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Axial capacity of post-heated square columns wrapped with FRP composites

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A B S T R A C T

This paper presents the experimental results of a study carried out to investigate the performance of post-heated reinforced concrete square columns repaired with unidirectional fibre reinforced polymers (FRP). The test specimens were divided into three groups: unheated columns, post-heated columns, and post-heated columns wrapped with a single layer of unidirectional glass or carbon fibre reinforced polymer jackets. All columns were tested under axial compression. The experimental results presented compare the effects of glass and carbon fibre reinforced polymer on the performance of post-heated square columns in terms of their stiffness, ductility, ultimate strain and the ultimate strength. The results indicated that a single layer of glass or carbon fibre reinforced polymer enhanced the ultimate strength, ductility and the ultimate strain significantly. However, the stiffness (secant stiffness) of post-heated columns was not improved when wrapped with a single layer of glass or carbon fibre reinforced polymer.

1. Introduction

The repairing and strengthening of concrete structures has become popular and common in the construction industry due to the financial benefits, in terms of direct and in-direct costs, compared to the alternative of demolition and total or partial reconstruction. In terms of structural performance under fire conditions concrete structures generally perform very well in fire due mainly to the low thermal conductivity and incombustibility of concrete. In addition, provided a fire is contained to a small compartment, a concrete structure with correct reinforcement tying details can effectively transfer load away from the heat effective areas towards the unheated structure outside the fire compartment. It is therefore very rare for concrete structures to collapse during a fire and post-fire repair is usually preferred compared to demolition and reconstruction. After a fire, owners and insurers demand reliable, economical and rapid repair techniques in order to avoid financial losses. Various methods for repairing and strengthening of concrete structures, following a fire, have been used in the past [1–5]. However, previous techniques are time consuming and are disruptive in that they involve removing and replacing the fire damaged structural material with a new stronger material. In recent years, fibre-reinforced composites have gained world-wide recognition as strengthening measures especially in the speedy repair and strengthening of existing reinforced concrete columns without any interruption in the use of the structure.

In the past, significant research has been reported on repairing and strengthening of reinforced concrete columns with fibre reinforced polymers (FRP), which have been damaged other than by fire [7–16]. Unfortunately, due to the uncertainties regarding the behaviour of fibre reinforced polymers in any subsequent fire following repair, limited research has been reported on the repairing of fire damaged concrete columns [18–20]. However, it has been found that with applied fire insulation fibre reinforced polymers can perform well in fire [22–29]. The commonly used square geometry for columns, and the lack of awareness of the use of fibre reinforced polymers after fire in concrete buildings, highlighted the need to investigate the behaviour of post-heated square columns wrapped with fibre reinforced polymers. In order to enhance the application of fibre reinforced polymers this experimental study provides useful information to designers who are involved in the field of repairing and strengthening existing fire damaged concrete structures with FRP.

The experimental study presented in this paper was undertaken to investigate the post repair compressive performance of post-heated reinforced concrete square columns wrapped with a single layer of unidirectional glass or carbon fibre reinforced polymers. Nine (one-third scale) square columns were tested in this study. The repair effects of the fibre reinforced polymer (FRP) wrapping on the strength ductility and stiffness (in terms of secant stiffness) were investigated. It was found that a significant enhancement in strength and ductility was achieved with the wrapping of a single layer of glass or carbon fibre reinforced polymers placed around the post-heated square reinforced concrete columns.

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2. Experimental programme

In order to investigate the effectiveness of (FRP) for the repair of heat damaged concrete columns, nine reinforced concrete square columns were cast with gravel aggregate concrete within the structural engineering laboratory at the University of Manchester. The columns were divided into three groups (Table 1):

(a) Without heating and without FRP.
(b) With heating but without FRP.
(c) With heating and repaired with FRP.

For the heated columns, all columns were heated to a uniform temperature of 500 °C at rate of 150 °C/h before being allowed to cool down. The temperature of 500 °C was chosen since at this temperature concrete generally retains a residual strength of approximately 50% of its original strength for normal strength gravel aggregate concrete [6].

For the repaired columns, a single layer of unidirectional glass fibre reinforced polymer (GFRP) or carbon fibre reinforced polymer (CFRP) jacket was used with the main fibres oriented in the lateral direction. A summary of the nine tested columns is given in Table 1. All columns were 200 mm × 200 mm in cross-section and 1000 mm in height. All the specimens were reinforced with eight 10 mm diameter longitudinal reinforcing deformed bars, resulting in a 1.6% longitudinal reinforcement ratio. All bars were evenly distributed throughout the cross-section with 25 mm cover to the main reinforcement (Fig. 1). In all columns, 6 mm diameter deformed bars were used as link bars spaced at 100 mm centres. The link bars were anchored with a 135° hook at each end, which extended approximately 60 mm into the concrete cores, as shown in Fig. 1.

