Available online at www.CivileJournal.org

Civil Engineering Journal

Vol. 5, No. 10, October, 2019

Effectiveness of Prolonged Air-Recuring on Strength of Fire Damaged RC Columns

M.Adeel Ehsan [a*], M.Yaqub [a], M. Uzair Khan [a], M.U.A.R. Amir [a], Iqrar Hussain [a]

[a] *Department of Civil Engineering, University of Engineering and Technology Taxila, Rawalpindi ,Punjab-North,47080,Pakistan.*

Received 13 May 2019; Accepted 13 September 2019

Abstract

In this study, the effect of post-fire prolonged air-recuring on the recovery of strength of reinforced concrete was investigated after exposing to elevated temperature levels varying from 300 ºC to 900 ºC. The main objective of the paper is to find the effectiveness of prolonged air-recuring on strength of fire damaged RC columns. In order to achieve this, a total of 24 reinforced concrete circular columns having size Ø 200×1200 mm were casted and tested under compressive loading. Eighteen specimens were heated in an electric furnace such that six specimens at each temperature level i.e. 300ºC, 500 ºC and 900 ºC. After heating, specimens were allowed to cool down naturally at ambient temperature and then specimens were air-recured in open environment (high humidity) following by repairing of respective specimens with various confinement techniques. After post fire prolonged air-recuring, test results showed that decrease in axial compressive at all temperature levels was less compared to values reported in literature for corresponding temperatures. This confirms the recovery of microstructure and thus increase in strength of post heated reinforced concrete circular columns due to prolonged air-recuring. It has been observed that both the confinement techniques i.e. single layer of CFRP only and epoxy injection, steel wire mesh filled with epoxy resin mortar along with CFRP wrapping restored the original strength or even more of air-recured post heated reinforced concrete circular columns compared to that of un-heated control specimens or even more.

Keywords: Fire Damaged; Modelling Temperature Levels; Prolonged Air-Recuring; Steel Wire Mesh; Epoxy Injection and Epoxy Resin Mortar Filling; Carbon Fibre Reinforced Polymer and Compressive Strength.

1. Introduction

It is common knowledge that structural fires have led to a great loss of buildings and damage to property in the past two decades. Therefore, there is a growing need to provide approaches for post-fire repair of structural members to enhance their structural safety. This paper presents a state-of-the-art review on the repair of fire-damaged reinforced concrete (RC) members with axial load [1]. Concrete provides better fire resistance in comparison to other various construction materials because of its incombustible behaviour. During service life of reinforced concrete structures, fire is considered as one of the most critical loading that can hit these structures. The overall structural performance, strength, stiffness, durability, modulus of elasticity, volume stability of concrete is affected by intensity and duration of fire among other mechanical properties. It is well known fact that chemical composition and physical structure of concrete changes when exposed to high temperatures [2]. The chemical changes occur mainly in hardened cement matrix initiating from dissociation/detaching/severance of calcium hydroxide above 300 ºC until complete dehydration of calcium silicate hydrate C-S-H at around 900 ºC.

* Corresponding author: adeelehsan786@gmail.com

http://dx.doi.org/10.28991/cej-2019-03091405

Dehydration involves release of chemically bound water from C-S-H which leads to increase in internal stresses. The increased internal stresses and thermal expansion of aggregates result in development of micro cracks, capillaries, pores and voids. Further decomposition of C-S-H starts at temperature exceeding 600 ºC which causes grinding, crumbling, disintegration and peeling of concrete at around 800 ºC. It is well established fact in literature that significant degradation in strength, varying from 55 to 70 %, occurred in concrete above 500 ºC [3]. But there are very less chances for reinforced concrete structures to be completely destructed even when the intensity of fire is high. The reason for this is that concrete possesses low thermal conductivity and therefore reduction in mechanical properties of concrete is reduced in case of fire. In case of fire damaged concrete structures, structural reliability, integrity and stability is maintained through transferring of loads from fire damaged portions to un-damaged portions proportional to the relative stiffness's of the structural elements. Mostly, it has been observed that in real fire cases, the integrity and duration of fire decreased the strength and stiffness of concrete structures up to the extent where financially and logically both, repair is practically feasible and worthwhile solution rather than destructing and reconstruction of fire damaged concrete structures. In order to provide direct profit to owners and investors fire damaged repaired buildings can be re-used earlier because of faster re-operational and re-functional of building [4-7]. For rehabilitation and re-instatement of fire damaged concrete elements and structures, a wide range of repair and strengthening techniques can be implemented. Conventional repairing techniques like concrete or steel jacketing can increase strength of fire damaged concrete. These schemes invite additional weight, interject/disturb function of building due to prolonged and delayed process of repairing and also involve issue of bond between old and new concrete.

The FRP jacket was effective only along the two diagonals of the cross-section. Confinement is generally more effective in specimens with circular cross-section than those with square cross-section. The change in cross section for some of the specimens from square to circle was implemented. To modify the shape, expansive cement concrete has been utilized to fill the gap between the circular and the square cross sections. The test results indicated that heating up to 500 °C caused a severe decline in compressive strength and the elastic modulus of concrete [8]. The experimental program of concrete jacketing used by Bassam A. Tayeh, showed the ultimate load capacities more than twice those of the unjacketed reference columns and the same axial capacity as the monolithically cast reference columns. Moreover, using the shear studs was found to be the most effective among the three surface preparation techniques [9]. For structural engineers it is a challenge to introduce reliable, operative, fast and cost-effective repairing method in order to restore the original strength and structural performance of the fire damaged concrete up to the level of damage. On the other side, for a variety of civil engineering applications, the introduction of fiber reinforced polymer FRP wraps is gaining acceptance and has become considerably striking/attractive in industry due to various advantages including high strength to weight ratio, high stiffness to weight ratio, better corrosion resistance, excellent fatigue response, ease of application and adaptability i.e. to be molded and shaped to the existing building [10-11]. In past studies the use of FRP composites for repairing, strengthening and retrofitting as an ideal solution has been explored. But only a few studies have recently revealed the feasibility of using FRP composites in repairing and reinstating fire damaged concrete. It was suggested by Yaqub et al. (2013) that combination of ferro-cement mesh along with CFRP may be effective to restore original strength, ductility, energy dissipation and stiffness simultaneously [12-18]. However, to the date it was not investigated experimentally whether this combination is effective or not.

The research work presented in this paper is to check the feasibility of using together FRP composite material, steel wire mesh wrap along with other advance and smart materials in order to introduce an effective repairing scheme which is time and cost effective as well. In this work RC circular columns exposed to the temperatures ranging from 300ºC to 900ºC were air recurred after heating in order to check the effect of pro-longed air re-curing on strength of post heated concrete. The past studies explored that air-re-curing for a shorter period of 28 days did not show any significant increase in microstructure and there was only a slight increase in volume of separated pores.

2. Materials and Methods

This experimental work is conducted to determine the effectiveness of CFRP wrap in order to repair fire damaged reinforced concrete (RC) circular columns. A total of 24 circular columns were casted in laboratory at University of Engineering and Technology, Taxila. Out of twenty four eighteen columns were heated in an electric furnace at different temperatures i.e. 300 ºC, 500 ºC, and 900 ºC, such that six specimens at each specified temperature. The columns were tested under axial compressive loading at different conditions and divided into following four major categories:

Test Matrix Category 1: Six Un-Heated (Control) Columns;

Test Matrix Category 2: Six Post-Heated and Non-Repaired Columns;

Test Matrix Category 3: Six Post-Heated and repaired with CFRP Only;

Test Matrix Category 4: Six Post-Heated and repaired with epoxy injection, epoxy resin mortar and confined with steel wire mesh and subsequently CFRP composite wrap.

