Fire endurance and residual strength of insulated concrete beams strengthened with Near Surface Mounted reinforcement

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Abstract: The comparatively weak performance of fiber reinforced polymer (FRP) strengthened members under fire exposure is a primary factor hindering the application of FRP strengthening in buildings in which fire risk is not negligible. As part of a larger ongoing study investigating the behavior of FRP strengthening systems under fire exposure an investigation was set-up to examine and document the performance of 6 full-scale near surface mounted (NSM) FRP reinforced concrete beams exposed to 1 hour of fire. All the specimens were pre-loaded to the service load of the strengthened member. Tests results indicated that, if appropriately insulated, the NSM FRP strengthened beams can achieve a satisfactory fire endurance of 1 hour as per EN1363-1 specifications. Moreover this paper presents also a study on the residual performance of fire tested beams. Results of this study suggest that, if the insulation system is able to maintain the adhesive temperature at relatively low value ($T_{adhesive} \leq 1.6 T_g$ for the beam configuration in this test program), the FRP concrete bond degradation under fire is limited and the FRP strengthened beam can retain a large part (in this test program up to 92%) of its original strength.

Keywords: Fiber reinforced polymer; Fire safety; Insulation; Residual strength; Structural members; Strengthening; NSM

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Introduction

Fiber reinforced polymer (FRP) materials are currently produced in different configurations and are widely used for the strengthening and retrofitting of concrete structures. The use of FRP bars and strips as near surface mounted (NSM) is a widely applied strengthening technique (Sena Cruz et al. 2004; De Lorenzis et al. 2004; Kotynia 2005; Teng et al. 2006; Seracino et al. 2007; Palmieri et al. 2010). FRP bars or strips are installed by grooving the surface of the member and embedding the FRP reinforcement into the grooves with high strength adhesive (epoxy or mortar). This method is relatively simple and enhances the bond of the FRP reinforcement, thereby using the material more efficiently (debonding typically at 70-80 % of the ultimate strain which is at higher levels than typically obtained for externally bonded reinforcement; Kotynia 2005, Raafat et al 2003).

Moreover NSM is particularly attractive for the flexural strengthening in negative moment regions of slabs and decks, where external reinforcement would be subjected to mechanical and environmental damage and would require protective cover. One of the main concerns in implementing FRP materials in buildings for which fire risk is not negligible is their weak performance under elevated temperature and fire exposure. Deterioration in mechanical properties of FRPs and adhesives, as well as reduction of bond strength at the concrete-adhesive interface can be expected at high temperatures (Blontrock et al 1999; Kodur et al. 2007; Wang et al. 2007, Foster et al. 2008, Weber A. 2008, Nigro et al. 2011a-b, Katz 2012, Nigro et al. 2012). This gives potential concerns regarding the structural integrity of FRP strengthened concrete structures during fire exposure.

Indeed as the temperature of the polymer matrix approaches its glass transition temperature, T_g , the matrix transforms to a soft, rubbery material with reduced strength and stiffness. Thus for epoxy resins, currently used as primer, adhesive and matrix for FRP strengthening systems the degree of reduction of the mechanical properties at temperatures close to their T_g (the T_g of ambient cured epoxies is usually in the range of 50-90 °C, as cited in Blontrock et al. 1999) is of relevant importance for the strengthened structures, mostly in relation to the bond performance. Due to the degradation of FRP materials at high temperature, guidelines for design of FRP strengthened structures (ACI 440.2R-2008 and fib Bulletin 14 2001) require that the strength of FRP is to be ignored unless a fire-protection system is used that can maintain the FRP temperature below its critical temperature (currently defined as the lowest T_g of its components). Thus, the use of FRP strengthening systems is mainly limited to applications where fire

aspects are not critical or where loss of the FRP under acting loads during fire are not critical for the structural integrity (as required by Equation 9.2 in ACI440.2R-08).

A number of research projects (Bisby et al 2008, Blontrock et al. 2001, Stratford et al 2010, and Williams et al. 2008) have shown that with an appropriate insulation, concrete structures strengthened with FRPs can achieve a satisfactory fire endurance rating though contribution of the FRP is generally assumed as lost during fire exposure. Kodur et al. (2010) have presented a numerical model for evaluating the fire performance of FRP RC (reinforced concrete) strengthened beams under fire conditions and concluded that supplemental fire insulation is often needed to satisfy fire resistance requirements by providing additional protection to the concrete and longitudinal tension steel. Fire tests were performed (two fire test series for a total of 12 NSM FRP strengthened and insulated beams), by the authors (Palmieri et al 2012) to evaluate the fire endurance of insulated and NSM FRP strengthened beams exposed to a standard fire of 2 h while subjected to the service load of the strengthened beam. In line with previous work (Blontrock et al. 2001, Bisby et al 2005, Bisby et al 2008, Burke et al. 2008, Foster et al. 2008, Williams et al 2008, Chowdhury 2008, Kodur et al 2010) the findings showed that the beams can achieve 2h of fire endurance ratings even after the adhesive temperature exceeds excessively the glass transition temperature. This relates to the fact that in case of accidental loss of FRP, load levels are generally limited to avoid collapse. Hence, fire ratings with respect to the concrete and the internal steel rebars are governing, considering the acting load level during fire. Care should be given to the fact that this acting load level is higher than for an unstrengthened reference situation, so that available concrete cover is not per definition sufficient. An insulation system might, depending on the situation (level of concrete cover and level of acting load), be needed and increases the overall fire rating in any case.