2.1. Material properties

The same concrete mix, comprising sand, gravel aggregate and Ordinary Portland Cement (OPC), was used for all specimens. The maximum size of aggregate used in the mix was 10 mm. The proportions of the cement content, water content, fine and coarse aggregate were 370 kg/m^3, 203.5 kg/m^3, 647.5 kg/m^3 and 1295 kg/m^3 respectively. The average cube strength at the time of testing was 53 MPa. The measured yield strength of the longitudinal and transverse reinforcement was 553 Mpa and 570 MPa respectively. The FRP composites used were Tyfo SEH-51A unidirectional glass fibre, Tyfo SCH-41 unidirectional carbon and Weber.tec force C-240 unidirectional carbon fabric sheets. The ultimate tensile strength in the primary fibre direction, tensile modulus, ultimate elongation and the thickness for all FRP composites used in this study, adopted from suppliers, are shown in Table 2. The epoxy material properties adopted from suppliers are shown in Table 3.

2.2. Casting of specimens

All columns were cast in a horizontal position using steel plates as formwork. For all columns, compaction was carried out using a vibrating table. Two columns were cast at a time from one batch of concrete along with three cubes of 100 mm size in order to monitor the strength at the time of testing. To avoid stress concentration in the fibre reinforced polymer jacket during testing the corners of the square columns were rounded using concave wood sections with a 25 mm radius, placed inside the formwork during casting. Two type K-thermocouples were embedded into each column during casting to monitor the temperature at the time of heating. One

Table 1
Summary of tested columns.

Group	No. of columns	Reference (for Figs. 5 and 7)	Testing conditions	Moisture content (%)	Failure load (kN)	Secant stiffness (kN/mm)
a	2	(1)	Unheated/non-jacketed	–	1965	3854
b	2	(2)	Post-heated/non-jacketed	3.6	1110	653
c	2	(3)	Post-heated/GFRP repaired	3.8	1396	859
	2	(4)	Post-heated/CFRP repaired (Weber.tec system)	3.5	1448	921
	1	(5)	Post-heated/CFRP repaired (Fyfe system)	3.4	1680	564

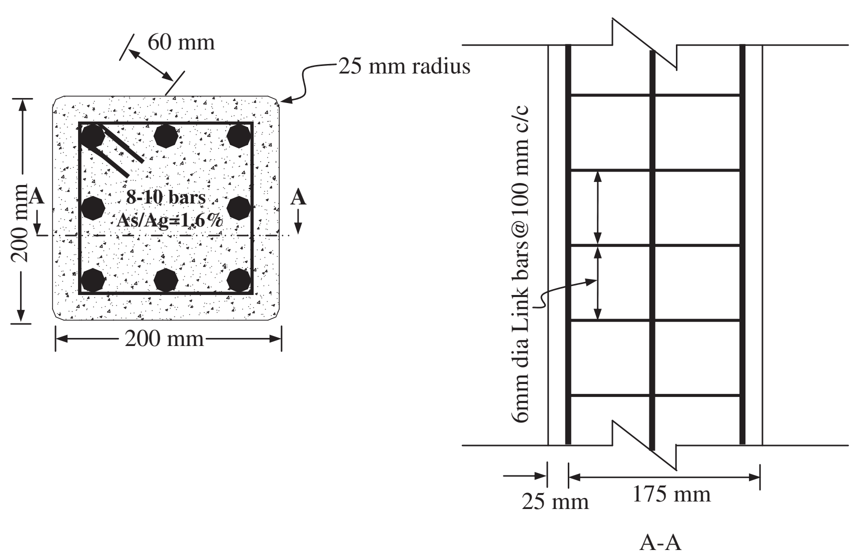

Fig. 1. Reinforcement arrangement in square columns.

Table 2
Composite typical dry fibre properties.

Composite typical dry fibre properties			
PROPERTY	TYFO SCH-41 Fyfe*	TYFO SEH-51A Fyfe*	C Sheet 240 Weber**
Tensile strength (GPa)	3.8	3.24	4.0
Tensile modulus (GPa)	230	72.4	240
Ultimate elongation (%)	1.7	4.5	1.6
Thickness (mm)	0.37	0.36	0.117

Table 3
Epoxy material properties.

Epoxy material properties			
Properties	Tyfo S epoxy Fyfe*	Weber.tec force EP primer**	Weber.tec force EP bonding adhesive**
Tensile strength (MPa)	72.4	19	17
Compressive strength (MPa)	–	100	80
Tensile modulus (GPa)	3.18	5	5
Elongation	5%	–	–
Flexural strength (MPa)	123.4	30	28
Flexural modulus (GPa)	3.12	–	–
Bond to concrete (MPa)	–	>5.3	>4
Coefficient of expansion	–	8×10^{-6} mm/mm °C	6×10^{-6} mm/mm °C
Glass transition temperature (°C)	82	60	60

* Fyfe Europe S.A, "The Fibrwrap Company" Web: http://www.fyfeco.com.
** Saint-Gobain Weber Ltd. Web: http://www.netweber.co.uk.

thermocouple was placed in the centre of the column, and the other was fixed to the longitudinal reinforcement, at the mid-height of the column. The specimens were cured using moist sacking. The columns were cured for 14 days after which they were left in the laboratory environment until the day of testing. All cubes were also kept with the columns for curing until they were tested.

