Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

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Notations

$\xi$ = Ratio between the residual compressive strength at a given temperature $T^\circ C$
and the original un-heated compressive strength of concrete at $20^\circ C$  

$f_{co}$ = Cylinder strength of concrete not exposed to fire

$f_r$ = Residual strength of concrete after fire

$f_c$ = Concrete stress

$\varepsilon_c$ = Concrete strain

$\varepsilon_o$ = Concrete strain corresponding to $f_r$

$f'_{ct}$ = Residual compressive strength of concrete after exposure to temperature $T$ (maximum temperature that concrete has been exposed to, before cooling).

$f'_c$ = Concrete compressive strength of the cylinder at ambient temperature ($20^\circ C$)

$f_{cu}$ = Cube compressive strength of concrete

$f_y$ = Yield strength of steel

$\Theta$ = Temperature

$\Theta_{max}$ = Maximum temperature

$\Theta_c$ = Concrete temperature

$\sigma$ = Stress

$\phi$ = Reduction co-efficient for concrete depending on the effect of thermal stresses in the fire situation.

$\varepsilon_{cu\theta}$ = Concrete strain in fire situation

$k_{c,\theta}$ = Reduction factor for strength of concrete depending upon temperature

a = shear span

d = effective depth.

FRP = Fibre reinforced polymer

CFRP = Carbon fibre reinforced polymer

GFRP = Glass fibre reinforced polymer

$k$ = Secant stiffness
$\Delta$ = Lateral displacement

L = Height of column between the point of application of lateral load and top level of column footing

$E_f$ = Modulus of elasticity of FRP

$f_{cc}'$ = Compressive strength of confined concrete

$f_l$ = Lateral confinement pressure

$f_f$ = Ultimate strain capacity of FRP

n = number of FRP layers

$k_a$ = Efficiency reduction factor

$t_f$ = Thickness of FRP layer

r = Radius of corner

$A_e$ = Effective confined area

$\varepsilon_f$ = FRP effective strain

$A_{ol}$ = Area of overlap of the parabolas
Abstract

In the light of extreme events of natural disasters (earthquakes or hurricanes) and accidents (fire or explosion), repairing and strengthening of existing concrete structures has become more common during the last decade due to the increasing knowledge and confidence in the use of composite advanced repairing materials. The past experience from real fires shows that it is exceptional for a concrete building to collapse as a result of fire and most fire-damaged concrete structures can be repaired economically rather than completely replacing or demolishing them. In this connection an experimental study was conducted to investigate the effectiveness of fibre reinforced polymer jackets for axial compressive and seismic shear performance of post-heated columns. This study also investigates the effectiveness of ferrocement laminate for the repairing of post-heated reinforced concrete columns.

A total of thirty-five reinforced concrete columns were constructed and then tested after categorising them into three main groups: un-heated, post-heated and post-heated repaired. The post-heated columns were initially damaged by heating (to a uniform temperature of 500°C). The concrete cubes were also heated to various temperatures to develop the relation between compressive strength and ultrasonic pulse velocity. The residual compressive strength of the concrete cubes and reinforced concrete columns were determined by ultrasonic testing. The post-heated columns were subsequently repaired with unidirectional glass or carbon fibre reinforced polymer and ferrocement jackets. The experimental programme was divided into two parts. The columns of experimental part-1 were tested under axial compressive loading. The columns of experimental part-2 with a shear span to depth ratio of 2.5 were tested under constant axial and reversed lateral cyclic loading.

The results indicated that the trend of reduction in ultrasonic pulse velocity values and in residual compressive strength of concrete was similar with increasing temperature. The reduction in residual stiffness of both post-heated square and circular columns was greater than the reduction in ultimate load. The circular sections benefited more compared to the square cross-sections with fibre reinforced polymers for improving the performance of post-heated columns in terms of compressive strength and ductility tested under axial compression. GFRP and CFRP jackets performed in an excellent way for increasing the shear capacity, lateral strength, ductility, energy dissipation and slowed the rate of strength and stiffness degradation of fire damaged reinforced concrete square and circular columns tested under combined constant axial and reversed lateral cycle loading. However, the effect of a single layer of glass or carbon fibre reinforced polymer on the axial stiffness of post-heated square and circular columns was negligible. The use of a ferrocement jacket for the repairing of post-heated square and circular columns enhanced the axial stiffness and ultimate load carrying capacity of columns significantly.
Declaration

No portion of the work referred to in the thesis has been submitted in support of an application for another degree or qualification of this or any other university or other institute of learning.

Muhammad Yaqub
(October, 2010)
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CHAPTER-1

1 INTRODUCTION

1.1 BACKGROUND

The repairing and strengthening of concrete structures has become extremely 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 due primarily to the concrete’s low thermal conductivity which results in the structure increasing in temperature at a slow rate during a fire. It is very rare for concrete structures to collapse during a fire and post-fire repair of concrete structures is typically an economical and viable option compared to demolition and reconstruction. The decision to repair, or demolish a building must be based on the economic considerations such as direct costs and time. Typically, the critical factor is time since the delay in repairing a building could result in serious indirect financial consequences to both the owners and insurers. Immediate reinstatement is generally required by the owners to save consequential loss of business due to the delay in the use of the building.

In fire damaged concrete structures, the knowledge of the residual properties of materials is required as a basis for the decision of reconstructing or repairing and for the economical design of the repair structure. For the evaluation of residual strength in existing fire damaged concrete structures, there are a number of in-situ and laboratory-based techniques available to diagnose the site condition of concrete. The in-situ techniques include visual inspection; non-destructive testing, taking samples of concrete and reinforcement for testing in the laboratory. The non destructive testing on site includes hammer soundings; rebound hammer (Schmidt hammer) and PUNDIT (Portable Ultrasonic Non-destructive Digital Indicating Tester). The rebound hammer test needs a flat surface to test and as a large number of tests are required
to reduce the effects of variability, it is not generally suitable for use on spalled surfaces, which is often the case with fire damaged concrete [1]. The core test provides the most reliable in-situ strength assessment but it is very risky to extract cores from existing reinforced concrete columns due to the possibility of cutting the main reinforcing bars during extracting cores from columns. Since the load carrying capacity of the whole structure mainly depends upon the strength of columns. Therefore extracting of cores direct from columns should be avoided.

The ultrasonic pulse velocity test is a well established and handy tool for practising engineers to evaluate the residual strength of reinforced concrete. Since the maximum pulse energy is transmitted at right angles to the face of the transmitter; therefore the direct method is the most reliable from the point of view of transit time measurements in concrete columns because it is easier to put transducers on opposite sides of columns during testing. However, there is no fundamental relationship between pulse velocity and the compressive strength of concrete. Therefore, it is necessary to obtain a direct relation between the residual compressive strength of fire damaged concrete and the ultrasonic pulse velocity to get reliable results of the residual compressive strength of fire damaged reinforced concrete. In this research project, a relationship between the ultrasonic pulse velocity and residual compressive strength of heat damaged concrete is investigated to predict the residual compressive strength of fire damaged concrete in existing reinforced concrete columns.

Various methods for repairing and strengthening of concrete structures, following a fire, have been used in the past [1, 2]. Unfortunately, all previous traditional available techniques for repairing and strengthening of concrete structures are associated with a time consuming and obstreperous process of removing and replacing the damaged materials with new and stronger materials. In recent years, fibre-reinforced polymer (FRP) has been recognized as a strengthening and repairing advanced technology especially in the speedy repair and strengthening measures for civil engineering infrastructures worldwide. In this advanced technology, reinforced concrete columns
can be easily and effectively be strengthened by wrapping layers of FRP around columns in existing conditions without any interruption to the use of the structure.

Crushing failure under gravity loading and brittle shear failure under reversal actions of earthquake forces, with coexisting axial loads, are the most common failure of reinforced concrete columns in buildings. In the past, a lot of research has been reported on repairing and strengthening for such a type of failure of reinforced concrete columns with fibre reinforced polymers (FRP) [3-111]. Unfortunately, due to the uncertainties regarding the behaviour of fibre reinforced polymer in any subsequent fire following repair, limited research has been reported on the repairing of fire damaged concrete columns [112-116]. However, it has been found that, with applied fire insulation, fibre reinforced polymer can perform well in fire [117-122]. The square and circular columns are commonly used in buildings and it is felt that the shape of the cross-section of the columns may affect the confinement effectiveness of externally bonded FRP jackets [43, 111]. This research study investigates the axial compressive and seismic shear performance of post-heated reinforced concrete square and circular columns (heated to a uniform temperature of 500°C) wrapped with a single layer of unidirectional glass or carbon fibre reinforced polymers.

It has been found that a ferrocement jacket can also provide an effective confinement in reinforced concrete columns [123-134] and therefore it has a great potential to be used as a strengthening material. The skill required for the fabrication of ferrocement is of low level and its constituents are typically locally available. Therefore ferrocement can be used as an ideal repairing technique for residential buildings in developing countries due to the availability of cheap labour. Although ferrocement is an old technology and has been used as a structural material for more than 50 years [135], its application in repairing of fire damaged reinforced concrete columns is a new one. This research study also investigates the performance of a ferrocement jacket for the repairing of post-heated reinforced concrete columns tested under axial compression.
1.2 AIMS, OBJECTIVES AND KEY HYPOTHESIS

The overall aim of the proposed research is to develop effective, economical and rapid repairing and strengthening methodology for reinforced concrete columns damaged by extreme loading.

The specific objectives of the research are:

1) To investigate the effectiveness of fibre reinforced polymers for the repairing of fire damaged reinforced concrete columns tested under axial compression.

2) To investigate the effectiveness of ferrocement for the repairing of fire damaged reinforced concrete columns tested under axial compression.

3) To investigate the repair effects of composite materials on the cross sectional shape of post-heated reinforced concrete columns.

4) To investigate the seismic shear performance of fire damaged reinforced concrete columns repaired with fibre reinforced polymers.

5) To evaluate the residual compressive strength of fire damaged reinforced concrete columns in existing conditions by non-destructive testing (Ultrasonic testing).

The objective of this study is to generate the required data for practising engineers who are involved in the field of repairing and strengthening of existing reinforced concrete structures damaged by extreme loading. This reported research work has considerable importance for the construction industry regarding repairing of fire damaged reinforced concrete columns with advanced composite materials.

1.3 BENEFITS

The incidences of natural disasters (earthquakes) and accidents (fire) are unfortunately quite frequent in the construction industry. The statistical survey indicates an upward trend of such incidences every year. The effects of these incidences when expressed in economical terms, the actual loss runs into astronomical figures. This research work contributes in the effective use of advanced composite materials for repairing and strengthening of infrastructure affected by such incidences.
1.4 ORGANIZATION OF CHAPTERS

The outline of the chapters incorporated within the thesis is summarized as below.

1.4.1 Chapter 2: Literature review
This chapter highlights the literature review relevant to the current research project.

1.4.2 Chapter 3: Material properties, construction and heating of specimens
This chapter describes the material properties used in the experimental work; casting and heating procedure of reinforced concrete columns.

1.4.3 Chapter 4: Repairing and testing procedure of columns
This chapter explains the various repairing techniques; instrumentation set up and testing procedure of post-heated columns.

1.4.4 Chapter 5: Results and analysis of residual compressive strength and ultrasonic testing
Experimental test results, analysis of residual compressive strength of concrete and ultrasonic testing are presented in Chapter-5.

1.4.5 Chapter 6: Analysis of results and discussion of axially loaded columns
This chapter describes the detailed analysis and discussion of the results obtained during the testing of axially loaded reinforced concrete columns (Experimental part-1).

1.4.6 Chapter 7: Analysis of results and discussion of critical seismic shear tested columns
Experimental test results, analysis and detailed discussions of reinforced concrete columns tested under combined constant axial and reversed lateral cyclic loading (Experimental part-2) are presented in Chapter-7.

1.4.7 Chapter 8: Conclusions and future recommendations
Finally, the conclusions of the whole research work and recommendations for future research are listed in Chapter-8.
CHAPTER-2

2 LITERATURE REVIEW

This chapter highlights some of the available literature relevant to the current study and is summarised as below.

2.1 EFFECTS OF FIRE ON CONCRETE STRUCTURES

Temperatures greater than 900°C are common in fires within buildings [136-139]. In terms of structural performance under fire conditions, concrete structures generally perform very well due primarily to the concrete’s low thermal conductivity which results in the structure increasing in temperature at a slow rate during a fire. Experience from real fires shows that it is rare for a concrete building to collapse as a result of fire and most fire-damaged concrete structures can be reinstated successfully. In fire damaged concrete structures, the knowledge of the residual properties of materials are required as a basis for the decision of its reinstatement. The reduction in load-bearing capacity of the concrete structural members in fire depends upon many factors [140-144] such as

1) Type and quality of reinforcement.
2) Type of aggregates.
3) Duration and the severity of fire exposure.
4) Conditions of testing: whether the concrete is tested in hot conditions or after cooling down; whether it is extinguished with water or allowed to cool down slowly in the furnace or in open air before testing.
5) Conditions of loading during heating of the specimens and whether the concrete specimens are restrained or unrestrained during heating.
6) The amount of moisture content present in the concrete specimens at the time of heating.
7) Shape and dimensions of the structural member.
8) Concrete cover and other protection of the reinforcement.
The mechanical properties of concrete such as compressive strength, tensile strength, Poisson’s ratio and modulus of elasticity change with the increase of temperature. These changes depend on the peak temperature, the rate of heating and cooling, the fire duration, the type of concrete and type of testing. Three types of test conditions, stressed, un-stressed, and un-stressed residual strength tests, have been used to investigate the performance of concrete in fire [145]. In a stressed test condition, a preload, normally in the range of 20 to 40 percent of the ultimate compressive strength at room temperature is applied to the concrete specimen prior to heating, and the load is maintained during the heating period. The specimen is heated continuously at a prescribed rate until a specified temperature is reached, and the temperature is maintained constant until a thermal steady state is achieved [145]. After achieving uniform temperature, the load is then increased at a prescribed rate until the specimen fails. In the un-stressed test method, the specimen is heated, without any loading, at a prescribed rate to the specified temperature, and is maintained constant until a thermal steady state is reached within the specimen. After achieving the target temperature, the specimen is loaded at a prescribed rate until failure occurs. The stressed and un-stressed test methods are performed to assess the properties of concrete at elevated temperature [145].

In the un-stressed residual strength test, the specimen is heated without any load at a prescribed heating rate. After achieving the target temperature, the temperature is kept constant until a thermal steady state is reached within the specimen. After heating, the specimen is then allowed to cool down to room temperature and loaded at room temperature up to the failure. This method is most suitable for assessing the post-fire or residual properties of concrete [145].

### 2.1.1 Residual compressive strength of concrete

Concrete strength decreases with increasing temperature and there is further decrease on cooling probably because of additional micro cracking. When concrete cools down, the quicklime (calcium oxide) absorbs moisture and converts to slaked lime (calcium hydroxide). When this happens, disintegration of the affected concrete will occur [146]. Generally, the residual strength of concrete remains approximately in
the range of 75% to 25% of the original strength when heated in the range of 300°C to 600°C respectively in most concrete structures [137-139, 145].

Many researchers investigated the effects of high temperatures on the compressive strength of concrete made with Ordinary Portland Cement tested at elevated temperatures and after cooling. Hertz [141] investigated the behaviour of siliceous, limestone, granite, sea gravel, pumice and expanded clay aggregate concrete at elevated temperatures and after cooling down. It was found that the relative residual compressive strength of heated siliceous concrete was about 20% smaller than the hot strength for temperatures above 400°C. Hertz [141] also investigated the behaviour of the granite, basalt, limestone, and sea gravel, which belongs to a group of aggregates that has lower thermal expansion and suffers less damage than siliceous aggregate concrete during fire. It was found that the reduction in compressive strength and the standard deviation for this reduction was nearly the same for granite, basalt, limestone, and sea gravel. Theconcretes made of aggregates (granite, basalt, limestone, and sea gravel) were tested considering one group called the main group by Hertz [141]. It was observed that the main group aggregate concrete showed higher values than siliceous aggregate concrete after cooling. It was also confirmed by Abrams [147] that, at temperatures above about 430°C, siliceous aggregate concrete lost a greater proportion of its strength than concretes made with limestone or lightweight aggregates but when the temperature reached approximately 800°C the difference disappeared [152].

Malhotra [142] tested concrete 50 mm × 100 mm cylinders using various mix proportions and water-cement ratios to determine the compressive strength. Malhotra [142] found that at temperatures less than 200°C, the reduction in compressive strength was small. At higher temperatures, the reductions were significant and depended on the test conditions. However, the reduction in compressive strength at 500°C for stressed and, unstressed specimens was about 35% and 60% of the unheated value respectively. The residual strength of unstressed specimens heated at 500°C and tested after cooling was reduced by up to about 80% of its initial strength.
It was observed that the reduction in compressive strength for concrete made with an aggregate-cement mix ratio 4.5:1 and water-cement ratio of 0.65 was greater than that of concrete made with an aggregate-cement mix ratio 6:1 and water-cement ratio of 0.5. It was concluded that the residual strength of the heated concrete was further reduced in compressive strength on cooling and was approximately 20% less than the corresponding hot strength in the temperature range of 200°C to 450°C for 1:4.5 and 1:6 mix concretes.

Lee et al.[143] tested cylinders to investigate the strength, stiffness and permeability of concrete heated to various temperatures and cooling scenarios. The results demonstrated that the residual strength and stiffness dropped rapidly when the temperature was increased above 400°C. At 600°C and 800°C the residual strength of concrete was found to be less than 57% and 18% of the original un-heated strength respectively.

Abrams [147] reported the results of compressive strength tests on 75 mm x 150 mm cylindrical specimens heated to temperatures of 93-871°C. The results demonstrated that the unstressed residual strength obtained after cooling showed the greater losses compared to unstressed and stressed strength tested at elevated temperature. The results of siliceous and expanded shale aggregate concretes showed that the specimens stressed during heating had smaller reductions in strength at high temperatures than unstressed specimens tested hot. This may be due to the reduction of crack formation owing to the imposed stress.

The siliceous aggregate concrete displayed about 75% of the original strength at 450°C in unstressed conditions. Above 450°C, the siliceous aggregate concrete had greater strength loss and retained less strength than that for the other types of concretes, in all three test conditions (unstressed residual tested at room temperature, unstressed and stressed tested at elevated temperature) due to the abrupt volume change that occurs because of the transformation of $\alpha - \beta$-quartz at about 573°C in the siliceous aggregates concrete. Compressive strengths of
specimens tested under stressed condition were generally 5 to 25% higher than those tested under unstressed conditions. The specimens tested under unstressed residual (tested at room temperature after heating) had the lowest strength compared with stressed and unstressed specimens. In all types of aggregates, it was found that the loss of compressive strength was independent of the original level of strength but the sequence of heating and loading had a significant affect on the residual strength.

The findings of Abrams [147] were very close to Malhotra [142], Saeman and Washa [148], and Harmathy and Berndth [149]. However, the reduction in strength up to 600°C, reported by Malhotra [142] was larger compared with Abrams [147]. The reduction in unstressed residual strengths of siliceous aggregates found by Zoldners [150] at 500°C was less compared to Abrams [147].

Nassif [151] reported the results of compressive strength tests on 75 mm diameter and 175 mm long concrete cores made with river aggregate, Ordinary Portland Cement and well graded marine sand. The results demonstrated that heating to temperatures less than 220°C had no significant effect on the compressive strength of the river gravel aggregate concrete. However, the residual strength of concrete was found to be 70% and 42% of the original value at temperatures of 320°C and 470°C respectively. The findings of Nassif [151] were very close to the Concrete Society Technical Report [137], CIB W14 report [138] and Hertz [141].

Chang et al. [153] tested cylinders of size 150 mm × 300 mm to investigate the effect of temperatures on the properties of concrete subjected to temperatures in the range of 100-800°C. It was observed that the residual strength at 200°C, 400°C, 600 °C and 800°C was found to be 90%, 65%, 40% and 15% of the original unheated value respectively. The findings of Chang [153] were very close to the results of Abrams [147] and BS EN 1994-1-2 [154].

Chan et al. [155] reported the results of compressive strength tests on 100 mm cubes heated to temperatures of 20-1200°C to investigate the effect of temperature on the
residual compressive strength of concrete. The results showed that, there were three temperatures ranges (20-400°C, 400-800°C, and 800-1200°C) in which the loss of compressive strength was noticeably different. Chan et al. [155] found that up to a temperature of 400°C, the loss in strength was 25% and severe strength loss occurred within the range of 400-800°C, due to dehydration of C-S-H gel.

Mohamedbhai [156] investigated the effect of varying the time of exposure and the rates of heating and cooling on the residual compressive strength of basalt aggregate concrete cubes (100 mm size). The cubes were heated to temperatures in the range of 200-800°C. The results demonstrated that the range of residual compressive strengths of concrete at temperatures, 20°C, 200°C, 400°C, 600°C and 800°C were 100, 50-92, 45-83, 38-69 and 20-36% respectively.

Xiao et al.[157] made a review of previous research on the mechanical behaviour of concrete both under and after high temperature exposure. The results illustrated that under high temperature up to 400°C, the compressive strength of concrete decreased somewhat at the start and then increased a little which could be considered as constant. After 400°C, the compressive strength dropped rapidly. When the temperature reached 800°C, the residual compressive strength was approximately 20% of that of room temperature.

Noumowe et al.[158] tested concrete cylinders at different values of high temperatures of two concretes (ordinary and high strength) made with calcareous aggregates. The change in the residual compressive strength up to 200°C temperature was small. Between 200°C to 350°C, the residual compressive strength was nearly constant and retained more than 80% of the initial strength. After 350°C, there was rapid drop in strength. At 600°C, the percentage residual compressive strengths were 38.6% and 46.5% for normal (NSC) and high strength concrete (HSC) respectively.
Lea [159] investigated the effect of high temperature on the residual strength of concrete made with gravel and sand. The results demonstrated that up to 450°C, the residual unstressed strength was not affected significantly but above 450°C, it dropped sharply due to loss of bond between the aggregate and the cement paste. The unstressed residual strength was lower than the unstressed strength. The specimens tested at high temperature were 10% stronger than those heated to the same temperature and allowed to cool down before the test.

Lankard et al.[160] reported the results of compressive strength tests carried out on two concretes made with gravel aggregates and with limestone aggregate, at temperatures up to 260°C. The results demonstrated that limestone and gravel aggregate concretes behaved similarly when heated up to about 260°C. The unsealed unstressed gravel aggregate concrete showed higher compressive strength than sealed unstressed gravel aggregate concrete when tested in both the hot and cold conditions. For both unsealed gravel and limestone aggregate concretes, there was an increase in the hot and cold strengths after heating at about 80°C and 120°C respectively. Specimens heated to above 190°C and cooled to room temperature before testing generally showed losses of strength in the range of 10 to 20%.

