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Performance of reinforced concrete elements retrofitted with SIFCON under elevated temperatures

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ABSTRACT

Reinforced concrete (RC) elements lose their structural strength by exposure to fire as the mechanical properties of the reinforcing steel and concrete deteriorate due to the heat. This study provides an understanding of the behavior of three structural concrete elements: beam, column, and slab at high temperatures using a finite element (FE) modeling produced in ABAQUS, as well as the potential of retrofitting these elements using Slurry Infiltrated Fiber Concrete (SIFCON). The use of SIFCON has shown excellent results compared to the fire-damaged elements and to the undamaged concrete elements in terms of various indices (ultimate load capacity, stiffness, toughness, and ductility).

1. Introduction

When exposed to high temperatures, concrete loses material characteristics including strength and elastic modulus and develops cracks and spalling. Also, it causes concrete's material properties to change [1]. Therefore, it is necessary to understand how material properties change at elevated temperatures in order to explore how fire-damaged constructions behave. Failure of concrete due to fire occurs from loss of bending or tensile strength, loss of bond strength, loss of shear or torsional strength, loss of compressive strength, and spalling of the concrete [2]. Khalaf & Huang [3] presented research on the bond behavior of reinforced concrete members under fire conditions. The findings showed that concrete cover, which serves as the limitation to the reinforcement for both the reinforced concrete beam and the slab has a significant impact on the bond strength.

The impact of elevated heat on the thermal and static mechanical characteristics of RC beams were investigated by Liu et al. [4] They used numerical simulation, theoretical analysis, and machine learning techniques. For four RC simply supported beams, fire tests and static stress testing following natural cooling were conducted. The outcomes revealed the maximum spalling depth rose and cross-sectional temperature curves were steadily elevated as fire exposure time increased. After long fire exposure times, RC beams' residual flexural strength dropped.

Ibraheem and Abdullah [5] concluded that steel structure members are susceptible to fire-induced damage or collapse due to high thermal conductivity and rapid loss of rigidity and strength. They carried out experimental tests to examine the response of steel beams to fire. The test

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results showed that the beams are prone to rapid failure due to a significant loss of yield and ultimate strength of the steel beam. The mechanical properties of steel that could also impact the strength and stiffness of reinforcement include yield strength, ultimate strength, modulus of elasticity, and stress-strain relationship. These types of attributes are influenced by temperature, reinforcing type, and heating and strain rates ,the strength and stiffness of reinforcement decrease with increasing temperature [6].

Buildings made of reinforced concrete have received more attention recently in terms of strengthening or rehabilitation. The need for strengthening or rehabilitation arises due to changes in code requirements, architectural layout changes that result in increased loading, weather-related degradation of reinforcement material, damage to the concrete's outer layer from corrosion and fire, and age-related of the concrete, among other factors [7]. To restore their original functionality, damaged structural components must be retrofitted. The size of the damage, the shape of the component, the repair materials, the cost, the duration, and the functions of the component are just a few of the many factors that need to be carefully considered before carrying out repairs for fire-damaged components [8]. The numerous research investigations on the structural restoration of RC have already been conducted [9, 10]. As for SIFCON, it can be seen as a new development in the field of civil Engineering, which has a significant impact on the strengthening and maintenance of concrete members after they have been damaged to return them to service again. SIFCON is created by integrating steel fibers into the cement mortar matrix to obtain high levels of mechanical properties [11]. In fact, SIFCON can be considered a type of fiber-reinforced concrete which characterized by good ductility and significant energy absorption capabilities. SIFCON is used in heat-resistant applications, explosion-proof buildings and pavement overlays [12]. The slurry of cement, fine sand, pozzolanic materials, water, and chemical additives is prepared to create SIFCON by filling a fiber-containing mold with the combination at a ratio of 4-20%, and it can reach 30% by volume [13]. Figure 1 identical casting procedure of SIFCON

Figure 1. Casting procedure of SIFCON

The aim of this study is to take advantages of the possibility of the ABAQUS software in studying the properties of concrete elements (beam, column, and slab) and the changes that occur in those properties after they are exposed to fire for two hours. Also, the performance of SIFCON confined concrete elements under elevated temperatures was investigated.