2.3. Heating of columns in the electric furnace

All the columns were heated in an electric furnace having 1.6 m × 1.2 m × 1.5 m size after 9 months storage. Six columns were heated at a time along with nine 100 mm cubes as control specimens, as shown in Fig. 2. An unstressed residual strength test was used to evaluate the residual strength of all the columns, which is considered to be more conservative in terms of design for assessing the post-fire or residual properties of concrete after fire [30–32]. The columns were heated to a uniform temperature of 500 °C at a rate of 150 °C/h. The temperatures within the furnace were controlled by two thermocouples installed at mid-height and at the top of the electric furnace. The tops of the columns were covered with an Isofrax blanket, as shown in Fig. 2. When the average furnace temperature reached 500 °C, the furnace temperature was kept constant until the temperature at the centre of columns reached the same temperature. Heating was stopped when the furnace temperature and temperature at the centre of column coincided. After the furnace was switched off, the columns were allowed to cool down naturally. The cover of the furnace was lifted off when the temperature inside the furnace reached below 200 °C. The time–temperature curve is shown in Fig. 3. After cooling the columns were removed from the furnace and kept in a dry condition at room temperature in the laboratory until testing.

2.4. Repairing of heat damaged columns with fibre reinforced polymer

Before applying the FRP, the rounded corners of the square columns were further ground to avoid any damage to the GFRP or CFRP jackets. The surface of the heat damaged columns was cleaned with a steel wire brush to remove dust and, grease according to the Concrete Society Report 69 [21] before the application of the GFRP or CFRP jackets.

The primer coat consisted of epoxy resin, hardener and Cab-o-sil M5 (micro silica) powder and was applied to the substrate of the five post-heated columns to fill all voids, cavities and micro-cracks. Commercially available unidirectional Tyfo SEH-51A glass fibre, Tyfo SCH-41 carbon fibre and Weber.tec force C-240 carbon fabric sheets were then wrapped around the heat damaged columns, with the main fibres oriented in the transverse direction, using a wet layup technique. The fabric sheets were cut according to the height and perimeter of the columns together with an additional 200 mm extra for overlapping.

Only one layer of fibre reinforced polymer was used in this study to investigate the effect of fibre reinforced polymer on the performance of post-heated columns. Irregularities and air pockets were removed using a roller and hand pressure. Rolling was continued until the fibres were firmly bedded onto the concrete surface and a surface coating of resin became apparent. A small gap of 25 mm was left between the fibre jackets and the ends of the column to prevent the jackets from direct axial loading during testing. The wrapped specimens were then cured in the laboratory environment at room temperature for approximately 1 month prior to testing.

2.5. Test instrumentation setup and testing procedure

All columns were tested under axial compression. Prior to testing all columns were capped with dental plaster at the top and bottom to ensure parallel surfaces and to distribute the load uniformly during testing. Three LVDTs having a gauge length of 275 mm were placed at the mid-height on all specimens whilst a fourth was used to measure the total axial shortening during the test over a 950 mm length. Three horizontal LVDTs of 85 mm gauge length were also attached to each column on three faces to measure any transverse displacements. In addition to the LVDTs, axial and transverse strains at mid-height were also measured using surface strain gauges having a gauge length of 30 mm. The LVDTs were initialised and then the load was applied at a rate of 1 kN/s until failure occurred. A 2500 kN capacity compression testing machine was used and all data was monitored and recorded in an automatic data acquisition system throughout the test.

2.6. Experimental results and discussions

2.6.1. Test observations and failure mode

The surface of the columns was carefully observed following heating. There was no evidence of spalling in any of the columns, even though the moisture content was between 3.4% and 3.8% (Table 1). The moisture content was measured from the cubes, which were heated at the same time as the columns. The colour of all the columns was found to be pink and a few micro-cracks were observed on the surface of each column. For all the columns a typical crushing failure mode was observed under axial compression. The failure of most of the columns was initiated at the top ends of the columns due to high stress concentration (caused by the platen effect of the testing machine) within this region [12,13]. However, a better response could be achieved if the heads of columns were further strengthened with stirrups or ferrocement meshes. For the unheated columns the failure was sudden and explosive, indicating the typical brittle behaviour of the concrete. In contrast, the