Kaplan et al.[161] conducted tests on unsealed concrete specimens made with siliceous aggregates and reported that for temperatures below 150°C the strength after cooling was greater than that tested while hot, whereas for temperatures above 150°C the hot specimens were stronger than cooled ones. The compressive strength of specimens tested at 400°C while hot was up to 20% less than the original unheated values, whereas after cooling to room temperature the specimens showed up to 40% reduction in compressive strength.

Khoury et al.[162] reported the results of compressive strengths of concretes made with limestone, basalt, siliceous and light weight aggregates. It was observed that after heating to 600°C, the residual strengths of the unstressed specimens were about 20, 30, and 40% for limestone, lightweight, and basalt aggregate respectively.
It was found that the siliceous gravel aggregate concrete at 600°C was so badly damaged and it could not be tested.

Harada et al.[163] investigated the compressive strength, elasticity and thermal properties of concrete during and after heating at various temperatures. The results showed that the residual compressive strengths of concrete were 80%, 75% and 60% of the original value at 100°C, 300°C, and 450°C respectively. The findings of Harada [163] were very close to Abrams [147], Chang [153] and BS EN 1994-1-2 [154].

Purkiss [164] reported the results of variation of residual compressive strength of concrete with temperature. It was found that the percentage loss in compressive strength of concrete quoted by Purkiss [164] was lower compared to Malhotra [142] and Abrams [147].

The Concrete Society is very active in producing comprehensive updates to assess the strength of fire damage concrete and repairing of fire damaged concrete structures. The Concrete Society TR15 [136] suggested a strength reduction curve based on the research of unstressed residual strength of concrete, as shown in Fig. 1. It has been found that the strength of the loaded specimens is generally higher than those of unloaded specimens during heating [141, 147]. Therefore the unstressed residual strength test is considered more conservative for assessing the post-fire or residual properties of concrete [141, 145, 147, 164]. In reality, all concrete structures would be stressed, at least under dead load, at the time of heating. Therefore, the Concrete Society had made some modifications in the TR33 report [137] and suggested the strength reduction curve shown in Fig. 2 based on the residual strength of heated stressed concrete after cooling.

Recently, the Concrete Society published a report TR68 [1] in which it is assumed that the concrete heated to above 300°C has lost all of its strength. It has been found that the normal grey colour of Ordinary Portland Cement Concrete turns to light pink
at around 300°C [165]. Therefore the 300°C temperature is considered as the boundary for the pink colour and it identifies the limit of damaged concrete. Since the change of colour is not evident for all types of aggregates. Therefore, the concrete exposed to more than 300°C should be removed prior to repairing. For the sake of assessment of fire damaged concrete structure on the safer side, the Concrete Society TR68 [1] assumed the zero strength of concrete above 300°C, as shown in Fig. 3. It is worth mentioning here that the colour changes shown in Figs. 1 to 2 are applicable for siliceous aggregate and may not be applicable for calcareous or crushed flint aggregate. Marchant [2] reported that concrete retains 50% of its compressive strength after exposing to temperature 500°C, as shown in Fig. 4.

Fig.1: Residual strength of heated un-stressed dense aggregate concrete after cooling [136]
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 2: Residual strength of heated stressed dense aggregate concrete after cooling [137]

Fig. 3: Residual compressive strength of concrete after cooling [1]
The variation in residual compressive strength of concrete reported by various researchers depends on many factors which contributed in the strength loss. Some of them are described in the following sections.

### 2.1.1.1 Effect of methods of cooling on the compressive strength of concrete

The relative residual compressive strength of concrete after various cooling regimes decreases monotonically. After cooling, the material takes time to come to equilibrium with its surroundings [141]. It is found that the compressive strength measured immediately after cooling is somewhat lower than that at high temperature [141, 142, 144, 147, 152, 159, 164] and there is further post-cooling variations due to re-absorption of moisture to the concrete [141, 144]. The post-cooling changes in strength that occurs as a concrete element approaches equilibrium with its surroundings are accompanied by dimensional changes within a structure. These changes would cause further redistribution of stress and possibly additional damage during the period immediately after fire [144].
The strength of a fire damaged concrete structure should be investigated after one week of fire, since the compressive strength is reduced a further 20% after cooling [141, 147]. The loss of strength of concrete with water cooling below 400°C is more than that of cooling in air. However above 600°C, the effect of cooling regime (either water or air) on the compressive strength is not significant [156]. It is also found that the damage of carbonated concrete heated above 500°C becomes more when cooled with water [157]. Under the same condition of cooling with water, the strength damage is even more with a sharper drop of the temperature (e.g. cold water) since some cement grains may be hydrated with the water [157]. However, the residual strength of concrete with cooling in the air continues to decrease for a period and then some recovery would occur slowly [143]. When cooling is carried out in the furnace, the strength loss of concrete is less as compared to concrete cooled in open air and in water [143, 157].

The application of water in fire causes more reduction in compressive strength due to setting up of larger temperature gradients in the concrete [152]. The cooling of heated concrete with tap water for five minutes reduces about 10% more the compressive strength of hot siliceous concrete [151]. Zoldners [150] found that at temperatures up to 500°C, the specimens that were cooled by quenching in water for 5 min had much lower compressive strength than the specimens cooled down slowly overnight in the closed furnace.

2.1.1.2 Effect of rate of heating and cooling on the compressive strength of concrete

The higher the rate of heating and cooling, the greater would be the reduction in residual compressive strength of concrete [141, 142, 150, 162, 166]. The residual strength is smaller for small temperatures at rapidly heating rate because the matrix does not have the same time to creep when the aggregate expands and the matrix shrinks [141]. The matrix therefore appears to be more brittle giving more micro cracks, when the specimen is heated rapidly. On the other hand for higher temperatures, the rapidly heated concrete is stronger because it takes time for the
calcium hydroxide to decompose and damage the concrete. More rapid heating usually occurs near the surface of a structure [141].

The rates of heating and cooling have a significant effect on the residual strength of concrete heated to temperatures lower than 600°C. However, the rates of heating and cooling have no significant effect on the residual strength of concrete heated to 600°C and above [156] because most of the micro cracking has been taken place and most of the moisture has been removed up to 600°C and above. If the heated concrete is allowed to cool down faster, the reformation of calcium hydroxide will take place at a slow rate in the days after cooling, and the concrete reaches its minimum strength several days after the fire. The speed of this regeneration process depends upon the moisture content of the ambient air and the size of the structure [141].

2.1.1.3 Effect of loading on the compressive strength of concrete

Under a combination of load and heat, which is likely the case during actual fire, the concrete retains a high percentage of its compressive strength. The presence of compressive stresses in stressed specimens reduces the growth of cracks [140, 147]. It was observed that the strength of the loaded specimens was generally 5 to 25% higher than those of unloaded specimens during heating [141, 147]. The strengths of specimens stressed in compression during heating were not significantly affected by the applied stress level, which ranged from 25 to 55% of the original strength [147]. The effect of loading should be considered only when the compressive load is known throughout the fire exposure period of the concrete, otherwise it is more conservative to use the values obtained from tests of unloaded specimens instead of the transient values of the compressive strength [141, 164].

2.1.1.4 Effect of shape and size of specimens on the compressive strength of concrete

The shape and size of the specimens may affect the compressive strength of Portland Cement Concrete at high temperatures. Cubes have greater residual strength than prisms [167]. The temperature at the centre of the larger size specimen will be lower than those of smaller size specimens at the same time due to the delay
of the heat transfer. Consequently, the loss of strength would be higher in the smaller size concrete specimen than in the larger size specimens when exposed to same fire duration [167-169]. However, the effect of specimen size on the retained compressive strength of concrete is not manifested when heated uniformly [170].

2.1.1.5 Effect of long term exposure to high temperature on the compressive strength of concrete

The longer the period of exposure to high temperatures, the greater would be the deterioration in compressive strength due to crack generation and material decay. Most of the reduction occurs within the first 30 days of long term exposure [152, 156, 160, 163, 171-174]. The residual strengths after one hour of exposure at 200, 400, 600 and 800°C were about 80, 70, 60, and 30% respectively while the residual strengths after 2 hours or more exposure were found to be about 70, 60, 45, and 25% [156]. At 300°C, residual compressive strength of about 65% was found after two days and 50% at the end of four months [174].

2.1.2 General expressions for residual compressive strength of fire damaged concrete

A simple expression for deterioration of concrete proposed by Hertz [141] is given below

\[
\xi = \frac{1}{1 + \frac{T}{T_1} + \left(\frac{T}{T_2}\right)^2 + \left(\frac{T}{T_8}\right)^8 + \left(\frac{T}{T_{64}}\right)^{64}}
\]

Where \(\xi\) is the ratio between the residual compressive strength at a given temperature \(T\) °C and the original un-heated compressive strength of concrete at 20°C. The parameters \(T_1, T_2, T_8, T_{64}\) with the unit °C are given in Table 1.
The residual compressive strength of concrete can be estimated using the relation given in BS EN 1994-1-2 [154] described in Section 2.1.6. The equation for estimating the residual compressive strength of unstressed concrete reported by Lie et al. [175] can be used to calculate the residual strength of fire damaged concrete. This equation is also confirmed analytically and experimentally by Lin et al. [176]. The expression for concrete after exposure to fire proposed by Lie et al. [175] is given below.

\[ f'_r = f'_{co} (1 - 0.001T) \text{ for } 0°C \leq T \leq 500°C \]  
\[ f'_r = (1.375 - 0.00175T)f'_{co} \text{ for } 500°C < T \leq 700°C \]  
\[ f'_r = 0 \text{ for } T > 700°C \]  

The stress-strain relationship for concrete after exposure to fire may be represented by the following [176]:

\[ f'_c = f'_{c0} \left[ 1 - \left( \frac{\varepsilon'_c - \varepsilon'_o}{\varepsilon'_o} \right)^2 \right] \text{ for } \varepsilon'_c \leq \varepsilon'_o \]  
\[ f'_c = f'_{c0} \left[ 1 - \left( \frac{\varepsilon'_c - \varepsilon'_o}{3\varepsilon'_o} \right)^2 \right] \text{ for } \varepsilon'_c > \varepsilon'_o \]  

In which,
\[ \varepsilon'_o = 0.0025 + (6.0T + 0.04T^2) \times 10^{-6} \]  

<table>
<thead>
<tr>
<th>Type of Aggregates and Conditions of Testing</th>
<th>( T_1 )</th>
<th>( T_2 )</th>
<th>( T_8 )</th>
<th>( T_{64} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Siliceous concrete (Hot)</td>
<td>15000</td>
<td>800</td>
<td>570</td>
<td>100,000</td>
</tr>
<tr>
<td>Siliceous concrete (Cold)</td>
<td>3500</td>
<td>600</td>
<td>480</td>
<td>680</td>
</tr>
<tr>
<td>Main Group aggregate Concrete (Hot)</td>
<td>100,000</td>
<td>1080</td>
<td>690</td>
<td>1000</td>
</tr>
<tr>
<td>Main Group aggregate Concrete (Cold)</td>
<td>10,000</td>
<td>780</td>
<td>490</td>
<td>100,000</td>
</tr>
<tr>
<td>Light aggregate concrete (Hot)</td>
<td>100,000</td>
<td>1100</td>
<td>800</td>
<td>940</td>
</tr>
<tr>
<td>Light aggregate concrete (Cold)</td>
<td>4000</td>
<td>650</td>
<td>830</td>
<td>930</td>
</tr>
</tbody>
</table>

Table 1: Parameters for concrete while hot and in cold condition [141]
Where

\[ f'_{ct} = \text{Cylinder strength of concrete not exposed to fire} \]
\[ f_r = \text{Residual strength of concrete after fire} \]
\[ f_c = \text{Concrete stress} \]
\[ \varepsilon_c = \text{Concrete strain} \]
\[ \varepsilon_o = \text{Concrete strain corresponding to } f_r \]
\[ T = \text{Highest temperature attained by the concrete} \]

Chang et al. [153] suggested the following temperature-dependent residual compressive strength equation for unstressed concrete.

\[ \frac{f'_{ct}}{f'_{c}} = 1.01 - 0.00055T \quad \text{for} \quad 20^\circ C < T \leq 200^\circ C \]  
\[ \frac{f'_{ct}}{f'_{c}} = 1.15 - 0.00125T \quad \text{for} \quad 200^\circ C < T \leq 800^\circ C \]  

Where

\[ f'_{ct} \] is the residual compressive strength of concrete after exposure to temperature
\[ T \] is maximum temperature that concrete has been exposed to, before cooling.
\[ f'_{c} \] is the concrete compressive strength at ambient temperature (20°C).

The findings of Chang [153] were very close to the results of Abrams [147] and BS EN1994-1-2 [154]. The curves for the residual compressive strength of concrete after fire reported by Marchant [2] and Concrete Society Reports TR15 [136] & TR33 [137] could also be helpful in predicting the residual strength of concrete exposed to various temperatures. None of the above relations could predict the exact value of residual strength. However, the combination of all could give a reasonable prediction about the residual strength of fire damaged concrete.

### 2.1.3 Effect of temperature on the tensile strength of concrete

The tensile or flexural strength of concrete made with Ordinary Portland Cement and conventional aggregates decreases when exposed to high temperatures [166]. At high temperature, the sensitivity of tensile strength of concrete is higher than the compressive strength. For the same heating temperature the reduction in tensile
strength is more than that in compressive strength [177, 178]. Therefore, the relative reduction in tensile strength of concrete is higher than the compressive strength at high temperature [150, 163, 179, 180].

The type of aggregate has a considerable effect on the tensile strength of concrete when exposed to high temperatures. The performance of concretes made with siliceous aggregates at high temperature is better than the concrete made with calcareous aggregates [163, 180]. However, the findings reported by Zoldners [150] were inconsistent with Harada et al.[163] and Schneider [180]. It was observed that concrete with smaller aggregate-cement ratio, had a greater reduction in tensile strength than concrete with high aggregate-cement ratios [142, 181]. It is worth noting that the residual tensile strength of concrete after cooling to room temperature is less compared to concrete tested in hot conditions [180, 181]. The results of the tensile strength of concrete are not considerably affected by the rate of heating [180]. The longer the period of exposure to temperatures higher than 150°C, the greater would be the reduction in tensile strength of concrete [163, 172].

2.1.4 Effect of temperature on the modulus of elasticity of concrete

In the evaluation of deformation of fire-damaged concrete structures, it is important to consider the effect of temperature on the elastic modulus of the concrete since concrete is softening with increasing temperature [153]. Concrete is normally made of carbonate, siliceous and lightweight aggregates. In all the three concretes, the modulus of elasticity is affected as the temperature increases [140]. The type of aggregates had a significant effect on the elastic modulus with increasing temperature [140]. The modulus of elasticity of the high strength concrete is decreased by 5 to 10 % when exposed to temperatures in the range of 100 to 300°C. At 800°C, the modulus of elasticity was only 20 to 25% of the value at room temperature [182]. The rate of heating affects the elastic modulus and compressive strength in a similar manner. The faster the heating, the lower would be the elastic modulus due to higher thermal stresses, higher pore pressure, higher stresses from thermal shrinkage, and greater damage to the microstructure [183].
For the siliceous aggregate concrete heated to 400°C, the value of elastic modulus after cooling was the same as that prevailing at the highest temperature [184]. The same finding was quoted by Harada [183] that the value of residual elastic modulus (after cooling) showed a similar drop with maximum temperature. This indicated that the elastic modulus experiences a permanent reduction in its value [174] due to the change of microstructure and bonding with increasing temperature. The reduction in elastic modulus is more than that in the compressive strength when exposed to fire [153]. The original strength has no significant effect on the elastic modulus after heating to various temperatures [153]. It was found that the values of elastic modulus at 200°C, 400°C, and 600°C were about 80%, 40% and 6% of the original unheated value respectively [153].

Nassif [151] reported the results of the residual initial elastic modulus strength carried out on 75 mm diameter and 175 mm long concrete cores made with siliceous aggregate concrete, which were subjected to different heating and cooling regimes in the temperature range of 217-470°C. Some cores were heated to basic heat exposure in which the cores were heated to a point where both the outer surface and the centre of the core reached the same temperature. At this point, the furnace was switched off and the cores were taken out to a controlled environment of 20°C where they were left for cooling. Some cores were further exposed to temperature after achieving the point of uniformity of temperature at both the outer surface and the centre of cores and were allowed to cool down in air in a controlled environment. The other specimens were sprayed with tap water for five minutes after getting the same temperature at the outer surface and in the centre and were left for cooling in the same environment. The results demonstrated that the residual stiffness of concrete soaked at the maximum temperature for 2 hours was less than that concrete subjected to basic heat treatment and cooled by quenching.

2.1.5 Effect of temperature on the Poisson ratio of concrete

The Poisson ratio of concrete varies with temperature in a manner similar to elastic modulus [185]. The variation in the Poisson ratio was observed from about 0.2 at
room temperature to about 0.1 at 400°C [186] due to the weakening of the micro structure resulting from rupture of bonds at high temperature.

### 2.1.6 Residual stress-strain relationship

For the evaluation and repairing of fire-damaged concrete structures, it is important to consider the stress-strain behaviour of post fire damaged concrete structures in order to envisage the whole structural behaviour in a future earthquake. The post-fire residual stress-strain relationship of fire-damaged concrete is a complex function of the method of cooling and duration of exposure as well as the maximum temperature [151]. The shape of the stress-strain curves of concrete does not change after heating. However, the peak in the curve is reduced due to induced degradation and occurs at a higher strain [144, 174]. Moreover, the slope of the descending portion of stress-strain curves is reduced with increasing temperature [144].

The compressive strength \( f_c \) of concrete will not recover to its initial value when it is heated to maximum temperature of \( \theta_{\text{max}} \), and allowed to cool down at ambient temperature [154]. BS EN1994-1-2 [154], considers the same peak strain for concrete during heating and cooling. Therefore, in the descending branch of the concrete heating curve, as shown in Fig. 5, the value of \( \varepsilon_{\text{cu}}, \theta \) and the value of the slope of the descending branch of the stress-strain relationship may both be maintained equal to the corresponding values for \( \theta_{\text{max}} \) as shown in Fig. 6.

The following relation gives the residual compressive strength of concrete heated to a maximum temperature \( \theta_{\text{max}} \) and having cooled down to the ambient temperature of 20°C.

\[
f_{c, \theta, 20^\circ C} = \varphi f_c'
\]

(10)

Where for

\[
20^\circ C \leq \theta_{\text{max}} < 100^\circ C; \quad \varphi = K_c \cdot \theta_{\text{max}}
\]

(11)

\[
100^\circ C \leq \theta_{\text{max}} < 300^\circ C; \quad \varphi = 1.0 - \left[0.235(\theta_{\text{max}} - 100)/200\right]
\]

(12)
\[ \theta_{\text{max}} \geq 300^\circ C; \varphi = 0.90 K_{c,\theta_{\text{max}}} \] Where \[ K_{c,\theta_{\text{max}}} = \frac{f'_{c,\theta}}{f'_c} \] (13)

During the cooling down of concrete with \( \theta_{\text{max}} \leq \theta \geq 20^\circ C \), the corresponding compressive cylinder strength \( f'_{c,\theta} \) may be interpolated linearly between \( f'_{c,\theta_{\text{max}}} \) and \( f'_{c,\theta,20^\circ C} \).

For any lower temperature obtained during the subsequent cooling phase as for \( \theta_3 \),

\[ f'_{c,\theta,20^\circ C} = (0.90 \times K_{c,\theta_{\text{max}}}) f'_c \] (14)

\[ f'_{c,\theta} = f'_{c,\theta_{\text{max}}} - \left[ f'_{c,\theta_{\text{max}}} - f'_{c,\theta_{20^\circ C}} \right] \left( \frac{\theta_{\text{max}} - \theta_3}{\theta_{\text{max}} - 20} \right) \] (15)

\[ \varepsilon_{cu,\theta_3} = \varepsilon_{cu,\theta_{\text{max}}} \]

\[ \varepsilon_{ce,\theta_3} = \varepsilon_{cu,\theta_3} + \left( \varepsilon_{ce,\theta_{\text{max}}} - \varepsilon_{cu,\theta_{\text{max}}} \right) \frac{f'_{c,\theta_3}}{f'_{c,\theta_{\text{max}}}} \] (16)

**Fig.5:** Heating and cooling of concrete [154]
However, the residual peak strain reported by Chang. et al.[153] for normal strength concrete made with siliceous aggregates was lower than BS EN1994-1-2 [154].

![Stress-Strain relationships of concrete](image)

**Fig. 6:** Stress-Strain relationships of the concrete strength class C40/C50 heated up to $\theta^\circ C, \theta_2^\circ C, \theta_{\text{max}}^\circ C$ and cooled down to $\theta_3^\circ C$ [154]

Many researchers investigated the stress-strain relationships of concrete during and after heating to high temperatures. Nassif [151] tested concrete cores to investigate the stress-strain relationships of river gravel concrete and limestone concrete. The results demonstrated that river gravel heated concrete after cooling with water showed a brittle failure mode below 320°C and exhibited gradual failure above 320°C due to increasing the internal plasticity, as shown in Fig. 7. On the other hand, in limestone concrete the brittle failure was observed only in un-heated cores while heated limestone concrete after cooling with water displayed a gradual type of failure, as shown in Fig. 8.
Fig. 7: Post fire full stress-strain relationship of quenched fire-damaged river gravel concrete [151]

Fig. 8: Post fire full stress-strain relationship of quenched fire-damaged limestone concrete [151]
Harada et al. [163] tested concrete cylinders having 50 mm diameter and 100 mm height. All specimens were heated in an electric furnace at a rate of 1.5°C/min and allowed to cool down gradually. The results demonstrated that the gradient of the curves becomes gradual with rise in temperature. The curve is convex in the upward direction up to 400°C and becomes concave at 500°C, as shown in Fig. 9. The residual strain increases with increasing temperature. At 500°C, the residual strain is more than 50% of the total strain.

Fig. 9: Residual stress-strain curves of sand stone aggregate concrete [163]
Castillo et al.[182] investigated the effects of transient high temperature on the load deformation behaviour of normal strength (27.6 Mpa) and high strength (62.1 Mpa) concrete. The concrete mix used composed of Ordinary Portland Cement, natural river sand and crushed limestone. The specimens were 51 mm x 102 mm cylinders. The maximum aggregate size used was 9.5 mm. The heating rate for all specimens was between 7 and 8°C/min. The results showed that for both the normal and high strength concretes, strain at peak load did not vary significantly within the temperature range of 100 to 200°C. Between 300 to 400°C, the strain corresponding to the peak load increased slightly. However, at temperatures ranging between 500 to 800°C, the strain at peak load increased significantly. At 800°C, the strain at peak load was three to four times the strain at room temperature, as shown in Figs. 10 and 11.