All specimens were uniform in shape and size when casted using steel mould having dimensions 1200 mm length and 200 mm diameter. Six 10 mm deformed bars were used as longitudinal reinforcement and uniformly distributed within the cross-section resulting 1.5% longitudinal reinforcement ratio. For all columns, eight 6mm diameter deformed bars were used as ties/link reinforcement with spacing 150 mm centre to centre. The tie bars were provided with a standard hook at the end as per ACI standard and extended into the concrete core. After heating, the columns were allowed to cool naturally at ambient temperature. Then all post heated columns were air-recured in the open moist environment (high humidity) in order to recover microstructure. During air-recuring the post heated specimens gain moisture from open environment/air due to which re-hydration process activates and re-generation of hydration products starts. This leads to decrease in total pore volume along with decrease in connectivity of pore spaces and thus eliminating micro-cracks, voids formed during heating which weaken the concrete core.

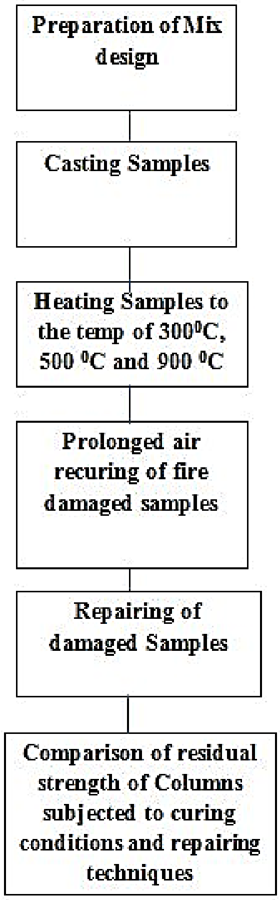

Figure 1. Flow chart showing research methodology

2.1. Materials and Casting

The normal strength concrete mix having 27 MPa cylindrical crushing compressive strength was used in casting of all the specimens. The mix design consisted of sand, gravel aggregate with maximum size of 19 mm and Ordinary Portland Cement (locally available). The concrete mix and its ingredients used for the casting of specimens are shown in Table 1.

Table 1. Properties of material used in casting of columns

Material	Ratio
Mix proportions (Cement : Sand : Aggregate)	1 : 1.25 : 2.5
Water Cement Ratio	0.40
Polypropylene Fibre (Sika Fibre 12)	0.6 kg per Cubic Meter of Concrete

The properties of epoxy adhesive, repair mortar Sikadur 31 CF Slow and steel wire mesh provided by the manufactures are shown in Tables 2 and 3. The typical properties of, Sika Fibre 12(Polypropylene), dry carbon fibre reinforced polymer CFRP Wrap CFW-600, Epoxy based impregnating resin Chemdur 300, and properties of laminate CFRP Wrap CFW-600 and Chemdur 300 provided by the manufacture are shown in Tables 4 to 7.

Table 2. Typical properties of epoxy adhesive and repair mortar

Product : Sikadur 31 CF Slow		
Property	**Value**	**Standard**
7 Days Compressive Strength (N/mm^2) at +25 ^0C	47-57	DIN EN 196
7 Days Flexural Strength (N/mm^2) at +25 ^0C	22-32	DIN EN 197
7 Days Tensile Strength (N/mm^2) at +25 ℃	10-16	ISO 527
7 Days Bond Strength (N/mm^2) with Steel at +25 ℃	13-17	EN ISO 4624, EN 1542 & EN 12188
8 Days Bond Strength (N/mm^2) with Concrete at +25 ℃	> 4	
Density (When Evacuated) at +23 ℃	1.93 kg/lt	
E-Modulus (14 Days at +35 ℃		
1- Tensile (N/mm^2)	~ 3000	ISO 527
2- Compressive (N/mm^2)	~ 2600	ASTM D 695
Elongation at Break after 7 Days at +35 ℃	0.6 ±0.1%	ISO 527
Thermal Expansion Co-Efficient (For +23 ℃ to +60 ℃	7.9 x 10-5 Per ^0C	EN 1770
Thermal Stability after 7 Day +35 ℃ and for Thickness of 10 mm	Heat Deformation Temperature (HDT)>50℃	ISO 75

Table 3. Properties of welded steel wire mesh

Property	**Value**
Wire Thickness (mm)	1.4
Mesh Opening	19 mm × 19 mm
Yield Strength of Wire in Tension (N/mm^2)	20.30
Ultimate Strength of Wire in Tension (N/mm^2)	32.48

Table 4. Typical properties of sika fibre 12

Products Name: Sika Fibre 12 (Polypropylene)		
Property	**Value**	**According to Standard**
Density (kg/lit)	0.91	US CONEG Legislation Limit for Heavy Metals, European Directive 94/62/EC of 20.12.94
Fibre Length (mm)	12	
Fibre Diameter (micron)	18	
Absorption	Nil	
Oftening Point (ºC)	160	
Specific Surface Area (m^2/kg)	200	
Thermal Conductivity	Low	
Alkali Resistance (%)	100	
Electrical Conductivity	Low	

Table 5. Typical properties of dry carbon fibre reinforced polymer

Product Name: CFRP Wrap CFW-600		
Property	**Value**	**According to Standard**
Areal Weight (g/mm^2) Fibers Only	610 ± 20	-
Fabric Design Thickness (mm)	0.337	-
Fabric Density (g/cm^3)	1.79	-
Tensile Modulus N/mm^2 (Minimal)	230,000	ISO 10618
Tensile Strength N/mm^2 (Minimal)	4,900	
Elongation at Break	1.50%	

Table 6. Typical properties of epoxy adhesive for carbon-fibre wrapping

Product Name: Chemdur 300		
Property	**Value**	**Standard**
7 Days Tensile Strength (N/mm^2) at +23 ºC	30	DIN 53452
Density (at +20 ºC)	1.31 ± 0.1 kg/lt	-
Toxicity	Non-Toxic	DIN 53452
Viscosity	Pasty, does not flow	-
Open Time (23 ºC and 35 ºC)	30 minutes	-
Colour & Mix Ratio Component A (4 Parts By Weight) Component B (1 Part By Weight)	 White Grey	-
Pot Life (600 ml) 15 ºC 35 ºC	90minutes 30minutes	-

Table 7. Properties of the laminate

Products Name: CFRP Wrap CFW-600 and Chemdur 300		
Property	**Value**	**According to Standard**
Laminate Thickness (mm) (Nominal)	1.4	ICBO ER 5558 (USA)
Based on typical laminate thickness of 1.4 mm		
Ultimate Load (kN/mm^2)	1000	
Tensile E- Modulus (kN/mm^2)	48	

All the specimens were cast using steel moulds from the same mix of concrete and curing was done as per standard procedure ASTM C31. The steel cylindrical moulds were cleaned properly and a coat of oil was applied on the inner surface of all moulds so that specimens would easily remove during de-molding. All nuts and bolts were tightly fixed with the base plate having no space or gap within all parts of the columns. The cylindrical steel moulds were placed on a clean, level and firm surface. The concrete was placed in to the moulds in layers and each layer. Each layer was compacted using 30 blows of 16 mm rod. The top surface of concrete was levelled using steel float. The moulds were left undisturbed for 24 hours. After 8 hours of casting, all the moulds were covered with damp hessian cloth (jute cloth). After 24 hours all the specimens were removed from the moulds and fully immersed under water. All the specimens were cured for 28 days after which they were left in the laboratory environment until the day of heating.

The duration of heating was selected ranging from minimum 300 minutes to maximum 655 minutes so as to cover a reasonable range of duration of heating based on the following:

1) In order to represent typical structural resistance of heating;

2) To extend the duration 655 minutes to determine if the duration of heating had a noticeable impact on the results (under the most severe heating condition).

