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# EFFECTS of ELEVATED TEMPERATURES and FIRE on BOND STRENGTH of PRESTRESSED STEEL and CARBON FRP BARS in HIGH PERFORMANCE SELF-CONSOLIDATING CONCRETE

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## Abstract

Novel structures are emerging utilizing high performance, self-consolidating, fibre-reinforced concrete (HPSCC) reinforced with high-strength, lightweight, and non-corroding prestressed reinforcement. One example of this is a new type of precast carbon fibre reinforced polymer (CFRP) pretensioned HPSCC panel intended as load-bearing panels for building envelopes. As for all load-bearing structural members in building applications, the performance of these members in fire must be understood before they can be used with confidence. In particular, the bond performance of CFRP prestressing reinforcement at elevated temperatures is not well known. This paper examines the fire performance of these new types of structural elements, placing particular emphasis on the bond performance of CFRP and steel wire prestressing reinforcement at elevated temperatures. The results of large scale fire tests and bond-pullout tests on CFRP and steel prestressing bars embedded in HPSCC cylinders are presented and discussed to shed light on the fire performance of these structural elements. An analytical model is proposed to determine the temperature at which first appearance of the cracking phenomenon and concrete cover failure occurred, in the large scale fire tests, due to incompatibility of thermal expansion between CFRP tendons and HPSCC. From this research it is proposed that temperatures in range of the glass transition temperature ($T_g$) of the CFRP tendon's epoxy matrix are critical for the bond strength capacity of the CFRP tendons in reinforced or pretensioned concrete members.

*Keywords:* fibre reinforced polymer, fire, high temperatures, pullout, thermal expansion, cracking, spalling

## Introduction

Current trends in construction are forcing the development of more durable and sustainable concrete structures. Careful selection, design, and optimization of both the concrete mixes and the reinforcing materials used are now commonplace. One result of this has been the emergence of structural elements incorporating optimized, high-performance, self-consolidating, fibre-reinforced concrete (HPSCC) and novel reinforcing and prestressing materials such as carbon fibre reinforced polymer (CFRP) tendons, which are high-strength, creep resistant, lightweight, non-corroding, and magnetically invisible. One example of such an element is precast CFRP pretensioned HPSCC members used as load-bearing panels for building envelopes (Figure 1). However, the performance of these HPSCC precast members in fire is not well known and must be understood before they can be used with confidence.

The bond between both steel and FRP reinforcing bars (prestressed and non-prestressed) and concrete deteriorates at elevated temperature. Indeed, for FRP reinforcement, bond strength reductions are thought

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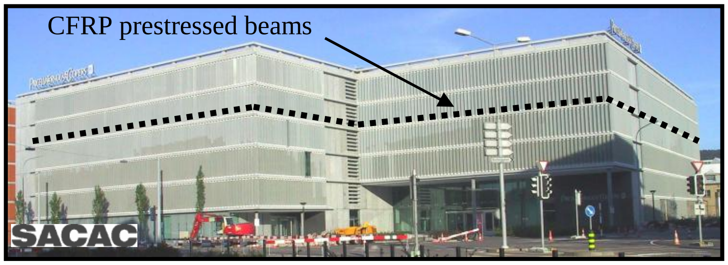

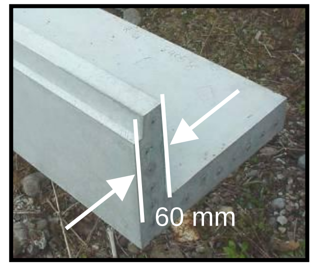


Figure 1. Use of precast CFRP pretensioned HPSCC members in a building envelope (Terrasi, 2007)

to be a limiting factor for the fire-safety of FRP reinforced or prestressed concrete (Bisby, Green and Kodur, 2005), although the precise magnitude of bond strength reductions and their impacts on the load-bearing capacity of heated reinforced (or prestressed) concrete structures have not been studied and remain unknown. To address all of the above issues, an ongoing study is underway which includes: (1) large-scale furnace tests on CFRP prestressed HPSCC panels, (2) transient bond pullout tests on CFRP tendons and steel prestressing wires, and (3) micro-mechanical characterization of the CFRP tendons. The goal in all cases is to better understand the response of CFRP prestressed HPSCC panels and to determine the factors that should be considered in their fire-safe design and application.