Fig. 10: Load deformation behaviour of normal strength concrete at high temperatures [182]
Diederichs et al. [187] performed unstressed tests on normal and high strength concrete made with Ordinary Portland Cement. The results demonstrated that high strength concrete failed in a more brittle manner than normal strength concrete, as shown in Figs. 12 and 13.

Fig. 11: Load deformation behaviour of high strength concrete at high temperatures [182]
Fig. 12: Stress-strain relationships of normal strength concrete [187].

Fig. 13: Stress-strain relationships of high strength concrete [187].
Furumura et al.\[188\] reported the results of stress-strain relationships of normal strength and high strength concretes. The results demonstrated that the stress-strain curves of high strength concrete (FR60) showed a steeper slope than the normal strength concretes (FR42, FR21) at 300°C, as shown in Fig. 14 (a). The 42 MPa concrete shows flatter slope above 300°C, as shown in Fig. 14 (b).

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{stress_strain_graph.png}
\caption{Comparison of stress-strain relationships for concretes at 300°C [188]}
\end{figure}
Felicetti et al. [189] conducted unstressed residual strength tests on high strength concretes (72 MPa and 95 MPa) made with siliceous aggregates. The specimens were heated to temperatures 105, 250, 400, and 500°C. All specimens were heated at a rate of 12°C/min. It was noted that the mode of failure was brittle when heated up to 250°C and after that it was more gradual. The peak in the curves is reduced due to induced degradation and occurs at a higher strain, as shown in Figs. 15 and 16. It was observed that the peak strain in high strength concrete was greater than in the normal strength concrete, as shown in Figs. 17 and 18.

Fig. 14 (b): Typical stress-strain relationships for 42 MPa concrete at different temperatures [188]
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 15: Residual stress strain curves for 72 MPa after heating to different temperatures[189]

Fig. 16: Residual stress strain curves for 95 MPa after heating to different temperatures[189]
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 17: Stress-strain curves of high strength concrete [190]

Fig. 18: Stress-strain curves of normal strength concrete [143]
2.1.7 Spalling

The spalling of concrete structures in fire is the violent or non-violent breaking off of layers or pieces of concrete from the surface of structural elements [191-194]. Spalling can occur in buildings in the form of aggregate spalling, surface spalling, corner spalling and explosive spalling [191-194]. All forms of spalling can cause unfavourable damage and may reduce the fire resistance of concrete structure [194, 195]. However, explosive spalling has very serious structural consequences compared to aggregate, surface and corner spalling. This phenomenon is unpredictable and can seriously affect the integrity and stability of concrete structures [195]. It may cause sudden and complete failure of the concrete structural members due to significant loss of section [194]. There are many factors which contribute to explosive spalling such as concrete age, heating rate, type and size of aggregate, moisture content, permeability, curing methods, concrete strength, cracking, concrete cover, thermal restraint and stress levels. However, at the present time explosive spalling, or the stochastic nature of spalling, is not fully understood [194]. It is considered that the combination of thermal stresses and pore pressure stresses have a key role in increasing the strain energy within the gel pores of the concrete which contribute to this phenomenon [194, 195]. The risk of explosive spalling can be minimized by using polypropylene fibres [191, 194]. The polypropylene fibres melt at 160°C and form many channels through which pore pressure is released in the form of vapours [194].

2.1.8 Residual strength of reinforcing steel

The effect of high temperature on the yield strength of typical reinforcing steel bars while hot and after cooling is shown in Figs. 19 and 20. Significant loss of strength occurs when the reinforcing steel is at high temperature and it is generally responsible for any excessive residual deflection. The yield stress reduces at temperatures above 300°C and retains 50% of the original yield strength at 550°C [1, 137], as shown in Fig. 19. The reduction in yield strength will occur further with increasing temperatures above 550°C.
The original yield stress after cooling is almost completely recovered from temperatures up to 450°C for cold worked steel and 600°C for hot rolled steel, as shown in Fig. 20. However, above these temperatures, there will be a loss in yield strength after cooling [137]. On cooling from 800°C, the yield stress is reduced by 30% for cold worked bars and 5% for hot rolled bars [144]. It is believed that the temperature rise of the reinforcing bars in the sides of the concrete columns is considerably lower than for those at the corners. This could be attributed to the fact that the concrete cover to the side bars remains in position longer than that over the corner bars [196] because the corners of columns heat up quicker. The actual loss in strength depends on the heating conditions and type of steel. However, the conservative values given in Fig. 20, for temperatures up to 700°C, would be sufficient for most purposes [137].

2.1.9 A simple expression for residual strength of fire damaged reinforcing steel

A simple expression for deterioration of reinforcing steel proposed by Hertz [141, 197, 198] is given below

\[
\xi = K + \frac{1 - K}{1 + \frac{T}{T_1} + \left(\frac{T}{T_2}\right)^2 + \left(\frac{T}{T_8}\right)^8 + \left(\frac{T}{T_{64}}\right)^{64}}
\]

Where \(\xi\) is the ratio between the residual tensile strength of reinforcing steel at a given temperature \(T\) °C and the original un-heated tensile strength of reinforcing steel at 20°C. The parameters \(T_1, T_2, T_8, T_{64}\) with the unit °C and \(K\) are given in the Table 2
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
<thead>
<tr>
<th>Type of steel and conditions of testing</th>
<th>$K$</th>
<th>$T_1$</th>
<th>$T_2$</th>
<th>$T_8$</th>
<th>$T_{64}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hot rolled bars, 0.2% stress</td>
<td>0.00</td>
<td>6000</td>
<td>620</td>
<td>565</td>
<td>11,00</td>
</tr>
<tr>
<td>Hot rolled bars, 2.0% stress</td>
<td>0.00</td>
<td>100,000</td>
<td>100,000</td>
<td>593</td>
<td>100,000</td>
</tr>
<tr>
<td>Hot rolled bars, 0.2% residual stress</td>
<td>1.0</td>
<td>100,000</td>
<td>100,000</td>
<td>100,000</td>
<td>100,000</td>
</tr>
<tr>
<td>Hot rolled bars, 2.0% residual stress</td>
<td>1.0</td>
<td>100,000</td>
<td>100,000</td>
<td>100,000</td>
<td>100,000</td>
</tr>
<tr>
<td>Cold-worked bars, 0.2% stress</td>
<td>0.00</td>
<td>100,000</td>
<td>900</td>
<td>555</td>
<td>100,000</td>
</tr>
<tr>
<td>Cold-worked bars, 2.0% stress</td>
<td>0.00</td>
<td>100,000</td>
<td>5000</td>
<td>560</td>
<td>100,000</td>
</tr>
<tr>
<td>Cold-worked bars, 0.2% residual stress</td>
<td>0.58</td>
<td>100,000</td>
<td>5000</td>
<td>590</td>
<td>730</td>
</tr>
<tr>
<td>Cold-worked bars, 2.0% residual stress</td>
<td>0.52</td>
<td>100,000</td>
<td>1500</td>
<td>580</td>
<td>650</td>
</tr>
</tbody>
</table>

Table 2: Parameters for reinforcing steel tested in cold condition [197]

Fig. 19: Yield strength of steels tested at elevated temperatures [138]
2.1.10 Effect of temperature on bond strength of concrete and reinforcing steel

Bond strength between reinforcing bars and concrete deteriorates when reinforcing concrete structural members are exposed to high temperature. When concrete is subjected to high temperature, the reduction in bond strength is higher compared to the reduction in the compressive strength and tensile strengths of concrete [179, 199-201]. The residual bond strength is not only dependent on the temperature reached during heating but also upon the test procedure and the shape of bar [138]. However, the percentage reduction in bond strength for deformed bars is less compared to plain round bars, as shown in Fig. 21. It was also observed that the reduction in bond strength in rusted plain round bars is lower compared to the fresh rolled bars with increasing temperatures [179, 200, 202].

The diameter of plain and deformed bars had little effect on the percentage reduction in bond strength [200, 202]. However, the type of aggregate has a significant effect
on the percentage reduction in bond strength at elevated temperature [169, 203]. It was found that the specimens tested under hot conditions experienced less reduction in bond strength compared to those tested after cooling [204]. The results of bond strength in stressed specimens are different from un-stressed specimens during heating [172, 200]. It was also found that the concrete structural members having smaller concrete cover experienced more reduction in bond strength than the structural members having more concrete cover [179].

The bond strength in reinforced concrete structures is generally not a problem even though bond strength is severely reduced in a fire [164]. The problem is more likely to be worse in pre-stressed concrete construction where the bond in the anchorage length is needed to transfer the pre-stress force to concrete [164]. However, a conservative damage factor for bond of 0.70 may be applied to reinforcement within the 100°C to 300°C zone [138]. A value of 0.80 might be considered for lower temperatures, small diameter bars (12mm or less) and concrete of lower compressive strength (25-30N/mm²) containing calcareous or lightweight aggregates or for reinforcement contained by stirrups [138].

![Graph showing bond strength ratio vs temperature for ribbed and plain round bars](image_url)
2.2 ASSESSMENT OF FIRE DAMAGED CONCRETE STRUCTURE

In general concrete structures are non-combustible and resist fires for a quite long period. However, long time exposure to fire results in deterioration of compressive strength and degradation of reinforcing steel which may cause loss of service ability or collapse of the structure. The concrete structure undergoes a number of changes when subjected to fire [137]. On heating it shows:

1) Surface crazing due to dehydration of cement paste.
2) Cracking and spalling due to heat transferred to the interior concrete.
3) Reduction in residual compressive strength of concrete due to cracking and spalling.
4) Reduction in yield strength of reinforcement due to rise in temperature.
5) Possible deflection of structural members due to heat transferred to the reinforcement (accelerated if spalling occurs) and expansion of aggregate particles resulting in significant levels of differential strain.

On cooling after fire:

1) Cracks close up and concrete could recover its original strength when subjected to temperature less than 300°C [141].
2) The strength loss is permanent when micro cracks appeared in the concrete [141].
3) The deflection recovery remains incomplete for severe fire and further deformations and cracking may result due to absorption of moisture from the atmosphere [1].
4) Reinforcing bars recover their original yield strength when subjected to temperatures less than 450°C [137] and any buckled bars remains buckled after cooling of the reinforcement.

After a fire, the concrete structure should be evaluated and the following deterioration should be assessed before carrying out any repair work [1].

1) Depth of damage (spalling) or loss of strength of concrete.
2) Loss in strength of reinforcement or embedded structural steel elements.
3) Damage or distress to the structure from movement, settlement or imposed loads.
The aim of an assessment of a fire damaged concrete structure is to propose suitable repair methods or to decide whether demolition of elements or the whole structure is required.

2.2.1 Stages in the assessment of fire damaged concrete structures

2.2.1.1 Health and safety

The first duty of the structural engineer is to consider the safety before going to start the actual work for the assessment of the fire damaged structure. Apart from the load carrying capacity of individual structural members, the overall stability of the building must be considered, since one weak member can, under certain circumstances, affect the safety of the building as a whole [2]. The following steps should be kept in mind while carrying out the preliminary inspection [1, 2].

1) The safety of the structure and the public safety should be considered at all assessment stages, from the initial phase to the final repair stage.
2) The beams and slabs which are in critical condition should be propped with temporary bracing.
3) Unexpected weakness may exist in suspended floors where the heat has caused most damage to the soffit, leaving the top surface apparently solid. Infill panels in ribbed floors may be weakened leaving them brittle and vulnerable to sudden collapse under foot traffic or other loading.
4) Repairing materials and equipment should be placed at suitable places, taking into account any additional loads that may apply to the weakened structure.
5) Temporary false work may be urgently required to secure the individual members and the stability of the structure as a whole.
6) Special care is required to avoid the transfer of excessive loads and stresses to other members especially where false work is being used to relieve a column at an intermediate floor level. Relieving false work may have to be carried through to foundation level to avoid creating excessive stresses in adjoining parts of the structure.
2.2.1.2 ASSESSMENT OF DAMAGE

There are two approaches which could be used for the assessment of fire damaged concrete structures separately or combined depending on the nature of the fire and type of the structure.

2.2.1.2.1 Method-1

The first methodology involves three steps.

1) Estimation of maximum temperature reached in fire and the duration of fire.
2) Heat transfer analysis.
3) Assessment of residual strength of the concrete and reinforcement.

- **Estimation of maximum temperature reached in fire and duration of fire**

There are many factors which contributed in the reduction of strength of concrete structures during fire. However, the primary effect of fire on concrete structures depends on the peak temperature reached during fire and the duration of fire [205]. The maximum temperature and duration of a fire are dependent on the fire load, ventilation conditions, geometry and material properties of the compartment. After making sure that the structure is safe to enter after fire, all possible clues to the history of the fire should be carefully recorded before debris is removed. In a real fire situation, the severity of fire is not uniform throughout the building and may have remained localized for a long time; the rate of temperature rise may have been faster, or slower, than in the standard test, or extensive spread may have occurred [206]. Different parts of the building may have reached different peaks of temperatures due to different fire intensities. Therefore, the building should be divided into horizontal and vertical zones so that the severity of fire can be assessed in detail for each zone. There are four feasible approaches to assess the severity of fire and the duration of fire [164].

The first approach is to use fire brigade records and the evidence of eyewitnesses which gives an indication about the times of starting and extinguishing of fire and the effort to control the fire. It gives only a qualitative judgment but not the quantitative assessment, as it provides the information whether the fire was small or large, more
or less damaging, of long or short duration and which particular areas had higher
temperatures than others. The second approach is to examine the maximum
temperature of the debris reached during fire. In buildings there is range of metallic
and non metallic materials, each of which has a different temperature at which it
suffers physical or chemical changes [207]. An examination of the debris may not
give an accurate picture of the temperature of the fire as it is subjected to local
fluctuations [1, 206]. However, Table 3 gives an approximate guide to the
assessment of temperature reached by selected materials and various components
in building fires [1, 206]. This gives only clue about maximum particular temperatures
but it does not give the direct indication regarding the actual temperature and the
total duration of exposure to that temperature.

<table>
<thead>
<tr>
<th>Substance</th>
<th>Typical Examples</th>
<th>Conditions</th>
<th>Approximate Temperature [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Paint</td>
<td></td>
<td>Deteriorates Destroyed</td>
<td>100</td>
</tr>
<tr>
<td>Polystyrene</td>
<td>Thin-wall food containers, foam, light shades, handles, curtain hooks, radio casings</td>
<td>Collapse, Softens Melts and flows</td>
<td>120, 120-140, 150-180</td>
</tr>
<tr>
<td>Polyethylene</td>
<td>Bags, films, bottles, buckets, pipes</td>
<td>Shrivels Softens and melts</td>
<td>120, 120-140</td>
</tr>
<tr>
<td>Polymethyl methacrylate</td>
<td>Handles, covers, skylights, glazing</td>
<td>Soften Bubbles</td>
<td>130-200, 250</td>
</tr>
<tr>
<td>PVC</td>
<td>Cables, pipes, ducts, linings, profiles, handles, knobs, house ware, toys, bottles</td>
<td>Degrades Fumes Browns Charring</td>
<td>100, 150, 200, 400-500</td>
</tr>
<tr>
<td>Cellulose</td>
<td>Wood, paper, cotton</td>
<td>Darkens</td>
<td>200-300</td>
</tr>
</tbody>
</table>
### Table 3: Melting temperatures for various materials [1]

<table>
<thead>
<tr>
<th>Substance</th>
<th>Typical Examples</th>
<th>Conditions</th>
<th>Approximate Temperature [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood</td>
<td>Ignites</td>
<td>240</td>
<td></td>
</tr>
<tr>
<td>Solder lead</td>
<td>Plumber</td>
<td>Melts</td>
<td>250</td>
</tr>
<tr>
<td></td>
<td>Joints, plumbing, sanitary installations, toys</td>
<td>Melts, sharp edges rounded drop formation</td>
<td>300-350</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>350-400</td>
</tr>
<tr>
<td>Zinc</td>
<td>Sanitary installations, gutters downpipes</td>
<td>Drop formations</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Melts</td>
<td>420</td>
</tr>
<tr>
<td>Aluminum and alloys</td>
<td>Fixtures, casings, brackets, small mechanical parts</td>
<td>Softens</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Melts</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drop formation</td>
<td>650</td>
</tr>
<tr>
<td>Glass</td>
<td>Glazing, bottles</td>
<td>Softens, sharp edges rounded</td>
<td>500-600</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flowing easily, Viscous</td>
<td>800</td>
</tr>
<tr>
<td>Silver</td>
<td>Jewellery, spoons, cutlery</td>
<td>Melts</td>
<td>900</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drop formation</td>
<td>950</td>
</tr>
<tr>
<td>Brass</td>
<td>Locks, taps, door handles, clasps</td>
<td>Melt(particularly edges)</td>
<td>900-1000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drop formation</td>
<td>950-1050</td>
</tr>
<tr>
<td>Bronze</td>
<td>Windows, fittings, doorbells, ornamentation</td>
<td>Edges rounded</td>
<td>900</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drop formation</td>
<td>900-1000</td>
</tr>
<tr>
<td>Copper</td>
<td>Wiring, cables, ornaments</td>
<td>Melts</td>
<td>1000-1100</td>
</tr>
<tr>
<td>Cast iron</td>
<td>Radiators, pipes</td>
<td>Melts</td>
<td>1100-1200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Drop formation</td>
<td>1150-1250</td>
</tr>
</tbody>
</table>
In the third approach, the duration and severity of the fire may be obtained from the depth of charred timber that has remained in place throughout the fire. For all practical purposes, the timber will char at a constant rate on each face in the standard furnace tests. As a rough guide, the char increases at a rate of 40mm per hour in the standard fire test. The information is less relevant if the timber had not remained in place and has fallen to the ground with other debris [1]. Attention should be given to the species of timber, position of timber and the nature of fire load. The timber fixed at higher levels in a room for instance would have been exposed to more severe conditions as compared to near the floor [207]. The rates which are given in Table 4 are based on Section 4.1 of BS:5268 [208] and allow an assessment to be made in terms of an equivalent fire resistance time.

<table>
<thead>
<tr>
<th>Timber</th>
<th>Depth of charring in 30 minutes</th>
<th>Depth of charring in 60 minutes</th>
</tr>
</thead>
<tbody>
<tr>
<td>All structural species except those listed below</td>
<td>20 mm</td>
<td>40 mm</td>
</tr>
<tr>
<td>Western red cedar</td>
<td>25 mm</td>
<td>50 mm</td>
</tr>
<tr>
<td>Hardwoods having a nominal density not less than 650kg/m³ at 18% moisture content</td>
<td>15 mm</td>
<td>30 mm</td>
</tr>
</tbody>
</table>

Table 4: Charring rate of wood [208]

The fourth method is to examine the colour of concrete. The colour of concrete changes during heating and it is irreversible. Therefore colouration of concrete at various depths allows an estimation of maximum temperature attained and the equivalent fire duration [136, 138, 165, 206]. The normal grey colour of Ordinary Portland Cement Concrete changes to light pink at around 300°C and becomes darker attaining the maximum intensity at about 600°C [207]. The temperature of 300°C is significant for three reasons. Firstly, a pink coloration occurs at this temperature [1, 136, 137]. Secondly, below that temperature the effect of heat on concrete strength is likely to be structurally insignificant [139, 147]. Thirdly, above
300°C, a pink discolouration indicates the inception of significant loss of strength due to heating [1]. The colour change to pink tends to be more prominent with siliceous aggregates. Calcareous and igneous crushed rock aggregates are less susceptible to this effect [137].

The change in colour is due to the transformation of ferric compounds present in aggregate or in the sand as impurities to ferric oxide [207]. However, in some cases these salts are not present in concrete. Therefore, concrete which does not show pink colour after heating, is not necessarily undamaged by fire [1]. The intensity of colour depends upon the level of impurity and the colour changes have been noticed even with limestone aggregate concrete when river sand is used as fine aggregate [207]. After removal of pink coloured concrete it may be assumed that remaining concrete has an average strength not less than 80% of the strength before fire [139]. Fig.22 shows the changes in colour of concrete at different temperatures [206].

To examine the change in colour, a piece of concrete from the damaged section should be removed, including the exposed surface in order to establish the spectrum of colour changes. The furthest depth at which pink colour can be seen may be taken as the boundary for the 300°C isotherm [207]. Where it is difficult to assess the depth of the pink layer, a small diameter core can be extracted from existing fire damaged portion of concrete in order to examine it accurately. Measurement of the extreme depth of discolouration enables an estimate to be made of the fire severity in terms of the equivalent duration of standard fire test.

The fifth approach is based on the equations proposed by Lie [209] and BS EN 1991-1-2 [210] for determining equivalent fire resistance. Estimations have to be made for each and every compartment, or floor, depending on the layout of building. This estimation needs the size of the compartment, size of openings (ventilation factor) and the fire load density. Since these equations based on the assumption that the whole fire load is burnt without any interruption and the whole ventilation is available from the start of the fire. Therefore, the result of these equations may not be totally
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

accurate for a large compartment fire. Practically none of the above approaches gives completely reliable results but the combination of all approaches gives a reasonable prediction about the maximum temperature and the duration of fire.

- **Heat transfer analysis**

There is no simple method of estimating the residual strength if the assessment of the distribution of temperature within the cross-section of structural member is not possible, because the loss of strength depends on time and temperature. There are three approaches to predict the distribution of temperature within the cross-section of structural members. Firstly, to use the charts for temperature distributions in dense concrete elements based on standard fire tests given in BS EN1992 Part 1-2 [211]. Secondly to use simple formulae proposed by Wickstrom [212]. The third one, which is considered a more realistic approach, is to estimate temperature time distributions within the cross-section of structural members using finite element methods for heat transfer analysis.

![Colour changes](image)

**Fig. 22: Colour changes in concrete [206]**

- **Assessment of residual strength of concrete and reinforcing steel**

The residual strength of concrete can be estimated by applying residual strength and temperature relationships reported by Hertz [141], Chang.et al.[153], BS EN 1994-1-2 [154], Concrete Society [1, 136, 137], CIB W14 report [138], Marchant.E.W [2]. The
residual strength of reinforcing steel can be estimated by applying residual strength and temperature relationships reported by Hertz [141] and the Concrete Society [1, 136, 137]. To determine the average damage in concrete structures, the structural members can be divided into inner, intermediate, and outer zones according to the temperature distribution, as shown in Fig. 23. The average damage factors may then be applied, taking a factor of 1.0 for all concrete subjected to temperatures less than 100°C (inner zone) [138].