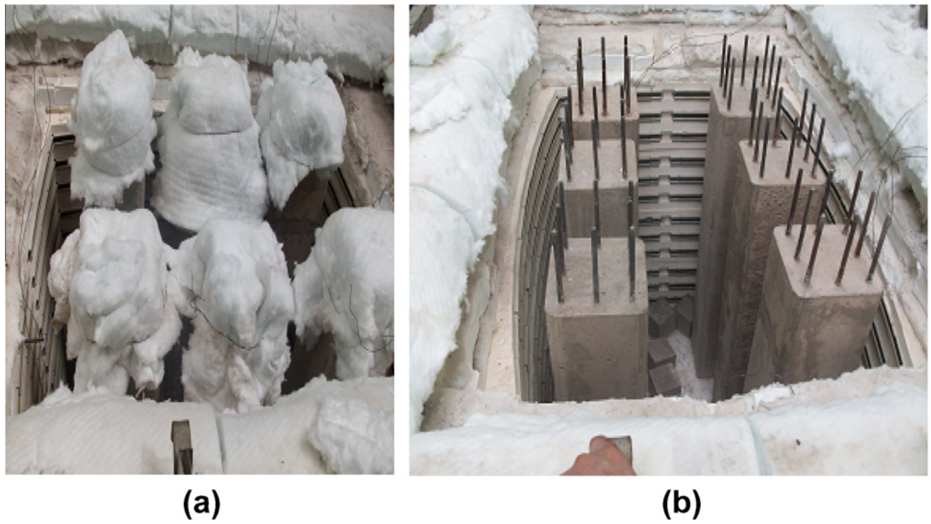

Fig. 2. (a) Insulation of protruded reinforcing bars with Isofrax blanket; (b) columns after heating.

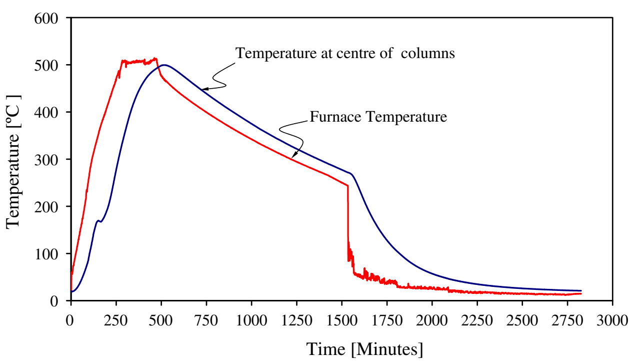

Fig. 3. Time–temperature relationship during column heating.

failure of the post-heated columns was more gradual indicating a more ductile behaviour of concrete after heating. The failure of the GFRP or CFRP wrapped post-heated columns took place by rupture of the fibre jackets at the corners, as shown in Fig. 4. The failure was preceded by a cracking sound, indicating significant activation of the fibre reinforced polymer jackets. As the failure load was approached white patches appeared on the surface of the glass and carbon fibre reinforced polymer close to the top end of the column due to stress concentration. Visible folds generated by lateral dilation were observed with increased loading in the transverse direction close to the top end followed by a sudden bursting failure of the fibre which took place at the corners of columns, accompanied by crushing of the concrete. This suggests that the stress in the square jacket tends to be concentrated at the corners. The sudden and explosive nature of the failure indicates the release of a significant amount of energy as a result of the uniform confining stress provided by the fibre jacket due to rounding of the square corners. More energy was released in the carbon fibre jacket compared to the glass fibre jacket in the form of a sudden and explosive noise, indicating a more brittle failure in the carbon fibre jacket. This could be due to a higher modulus of elasticity of carbon fibre reinforced polymer (Table 2). It also indicates more confining pressure was developed by the carbon fibre jacket compared to the glass fibre jacket. Visual inspection of the damaged columns showed good contact between the jacket and the concrete indicating that no de-bonding had taken place at any stage during the test.

Fig. 4. Failure pattern of FRP composites.

2.6.2. Effect of GFRP and CFRP jackets on ultimate strength

Fig. 5 shows the effect of unidirectional glass and carbon fibre reinforced polymer on the strength of post-heated square columns. The numbers on the *x*-axis represented in Fig. 5 relate to:

1. The average strength of two unheated columns (Tests 1 and 2).
2. The average strength of two post-heated columns after heating to 500 °C (Tests 3 and 4).
3. The average strength of two post-heated square columns (to 500 °C) wrapped with a single layer of Tyfo SEH-51A GFRP jacket (Tests 5 and 6).
4. The average strength of two post-heated columns (to 500 °C) repaired with a single layer of Weber.tec force C-240 CFRP jacket (Tests 7 and 8).
5. The strength of a post-heated square column wrapped with Tyfo SCH-41 CFRP jacket (Test 9).

From Fig. 5, it can be seen that the strength of columns was reduced significantly after heating to 500 °C. However, consider-

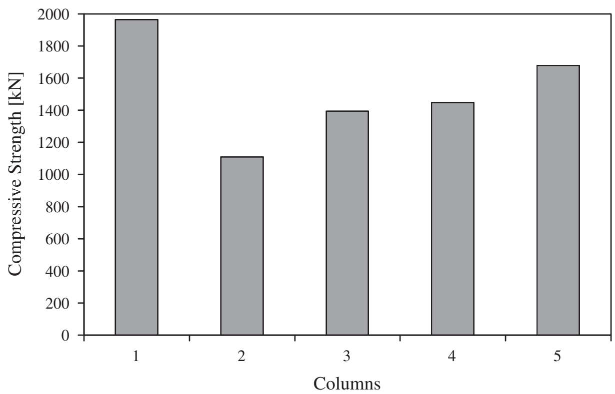

Fig. 5. Compressive strength comparison of unheated, post-heated and post-heated FRP wrapped columns.