## Large Scale Fire Tests

Seven large scale fire tests were performed on CFRP prestressed HPSCC slabs in a floor furnace at EMPA (ISO 834 Fire). Figure 2 shows schematics of the specimens and test setup, and Table 1 provides details of the experimental program. Initial scoping tests (not presented) performed on small scale slabs (Terrasi, Stutz, Barbezat and Bisby, 2010) indicated that loss of bond between the FRP tendons and the concrete was a governing factor in determining the fire resistance of the CFRP prestressed slabs. Thus, the testing programme included slabs with unheated overhangs (of varying length) at each end to provide a cold anchorage region during fire testing. The smallest anchorage length (160mm) represented the room temperature prestress development length for a tendon stress of 1200MPa, as determined from previous testing. The slab thickness, and hence the cover to the reinforcement, varied between 45mm and 75mm. The 100MPa concrete incorporated 2kg/m$^3$ of short polypropylene (PP) fibres and had high moisture content at the time of testing (4.4-4.8%). The load in the central span corresponded to a typical service load condition (Terrasi, 2007). One slab used 6mm diameter cold-drawn steel prestressing wire stressed to 1200MPa while the others had 5.4mm diameter CFRP tendons.

The fire resistance of the slabs varied between 24min and 91min, with the thicker slabs generally achieving higher fire endurances. The notable exception was the steel prestressed slab, which suffered severe spalling early in the test (likely due to its very young age at the time of testing). This test is being repeated in May 2010 to properly account for the differences in slabs' age at the time of testing.

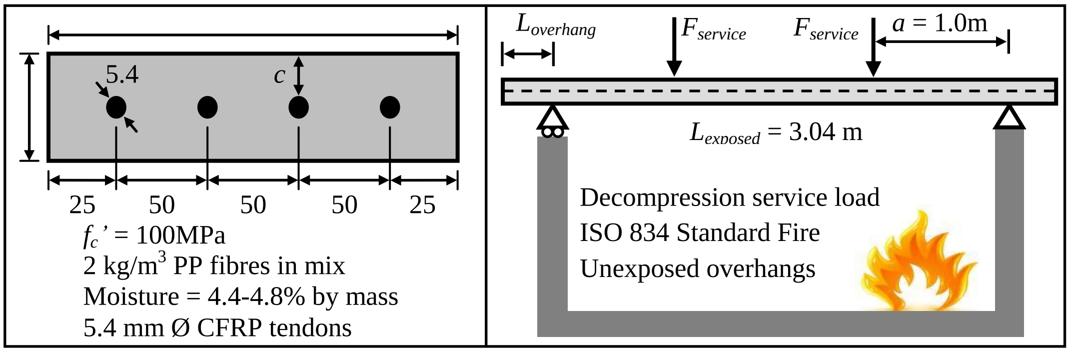


Figure 2. Details of fire test specimens and fire test setup

Despite including PP fibres in the mix, the dominant failure mode was sudden collapse due to accumulated HSPCC spalling, which reduced the slabs' cross-sections until they failed in bending due to crushing of the remaining concrete under the service load. Spalling was first localized in the shear and bending span (i.e. near the supports where the bending moment is low and the exposed face of the slab is most precompressed). It is widely known that concrete's propensity for spalling is increased by compressive stress, so the location of first spalling is unsurprising.

Longitudinal splitting cracks were observed on the non-exposed surface and to a smaller extent on the exposed surface, prior to failure for 45 and 46mm thick slabs and, to a smaller extent, for the thicker slabs. These were possibly caused by thermal incompatibility between HPSCC and CFRP tendons. Significantly, tendon slip versus time measurements showed no evidence of slip increases during the tests, indicating that the anchorage length of 160mm was sufficient to prevent bond failure. Tendon temperatures recorded in the fire exposed spans during these tests indicated that the tensile strength of the CFRP was maintained at temperatures above 330°C. Full details of these tests will be presented elsewhere (Terrasi et al., 2007).

