SMAR 2019 – Fifth Conference on Smart Monitoring,
Assessment and Rehabilitation of Civil Structures

SMAR 2019

Influence of FRP Repair on the Axial Behavior of Fire Damaged Concrete

Ugur DEMIR [1], Goktug UNAL [1], Ahmet Faik SERT [1], Ramazan Okan CALIS [1], Alper ILKI [1]
[1] Istanbul Technical University, Istanbul, Turkey

Contact e-mail: udemir@itu.edu.tr

ABSTRACT: The exposure of concrete to elevated temperatures detrimentally affects its mechanical properties and thus, efficient repair materials and schemes would be needed to regain these properties. This study reports the results of an experimental work on the influence of fiber reinforced polymer (FRP) repair on the axial stress–strain behavior of concrete cylinders exposed to fire. Within the scope of the study, twelve plain concrete cylinders with dimensions of 150×300 mm were cast. While three of the cylinders were kept in ambient conditions, nine of them were exposed to ISO-834 standard fire for 60 minutes. After natural cooling, i) three of the fire exposed specimens were kept without repair, ii) three of them were repaired by jacketing with two layers of carbon FRP sheets and iii) other three of specimens were jacketed with four layers of FRP sheets. Then, all of the specimens were tested under uniaxial compression. The test results indicated that exposure to elevated temperatures leads to a reduction in compressive strength and modulus of elasticity but an increase in the axial strain corresponding to peak stress for the heated plain specimens. When confined with FRP jackets, the compressive strength and deformation capacity of these fire damaged specimens enhanced remarkably. On the other hand, the repair technique was found to be ineffective on reinstating the axial stiffness of the specimens which was reduced after fire exposure. Furthermore, the prediction performance of a unique model available in the literature that has been proposed for predicting the axial stress- strain behavior of fire damaged FRP confined circular columns was investigated. It was seen that the model did not exhibit a reasonable performance for the specimens tested in this study.

1 INTRODUCTION

Real fire experiments show that it is rare for a concrete building to collapse as a consequence of fire and most fire-damaged concrete structures can be successfully rehabilitated (Yaqub and Bailey, 2011). Depending on the predefined damage class, repairing after fire is recommended to be performed with one or a combination of different techniques (e.g. aesthetical renovation, using shotcrete, addition of extra reinforcement using glued carbon or glass-fiber laminates (FRP), addition of extra fire-safety equipment, by injecting resins or cement slurries) (fib-46, 2008; Concrete Society, 2008). Other conventional techniques for repairing fire-damaged columns are strapping by steel plates, jacketing using conventional reinforced concrete, adding columns at new locations of the facility or combinations of these techniques (Lin et al. 1995). Most of these techniques are generally criticized as being time demanding, destructive, ineffective, sometimes expensive and incurring additional weight (Al-Nimry et al. 2013). In order to overcome the issues associated with the conventional strengthening techniques, use of FRP is one of the latest and most promising approaches (Smyrou et al. 2015). A particularly attractive use of FRPs in structural engineering applications involves strengthening RC (reinforced concrete) columns by circumferential FRP wraps (e.g. Ilki et al. 2004, Kodur et al. 2006, Ilki et al. 2008). With the increasing trend in use of FRP wraps in structural repair applications, concern has developed regarding their performance after fire (Bisby et al. 2011, Yaqub et al. 2011, Yaqub and Bailey 2011, Al-Nimry et al. 2013). However, among these studies, only the study conducted by Bisby et al. (2011) provides a quantified design guidance for practical applications of FRP confinement in fire-damaged structures. In their paper, they reported an experimental and analytical work consisted of uniaxial compressive tests on 33 plain concrete cylinders of 100 mm in diameter and 200 mm in height. The aim of the study was to investigate the compressive strength and stress–strain behavior of both unconfined and FRP confined plain concrete cylinders which were repaired after being heated to various elevated temperatures and cooled down to room temperature. For the heated specimens, for a given exposure temperature (i.e. 300, 500, or 686 °C), the furnace was programmed to heat up to the desired temperature and then to hold that temperature for the required duration of heating (120 or 240 min). According to the test results of 33 cylinder specimens, the authors have proposed a unique model to predict the axial stress–strain characteristics of fire-exposed FRP-confined concrete under slow heating rates (5-15 °C/min). The duration between the fire and compression tests were 6 to 17 days corresponding to the first stage of cooling during which compressive strength reduction due to the elevated temperatures has not been completed yet according to Harada et al. (1972). However, it is worth to note that this study is intentionally representing a worst-case scenario in which the real behavior is expected to be better. On the other hand, only the presented study considers a standard ISO-834 fire which is a more representative of a real fire among the all above-mentioned works. Furthermore, when compared to Bisby et al. (2011), the presented study also takes a longer cooling phase (6 months) into account allowing for a full recovery on residual properties of concrete which is of vital importance in terms of eliminating the residual behavior variations depending on duration after fire. When the literature is mined, it is clear that there is an obvious need for more research since very little information exists regarding the ability of FRP strengthening systems to retain structural effectiveness under sustained service loads after post-fire cooling. The primary objective of the research presented in this paper is to enhance the existing knowledge to demonstrate the effectiveness of externally confined FRP sheets for reinstating the load-bearing and ductility capacity of fire-damaged circular concrete compressive members. Thus, more realistic design guidance can be provided for practical applications. This study reports preliminary results of a comprehensive work on the repair of fire exposed RC.