The time duration between heating and testing is potentially important when studying the residual compressive strength of concrete. The less time between casting and heating may cause spalling due to the presence of higher moisture content. However, the longer duration between casting and heating may reduce the chances of spalling due to less moisture content. Therefore, the specimens were installed in the furnace at least seven month after casting so that they had low moisture content to avoid spalling. Also 0.025% of polypropylene fibers (Sika Fibre 12) by weight of mix batch was mixed during casting of specimens in order to avoid explosive spalling.

2.2. Heating

Heating of reinforced concrete columns was carried out after seven months of casting in the Carbottom Heat-Treatment Furnace A-49 (S 209), except those at room temperature. The dimensions and specifications of gas furnace used in the current research which was available in the hydraulic shop of Heavy Mechanical Complex (HMC), Taxila, Pakistan, are given in Table 8. LPG fuelled burners were used to control the temperature furnace. The temperature of furnace was monitored by six DT1- thermocouples installed within the furnace at different locations. All the thermocouples were connected with a digital data logging system as shown in Figure 2.

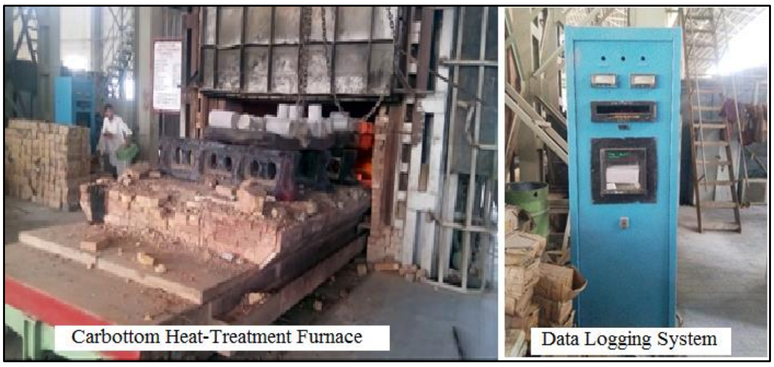

Figure 2. Heating of specimens

Table 8. Heating facility specifications

Name of the Furnace: Car bottom Heat-Treatment Furnace A4-9 (S 209)	
1. Effective Dimensions	2.54 × 4.64 × 11.79 m
2. Maximum Temperature	1150
3. Loading Capacity	30 ton
4. Fuel	Natural Gas
5. Fuel Consumption	165 Normal Cub m/m
6. Gas Pressure	500 mm Hg
7. Air Pressure	200-300 mm Hg
8. Type and No. of Burners	DT1-14 Pcs
9. Power Capacity	
Car Pulling Mechanism	10 KW
Door Lifting Mechanism	2.2 KW
Power for Burner	5.5 KW

All the specimens were heated upto the chosen/required furnace target temperature separately not in one go i.e. the specimens required to heat upto 300 ºC were loaded first, un-loaded when furnace target temperature of 300 ºC was achieved and then switched off. Similar process was repeated to heat other specimens at temperature levels of 500 ºC and 900. The rate of heating was set at 1.6 ºC/minute. It should be noted that the heating rate of 1.6 ºC/minute refers to the rate of temperature rise within the furnace, not the temperature rise inside the concrete columns.

In the current study, the furnace used was not able to achieve rapid heating rates which are not representative of the standard fire scenario. However, the current study is mainly concentrated on the peak furnace temperature exposure because the standard fire does not necessarily reflect or simulate the realistic heating of concrete within a real structure during a real fire [5]. In real fire, the severity of fire depends on the external peak temperature and duration of fire. The outer layers of concrete members are subjected to higher temperature compared to the temperature levels at the inner layers of the structural members during the real fire. The peak temperatures and duration adopted in the current study is shown in the time temperature curve (refer to Figures 3 to 5). After heating and cooling, all the specimens were safely shifted to the structural testing laboratory for repairing process which started almost 90 days after heating.