However, residual flexural strength tests on fire tested beams previously tested (Palmieri et al. 2012) have tentatively demonstrated that in some cases the FRP seems to be able to retain part of the bond strength to the concrete for the beams where the adhesive temperature remained less than about 2.5 times T_g . Therefore, the research presented in this article focuses on this aspect of critical FRP bond adhesive temperature in relation to the glass transition temperature, for RC beams strengthened in flexure with NSM. In this respect the authors designed the insulation to limit the adhesive temperature, such to avoid significant dysfunction of the FRP in terms of stress transfer compatibilities between the FRP and the RC beams during or after fire.

The presented research can be considered as an extension of the previous work (Palmieri et al 2012).

Indeed six additional concrete beams, with the same configuration and mechanical properties and using the insulation materials which gave the best results in the preceding fire test, were tested under fire in order to: (1) investigate the reliability of the previous test results; (2) investigate if the NSM FRP strengthened system is active during fire by testing one of the beams till failure at 1 hour of fire exposure; (3) investigate the adhesive bond degradation at temperature higher than the adhesive glass transition temperature (in order to do this, different insulation thicknesses have been investigated in order to achieve different temperatures into the adhesive and a time of 1 hour of fire exposure was chosen to avoid loss of composite action due to an excessive heating of the adhesive); and (4) investigate the influence of using an expansive mortar, alternative to epoxy based adhesive. In view of point (3) structural testing to failure at room temperature of the fire tested beams was carried out to evaluate their residual strength after fire exposure.

Experimental program

Test specimens

The testing program involved the design and fabrication of 10 steel reinforced concrete beams with rectangular cross section (200 x 300 mm in cross section) simply supported over 3000 mm clear span and tested in four point bending with a shear span of 1000 mm and a constant moment region of 1000 mm. Details of the specimens are shown in **Figure 1**. The longitudinal lower reinforcement consisted of two steel rebars with diameter 16 mm, the longitudinal upper reinforcement consisted of two steel rebars with diameter 10 mm. Shear reinforcement consisted of steel stirrups with 8 mm diameter spaced at 100 mm and 150 mm (see **Figure 1**). The concrete cover was 30 mm. The FRP reinforcement consisted of CFRP (carbon fiber reinforced polymer) rods (type Aslan 200, supplied by Fortius/Hughes Brothers) with nominal diameter of 9.53 mm, and GFRP (glass fiber reinforced polymer) rods (type Combar, supplied by Schöck) with a nominal diameter of 12 mm. The groove dimensions (see **Figure 1**) were defined in order to be at least 1.5 times the diameter of the NSM FRP bars. Four reference beams (one unstrengthened and three strengthened beams) were tested to failure at room temperature (Palmieri et al. 2010). Six strengthened and insulated beams were subjected to fire.

Materials properties and insulation materials

The concrete mixture design was identical for all the beams and incorporates siliceous aggregates with a maximum diameter of 16 mm. The average cylinder compressive strength, f_c , at the time of testing (196

days after casting the concrete) was 42.0 N/mm^2 . The GFRP rods, as reported by the manufacturer, had 1350 MPa tensile strength and 60 GPa Young's modulus, the CFRP bars 1900 MPa tensile strength and 126 GPa Young's modulus (as reported by the manufacturer). An epoxy resin (type Sikadur-30, supplied by Sika) with a tensile strength of 30 MPa and elastic modulus of 12.8 GPa (as reported by the manufacturer) and a glass transition temperature, T_g , of 62°C was used to embed the GFRP bars into the grooves. For the CFRP bars two kind of adhesives were used: an epoxy resin (type Fortresin CFL, supplied by Fortius) with a tensile strength of 32 MPa (as reported by the manufacturer) and a glass transition temperature, T_g , of 65°C was used to embed the CFRP bars into the grooves of beam B2-F3-1 and an expansive cementitious mortar (type Sikagrout 212, supplied by Sika) with a tensile strength of 4.1 MPa (as reported by the manufacturer) was used to embed the CFRP bars into the grooves of beam B4-F3-1. An overview of the fire test matrix is given in **Table 1**. The glass transition temperature of the two resins was experimentally determined on the basis of DSC (differential scanning calorimetry), according to ISO 11357-2. The investigated fire insulation systems were: a calcium silicate protection board (Type Promatect L-500 supplied by Promat) and one insulation system composed by two ceramic based coating (type Hot Pipe Coating and Omega Fire supplied by Superior Product Europe). **Figure 2** shows the layout of the different fire insulation systems. As in previous experimental results (Palmieri et al. 2012) beams B1-F3-1 and B2-F3-1 were protected with Promatect L-500 fixed in a U-shaped form. The thickness of the plate at the bottom was 100 mm for beam B1-F3-1, composed of Promat L- 500 200 mm wide plates with a thickness of 50 mm, as indicated in **Figure 2**. The first 50 mm of protection was composed of two plates with a length of 1450 mm joint together in the length direction by means of silicate glue (type Promacol k84 supplied by Promat), so to achieve a total length of 2900 mm. These two plates were attached to the bottom surface of the beam by mean of silicate glue. At the bottom of the first layer, two additionally plates with the same geometry were mechanically fixed with screws placed at a distance of 200 mm in longitudinal direction. The configuration of the bottom protection of beam B2-F3-1 was the same adopted in beam B1-F3-1, but plates with a thickness of 30 mm were used. The insulation at the side faces was composed of Promat L-500 180 mm wide (beam B1-F3-1) and 140 mm wide (for beam B2-F3-1) with a thickness of 20 mm (previous tests, Palmieri et al. 2012, have shown that a side thickness of 20 mm was enough to delay the increase of temperature in the longitudinal steel reinforcement and FRP bars). Screws, to create the U-shaped insulation form, are provided each 150 mm for the connection between the side plates and bottom plates and each 200 mm for the connection of the