The damaging factor of 0.85 is for concrete, which is heated to in the range of 100°C to 300°C temperatures (intermediate zone) [138]. A damage factor of 0.40 is for concrete within the 300°C to 400°C zone. A damage factor of 0.50 is for concrete within the 400°C to 500°C zone [2]. A damage factor of 0.20 for concrete within the 500°C to 600°C. At temperatures between 600°C to 900°C concrete becomes weak and friable and may be subjected to 100 percent strength loss. However, for the sake of assessment of fire damaged concrete structure on the safer side, the Concrete Society TR68 [1] assumed the zero strength of concrete above 300°C [outer zone]. After the removal of pink coloured concrete (above 300°C) it may be assumed that the remaining concrete has an average strength not less than 80% of the strength before fire.
Method-2 involves the in-situ testing of the fire-damaged reinforced concrete structures in order to assess the concrete quality and the residual strength of reinforcement. The following steps are carried out to test fire damaged concrete structures:

**a) Preliminary investigation**
1) Visual inspection of structure
2) Estimation of severity and duration of fire

**a) Field Testing**
1) Non-destructive testing.
2) Coring, sampling of reinforcement and subsequent laboratory testing
3) Load testing

**a) Preliminary investigation**

**1) Visual inspection of structure**

Aftermath of fire events, it is essential first to ensure the safe entry to those parts of the building that require inspection. The inspection of fire damaged concrete usually starts with visual observations of the structural members in affected areas for physical appearance. The building should be divided into three zones, serious damage, minor damage and un-damage [1]. Since the stiffness of fire damaged structural members is reduced due to removal of moisture and micro cracking. The load is redistributed in a fire damaged building according to the stiffness’s of members. Therefore, it is important to extend the inspection to any part of structure which is not directly damaged by the fire because it is possible that a substantial redistribution of forces can occur into the unaffected part of the structure. Apart from the load bearing capacity of individual structural members, the overall stability of the building must be considered, as one weak member can, under certain circumstances, affect the safety of the building as a whole [2].

Visual inspection may be aided by the use of magnifying hand lens (typically $\times 10$ magnifications) or field microscope (typically $\times 50$ magnifications) [213]. Inspection of cracking, spalling, deflections, surface crazing, colour changes, smoke damage and exposed steel reinforcement provide valuable information. Distortion and misalignment of vertical load bearing members due to expansion must be carefully examined. Misalignment of columns indicates that secondary moments are induced due to the $P-\Delta$ effects. Excessive deflection, large extensive cracks, misalignment and distortion of members shows that the load carrying capacities are reduced significantly and it is important to give more concentration to that zone of fire damaged structure [214]. Various measurements of geometry, deflections, and deformations of fire affected members should be taken and compared with undamaged members of the same structure to get more clear ideas. Visual inspection of reinforced concrete structures is performed to assign each structural
member a class of damage ranging from zero to four suggested by the Concrete Society [1], as shown in the Table 5

2) Estimation of severity and duration of fire

The second phase of preliminary investigation involves the examination of debris found at the scene as described in Section 2.2.1.2.1

b) Field testing

When it becomes clear from the preliminary investigation that a detailed investigation must be required then it is more realistic to specify a procedure to test the structure. The field testing can vary from simple hammer testing to costly load testing in serious, minor and un-damaged zones of concrete structure. The number of tests, as well as the methodology is a compromise between accuracy and effort, cost and damage, the availability of equipment and the budget, the time available and other constraints. Engineering judgment is thus required to determine the number and location of tests. Samples from fire affected concrete and unaffected concrete should be taken for comparison to become more confident. Health and safety must be considered at all the times, particularly when extracting core samples from critical locations. Care must be taken not to cause more damage than the fire [1].

1) Non-destructive testing

The non-destructive test methods most commonly used for the assessment of fire-damaged concrete structures could include soundness test, rebound hammer test and the ultrasonic pulse velocity test.

a) Soundness test

Sound tests can be used as a screening test to locate areas where detailed investigation is required. It may be sufficient to take soundings on the damaged concrete to determine the degree of deterioration. When concrete is struck with a hammer good materials will give the impression of being solid or hard, whereas damaged materials will sound hollow and the dull thud of weak material is readily distinguished, and this test may be successfully done with hammer and chisel [1]. Drilling resistance test can be used as an alternative of soundings with a hammer
and chisel to determine the depth of weakened concrete. A screwdriver or chisel is also useful to investigate surface areas for softened spots due to fire.

<table>
<thead>
<tr>
<th>Class of damage</th>
<th>Repair classification</th>
<th>Repair requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 0</td>
<td>Decoration</td>
<td>Redecoration</td>
</tr>
<tr>
<td>Class 1</td>
<td>Superficial</td>
<td>Superficial repair of slight damage</td>
</tr>
<tr>
<td>Class 2</td>
<td>General repair</td>
<td>Minor structural repair to restore the concrete cover which has been partly lost</td>
</tr>
<tr>
<td>Class 3</td>
<td>Principle repair</td>
<td>Strengthening repair reinforced according to the load carrying requirement of the member. Concrete and reinforcement strength may be significantly reduced requiring check by design procedure</td>
</tr>
<tr>
<td>Class 4</td>
<td>Major repair</td>
<td>Major strengthening repair with original concrete and reinforcement assessed down to have little or no strength or demolition and replacement</td>
</tr>
</tbody>
</table>

Table 5: Damage classification after fire [137]

b) Rebound hammer test

The Swiss Engineer Ernst Schmidt first developed a practicable rebound hammer test in the late 1940s. The test instrument consists of a metal housing, a spring loaded mass (the hammer) and a steel rod (the plunger). To perform a test, the plunger is placed perpendicular to the concrete surface and the housing is pushed towards the concrete. This action causes the extension of a spring connected to the hammer. When the instrument is pushed to its limit, the hammer is propelled towards the concrete and it impacts a shoulder on the plunger. There is no direct relationship between the measurement of surface hardness and strength. As flat surface is needed to perform this test and large number of tests are worthwhile to reduce the
effects of variability, therefore the rebound hammer is not in general suitable for use on spalled surfaces, which is commonly the case with fire-damaged concrete [1]. The results of this test on fire damaged concrete, even on flat surfaces, are fairly variable [1]. However, the apparatus is frequently available and testing procedure is given in BS EN 12504 [215].

c) Ultrasonic pulse velocity method

The test equipment includes a transmitter, a receiver and electronic instrumentation. The test consists of measuring the time it takes for a pulse of vibration energy to travel through a concrete member. The vibration energy is introduced into the concrete by the transmitting transducer, which is attached to the surface with petroleum jelly. The distance between the transducers is divided by the transit time to obtain the pulse velocity through the concrete. The pulse velocity is proportional to the square root of the elastic modulus and inversely proportional to the mass density of the concrete. The elastic modulus of concrete has been found to vary in proportion to the square root of the compressive strength. The pulse velocity depends on a number of factors such as aggregate type and moisture content. In addition, the presence of reinforcing steel aligned with the pulse travel path can also significantly increase the pulse velocity. There is no fundamental relationship between pulse velocity and strength. The testing procedure is given BS EN 12504-4 [215].

2) Coring, sampling of reinforcement and subsequent laboratory testing

The most direct method for examination and compression testing of in-situ concrete is by testing cores extracting from fire damaged and undamaged concrete. This is a well-established method, facilitating visual inspection of the interior regions of a member to be coupled with strength examination. The other physical properties such as density, water absorption, and indirect tensile strength can be measured from the extracted cores. Cores are also normally used as samples for chemical analysis before going to strength testing. The penetration of heat can be studied on core samples through the examination of colour changes of concrete in the sample. The procedure for core testing is given in BS EN 12390-3 [216]. Large cores should not be taken from positions where they would cause a significant loss in structural
strength [1]. The main purpose of core testing is to determine the original strength of undamaged concrete and to determine the strength of fire damaged concrete.

To determine the residual strength of reinforcing steel bars, samples of reinforcing steel bars should be taken from the fire exposed or seriously spalled areas of concrete where reinforcing steel has apparently reached elevated temperature [217]. The strength of reinforcing steel bars in fire exposed areas may reduce significantly. To determine the extent of damage, rebar samples should also be taken from adjoining areas where concrete seems to be sound. All samples should be sent to the laboratory for tensile, yield and elongation testing. The extent of damage and residual strength can be estimated by comparing the results of rebar from undamaged areas with the rebars in fire damaged areas [217].

3) Load testing

The structural soundness before the reinstatement and after reinstatement of fire damaged concrete structures should be evaluated by load testing. Although the indirect information about the structural capability of individual structural members is known, it may still become necessary to make sure the actual loading capacity of an integrated system of structural members after repair. For this an actual load test must be performed to check the satisfactory performance by measurement of deflection under the load, which may be sustained for a specified period. The bays and beams which exhibited serious spalling and their adjacent bays should be preferred for load testing of fire damaged reinforced concrete structures. The procedure for performing the load test is described in BS 8110 -2 [218].

2.3 REPAIRING AND STRENGTHENING OF FIRE DAMAGED CONCRETE STRUCTURES

Repairing and strengthening of existing concrete structures has become more common during the last decade due to the increasing knowledge and confidence in the use of advanced repairing materials, as well as the economical and environmental benefits of repairing or strengthening of structures compared to demolition and rebuilding. In terms of fire, concrete structures usually offer good fire
resistance due primarily to concrete’s low thermal conductivity which results in the temperature within structural members, away from the exposed face, remaining low during a fire. Experience from real fires shows that it is rare for a concrete building to collapse as a result of fire and most fire-damaged concrete structures can be successfully repaired. After a fire, owners and insurers demand reliable, economical and rapid repair techniques in order to avoid financial losses due to the delay in the use of the building.

Fibre reinforced polymer (FRP) has been recognized as a strengthening and repairing advanced technology for civil engineering infrastructures worldwide. The main application of this technology is the use of fibre reinforced polymer jackets in building and bridge columns. All previous traditional available techniques for repairing and strengthening of concrete columns are associated with a time consuming and obstreperous process of removing and replacing the damaged materials with new and stronger materials. In this advanced technology, concrete columns can be easily and effectively be strengthened by wrapping layers of FRP around columns in existing conditions without any interruption to the use of the structure. To date the main thrust of research in the area of FRP has been focused on strengthening and repairing of columns damaged other than due to fires [3-111]. Unfortunately, due to the uncertainties regarding the behaviour of fibre reinforced polymer in any subsequent fire following repair, limited research has been reported on the repairing of fire damaged concrete columns [112-116]. However, it has been found that, by incorporating suitable fire protection measures into FRP wrapped structural system, fibre reinforced polymers can perform well in fire [117-122].

Ferrocement is a composite material consisting of rich cement sand mortar and reinforcing thin wire mesh. The existing literature review [123-134] has shown that the ferrocement provides an effective confinement in concrete columns and has a great potential to be used as a strengthening material in developing countries. Since the skill required for the fabrication of ferrocement is of low level and its constituents are locally available. Therefore ferrocement can be an ideal repairing technique for
residential building in developing countries due to the availability of cheap labour. Although ferrocement is an old technology and has been used as a structural material for more than 50 years [135] its application in repairing fire damaged columns is a new one.

2.3.1 Fibre reinforced polymers (FRP)

Fibre reinforced polymers are the matrix of polymeric material reinforced by fibres. The use of fibre-reinforced composite materials in civil engineering structures has increased at a very rapid rate in the recent years. These high performance materials have unique properties and numerous advantageous over conventional construction materials which make them extremely attractive for structural applications. FRP composites have excellent corrosion resistance, high strength to weight ratio, excellent fatigue strength and creep/relaxation performance and satisfactory chemical resistance [219]. They are electronically neutral and allow easy handling and installation. Fibre reinforced polymers are now recognized as an effective and efficient construction material.

Fibre reinforced polymers (FRP) are typically made of carbon fibres, glass fibres and aramid. Moreover, as the fibre types (glass, carbon, aramid) and fibre volumes can be combined in numerous ways with a large variety of matrices, their overall mechanical properties can be modified to provide optimum solutions to a wide range of structural problems [219]. The various shapes and types of fibre reinforced polymers are shown in Fig. 24. One area where the use of fibre reinforced composites has attracted considerable interest is in the strengthening and repairing of reinforced concrete structures. Indeed, fibre reinforced composites are expected to become the materials of choice in the future. The cost of fibre is more than mild steel. However, when cost comparisons are made on strength basis and life cycles and other than materials costs are taken into account (transportation, handling, labour, obstruction of occupancy, etc.), fibre reinforced polymers are quite often cost-effective [219].
2.3.2 Basic components of fibre reinforced polymers

A fibre reinforced polymer is composed of fibres and polymer matrix [219], as shown in Fig. 25

2.3.2.1 Fibres

The fibres provide the strength and stiffness in a fibre reinforced polymer and fibres reinforced polymers are much stronger and stiffer in the fibre direction. The fibres are selected on the following basis [219].

1) High stiffness
2) High ultimate strength
3) Low variation of strength between individual fibres
4) Stability durability
5) Uniform diameter
The following three fibres are commonly used in civil engineering applications

2.3.2.1.1 Carbon fibre

Carbon fibre is a polymer which is a form of graphite. Graphite is a form of pure carbon. In graphite the carbon atoms are arranged into big sheets of hexagonal aromatic rings. The sheets look like chicken wire. The carbon fibres are available in the following classes based on their elastic modulus [219].

1) Standard 250-300 GPa
2) Intermediate, 300-350 GPa
3) High, 350-550 GPa
4) Ultra-high, 550-1000 GPa

Carbon fibres are used as structural fibre reinforced polymer wraps for repair and strengthening of columns, beams and slabs. The use of carbon fibre is increasing day by day due to their steadily decreasing cost, their high elastic modulus and available strength, their low density and their outstanding resistance to thermal,
chemical and environmental effects [219]. The choice of carbon fibre is ideal for strengthening and repairing of structures which are sensitive to weight and deflection [219].

2.3.2.1.2 Glass fibre

Glass fibre is a material made from extremely fine fibres of glass. Glass fibres are the most economical fibres commonly used in structural applications. Glass fibres are available in several grades. The most commonly used grades of glass fibres are E-glass, R-glass, AR-glass fibres. Glass fibres have high strength, moderate modulus of elasticity, moderate density and low thermal conductivity. Glass fibres are commonly used for structures which are not critical for weight and deflections [219].

2.3.2.1.3 Aramid fibre (kevlar)

Aramid fibres are family ofnylons and are manufactured from synthetic compound called aromatic polyamide. Aramid fibres are commonly available in two grades 60MPa and 120MPa. Aramid fibres have high strength, high stiffness, toughness, thermal stability, low density and moderate elastic modulus. Aramid fibres are vulnerable to ultra violent rays and moisture absorption [219].

2.3.2.2 Matrix

The matrix is the binder of the fibre reinforced polymer. The matrix is used to bind the fibres together and to protect the fibres from abrasion and from environmental effects. The matrix is used to transfer forces between the individual fibres [219]. A polymer matrix is an organic compound of long chain molecules and is composed of smaller repeated units called monomers.

2.3.3 Quantitative and qualitative comparison of FRP

The quantitative and qualitative comparison of fibre reinforced polymers is shown in the Tables 6 and 7 [219].
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>Ultimate strength (MPa)</th>
<th>Elastic modulus (GPa)</th>
<th>Failure strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glass-FRP</td>
<td>517-1207</td>
<td>30-55</td>
<td>2.4-5.5%</td>
</tr>
<tr>
<td>Carbon- FRP</td>
<td>1200-2410</td>
<td>147-165</td>
<td>1.1-5%</td>
</tr>
<tr>
<td>Aramid -FRP</td>
<td>1200-2068</td>
<td>50-74</td>
<td>2.2-6%</td>
</tr>
<tr>
<td>Steel</td>
<td>483-690</td>
<td>200</td>
<td>&gt;10%</td>
</tr>
</tbody>
</table>

Table 7: Quantitative Comparison of FRP [219]

2.3.4 Ferrocement

Ferrocement is a type of reinforced concrete having closely spaced multiple layers of reinforcing wire mesh completely embedded in mortar. The wire meshes are available in hexagonal or square openings. Meshes with square openings are mostly preferred in ferrocement construction due to higher efficiency. Meshes with square openings are available in welded or woven form. The ranges of mix proportions recommended for common ferrocement construction are: sand-cement ratio by weight, 1.5 to 2.5, and water-cement ratio by weight, 0.35 to 0.5 [133]. Normally, the aggregate consists of well graded sand that passes 2.36 mm sieve (BS Sieve No.7). The stiff mix is preferred, provided it does not prevent full penetration of the mesh. To improve the workability, the normal and high range superplasticizers are used. There are many methods for the construction of ferrocement but the most common method is mortar placement. In this method the mortar is forced through
mesh by hand plastering. Moist or wet curing is essential for ferrocement construction. Ferrocement is used for many purposes especially in structures or structural components where a low level of skill is required.

2.3.5 Application of fibre reinforced polymers (FRP) in strengthening of concrete structures

Fibre reinforced polymers sheets or plates are bonded to the exterior of reinforced concrete members on the tension side of flexural deficient members to increase the flexural capacity or on their side faces to increase the shear capacity using the wet lay-up procedure with an epoxy resin or adhesive [220].

2.3.5.1 Axial strengthening of reinforced concrete columns

Fibre reinforced polymers are most effective for strengthening of existing concrete columns. Fibre reinforced polymers are wrapped around the columns in the circumferential and longitudinal direction to provide confinement in order to increase both strength and ductility. When a column is subjected to an axial compressive load it expands laterally. This dilation causes tensile stress in the fibre reinforced polymer wrap and confines the concrete which keeps the concrete of the column in a state of tri-axial (three dimensional) stress. Due to this three dimensional state of stress, the load and deformation carrying capacity is increased significantly which results in a more strong and ductile structural member [220].

2.3.5.2 FRP confinement models for reinforced concrete columns

The current existing international design guidelines provide predictive design equations for the strengthening of un-heated reinforced concrete columns with fibre reinforced polymer tested under axial loading. The models for the column confinement with FRP were generally developed based on the laboratory tests of small specimens. Extensive research has been published on the FRP confinement models. However, the design guidelines included in the literature review are American Concrete Institute (ACI Committee 440.2R-02, 2002) [238], Canadian Standard Association (CSA S806-02, 2002) [239], Technical Report No.55 by the
Concrete Society (TR 55, 2004) [240]. The design equations reported by the mentioned international design codes are explained in the following sections.

**a) American Concrete Institute (ACI Committee 440.2R)**

The axial load carrying capacity of the strengthened column according to ACI Committee 440.2R [238] can be calculated using the following equation.

\[
P_a = 0.85 f'_{cc} (A_x - A_s) + f_y A_s
\]

Where \( P_a \) = Axial load carrying capacity

\( f'_{cc} \) = Compressive strength of confined concrete

\( A_x \) = Cross-sectional area of the confined concrete

\( A_s \) = Longitudinal reinforcing steel area

\( f_y \) = Yield strength of longitudinal reinforcing bars

The compressive strength of confined concrete \( f'_{cc} \) can be calculated using the following equation

\[
f'_{cc} = f_c \left[ 2.25 \sqrt{1 + 7.9 \frac{f_l}{f_c}} - 2 \frac{f_l}{f_c} - 1.25 \right]
\]

Where \( f_c \) = Unconfined compressive concrete strength

\( f_l \) = Lateral confinement pressure

The lateral confinement pressure \( f_l \) can be calculated by using the following equation

\[
f_l = \frac{k_a \rho f_{ce} E_f}{2}
\]

Where \( k_a \) = Efficiency reduction factor that can be taken as equal to 1.0 for circular sections and is expressed as follows non circular sections

\[
k_a = 1 - \frac{(b - 2r)^2 + (h - 2r)^2}{3bh(1 - \rho)}
\]

Where \( b, h \) and \( r \) represent short side, long side and radius of the edges of the section, the longitudinal reinforcement ratio \( \rho_s \) is calculated as the ratio of the longitudinal steel area to the gross area of column.
The volumetric ratio of FRP reinforcement $\rho_f$ can be calculated by using the following equations

$$\rho_f = \frac{4nt_f}{D} \quad \text{(Circular sections)}$$

Where $n$ = number of FRP layers

$$t_f = \text{Thickness of FRP layer}$$

$$D = \text{Diameter of circular column}$$

$$\rho_f = \frac{2nt_f(b + h)}{bh} \quad \text{(Non circular sections)}$$

Where 'b' and 'h' are the short and long dimensions of the cross-section of column

The FRP effective strain $\varepsilon_{fe}$ should be lesser of (0.004 or 0.75*ultimate FRP strain) and $E_f$ is the modulus of elasticity of FRP.

b) Canadian Standard Association (CSA- S806-02)

According to the Canadian Standard Association S806-02(CSA 2002) [239], the axial load carrying capacity of the strengthened column can be calculated using the following equation

$$P_a = \alpha f_{cc}^{'} (A_x - A_n) + A_n f_y$$

where $\alpha = 0.85 - 0.0015 f_c^{'} \geq 0.67$

The compressive strength of confined concrete $f_{cc}^{'}$ can be calculated using the following equations

$$f_{cc}^{'} = 0.85 f_c^{'} + k_s f_s f_l$$

Where $k_s = 6.7(k_s f_c)^{0.17}$

The shape factor $k_s$ can be taken as 1.0 for circular section and 0.25 for non circular section.

The lateral confinement pressure $f_l$ can be calculated by using the following equation

$$f_l = \frac{2nt_f E_f E_{fe}}{D} \quad \text{(For circular sections)}$$
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Where \( D \) = Diameter of circular section

The FRP effective strain \( \varepsilon_{ef} \) should be lesser of (0.004 or 0.75*ultimate FRP strain) and \( E_f \) is the modulus of elasticity of FRP

\[
f_i = \frac{2n t f \varepsilon_{ef} E_f}{D} \quad \text{(For non-circular sections)}
\]

(27)

Where \( D \) = lesser of ‘b’ and ‘h’ dimensions of the cross-section

c) Concrete Society Technical Report No.55 (TR55)

According to the Concrete Society Technical Report No.55 \cite{240}, the value of the confined concrete compressive strength \( f_{cc}' \) which is based on the characteristic unconfined cube strength \( f_{cu} \) can be calculated using the following equations

\[
f_{cc}' = f_{cu} + 0.08 \left( \frac{2n t f}{D} \right) E_f \quad \text{(For circular sections)}
\]

(28)

Where \( f_{cu} = 0.67 f_{cu} \) = Unconfined concrete compressive strength

\( D \) = Diameter of circular section

\( E_f \) = Tensile modulus of FRP

\[
f_{cc}' = f_{cu} + 2k_s f_i \quad \text{(For non-circular sections)}
\]

(29)

Where the shape factor \( k_s = \frac{b A_s}{h A_g} \)

\[
\frac{A_s}{A_g} = 1 - \frac{(h - 2r)^2 + (b - 2r)^2 - 3 A_g}{3 A_s - 3 A_g - \rho_s}
\]

Where

\( b \) = Length of short side

\( h \) = Length of long side

\( r \) = Radius of corner

\( A_s \) = Effective confined area

\( A_g \) = Total cross-sectional area = \( bh - (4 - \pi) r^2 \)

\( A_o \) = Area of overlap of the parabolas

\( = 0 \) if \( 2b \geq (h - 2r) \)
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

\[
\text{Length of overlap region} = \frac{4l_{ol}^3}{3(h-2r)} + l_{ol} (2b-(h-2r)) \text{ if } 2b < (h-2r)
\]

Where

\[
l_{ol} = \sqrt{\frac{(h-2r)^2}{4} - \frac{b(h-2r)}{2}}
\]

The lateral confinement pressure \(f_i\) can be calculated by using the following equation

\[
f_i = \frac{2f_f t_f}{\sqrt{b^2 + h^2}}
\]

Where \(f_f\) = Ultimate strain capacity of FRP

2.3.5.3 Seismic strengthening of reinforced concrete columns

When a reinforced concrete column is subjected to seismic loading, its energy absorption capacity, rather than its load capacity, is the main concern [17]. Traditionally, the energy absorption capacity of columns is increased by reinforced concrete jacketing or steel jacketing. However, steel and concrete jacketing results in a significant increase of column stiffness, which may lead to additional earthquake forces in columns. There are two methods for seismic strengthening of concrete columns [17].