Table 1. Large scale fire test programme and results (Refer to Figure 2)

| No. | Age (mths) | Tendon type | Prestress (MPa) | Cover *c* (mm) | Thickness *t* (mm) | Overhang (mm) | Failure time | Failure mode |
|---|---|---|---|---|---|---|---|---|
| 4 | 9.3 | CFRP | 1200 | 19.8 | 45 | 160 | 26'12" | spalling → crushing |
| 7 | 8.8 | CFRP | 1200 | 20.3 | 46 | 280 | 34'36" | spalling → crushing |
| 8 | 8.4 | CFRP | 1200 | 28.3 | 62 | 280 | 24'12" | spalling → crushing |
| 5 | 8.4 | CFRP | 1200 | 27.8 | 61 | 160 | 47'00" | spalling → crushing |
| 9 | 9.3 | CFRP | 1200 | 34.8 | 75 | 280 | 1h00'24" | spalling → crushing |
| 6 | 9.3 | CFRP | 1200 | 34.8 | 75 | 160 | 1h31'36" | spalling → crushing |
| 40 | 1.0 | Steel | 1200 | 34.8 | 75 | 160 | 29'00" | spalling → crushing |

## Bond Strength at Elevated Temperature

Past research on the bond performance of FRP and steel reinforcing bars at high temperatures has often been performed by heating a pullout sample without any load applied and then loading it to failure once a target temperature is reached (Lublóy, Balázs, Borosnyói and Nehme, 2005; Bingöl and Gül, 2009). This is not representative of conditions in a prestressed concrete structure in a real fire, where materials are heated under sustained load. Furthermore, in pretensioned prestressed elements there is considerable bond strength demand throughout the structure's lifetime needed to develop and maintain the required prestressing forces. It seems likely that sustained stresses are likely to be much more important than short terms loads for CFRP tendons at elevated temperature, since the tendon's epoxy matrix may undergo considerable creep deformation under sustained load at elevated temperature. Thus, in the current research bond pullout testing has been performed by applying a sustained load to a predefined bonded length and then heating the bond line at a prescribed rate until failure occurs; this is more representative of the state of stress within a real FRP prestressed structural element during a fire. Although the stress conditions in a reinforced concrete element differ greatly from those produced in a pullout test (ACI 408R, 2003), this type of test has been widely adopted in the assessment of bond performance of steel reinforcing bars in concrete. The pullout test setup adopted in the current study is shown in Figure 3.

A total of 18 pullout specimens were tested, nine with CFRP tendons and nine with steel prestressing wire. Round, sand-coated unidirectional CFRP tendons supplied by SACAC, Switzerland were used in the pullout tests (identical to those used in the large scale tests described previously). The tendons' design ultimate tensile strength is 2000MPa, with an elastic modulus of 150GPa, a linear-elastic tensile stress-strain response to failure, and an ultimate strain of 1.33%. The maximum prestress level for the CFRP tendons is governed by anchorage issues during stressing and is currently 1200MPa. To compare the bond performance of the CFRP tendons against conventional steel prestressing wire, 6mm diameter steel wire produced by NEDRI Spanstaal BV, specifically for prestressing applications, was also studied (identical to that used in Slab 40 in Table 1). The wire's design yield strength is 1592MPa (0.2% offset) and its modulus is 210GPa.