side plates to the beam. The screws were additionally protected with silicate glue in order to minimize the effect of screws on heat transfer. Beams B1-F3-2, B1-F3-3 and B1-F3-4 were insulated with Hot Pipe Coating (HPC), which was spray-applied with a thickness of 25mm (beam B1-F3-2), 35 mm (beam B1-F3-3) and 20 mm (beam B1-F3-4) to the bottom and 10 mm to the sides. The total thickness was built up in different layers in a range of 0.2-1 mm each. On top of the HPC a layer of Omega Fire was spray applied with a thickness of 20 mm (beams B1-F3-2 and B1-F3-3) and 15 mm (beam B1-F3-4) to the bottom and 10 mm to the sides. For beam B4-F3-1, in which the NSM FRP bar was embedded with an expansive mortar, a layer of HPC and Omega Fire (10+10 mm at the bottom and the sides) was provided for the insulation. Insulation system details and properties, according to the manufacturers, are provided in **Table 1**. The fire insulation systems were applied to the beams over a total length of 2900 mm in order to avoid any damages of the fire protection by touching the furnace wall during the increase of beams deflection. The small gap (approximately 50 mm) between the furnace walls and the insulation was fulfilled with glass wool attached with Promakol k84.

Experimental set-up

The beams were tested simultaneously in a horizontal furnace of 6 m long by 3 m wide. During the fire test all the beams were loaded in 4 point bending in a purpose built loading frame as shown in **Figure 3**. The beams were mounted in a steel ring frame that was then lifted and placed on the top of the furnace chamber. The openings in the frame on either side of the beams were closed with aerated concrete slabs. These slabs were 150 mm deep and 600 mm wide and were insulated on their sides with 20 mm thick glass wool. Therefore, the beams were exposed to fire from three sides (bottom of the beams and lateral sides for a height equal to 150 mm see **Figure 3**), and the top surface was exposed to ambient temperature. The beams were placed in the transverse direction of the furnace (the clear span of the beams being 3 m) as shown in **Figure 3**. Fire testing standards require that structural elements need to resist the service loads during the fire test. Thus, before starting the fire test the beams were loaded to the service load of the respective strengthened beams ($Q_k = 2 \times 40.50$ kN for beam B2-F3-1; equal to 40% of the ultimate load, or 71% of the ultimate load of an equivalent unstrengthened beam; $Q_k = 2 \times 36.00$ kN for beams B1-F3-1, B1-F3-2, B1-F3-3, B1-F3-4; equal to 37% of the ultimate load, or 63% of the ultimate load of an equivalent unstrengthened beam and $Q_k = 2 \times 40.50$ kN for beam B4-F3-1; equal to 54% of the ultimate load, or 71% of the ultimate load of an equivalent unstrengthened beam). A calculation of the service load and ultimate load (ultimate load was calculate considering all the material

safety factors equals to 1) was made in accordance to Eurocode 2 and *fib* Bulletin 14. For all the beams the service load, Q_k , was governed by the allowable compressive stress in the service limit state (SLS). The service load was applied by a separate hydraulic unit in function of the required load, so that the beams were loaded in 3 groups. The load was kept constant during the fire test. A maximum of 1 h fire exposure was chosen; after which the test was halted. All the beams were exposed to EN 1363-1 standard fire; the furnace temperature was controlled to follow the standard time-temperature curve according to ISO 834. This standard prescribes the heating by the combustion gases as function of the time.

