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# Performance of reinforced concrete elements strengthened with carbon fiber CFRP at elevated temperatures

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The authors declare that there is no conflict of interest.

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**Abstract**. The importance of the research topic is established by the problems that occur in structural buildings when exposed to fire accidents, where the concrete loses much part of its mechanical properties and therefore becomes out of service. Because reconstruction of damaged buildings has a high financial cost, it is necessary to focus on the restoration of damaged concrete members with performant techniques and proven efficiency in terms of increasing the strength of concrete and its resistance to high temperatures. The authors conduct a numerical investigation on the use of carbon fiber-reinforced polymer sheeting CFRP to restore various structural concrete elements such as beams, columns, and slabs damaged in fire accidents for two types of normal and high-strength concrete, in addition to studying the behavior of concrete after strengthening it with CFRP sheets. The results by showed that load capacity, stiffness index, and absorption energy index have been improved by using CFRP in comparison with undamaged and fire-damaged elements.

**Keywords:** Beam, column, slab, carbon fiber, concrete

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# Характеристики железобетонных элементов, усиленных углепластиком CFRP, при повышенных температурах

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**Аннотация.** Актуальность темы исследования обусловлена проблемами, возникающими в несущих зданиях при пожарах, когда бетон теряет большую часть своих механических свойств и, следовательно, выходит из строя. Поскольку реконструкция поврежденных зданий требует высоких финансовых затрат, необходимо сосредоточиться на восстановлении поврежденных бетонных элементов с использованием надежных методов и доказанной эффективности с точки зрения восстановления прочности бетона и повышения устойчивости к высоким температурам. В исследовании численно исследуется использование углепластика CFRP, для восстановления различных структурных бетонных элементов, таких как балки, колонны и плиты, поврежденных в результате пожара, для двух типов нормального и высокопрочного бетона, а также изучается поведение бетона после укрепления его листами углепластика. Результаты показали, что несущая способность, индекс жесткости и индекс энергии поглощения были улучшены при использовании углепластика по сравнению с неповрежденными и поврежденными огнем элементами.

**Ключевые слова:** балка, колонна, плита, углепластик, бетон

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## 1. Introduction

Concrete is the most often used man-made material and the second most consumed substance in the world, next to water, due to its necessity for various construction applications and the long-term demand for them [1]. Concrete is a composite material comprised of various components, including aggregates, water, cement, and other cementitious elements as binder ingredients [2].

Concrete's preserved qualities after cooling from being subjected to high temperatures are typically referred to as residual properties. The period of exposure, the features, and the material composition of concrete can all have an impact on how these properties change significantly within the high temperature range associated with an exposed fire [3]. Compressive strength, tensile strength, elastic modulus, and stress-strain response are the principal mechanical parameters of concrete that are of interest after exposure to high temperatures. These characteristics are frequently used to evaluate how much strength concrete loses and degrades at high temperatures [4].

When heated to 300 °C, concrete loses around 25 % of its initial compressive strength, and when subjected to temperatures beyond 600 °C, it loses about 75 % [5; 6]. The tensile strength decreases in concrete with an increase in temperature [7]. The mechanical property of concrete that is commonly regarded as being most impacted by exposure to high temperatures is its modulus of elasticity. In comparison to compressive and tensile strength, the degradation of elastic modulus occurs much more quickly [8]. While considering the critical heating

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level for residual mechanical strength characteristics of fiber-reinforced concrete, Eidan et al. [9] showed that, at 400 ºC, the residual factors for fiber-reinforced concrete are often better than those of plain concrete.

At high temperatures under 1000 °C, the compressive strength significantly decreased, but steel fiber-reinforced concrete with a 1 % addition outperformed non-steel fiber-reinforced concrete [10]. Moghadam & Izadifard [11] compared the impact of steel fiber and glass fiber on the strength of concrete at high temperatures. While no chemical changes were observed at this temperature range, both for steel and glass fiber at tested temperatures up to 800 ºC, they noticed that the compressive strength of normal, steel, and glass fiber concrete decreased as the temperature rose to 100 ºC.