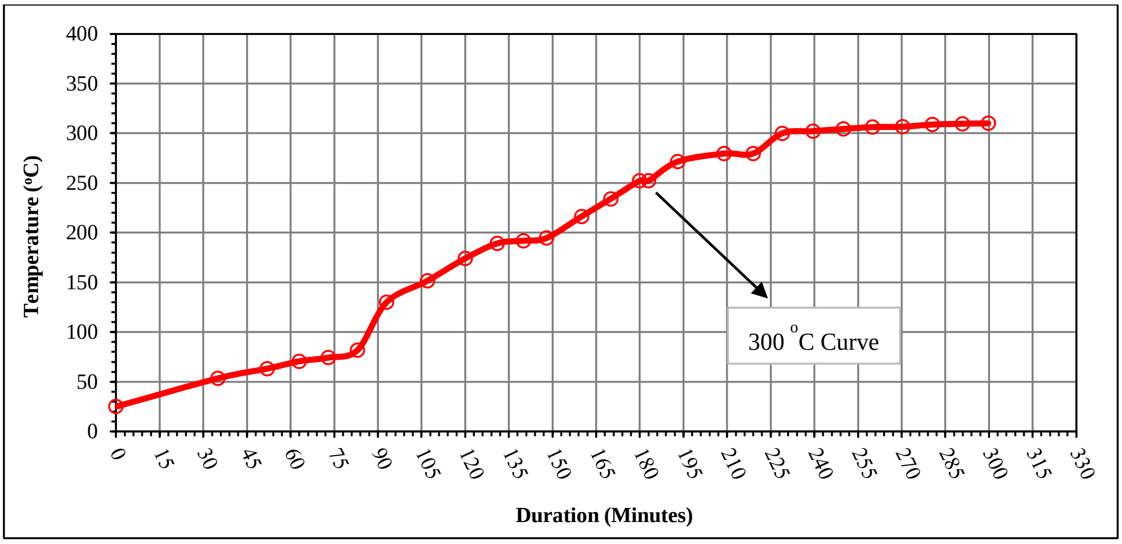

Figure 3. Time ~ Temperature Curve for 300 ºC

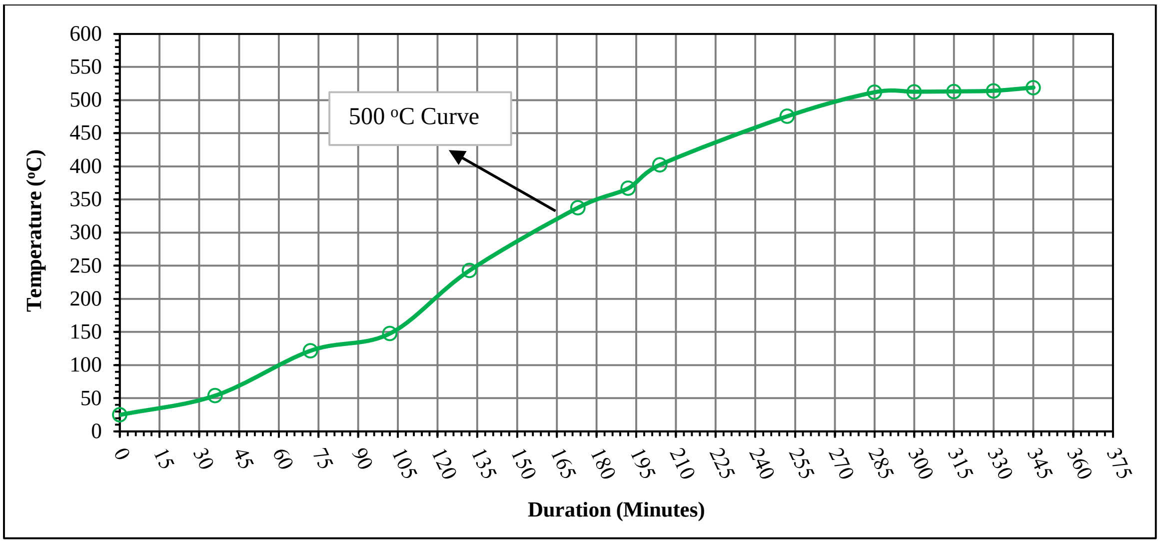

Figure 4. Time ~ Temperature Curve for 500 °C

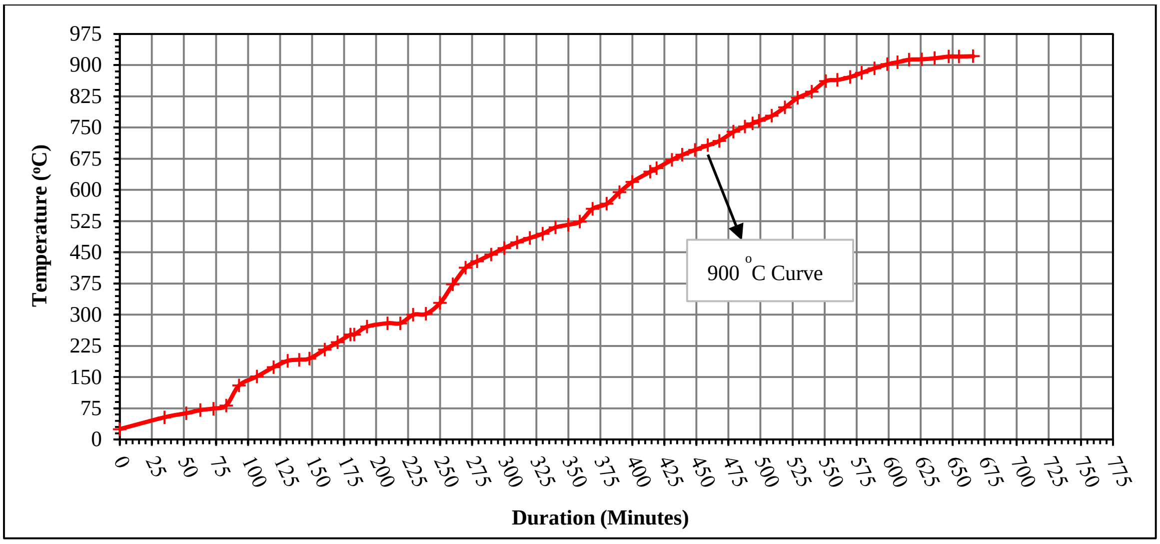

Figure 5. Time ~ Temperature Curve for 900 °C

3. Repairing Process of Air-Recured Post Heated Reinforced Concrete Circular Columns

3.1. Surface Preparation

The substrate of the post-heated reinforced concrete circular columns was prepared by removing loose and friable particles with a steel wire brush. Sikadur 31 CF Slow epoxy resin mortar was used to fill the cavities, micro cracks, voids and dikes of post heated specimens to get smooth surface. Sikadur 31 CF Slow adhesive and repair mortar components A and B were mixed in the ratio of 2:1 by weight for at least 3 minutes with a mixing spindle attached to a slow speed electric drill (maximum 300 rpm) until the paste became uniform in consistency and greyish colour was achieved. While mixing aeration was avoided. The whole mix was poured into a clean container and stirred again for approximately 1 minute at low speed to minimize air entrapment. It was observed that there was no loss of section for columns except those exposed to 900 °C temperatures. Therefore, a thin layer of not more than 2 mm epoxy resin mortar was applied on the surface of columns using spatula and trowel in order to fill cavities, micro cracks.

3.2. Repairing Technique using Steel Wire Mesh and Epoxy Resin Mortar

The surface of specimens was made smooth with grinder having steel disc attached to it and further substrate of post-heated reinforced concrete columns was prepared by removing loose and friable particles using steel wire brush. In this technique, the same size of holes 10 mm in diameter and 76.3 mm in depth were made normal to the longitudinal axis of the post-heated columns using drilling machine. The same numbers of holes i.e. 24 were made on the surface area of the post-heated circular columns in a staggered manner, such that six holes in one layer along longitudinal axis. The surface of the post-heated columns was cleaned using steel wire brush and the dust due to drilling was removed from the holes using blower cleaner. Sikadur 31 CF Slow adhesive and repair mortar components A and B were prepared by mixing in the ratio of 2:1 by weight for at least 3 minutes with a mixing spindle attached to a speed electric drill (maximum 300 rpm) until the material becomes smooth in consistency and a uniform grey colour. The aeration was avoided while mixing. The whole mix was poured into a clean container and stirred again for approximately 1 minute at low speed to keep air entrapment at a minimum. The prepared epoxy resin mortar Sikadur 31 CF Slow was filled into the drilled holes with the help of thin steel rod. The epoxy resin mortar was pressed into the holes using thin steel scrapers. After filling the holes with epoxy resin mortar Sikadur 31 CF Slow, the specimens were left undisturbed up to 2 days for subsequent steel wire mesh wrapping. The length and width of the wire mesh was cut according to the perimeter and height of the post-heated concrete columns together with an additional 200 mm extra for overlapping. A wooden hammer was used to keep the wire mesh as adjacent to the surface of post-heated columns as possible. Figure 5 (b and c) shows the specimens after wrapping with single layer of steel wire mesh and ready for filling of epoxy resin mortar Sikadur 31 CF Slow. A primer coat of epoxy Sikadur 52 LP was applied on the surface of the post-heated epoxy resin mortar injected specimens confined with the steel wire mesh jacket. The epoxy resin mortar Sikadur 31 CF Slow was poured into the steel wire mesh by hand using thin steel scrapper to form 15 mm thick epoxy resin mortar steel wire mesh jacket. A clear cover of approximately 2.5 mm was provided to the outer face of the jacket. A gap of 12.5 mm was maintained at two ends of the post-heated concrete columns, between surface and the steel wire mesh jacket to avoid direct axial loading to the steel wire mesh jacket while testing. Thereafter, the post-heated repaired concrete columns were air cured at room temperature in the laboratory environment for up to 2 days for CFRP wrapping consequently.

Figure 6. Repairing of Specimens

3.3. Application of CFRP Jacket

After preparing surface of post-heated reinforced concrete columns and repaired with technique mentioned in sections were wrapped with carbon fiber reinforced polymer jacket. Only single layer of unidirectional carbon fibre reinforced polymer (CFRP) Imporient wrap CFW 600 (0.337 mm thick) was used in this research to investigate the effect of fibre reinforced polymer on strength, stiffness, energy dissipation and ductility of the post-heated repaired reinforced concrete circular columns. The fabric sheet was cut according to the perimeter and height dimension of the specimens to be wrapped including a hoop overlap of 200 mm in accordance with the manufacture's recommendations. The CFRP fabric was impregnated with epoxy Chemdur-300 and then wrapped around the heat damaged repaired specimens with the main fibres oriented in the transverse direction using a wet layup technique as shown in Figure 6. A steel roller was used in order to distribute the epoxy on the CFRP layer to ensure good impregnation and to remove all entrapped air bubbles. A gap of 25 mm was maintained at top and bottom of the reinforced concrete circular columns between the end and the CFRP jacket to avoid direct axial loading to the CFRP jacket while testing. The CFRP wrapped specimens were allowed to cure in the open environment before testing.