Instrumentations

The beams were instrumented to measure temperature distributions throughout the cross section. Twenty thermocouples, type K, were placed inside the concrete at two different locations (each section was placed at a distance equals to 375 mm from the middle of the beam) along the span of the beams. In each concrete section ten thermocouples were placed: one was placed at the bottom face of the concrete beam, one at the interface between the adhesive and the FRP reinforcement, two were placed on the lower steel reinforcement, one at the unexposed upper concrete surface and the remaining thermocouples in the concrete section. **Figure 4** shows a schematic representation of the thermocouple locations within the two sections. In addition a displacement transducer (LVDT) was connected to the unexposed surface of each beam to measure the deflection at midspan in the pre-load phase and during fire testing.

Test results and observations

Despite the different insulation types and/or thicknesses all the insulated beams could withstand the 1 hour fire test, while submitted to their service load. Visual observations during the fire test were made through a number of small view ports around the furnace walls. Upon completion of the test, the fire exposed faces of the beams were examined; **Figure 5** shows pictures of beam B1-F3-1, B1-F3-3 and B4-F3-1 after fire exposure. Except for some discoloration and small cracks in the lower bottom protection board of beams insulated with Promatect L-500 (beams B1-F3-1 and B2-F3-2), the fire insulation board system was intact and few signs of deterioration were observed (see **Figure 5.a** as reference). After fire exposure the boards were carefully removed and no signs of damage were observed to the adhesive. At approximately 18 min into the tests, for all the beams insulated with HPC/Omega fire system, several cracks were observed for the outer layer of Omega Fire (the temperature in the furnace was approximately equal to 770 °C at that time) with surface flaming of the product. At approximately 30 minutes into the test, in the two lateral sides of the beams, the Omega Fire insulation layer started to

detach (as shown in **Figure 5.b**). At the same time cracks, at the bottom, appeared to widen as the test progressed. At 50 minutes into the test the layer of Omega Fire detached from the HPC layer at the bottom of the beams (temperature in the furnace was approximately 900 °C). By observation, the HPC fire insulation system performed well for all the duration of fire exposure. Observations after fire exposure revealed that the HPC protection coating was consumed (the outer few millimetres of the layer burned away) in portions of the exposed face, presenting cracks for some of the beams in the outer layers (although the precise description of consumption of a certain amount of insulation or possible cracks could not be observed; see **Figures 5.b** and **5.c** for reference). After fire exposure the HPC layer was carefully removed and no signs of damage were observed to the adhesive (resin or mortar).

Thermal performance of the insulation

The performance of the insulation played a key role in limiting the temperatures in the concrete, steel rebars, FRP reinforcement and epoxy adhesive. The fire endurance (with respect to the 1 h fire duration of this test program) was defined as the amount of time that (1) the structural members must sustain the applied load without structural failure (according to the EN 1363-1), (2) the average temperatures of the unexposed concrete should not increase the initial average temperature by more than 140°C or the temperature at any location of the concrete part should not increase above the initial average temperature by more than 180°C (in case the specimen should fulfil a separating function during fire, in accordance to EN 1363-1) and (3) the temperature in the reinforcing steel should not increase more than a critical temperature equal to 593 °C (this critical temperature, according to table 3.2a of Eurocode 2 and as reported in previous work of Kodur et al. 2010, can be assumed as approximately the temperature where the steel has lost approximately 50% of its yield strength from that at room temperature).

The experimental data demonstrates that all the beams obtained the fire endurance ratings of one hour by satisfying both thermal and load bearing criteria described above. A summary of the maximum temperature recorded at the unexposed concrete surface, at the bottom steel reinforcement and at the adhesive interface (epoxy or mortar) after 1 h of fire exposure is reported in **Table 2**. The time when the epoxy reached the T_g , for the beams strengthened with FRP bars embedded with the epoxy resin, is also reported in **Table 2**. For all the beams the recorded temperatures of the concrete at the unexposed side and the longitudinal bottom reinforcement remained after 1 hour of fire well below the before mentioned critical temperatures of 140°C and 593 °C. This can be expected (and as verified in previous reference

fire tests), given the 30 mm concrete cover and the beam geometry. The insulation only lowered the temperatures even more below the critical temperature.