2 EXPERIMENTAL WORK

Twelve concrete cylinder specimens with circular cross-sections were fabricated. The mix proportions of concrete are given in Table 1. All specimens had 150 mm diameter and 300 mm height. While three of the cylinders were not subjected to elevated temperatures, nine of them were exposed to ISO-834 standard fire for 60 minutes. The fire test was carried out at a large-scale multipurpose furnace with 3.2 m depth, 4 m width and 3 m height. Eight natural gas burners located within the furnace provide thermal energy, while eight thermocouples in the test chamber monitor the furnace temperature during the fire test. Two Type-K NiCr-Ni thermocouples of 0.91 mm thick were installed at the mid-height center and the surface of the specimens for measuring the concrete temperatures during fire exposure. The fire tests were conducted 14 months after casting. Figure 1 shows time-temperature curve in the furnace during the fire test and after cooling. In this figure, average thermocouple measurements obtained at the center of the cross-section and at the surface of the specimens as well as ISO-834 standard fire curve and the average temperatures measured in the furnace (denoted as 60 minutes) are shown. After natural cooling, i) three of the fire exposed specimens were kept without repair, ii) three of them were repaired by jacketing with two layers of carbon FRP sheets and iii) other three specimens were jacketed with four layers of FRP sheets. Then, all of the specimens were tested under uniaxial compression.

Table 1. Mix proportions of concrete

Material	Amount (kg/m^3)
Cement (CEM 42.5 R)	300
Fine aggregate (5-12 mm)	920
Coarse aggregate (12-22 mm)	997
Superplasticizer (Sikament-300)	3.6
Water	124

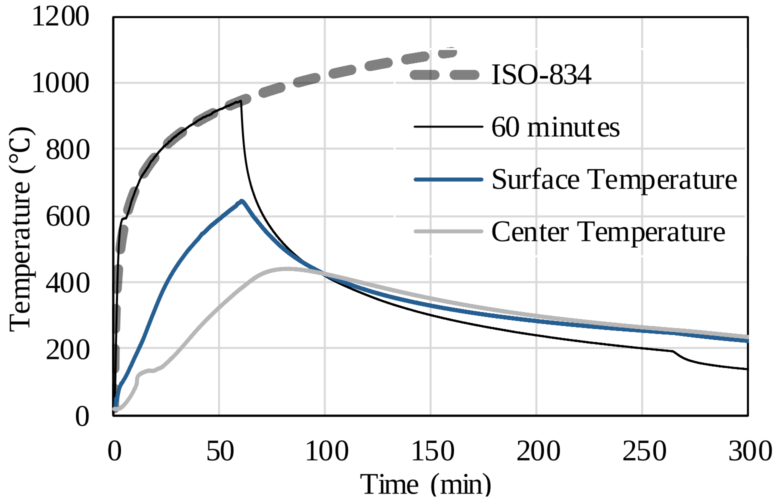

Fig. 1. Time – temperature curves during the fire test

Details of the test specimens are summarized in Table 2. In this table, for instance, 60m-r2 demonstrates the specimen subjected to ISO-834 fire for 60 minutes (60m) and jacketed with two layers of FRP (r2) while ref indicates the unheated and unconfined reference specimens. All compression tests were performed in triplicate.