1) Strength-oriented.
2) Ductility-oriented.

1) Strength-Oriented

In this method the main fibres of fibre reinforced polymer are bonded longitudinal in order to increase the flexural strength of columns.

2) Ductility-Oriented

In this method the main fibres of fibre reinforced polymer are wrapped in the hoop or transverse direction to increase the ductility of reinforced concrete columns. Both the above methods can enhance the energy absorption capacity of the column.

2.3.6 FRP jackets with horizontal oriented fibres

To improve the shear capacity and the ductility of concrete columns against seismic forces, the columns should be wrapped with fibre sheets or winding fibre strands with
resin in a wet lay-up process in which the main fibres are generally oriented in the transverse or hoop direction. Under shear forces, the tensile stresses in the fibre reinforced polymers contribute to the overall shear resistance of the columns. In the shear strengthening of reinforced concrete columns, the fibre reinforced polymer jacket is generally required to the full entire column height while for plastic hinge confinement and for lap splices clamping, the fibre reinforced polymer is required in the plastic hinge and near by end regions [90]. In order to prevent the FRP jacket from direct axial loading, a small gap up to 20 mm has been recommended between the ends of the jacket and any adjacent transverse structural member or footing [17].

2.3.7 FRP jackets with longitudinal oriented fibres

The fibre reinforced polymer jackets with main fibres oriented in the hoop or transverse direction of columns increase its flexural capacity mostly if a significant axial load is present. When the lateral FRP confinement is not sufficient for seismic loads, an additional increase may be attained by bonding FRP sheets with the main fibres oriented in the longitudinal direction, leading to the combined use of lateral and longitudinal fibre reinforced polymers [17]. The bonding of FRP sheets with the main fibres oriented in the longitudinal direction is very effective for flexural deficient columns at cut off sections of longitudinal reinforcement [17]. It was observed that the combination of laterally bonded FRP with longitudinal bonded FRP jackets over cut-off sections can effectively shift flexural failure to the potential plastic hinge regions under maximum moments [17].

2.3.8 Typical failure of reinforced columns under seismic attack

In earthquakes, the reinforced concrete structures are subjected to lateral cyclic loads with coexisting axial loads. The failure of reinforced concrete columns in earthquakes is due to deficient shear strength, insufficient flexural strength or lack of ductility. The columns having a smaller section and larger shear span-to-depth ratio experienced large deformations due to the yielding of the main reinforcement and failed in a flexural mode [221]. The flexural failure due to more inelastic deformations is generally less destructive and no sudden collapse has happened in this kind of mode of failure. Additionally, the flexural failure allows stress re-distribution and the
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

columns are continuing to support the super-structure which gives warning to the occupants to leave the buildings without any loss. Flexural failures typically occur at the column ends, with some displacement ductility, and are limited to smaller regions in the column [17, 90]. Therefore, the flexural damaged reinforced concrete columns can be repaired easily [221].

The unstable and brittle catastrophic shear failure has been experienced in columns having comparatively smaller shear span-to-depth ratios, larger sections, less main reinforcement ratios and insufficient transverse reinforcement ratios during earthquakes [221, 222], as shown in Figs. 26 and 27. The brittle shear failure is very dangerous and should be avoided in case of future expected strong earthquakes [221].

In earthquakes, the shear failure always prevails in short columns having smaller shear span-to-depth ratios rather than in longer columns [90, 221-223]. Short columns in reinforced concrete frame buildings are considered sometimes intentionally in the design or produced unintentionally in buildings when buildings are constructed on sloping ground or buildings with a mezzanine floor or the clear height of building frame is reduced due to presence of stiff structural or non-structural elements such as a placing of a masonry infill wall of partial height to fit a window over the remaining height, as shown in Figs. 28,29,30 [224].

As the effective height over which a short column can under go deformation freely during earthquake becomes small it offers more resistance to horizontal motion, as shown in Fig. 31. Therefore, a short column attracts more earthquake forces as compared to long columns. Under reversal actions of earthquake forces with coexisting axial loads, the short columns due to diagonal cross shear cracking suddenly lose their load carrying capacity and are unable to support the super structure [92]. The columns failing in a shear mode are very dangerous and must be avoided if possible by increasing the shear resistance of short columns. This must be
done using effective and economical strengthening techniques, and the strengthened column in existing structures must fail in a flexural mode rather than in shear.

Fig. 26: Shear failure of columns 1995, Kobe Japan Earthquake [221]

Fig. 27: Shear failure of columns due to short column effect [225]
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 28: Building with short columns [224]

Fig. 29: Building on sloping ground with short columns [224]
During an earthquake, a long column and a short column of the same cross section move horizontally by same amount. A short column is stiffer than long column and it attracts more horizontal earthquake force [224].
2.3.8.1 Hysteretic response of reinforced concrete column

The response of a structure to static and dynamic loading is assessed in terms of energy dissipation. The evaluation of energy dissipation depends upon the hysteretic response of structure. When the ductility level increases, the hysteretic loops get wider and more energy is dissipated. The energy that a structural system can dissipate in an earthquake is a function of its inelastic deformation. Therefore it is supposed to be more economical to consider the energy dissipation of structure in the inelastic deformation range in the design rather than in the elastic range [226]. This requires understanding of the hysteretic behaviour of the structural members.

2.3.8.2 Residual axial load capacity of reinforced concrete column

When a reinforced concrete column is subjected to constant axial load and lateral cyclic load simultaneously, then yielding of the longitudinal reinforcement takes place at the failure load. After the yielding of longitudinal reinforcement, the column sustains the gravity load (axial load) and lateral loads (shear loads) until the shear demand on the column exceeds the ultimate shear capacity of reinforced concrete column. After shear failure in the reinforced concrete column, the gravity load (axial load) is supported by shear friction forces developed within the column and the column still continues to retain some axial capacity after shear failure [227]. When the axial load exceeds the shear friction forces then axial failure occurs in the column and total collapse of the reinforced concrete column takes place. The inclined shear failure surface is supposed to occur at a critical angle which represents the inclined crack resulting from shear failure in the column. When the effective friction coefficient or critical crack angle is not known, the residual axial capacity of column can be estimated as 10% of the undamaged axial load capacity prior to testing of reinforced concrete columns [227].

2.3.8.3 Effect of shear span to effective depth ratio

A column fails in a shear mode when the shear span to effective depth ratio remains less than or equal to 2.5 \((a/d\leq 2.5)\). When the shear span to effective depth ratio varies between 2.5 to 5 \((5>a/d>2.5)\) then the column fails in a flexure-shear mode.
When the shear span to effective depth ratio exceeds 5 (a/d>5) the column fails in a flexural mode [228].
CHAPTER-3

3 MATERIAL PROPERTIES, CONSTRUCTION AND HEATING OF SPECIMENS

3.1 INTRODUCTION

An experimental programme developed by the author was conducted to investigate the effectiveness of advanced composite materials for the repairing of fire damaged reinforced concrete square and circular columns. A total of thirty five columns were constructed in this study. The experimental programme was divided into two parts. The experimental part-1 consisted of ten square and nine circular columns tested under gravity loading only. The experimental part-2 comprised eight square and eight circular columns tested under combined constant axial and lateral reversal cyclic loading in order to simulate the gravity and earthquake loading.

The experimental work was carried out into the following five stages.

1. Casting
2. Heating
3. Non destructive testing
4. Repairing
5. Testing

This chapter presents the properties of materials used in the experimental work, a description of all test specimens, construction procedures, heating and non-destructive testing of specimens. The repairing and testing procedure for columns is described in Chapter-4

3.2 MATERIALS

3.2.1 Concrete

The same concrete mix, comprising sand, gravel aggregate and Ordinary Portland Cement (OPC), was used for the construction of all specimens in both parts of experimental work. 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 28 days compressive strength of the cubes is shown in Table A.2 in the Appendix. The average cube strength of un-heated and post-heated columns at the time of testing was 54 MPa and 23 MPa respectively, as shown in Tables 8 and 9.

### 3.2.2 Reinforcing steel

Deformed cold worked reinforcing steel bars having 6 mm and 10 mm diameter were used in the construction of all specimens. Tension tests were performed on steel samples of each bar size. The mechanical properties of pre-heated and post-heated reinforcing steel bars are given in the Table 10. The pre-heated cold worked steel bars did not give a well-defined yield plateau while post-heated cold worked steel bars displayed a well defined yield plateau, as shown in Figs.A-1 to A-4. For steel without clear yield strength, the yield strength was determined at the 0.2% strain offset.

### 3.2.3 Fibre reinforced polymer (FRP)

The FRP composites used in the present study were unidirectional Tyfo SCH-41 carbon, Tyfo SEH-51A glass and Weber.tec force C-240 unidirectional carbon sheets. The properties of the dry fibre reinforced polymers and composite laminates were adopted from suppliers, as shown in Tables 11 and 13.

### 3.2.4 Tyfo S Epoxy

The Tyfo S epoxy is a two component epoxy matrix material and is used for bonding applications of Tyfo SCH-41 carbon, Tyfo SEH-51A glass fabric sheets. The properties of Tyfo S epoxy adopted from the supplier are shown in Table 12.

### 3.2.5 Weber.tec force EP primer

Weber.tec force EP primer is a two component epoxy resin primer and is used to seal and stabilise prepared surfaces of the concrete before the application of bonding fibre sheets for structural strengthening. The properties of Weber.tec force EP primer adopted from the supplier are shown in Table 12.
3.2.6 Weber.tec force EP bonding adhesive

Weber.tec force EP bonding adhesive is a two-component epoxy resin adhesive for concrete and is used to structurally fasten the Weber.tec force composite sheets and transfers all loads into the fibre composites. The properties of the Weber.tec force EP bonding adhesive provided by supplier are shown in Table 12.

3.2.7 Cabo-o-sil M5 powder

Cabo-sil M5 (fumed silica) is a light, fluffy powder having chain-forming tendencies and is white in appearance. The cabo-sil M5 powder added with Tyfo S epoxy was used to fill voids, cavities and micro cracks of post-heated columns.

3.2.8 Weber.tec EP bonding aid

Weber.tec EP bonding aid is a highbuild epoxy resin for priming epoxy repair mortars for bonding to concrete surfaces. It is used to bond freshly-mixed concrete, mortars to existing sound concrete substrate. The properties of bonding aid adopted from the supplier are shown in Table 14.

3.2.9 Weber.tec EP highbuild

Weber.tec EP highbuild composed of epoxy resin, hardener and special lightweight aggregates (in the powder form). The mortar has low slump characteristics and is suitable for the repair of damaged concrete on vertical and overhead applications. It can be applied in layers up to 75 mm thick on vertical surfaces or up to 30 mm deep on soffits. The highbuild epoxy resin mortar was used to repair seriously spalled circular columns. The properties of mortar adopted from the supplier are shown in Table 15.

3.2.10 Ferrocement

Ferrocement is a composite material consisting of rich cement sand mortar and very thin reinforcing wire mesh.

3.2.10.1 Cement sand mortar

The cement sand mortar of ferrocement consisted of Ordinary Portland Cement (OPC), sand passing through BS No.7 sieve and silica fume. The cement and sand
was mixed in the ratio of 1:2 by weight with a 0.35 water cement ratio. To improve the compressive strength of mortar 10% silica fume was added by weight of cement. Supperplasticizer with ratio of 1.2% by weight of cement/silica fume was added to improve the workability of mortar. The average cube strength at the time of testing was 66 MPa.

3.2.10.2 Reinforcing wire mesh

A galvanised square welded mesh made in the UK with 1.6 mm wire diameter and 12.5 mm mesh opening was used in the construction of ferrocement. The repair comprised four layers of galvanised square wire welded meshes with 354 MPa yield strength (adopted from the supplier).

3.2.10.3 Polyvinyl acetate (PVA)

Polyvinyl acetate is used for filling micro cracks and as a bonding agent for the repairing of post-heated columns with ferrocement.
### Compressive strength of controlling cubes for square columns

<table>
<thead>
<tr>
<th>Testing conditions</th>
<th>Specimen</th>
<th>Testing period (Months)</th>
<th>Compressive strength MPa (100 mm cubes)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>After casting</td>
<td>After heating</td>
<td>Un-heated</td>
</tr>
<tr>
<td>Un-heated</td>
<td>S1</td>
<td>13</td>
<td>-</td>
<td>58</td>
</tr>
<tr>
<td>Un-heated</td>
<td>S2</td>
<td>13</td>
<td>-</td>
<td>56</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S3</td>
<td>10</td>
<td>2</td>
<td>53</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S4</td>
<td>10</td>
<td>2</td>
<td>53</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S5</td>
<td>14</td>
<td>2</td>
<td>50</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S6</td>
<td>14</td>
<td>2</td>
<td>52</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S7</td>
<td>12</td>
<td>2</td>
<td>53</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S8</td>
<td>19</td>
<td>2</td>
<td>55</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S9</td>
<td>19</td>
<td>2</td>
<td>52</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S10</td>
<td>19</td>
<td>2</td>
<td>52</td>
</tr>
<tr>
<td>Un-heated</td>
<td>S11</td>
<td>16</td>
<td>-</td>
<td>56</td>
</tr>
<tr>
<td>Un-heated</td>
<td>S12</td>
<td>16</td>
<td>-</td>
<td>58</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S13</td>
<td>11</td>
<td>2</td>
<td>53</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S14</td>
<td>11</td>
<td>2</td>
<td>54</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S15</td>
<td>15</td>
<td>2</td>
<td>54</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S16</td>
<td>16</td>
<td>3</td>
<td>51</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S17</td>
<td>12</td>
<td>3</td>
<td>53</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>S18</td>
<td>12</td>
<td>3</td>
<td>53</td>
</tr>
</tbody>
</table>

|                        |          |            |            | Average compressive strength | 54 | 23 |
| Standard deviation     | 2.135744| 1.624221   |            | Compressive strength percentage loss | 57.00 % |

Table 8: Crushing strength of controlling cubes for square columns
### Compressive strength of controlling cubes for circular columns

<table>
<thead>
<tr>
<th>Testing conditions</th>
<th>Specimen</th>
<th>Testing period (Months)</th>
<th>Compressive strength MPa (100 mm cubes)</th>
<th>After casting</th>
<th>After heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Un-heated</td>
<td>C1</td>
<td>12</td>
<td>-</td>
<td>58</td>
<td>-</td>
</tr>
<tr>
<td>Un-heated</td>
<td>C2</td>
<td>13</td>
<td>-</td>
<td>55</td>
<td>-</td>
</tr>
<tr>
<td>Un-heated</td>
<td>C10</td>
<td>19</td>
<td>-</td>
<td>56</td>
<td>-</td>
</tr>
<tr>
<td>Un-heated</td>
<td>C11</td>
<td>19</td>
<td>-</td>
<td>57</td>
<td>-</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C3</td>
<td>12</td>
<td>2</td>
<td>52</td>
<td>22</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C4</td>
<td>12</td>
<td>2</td>
<td>53</td>
<td>22</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C5</td>
<td>14</td>
<td>2</td>
<td>52</td>
<td>24</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C6</td>
<td>19</td>
<td>4</td>
<td>51</td>
<td>22</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C7</td>
<td>11</td>
<td>2</td>
<td>53</td>
<td>22</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C8</td>
<td>14</td>
<td>2</td>
<td>54</td>
<td>23</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C9</td>
<td>14</td>
<td>5</td>
<td>53</td>
<td>21</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C12</td>
<td>16</td>
<td>4</td>
<td>52</td>
<td>25</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C13</td>
<td>16</td>
<td>4</td>
<td>53</td>
<td>25</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C14</td>
<td>14</td>
<td>4</td>
<td>54</td>
<td>24</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C15</td>
<td>14</td>
<td>4</td>
<td>53</td>
<td>24</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C16</td>
<td>16</td>
<td>2</td>
<td>52</td>
<td>22</td>
</tr>
<tr>
<td>Post-heated(500°C)</td>
<td>C17</td>
<td>18</td>
<td>10</td>
<td>53</td>
<td>22</td>
</tr>
<tr>
<td><strong>Average compressive strength</strong></td>
<td></td>
<td></td>
<td></td>
<td>54.0</td>
<td>23</td>
</tr>
<tr>
<td><strong>Standard deviation</strong></td>
<td></td>
<td></td>
<td></td>
<td>1.851514</td>
<td>1.268814</td>
</tr>
<tr>
<td><strong>Compressive strength percentage loss</strong></td>
<td></td>
<td></td>
<td></td>
<td>57.0%</td>
<td></td>
</tr>
</tbody>
</table>

Table 9: Crushing strength of controlling cubes for circular columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

### Properties of reinforcing steel

<table>
<thead>
<tr>
<th>Test conditions</th>
<th>Size (mm)</th>
<th>Yield strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Un-heated</td>
<td>6 mm</td>
<td>570</td>
</tr>
<tr>
<td></td>
<td>10 mm</td>
<td>553</td>
</tr>
<tr>
<td>Post-heated</td>
<td>6 mm</td>
<td>498</td>
</tr>
<tr>
<td></td>
<td>10 mm</td>
<td>487</td>
</tr>
</tbody>
</table>

Table 10: Properties of reinforcing steel

### Composite typical dry fibre properties

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>Tyfo SCH-41 Fyfe*</th>
<th>Tyfo SEH-51A Fyfe*</th>
<th>C Sheet 240 Weber**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength GPa</td>
<td>3.8</td>
<td>3.24</td>
<td>4.0</td>
</tr>
<tr>
<td>Tensile modulus GPa</td>
<td>230</td>
<td>72.4</td>
<td>240</td>
</tr>
<tr>
<td>Ultimate elongation (%)</td>
<td>1.7</td>
<td>4.5</td>
<td>1.6</td>
</tr>
<tr>
<td>Fibre thickness (mm)</td>
<td>0.37</td>
<td>0.36</td>
<td>0.117</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>1.74</td>
<td>2.55</td>
<td>1.7</td>
</tr>
<tr>
<td>Fabric Width [m]</td>
<td>0.61</td>
<td>1.37</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Table 11: Composite typical dry fibre properties

### Epoxy material properties

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength (MPa)</td>
<td>72.4</td>
<td>19</td>
<td>17</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>-</td>
<td>100</td>
<td>80</td>
</tr>
<tr>
<td>Tensile modulus (GPa)</td>
<td>3.18</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Elongation</td>
<td>5%</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Flexural strength (MPa)</td>
<td>123.4</td>
<td>30</td>
<td>28</td>
</tr>
<tr>
<td>Flexural modulus (GPa)</td>
<td>3.12</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Bond to concrete (MPa)</td>
<td>-</td>
<td>&gt;5.3</td>
<td>&gt;4</td>
</tr>
<tr>
<td>Coefficient of expansion</td>
<td>-</td>
<td>8X10⁻⁶ mm/mm°C</td>
<td>6X10⁻⁶ mm/mm°C</td>
</tr>
<tr>
<td>Glass transition temperature</td>
<td>82°C</td>
<td>60°C</td>
<td>60°C</td>
</tr>
</tbody>
</table>

Table 12: Epoxy material properties
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
<thead>
<tr>
<th>Composite Laminate Properties*</th>
<th>Tyfo SCH-41 Fyfe*</th>
<th>Tyfo SEH-51A Fyfe*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate tensile strength in primary fibre direction (MPa)</td>
<td>986</td>
<td>575</td>
</tr>
<tr>
<td>Elongation at break (%)</td>
<td>1</td>
<td>2.2</td>
</tr>
<tr>
<td>Tensile Modulus (GPa)</td>
<td>95.8</td>
<td>26.1</td>
</tr>
<tr>
<td>Ultimate tensile strength 90 degrees to primary fibre (MPa)</td>
<td>-</td>
<td>25.8</td>
</tr>
<tr>
<td>Nominal laminate thickness (mm)</td>
<td>1.0</td>
<td>1.17</td>
</tr>
</tbody>
</table>

Table 13: Composite laminate properties

<table>
<thead>
<tr>
<th>Weber tec. EP bonding aid**</th>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (MPa)</td>
<td>100</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>19</td>
</tr>
<tr>
<td>Flexural strength (MPa)</td>
<td>30</td>
</tr>
<tr>
<td>Modulus of elasticity (MPa)</td>
<td>6000</td>
</tr>
<tr>
<td>Bond to abraded steel surface (MPa)</td>
<td>17</td>
</tr>
<tr>
<td>Bond to concrete (MPa)</td>
<td>&gt;5.3</td>
</tr>
<tr>
<td>Bond of new to old concrete (MPa)</td>
<td>&gt;2.8</td>
</tr>
<tr>
<td>Pot life of 1 litre (Minutes)</td>
<td>12</td>
</tr>
<tr>
<td>Coverage on rough concrete (m²/litre per coat)</td>
<td>1.5-3</td>
</tr>
<tr>
<td>Minimum temperature use</td>
<td>5°C</td>
</tr>
</tbody>
</table>

Table 14: Properties of Weber.tec EP bonding aid

<table>
<thead>
<tr>
<th>Weber.tec EP highbuild mortar**</th>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td>Days 7</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>Days 7</td>
</tr>
<tr>
<td>Flexural strength</td>
<td>Days 7</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td></td>
</tr>
</tbody>
</table>

Table 15: Weber.tec EP highbuild mortar properties

** Saint-Gobain Weber Ltd Web: http://www.netweber.co.uk
3.3 TEST SPECIMENS
3.3.1 Experimental part-1: (Tested under axial loading)

3.3.1.1 Circular reinforced concrete columns

The objective of the experimental programme was to investigate the effectiveness of fibre reinforced polymer (FRP) and ferrocement jackets for the repairing of heat damaged circular concrete columns. The columns were cast with gravel aggregate concrete within the laboratory at the University of Manchester UK. The columns were tested under eight different conditions.