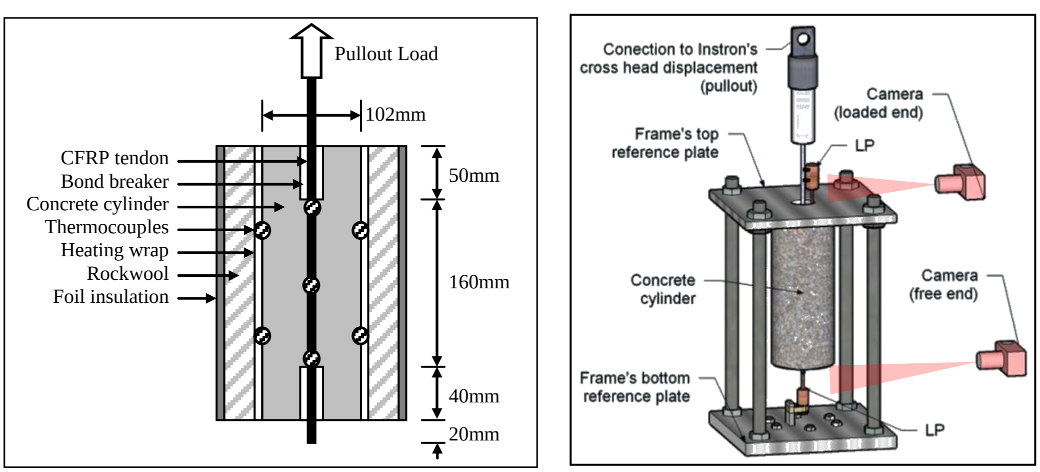


Figure 3. Schematic of pullout test specimens and experimental setup for pullout tests

The relatively high cost of CFRP tendons necessitates a correspondingly high quality concrete. A high performance self-consolidating concrete (HPSCC) was designed for a strength of 90-100MPa at 28 days; 2kg/m3 of short PP fibres were included in the concrete to simulate the concrete mix used in the fire tests. The pullout samples were 102mm Ø concrete cylinders with a length of 250mm. The moulds were designed in such way that the tendons/wires were placed vertically and axisymmetrically. The tendons/wires were debonded over a portion of their embedded length at the top and bottom of the cylinders to: (1) allow for the bonded length to be equal to the prestress transfer length of the CFRP prestressing tendons (160mm); (2) prevent localized artificial confinement of the bonded length due to compressive load on the concrete at the loaded end; and (3) promote an axisymmetric heat transfer condition along the bonded length and assure a uniform bond line temperature. Special consideration was given to the accuracy of the measurements of bar slip at both the loaded and free ends of the specimens. A unique digital image correlation analysis (White and Take, 2002) was used to measure slip (Figure 3).

The behaviour of the specimens during heating turned out to be far more complex than expected, resulting in three distinct types of bond tests:

1. Regular pullout test (RPOT): Specimens were loaded at room temperature under displacement control until pullout failure occurred.
2. Regular prestress and heated pullout test (Regular PHPOT): Specimens were loaded at room temperature to a prescribed load under load control and then heated under sustained load until pullout occurred.
3. Extended PHPOT: Identical to regular PHPOTs except that bond stress was insufficient to produce failure of the bond interface on heating. After 230mins at steady state temperature the load was increased until failure.

The prescribed sustained loads were taken as increasing percentages of the average strength of RPOT tests: for CFRP tendons these were 15, 30, 38, 45, 53, 60 and 68% and for steel prestressing wires they were 37, 46 and 55%.

**Steel Pullout Test Results**

A summary of the test results for all of the pullout tests is presented in Table 2. Steel RPOT samples failed by tensile rupture of the steel wire at the loaded end (i.e. the bond failure capacity was greater than the tensile failure capacity of the wire). The average RPOT failure load of $\tau_{ave,\ s}$ = 16.6MPa was used to define subsequent test loads.

Extended PHPOTs executed on steel pullout samples were stressed to 37% and 46% of $\tau_{ave,\ s}$. Under $0.37\tau_{ave,\ s}$ the samples were heated to a bond line temperature of about 162°C and were then loaded to failure of the bond interface, which occurred at an average bond stress of 12.6MPa. Under 0.46$\tau_{ave,\ s}$ the samples were heated to about 166°C and were then loaded to failure of the bond interface, which occurred at an average bond stress of 13.2MPa.