Table 2. Details of test specimens

Designation	Number of specimens	Number of FRP layers	FRP volumetric ratio (ρ_f)
ref	3	N.A.	-
60m	3	N.A	-
60m-r2	3	2	0.0044
60m-r4	3	4	0.0088

The volumetric ratios of FRP (ρ_f) for the specimens given in Table 2 are calculated using Eq. (1) as recommended by Ilki et al. (2004). In Eq. 1, t_f is the total effective thickness of FRP and D is the diameter of the specimens.

$$\rho_f = \frac{4t_f}{D} \tag{1}$$

Mechanical properties of unidirectional carbon FRP sheets provided by the manufacturer (Akfibre DKM 42 12 K) are given in Table 3. The epoxy used for bonding FRP sheets consisted of a resin binder and hardener, which are mixed, in a ratio of 2/1 by weight. Jacketing of specimens was executed in five steps as 1) cleaning surface of each specimen, 2) partial repair of the specimen surface with epoxy mortar only to fill the voids over the surface, 3) applying epoxy, 4) FRP

application over epoxy, and 5) forming an overlap of 150 mm at the end of FRP jacket. The FRP was applied using a hand
lay-up procedure. After jacketing process is completed, the specimens were left in laboratory for 6 months until the test day to represent a full recovery after cooling.

Table 3. Mechanical properties of FRP sheets given by manufacturer

Tensile strength (MPa)	Longitudinal elasticity modulus (MPa)	Ultimate tensile strain (%)	Effective thickness (mm)
4200	240000	1.8	0.166

Sequentially, compression tests were carried out under monotonic uniaxial compressive loading using an Instron testing machine with a capacity of 5000 kN. The tests were displacement controlled with a loading rate of 0.5 mm per minute. For measuring the axial deformations, two linear variable differential transducers (LVDTs) were located at mid height of the specimens through a compressometer with a gage length of 150 mm. Additionally, four LVDTs were placed between the loading and supporting steel plates along the height of the specimens (Fig. 2). All LVDTs had a stroke length of 25 mm and sensitivity of 500×10^{-6} strain/mm. To measure the hoop rupture strains, strain gages were placed along the transverse direction. The gage length of each strain gage was 60 mm and these strain gages were placed at the mid-height of the specimens. A TML-TDS-303 data logger was used for data acquisition. Axial loads applied during the tests were taken directly from the built-in load cell of the Instron testing machine.

Fig. 2. Test setup for specimens illustrating LVDT and strain gauge application

The axial behavior of unconfined and externally FRP jacketed specimens can be presented with stress-strain relationships. These relationships and quantitative test results are given in Fig. 3 and Table 4, respectively. Figure 3 highlights the effect of fire on the axial strength, stiffness, and full stress–strain response of specimens. In this plot, the negative part of the horizontal axis is used to show strains in hoop direction whereas the positive parts are used for axial strains (ε_c). In Table 4, f'_{co} and f'_{cc} are the unconfined and confined axial strengths, while ε_{co} and ε_{cc} are the axial strains corresponding to axial strength for the unconfined and confined specimens, respectively. The axial strains were calculated by dividing the average displacement readings of the two LVDTs at mid-height to the gage length of 150 mm, whereas the hoop rupture strains were obtained from the average strain gage readings measured along the transverse direction (Fig. 2).