The thermal criterion for the FRP or adhesive temperature during fire exposure, is currently limited in design guidance documents (*fib* bulletin 14 and ACI 440.2R-2008) by considering T_g as limiting value. The obtained experimental results demonstrate that even if the recorded temperature of the epoxy resin, for all the tested beams, exceed the glass transition temperature in a time range between 33 min and 49 min, depending on the type of fire insulation and thickness, no impending failure of the protected beams was observed during fire test, and all the beams were able to withstand the service load of the strengthened beams for 1h of fire exposure. Therefore, to have a better understanding of the thermal performance at the bond interface, **Figure 6** shows the increase of temperature recorded by the thermocouples (average values of the two thermocouples) inserted at the bar adhesive interface. The experimental data demonstrates that, for beams B1-F3-1 and B2-F3-1 insulated with Promatect L-500 (see **Figure 6.a**), despite the different thickness, both beams performed similarly (more investigations are needed to clarify this aspect). For both beams the temperature increased slightly with fire exposure time. The temperature in the adhesive reached its glass transition temperature ($T_g = 62^\circ\text{C}$ and $T_g = 65^\circ\text{C}$ for beam B1-F3-1 and 2-F3-1 respectively) at around 39 min of exposure and the rate of increase of temperature remained steady during the fire exposure time without showing any abrupt increase. After 1 h of fire exposure the maximum recorded temperature into the adhesive was equal to 116°C for beam B1-F3-1 and 111°C for beam B2-F3-1 corresponding to $1.87 T_g$ and $1.70 T_g$ respectively. A different behaviour was observed for the beams insulated with HPC and Omega fire, in which the thermal performance of the insulation depends heavily on its overall thickness (see **Figure 6.b**). For instance for beams B1-F3-2, B1-F3-3 and B1-F3-4 for which the FRP bars were embedded with the same adhesive and were insulated with the same material, increasing the thickness from 20 mm of HPC and 15 mm of Omega fire to 35 mm of HPC and 20 mm of Omega fire resulted in an increasing of reaching the glass transition temperature from about 33 min to 49 min. Temperature increase for beam B1-F3-2 and B1-F3-4 was equal up to approximately 30 min into the fire. At that moment the increase of temperature into the adhesive of beam B1-F3-4, despite the lower thickness of insulation, became lower than that of beam B1-F3-2. This can be related to a different deterioration or cracks propagation of the inner layer of HPC along the beams. After 1 hour of fire exposure the maximum recorded temperature into the adhesive for beams B1-F3-2, B1-F3-3 and B1-F3-4 was 130°C ($2.1 T_g$), 102°C ($1.63 T_g$) and 101°C ($1.63 T_g$)

respectively. Beam B4-F3-1 with mortar as adhesive and with insulation thickness of 10 mm of HPC and 10 mm of Omega fire, presented the higher recorded increase of temperature into the bonding agent. After 1 h of fire exposure the temperature into the expansive mortar was about 163 °C. To know if the FRP strengthening is still active in some degree during or after 1h of fire exposure, deflection curves during fire were observed (next section) and residual strength testing was performed as discussed in section Beam's residual strength.

Structural performance at fire exposure

The deflections of each beam were measured in the pre-load phase and during fire testing. **Figure 7** shows the time-increase of deflection curves of all the beams under fire exposure. This increase of deflection is the additional deflection during fire, with respect to the initial deflection at 20°C and under the applied service load. From experimental outcomes it is clear that all the insulated beams were able to support the service load of the strengthened beam throughout the 1 hour fire tests without any signs of impending failure. No significant changes, in terms of sudden increase of deflection or rate of deflection, in the slope of the time deflection curves were observed for all the tested beams. At the end of the 60 min, the fire was halted and all the beams were unloaded except for beam B1-F3-2, for which the applied load was increased up to failure (see **Figure 8**). At that moment the temperature of the epoxy resin was about $T_{adhesive}=131^{\circ}\text{C}$ equal to $2.1 T_g$ and was constant during the increase of the load. The midspan deflection of beam B1-F3-2 increased slightly with increasing fire exposure time. When the load was increased, at 60 min of fire exposure, the deflection increased accordingly up to the point at which debonding of the bars occurred (at that moment the load was about $Q_u=75.2$ kN); after debonding the load dropped suddenly to approximately that of the corresponding unstrengthened beam ($Q_u=55.0$ kN) and the deflection increased until the concrete crushed. The failure of the beam was preceded by extended flexural vertical cracking of the beam in the pure bending region that led to yielding of the steel and loss of bond of the NSM FRP reinforcement followed by concrete crushing. This test clearly demonstrates that the adhesive likely started to lose strength and stiffness when the glass transition temperature was reached, but it was still capable to transfer stresses for the load levels under consideration. Indeed, the recorded failure load $Q_u=75.2$ kN was equal to 131% of that of the unstrengthened beam ($Q_{u,unstr.}=57.3$ kN) and equal to 77% of that of the same strengthened beam tested at ambient condition up to failure ($Q_{u,strength.}=96.9$ kN; Palmieri et al. 2012). It has to be noted that, based on experimental bond shear tests performed by the authors (Palmieri 2012), the adhesive kept

approximately 10% of its bond strength at a temperature equal to 120°C with respect to ambient (20°C) conditions.