Fixing damaged concrete members frequently involves building an external reinforced concrete support or concrete jacket or epoxy-bonding metal plates for damaged component, among other methods [12]. Using a laminate made of fiber-reinforced composite materials, such as carbon and glass fiber-reinforced polymers, in place of the steel plates is a unique technology. In structural repairs and the restoration of reinforced concrete components, the use of high-performance fiber-reinforced cementitious composites has gained significance [13]. The CFRP has been extensively employed to strengthen various structural elements, such as beams, columns, and slabs, from the outside [14]. Due to the low weight, corrosion-resistance, and tensile strength of FRP materials, which were employed in the space industry in the 1970s, this application became quite popular. The RC columns are passively contained by the CFRP jacket, which is only stressed when a column is subjected to an additional axial force that produces dilatation. Several factors affect the degree of confinement and the overall improvement in strength of confined concrete columns' sizes, shapes, and fiber modulus, as well as their thickness and fiber rupture strain (circular, square, or rectangular) [15].

Several scholars, for example, Ashteyat et al. [16], Shehata et al. [17], and Mhanna et al. [18], have thoroughly studied carbon fiber-reinforced polymers. They investigated the effects of corner roundness, column height, cross-section form, layer count, and wrapping method (full or partial) on the strength and ductility of the RC column. They discovered that when the corner radius was rounded until it approached the circular section, and as the CFRP's thickness rose, the strength improved. When exposed to repeated loading, CFRP has the benefit of displaying a much higher tensile strength. They can also be easy to use on site without the need for specialized tools or labor and are highly resistant to corrosive effects due to the many properties of CFRP, including mechanical properties, spacing, dimensions, and configuration [16–18].

According to the previous analysis of the literature, it should be noted that while numerous studies have been conducted to examine the behavior of concrete members when exposed to fire, little is known about the behavior of these members when strengthened with CFRP. Hence, this study is devoted to the effectiveness of employing CFRP to repair damaged concrete elements.

## 2. Methodology

The obtained results from the numerical simulation for the beam, slab, and column under ambient temperatures, elevated temperatures, and, lastly, after strengthening the fire-damaged concrete element, will be included in this study. Each concrete element has two concrete strength types (normal 25 MPa and high 65 MPa), in addition to two thicknesses for CFRP. The simulations shall be divided into the referenced undamaged models, the fire-damaged models, and the strengthened models. ABAQUS offers a variety of material attributes that reflect how those materials behave under various simulations. The Poisson's ratio and the modulus of elasticity are two parameters that are frequently used to describe the elastic phase in isotropic materials. The concrete damaged plasticity model CDP is utilized in this study to characterize the concrete plastic phase because of its effectiveness in predicting the behavior of the concrete under a variety of conditions, including monolithic and repeated loadings, plain and reinforced concrete, and application, which depends on the material loading rate.

In this paper, the behavior of the various concrete elements under various thermal conditions and repair techniques will be presented and discussed in detail, showing the level of degradation and improvement in the elements' performance in terms of various flexural indices. Each concrete member category was designated according to the type of the element.

The concrete beam has a full-scale dimension of 5500 mm in length, 300 mm in width, and 500 mm in thickness, which is one of the three main concrete components that make up the majority of such constructions. The beam was strengthened with three steel bars, top and bottom, using 16 mm steel bars, stirrups along the length of the beam with consistent spacing using steel bars with a diameter of 10 mm each at 200 mm, and three steel bars at the top and bottom using 16 mm steel bars. While for the concrete column element, the dimensions were selected to ensure that the column is classified as a short column, and the failure will occur due to concrete crushing.

The concrete column cross section dimensions are 300 mm by 400 mm and the length is 3000 mm; the longitudinal steel reinforcement was six bars with a diameter of 16 mm; and the transverse reinforcement (ties) was steel bars with a diameter of 10 mm and 200 mm spacing.