Regular PHPOTs on steel tendons were loaded at a sustained stress of 0.55$\tau_{ave,\ s}$. In these cases failure occurred by transverse splitting failure of the concrete cylinder in less than six minutes of heating when the average bond stress was being maintained at 9.1MPa and the bond line temperature had not yet increased. Failure of these tests was likely the result of the concrete's tensile strength being exceeded due to the summation of the mechanical stresses produced by the pullout conditions and the thermal stresses produced by the steep thermal gradient in the concrete.

The results show that the bond performance of steel prestressing wire in concrete is influenced by a number of parameters, notably including transverse splitting cracking of the concrete. However, on the basis of the current results it appears that the bond between the steel wires and the concrete was reduced by up to 30% at a temperature of 166°C. Considerable additional research is needed before meaningful conclusions can be drawn. Such research appears to be warranted.

### CFRP Pullout Test Results

CFRP RPOT samples failed by slipping at the bond interface between the sand coating on the CFRP tendons and the tendon. After failure occurred, pullout continued at a constant slip rate and a residual bond strength capacity of 66-77% of the peak load was measured. The average RPOT failure load was $\tau_{ave,\ f}$ = 4.9MPa.

Table 2. Pullout test programme and results

| Test | Prestressing Condition | | | Failure Condition | | | | | Residual bond strength (MPa) |
|---|---|---|---|---|---|---|---|---|---|
| | Bond stress (MPa) | Tensile stress (MPa) | Pullout load (kN) | Bond stress (MPa) | Tensile stress (MPa) | Pullout load (kN) | Temperature Bar (°C) | Temperature Blanket (°C) | |
| CFRP RPOT | - | - | - | 5.3 | 633 | 14.5 | 21 | 24 | 4.1 |
| CFRP RPOT | - | - | - | 4.4 | 524 | 12.0 | 20 | 24 | 2.9 |
| CFRP 15% | 0.7 | 87 | 2.0 | 3.9 | 461 | 10.6 | 166 | 182 | 2.3 |
| CFRP 30% | 1.5 | 175 | 4.0 | 5.1 | 601 | 13.8 | 166 | 182 | 2.2 |
| CFRP 38% | 1.8 | 218 | 5.0 | 4.3 | 513 | 11.8 | 164 | 185 | 2.3 |
| CFRP 45% | 2.2 | 262 | 6.0 | - | - | - | 109 | 169 | 0.65 |
| CFRP 53% | 2.6 | 306 | 7.0 | - | - | - | 102 | 155 | 0.89 |
| CFRP 60% | 3.0 | 349 | 8.0 | - | - | - | 95 | 148 | 0.92 |
| CFRP 68% | 3.3 | 393 | 9.0 | - | - | - | 21 | 76 | - |
| Steel RPOT | - | - | - | 16.6 | 1774 | 50.2 | 22 | 24 | - |
| Steel RPOT | - | - | - | 16.6 | 1772 | 50.1 | 21 | 24 | - |
| Steel 37% | 6.1 | 648 | 18.3 | 11.8 | 1257 | 35.5 | 164 | 184 | - |
| Steel 37% | 6.1 | 648 | 18.3 | 13.5 | 1434 | 40.6 | 160 | 178 | - |
| Steel 46% | 7.6 | 810 | 22.9 | 12.8 | 1365 | 38.6 | 170 | 192 | - |
| Steel 46% | 7.6 | 810 | 22.9 | 13.7 | 1460 | 41.3 | 162 | 179 | - |
| Steel 55% | 9.1 | 972 | 27.5 | - | - | - | 31 | 155 | - |
| Steel 55% | 9.1 | 972 | 27.5 | - | - | - | 25 | 118 | - |
| Steel 55% | 9.1 | 972 | 27.5 | - | - | - | 25 | 105 | - |

Extended CFRP PHPOT samples were prestressed to 15, 30 and 38% of $\tau_{ave, f}$. In all cases the samples were heated to a bond line temperature of about 170°C and were then loaded to failure, which occurred at an average bond stress of about 4.4MPa. Again, after failure occurred, pullout continued and residual bond strength of 44-58% was measured. The residual bond strengths observed in Extended PHPOTs on CFRP were consistently larger than the bond stresses at which the bond interface failed *during* heating in Regular PHPOTs at higher stress levels. This suggests a time dependency of bond strength related to the duration of loading at a given stress level, likely related to the time-temperature-stress dependent creep properties of the polymer coating at the surface of the CFRP tendons.