1) Two un-heated columns (C1 and C2).
2) A post-heated column without any spalling (C3).
3) A post-heated column without any spalling wrapped with a unidirectional glass fibre reinforced polymer (Tyfo SEH-51A GFRP) jacket (C4).
4) A post-heated column with serious explosive spalling and repaired with epoxy resin mortar (C5).
5) A post-heated column with serious explosive spalling repaired with both epoxy resin mortar and Tyfo SEH-51A GFRP jacket (C7).
6) A post-heated column without any spalling wrapped with a carbon fibre reinforced polymer (Weber. tec force C-240 CFRP) jacket (C6).
7) A post-heated column with serious spalling repaired with both epoxy resin mortar and Tyfo SCH-41 CFRP jacket (C8).
8) A post-heated column with serious spalling repaired with a both concrete and ferrocement jacket (C9).

All columns were identical when cast, having a 1000 mm length and 200 mm diameter. All the specimens were reinforced with six 10 mm diameter longitudinal ribbed reinforcing bars, resulting in a 1.5% longitudinal reinforcement ratio. All bars were evenly distributed throughout the cross-section with 30 mm cover to the main longitudinal reinforcement. For all columns, 6mm diameter ribbed bars were used as link-bars spaced at 100 mm centres. The link-bars were provided with a 60 mm
overlap at the ends without any extension into the concrete cores, as shown in Fig. 32.

Seven columns were heated in an electric furnace to 500°C uniformly at a rate of 2.5°C/min. The furnace remained at 500°C until the columns reached a uniform temperature of 500°C. During heating; four columns were seriously damaged by explosive spalling. Three spalled columns were repaired with highbuild epoxy resin mortar before being wrapped with the FRP. Two post-heated columns were repaired with unidirectional glass fibre, and two columns were repaired with unidirectional carbon fibre reinforced polymer. One seriously spalled column was repaired with original concrete mix prior to being repaired with ferrocement jacket. All columns were tested under axial compression to determine their ultimate axial strength, stiffness and ductility.

### 3.3.1.2 Square reinforced concrete columns

In order to investigate the effectiveness of fibre reinforced polymer (FRP) and ferrocement jackets for the repairing of heat damaged columns, ten reinforced concrete square columns were cast with gravel aggregate concrete within the same structural engineering laboratory as described previously. The columns were divided into six categories.

a) Without heating and without FRP (S1&S2).
b) With heating but without FRP (S3&S4).
c) With heating and repaired with Tyfo SEH-51A GFRP jacket (S5&S6)
d) With heating and repaired with Tyfo SCH-41 CFRP jacket (S7)
e) With heating and repaired with Weber. tec force C-240 CFRP (S8&S9)
f) With heating and repaired with ferrocement (S10).

For the heated columns, all columns were heated to a uniform temperature of 500°C before being allowed to cool down. The temperature of 500°C was chosen for both square and circular columns since at this temperature concrete generally retains a residual strength of approximately 50% of its original strength [2]
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 while four layers of wire mesh were used for ferrocement jacket. 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. 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. 36.

3.3.2 Experimental part-2: (Tested under combined axial and lateral reversal cyclic loading)

3.3.2.1 Circular reinforced concrete columns

The aim of the current experimental study is to determine the effectiveness of fibre reinforced polymer for seismic shear strengthening of heat damaged circular reinforced concrete columns. In the reported study, eight circular reinforced concrete columns were cast with gravel aggregate concrete and tested under combined constant axial and lateral reversal cyclic loading. The columns were divided into four groups.

1) Two un-heated columns (C10&C11)
2) Two post-heated columns without any spalling (C12&C13)
3) Two post-heated columns (without any spalling) wrapped with unidirectional glass fibre reinforced polymer (Tyfo SEH-51A GFRP) jacket with main fibres oriented in the transverse direction (C14&C15)
4) Two post-heated columns: one without any spalling wrapped with a Tyfo SCH-41 CFRP jacket only and other with spalling repaired with both concrete and a Tyfo SCH-41 CFRP jacket with the main fibres oriented in the transverse direction (C16& C17).
All columns have 200 mm diameter and overall height 1000 mm. All the specimens were reinforced longitudinally with six No.10 deformed bars similar to circular columns as described earlier in Section 3.3.1.1. The longitudinal bars were extended up to 80 mm through the formwork in all columns from the bottom side at the time of casting. One end of all the longitudinal bars were threaded up to 40 mm in order to bolt them to a 40 mm base stiff steel plate at the time of testing. Each column had a strong steel stub at its base representing the column footing junction or beam column joint.

The test specimens were shear critical type intentionally due to a small shear span to depth ratio of 2.5 [228]. The shear span was taken as a distance from the point of application of lateral cyclic load to the interface of column footing. All specimens were tested under combined constant axial and lateral reversal cyclic loading representing gravity and seismic loading. The cyclic loading was applied at the height of 425 mm from the centre of the pin of the actuator (where the cyclic loading was applied) to the face of column footing.

### 3.3.2.2 Square reinforced concrete columns

The objective of this study is to investigate, experimentally, the effectiveness of unidirectional glass or carbon fibre reinforced polymers for shear strengthening of post-heated reinforced concrete square columns. In this study, a total of eight reinforced concrete square columns were cast with gravel aggregate concrete and tested under combined constant axial and lateral reversal cyclic loading. The test specimens were divided into five test series.

a) Two un-heated columns (S11&S12)
b) Two post-heated columns (S13&S14)
c) A post-heated column wrapped with unidirectional glass fibre reinforced polymer (Tyfo SEH-51A GFRP) jacket with main fibres oriented in the transverse direction (S15).
d) A post-heated column wrapped with unidirectional glass fibre reinforced polymer (Tyfo SEH-51A GFRP) jacket with main fibres oriented in the longitudinal direction (S16)
e) Two post-heated columns wrapped with unidirectional carbon Tyfo SCH-41 CFRP jacket with main fibres oriented in the transverse direction (S17 & S18).

All square columns had the same geometry and reinforcement, as explained earlier in Section 3.3.1.2. However all longitudinal bars were threaded from one end and extended up to 80 mm from the formwork at the time of casting similar to the circular columns explained earlier in Section 3.3.2.1. The test specimens were vertical cantilever columns with a strong steel stub at the base. In order to ensure the shear failure mode, all columns were tested with a shear span to depth ratio of 2.5 [228] under bi-directional loading similar to circular columns (as explained in Section 3.3.2.1) simulating the gravity and unidirectional seismic loading.

It is worth to mention here that three presumptions were made in testing of square and circular columns of experimental part-2.

1) All columns were heated without any load because the un-stressed residual strength test is more conservative for assessing the post-fire or residual properties of concrete [141, 145, 147, 164].

2) In order to ensure the shear failure mode, all columns were tested with shear span to depth ratio of 2.5 [228] under bi-directional loading which simulates the gravity and unidirectional seismic loading.

3) Only a single layer of glass or carbon fibre reinforced polymer with main fibres oriented in the transverse direction was used in this study to confirm the effectiveness of fibre reinforced polymers for the shear strengthening of post-heated columns.
3.4 CASTING OF SPECIMENS

3.4.1 Experimental part-1(Tested under axial loading)

3.4.1.1 Circular reinforced concrete columns

The test specimens were cast vertically using PVC pipe formwork, as shown in Fig. 33 (a). The moulds were properly oiled on the inner sides and the prepared cage of reinforcement shown in Figs. 33 (b) and 34 (a) was kept in the moulds. Concrete spacers of 30 mm size were attached to the main reinforcing bars using binding wires to maintain 30 mm concrete cover to the main reinforcement, as shown in Fig. 34 (a). Two columns were cast from a single batch of concrete. Each column was cast along with three cubes of 100 mm size in order to monitor the concrete strength of columns at the time of testing. The concrete mix was filled into the PVC moulds in layers and was compacted using an internal poker vibrator to remove air voids.

Two type K-thermocouples were embedded in each column during casting in order to monitor the temperature at the time of heating. One thermocouple was placed at the centre of the columns at mid-height, and the other was attached to one of the longitudinal reinforcing bars at mid-height. The columns and cubes were demoulded after 24 hours of casting and then cured under moist canvas that was kept continuously wet with water. All specimens were covered with a plastic sheet for fourteen days to prevent the loss of moisture. After fourteen days, the columns along with cubes were then air cured at room temperature in the laboratory environment until heating and testing, as shown in Fig. 35.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 32: Arrangement of reinforcement in circular columns

Fig. 33: (a) PVC moulds; (b) prepared cage of reinforcement
Fig. 34: (a)-(b) Prepared cage of reinforcement in PVC moulds; (b) Casting of concrete.
3.4.1.2 Square reinforced concrete columns

The reinforcement was fixed in all square columns in a pattern, as shown in Fig. 36. All square columns were cast in a horizontal position using steel moulds for the formwork, as shown in Fig. 37. The steel moulds were properly oiled on the inner sides for easy removal of the specimens at the time of demoulding. The prepared cage of reinforcement was kept in the moulds carefully. Concrete spacers of 25 mm size were used to maintain 25 mm concrete cover to the main reinforcement using binding wire. The concrete was poured in three layers and compaction of each layer was carried out using a vibrating table. Two columns were cast at a time from a single batch of concrete along with three cubes of 100 mm size for each column in order to monitor the strength at the time of testing. To avoid stress concentration at the corners, the corners of square columns were rounded using concave wood.
sections with a 25 mm radius, placed inside the formwork during casting, as shown in Fig. 37. Two type K-thermocouples were embedded into each column during casting to monitor the temperature at the time of heating. One thermocouple was placed in the centre of the column, at mid-height, whilst the other was attached at mid-height to the reinforcement. After 24 hours of casting, the specimens were cured using moist sacking. All specimens were covered with plastic sheet to prevent the loss of moisture. The columns were cured for fourteen days after which they were left in the laboratory environment until the day of heating and testing. All cubes were also kept with the columns for curing until they were heated and tested, as shown in Fig. 38.

Fig. 36: Reinforcement arrangement in square column

Fig. 38: Specimen placement and reinforcement details
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 37: Fixing of reinforcing steel in square columns

Fig. 38: Curing of square columns and cubes
3.4.2 Experimental part-2 (Tested under combined axial and lateral reversal cyclic loading)

3.4.2.1 Circular reinforced concrete columns

All circular columns were constructed in an upright position using PVC moulds formwork in a similar fashion as described in Section 3.4.1.1

3.4.2.2 Square reinforced concrete columns

All square columns were cast in a horizontal position using well oiled steel moulds formwork adopting the same procedure as illustrated in Section 3.4.1.2.

3.5 HEATING OF SPECIMENS

3.5.1 Heating of cubes

Thirty 150 mm × 150 mm × 150 mm size and twelve 100 mm × 100 mm × 100 mm size cubes were cast from the same batches of concrete (which were used for the construction of square and circular columns). Fifteen 150 mm × 150 mm × 150 mm size and six 100 mm × 100 mm × 100 mm cubes were heated in an electric furnace having 1.6 m × 1.2 m × 1.5 m size after more than nine months of casting at a rate of 2.5°C/min. Three cubes of 150 mm × 150 mm × 150 mm size were heated at the same time in the electric furnace at each temperature of 200°C, 300°C, 450°C, 500°C and 550°C respectively in order to estimate the retained concrete compressive strength. At 450°C and 550°C, six cubes of size 100 mm × 100 mm × 100 mm were also tested along with 150 mm × 150 mm × 150 mm cubes to consider the effect of specimen size on the retained compressive strength. An unstressed residual strength test was used to evaluate the residual strength of the cubes, which is considered to be more conservative in terms of design for assessing the post-fire or residual properties of concrete after fire [145, 147, 164].

One K-type thermocouple was embedded at mid-height in the centre of each cube to monitor the temperature in the cubes. The temperatures within the furnace were controlled by two type-K thermocouples installed at mid-height and at the top of the
electric furnace. When the average furnace temperatures reached specified temperatures, the furnace temperatures were kept constant until the temperature at the centre of cubes coincided with the target temperature. After achieving the point of uniformity of temperature inside the furnace and at the centre of the cubes, the furnace was switched off and the cubes were allowed to cool down naturally inside the furnace. The cover of the furnace was lifted off when the temperature inside the furnace reached ambient temperature. The specimens were then taken out of the furnace and stored in a dry condition at room temperature for 7 days until testing. The time temperature curves of cubes heated to 200°C, 300°C, 450°C, 500°C and 550°C respectively are shown in Figs. 39 to 43.

![Time-temperature curve for cubes exposed to 200°C](image-url)
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 40: Time-temperature curve for cubes exposed to 300°C

Fig. 41: Time-temperature curve for cubes exposed to 450°C
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 42: Time-temperature curve for cubes exposed to 500°C

Fig. 43: Time-temperature curve for cubes exposed to 550°C
3.5.2 Heating of columns

The columns were heated at an age of more than nine months after casting. An electric furnace having a size of $1.6 \times 1.2 \times 1.5$ m was used to heat all columns. At first, two columns, one square (S3) and one circular (C7), cast from single batch of concrete were heated together at the same time at a rate of 150°C/hour. The circular column (C7) was serious damaged by explosive spalling while no any evidence of spalling was observed in the square column (S3), as shown in Fig. 44. After facing the problem of serious explosive spalling in circular column, it was decided to heat square and circular columns separately. In order to protect the furnace from the damage due to explosive spalling, a cage made of wire mesh, as shown in Fig. 45, was provided around all circular columns when placed in an electric furnace for heating.

Fig. 44: Serious explosive spalling experienced in a circular column during heating
3.5.2.1 Heating of circular reinforced concrete columns

Thirteen circular columns of experimental part-1 and part-2 were heated at an age of more than nine months after casting in an electric furnace having a size of 1.6 m × 1.2 m × 1.5 m. A maximum of six columns were heated at a time in an unstressed condition, along with nine 100 mm cubes as control specimens. The bars shown protruding from the columns in Figs. 46 (a), 49, 50 had two functions. (1) To aid lifting of the columns in and out of the furnace. However, in practice it was found that they were not required and it was possible to lift the columns easily. The bars were cut prior to being tested under axial load (in the case of experimental part-1 columns). (2) The protruded part of bars was threaded up to 40 mm length prior to casting of
columns. The threaded part of all bars was completely extended from the bottom of columns tested under combined axial and reversed lateral cyclic loading (experimental part-2 columns) in order to screw and bolt them to the 40 mm stiff steel plate at the time of testing. The arrangement of the columns in the furnace and the furnace set up is shown in Fig. 46.

An average heating rate of 150°C/hour was used in the experiment during heating of all circular columns. The temperature at the centre of each concrete column and of one of the longitudinal reinforcement bars at mid-height was monitored using two type K-thermocouples, which were placed in position prior to casting the columns as described previously. The temperatures within the furnace were controlled by two K-thermocouples; one installed at mid-height and the other installed at the top of the furnace. The surface temperature was also monitored by attaching an additional two K-type thermocouples on the surface of each column. During heating, the extended reinforcing bars shown in Fig. 46 were insulated with Isofrax blanket.

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 [140, 145, 147]. When the average furnace temperature reached 500°C, the furnace temperature was kept constant until achieving the point of uniformity of temperature, inside the furnace, at the surface of columns and at the centre of each column. At this point, the furnace was switched off and the columns were then allowed to cool down naturally within the furnace.

The cover of the furnace was lifted off when the temperature inside the furnace reached 200°C in the cooling phase in the case of experimental part-1, while the cover of the furnace was lifted off when the temperature inside the furnace reached 36°C in case of experimental part-2. The measured time-temperature relationship for experimental part-1 and part-2, are shown in Figs. 47 and 48 respectively. After cooling down, the columns were removed from the furnace and kept at room
temperature in the laboratory until repairing and testing. It is interesting to note that out of thirteen circular columns heated, five columns experienced serious explosive spalling while others did not. The summary of exploded columns is shown in Table 16.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 47: Time-temperature curve for circular columns (experimental part-1) exposed to 500°C

Fig. 48: Time-temperature curve for circular columns (experimental part-2) exposed to 500°C
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 49: Circular columns after heating in an electric furnace

Fig. 50: Explosive spalling experienced in circular columns
3.5.2.2 Heating of square reinforced concrete columns

Fourteen reinforced concrete square columns (experimental part-1 and part-2) were heated in the same furnace, as described in Section 3.5.2.1. The furnace set up for heating of square columns is shown in Fig. 51. A maximum of six columns were heated at a time along with nine 100 mm cubes as control specimens similar to the circular columns, as shown in Figs. 52 and 53. The average heating rate used in the experiment was 150°C/hour similar to the circular columns. An unstressed residual strength test was also used in this study to evaluate the residual strength of square columns. The square columns were heated to a uniform temperature of 500°C. The temperatures within the furnace were controlled by two type-K thermocouples installed at mid-height and at the top of the electric furnace.

The reinforcing bars protruded from one end of all columns, as shown in Figs. 51 and 53 had the same two functions as described earlier in Section 3.5.2.1. The protruded top reinforcing bars were insulated with an Isofrax blanket, as shown in Fig. 52, at the time of heating. 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. After achieving the point of uniformity of temperature inside the furnace and at the centre of columns, the furnace was switched off. The columns and cubes were allowed to cool down naturally within the furnace. The cover of the furnace was lifted off when the temperature inside the furnace reached 243°C and 122°C in the cooling phase for experimental part-1 and part-2 columns respectively. The time-temperature curves for experimental part-1 and part-2 are shown in Figs. 54 and 55 respectively. After cooling, the columns were removed from the furnace and kept in a dry condition at room temperature in the laboratory until repairing and testing.
Fig. 51: Electric furnace set up and columns before going into furnace

Fig. 52: Insulation of top extended reinforcing bars with Isofrax blanket
Fig. 53: Square columns after heating in an electric furnace

Fig. 54: Time-temperature curve for square columns (experimental part-1) exposed to 500°C
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Table 16: Explosive spalling in square and circular columns cast from single batch of concrete mix

<table>
<thead>
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<th>Batch</th>
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<th>Circular columns</th>
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<td>Specimen</td>
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<td></td>
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</tr>
</tbody>
</table>

Fig. 55: Time-temperature curve for square columns (experimental part-2) exposed to 500°C
3.6 MOISTURE CONTENT

The weight of controlling concrete cubes for square and circular columns was measured before and after heating. The moisture content was calculated by subtracting the weight of post-heated cubes from the measured weight of un-heated cubes and divided by the weight of post-heated cubes. The measured moisture content in the controlling cubes for square and circular columns is shown in Table 17.

3.7 NON DESTRUCTIVE TESTING

Pundit with direct transmission mode was used to measure the ultrasonic pulse velocity of concrete adopting the procedure given in BS EN 12504-4:2004 [215]. After more than nine months of casting of concrete cubes and columns from the same mix and cured under identical conditions, pulse velocity measurements were made with Pundit in pre-heated and post-heated cubes and columns. Petroleum Jelly was used to make smooth surfaces both for concrete cubes and columns. The transmitting and receiving transducers were placed on the opposite faces of the concrete cubes and columns. The distance between the transducers was divided by the transit time to obtain the pulse velocity through the concrete. Three readings were taken in each cube.

In the reinforced concrete columns, the ultrasonic pulse velocity was measured at two positions top and middle and three readings were taken at each position. To avoid reinforcement, the positions of the transducers were selected between the spacing of link bars and main longitudinal reinforcement. The cubes and columns were placed in an electric furnace after measuring the ultrasonic pulse velocities, and heated up to the temperatures described in Sections 3.5.1, 3.5.2.1 and 3.5.2.2. The pulse velocity was measured again in the post-heated concrete cubes and columns on the 7th day of cooling after exposing to temperatures described in Sections 3.5.1, 3.5.2.1, and 3.5.2.2 at the same positions. The concrete cubes after measuring the pulse velocities were tested in a compression testing machine on the 7th day of cooling after exposing to various high temperatures.
<table>
<thead>
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<th>Test condition</th>
<th>Moisture content in square and circular columns</th>
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<tr>
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<td>Square columns</td>
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<tr>
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<td>Specimen</td>
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<tr>
<td>Post-heated (500°C)</td>
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</tr>
<tr>
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<td>S3</td>
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<td>S17</td>
</tr>
<tr>
<td></td>
<td>S18</td>
</tr>
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<td>Average moisture content</td>
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</tr>
</tbody>
</table>

Table 17: Moisture content in square and circular column
CHAPTER-4

4 REPAIRING AND TESTING PROCEDURE OF COLUMNS

4.1 INTRODUCTION

This chapter describes the repairing and testing procedure of circular and square columns of experimental part-1 (tested under axial loading) and experimental part-2 (tested under combined axial and reversed lateral cyclic loading). The test specimens in each part were categorised into three main groups, un-heated, post-heated and post-heated repaired columns. The post-heated columns of part-1 (tested under axial loading only) were repaired using the following number of techniques:

1. Epoxy resin mortar (for the seriously spalled circular columns)
2. Unidirectional Tyfo SEH-51A glass fibre reinforced polymer jackets (square and circular columns)
3. Unidirectional Tyfo SCH-41 carbon fibre reinforced polymer jackets (square and circular columns)
4. Unidirectional Weber.tec force C-240 carbon fibre reinforced polymer jackets (square and circular columns)
5. Ferrocement (square and circular columns)

The post-heated columns of experimental part-2 (tested under simulated gravity and earthquake loadings) were repaired using the following methods:

1. Unidirectional Tyfo SEH-51A glass fibre reinforced polymer (square and circular columns)
2. Unidirectional Tyfo SCH-41 carbon fibre reinforced polymer (square and circular columns)

4.2 REPAIRING OF SERIOUSLY SPALLED CIRCULAR COLUMNS

4.2.1 Repairing of explosive spalled circular columns with epoxy resin mortar

For the columns which spalled, as shown in Fig. 56 (a), an epoxy resin mortar was used for three columns before wrapping with GFRP or CFRP jackets. The loose concrete was removed with a steel wire brush and the concrete and steel substrate
was prepared according to the procedure given in BS EN1504 [229]. After surface preparation a primary coat of Weber tec.EP bonding aid was applied to the spalled surface of the column with a brush to achieve a good bond between the old and new concrete, as shown in Fig. 56 (b). The epoxy resin mortar was prepared by mixing the resin and hardener components in a clean plastic bucket.

The resin and harder components were mixed in the ratio according to the volumes of measuring cups provided by supplier. The resin and harder components were mixed thoroughly until uniform colour and consistency was achieved. The filler (special lightweight aggregates in powder form) was added in the bucket slowly and was mixed all the time. After preparation, the epoxy resin mortar was applied to the tacky primed surface with a steel trowel and was pressed firmly into position, as shown in Fig. 57. The surface was finished using a steel float and kept for curing for one month in the laboratory environment until testing, as shown in Fig. 57 (b).