Beams' residual strength

Another potentially important aspect of fire performance of FRP strengthened concrete structures is their residual behaviour after fire exposure. The post-fire residual behaviour of RC beams depends on the internal temperatures attained in fire, the load experienced by the beams in fire, the cooling method (air cooled method for this test program) and the strength recovering time following the cooling period. The fire damaged beams were stored for approximately one month at laboratory ambient temperature and then tested up to failure to determine their residual strength. The test set-up was the same adopted for the fire test and to test the reference beams at ambient temperature ($T=20\text{ }^{\circ}\text{C}$) (Palmieri et al 2010). The fire damaged beams were all tested up to failure in 4 point bending, and were instrumented with LVDTs and dial gauges in order to measure electronically and manually the beams deflection at midspan, under the point loads and at both supports. **Table 3** summarizes the experimental results in terms of ultimate load capacity, $Q_{u,residual}$, increase of flexural strength with respect to that of the unstrengthened beams at room temperature, $Q_{u,residual}/Q_{u,unstr.}$, percentage of residual strength with respect of the strengthened beam tested at room temperature, $Q_{u,residual}/Q_{u,str.}$, failure mode and temperature of adhesive after one hour of fire exposure as a function of the adhesive glass transition temperature ($T_g=62^{\circ}\text{C}$ for beams B1-F3-1, B1-F3-2, B1-F3-3 and B1-F3-4 and $T_g=65^{\circ}\text{C}$ for beam B2-F3-1). Based on the internal temperatures of the concrete compression zone and tension steel, well below the critical temperature of 140 °C and 570 °C respectively (see **Table 2**) the beams were expected to recover at least all of their unstrengthened flexural strength after the recovery time. This was indeed observed for all the tested beams. Moreover, for all the beams in which the FRP bars were embedded with an epoxy adhesive, the insulation systems were able to keep the adhesive temperature at relatively low temperature (in a range between 101 °C and 131 °C) so that they retained essentially almost their complete original strength at room temperature. For instance, even after 1h of fire exposure, all the tested beams were able to increase their flexural strength up to 56% in comparison to that of the unstrengthened beam B0. This residual strength was in a range between 86% and 92% in comparison to the FRP strengthened beams tested at ambient temperatures. Also beam B4-F3-1, for which the FRP bars were embedded with an expansive mortar, retained a great portion of its original strength. The primary beneficial effect of using expansive mortar as bond adhesive, instead of using epoxy resin, is that the mortar does not experience significant loss of mechanical and

bond properties in the range of the epoxy glass transition temperature (usually in the range between 50-90 °C for ambient cured epoxies) [Burke et al. 2008]. Indeed experimental results demonstrated that despite the lower insulation thickness of beam B4-F3-1 with respect of that of all the other beams and consequently the relatively higher adhesive temperature value ($T_{adhesive} = 167^{\circ}\text{C}$) recorded after 1h of fire exposure, beam B4-F3-1 was able to increase the flexural strength up to 17% in comparison to that of the unstrengthened beam B0 showing a residual strength equal to 91% of the FRP strengthened beam tested at room temperature. Although more tests are needed to confirm the behaviour under fire exposure of the mortar as bond adhesive, it seems to be a good alternative to the epoxy resin for strengthening concrete structures in which moderate flexural strength increase is needed, and this is in agreement with the findings of previous research (Burke 2008).

Finally recorded load-midspan deflection curves are shown in **Figure 9.a-d**. Because the beams were pre-cracked from the service load applied during the fire test, none of the curves for the residual strength testing demonstrate a cracking load. A part from this the load-midspan deflection curves were in close agreement with that of the FRP strengthened beams tested at room temperature until they start failing under the applied loads. In the final stage the midspan deflection indicates a certain reduction in the bond adhesive (epoxy or mortar) strength and stiffness.

Conclusions

Based on the results of fire test discussed herein, the following conclusions can be made. Despite the higher service load applied to the fire tested beams, the experimental results have clearly demonstrated the feasibility of providing 1h fire endurance rating under service load of the strengthened beam, without obvious dysfunction of the FRP in terms of strength compatibilities between the FRP and the RC beam during and after fire, if adequate protection against fire is provided. For none of the strengthened beams FRP NSM detached visibly. The insulation systems evaluated herein appear to have effectively protected the NSM FRP strengthened beams from heat penetration during 1 hour of fire exposure. The adhesive's temperature was maintained to low temperatures ($T_{adhesive} = 130^{\circ}\text{C}$ and $T_{adhesive} = 167^{\circ}\text{C}$ for epoxy resin and expansive mortar) and no impending failure was observed during the 1h fire exposure. The fire resistance effectiveness of the FRP strengthening system after fire exposure was evaluated in two different ways. For one of the tested beam (B1-F3-2) the load was increased immediately after the 1 h of fire exposure, keeping the temperature constant. This beam B1-F3-2 achieved a residual strength capacity equal to 77% with respect to that tested at room temperature, yet 127% of the unstrengthened

beam. All the others fire damaged beams were tested up to failure after been air cooled and stored in the laboratory for approximately one month. As the temperature of the concrete (at compression side) and the longitudinal bottom steel reinforcement were below critical temperatures, even in case of accidental drop out of FRP, the beams will not collapse for the acting service load. Moreover the residual strength tests demonstrated that, if the insulation is able to maintain the adhesive temperature at relatively low temperature ($T_{adhesive}=100\text{ }^{\circ}\text{C}$ to $130\text{ }^{\circ}\text{C}$ for epoxy resin and $T_{adhesive}=167\text{ }^{\circ}\text{C}$ for expansive mortar) the FRP is able to retain bond strength to the concrete (in agreement with work on double bond shear test by Foster and Bisby 2008 and Palmieri 2012) and the beam is still able to retain a considerable part (in this test program up to 92%) of the flexural capacity of the FRP strengthened beam at room condition.