Regular PHPOTs on CFRP were prestressed to 45, 53, and 60% of $\tau_{ave, f}$. These samples failed during heating with decreasing failure temperatures observed as the sustained bond stress level increased. Figure 4 shows a plot of loaded end slip versus bond line temperature for these three tests, where the correlation between bond stress, temperature, and slip initiation is clear. For regular PHPOTs, pullout continued after the initial bond failure as the loading frame attempted to maintain the load, and a residual strength of about 29-35% of the bond stress at which the tendons were prestressed was observed. The Regular PHPOTs on CFRP at 68% of $\tau_{ave, f}$ failed soon after heating began. This was likely initiated by a small slip produced by longitudinal thermal expansion of the concrete during initial heating.

On the basis of the pullout tests on CFRP tendons, it appears that the bond between CFRP tendons and concrete is damaged by exposure to temperatures in the range of 90-120°C. Significantly, the glass transition temperature ($T_g$) for the epoxy matrix used in the fabrication of these tendons was measured at EMPA by DMTA as 125°C (as $T_g$-onset, the temperature above which the polymer softens and suffers a reduction in strength and stiffness of several orders of magnitude). This idea is supported by previous research studying the bond strength of Glass FRP bars used as non prestressed reinforcement for concrete structures (Bisby et al., 2005). However, considerable additional testing is needed to fully understand the complex interactions between time, stress, temperature, and strength that eventually lead to bond failure.

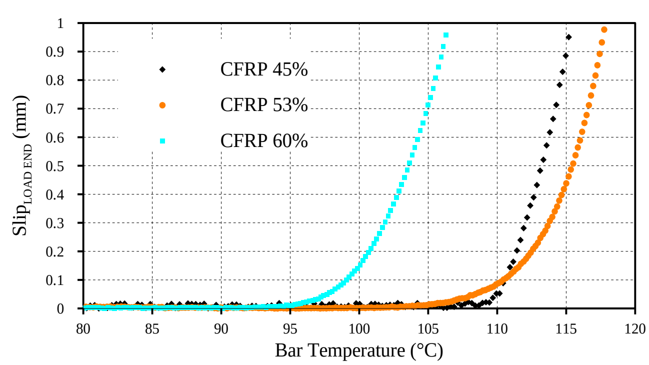


Figure 4. Loaded end slip versus bar temperature for Regular PHPOTs on CFRP tendons

## CFRP-HPSCC Thermal Incompatibility Model

An analytical model proposed by Aiello, Focacci and Nanni (2001) was modified to determine the temperatures corresponding to the first appearance of longitudinal cracks in the large scale fire tests performed at EMPA. The transverse section of a slab (45mm thick) tested in the large scale fire tests is shown in Figure 5. This model is a theoretical, analytical approach to predicting the cracking phenomenon observed in these tests, and is based on the assumptions that:

- each CFRP tendon is treated independently, meaning that the clear spacing between two adjacent tendons is sufficient to avoid the occurrence of horizontal splitting cracks at the tendon's level;
- absence of CFRP tendon's boundary conditions at the bar terminations with the aim of focusing the analysis on the effects of stress interaction between CFRP tendons and HPSCC produced by thermal actions;
- the temperature in the CFRP tendon and concrete increases uniformly in concrete and CFRP tendon (i.e. there is no thermal gradient) with the aim of focusing the analysis on the effects mentioned above; and
- the elastic modulus and tensile strength of HPSCC decreases at high temperatures.