4.2.2 Repairing of explosive spalled circular columns with concrete

Two seriously spalled columns were placed in an upright position and repaired with the original concrete mix which was used in the casting of all columns. The loose fire damaged concrete was removed with a steel wire brush. After surface preparation, a primary coat of Weber.tec.EP bonding aid was applied to the spalled surface of the column with a brush to achieve a good bond between the old and new concrete, as shown in Fig. 58 (b). A well oiled original PVC pipe mould which was used in the casting of all circular columns was placed around the spalled columns, as shown in Fig. 59. The concrete mix was filled into the mould in layers and was compacted using an internal poker vibrator to remove air voids. For each column, three cubes of 100 mm size were cast in order to monitor the concrete strength of columns at the time of testing. After 24 hours, the columns were taken out from the moulds. The repaired columns were wrapped with wet gunny bags and covered with a plastic sheet to prevent the loss of moisture, as shown in Fig. 60. The moist curing was continued for fourteen days. After fourteen days, the columns along with cubes were then air cured at room temperature in the laboratory environment until testing.
Fig. 56: (a) Explosive spalled columns; (b) Application of Weber.tec EP bonding aid
Fig. 57: (a)-(b) Repairing of seriously explosive spalled circular columns with Weber.tec EP highbuild mortar
Fig. 58: (a) Explosive spalled columns; (b) Application of Weber.tec EP bonding aid

Fig. 59: Placing of PVC mould around seriously spalled circular columns
4.3 REPAIRING OF POST-HEATED COLUMNS WITH FIBRE REINFORCED POLYMERS (FRP)

4.3.1 Circular reinforced concrete columns (Experimental Part-1)

The surface micro cracks, voids and cavities were filled with thickened Tyfo S epoxy. The epoxy (Component-A) and hardener (Component-B) were mixed in the ratio of 100 to 42 parts by volume in clean plastic bucket until uniform colour and consistency was achieved. The powder Cab-o-sil M5 was added slowly and was mixed all the time until a thick paste of epoxy was achieved. A thin layer of thickened epoxy was applied on the substrate of all columns to fill all voids, cavities and micro-cracks, as shown in Fig. 61 (a). After filling all voids and cracks, one primer coat of Tyfo S epoxy without Cab-o-sil M5 was thoroughly coated on the surface of three columns while Weber.tec force EP primer coat was applied on the surface of one column by using a brush, as shown in Fig. 61 (b). The columns after application of the primer coat were allowed to become tacky to touch.
The wet lay-up technique was used to wrap the commercially available Tyfo SEH-51A glass, Tyfo SCH-41 carbon and Weber.tec force C-240 carbon jackets around columns. The glass fibre was wrapped around two post-heated columns, one without any spalling and the other having serious explosive spalling following repair with a highbuild epoxy resin mortar. Weber.tec force C-240 carbon sheet was wrapped around a post-heated column without any spalling. Tyfo SCH-41 carbon jacket was wrapped around a column which was seriously spalled during heating and repaired with an epoxy resin mortar. The Tyfo SEH -51A glass (single piece), Tyfo SCH-41 carbon (two pieces) and Weber.tec force C-240 carbon (three pieces) fabric sheets were cut according to the perimeter and height dimensions of the column plus 200 mm extra for overlapping in the circumferential direction. An overlap of 100 mm was provided in the longitudinal direction for the FRP (when used in more than one piece). The Tyfo SEH -51A glass and Tyfo SCH-41 carbon fabric sheets were saturated on both sides with a prepared standard Tyfo S epoxy mix, as shown in Figs. 62 and 63. The standard Tyfo S epoxy was prepared by mixing the epoxy (Component-A) and hardener (Component-B) in the ratio of 100 to 42 parts by volume as described earlier.

The Weber.tec force C-240 carbon sheet was saturated with adhesive (harder and epoxy) mixed at 1:1 by volume. The saturated fabric sheets were wrapped around columns with main fibres placed in the circumferential direction when the surface of the columns became tacky to touch. Only one layer of Tyfo SEH -51A glass, Tyfo SCH-41 carbon and Weber.tec force C-240 carbon fibre reinforced polymer were wrapped around the columns. The entrapped air bubbles were removed with a roller, as shown in Fig. 64. The roller was used continuously until the layer of fibre with proper orientation was firmly bedded to the concrete surface of the column. A gap of 25 mm was provided between the fibre jackets and the column ends in order to prevent the jackets from direct axial loading. The wrapped specimens were left for curing in the laboratory environment at room temperature for at least one month before testing.
Fig. 61: (a) Application of thickened Tyfo S epoxy with Cab-o-sil M5 (b) Application of primer adhesive (Tyfo S epoxy and Weber.tec force EP primer)

Fig. 62: Saturating of Tyfo SEH-51A glass fabrics with adhesive (harder and epoxy)
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 63: Saturating of Tyfo SCH-41 carbon fabrics with adhesive (harder and epoxy)

Fig. 64: Removal of air bubbles from the FRP jacket wrapped around circular columns using roller and hand pressure
4.3.2 Square reinforced concrete columns (Experimental Part-1)

Before applying the FRP, the rounded corners and faces of the square columns were smoothed by grinding using an electric grinder, to avoid any damage to the GFRP or CFRP jackets, as shown in Fig. 65 (a). The surface of the heat damaged columns was cleaned with a steel wire brush to remove dust, grease and completely dried before the application of a primer coat. A thin layer of thickened epoxy with Cab-o-sil M5 powder (described earlier in Section 4.3.1) was applied to the substrate of columns to fill all voids, cavities and micro-cracks, as shown in Fig. 65 (b).

Following the application of the epoxy, once it had cured, a primer coat of Tyfo S epoxy was applied on the substrate of three columns and Weber.tec force EP primer was applied on the substrate of two columns using a brush, as shown in Fig. 65(c). All columns were allowed to become tacky to the touch. Three types of commercially available unidirectional fibres, Tyfo SEH-51A glass fibre, Tyfo SCH-41 carbon fibre and Weber tec force C-240 carbon fabric sheets were used for the repairing of post-heated square columns of experimental part-1. Unidirectional fibre reinforced polymer sheets were then wrapped around the heat damaged square columns using a wet lay up technique similar to circular columns (explained in Section 4.3.1), as shown in Fig. 66. 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 square columns. Irregularities and air pockets were removed using a roller, as shown in Fig. 66. The wrapped specimens were then air cured in the laboratory environment at room temperature for approximately one month prior to testing, as shown in Fig. 67.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 65 (a) Grinding of corners and faces of columns ;( b) Application of Cab-o-sil M5 mixed with epoxy; (c) Application of primer coat.

Fig. 66: Wrapping of GFRP and CFRP jackets around square columns
4.3.3 Circular reinforced concrete columns (Experimental part-2)

All loose concrete, dust and grease was removed with steel wire brush from the surface of post-heated circular columns similar to columns described in Section 4.3.1. The surface of post-heated columns was roughened by sandblasting. The voids, cavities and micro cracks on the substrate of the post-heated columns were filled by applying a thin layer of primer coat consisting of epoxy, hardener and Cab-o-sil M5 powder as described previously. After filling all voids and cracks, one primer coat of Tyfo S epoxy was applied in a similar way as described earlier in Section 4.3.1 on the substrate of the post-heated columns by using a brush and was allowed to become tacky to touch.

The manufacturer’s instructions were strictly followed for the application of GFRP or CFRP jackets. Two types of FRP, Tyfo SEH -51A glass and Tyfo SCH-41 carbon

Fig. 67: Curing of GFRP and CFRP wrapped square columns
were used in this study. The wet lay-up technique was used to wrap commercially available Tyfo SEH-51A glass or Tyfo SCH-41 composite carbon fibre jackets around the columns. The glass fibre was wrapped around two post-heated columns without any spalling. The carbon fibre was wrapped around two post-heated columns, one without any spalling and the other with explosive spalling following repair with concrete.

The epoxy (Component-A) and hardener (Component-B) were mixed in the ratio of 100 to 42 parts as described earlier. The fabric sheets were impregnated on both sides with a prepared standard Tyfo S Epoxy. When the surfaces of columns became tacky to touch, the impregnated glass or carbon fabric sheets were tightly wrapped around the columns with main fibres in the circumferential direction. Only a single layer of glass or carbon fibre reinforced polymer was used in this study. The GFRP or CFRP jackets were wrapped tightly around post-heated columns without any distortion. The entrapped air bubbles were removed using a roller. The wrapped specimens were left for curing in the laboratory environment at room temperature for about three months before testing, as shown in Fig. 68

4.3.4 Square reinforced concrete columns (Experimental part-2)

The same procedure was adopted for the surface preparation and filling of the cracks, voids and cavities as described earlier in Section 4.3.2. Two types of commercially available fibre reinforced polymers Tyfo SEH-51A unidirectional glass or Tyfo SCH-41 unidirectional carbon composites sheets were used for the repairing of post-heated square columns of experimental part-2. The Tyfo SEH-51A unidirectional glass or Tyfo SCH-41 unidirectional carbon composites sheets were wrapped around square columns in a similar fashion as described in Sections 4.3.1, 4.3.2 and 4.3.3.

In order to investigate the effect of the orientation of the main fibre direction, two columns were wrapped with carbon and one column was wrapped with glass fibre reinforced polymer with main fibres oriented in the transverse direction. The last column was wrapped with glass fibre reinforced polymer with main fibres oriented in
the longitudinal direction. The glass and carbon fibre reinforced polymer was wrapped along the full length of the columns and an overlap was provided in all columns in the lateral and longitudinal direction similar to columns described in Section 4.3.1. Only a single layer of Tyfo SEH-51A glass and Tyfo SCH-41 carbon fibre reinforced polymer was used in this study. The wrapped specimens were air cured in the laboratory environment for approximately three months prior to testing similar to circular columns, as shown in Fig. 68.

Fig. 68: Curing of FRP wrapped columns

4.4 REPAIRING OF POST-HEATED COLUMNS WITH FERROCEMENT

4.4.1 Circular reinforced concrete column (Experimental part-1)

The surface of the seriously spalled post-heated circular column repaired with concrete was cleaned with a steel wire brush to remove dust and grease before the application of a ferrocement jacket. After surface preparation, the post-heated circular
column was held in an upright position and a coat of polyvinyl acetate (PVA) was applied on the surface of the column to fill micro cracks and to provide a bond between the post-heated column concrete and the ferrocement laminate. The length and width of the wire mesh was cut according to the perimeter and height of column together with an additional 200 mm extra for overlapping from the roll of wire mesh. The column was wrapped with continuous four layers of wire mesh.

A wooden hammer was used to make the wire mesh tight and keep as close to the surface of column as possible. An overlap was provided over 200 mm length in the circumferential direction for the last layer. Fig. 69 (a) shows a column after wrapping with four layers of wire mesh and ready for the application of mortar. After wrapping with four layers of wire mesh, the cement sand mortar was forced into the mesh by hand pressure using a steel trowel to form a 16mm thick ferrocement jacket all around the column, as shown in Fig. 69 (b)-(c). A clear cover of approximately 2 mm was provided to the outer face of the jacket. A small gap of approximately 25 mm was left between the ferrocement jacket and the ends of the column by providing a layer of mortar to prevent the jacket from direct axial loading during test. Fig. 69 (d) shows the column after plastering.

Three cubes of 100 mm size were cast from the same mortar in order to monitor the mortar strength of ferrocement jacket at the time of testing. The column and mortar cubes were then cured under a wet canvas that was kept continuously moist with water for fourteen days. All specimens were covered with a plastic sheet for fourteen days to prevent the loss of moisture. After fourteen days, the column along with three cubes was then air cured at room temperature in the laboratory environment until testing.
Fig. 69: (a) Column ready for plastering after wrapping with four layers of wire mesh;
(b) Forcing of cement sand mortar into the mesh by hand pressure using steel trowel;
(c) Forcing of cement sand mortar into the mesh by hand pressure using steel trowel;
(d) Circular column after plastering.
4.4.2 Square reinforced concrete column (Experimental part-1)

The grease, dust and loose concrete from the surface of the post-heated square column were carefully removed with a steel wire brush before the application of the ferrocement jacket. The round corners of the post-heated square columns were further ground with an electric grinder as described earlier in Section 4.3.2 to reduce the stress concentration at the corners of the ferrocement jacket. After surface preparation, the post-heated square column was placed vertically and a coat of polyvinyl acetate (PVA) was applied to the surface of the column to fill micro cracks and to provide bond between the post-heated column concrete and the ferrocement laminate, similar to circular columns described in Section 4.4.1. The post-heated column was wrapped with four layers of wire mesh reinforcing steel. The lengths and width of wire mesh were cut according to the exact perimeter of three sides of square column in order to wrap them around the column in the form of an U-shape with 50 mm extra. The wire mesh sheet was bent in an U-shape using a sheet metal folder available within the structural engineering laboratory at the University of Manchester, as shown in Fig. 70 (a).

The U shape wire mesh was placed on three sides along the length of column. After placing the U-shape wire mesh sheet around the post-heated square column, the two ends of U-shape wire mesh sheet were rounded with a wooden hammer and extended up to 25 mm from each side on the fourth face, as shown in Fig. 70 (b). After wrapping the first layer of wire mesh around the column, the ends of first layer of wire mesh, at several places along the length of column were tied together using binding wire. A wooden hammer was used to firmly bed the U-shape wire mesh sheet onto the concrete surface and was tightened further using the binding wire on the fourth face of the column. The other U shape wire mesh sheet was placed on the opposite face and every effort was made to keep it as close to the surface of the column as possible using a wooden hammer. After placing opposite U-shape wire mesh sheet, the two ends of U-shape wire mesh sheets were again tied together along the length of the column at many places using binding wire. The third and fourth U shape wire mesh sheets were placed opposite to each other and transverse...
to the faces of previously applied U-shape sheets in order to ensure as even distribution of mesh as possible. The ends of both U-shape sheets were tightened by using the same technique as described earlier. The fifth and sixth U-shape wire mesh sheets were applied by adopting the same procedure. In each layer of wire mesh the ends of U-shape sheets were rounded with a hammer up to 25 mm length from both sides projected on the fourth face of the column, as shown in Fig. 70 (b).

Fig. 71 (a) shows the column after wrapping with single layer of mesh. Fig. 71 (b) shows the column after four layers of wire mesh and ready for application of mortar. A twenty-five millimetre gap was left between the ferrocement jacket and the ends of the column by placing a layer of mortar at the ends to prevent the strengthening jacket from carrying any direct axial loading during test. The cement sand mortar was carefully forced into the mesh opening with steel trowel using hand pressure to ensure full penetration of mortar into mesh, as shown in Fig. 72 (a)-(b). A 16mm thick ferrocement jacket was provided all around the column with 2 mm clear cover. The surface was finished using a steel float. Three cubes of 100 mm size were cast from the same mortar mix at the time of repairing of the column in order to monitor the mortar strength of the ferrocement jacket. The column and mortar cubes were then cured under identical conditions using moist canvas that was kept continuously wet with water. All specimens were covered with a plastic sheet for fourteen days to prevent the loss of moisture. After fourteen days the columns, along with three cubes, were air cured at room temperature in the laboratory environment until testing.
Fig. 70: (a) Bending of wire mesh sheet in a U-shape using a sheet metal folder; (b) Final shape of U-shape mesh
Fig. 71: (a) Wrapping of U-shape wire mesh around square column; (b) Column ready for plastering after wrapping with four layers of U-shape wire mesh.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 72: (a)-(b) Forcing of cement sand mortar into the mesh using hand pressure and steel trowel

(a)

(b)

(a) and (b) show the process of forcing cement sand mortar into the mesh using hand pressure and a steel trowel. The images illustrate the repair technique used for post-heated columns.
4.5 INSTRUMENTATION AND TESTING PROCEDURE FOR AXIALLY LOADED COLUMNS (EXPERIMENTAL PART-1)

4.5.1 Circular reinforced concrete columns (Axially loaded)

All columns were capped with dental plaster (Dentstone KD plaster) at their ends, top and bottom before testing to ensure parallel surfaces and to distribute the load uniformly. Each column was instrumented with three linear variable displacement transducers (LVDTs) along the longitudinal axis. All three LVDTs were placed at mid-height on all specimens over a total length of 275 mm. The LVDTs were attached between an aluminium angle and pin which were fixed to the column using a glue gun.

In addition to the LVDTs, all columns were also instrumented using four surface strain gauges, two in the longitudinal direction and the other two in the hoop direction, glued either onto the concrete surface for the control specimens or onto the fibre reinforced polymer sheet, or onto the outer surface of ferrocement jacket for the repaired specimens, as shown in Fig. 73. The gauges were bonded at the mid-height and on the opposite faces of each column. All strain gauges were 30 mm long and type PFL-30-11. The load was applied, using load control, at a loading rate of 1 kN/s. The columns were tested to failure under axial compression using a 2500 kN capacity compression testing machine and the data was monitored and logged using an automatic data acquisition system.
Fig. 73: Test set up and Arrangement of instruments (a) on concrete surface (b) on ferrocement surface (c) on GFRP surface (d) on CFRP surface
4.5.2  Square reinforced concrete columns (Axially loaded)

All of the square columns were instrumented with three vertical LVDTs (Linear variable displacement transducers) having a gauge length of 275 mm. The LVDTs were placed at the mid-height on all specimens to measure the axial shortening during the test over a 275 mm length similar to circular columns described in Section 4.5.1. Three horizontal LVDTs of 85 mm gauge length were also attached to each column on three faces to measure any transverse displacements. The axial and transverse strains at mid-height were also measured using surface strain gauges having a gauge length and type as explained earlier in Section 4.5.1. Prior to testing, a thin layer of dental plaster was placed on both ends of columns to ensure the contact surfaces were flat and parallel to distribute the load evenly during testing. The loading test setup is shown in Figs. 74 and 75. The specimens were loaded and tested using the load control method at a same loading rate in the same compression testing machine as described earlier in Section 4.5.1.

Fig. 74: Test up and Instrumentation (a) On concrete surface (b) On ferrocement surface
4.6 TEST SET UP AND LOADING PROCEDURE FOR CRITICAL SHEAR COLUMNS (EXPERIMENTAL PART-2)

4.6.1 Circular reinforced concrete columns (tested under combined axial and lateral reversal cyclic loading)

4.6.1.1 Test set up

All columns were tested in a vertical cantilever position using the test set up shown in Figs. 76 to 79. The lower ends of columns were securely tied to two heavy steel beams of the testing frame through a 40 mm thick stiff steel plate in order to avoid rotation or slip between the column base and platform surface. The columns were confined with a very rigid, high strength, 200 mm high steel pipe at the bottom. The steel pipe was designed and made of two separate halves of 200 mm height and 15 mm thickness with a radius to fit the columns. The steel pipe was welded with two
buttress supports on each side in the pulling and pushing directions of loading, as shown in Fig. 81. The two halves of the steel pipe that tightly fitted around the circular section of column, together with the 40 mm thick steel plate, provided the effective rotational confinement at the bottom of the columns.

To apply a horizontal cyclic load, 425 mm above footing, a wooden box made of two separate halves with a concentric hole that exactly fitted the specimen at the point of application of lateral loading was designed to distribute the load evenly around the perimeter of circular column. The cross-section of the wooden box used to confine the circular column at the level of lateral cyclic loading is shown in Fig. 80. The two halves of the wooden sections were pushed against the specimen and sandwiched in the direction of loading between two steel plates of size 200 mm \( \times \) 300 mm. The whole unit was connected together using six high strength steel bars. One side of the steel plate was connected with the pin of the hydraulic actuator to apply shear cyclic loading in the pushing and pulling directions. The reversed lateral cyclic load, representing seismic forces, was applied using a servo controlled hydraulic actuator to the columns fixed in a condition of vertical cantilever.

The axial load (representing gravity load) was applied to the columns through two high strength 25 mm diameter calibrated macalloy bars. The top ends of the macalloy bars were connected to a 40 mm thick steel plate fitted on top of the columns while the bottom ends of the macalloy bars were fixed to a high strength steel rod running through the bottom of the steel beams of the testing frame, as shown in Figs. 76 to 79. The axial tensile forces were applied to the macalloy bars evenly using a 100 kN capacity hydraulic jack and by tightening the bolts at the top of the mounted steel plate. The tensile forces in the macalloy bars were measured using two Vishay strain indicator controlling boxes connected with strain gauges fixed to each macalloy bar, as shown in Figs. 76 and 77. The tensile forces in the bars were transferred to apply a compression load to the column through the 40 mm thick steel plate mounted on top of the column, as shown in Figs. 76 to 79.
To apply horizontal load the actuator was fixed to the vertical two steel columns of the testing frame, as shown in Figs. 76 and 77. Two linear variable displacement transducers (LVDTs) having a 100 mm stroke length were used to measure the displacement at the application of lateral cyclic loading in order to account for the sway of the frame and displacement at the tip of column.

4.6.1.2 Loading procedure

All specimens were tested under similar loading condition. Each specimen was subjected to bidirectional loading at the same time, simulating the gravity and unidirectional seismic loading. The applied compressive axial load was 10% of the axial capacity of the column and was maintained constant throughout the test. The lateral cyclic load was applied by a lateral displacement method controlling the drift ratios using an Instron servo-controlled hydraulic actuator with a 250 kN load capacity and a ±125 mm stroke capacity. All columns were subjected to the same convenient reversal lateral loading history (simulating earthquake loading), as shown in Fig. 85. The lateral cyclic load was applied with the centre placed at 425 mm above the column support.

The actuator was operated by using the test Xpert CAT3 software. The first lateral reversal loading cycle was applied at a 0.5% drift ratio $\Delta/L$, for the start up of the actuator. Where ‘$\Delta$’ represents the lateral displacement and ‘L’ represents the height of column between the point of application of the lateral loading and top of the column support. Then; three repetitive fully reversed lateral cyclic loading were attempted there after at each peak drift ratios of $\Delta/L=1\%, 2\%, 3\%, 4\%, 5\%, 6\%, 7\%, 8\%, 9\%, 10\%, 11\%, 12\%$ etc. The drift cycles were automatically generated using a computerized load/drift control system and were increased gradually. Such a loading pattern was attempted until the peak of the lateral loading of the column under testing was reached and the lateral load started to decrease on further loading. All measurements were monitored using a data acquisition system.
Fig. 76: Test set up and Instrumentation for unwrapped circular columns tested under combined constant axial and reversal cyclic lateral loading
Fig. 77: Test set up and Instrumentation for FRP wrapped circular columns tested under combined constant axial and reversal cyclic lateral loading
Fig. 78: Testing frame set up for columns tested under combined axial and lateral reversal cyclic loading
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 79: Detail of set up of circular column for testing under combined constant axial and lateral reversal cyclic loading

Fig. 80: Cross-section of wood used to confine the circular column at the level of lateral reversal cyclic loading
4.6.2 Square reinforced concrete columns (tested under combined axial and lateral reversal cyclic loading)

4.6.2.1 Test set up

The experimental setup for the testing of square columns under bi-directional loading, which simulates the gravity and unidirectional seismic loading, is shown in Figs. 82, 83 and 86. The test specimens were cantilever columns. The bottom of the square columns was securely tied with two heavy steel beams of the testing frame through a 40 mm thick steel plate similar to circular columns, as shown in Fig. 78 discussed in Section 4.6.1.1, in order to avoid rotation or slip between the column base and platform surface. To attain this, the columns were placed on a 40 mm thick stiff steel plate and the steel plate was fixed with two heavy steel beams of the testing frame using high strength bolts. The base steel plate (40 mm thick) and the steel beams of the testing frame were strong enough to provide the rotational restraint to the columns.
Two separate steel plates (L-shape having 15 mm thickness and 200 mm height welded with two buttress supports) were placed on two opposite sides of the columns in the pulling and pushing direction of loading, as shown in Figs. 82 to 84 and 86. The two opposite steel plates were connected with each other by four high strength steel bolts and fixed with a 40mm thick base steel plate using six high strength bolts in order to hold the column at the bottom at the time of testing. The two stiff steel plates on opposite sides of column and 40 mm thick base steel plate may act as one unit and represents a strong steel stub, as shown in Fig. 84. This strong steel stub provided effective rotational confinement at the base up to 200 mm height of the specimens.