In Fig. 3, it can be clearly noticed that, exposure to elevated temperatures leads not only to a reduction in compressive strength, but also a significant reduction in the modulus of elasticity (or axial stiffness). Axial stiffness, calculated through tangent modulus (slope of stress-strain curve between 5-40% of the peak strength) of the fire exposed specimens was reduced to about 20% of the unheated value. In addition, an increase in the axial and lateral strains corresponding to peak stress is observed. Concrete exposed to 60 minutes of ISO-834 standard fire experienced an average reduction of 58% in compressive strength, whereas fire exposure resulted with an increase in axial and lateral strains at peak stress in the order of 165% and 400%, respectively. This can be attributed to the reduction in stiffness and excessive increase in dilation of the concrete due to fire exposure, respectively. On the other hand, both the axial stress and strains at failure were higher for the confined specimens than those for the unconfined ones.

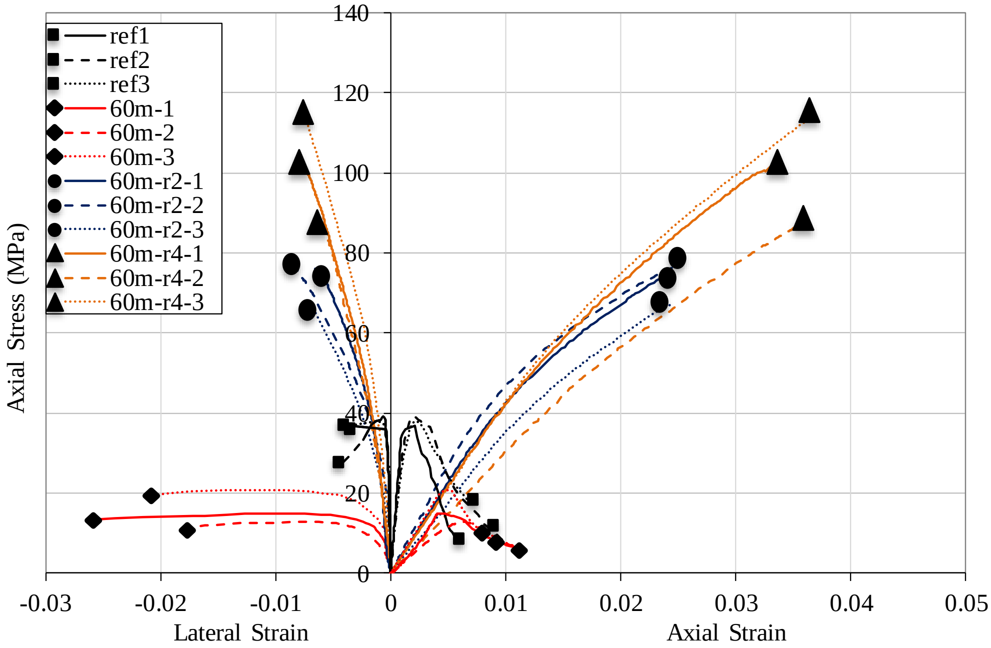

Fig. 3. Axial stress-axial strain and axial stress-lateral strain curves

Table 4. Test results and comparison with the model predictions

Specimen	$f'_{co,exp}$ or $f'_{cc,exp}$ (MPa)	$f'_{cc,analy}$ (MPa)	$\varepsilon_{co,exp}$ or $\varepsilon_{cc,exp}$	$\varepsilon_{cc,analy}$	$\varepsilon_{h,rup}$	$f'_{cc}/f'_{co}{}^{a}$	$\varepsilon_{cc}/\varepsilon_{cc}{}^{a}$	Average $f'_{cc}/f'_{co}{}^{a}$	Average $\varepsilon_{cc}/\varepsilon_{co}{}^{a}$
ref1	36.9	-	0.0021	0.0020	-	-	-		
ref2	39.3	-	0.0020	0.0020	-	-	-	-	-
ref3	38.0	-	0.0022	0.0020	-	-	-		
60m-1	12.7	16.2	0.0061	0.0079	-	-	-		
60m-2	15.0	16.2	0.0055	0.0079	-	-	-	-	-
60m-3	20.8	16.2	0.0050	0.0079	-	-	-		
60m-r2-1	74.8	50.9	0.0244	0.0184	0.007	4.62	4.41		
60m-r2-2	77.0	50.9	0.0249	0.0184	0.007	4.76	4.50	4.50 3.14[b]	4.42 2.33[b]
60m-r2-3	66.8	50.9	0.0240	0.0184	0.007	4.13	4.34		
60m-r4-1	102.6	85.6	0.0336	0.0286	0.007	6.35	6.08		
60m-r4-2	86.6	85.6	0.0355	0.0286	0.007	5.35	6.42	6.27 5.29[b]	6.30 3.62[b]
60m-r4-3	114.9	85.6	0.0370	0.0286	0.006	7.11	6.41		

