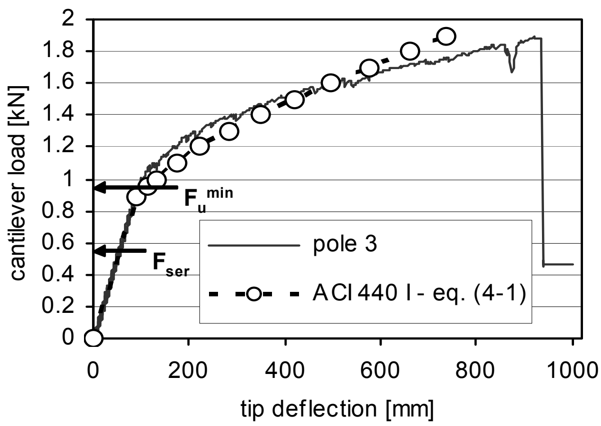

Figure 9. Cantilever load vs. tip deflection curve for a lighting column tested in bending/torsion

The cantilever load vs. deformation curve in Figure 9 shows exemplarily for all specimens how the type of column examined did fulfil all the requirements for experimental certification according to EN40-3-2 (CEN/TC 50, 2000). This is so whether or not shear reinforcement or supplementary passive reinforcement is used and irrespective of the position of the 300 x 75 mm opening for the electrical fuses in relation to the plane of bending tested (Terrasi & Lees, 2003). In particular, the full prestressing for service loads limits the deflection and the torsion angle at the tip of the column under maximum working load. The measured residual deflections after relaxation from the service state were only one fifth of the values defined by EN40-3-2 (CEN/TC 50, 2000). The effective total failure safety factor of the samples ranged from 2.8 to 3.5 (Terrasi & Lees, 2003), which is significantly more than the 1.7 required by the standard. In addition to this, all columns showed a high tip deflection at failure (tensile failure of the CFRP wires or compressive failure of the HPC) way above 0.4 m (which corresponds to 5% of the nominal height) – the relatively good bond of the coated CFRP prestressing wires to the HPC matrix enabled a reasonable number of fine bending cracks to open in which the bending rotation was concentrated. The deflections calculated using the ACI 440.4R-04 recommendations (American Concrete Institute, 2004) were found to correlate well with the measured results (Figure 9). The high rotation capacity of the cracked cross sections yielded the desired warning effect when the columns have to resist unusually extreme loads (well above the designed service load) as can be seen in Figure 8.

The high bending stiffness of the filigree columns in the working state (F_{ser}, Figure 9) achieved by full prestressing and their (for concrete) small mass distribution led to a fundamental frequency of 2.4 Hz which was determined from several free bending vibration decay tests on two uncracked samples (Terrasi & Lees, 2004).

Thus, the columns tested are more difficult to damage by vandalism than aluminium columns, for example.

The high durability of CFRP prestressed HPC was first verified in a freeze-thaw temperature exposure test on a 175 x 40 mm plate 2.3 m long which was prestressed with the 4.2 mm diameter prestressing wires used in the columns. Fifty one-day long freeze-thaw temperature cycles between -25 and +50 °C, following the procedure given in (Terrasi et al., 2001) did not produce cracks in the specimen and the loss of concrete-prestress was negligible. In a second test phase, two 1 m long CFRP prestressed spun HPC column sections with and without PVA fibre-reinforced grid and having the opening for the electrical fuses in the centre were subjected to 100 freeze-thaw temperature cycles between -25 and +60 °C (same procedure as reported in (Terrasi et al., 2001), except that the oven exposure was at 60 °C instead of 50 °C). These specimens were compared with the standard SACAC product, an 8 diameter 6 mm grade B500B steel (SIA, 2003) reinforced spun concrete column section (spiral reinforcement diameter 3 mm B500B with 30 mm pitch) having same outer dimensions (Terrasi & Lees, 2003) but a higher wall thickness of 55 mm. In this case at the end of testing 10-12 thin (width < 0.1 mm) thermal cracks were observed for the CFRP prestressed HPC column sections. These concrete splitting cracks are caused by the superposition of the tensile thermal compatibility stresses between CFRP and HPC with the bursting stresses in the prestress transfer zone of the columns. The appearance of thin thermal cracks after 100 extreme freeze-thaw-heat cycles is not considered to be dramatic, since the penetration of water into these cracks did not cause any damage or spalling of the concrete cover when freezing during the performed thermal cycles. Moreover the reference steel reinforced concrete specimen showed a lower durability than the CFRP HPC specimens with 20 thermal cracks and a maximum crack width of 0.2 mm observed at the end of testing.