able strength was recovered when the post-heated square columns were wrapped with a single layer of Tyfo SEH-51A GFRP (Tests 5 and 6), Weber.tec force C-240 CFRP (Tests 7 and 8) or Tyfo SCH-41 CFRP (Test 9) jackets. This could be attributed to the fact that concrete dilates laterally under axial compression and this dilation causes tensile stresses in the GFRP or CFRP jackets. Due to this tensile stress, the fibre reinforced polymer jacket confines the column and keeps the column in a three-dimensional state of stress. Subsequently, due to this three-dimensional state of stress, the load carrying capacity of the columns was increased significantly. Fig. 5 also compares the improvement in the strength of post-heated columns wrapped with different FRP. It is seen that the ultimate axial load capacity of the post-heated columns was increased by 26%, 31% and 51% of the strength of post-heated columns by wrapping with a single layer of Tyfo SEH-51A GFRP (Tests 5 and 6), Weber. tec force C-240 (Tests 7 and 8) and Tyfo SCH-41 CFRP (Test 9) jackets respectively. However, the efficiency of Tyfo SCH-41 CFRP (Test 9) jacket for the improvement of strength is considerably higher than Tyfo SEH-51A GFRP (Tests 5 and 6) and Weber.tec force C-240 CFRP (Tests 7 and 8) jackets. The confinement effect of Tyfo SCH-41 CFRP jacket (Test 9) is higher due to a greater modulus of elasticity and ultimate tensile strength compared to the Tyfo SEH-51A GFRP (Tests 5 and 6). The strength of post-heated square columns was restored up to 86%, 74% and 71% of the strength of the unheated reference columns when wrapped with Tyfo SCH-41 CFRP (Test 9), Weber.tec force C-240 CFRP (Tests 7 and 8) and Tyfo SEH-51A GFRP (Tests 5 and 6) jackets respectively. This indicates that the strength of post-heated square columns could be restored to some extent, but not up to the original level of the undamaged reference concrete columns when wrapped with a single layer of GFRP or CFRP. This is due to the fact that in a square section, the distribution of lateral confining pressure of the GFRP or CFRP jackets varies from a maximum at the corners to a minimum between the corners due to arching action [10,11,13,14]. Fig. 6 shows the arching action, which takes place in the cross-section of a square column, defining regions of unconfined concrete and regions of effectively confined concrete.

Since no design method is currently available to determine the number of required layers of fibre reinforced polymer for the repair of fire damaged concrete square columns, only a single layer of GFRP or CFRP jacket has been used in this study to validate the effectiveness of this method. However, increasing the number of layers of the fibre reinforced polymer, or increasing the corner radius, improves the strength of square columns, as shown from previous work [10,11,13,15,16].

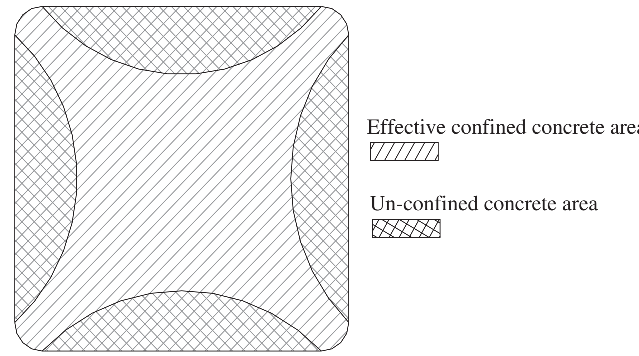

Fig. 6. Arching action of square columns wrapped with GFRP or CFRP jackets.

2.6.3. Effect of GFRP and CFRP jackets on stiffness

Fig. 7 shows the effect of GFRP or CFRP on the stiffness of post-heated columns. The numbers representing the columns on the *x*-axis in Fig. 7 show:

(1) The average secant stiffness of two unheated columns (Tests 1 and 2).
(2) The average secant stiffness of two post-heated columns (Tests 3 and 4).
(3) The average secant stiffness of two post-heated columns wrapped with a single layer of Tyfo SEH-51A GFRP jacket.
(4) The average secant stiffness of two post-heated columns wrapped with a single layer of Weber.tec force C-240 CFRP jacket (Tests 7 and 8).
(5) The secant stiffness of post-heated column wrapped with a single layer of Tyfo SCH-41 CFRP jacket (Test 9).