3.4. Instrumentation and Testing Procedure

All columns were capped with plaster cement as per ASTM C 617-98 "Standard Practice for Capping Cylindrical Concrete Specimens at their ends, top and bottom to confirm parallel surfaces and distribute load uniformly before testing. Each column was instrumented with three linear displacement sensors (LDSs) and one linear variable displacement transducers (LVDTs). Two LDS were fixed at mid-height on all specimens over a total length of 355 mm in the longitudinal direction. These LDSs were fixed between two steel angles which were attached to the column surface using a screw driven into drilled hole of 5 mm depth. Further each column was instrumented with one LDS and one LVDT in hoop direction at mid-height over the CFRP wrapped surface to determine lateral. The sensors were attached at the mid height and on the opposite faces of each column. The load was applied, using load control, at a loading rate of 0.05 MPa/s. The columns were tested to failure under axial compression using 5000 kN capacity compression testing machine and the data was monitored and logged using an automatic data acquisition system (P3 box) as shown in Figure

6. Steel collars of about 5 mm thickness were used up to depth of about 200 mm (equals to diameter of specimen) at top and bottom of columns in order to avoid local failure near ends due to machine platen effect at column edges and transfer load till centre so that failure would occur nearly at centre and full capacity of column would be assessed.

Figure 7. Testing Setup

4. Results and Discussion

4.1. Explosive Spalling

In this experimental study, although spalling was not major parameter to be explored. Due to significant loss of concrete, explosive spalling can cause complete and abrupt failure of concrete members. Explosive Spalling depends upon many factors like moisture content, size and type of aggregates, heating rate, cracking and permeability. It is considered that pore pressure stresses plays major contribution to explosive spalling. In the present study, polypropylene fibres were used in concrete during casting to avoid explosive spalling. It is interesting to note that no phenomenon of explosive spalling was observed in any specimen even at high temperature level of 900 ^{o}C. As the temperature of concrete exposed to fire increases, these fibres work effectively allowing moisture and water vapors to escape. There is poor adhesion between concrete and fibre due to polarity mismatch which facilitates moisture to emit through cannel between concrete and fibre. During heating as the temperature increases the fibre contracts in length and expands in width resulting in cavities within the concrete matrix and thus allowing gas to move. In short, during heating of concrete, polypropylene fibre enables to form a network of interconnecting channel due to which explosive spalling was reduced caused by expansion of water

4.2. Failure Patterns and Modes

All the circular columns including un-heated, air-recured, post-heated and air-recured post-heated repaired with CFRP wrap and using another technique i.e. epoxy injection along with steel wire mesh filling with epoxy-resin mortar and following/subsequently/substantially single layer of CFRP wrap together were tested under axial compressive loading exactly in similar way.

4.3. Non-Heated / Non-Repaired And Air-Recured Post Heated / Non-Repaired Columns

In all un-heated/unrepaired & post-heated/unrepaired air recured columns the failure mode was typically crushing. It was observed that in un-heated/unrepaired columns that failure was quite sudden, brittle in nature and pieces of concrete material were chipped off/detached from the surface of columns as shown in Figure 7(c). But failure pattern for air-recured post-heated/un-repaired columns was quite steady and slow/gradual as compared to that of un-heated/un-repaired columns & tiny particles of concrete material were started chipping off before failure which shows ductile behavior. This leads to removal off large pieces of concrete at failure as shown in Figure 7(b). For all these columns vertical cracks were started at the top & bottom of columns and propagated till/up to center as shown in Figure 7(a, b, c) resulting failure/following by crushing, ensuring utilization of full capacity of columns, which shows that setup was effective which was used to avoid local failure at top and bottom edges.

4.4. Air-Recured Post Heated / Repaired Columns

It has been seen that failure of CFRP wrapped reinforced concrete circular columns occurred due to rupture of CFRP as shown in Figure 7. The rupture of fiber was started/initiated at the top and bottom ends propagating or moving till center followed by ultimate failure. This is because the stress created by confinement/jacket intensified at their ends due to effect of top & bottom platens of testing machine. For all air-recured post heated repaired columns in which confinement was provided through carbon fiber, it has been seen that fiber was fully removed/detached outside from overlapping region as shown in the figure. The ultimate failure occurred when recorded lateral strain of specimen exceeded the limit strain of wrap provided by the manufacturer. The failure of air-recurred post heated RC circular columns repaired with CFRP only was started through separation/rupture of CFRP wrap mentioned above. A typical sound was started hearing at a stage when concrete capacity was completely utilized and particles were trying to knock out but jacket/wrap held/packed them tightly till ultimate failure. Also a particular cracking noise was observed before ultimate failure of fiber indicating de-bonding of fiber and full utilization/activation of jacket capacity. At the end, the ultimate failure was undertook sudden, abrupt and bursting sound/noise for all wrapped columns was observed and thus more energy was dissipated in this form.

The ultimate/end failure of epoxy injected steel wire mesh epoxy resin mortar along with CFRP wrapped was almost same i.e. bursting, explosive & abrupt in nature. Vertical cracks were initiated at their ends & propagated till center and fiber was ruptured nearly at center as shown in Figure 8. For air-recured steel wire mesh filled with epoxy-resin mortar along with single layer of CFRP wrap, a slight tilt was observed along the longitudinal axis at failure as shown in Figure 7. The reason behind this was that during/while testing when heated concrete completely crushed/failed/reached its ultimate capacity and tried to split out, but the confining pressure of wire mesh held/grasped it tightly till its ultimate passive pressure. After this confining pressure of fiber/ wrap was activated and grip the concrete inside till ultimate rupture. After detaching of fiber at ultimate load it was noted that no de-bonding was occurred between fire damaged concrete core and steel wire mesh jacket indicating a good contact between concrete core and steel wire mesh epoxy resin mortar jacket.