As stated before, the reasoning behind using CFRP prestressed HPC aims at achieving the optimum serviceability for the lighting column developed, where excellent corrosion resistance is the main factor, particularly in the area of fixture and the opening for the electrical fuses for the lighting system. It should be borne in mind that different types of very aggressive media could have an effect on the durability of a lighting column (particularly de-icing salt, dog’s urine and garden chemicals).

One favourable side effect of the CFRP HPC columns is the reduction in weight from minimising the concrete cover. The lighting columns reported in (Terrasi & Lees, 2003) weigh 350 kg and are therefore 30% lighter than the equivalent steel-reinforced spun concrete product. This weight reduction is an advantage, particularly during the transport and erection of the taller lighting columns.

In 2003, SACAC launched the new product line of durable, lightweight, 4.2 to 18 m high lighting columns on the Swiss market (Figure 10). Today SACAC is producing approximately 500 to 600 of these lighting columns per year.

Figure 10. CFRP prestressed spun HPC residential street lighting columns in the Swiss town of Binningen

5. Thin-walled Building Façade Elements made of CFRP Prestressed Self Compacting Concrete

In 2005-2006 the commercial breakthrough for this novel material combination was achieved in the architectural field with the realisation of two building façades using thin-walled CFRP prestressed high performance self compacting concrete (HPSCC) elements (strength class C90).

In a first project 250 slender façade beams of length up to 8.8 m were designed and manufactured by SACAC for a 6-storey office building in Zurich – Oerlikon. Beams with an L-cross-section of 267 mm x 385 mm and a wall thickness of 40-70 mm (Figure 11) were chosen by the architect Kaufmann, van der Meer + Partner AG Zurich in order to form continuous horizontal bands at the height of the floor slabs, running around the entire concrete-glass building façade (Figure 12).

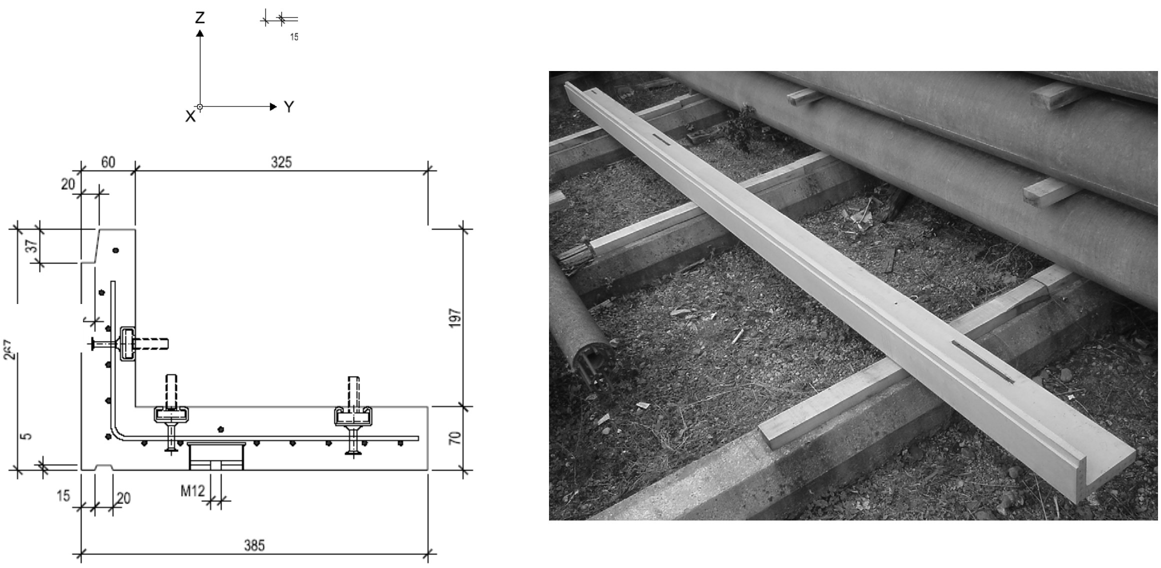

Figure 11. 8.8 m long CFRP prestressed HPSCC façade beam with L cross-section 267 x 385 mm