It can be seen from Fig. 7 that the exposure of columns to 500 °C results in significant loss of stiffness. The degradation of post-heated columns stiffness is mainly caused by the softening of concrete after heating to 500 °C due to micro-cracking. Voids in the micro-structure of the concrete define its porosity and have a significant influence on its stiffness. The porosity of concrete depends on factors related to the water-cement ratio and on the level of internal micro-cracking. On heating, the porosity of concrete is increased due to loss of moisture and due to the development of internal micro-cracking which ultimately results in a loss of stiffness. In the present study the secant stiffness of all columns was calculated from the measured ultimate axial load divided by the measured ultimate axial displacement. The secant stiffness provides a useful comparison for the degradation of axial stiffness of unheated, post-heated and post-heated columns wrapped with GFRP or CFRP jackets.

It is evident from Fig. 7 that the effect of GFRP or CFRP on the stiffness improvement of post-heated columns is negligible. This is due to the fact that the GFRP or CFRP jackets had little confining effect on post-heated square columns from the beginning to the middle stages of the loading [19,20]. As the axial load was increased on the wrapped post-heated column, the concrete compressed due to the micro-cracking and voids created due to heating. This compression took place with little lateral expansion of the column. As the failure load was approached lateral expansion of the column took place activating the GFRP or CFRP confinement effect. Thereafter the GFRP or CFRP jackets exerted continuous pressure in terms of lateral confinement until the rupture of the GFRP or CFRP jackets. Therefore the behaviour in terms of stiffness of GFRP or CFRP wrapped post-heated columns was similar to that of the post-heated unwrapped columns up to the failure of post-heated columns without FRP, as shown in Fig. 13.

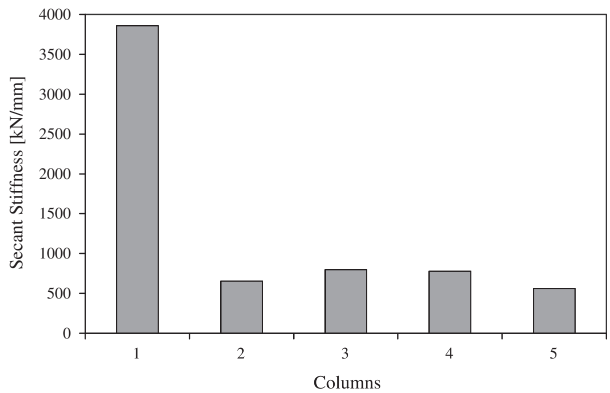

Fig. 7. Secant stiffness comparison of unheated, post-heated and post-heated FRP wrapped columns.

Comparing Figs. 5 and 7, it can be seen that the reduction in residual stiffness of the post-heated columns is higher than the reduction in ultimate load. Since load is redistributed according to the stiffness of the unheated and post-heated parts of the concrete building after fire, it is important to consider the residual stress redistribution prior to repair.

2.6.4. Effect of GFRP and CFRP on ultimate strain

Figs. 8–12 show the average axial stress plotted against lateral and axial strains of unheated, post-heated and post-heated columns wrapped with a single layer of GFRP or CFRP jacket. The axial and lateral strains were measured at the mid-height of all the columns while the axial stress was calculated from the measured axial load divided by the cross-sectional area of the square columns, where the contribution from the GFRP or CFRP was ignored. In Figs. 8–12, the curves ‘S1’ and ‘S2’ indicate the stress–strain relationships of Tests 1 and 2 (unheated/non-jacketed columns). The curves ‘S3’ and ‘S4’ represent the stress–strain relationships of Tests 3 and 4 (post-heated/non-jacketed columns). The curves ‘S5’ and ‘S6’ refer to stress–strain relationships of Tests 5 and 6 (post-heated columns wrapped with Tyfo SEH-51A GFRP jackets). The curves ‘S7’ and ‘S8’ show the stress–strain relationships of Tests 7 and 8 (post-heated columns wrapped with Weber.tec force C-240 CFRP jackets). The curve ‘S9’ shows the stress–strain relationship of Test 9 (post-heated column wrapped with Tyfo SCH-41 CFRP jacket).

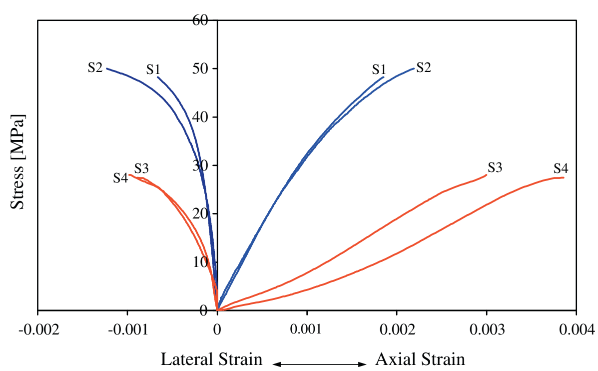

Fig. 8. Axial and lateral strains comparison of unheated/non-jacketed and post-heated/non-jacketed columns.