2. Material and methods

The full-scale dimension of three main concrete elements that forming most of buildings are modelled in ABAQUS. For beams, the dimension are 5500 mm, 300 mm, and 500 mm for length, width, and thickness, respectively. The beam was reinforced 3 steel bars top and bottom using 16 mm bar diameter, and transverse reinforcement (stirrups) with constant spacing along the beam length using steel bar diameter of 10 mm each 200 mm. While for column, the dimensions are selected to ensure that column is classified as short column and the failure mode will occur due to concrete crushing. The column cross section dimensions are; 300 mm by 400 mm, and length of 3000 mm. The longitudinal steel reinforcement was 6 bars with diameter bar of 16 mm, and the transverse reinforcement (ties), steel bar diameter of 10 mm each 200 mm spacing. Finally, for slab, the dimensions were selected to ensure that its behavior is classified as two-way action slab. The slab depth is 200mm with dimensions of 5500 mm by 5500 mm. The embedded steel reinforcement is two meshes (top and bottom) using steel bar diameter of 12 mm each 200 mm.

Validation using previous experimental work is required before beginning any parametric investigation using the finite element simulation. Thus, a previous study conducted by Bin Cai et al. has been used as the basis for the validation process [14]. The numerical validation showed a very close results to the adopted experimental research work in two aspects: the thermal loading and mechanical performance in terms of load displacement curves. Also, the numerical results of the performed phases showed that elevated temperatures have significant effect on the concrete member's performance. For geometrical modelling, the same dimension for the concrete elements and steel reinforcement were used, except the jacketing. A 3D deformable solid part, with two different thickness of 20 and 30 mm were modeled using the extraction method, with the same length of concrete elements, as illustrated in Figure 2.

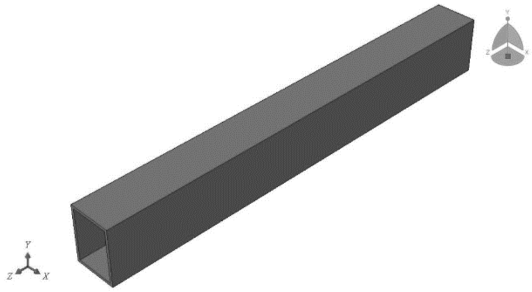

Figure 2. Modeling part of the jacketing

SIFCON layer jacketing is applied as a strengthening technique after subjecting the concrete elements to high temperatures. 20 mm and 30 mm SIFCON layer thickness are used. These layers are assumed to be casted on the damaged concrete faces, one face (the bottom one) for the slab element, three faces for the beam (sides and the bottom faces as U shape), and four faces for the column element.

This research work is carried out using the finite element method within ABAQUS software and is divided into three stages as following:

1. The first stage shows the effect of high temperatures on the performance of various concrete elements (beam, column, slab), the heat distribution map on the cross sections of the section of concrete elements is shown using thermal analysis.
2. The second stage involves mechanical loading as well as the effect of thermal loading performed at the first stage to check the residual capacity of the selected concrete members using paired temperature displacement analysis.

3. The third stage involves strengthening the fire-damaged concrete members with (SIFCON) jacket using the same type of analysis as the second stage.

The study will assume the fire will last for 2 hours on these members to evaluate the nominal strength after damage. The proposed numerical method for evaluating the post-fire residual load-bearing capacity of reinforced concrete structural elements includes creating interaction diagrams using the residual compressive strength of concrete, which is a function of the highest temperature each finite element that makes up the cross-section reached during the fire. Load carrying capacity is defined as the ultimate load that can be sustained by the concrete element. The load-carrying capacity evaluation process involves measuring the concrete member's existing dimensions and estimating its reinforcement area and concrete strength. When subjected to high temperatures, the load-carrying capacity will be greatly reduced. Equation (1) can be used to calculate the concrete element's capacity to carry axial loads.

$$Pu,(t) = A_c \cdot f_{ck} + A_{sc} \cdot f_y \tag{1}$$

In this equation, Pu(t) refers to the concrete element's axial load carrying capacity, A_c for the concrete's cross-sectional area, f_{ck} for its average strength (HSC: 65 MPa, NSC: 25 MPa), A_{sc} for steel's cross-sectional area, and fy for reinforcing steel's yield strength.