This analytical model describes the effect of a temperature increase $(\Delta T)$ on a CFRP tendon embedded in a concrete cylinder, the diameter of which is equal to the slab's thickness. A radial stress $(\sigma_T)$ acts at the concrete cover, as shown in Figure 5. Three cases must be considered (see Figure 6); (a) before the stress in concrete reaches the tensile strength, (b) after the first cracks appear, and (c) after the cracks reach the outer radius of the cylinder. In CFRP tendons, the epoxy resin plays an important role in the thermo-mechanical properties of the CFRP tendon's transverse direction. Steel reinforcements have relatively similar values of the coefficient of thermal expansion (CTE) to that of concrete. Similar CFRP tendons to the ones used in this study have more than double the CTE, in the transverse direction, of that of concrete (ACI 440.4R, 2004). The transverse CTE of CFRP tendons can be up to six times that of concrete in some commercial FRPs brands (Gentry and Hudak, 1996).

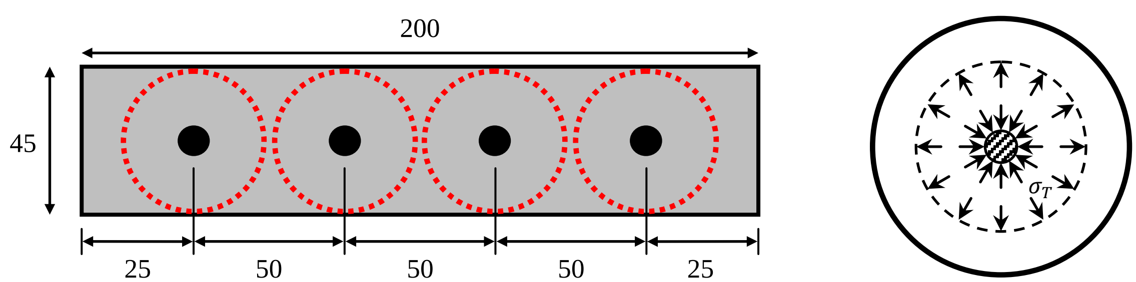


Figure 5. CFRP tendon and concrete effective area for a 45mm thick slab from the EMPA large scale fire test and radial stress acting at the interface CFRP-HPSCC under temperature increase

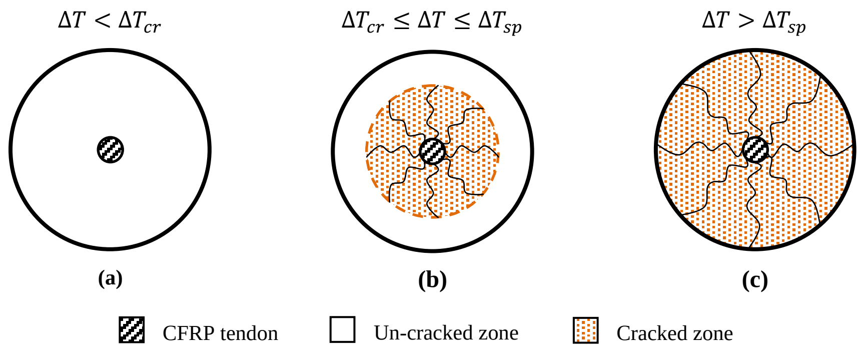


Figure 6. Three cases of the thermal incompatibility in the analytical model

The model was executed for the slabs tested in the EMPA large scale fire test with 45, 62 and 75mm thickness. The temperature, at which longitudinal splitting cracks first appeared on the slabs' top surface in the large scale fire tests was not recorded. Further experimental tests should be executed to validate this model. The model was executed under two conditions:

1) The system (CFRP tendon and concrete) increases its temperature uniformly, which is a first order approximation modelling of what happens at the fire exposed surface of the slab.
2) The concrete was kept at ambient temperature (20°C) while the tendon was heated, which is a first order approximation modelling of what happens at the non-fire exposed surface of the slab – ST.

Results of the model were plotted in a temperature-crack tip location graph for both conditions (see Figure 7). The plot indicates the temperature at which the crack first initiated (red circle) and the temperature at which the crack reached the outer radius of concrete (green rhombus).