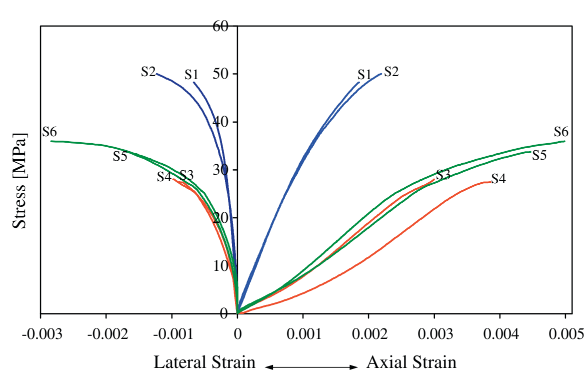

Fig. 9. Axial and lateral strains comparison of unheated/non-jacketed, post-heated/non-jacketed and post-heated columns wrapped with Tyfo SEH-51A GFRP jacket.

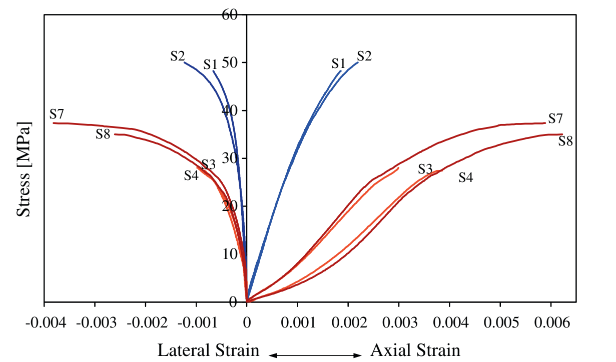

Fig. 10. Axial and lateral strains comparison of unheated/non-jacketed, post-heated/non-jacketed and post-heated columns wrapped with Weber.tec force C-240 CFRP jacket.

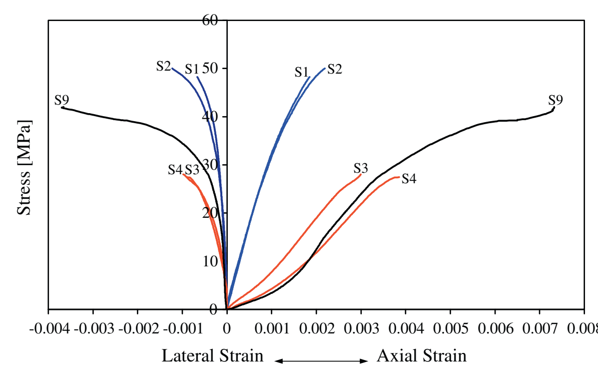

Fig. 11. Axial and lateral strains comparison of unheated/non-jacketed, post-heated/non-jacketed and post-heated columns wrapped with Tyfo SCH-41 CFRP jacket.

It can be seen from Fig. 8 that the values of axial strains at the ultimate loads were significantly lower in the unheated columns compared to the post-heated columns. This could be attributed to the micro-cracking and loss of stiffness of concrete after heating to 500 °C. Figs. 9–11 demonstrate that the post-heated columns wrapped with a single layer of Tyfo SEH-51A GFRP, Weber.tec force C-240 CFRP and Tyfo SCH-41 CFRP jacket withstood significant higher ultimate axial and lateral stains compared to the unheated and post-heated columns. In unheated and post-heated columns without FRP the strain in the lateral direction reached the ultimate tensile strain of the concrete at a relatively lower axial strain (Fig. 8). However, in the GFRP or CFRP wrapped post-heated columns the fibre reinforced polymer jackets had restrained this lateral movement and increased the capacity to withstand a much higher value of axial strain and lateral strain as shown in Figs. 9–11. The failure of GFRP or CFRP wrapped post-heated columns took

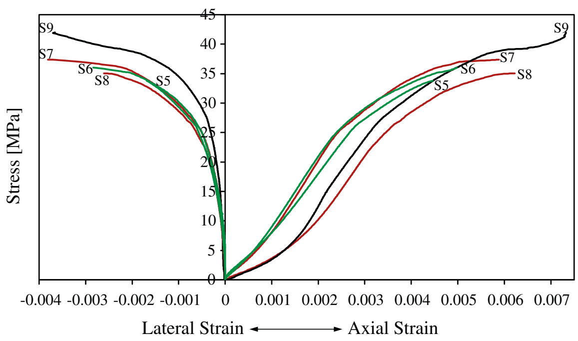

Fig. 12. Axial and lateral strains comparison of post-heated columns wrapped with Tyfo SCH-41 CFRP, Tyfo SEH-51A GFRP and Weber.tec force C-240 CFRP jackets.

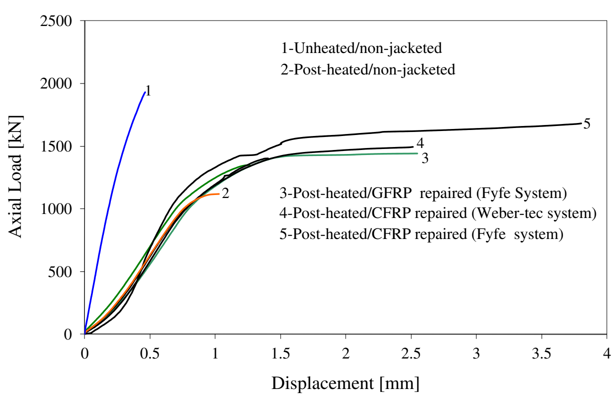

Fig. 13. Load–displacement comparisons of unheated and post-heated/non-jacketed columns and post-heated/jacketed columns.

place when the lateral strains exceeded the ultimate strain of the fibre or the crushing strain of concrete. At the same stress level, the increase in axial strain of the Tyfo SEH-51A GFRP, Weber.tec force C-240 CFRP and Tyfo SCH-41 CFRP wrapped post-heated columns is greater than the increase in transverse strain.