The slab's concrete proportions were also chosen to guarantee that its behavior qualifies as a two-way action slab. The slab's thickness was 200 mm, its aspect ratio was 1, and its dimensions were 5500 mm by 5500 mm. Two steel bar meshes (top and bottom) with a 12 mm diameter and a 200 mm spacing made up the embedded steel reinforcement.

Each member's concrete strength needs to be examined, as each type of concrete member shows a varied level of fire temperature decrease. The thermal destruction carried on by subjecting the faces of the concrete elements to the ISO-834 standards differs since there are four, three, and one subjected face in column, beam, and slab elements, respectively. The effects of high temperatures also differ depending on the type of concrete. The temperature starts to rise from the bottom of the exposed face [19].

## 3. CFRP material's characterization and modeling

In ABAQUS, the damage initiation criteria for fiber-reinforced composites are based on Hashin's theory (see Hashin and Rotem, 1973). The Hashin damage model predicts anisotropic damage in elastic-brittle materials [21]. It is primarily intended for use with fiber-reinforced composite materials and takes into account four different failure modes: fiber tension, fiber compression, matrix tension, and matrix compression [20]. Below are the equations for these failure modes (ABAQUS manual).

Fiber tensile failure criteria:

$$\left(\frac{\sigma_1}{\sigma_{1u}^t}\right)^2 + \left(\frac{\tau_{12}}{\tau_{12u}}\right)^2 = 1(\sigma_1 > 0). \tag{1}$$

Fiber compressive failure criteria:

$$\frac{|\sigma_1|}{\sigma_{1u}^c} = 1(\sigma_1 < 0). \tag{2}$$

Matrix tensile failure criteria:

$$\left(\frac{\sigma_2}{\sigma_{2u}^t}\right)^2 + \left(\frac{\tau_{12}}{\tau_{12u}}\right)^2 = 1(\sigma_2 > 0). \tag{3}$$

Matrix compressive failure criteria:

$$\left(\frac{\sigma_2}{2\tau_{23u}}\right)^2 + \left[\left(\frac{\sigma_{2u}^c}{2\tau_{23u}}\right)^2 - 1\right]\frac{\sigma_2}{\sigma_{2u}^c} + \left(\frac{\tau_{12}}{\tau_{12u}}\right)^2 = 1, \tag{4}$$

where $\sigma_1$ is the stress in direction 1, $\sigma_1 ut$ is the ultimate tensile stress in direction 1 (maximum tensile longitudinal strength), $\sigma_1 uc$ is the ultimate compressive stress in direction 1 (maximum compressive longitudinal strength), $\sigma_2$ is the stress in direction 2, $\sigma_2 ut$ is the ultimate tensile stress in direction 2 (maximum tensile transversal strength), $\sigma_2 uc$ is the ultimate compressive stress in direction 2 (maximum compressive transversal strength), $\sigma_3$ is the stress in 3 direction 3, $\tau_{13}$ is the shear stress in plane 1–3, $\tau_{23}$ is the shear stress in plane 2–3, $\tau_{23} u$ is the inter laminar ultimate shear strength in plane 2–3 (maximum shear strength in plane 2–3), $\tau_{12}$ is the shear stress in plane 1–2, $\tau_{12} u$ is the ultimate shear stress in plane 1–2 (maximum shear strength in plane 1–2).

Such a model requires defining the elastic properties matrix, such as the elastic and shear modulus for both directions (E1, E2, G11, G13, and G23), in addition to the longitudinal and transverse Poisson ratio of the composite. The damage initiation can be defined by assigning the damage variables as listed in Table 1, they are the longitudinal and transverse values for tensile, compressive, and shear strength. In total, six parameters should be assigned to define the elastic properties in addition to six parameters to define the damage initiation response of the CFRP sheet.