[a] Values are with respect to heated specimens
[b] Values are predicted by Bisby et al. (2011) and discussed in the following section
exp: experimental values; analy: analytical values predicted by Bisby et al. (2011)

For the specimens confined with two and four layers of FRP, axial strengths were enhanced by an average value of 350% and 527%, respectively, while axial strains were improved by 342% and 530%, respectively compared to the fire-damaged specimens. As seen in Fig. 3 and Table 4, none of the specimens could regain the original axial stiffness as previously reported by Bisby et al. (2011) and Al-Nimry et al. (2013). The residual axial stiffness of the FRP confined specimens was similar to the unconfined ones (about 80% reduction with respect to unheated specimens). It is also clear from Fig.3 that heterogeneous characteristic of concrete is exacerbated by fire exposure resulting with a variability in axial stiffness of heated specimens. Consequently, it is apparent that external FRP jacketing significantly improves the strength and deformation capacity of the specimens, but it is ineffective in reinstating the original axial stiffness. All FRP-jacketed specimens failed with an explosive rupture of the FRP jackets. The confined specimens, which displayed almost identical failure modes, reached similar FRP hoop rupture strains independent of FRP thickness (Table 4).

2 ANALYTICAL WORK

Mechanical properties of concrete after fire exposure are temperature dependent and sensitive to parameters such as heating rate, temperature gradient, and so on. However, there is only one model in the published literature, which has been proposed specifically to predict the axial behavior of concrete jacketed with FRPs after being subjected to elevated temperatures and cooled down to room temperature. Therefore, the prediction performance of the existing model developed by Bisby et al. (2011) is investigated herein. For predicting the axial stress and strain response of fire-damaged FRP confined concrete, Bisby et al. (2011) recommended Eqs. 2 and 3, respectively. In these equations, $f'_{cc\Theta}$ is the confined compressive strength of concrete after cooling, $f'_{c\Theta}$ is the unconfined compressive strength of concrete after cooling, f_{lu} is the effective confinement pressure, $\varepsilon'_{ccu\Theta}$ is the confined ultimate strain of concrete after cooling, $\varepsilon'_{c\Theta}$ is unconfined strain at peak strength in concrete after cooling, ε'_{ccu} is the confined ultimate strain of unheated concrete and ε'_{c} is the unconfined strain at peak strength in unheated concrete. The effective confinement pressure is calculated using Eq. 4 in which t_f is the total thickness of the FRP wrap, E_f is the modulus of elasticity of the FRP wrap in the hoop direction, ε_f is the effective strain in the FRP wrap at the ultimate state, and D is the column diameter. According to the authors, in these equations, compressive strength of heated concrete can be estimated through a reduction factor available in variety of sources in literature and in this study experimentally obtained values are considered as similarly done by Bisby et al (2011). Strain at peak strength of unheated specimens are estimated using the approach described by Lee et al. (2008). The same approach was followed while assessing the prediction performance of the model.

$$f'_{cc\theta} = f'_{c\theta} + 3.3 f_{lu} \tag{2}$$

$$\varepsilon'_{ccu\theta} = \varepsilon'_{c\theta} + (\varepsilon'_{ccu} - \varepsilon'_{c}) \tag{3}$$