Fig. 12 compares the ultimate axial stress, ultimate lateral and axial strains of post-heated columns wrapped with GFRP or CFRP. It is interesting to note that the ultimate strains in the Tyfo SEH-51A GFRP wrapped post-heated columns were lower compared to the Tyfo SCH-41 CFRP and Weber.tec force C-240 CFRP wrapped post-heated columns, although the modulus of elasticity of the CFRP is higher compared to the GFRP. This indicates that, due to stress concentration at the corners, the full strength of the GFRP was not fully utilized and the failure of the GFRP jacket occurred prematurely. Comparing Tyfo SCH-41 CFRP and Weber.tec force C-240 CFRP wrapped post-heated columns, the ultimate strains in the Tyfo SCH-41 CFRP were higher than the Weber.tec force C-240 CFRP. The efficiency of Tyfo SCH-41 CFRP jackets in terms of ultimate strains was higher compared to the Tyfo SEH-51A GFRP and Weber.tec force C-240 CFRP jackets.

2.6.5. Effect of GFRP and CFRP jackets on ductility

The ductility of structural members is considered one of the most critical parameters for evaluating its performance. The concept of structural ductility is its ability to sustain inelastic deformation without loss in its load carrying capacity until failure [9]. The absolute definition of ductility is still a disputed issue and several methods exist to quantifying the ductility of structures [17]. However, based on the stress–strain response of unheated, post-heated and post-heated GFRP or CFRP wrapped columns, shown in Figs. 9–11, it is clear that confinement with GFRP or CFRP improves the ductility of columns. This could be attributed to the fibre reinforced polymer laminate providing confinement to the micro cracked post-heated concrete resulting in an increase in the column's load carrying capacity with a higher value of axial strain and lateral strain.

The ductility of the post-heated columns wrapped with Tyfo SCH-41 CFRP jackets is greater compared to the post-heated columns wrapped with Tyfo SEH-51A GFRP and Weber.tec force C-240 CFRP jackets, as shown in Fig. 12. The ductility of the post-heated Tyfo SEH-51A GFRP wrapped columns is lower than the ductility of the post-heated columns wrapped with Weber.tec force C-240 CFRP jackets. It is also observed that improvement in ductility due to confinement with GFRP or CFRP is more pronounced than the increase in ultimate load.

3. Conclusions

This paper presents an experimental study on the performance of post-heated axially loaded (one-third scale) reinforced concrete square columns repaired with a single layer of GFRP or CFRP jacket. Based on the results of the experimental investigation, the following conclusions can be drawn.

1. The axial load carrying capacity of square columns after exposure to 500 °C was reduced by up to 44%.
2. The glass and carbon fibre jackets failed at the corners of the square cross-section of the post-heated columns, due to the stress concentration in these regions.
3. The glass and carbon fibre reinforced polymer jackets increased both the strength and ductility of the post-heated square concrete columns under axial loading.
4. A single layer of GFRP or CFRP could restore, up to some extent, the load bearing capacity of square columns heated to 500 °C. However, the original strength of the unheated square columns was not restored using a single layer of GFRP or CFRP jacket. This could be due to the fact that the CFRP or GFRP jackets failed prematurely due to stress concentration at the corners of the square cross-section of the columns and the full strength of the GFRP or CFRP jackets could not be utilized to allow the original strength to be regained.
5. The concrete became more porous after heating and the stiffness of the post-heated columns was reduced significantly. However, the stiffness of post-heated columns did not improve by using a single layer of GFRP or CFRP jacket since the jacket confinement was only significantly activated after approaching the maximum strength of the repaired post-heated column. This was due to the initial axial compression of the post-heated column being accommodated by the micro-cracks and voids created during heating, with less lateral expansion in this initial load period compared to an unheated column. Once this initial compression took place, and the failure load approached, the column began to expand laterally and initiate the confining effects of the jacket.
6. Under axial compression, the lateral expansion in the post-heated concrete column was greater due to the heat induced damaged concrete being 'softer'. The failure of the columns was primarily due to lateral tensile strains. However, in the FRP wrapped post-heated columns the fibre reinforced polymer laminate resisted the lateral tension and consequently increased the column's capacity to carry a higher value of axial compressive load with more lateral strain.

On the basis of the results of the experimental tests, GFRP and CFRP jackets provide effective confinement near ultimate conditions and could be used to enhance the strength and ductility of fire damaged reinforced concrete square columns.

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