Figure 12. CFRP prestressed HPSCC L-beams as structural façade elements

1'600 m CFRP HPSCC façade beams were produced in four 22 m long pre-tensioning beds within two months with a prestress release/demoulding time of 36 hours after casting the concrete. The static system chosen was an asymmetrically supported continuous girder (Gerber beam) under a combination of wind loads, self weight, snow load, 2 maintenance personnel of variable position, and temperature variation following (SIA, 2003). The deflection criteria for the standard 8.4 m long L-beam to be fulfilled in service were: field (l) deflection < l/300, with l = 5.04 m and cantilever (a_1, a_2) deflection < a_i/300 (i = 1, 2), with a_1 = 2.37 m, a_2 = 0.97 m. Hence the beams were fully prestressed for maximum service loads in order to avoid cracks and limit deflections. Cross-sectional data of the beams are given in Table 2. The elements are prestressed by 14 CFRP wires (Figure

11) diameter 5.4 mm (f_{pk} = 2000 MPa, see Table 1) at a total prestress of 260 kN. The connection of the façade beams to the building's slabs was made possible by standard concrete inlays (stainless steel Halfen-rails™ with nail anchors and local confinement by diameter 5 mm stainless steel rebars, see Figure 11). On their lower flange the beams are supporting the rotation axes of a glass lamellae front in which 3.5 m long x 0.5 m wide vertical glass lamellae assume the function of sunblinds that can be electrically controlled by the office occupants. Besides that, the L-beams are acting as casing for a steel-superstructure carrying the electric control units and the lover pin-joints of the glass lamellae axes.

Table 2. Cross-sectional data of CFRP prestressed HPSCC façade beams with L-cross-section

Bending moment	Cracking Moment (i = crack, for f_{ct} = 5.0 MPa)	Moment of Resistance (i = R)
M_i^{y-}	-10.0 kN•m	-27.0 kN•m
M_i^{y+}	14.0 kN•m	26.0 kN•m
M_i^{z-}	-19.0 kN•m	-47.0 kN•m
M_i^{z+}	23.5 kN•m	51.0 kN•m

A second project in 2005-2006 required the production of 905 m of filigree rectangular beams in CFRP prestressed HPSCC for the extension of a college building (Falletsche, architect Rolf Mühlethaler Berne) at Zurich-Leimbach. These 157 façade 'columns' are supporting glass-windows and have a length varying between *l* = 0.5 m to 11.1 m with a cross-section of 100 mm x 300 mm (Figure 13).

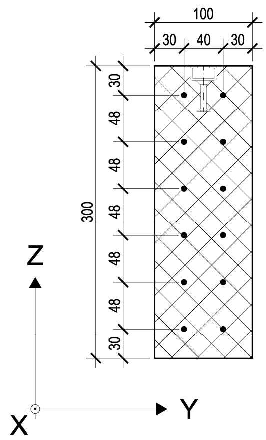

Figure 13. Cross-section of three-storey rectangular HPSCC façade column

Figure 14. 11.1 m long CFRP prestressed HPSCC column during surface finishing operations

Figure 14 shows a three-storey (11.1 m) column during surface finishing operations in the SACAC production plant. The columns were manufactured in a pre-tensioning bed with 8, 10 or 12 (Figure 13) pultruded and coated CFRP prestressing wires of diameter 5.4 mm (Table 1) at a total prestress of 146 kN, 183 kN, 220 kN respectively. The static system chosen was a simply supported beam at the ends under self weight (determining load case: transport and installation of the façade column). The deflection criteria to be fulfilled by the beams under maximum service load was field (l) deflection < l/300, with l = 0.5 to 11.1 m. Therefore the beams were fully prestressed for maximum service loads in order to avoid cracks and limit deflections. Table 3 gives relevant cross-sectional data of the rectangular façade columns prestressed with 12 CFRP wires. The connection of the façade columns to the building's structure/windows was solved again by standard inlays (stainless steel Halfen-rails™ with nail anchors, see Figure 13) in the HPSCC. In addition to this 400 m (326 pieces) of thin-walled (thickness 50 mm) CFRP HPSCC screen-panels with L-profile and length 1-1.5 m were manufactured that serve as horizontal joining elements in the façade (Figure 15).

Table 3. Cross-sectional data of CFRP prestressed HPSCC façade columns with rectangular cross-section

Bending moment	Cracking Moment (i = crack, for f_{ct} = 5.0 MPa)	Moment of Resistance (i = R)
M_i^{y-}	-18.8 kN•m	-48.5 kN•m
M_i^{y+}	18.8 kN•m	48.5 kN•m
M_i^{z-}	-6.2 kN•m	-14.6 kN•m
M_i^{z+}	6.2 kN•m	14.6 kN•m

Figure 15. College building Zurich Falletsche: filigree vertical CFRP prestressed HPSCC façade columns

6. Conclusions and Design Recommendations

The prefabrication of CFRP prestressed high performance concrete (HPC) elements permits the realisation of thin-walled, light-weight and durable structural elements with excellent service characteristics (no susceptibility to corrosion or ageing, high bending stiffness and high fatigue strength). Lower cost for maintenance, transportation and installation with respect to conventional reinforced concrete elements make CFRP prestressed HPC economically interesting. The new shaping possibilities in conjunction with advanced concrete compaction techniques (self compacting HPC) make these novel concrete elements fascinating from the aesthetic/architectural point of view.