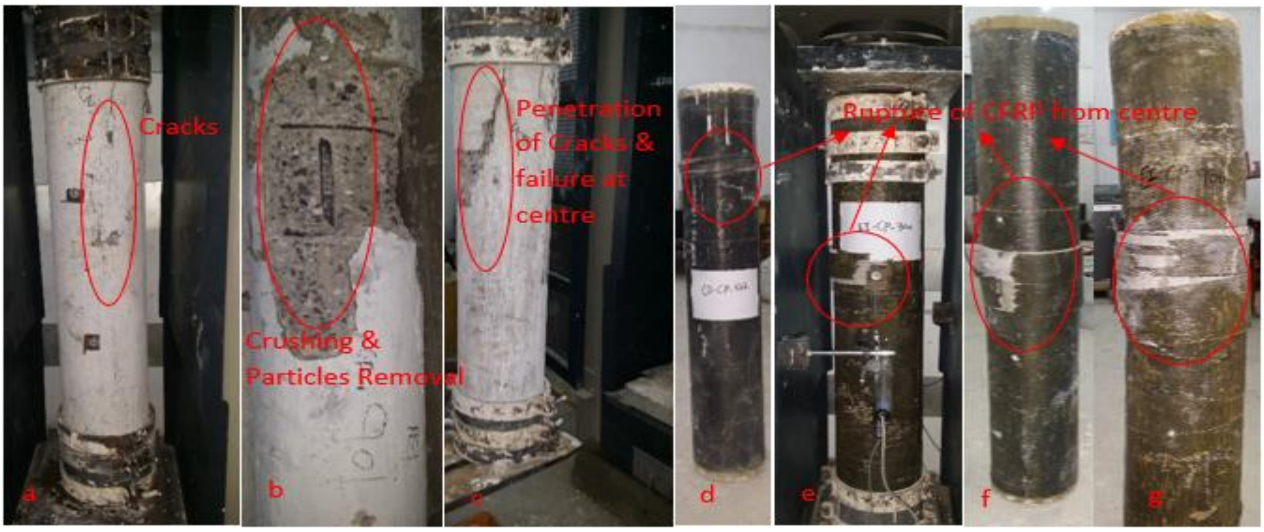

Figure 8. Failure Modes

4.5. Effect of confinement techniques on Compressive Strength

The un-heated un-confined and air-recured post heated unconfined RC circular columns were tested under axial compressive loading to make comparison with those of air-recured post heated specimens exposed to different temperature levels varying from 300 ºC–900 ºC and repaired with various confinement techniques. It is well established fact in literature that there is significant reduction in compressive strength of concrete, varying between 15-40% at around 300 ºC and 41-80% at and above 500 ºC (AL-Nimry et al. [16]). This certified and authorized comparison of these reported values to those of pro-longed air-recuring of post heated columns exposed to levels ranging between 300 ºC -900 ºC. The compressive strength of structural elements/members depends on the maximum axial stress that can withstand/resist by these members. The compressive strength of RC circular columns was calculated by dividing the maximum observed load while testing to the cross-sectional area of the specimen upon which load was transferred/applied. The numbers 1-16 presented on X-axis in Figure 8 represent: Six un-heated unconfined concrete control specimens (Tests 1-6)Two air-recured post-heated (exposed to 300 ºC) unconfined RC control specimens (Tests 7, 8)Two air-recured post-heated (exposed to 300 ºC) singlehandedly/only CFRP confined RC circular columns (Tests 9, 10) Two air-recured post-heated (exposed to 300 ºC) epoxy injection steel wire mesh epoxy resin mortar along with CFRP confined RC circular columns (Tests 11 ,12)Two air-recured post-heated (exposed to 500 ºC) unconfined RC control specimens(Tests 13, 14)Two air-recured post-heated (exposed to 500 ºC) singlehandedly/only CFRP confined RC circular columns (Tests 15, 16)Two air-recured post-heated (exposed to 500 ºC) epoxy injection steel wire mesh epoxy resin mortar along with CFRP confined RC circular columns (Tests 17 ,18) Two air-recured post-heated (exposed to 900ºC) unconfined RC control specimens (Tests 19.20)Two air-recured post-heated (exposed to 900 ºC) singlehandedly/only CFRP confined RC circular columns (Tests 21, 22)Two air-recured post-heated (exposed to 500 ºC) epoxy injection steel wire mesh epoxy resin mortar along with CFRP confined RC circular columns (Tests 23, 24). Figure 8 shows the degradation of compressive strength exposing to the temperatures ranging from 300 ºC to 900 ºC and its restorability using the various repairing confinement techniques. It is evident from Figure 8 that the compressive strength of the RC specimens was degraded after heating and the rate of strength degradation increases with the increasing level of temperature ranging from 300 ºC to 900 ºC.

It can be seen from Figure 9 and Table 9 that the compressive strength of RC columns was degraded 8%, 12% and 30.42% when exposed to temperatures 300 ºC, 500 ºC and 900 ºC respectively. The strength of the post-heated RC circular columns exposed to the temperature 300 ºC was increased up to 78.70%, 83.09% with respect to un-heated control specimens and 94.26%, 99.02% with respect to air-recured post-heated control specimens using singlehandedly CFRP, and epoxy injected steel wire mesh filled with epoxy resin mortar along with CFRP jacketing techniques respectively. However, the strength of the post-heated reinforced concrete (exposed to temperature 500 ºC) was increased by 72.17%, 75.64% with respect to the un-heated RC control specimens and 95.69%, 99.63% with respect to the air-recured post-heated RC control specimens using singlehandedly CFRP, and epoxy injected steel wire mesh filled with epoxy resin mortar CFRP composite jacketing techniques (refer to Table 9) respectively. It is evident from Figure 9 that even severely degraded compressive strength of the post-heated concrete (exposed to temperature 900 ºC) was found to be increased by 60.19%, 64.21%, with respect to the un-heated concrete control specimens and 130.23%, 136.01% with respect to the air-recured post-heated concrete control specimens using the singlehandedly CFRP wrapping, and epoxy injected steel wire mesh filled with epoxy resin mortar along with CFRP composite jacketing techniques (see Table 9) respectively. It is worth to highlight that all the CFRP confinement techniques showed higher strength compared to the unconfined un-heated and un-confined air-recured post-heated RC specimens. This attributes to the fact that the CFRP wrapping system provides a circumferentially uniform confining pressure to the radial expansion of the un-heated or air-recured post-heated RC columns tested under axial compression when the single layer of unidirectional carbon fibre was wrapped transverse to the longitudinal direction of the tested cylinders. It is also interesting to note that the effect of CFRP confinement techniques on the axial compressive strength of the air-recured post-heated concrete was increasing with increasing the heat damaging level ranging from 300 ºC to 900 ºC in the way when compared to respective heat damaging level as shown in Table 9. This could be due to the fact that reduction in compressive strength and under axial compression, the radial expansion of the post-heated concrete was increasing with increasing the level of temperature exposure and thus allowing more tensile stress to be produced in the CFRP composite wrapping system.