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References

- American Concrete Institute (ACI). (2008). "Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures." *ACI 440.2R-08*, ACI Committee 440, Farmington Hills, MI, USA.
- Bisby, L. A., Green, M. F., and Kodur, V. K. R. (2005). "Fire Endurance of Fiber-Reinforced Polymer-Confined Concrete Columns." *ACI Structural Journal*, 102 (6), 883-891.
- Bisby, L.A., Burke, P.J., and Green, M.F. (2008). "Comparative performance of Externally-Bonded and Near Surface Mounted FRP Strengthening System at High Temperatures." *Proc., 5th Int. conf. on Advanced Composite Materials in Bridge and Structures (ACMBS)*, Winnipeg, Canada.
- Blontrock, H., Taerwe, L., Matthys, S. (1999). "Properties of Fibre Reinforced Plastics at Elevated Temperatures with Regard to Fire Resistance of Reinforced Concrete Members". *ACI Structural Journal*, 188(5), 43-54.
- Blontrock, H., Taerwe, L., Vandeveld, P. (2001). "Fire Testing of Concrete Slabs Strengthened with Fibre Composite Laminates." *Proc., of FRPRCS-5*, London, 547-556.
- Burke, P.J., Bisby, L.A. and Green, M.F. (2008). "Structural Performance of NSM FRP Strengthened Concrete Slabs at Elevated Temperatures." *Proc., Structural Faults and Repair*, Edinburgh, UK.
- Chowdhury, E.U., Bisby, L.A., Green, M.F. and Kodur, V.K.R., (2008) "Residual Behaviour of Fire-Exposed Reinforced Concrete Beams Prestrengthened in Flexure with Fiber-Reinforced Polymer Sheets." *J. Compos. Constr.*, ASCE, 1(12), 61-68.
- De Lorenzis, L., Lundgren, K., Rizzo, A., (2004). "Anchorage length of near-surface mounted FRP bars for concrete strengthening – Experimental investigation and numerical modeling". *ACI Structural Journal*, 101(2), 269-278.
- EN-1363-1 (1999). "Fire Resistance Tests – General Requirements – Part I" *European Committee for Standardization*.
- EN 1992-1-1 (2004). "Eurocode 2- Design of concrete structures – Part 1-1: General rules and rules for buildings". *European Committee for Standardization*.

- fib Task Group 9.3 (2001). "Externally bonded FRP reinforcement for RC structures." *International Federation for Structural Concrete, fib Bulletin* 14.
- Foster, S.K., and Bisby, L.A. (2008) "Fire Survivability of Externally-Bonded FRP strengthening Systems" *J. Compos. Constr.*, ASCE, 12(5), 553-561
- ISO 834 (1999). "Fire Resistance Tests – Elements of Buildings Construction – Part I – General Requirements". *International Organization for Standardization*.
- ISO 11357-2 (1999). "Plastics- Differential Scanning Calorimetry (DSC)- Part 2: Determination of Glass Transition Temperature".
- Katz, A., Berman, N., and Bank, L. (1999). "Effect of High Temperature on Bond Strength of FRP Rebars." *J. Compos. Constr.*, ASCE, 3(2), 73-81.
- Kodur, V., Bisby, L., and Green, M.F. (2007). "Guidance for the design of FRP-strengthened concrete members exposed to fire." *Journal of Fire Protection Engineering*, 17 (5), 5-16.
- Kodur V, Ahmed, A. (2010). "Numerical Model for tracing the Response of FRP-Strengthened RC Beams Exposed to Fire" *J. Compos. Constr.*, ASCE, 14(6), 730-742.
- Kotynia R. (2005). "Strain Efficiency of Near-Surface Mounted CFRP-strengthened Reinforced Concrete Beams". *Proc. of International Conference on Composites in Construction*, 35-42.
- Nigro, E., Cefarelli, G., Bilotta, A., Manfredi, G. and Cosenza, E. (2011a). "Fire resistance of concrete slabs reinforced with FRP bars. Part I: experimental investigations on the mechanical behavior". *Composites: Part B*, 42 (2011), 1739-1750.
- Nigro, E., Cefarelli, G., Bilotta, A., Manfredi, G. and Cosenza, E. (2011b). "Fire resistance of concrete slabs reinforced with FRP bars. Part II: experimental results and numerical simulations on the thermal field". *Composites: Part B*, 42 (2011), 1751-1763.
- Nigro, E., Bilotta, A., Cefarelli, G., Manfredi, G. and Cosenza, E. (2012). "Performance under fire situations of concrete members reinforced with FRP rods: bond models and design nomograms". *Journal of Composites for Construction*. Vol. 16, No. 4, August 1, 2012.
- Palmieri, A., Matthys, S., and Taerwe, L. (2010). "Strengthening with Near Surface Mounted Reinforcement: Structural and Fire behavior." *Proc. of 3rd fib International Congress (fib-2010)* Washington DC.
- Palmieri, A., Matthys, S., and Taerwe, L. (2012a). "Experimental investigation on fire endurance of insulated concrete beams strengthened with near surface mounted FRP bar reinforcement", *Composites Part B: engineering*, 43 (3), 885-895.
- Palmieri, A. (2012). "Evaluation and optimization of fire safety of structures strengthened with NSM reinforcement", *Ph.D. thesis*, Ghent University, Ghent, Belgium.
- Raafat, E.H., and Rizkalla, S.H., (2003), "Investigation of Bond in Concrete Structures Strengthened with Near Surface Mounted Carbon Fiber Reinforced Polymer Strips"; *J. Compos. Constr.*, ASCE, 7(3):248-257
- Sena Cruz, J.M., and Barros, J. (2004). "Bond between Near Surface Mounted Carbon-Fiber-Reinforced Polymer Laminate Strips and Concrete", *J. Compos. Constr.*, ASCE, 8(6), 519-527.
- Seracino, R., Saifulnaz, M.R.R. and Oehlers, D.J. (2007). "Generic Debonding Resistance of EB and NSM Plate-to-Concrete Joints", *J. Compos. Constr.*, ASCE, 11(1): 62-70.
- Stratford, T.J. and Bisby, L. (2012). "Effect of Warm Temperatures on Externally Bonded FRP Strengthening." *J. Compos. Constr.*, 16(3), 235-244.
- Stratford, T.J., Gillie, M., Chen, J.F., Usmani, A.S., (2010). "Bonded Fibre Reinforced Polymer Strengthening in a Real Fire", *Advances in Structural Engineering*, 12(6), 867-878
- Teng, J.G., De Lorenzis, L., Wang, Bo, Rong, Li, Wong, T.N., Lik Lam (2006). "Debonding failures of RC beams strengthened with near surface mounted CFRP strips" *J Compos Constr.*, ASCE, 10(2):14.
- Wang Y.C., Wong P.M.H., Kodur V. (2007). "An experimental study of the mechanical properties of fibre reinforced polymer (FRP) and steel reinforcing bars at elevated temperatures", *Composite Structures*, 80(1), 131-140