*Table 1*

**Elastic and failure parameters of used CFRP laminate [22]**

| Parameter | Quantity |
|---|---|
| Elastic modulus of fabric, $E_1, E_2$, respectively | 230 and 16.58GaPa |
| Longitudinal and transverse Poisson's ratio | 0.30 |
| Shear modulus $G_{12}, G_{13}, G_{23}$, respectively | 9188.5, 12259 and 5911 MPa |
| Longitudinal tensile and compressive strength | 3900 and 3120 MPa |
| Transverse tensile and compressive strength | 210.6 and 64.5 MPa |
| Longitudinal and transverse shear strength | 210.6 and 276.9 MPa |

In this study, the obtained results shall be presented in detail in the main part. This part includes the results of the numerical simulation after the verification process for the beam, slab, and column under different conditions, i.e., phase 1, under ambient temperatures; phase 2, under elevated temperatures, and finally, phase c, after strengthening of the fire damaged concrete element. The simulation works shall be divided into three phases or stages: Phase 1 for the referenced undamaged models, Phase 2 for the fire-damaged models, and finally Phase 3 for the strengthened models.

## 4. Analysis and Results

The indicators (ultimate load capacity, stiffness, ductility, and toughness) were tested at each of the three stages of work for all structural elements and for both types of concrete (NSC and HSC). The ultimate load capacity results of the reference and fired and after strengthening of normal and high strength concrete for three elements (beam, column, slab) are illustrated in Table 2. (R) refers to the reference undamaged concrete member, and (F) refers to the concrete member after exposure to fire.

*Table 2*

**The effect of high temperature on ultimate load capacity**

| Structural member cases | Ultimate load capacity | | | | | |
|---|---|---|---|---|---|---|
| | Column | | Beam | | Slab | |
| | NSC | HSC | NSC | HSC | NSC | HSC |
| R | 4783.3 | 8480 | 2521.2 | 4181.4 | 5941.6 | 10190.5 |
| F | 2582.6 | 7179.7 | 1236.8 | 2937.0 | 4143.7 | 8411.1 |
| CFRP 1.5 | 3040 | 9676 | 1787.2 | 3666.8 | 5080.7 | 9599.9 |
| CFRP 2.5 | 3826 | 10945 | 1975.8 | 4011.4 | 6405 | 11149.8 |

In HSC beams, the enhancement in stiffness is higher than the fire-damaged beam by about 150%. While an enhancement in stiffness was found to be approximately the same in terms of NSC beams, in the case of the stiffness index in slabs, neither the NSC slabs nor the HSC slabs succeed in recovering the initial stiffness, so it can be concluded that even with increasing the CFRP sheet layer thickness to 166 %, the stiffness was the same. For NSC columns, increasing CFRP layer thickness by 166 % resulted in increasing the level of performance by 130 %. While for the HSC column, increasing the CFRP layer thickness by 166 % resulted in increasing the level of performance by 175 %. For both of the concrete strength classes, the stiffness index results of the reference, fired, and post-strengthened elements (beam, column, slab) are illustrated in Table 3.

*Table 3*

**The effect of high temperature on stiffness index**

| Structural member cases | Stiffness index | | | | | |
|---|---|---|---|---|---|---|
| | Column | | Beam | | Slab | |
| | NSC | HSC | NSC | HSC | NSC | HSC |
| R | 790.67 | 1716.67 | 143.10 | 188.74 | 216.53 | 376.14 |
| F | 249.3 | 1188.46 | 54.85 | 98.39 | 115.42 | 300.75 |
| CFRP 1.5 | 603 | 1798 | 56.36 | 155.61 | 115.87 | 299.65 |
| CFRP 2.5 | 699 | 1922 | 69.08 | 140.94 | 116.74 | 300.36 |