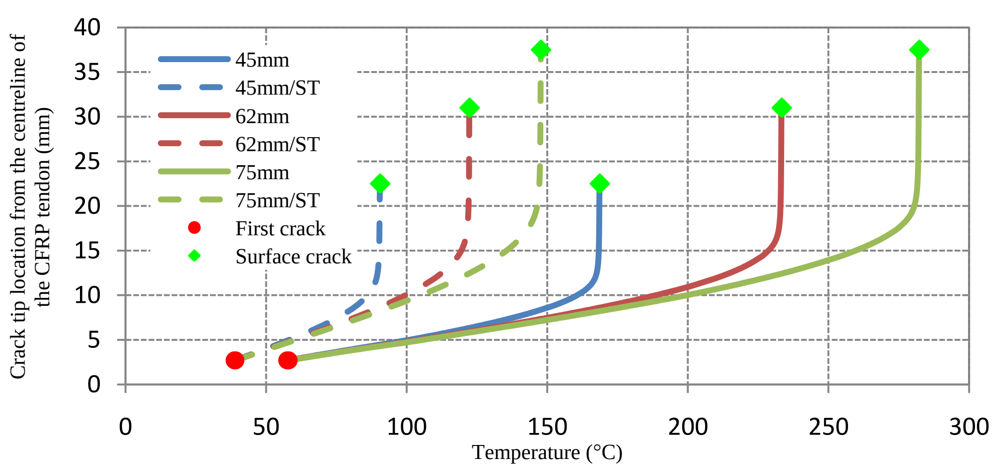


Figure 7. Correlation between crack propagation and temperature

Independent of the amount of concrete cover, the crack first initiated at 58°C for the models in which the concrete experienced a uniform increase of temperature, and at 39°C for the models in which concrete was kept at ambient temperature while only the tendon was heated.

As the crack propagated from the CFRP tendon, unstable crack propagation occurred as it approached the outer radius of concrete. Relative to the concrete cover, the crack reached the outer radius of the concrete at 169, 233 and 282°C (slabs 45, 62 and 75mm thick respectively) for the models in which concrete uniformly increased in temperature. For the models in which concrete was kept at ambient temperature while the tendon was heated, the crack reached the outer radius at 91, 122 and 148°C (slabs 45, 62 and 75mm thick respectively).

The model revealed a possible explanation for why longitudinal cracks were observed in the large scale fire tests earlier on the top (unheated) surface of the slabs. Even though the exposed surface of the slab suffered tensile strength and elastic modulus degradation of concrete, the crack initiated and reached the surface of the slab at lower temperatures for the case of cracking on the unexposed face. Additional modelling is needed to better understand the implications of the thermal gradient in the concrete in the real fire tests on the appearance of splitting cracks in the concrete during the transient heating of a fire test.

## Summary & Conclusions

Several conclusions can be drawn on the basis of the data presented herein:

- The HPSCC concrete used in these elements experienced considerable spalling, which eventually led to structural failure during large scale furnace tests. This was despite the concrete containing 2kg/m$^3$ of short PP fibres. Additional research to mitigate spalling is badly needed. When CFRP anchorage is maintained, CFRP prestressed concrete can perform as well or better than steel prestressed concrete in fire, achieving fire resistances of 30mins or more.
- Loss of bond (anchorage) is potentially a governing factor for CFRP prestressing tendons in concrete at elevated temperatures. It appears that temperatures in range of the glass transition temperature ($T_g$) of the tendon's epoxy matrix (used also for the bond enhancing sand coating) are critical for maintaining anchorage.
- A discussion should be made about the appearance of longitudinal cracks for slabs tested in large scale fire tests was an effect of the thermal incompatibility of the orthotropic CFRP tendon with the HPSCC. The experimental data and the thermal compatibility model proved that cracking was more common on the non-exposed surface of the slab. Since most reinforcements have high thermal conductivity, under fire conditions, it is very common to

have sections in which reinforcement has a high temperature in a non-exposed section of the structural element, which is the worst case scenario for the proposed model.

- Many aspects of bond performance at elevated temperature (for both FRP tendons and steel prestressing wires) remain poorly understood and require additional investigation.

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