A first full-scale structural element made from HPC prestressed with pultruded CFRP wires was realised in the year 2000 by the Swiss prefabrication element plant SACAC Schleuderbetonwerk AG in the form of a thin-walled overhead power line pylon. This 27 metres high tubular pylon for high voltage transmission lines was produced by a centrifugally casting technique with a concrete wall-thickness of merely 48 mm. It was therefore 40% lighter than a conventional steel-reinforced concrete pylon used for the same purpose. The transport and erection costs were reduced which, with an expected maintenance-free service life of 50 years, should result in lower life-cycle costs than those of tubular steel or steel lattice pylons which need a new coat of corrosion protection paint after approximately 20 years. Using CFRP prestressed HPC has also proved beneficial in other pilot applications: For example, in the spring of 2001, a series of filigree (minimum profile wall thickness of 50 mm) balcony sills and window sills 3.6 to 5.3 m long were manufactured for a multiple dwelling in the conurbation of Zurich. In 2002 this technology, originally developed in close collaboration with the Swiss Federal Laboratories for Materials Science and Technology Empa, was further optimized by SACAC to produce durable and lightweight CFRP prestressed concrete lighting columns. A series of the first type of column from this product line was subjected to a very detailed experimental certification procedure with static, dynamic and durability tests at the University of Cambridge in England. Since 2003 these durable, lightweight, 4.2 to 18 m high lighting columns are produced and sold on the Swiss market. Furthermore, in 2005-2006 aesthetics, installation and structural considerations lead to the realisation of two large building façades in Zurich, Switzerland using slender and thin-walled CFRP prestressed self compacting HPC profiles. All these applications have shown outstanding durability up to the current service time of 12 years, with no cracks arising or relevant deflection increases under service loads in outdoors mid-European climatic conditions.

The thin-walled HPC elements prestressed with CFRP wires described are designed as members subjected primarily to flexure. In terms of design criteria, the following has to be considered:

• The total bending safety factor for these elements is typically in the range of 1.8, meant as the ratio between design failure moment to maximum service moment according to (SIA, 2003). No preference is made between the two possible bending failure modes, i.e., “crushing of the concrete” or “tensile failure of the CFRP wires”, since both of them are brittle failures. The controlling failure mode depends on the section's geometry, the concrete's strength, the configuration of the prestressing reinforcement (at present the cost determining factor) and the prestressing level. The latter is determined by the serviceability issue of fully prestressing the element for service loads. Furthermore, the anchorage of the highly prestressed CFRP wires must be carefully verified (Terrasi, 2001, 2012).

• In most cases, the design is controlled by the fulfillment of the serviceability criteria. In particular, deflections under service loads can be limited by fully prestressing: The formation of bending cracks is avoided so that the moment of inertia of the entire cross section is available for the service moment reception. The maximum service moment defines the minimum cracking moment that has to be guaranteed at the critical cross section of the element. This can be calculated under consideration of a part of the tensile strength of the concrete (f_{ct} = 5 MPa for the HPC studied) and is principally controlled by the height of the total prestressing force (Terrasi, 1998). In order to make rational use of the still expensive CFRP wires and considering the space limitations in the slender elements, the wires should be prestressed at an initial prestressing degree of 60%-70% of their guaranteed tensile strength (ACI, 2004) of 2200 - 2500 MPa (Table 1). It is advisable to use small diameter CFRP wires (typically 3–6 mm) in order to obtain a smooth transfer of the prestressing forces, to prevent bond failures through longitudinal cracks of the low HPC covers (in the range of 20 mm) and to elude thermal compatibility cracks (Matthys, 1996). A complete loss of prestress would be the consequence of these cracks and has clearly to be avoided (Terrasi, 1998).

• One additional requirement to be fulfilled by the chosen design of the flexural element is the attainment of a high deflection at failure (i.e. in the cracking stage, see Figures 8 and 9) (Terrasi, 2004), which can be interpreted as a warning by the system when loaded at an unforeseen high level.

The fulfilment of the design criteria listed above allows a weight reduction of up to 40% over traditional steel reinforced and prestressed concrete flexural elements.

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