CFRP has a greater impact on the ductility performance of the beam in normal-strength concrete beams than in the high-strength class. On the other hand, for high-strength concrete, a 166 % increase in the thickness of the CFRP layer resulted in an 188 % increase in column ductility. Compared to normal-strength concrete, which received no benefit from the increase, for both normal and high-strength concrete slabs, the 1.5 mm CFRP layer thickness had the same efficiency. Compared to the 2.5 mm thickness, which showed that the improvement in the normal strength was nearly 1.5 times greater than the high-strength slab. These findings regarding the ductility index on the CFRP-treated columns reveal that a notable improvement has been made with this treatment, and full recovery of the column's ductility has been achieved for both types of concrete strength. The ductility index results of the reference and fired and after strengthening of normal and high-strength concrete for three elements (beam, column, slab) are illustrated in Table 4.

*Table 4*

**The effect of high temperature on ductility index**

| Structural member cases | Ductility index | | | | | |
|---|---|---|---|---|---|---|
| | Column | | Beam | | Slab | |
| | NSC | HSC | NSC | HSC | NSC | HSC |
| R | 2.07 | 1.68 | 2.63 | 2.79 | 4.54 | 3.46 |
| F | 2.09 | 1.99 | 2.79 | 3.12 | 3.71 | 3.15 |
| CFRP 1.5 | 3.13 | 4.20 | 3.69 | 3.02 | 3.65 | 3.20 |
| CFRP 2.5 | 2.81 | 7.93 | 6.02 | 3.77 | 8.24 | 4.62 |

The CFRP-strengthened members have sufficiently improved for normal-strength concrete when compared to the fire-damaged beam type. Furthermore, in high-strength concrete, the improved beams' toughness was approximately 138 % and 117 %, respectively, higher than the reference undamaged beam. In terms of columns, the absorption energy has been recovered successfully for both types of concrete strengths. Moreover, for normal and high-strength concrete slabs, respectively, increasing the CFRP layer thickness by 166 % increased the slab's capacity for absorption energy by around 581 and 238 %, respectively. The absorption energy index results of the reference and fired and after strengthening of normal and high-strength concrete for three elements (beam, column, slab) are illustrated in Table 5.

*Table 5*

**The effect of high temperature on absorption energy index**

| Structural member cases | Absorption energy index | | | | | |
|---|---|---|---|---|---|---|
| | Column | | Beam | | Slab | |
| | NSC | HSC | NSC | HSC | NSC | HSC |
| R | 39.199 | 51.335 | 84.352 | 189.001 | 513.427 | 591.032 |
| F | 23.120 | 43.956 | 48.188 | 133.99 | 314.974 | 491.251 |
| CFRP 1.5 | 51.318 | 118 | 166.444 | 261.718 | 278.399 | 509.853 |
| CFRP 2.5 | 200.310 | 385 | 413.190 | 221.522 | 1617.729 | 1217.200 |

## 5. Conclusion

The use of CFRP as effective reinforcement and repair techniques for improving the characteristics of various reinforced concrete members both before and after exposure to high temperatures was discussed in this research. We draw the following conclusions from this study:

1. HSC fire damaged columns, confined with 1.5 and 2.5 mm of CFRP layer thickness, improved the load capacity significantly by about 134 and 152 %, respectively. While NSC Column, confined with 1.5 and 2.5 mm CFRP layer thickness resulted in improving the load capacity significantly by about 117 and 148 % respectively.

2. The load carrying capacities were enhanced by about 144 and 159 % for 1.5 mm and 2.5 mm CFRP strengthened NSC beams, respectively. The 1.5 and 2.5 mm CFRP strengthened HSC beams have reflected load capacities higher than the fire damaged beam by about 124 and 136 %, respectively.

3. It can be noticed that the load capacity has been recovered in the NSC slab by about 122 % and fully recovered by 154 % for the 1.5 and 2.5 mm CFRP thickness layers, respectively. While in HSC slabs, load capacity has been recovered to about 114 % and fully recovered to 132 % for 1.5 and 2.5 mm CFRP thickness layers.

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