Weber A. (2008). "Fire-resistance tests on composite rebars". In: Proceedings of CICE2008, Zurich, Switzerland, 2008 Motavalli eds.

Williams, B, Kodur, V., Green. M., Bisby, L. (2008). "Fire endurance of Fiber-Reinforced Polymer Strengthened Concrete T-Beams", *ACI Structural Journal*, 105(1):60-67.

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Table 1- Test matrix

Beam	FRP	Adhesive	T _g [°C]	Insulation	Thick _{bottom} [mm]	Thick _{side} [mm]	Density [kg/m ³]	Conductivity [W/mK]
B1-F3-1	GFRP	Epoxy Sikadur 30	62	Promat L-500	50+50	20	500	0.09
B1-F3-2	GFRP	Epoxy Sikadur 30	62	HPC+Omega	25+20	10+10	527	0.07
B1-F3-3	GFRP	Epoxy Sikadur 30	62	HPC+Omega	35+20	10+10	527	0.07
B1-F3-4	GFRP	Epoxy Sikadur 30	65	HPC+Omega	20+15	10+10	527	0.07
B2-F3-1	CFRP	Epoxy Fortresin	65	Promat L-500	30+30	20	500	0.09
B4-F3-1	CFRP	Mortar SikagROUT 212	65	HPC+Omega	10+10	10+10	527	0.07

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Table 2- Maximum recorded temperatures after 1 hours of fire exposure

Beam	Insulation	T _{unexposed} [°C]	T _{steel} [°C]	T _{adhesive} [°C]	T _{adh} =T _g [min]
B1-F3-1	Promat L-500	35	126	116	39
B1-F3-2	HPC+Omega	34	127	130	33
B1-F3-3	HPC+Omega	34	135	102	49
B1-F3-4	HPC+Omega	35	122	101	37
B2-F3-1	Promat L-500	39	135	111	38
B4-F3-1	HPC+Omega	45	160	163	-

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Table 3- Experimental results residual strength test

Beam	Test	$Q_{u,residual}$ [kN]	$Q_u/Q_{u,unstr.}$ [-]	$Q_u/Q_{u,str.}$ [-]	Failure Mode	$T_{adhesive}$ [-]
B0-20°C	Room Temperature	57.3	1.00	-	Concrete crush	-
B1-20°C		96.9	1.69	-	FRP debonding	-
B2-20°C		101.5	1.77	-	FRP debonding	-
B4-20°C		73.3	1.27	-	FRP debonding	-
B1-F3-1	Residual strength test	85.0	1.48	0.87	FRP debonding	1.87T _g
B1-F3-2		-	-	-	FRP debonding	2.11T _g
B1-F3-3		89.7	1.56	0.92	FRP debonding	1.63T _g
B1-F3-4		88.7	1.54	0.91	FRP debonding	1.63T _g
B2-F3-1		87.7	1.53	0.86	FRP debonding	1.78T _g
B4-F3-1		67.3	1.17	0.91	Concrete crush	-

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