The effect of the elevated temperatures on flexural indices in terms of stiffness index is defined as the initial stiffens at the elastic stage which is equal to the final load at the elastic stage over corresponded displacement, the stiffness index (KI) is termed as the ratio of axial stiffness corresponding to 30% of the ultimate load of the temperature-exposed concrete elements surfaces with SIFCON (K, (s)) to the axial stiffness of the unprotected (reference) elements (K, (r)). KI can be calculated using Equation (2)[16].

$$KI = K(s)/\,K(r) \tag{2}$$

The effect of high temperatures on the performance of the concrete elements in terms of ductility is defined by the ratio between the displacement at peak load (Δu) and the notional yield displacement (Δy), The notional yield displacement (Δy) is defined as the junction of the two straight lines correlated with the load displacement curves at the elastic and post elastic stages, respectively. Equation (3) below was used to calculate the ductility index:

$$\text{The displacement ductility index} = (\Delta u)/\,(\Delta y) \tag{3}$$

In term of Toughness, the area under the load-displacement curve, which represents the energy absorption of the concrete elements that might be sustained before demonstrating a significant drop in load carrying capability, can be used to describe the concrete element's energy absorption capacity. According to previous studies, concrete structures' ability to absorb energy is the best indicator of how they will respond structurally to earthquake motion as well as fires [17].

2.1 Materials models

In ABAQUS, the stress strain data was refined and the elastic phase was separated from each other. The elastic modulus defined as elastic isotropic model, while for the plastic phase behavior concrete damaged plasticity (CDP) had adopted, CDP model was created for applications where the concrete is subjected to arbitrary loading scenarios, including cyclic loading, and is based on the assumption of scalar (isotropic) damage. The model accounts for the loss of elastic stiffness carried on by plastic straining in both tension and compression. It also takes into account the impacts of stiffness recovery during cyclic load [18]. Two types of concrete strength shall be investigated for each member. The first type is High Strength Concrete (HSC) with a compressive strength 65 MPa. The second type is Normal Concrete Strength (NSC) with a compressive strength (25 MPa). The selected CDP material parameter for unfired concrete for (normal and high strength concrete) and the assumed CDP material parameter for SIFCON are listed in table 1 and table 2 respectively, and SIFCON proportion mix for 1 m3 listed in table 3.

Table 1. Selected CDP material parameter for unfired concrete.

Parameter	Selected value
Material model	CDP model
E, MPa	23500 MPa for C25 and 38000 MPa for C65
Poisson’s ratio	Varied according to the temperature
Dilation angel	30 for C25 MPa and 40 for C65
*Ecc	0.10
*Fb0/fc0	1.16
*K	2/3
*Viscosity parameter	0.001

*As recommended by ABAQUS manual

Table 2. Selected CDP material parameter for SIFCON.

Parameter	Selected value
Material model	CDP model
E, MPa	42000
Poisson’s ratio	0.3
Dilation angel	45
*Ecc	0.1
*Fb0/fc0	1.16
*K	2/3
*Viscosity parameter	0.1

*As recommended by ABAQUS manual

Table 3. The assumed mix proportion of SIFCON

Constitutive type	Quantities
Cement kg/m3	872.1
Sand kg/m3	969.0
Silica fume kg/m3 10% replacement	096.9
Steel fiber %	006.1
w/b or w/c ratio	00.33

3. Analysis and results

The ultimate load capacity, stiffness, ductility and toughness indicators were tested at each of the three stages of work and for all structural elements and for both types of concrete (NSC& 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 3. (R) refers to the concrete member in the case of reference undamaged, and (F) refers to the concrete member after exposure to fire.