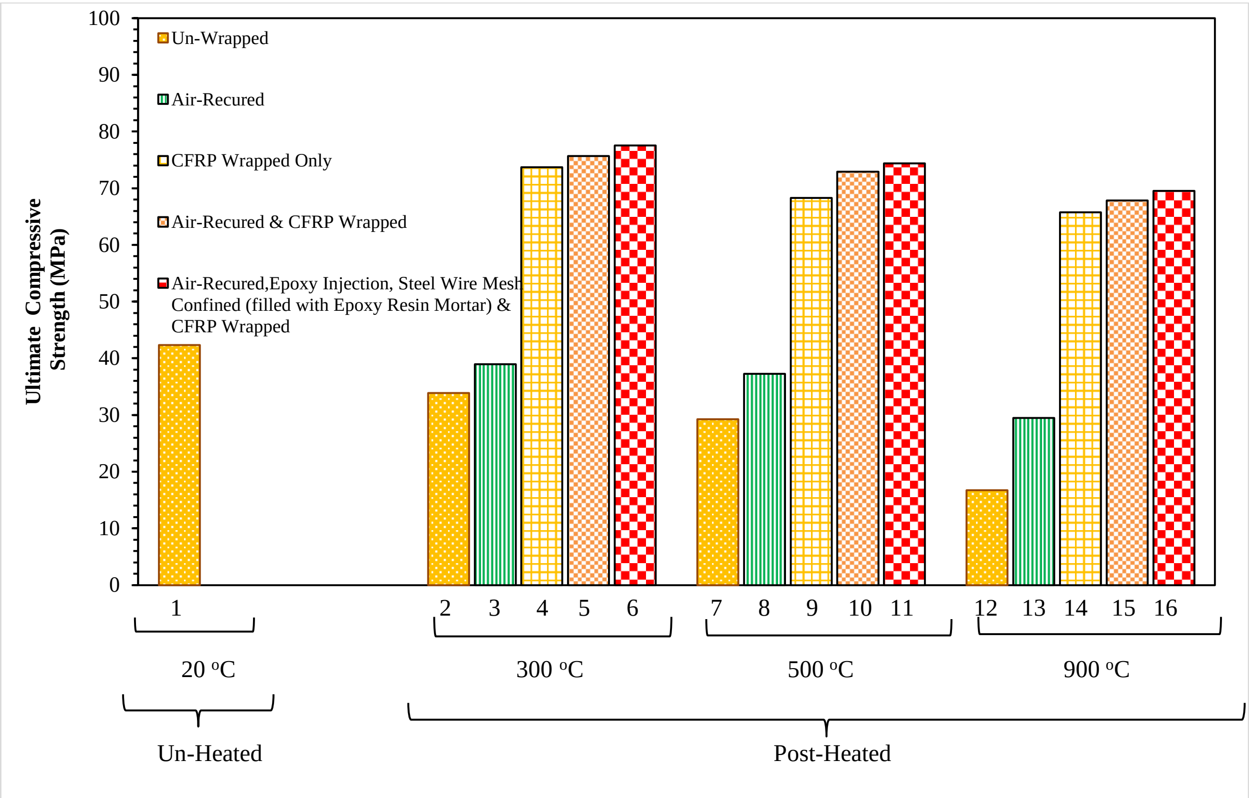

Figure 9. Ultimate Compressive Strengths at various Temperatures

It can be seen from Figure 10 and Table 9 that the CFRP composite jacket restored the original capacities of the axial compressive strength up to un-damaged level of concrete or even more. This indicates that the CFRP composite wrapping system is highly effective when the restoration of the axial compressive strength of the fire damaged structural members is the major concern. It is also interesting to note from Figure 10 that CFRP composite wrapping system showed large increase in compressive strength for higher levels of damage relative to respective degree of damage. It is important to note that reduction in compressive strength shown by air-recured post heated exposed to elevated temperature levels of 300 ºC, 500 ºC and 900 ºC was just 8%, 12% and 30% .These values are not consistent with those reported in literature i.e. 15-40% for 300 and 41% to 80% for 500 and 900 which shows the effect of prolonged air-recuring in open (high humidity) environment. This shows considerable recovery in strength at all temperature levels which attributes to micro-structure recovery and regeneration of hydration products. It is noteworthy that the epoxy injection, steel wire mesh filled with epoxy resin mortar subsequently CFRP composite jacketing technique showed 6% higher compressive strength compared to singlehandedly CFRP composite jacketing technique for the air-recured post-heated concrete exposed to 300ºC,500ºC and 900ºC respectively.

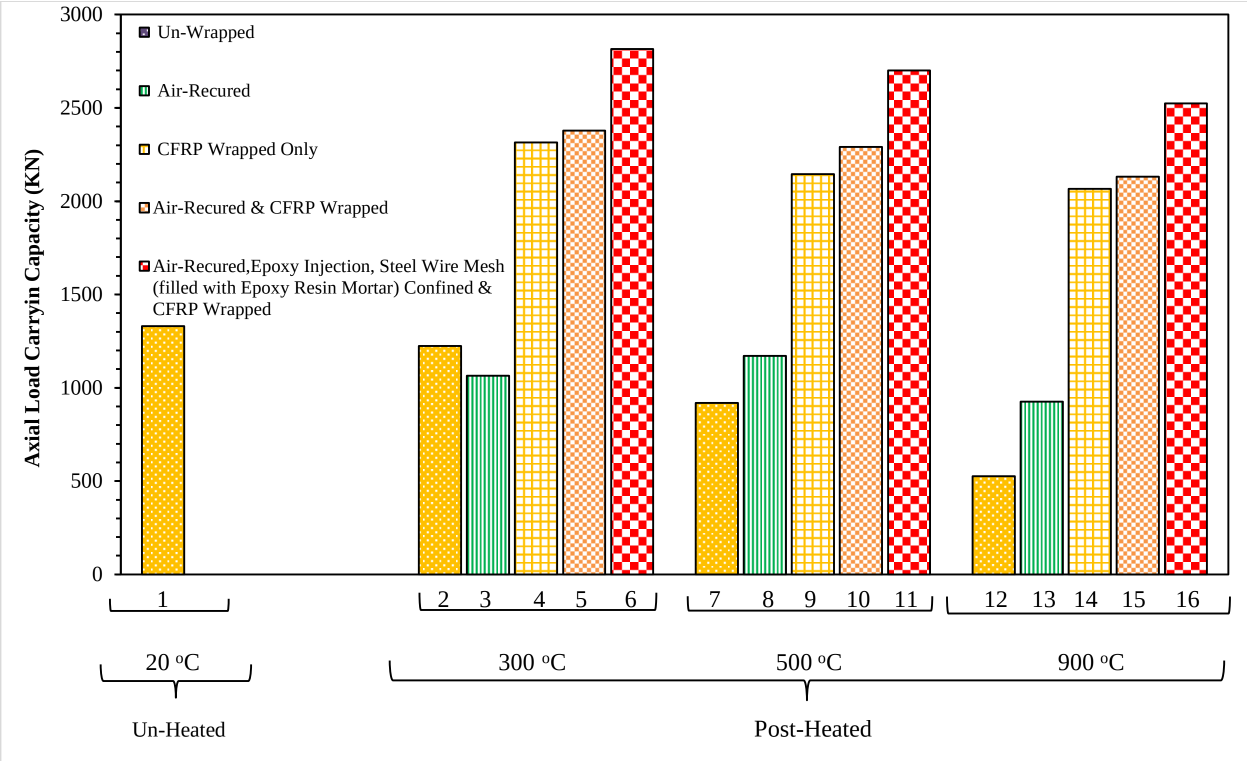

Figure 10. Axial Load Carrying Capacities at various Temperatures

Table 9. Axial Compressive Strength of RC Circular Columns

Test No.	Exposure Temperature	Description of Columns Tested Under Axial Compression	Failure Loads (kN)		Ultimate Strength (MPa)	% Age diff. w.r.t Reference Un-Heated/Un-Repaired Column	% Age diff. w.r.t Reference Corresponding Post-Heated/Un-Repaired Column	% Age diff w.r.t Reference Un-Heated/Un-Repaired Column	Age diff w.r.t Reference Corresponding Post-Heated/Un-Repaired Column
6-Jan	20 ºC	Un-Heated and Un-Repaired/Wrapped	1351.8 1388 1278 1375.9	1330	42.35	-	-	-	-
7-8		Post Heated to 300 ºC and Un-Repaired/Wrapped	1091	1064	33.87	-20.03	-	-20.03	-
9-10	300 ºC	Post Heated to 300 ºC Air Recured and Un-Repaired/Wrapped	1212.8	1224	38.96	-8.01	15.03	-8.01	15.03
11-12		Post Heated to 300 ºC and Wrapped with CFRP Only	2305.1	2316	73.7	74.04	117.62	74.04	117.62
13-14	300 ºC	Air Recured and Wrapped with CFRP	2365.5	2377.5	75.68	78.7	123.45	78.7	123.45
15-16		Air Recured Epoxy Injected, Wire Mesh filled with Epoxy Resin Mortar and Wrapped with CFRP	2810.3	2814.9	77.53	111.58	164.56	83.09	128.93

17-18	500 ºC	Un-Repaired/Wrapped	939.9	919.35	29.26	-30.9	-	-30.9	-
19-20		Air Recured and Un-Repaired/Wrapped	1192	1170.5	37.26	-12.02	27.32	-12.02	27.32
21-22		Wrapped with CFRP Only	2120	2144.9	68.27	61.22	133.31	61.22	133.31
23-24		Air Recured and Wrapped with CFRP	2298.8	2290.6	72.91	72.17	149.15	72.17	149.15
25-26	500 ºC	Post Heated to 500 ℃, Air Recured Epoxy Injected, Wire Mesh filled with Epoxy Resin Mortar and Wrapped with CFRP	2706	2700	74.38	102.97	193.72	75.64	154.17
27-28	900 ℃	Post Heated to 900 ℃ and Un-Repaired/Wrapped	498.1	526	16.75	-60.44	-	-60.44	-
29-30		Post Heated to 900 ℃ , Air Recured and Un-Repaired/Wrapped	870	926	29.47	-30.42	75.87	-30.42	75.87
31-32		Post Heated to 900 ℃ and Wrapped with CFRP Only	2082	2066	65.75	55.26	292.44	55.26	292.44
35-36		Post Heated to 900 ℃, Air Recured Epoxy Injected, Wire	2561	2525	69.54	312.07	313.07	314.07	315.07

5. Conclusions

On the basis of tests results following conclusions are drawn:

- After exposure to elevated temperature levels of 300ºC, 500ºC and 900ºC, the degradation/reduction level of various mechanical properties like strength, stiffness and energy dissipation capacities was found less than those of reported in literature for similar heating conditions. This is attributed to prolonged air-recuring, resulting in recovery of microstructure of post-heated specimens.
- The reduction values for strength reported in literature for 300ºC-500ºC are 15%-60% and for 900ºC 61%-80% and it is also stated that substantial reduction in stiffness level is even more severe than reduction in compressive strength. After post-fire prolonged air-recuring the reduction in compressive strength was 8%, 12% and 30% for temperature levels of 300ºC, 500ºC and 900ºC respectively with respect to un-heated control specimens and similarly the reduction in axial stiffness was 9%, 18% and 35% for temperature levels 300ºC -900ºC respectively.
- A circumferentially uniform confining pressure to the radial expansion of the air-recured post-heated RC columns was applied when tested under axial compression by CFRP composite wrapping system. So, both the CFRP confinement repairing techniques used in this study were found effective in restoring the original axial compressive strength of air-recured post heated RC circular columns (exposed to temperature levels of 300ºC - 900ºC) up to the original level of un-damaged concrete or even more. But the efficiency of CFRP was found increased with increasing temperature level. This is credited to the fact that under axial compression, the radial expansion of the post-heated concrete was increased with increasing the level of exposure temperature and thus allowing more tensile stress to be produced in the CFRP composite wrapping system.

6. Funding and Acknowledgment

This research work was financially supported by the Board of Directorate of Advance Studies and Technological Development (ASR&TD), the University of Engineering & Technology Taxila, Pakistan. The authors are grateful to Sika Pakistan (Pvt) LTD and Imp orient Chemicals (Pvt) LTD for providing technical support to use the advanced repairing materials. The authors are also grateful to the technical staff of Heavy Mechanical Complex (HMC) Taxila for providing the technical support in heating of specimens. The authors are grateful to the technical staff in the Structural laboratory at the University of Engineering and Technology Taxila whose contribution to the experimental work is appreciated.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References

[1] Zhou, Jun, and Lu Wang. "Repair of Fire-Damaged Reinforced Concrete Members with Axial Load: A Review." Sustainability 11, no. 4 (February 13, 2019): 963. doi:10.3390/su11040963.

[2] Yaqub, M., C.G. Bailey, P. Nedwell, Q.U.Z. Khan, and I. Javed. "Strength and Stiffness of Post-Heated Columns Repaired with Ferrocement and Fibre Reinforced Polymer Jackets." Composites Part B: Engineering 44, no. 1 (January 2013): 200–211. doi:10.1016/j.compositesb.2012.05.041.

[3] Yaqub, M., and C.G. Bailey. "Repair of Fire Damaged Circular Reinforced Concrete Columns with FRP Composites." Construction and Building Materials 25, no. 1 (January 2011): 359–370. doi:10.1016/j.conbuildmat.2010.06.017.

[4] Bisby, L.A., J.F. Chen, S.Q. Li, T.J. Stratford, N. Cueva, and K. Crossling. "Strengthening Fire-Damaged Concrete by Confinement with Fibre-Reinforced Polymer Wraps." Engineering Structures 33, no. 12 (December 2011): 3381–3391. doi:10.1016/j.engstruct.2011.07.002.

[5] Yaqub, M., and C.G. Bailey. "Non-Destructive Evaluation of Residual Compressive Strength of Post-Heated Reinforced Concrete Columns." Construction and Building Materials 120 (September 2016): 482–493. doi:10.1016/j.conbuildmat.2016.05.022.

[6] Al-Kamaki, Yaman SS, Riadh Al-Mahaidi, and Ian Bennetts. "Strength Model for Heat-Damaged Reinforced Concrete Circular Columns Confined with Carbon Fibre Reinforced Polymer Fabrics." Journal of Reinforced Plastics and Composites 34, no. 22 (August 21, 2015): 1833–1855. doi:10.1177/0731684415601901.

[7] Huo, Jingsi, Guowang Huang, and Yan Xiao. "Effects of Sustained Axial Load and Cooling Phase on Post-Fire Behaviour of Concrete-Filled Steel Tubular Stub Columns." Journal of Constructional Steel Research 65, no. 8–9 (August 2009): 1664–1676. doi:10.1016/j.jcsr.2009.04.022.

[8] Anasco Bastin, Danie Roy, Umesh Kumar Sharma, and Pradeep Bhargava. "A Study on Different Techniques of Restoration of Fire Damaged Reinforced Concrete Flexural Members." Journal of Structural Fire Engineering 8, no. 2 (June 12, 2017): 131–148. doi:10.1108/jsfe-03-2017-0026.

[9] Hosseinpour, M., M. Celikag, and H. Akbarzadehbengar. "Strengthening and Shape Modification of Fire-Damaged Concrete with Expansive Cement Concrete and CFRP Wrap." Scientia Iranica 26, (November 14, 2017): 699-708. doi:10.24200/sci.2017.4592.

[10] Tayeh, Bassam A., Mohammed Abu Naja, Samir Shihada, and Mohammed Arafa. "Repairing and Strengthening of Damaged RC Columns Using Thin Concrete Jacketing." Advances in Civil Engineering 2019 (June 11, 2019): 1–16. doi:10.1155/2019/2987412.

[11] Chen, G.M., Y.H. He, H. Yang, J.F. Chen, and Y.C. Guo. "Compressive Behavior of Steel Fiber Reinforced Recycled Aggregate Concrete after Exposure to Elevated Temperatures." Construction and Building Materials 71 (November 2014): 1–15. doi:10.1016/j.conbuildmat.2014.08.012.

[12] Poon, Chi-Sun, Salman Azhar, Mike Anson, and Yuk-Lung Wong. "Strength and Durability Recovery of Fire-Damaged Concrete after Post-Fire-Curing." Cement and Concrete Research 31, no. 9 (September 2001): 1307–1318. doi:10.1016/s0008-8846(01)00582-8.

[13] Ma, Ying, Yi Che, and Jinxin Gong. "Behavior of Corrosion Damaged Circular Reinforced Concrete Columns Under Cyclic Loading." Construction and Building Materials 29 (April 2012): 548–556. doi:10.1016/j.conbuildmat.2011.11.002.

[14] Akbas, B., J. Shen, and H. Hao. "Energy Appproach in Peformance-Based Seismic Design of Steel Moment Resisting Frames for Basic Safety Objective." The Structural Design of Tall Buildings 10, no. 3 (2001): 193–217. doi:10.1002/tal.172.

[15] El-Gamal, Sherif, Khalifa Al-Jabri, Ahmed Al-Mahri, and Saud Al-Mahrouqi. "Effects of Elevated Temperatures on the Compressive Strength Capacity of Concrete Cylinders Confined with FRP Sheets: An Experimental Investigation." International Journal of Polymer Science 2015 (2015): 1–10. doi:10.1155/2015/549187.

[16] Al-Nimry, Hanan, Rami Haddad, Saad Afram, and Mohammed Abdel-Halim. "Effectiveness of Advanced Composites in Repairing Heat-Damaged RC Columns." Materials and Structures 46, no. 11 (February 6, 2013): 1843–1860. doi:10.1617/s11527-013-0022-8.

[17] Zahid, MZA Mohd, BH Abu Bakar, FM Nazri, MM Ahmad, and K Muhamad. "Review of Repair Materials for Fire-Damaged Reinforced Concrete Structures." IOP Conference Series: Materials Science and Engineering 318 (March 19, 2018): 012023. doi:10.1088/1757-899x/318/1/012023.

[18] Elsanadedy, Hussein, Tarek Almusallam, Yousef Al-Salloum, and Rizwan Iqbal. "Effect of High Temperature on Structural Response of Reinforced Concrete Circular Columns Strengthened with Fiber Reinforced Polymer Composites." Journal of Composite Materials 51, no. 3 (July 28, 2016): 333–355. doi:10.1177/0021998316645171.