$$f_{lu} = \frac{2t_f E_f \varepsilon_f}{D} \tag{4}$$

Predicted axial strength and strain values by the model is given in Table 4. As seen in this table, the investigated model tends to underestimate both the axial strength and ultimate axial strain of fire-damaged FRP confined specimens. This can be attributed to the formulation for both strength and strain enhancements indicating an obvious need for a new model. Furthermore, the formulation of Lee et al. (2008) in order to obtain unconfined strain at peak strength ($\varepsilon'_{c\Theta}$, 0.0079) overestimated the experimental value (0.0055). This may be because of the heat exposure and cooling period variations between the researches as well as concrete properties. On the other hand, the average value of the hoop rupture strain is experimentally obtained as 0.007 for all confined specimens. Thus, strain efficiency factor, which is the ratio of the hoop strain in the FRP at the ultimate state ($\varepsilon_{h,rup}$) to the ultimate tensile strain of FRP given by manufacturer based on coupon tests (Table 3), was obtained as 0.40 for all FRP confined specimens. Bisby et al. (2011) recommended a strain efficiency value of 0.55 resulting with higher effective confinement ratios. In the light of analytical comparison of the model performances with the experimental data presented herein, it may be commented that, the investigated model performance is not reasonable in agreement with the test results reported in the current study.

In order to evaluate the confined strength and ultimate strain prediction performances of the model statistically, the average absolute error (*AAE*) and standard deviation (*SD*) are calculated using Eqs. (5 and 6). In these equations, mod_i and exp_i indicate the predicted value by the model and the corresponding value determined by the test, respectively; mod_{avg} and exp_{avg} are the average values predicted by the model and determined by the test, respectively; and n is the number of test specimens. Table 5 presents the calculated *AAE* and *SD* values for the model.

$$AAE = \frac{\sum_{i=1}^{n}\left|\frac{\mathrm{mod}_i - \exp_i}{\exp_i}\right|}{n} \tag{5}$$

$$SD = \sqrt{\frac{\sum_{i=1}^{n}\left(\frac{\mathrm{mod}_i}{\exp_i} - \frac{\mathrm{mod}_{avg}}{\exp_{avg}}\right)^2}{n-1}} \tag{6}$$

Table 5. Statistics on the prediction performance of the model

f'_{cc}/f'_{co}		$\varepsilon_{cc}/\varepsilon_{co}$	
AAE	*SD*	*AAE*	*SD*
0.23	0.62	0.45	0.90

Wu and Zhou (2010) have defined the accuracy of a model based on *AAE* values i.e. Category I ($AAE \leq 0.15$), Category II ($0.15 < AAE \leq 0.30$) and Category III ($AAE > 0.30$). According to this approach, the strength and ultimate strain predictions of the investigated model are ranked in Category II and Category III for strength and strain enhancements, respectively. This approach also confirms the need for a new model which is not assessed in the context of this paper.

3 CONCLUSIONS

On the basis of the presented preliminary experimental and analytical work on i) unheated and unconfined, ii) heated and unconfined and iii) heated and FRP confined specimens, the following conclusions can be drawn:

For the unconfined concrete specimens, exposure to elevated temperatures leads not only to a reduction in compressive strength, but also a reduction in stiffness and an increase in the axial strain corresponding to peak stress. Sixty minutes of ISO-834 standard fire resulted with a reduction of 58% in compressive strength and about 80% in axial stiffness with respect to the unheated ones. Fire exposure also resulted with an increase in axial and lateral strains at peak stress in the order of 165% and 400%, respectively for the heated specimens.

External FRP jacketing significantly improves the strength and deformation capacity of the specimens, but it is ineffective in reinstating the original axial stiffness. For the specimens confined with two and four layers of FRP, axial strengths were enhanced by an average value of 350% and 527%, respectively, while ultimate axial strains were improved by 342% and 530%, respectively as compared to the heated specimens. When compared to unheated specimens, a remarkable enhancement in both axial strength and deformability was also obtained.

The only available model in the literature which has been proposed for the axial behavior of fire-damaged and FRP repaired columns (according to the best knowledge of the authors), did not exhibit a satisfactory performance in terms of predicting the axial stress-strain response of FRP repaired fire-damaged concrete members.

Experimental and analytical results strongly indicate that further work is required for modelling the stress – strain behavior of sequentially fire-damaged, cooled down to room temperature and FRP confined concrete as well as considering different fire exposure durations and volumetric FRP ratios.

ACKNOWLEDGEMENTS

The authors acknowledge the material and workmanship support of the companies DowAksa, Ak-Kim and Oftek. Prof. Mark F. Green is also kindly acknowledged for giving ideas about the fire test.

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