Table 3. The effect of high temperature on ultimate load capacity

	Ultimate load capacity					
	Column		Beam		Slab	
	NSC	HSC	NSC	HSC	NSC	HSC
R	4783	8480	2521	4181	5941	10190
F	2582	7179	1236	2937	4143	8411
SIFCON 20 mm	6683	28618	1937	3607	5947	10217
SIFCON 30mm	8876	31697	2159	3890	6698	11241

The stiffness index for the NSC column improved as the SIFCON layer thickness increased, in contrast to the HSC column, which did not indicate an increase in stiffness when the SIFCON layer was similarly increased. While the increasing SIFCON layer thickness resulted in a noticeable improvement the stiffness values in both normal and high strength concrete beam, moreover in slab both of normal and high strength concrete treated with 30 mm SIFCON layer thickness were successfully achieved the reference stiffness value. For both of the concrete strength classes, the stiffness index results of the reference and fired and after strengthening for three elements (beam, column, slab) are illustrated in table 4.

Table 4. The effect of high temperature on stiffness index

	Stiffness index					
	Column		Beam		Slab	
	NSC	HSC	NSC	HSC	NSC	HSC
R	790.7	1716.67	143.1	188.74	216.53	376.14
F	249.3	1188.46	54.85	98.39	115.42	300.75
SIFCON 20 mm	487.8	1188.46	82.93	108.73	176.57	302.03
SIFCON 30mm	616.1	616.10	83.66	131.08	222.03	380.19

Both treated beams with SIFCON reflected high ductility values than the fire damaged and reference beams, for both concrete strength classes. On the other hand, both of SICON layer thickness treated slabs were unable to recover the reference ductility. The recovery percentages were 79.9% and 80.1% for normal and high strength concrete, respectively. While Treating fire-damaged columns with SIFCON resulted in improving the ductility by about 6 times and 4 times for normal and high strength concrete, respectively. 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 5. The results indicated that increasing SIFCON layer thickness about 150% resulted in improving the beam toughness, SIFCON contribution is greater in NSC beam than HSC beam. While for slabs NSC, a 150% increase in SIFCON layer thickness has no impact on toughness improvement, and it has very little of an impact on HSC slabs. More than 50 times and 100 times improvement in the absorption energy has been observed for normal, and nigh strength SIFCON treated columns, 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 6.

Table 5. The effect of high temperature on ductility index

	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
SIFCON 20 mm	12.05	8.89	3.52	3.28	3.62	4.70
SIFCON 30mm	13.77	13.77	2.87	2.94	3.64	4.28

Table 6. The effect of high temperature on absorption energy index

	Absorption energy index					
	Column		Beam		Slab	
	NSC	HSC	NSC	HSC	NSC	HSC
R	39.19	51.33	84.352	189.00	513.43	591.03
F	23.12	43.95	48.188	133.99	314.97	491.25
SIFCON 20 mm	01.04	04.89	131.915	181.54	483.04	856.67
SIFCON 30mm	01.59	01.59	155.793	196.34	546.38	936.91

4. Conclusion

Through this study, we conclude the following:

1. Treating the fire damaged NSC column with 20 mm and 30 mm SIFCON layer thicknesses, resulted in enhancing the load capacity by about 260% and 345%, respectively. While treating the fire damaged HSC column with 20 mm and 30 mm SIFCON layer thicknesses enhance the load capacity about 400% and 441%, respectively when compared with the fire damaged column.
2. Increasing SIFCON layer thickness by about 150% (from 20 mm to 30 mm) improved the load carrying capacity for NSC beam by about 122%-174%. While treating the fire damaged HSC beam with 20 mm and 30 mm SIFCON layer thicknesses enhance the load capacity about 122% and 132%, respectively when compared with the fire damaged beam.
3. Treating the fire damaged NSC slab with 20 mm and 30 mm SIFCON layer thicknesses, resulted in enhancing the load capacity by about 143% and 161%, respectively. While treating the fire damaged HSC slab with 20 mm and 30 mm SIFCON layer thicknesses enhance the load capacity about 121% and 131%, respectively when compared with the fire damaged slab.

Authors' contribution

All authors contributed equally to the preparation of this article.

Declaration of competing interest

The authors declare no conflicts of interest.

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