At the point of application of horizontal loading, the column was confined up to 200 mm height from two opposite sides with another two 15 mm thick 200 mm × 300 mm steel plates. The plates were connected to each other with six high strength steel bars to act as a one unit in order to confine the column at the level of cyclic loading. To apply horizontal shear loading on to the column the pin of the hydraulic actuator was connected with one of two steel plates at the centre. The distance from the centre of the hydraulic actuator pin to the interface of bottom column support was 425 mm (taken as shear span for horizontal loading). The gravity load was applied by a hydraulic jack of 100 kN capacity at the top of columns. To apply a horizontal load, the hydraulic actuator with 250 kN capacity was fixed with the vertical two steel columns of the testing frame, as shown in Figs. 82 and 83. Two linear variable displacement transducers (LVDTs) having 100 mm stroke length were used, one at the application of lateral cyclic loading to measure the sway of steel frame and other at the top of column to measure the displacement at the tip of column.

4.6.2.2 Loading procedure

The loading procedure for un-heated, post-heated and post-heated square columns following repaired with GFRP or CFRP jackets was similar to that described in Section 4.6.1.2.
Fig. 82: Testing frame set up for unwrapped square columns tested under combined axial and lateral reversal cyclic loading

Fig. 83: Testing frame set up for FRP wrapped columns tested under combined constant axial and lateral reversal cyclic loading
Fig. 84: Steel stub to confine reinforced concrete square column at bottom

Fig. 85: Cyclic loading history
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 86: Detail of set up of square column for testing under combined constant axial and lateral reversal cyclic loading
CHAPTER-5

5 RESULTS AND ANALYSIS OF RESIDUAL COMPRESSION STRENGTH AND ULTRASONIC TESTING

5.1 INTRODUCTION

This section addresses the visual inspection of cubes and reinforced concrete columns after heating, explosive spalling, analysis and results of residual compressive strength and ultrasonic testing. The variation of ultrasonic pulse velocity and residual compressive strength of cubes subjected to various temperatures is presented graphically.

5.2 VISUAL INSPECTION

5.2.1 Visual inspection of post-heated cubes

The surface and colour of cubes was carefully observed after removing from the furnace. There was no visible evidence of cracks on the surface of cubes heated up to 450°C. Concrete started to crack when cubes were subjected to 500°C and above. It is worth noting that these cracks were not very prominent on the 7th day after cooling. However, the cracks were more visible after 2 months of heating. This is due to the fact that for longer exposure to ambient air, the quicklime (calcium oxide) absorbed more moisture and converted to slaked lime (calcium hydroxide) resulting in more cracking and disintegration of concrete [146]. At 550°C significant cracks were observed on the surface of all cubes on the 7th day after cooling, as shown in Fig. 87 (e). This attributes to continuous crack formation due to high temperature [170]. The colour of all cubes was turned to pink when heated above 300°C.
Fig. 87: Cubes after heating to (a) 200°C (b) 300°C (c) 450°C (d) 500°C (e) 550°C.
5.2.2 Visual inspection of post-heated columns

The surface of the columns was carefully observed after removing from the furnace. The colour of all the columns was found to be pink and a few micro cracks were observed on the surface of each column. No evidence of spalling was observed in all the square columns. However, very serious explosive spalling was found in some circular columns.

5.3 EXPLOSIVE SPALLING

Although spalling was not the main parameter to be investigated, major explosive spalling was found in some circular columns during heating in the range of 300°C to 400°C furnace temperature. It is interesting to note that explosive spalling occurred only in some circular columns and no spalling was observed in the square columns, as shown in Figs. 44, 49, 50, 56(a) and 58(a). The square and circular columns were cast from the same concrete mix, and even some square and circular columns were cast exactly from the single batch of concrete, as shown in Table 16. The only difference between the circular and square columns was the concrete cover (30 mm in the circular columns and 25 mm in the square columns), the cross-sectional area (31416 mm² in the circular and 40000 mm² in the square), and very minor difference in percentage of longitudinal reinforcement ratio (1.5% in the circular and 1.6% in the square).

The link bars provided in both the circular and square columns had the same diameter and were spaced at the same distance. The only difference was that the link bars in the square columns were anchored with a 135° hook at each end and extended 60 mm into the concrete core. In the circular columns a 60 mm overlap was provided to the link bars at the ends without any extension into the concrete cores, as shown in Figs. 32 and 36. The measured moisture content ranged between 3.4 to 3.8% by weight in both the square and circular columns after more than nine months of casting, as shown in Table 17.
A total of fourteen square and thirteen circular columns were heated. The test procedure for heating the columns was such that six columns were heated at the same time and at the same heating rate. Only five circular columns seriously spalled during heating while the other eight within the furnace under identical conditions did not spall. For all the square columns no any spalling occurred. The same findings were observed by Khoury and Anderberg [194] who critically reviewed the research carried out over a number of years. There are many factors contributing in explosive spalling but still the process, and the probabilistic nature of spalling, is debateable [194]. The combination of thermal stresses and the pore pressure stresses have a key role in increasing the strain energy within the gel pores of the concrete [194, 195]. Any internal cracking and moisture content has significant contribution in producing explosive spalling [194]. As moisture content increases the resulting pore pressure will increase the probability of spalling. The micro cracks also accelerate the process of spalling by providing sources for crack propagation [194]. In the present study, it was observed that local micro surface shrinkage cracks were present in some circular columns before heating. The micro shrinkage cracks together with the 3.4 % to 3.8% moisture content, as shown in Table 17, could have initiated explosive spalling although the exact reason why only five circular columns spalled is unknown.

5.4 RESIDUAL COMPRESSIVE STRENGTH OF POST-HEATED CONCRETE

The residual compressive strength values obtained from the compressive strength tests of the concrete specimens on the 7th day of cooling in ambient air after exposing to various high temperatures are shown in Fig. 88. The residual compressive strength was calculated as the percentage compressive strength of post-heated concrete cubes with respect to un-heated concrete cubes. In this experimental work, all specimens were tested at room temperature after cooling without any application of compressive loads at the time of heating. Since the strength reduction in unstressed concrete after cooling is greater than stressed and unstressed concrete at elevated temperature [147], therefore, it is believed that the unstressed residual strength test is more conservative for assessing the post-fire or residual properties of concrete
[141, 145, 147, 164]. Since the compressive strength is reduced further during cooling and the minimum strength is found up to a week after fire [141], therefore, all specimens were tested on the 7th day of cooling. At 450°C and 550°C, 100 mm and 150 mm concrete cubes were tested to investigate the effect of specimen size. Tables A.3 and A.4 (Appendix) show all the data points measured for compressive strength of each cube. It is worth highlighting that the residual compressive strength of post-heated cubes with different size has no evident difference, as shown in Table 20. This supports the previous findings of Arioz [170].

No evidence of any visible cracks was found in the cubes subjected to temperatures in the range of 200 to 450°C. However, minor cracks were initiated when cubes were subjected to 500°C. At 550°C significant visible cracks were seen on the surface of all cubes, as highlighted in the circles in Fig. 87 (e). It can be seen from Fig. 88 and Table 20 that the residual unstressed strength was slightly affected up to 300°C. Above 450°C it dropped sharply due to loss of bond between the aggregate and the cement paste. The residual compressive strength on the 7th day of cooling after heating to 200°C, 300°C, 450°C, 500°C and 550°C was found to be 80%, 76%, 60%, 47% and 30% respectively, as shown in Fig. 88 and Table 20. It is worth highlighting that the residual compressive strength of 100 mm controlling cubes of square and circular columns subjected to 500°C was further reduced approximately up to 8% when tested after 2-3 months of cooling in ambient air, as shown in Tables 8 and 9. This is likely due to the fact that for longer exposure to ambient air the quicklime (calcium oxide) absorbed moisture and converted to slaked lime (calcium hydroxide) resulting in more cracking and reduction in residual compressive strength of concrete [146].

5.5 ULTRASONIC TESTING

5.5.1 Cubes

Fig. 89 shows the results of pulse velocity measurements of concrete cubes on the 7th day of cooling after exposing to 200°C, 300°C, 450°C, 500°C and 550°C. Each point in Fig. 89 represents the average of three ultrasonic pulse velocities. However,
the Tables A.3 and A.4 (Appendix) indicate all the data points of the measured pulse velocity for each cube. The ultrasonic pulse velocities on the 7th day of cooling after heating to 200°C, 300°C, 450°C, 500°C and 550°C were found to be 4.6 km/s, 4.1 km/s, 3.9 km/s, 3 km/s, 2.3 km/s and 1.1 km/s respectively, as shown in Fig. 89 and Table 18. From Fig. 89 and Table 18, it is evident that the ultrasonic pulse velocity of post-heated concrete decreases markedly with increasing temperature and the reduction in ultrasonic pulse velocity is notably above 450°C. This indicates that up to 450°C, the residual unstressed strength reduced gradually and above this temperature, it dropped sharply. This is due to the fact that above 450°C, the calcium silicates commence to decompose into quick lime and silica. This process is irreversible and there is a progressive loss of strength [146]. It is worth mentioning here that there exist very low pulse velocity values labelled as a star mark in the Tables 18, 19, 24 and 25. This could be due to the fact that the development of extensive cracks in post-heated concrete prevented the propagation of ultrasonic pulse velocity resulting in abnormal values [230, 231]. From Fig. 89 it can be seen that the trend of reduction in pulse velocity is similar as it was observed in the reduction of residual compressive strength of concrete, as shown in Fig. 88.

5.5.2 Columns

Tables 21 to 25 show the measured values of ultrasonic pulse velocities of un-heated and post-heated columns and their corresponding controlling cubes. The pulse velocity values were measured at the middle and at the top position of columns and at the middle positions of controlling cubes. Each value presented in the Tables 21, 22, 24 and 25 represents the average of the three readings taken at the middle and at the top positions of pre-heated, post-heated columns and cubes. The Tables 24 and 25 show that the values of ultrasonic pulse velocities measured at the middle positions of both post-heated square and circular columns were slightly lower than the values measured at the top positions of columns. This could be due to more cracking in the middle than the top positions of the columns. The measured values of the ultrasonic pulse velocities in post-heated circular columns were slightly lower than post-heated square columns. This is likely due to more cracking in circular than
square columns. The measured values of pulse velocities in pre-heated columns and cubes were found in the range of 4.76 km/sec to 4.54 km/sec. This could be due to the fact of the difference in moisture content [231], as shown in Tables 17 and 19. The pulse velocity measurements were made in pre-heated and post-heated columns and 100 mm controlling cubes. It was observed that the effect of shape and size of specimens on pulse velocity values was negligible, as shown in Tables 21 to 25.

5.6 RELATIONSHIP BETWEEN RESIDUAL COMPRESSION STRENGTH AND ULTRASONIC PULSE VELOCITY

Fig. 90 demonstrates the relationship between the ultrasonic pulse velocity and residual compressive strength of concrete cubes on the 7th day of cooling in ambient air subjected to 200°C, 300°C, 450°C, 500°C and 550°C. The pulse velocity measurements were also made in pre-heated and post-heated columns along with their corresponding 100 mm controlling cubes, as shown in Tables 21, 22, 24 and 25. In post-heated columns and their corresponding post-heated cubes subjected to 500°C, the ultrasonic pulse velocities were measured on the 7th day of cooling in ambient air. A relationship between the residual compressive strength and pulse velocity was developed for concrete cubes subjected to 200°C, 300°C, 450°C, 500°C and 550°C cast from the same batch of concrete mix which was used for casting the reinforced concrete columns. This relationship has been verified for ultrasonic pulse velocities for reinforced concrete columns and for their controlling cubes heated to 500°C. The quality of concrete can be classified on the basis of pulse velocity as specified in [232]

<table>
<thead>
<tr>
<th>Pulse Velocity (km/s)</th>
<th>Quality of Concrete</th>
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</thead>
<tbody>
<tr>
<td>&gt;4.5</td>
<td>Excellent</td>
</tr>
<tr>
<td>3.5-4.5</td>
<td>Good</td>
</tr>
<tr>
<td>3.0-3.5</td>
<td>Doubtful</td>
</tr>
<tr>
<td>2.0-3.0</td>
<td>Poor</td>
</tr>
<tr>
<td>&lt;2.0</td>
<td>Very poor</td>
</tr>
</tbody>
</table>

According to the above classification, the unheated concrete specimens having pulse velocities of more than 4.5 km/s would be considered as in excellent condition.
Concrete samples subjected to 200°C and 300°C temperature have pulse velocities in the range of 4.1 km/s to 3.9 km/s, therefore would be considered as in good condition. The samples heated to 300°C and 450°C have pulse velocity values 3.9 km/s to 3 km/s, therefore it would be considered in doubtful and poor condition. The concrete subjected to 500°C and 550°C have pulse velocities in the range of about 2.3 km/s to 1.1 km/s and it would be considered in a poor and very poor conditions.

The relationship between the pulse velocity and compressive strength proposed by various investigators can vary between concrete mixes, due to the different type of aggregates, aggregate-cement ratio, water-cement ratios, age of concrete, size and grading of aggregate, curing conditions, air content, moisture content and density [175, 215, 231, 233]. It was found that due to the dehydration of concrete by driving out of moisture content at elevated temperature and micro cracking, there was notable difference in pulse velocities for a given compressive strength of fire damaged and undamaged concrete [234]. It was also observed that the reinforcing bars running transverse to the pulse velocity measuring path had no significant effect upon measured pulse velocities values. However, the pulse velocities measured values were affected considerably by the presence of reinforcing bars running along the pulse velocity measuring path [231]. Based on the results shown in Tables 18, 21-25 it is evident that approximately the same ultrasonic pulse velocities values were found in 100 mm cubes, 150 mm cubes without any reinforcement, 200 mm square and 200 mm diameter circular reinforced concrete columns at ambient and after subjecting to 500°C. This indicates that ultrasonic pulse velocity was not affected by the size, shape of specimen and the presence of longitudinal reinforcement.

In reinforced concrete columns normally higher concrete strength is used as compared to beams and slabs. Therefore, in this study 54 MPa cube strength was selected considering the practical utility of concrete in columns. Measuring directly the ultrasonic pulse velocities from existing fire damaged concrete column without
extracting cores gives the picture of the loss of residual strength of concrete. The ultrasonic pulse velocity values of 4.6 km/s indicate the strength of concrete in reinforced concrete column is in excellent condition and its cubic compressive strength would be considered approximately 54 MPa. The ultrasonic pulse velocities 4.1 km/s to 3.9 km/s shows that the concrete in reinforced concrete column is in good condition and its residual value would be considered in the range of 80% to 76% of its original value. If the pulse velocity value is in the range of 3.9 km/s to 3 km/s, the concrete of the reinforced concrete column would be in doubtful and poor condition. The residual compressive strength of such concrete would in the range of 76% to 60% of its original value. If the ultrasonic pulse velocity value is in the range of 3 km/s to 2.3 km/s, the residual compressive strength of fire damaged concrete in a reinforced concrete column would be considered poor in the range of about 60% to 47% of its original value. The 1.1 km/s ultrasonic pulse velocity value indicates very poor quality of concrete in fire damaged reinforced concrete columns.

![Fig. 88: Effect of temperature on residual compressive strength](image-url)
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 89: Effect of temperature on pulse velocity

Fig. 90: Relationship between residual compressive strength and pulse velocity
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
<thead>
<tr>
<th>Test conditions</th>
<th>Pulse velocity [km/sec]</th>
<th>Compressive strength [MPa]</th>
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<tr>
<td></td>
<td>150 mm cubes</td>
<td>100 mm cubes</td>
</tr>
<tr>
<td>Un-heated</td>
<td>4.59</td>
<td>4.68</td>
</tr>
<tr>
<td>Post-heated (200°C)</td>
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<td>-</td>
</tr>
<tr>
<td>Post-heated (300°C)</td>
<td>3.95</td>
<td>-</td>
</tr>
<tr>
<td>Post-heated (450°C)</td>
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<td>3.02</td>
</tr>
<tr>
<td>Post-heated (500°C)</td>
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<td>2.40</td>
</tr>
<tr>
<td>Post-heated (550°C)</td>
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<td>1.18*</td>
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</table>

Table 18: Pulse velocity and compressive strength of cubes

<table>
<thead>
<tr>
<th>Test conditions</th>
<th>Pulse velocity [km/sec]</th>
<th>Moisture content [%]</th>
</tr>
</thead>
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<tr>
<td></td>
<td>150 mm cubes</td>
<td>100 mm cubes</td>
</tr>
<tr>
<td>Un-heated</td>
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</tr>
<tr>
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<tr>
<td>Post-heated (300°C)</td>
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<td>2.40</td>
</tr>
<tr>
<td>Post-heated (550°C)</td>
<td>0.98*</td>
<td>1.18*</td>
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</tbody>
</table>

Table 19: Moisture content in cubes

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<th>Test conditions</th>
<th>Compressive strength [MPa]</th>
<th>Average Compressive strength (%)</th>
<th>Residual Compressive strength (%)</th>
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<tr>
<td></td>
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<td>Un-heated</td>
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<tr>
<td>Post-heated (200°C)</td>
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<td>Post-heated (300°C)</td>
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<td>Post-heated (450°C)</td>
<td>32</td>
<td>33</td>
<td>32.5</td>
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<td>Post-heated (500°C)</td>
<td>25</td>
<td>26</td>
<td>25.5</td>
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<tr>
<td>Post-heated (550°C)</td>
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Table 20: Residual compressive strength of cubes
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
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<th>Test condition</th>
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<td>Specimen</td>
<td>Column position</td>
<td>Cubes</td>
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<tr>
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<td></td>
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<td>Middle</td>
<td>Middle</td>
</tr>
<tr>
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<td>4.76</td>
<td>4.76</td>
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<td></td>
<td>S2</td>
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<td>S18</td>
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<td>Standard deviation</td>
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<td></td>
<td>Overall standard deviation</td>
<td>0.077898</td>
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Table 21: Pulse velocity measurements in pre-heated square columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

### Table 22: Pulse velocity measurements in pre-heated circular columns

<table>
<thead>
<tr>
<th>Test condition</th>
<th>Pulse velocity [km/sec]</th>
<th>Cubes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular columns</td>
<td>Specimen</td>
<td>Column position</td>
</tr>
<tr>
<td>Un-heated</td>
<td>C1</td>
<td>4.72</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>4.66</td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>4.58</td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>4.58</td>
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<td>Average pulse velocity</td>
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<td>Standard deviation</td>
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<td>Overall standard deviation</td>
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### Table 23: Average pulse velocity measurements in square and circular columns

<table>
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<th>Average Pulse Velocity (Km/Sec)</th>
<th>Square Columns</th>
<th>Circular Columns</th>
<th>Cubes</th>
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<td>Before heating</td>
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<td>Average</td>
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<tr>
<td>Average compressive strength(MPa)</td>
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Table 22: Pulse velocity measurements in pre-heated circular columns

Table 23: Average pulse velocity measurements in square and circular columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
<thead>
<tr>
<th>Test condition</th>
<th>Pulse velocity [km/sec]</th>
<th>Square columns</th>
<th>Column position</th>
<th>Cubes</th>
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<td>Middle</td>
</tr>
<tr>
<td></td>
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<td>S18</td>
<td>2.79</td>
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Average pulse velocity 2.50 2.30 2.5
Standard deviation 0.157651 0.158046 0.347079
Overall standard deviation 0.253715

Table 24: Pulse velocity measurements in post-heated square columns

<table>
<thead>
<tr>
<th>Test condition</th>
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<th>Circular columns</th>
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<th>Cubes</th>
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<td>Middle</td>
</tr>
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<td>1.94</td>
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<td>C17</td>
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Average pulse velocity 2.40 2.0 2.60
Standard deviation 0.226665 0.146734 0.287487
Overall Standard deviation 0.352853

Table 25: Pulse velocity measurements in post-heated circular columns
CHAPTER-6

6 ANALYSIS OF RESULTS AND DISCUSSION OF AXIALLY LOADED COLUMNS (EXPERIMENTAL PART-1)

6.1 INTRODUCTION

In this chapter, the test observations of axially loaded columns are presented detailing key events throughout each test. The main variables investigated in this part of the study were the shape of columns, the presence of heat damage, and type of composite repairing materials (FRP, ferrocement). The results are presented graphically in terms of load-deformation and stress-strain response based on the test data. The axial compressive behaviour of repaired columns in terms of their stiffness, ductility, ultimate strain and the ultimate strength is evaluated and compared to those of the control original un-heated and post-heated reference columns.

6.2 TEST OBSERVATIONS AND FAILURE MODE

6.2.1 Circular columns (repaired with epoxy resin mortar, FRP jackets, ferrocement jacket, tested under axial compressive loading)

All columns were tested under axial compression in a similar manner. For all columns a typical crushing failure mode was observed. It was noted that all columns failed at their ends due to the effect of stress concentration in these regions [11]. The failure of unheated columns was sudden and explosive with a loud booming noise, indicating a typical brittle mode of failure. However, the failure of the heat damaged column was ductile. The visual inspection of damaged columns showed that the reinforcing ties were opened at the overlap position during testing, as shown in Figs. 91 and 92. The overlap of 60 mm within the link bars without any hook and extension into the concrete cores was not sufficient to provide proper confinement. The failure of the seriously spalled, mortar repaired, column was ductile. The visual inspection of the failure of the mortar repaired column showed a good bond between the old
concrete and epoxy resin mortar, indicating that no debonding took place at any stage throughout the loading process, as shown in Fig. 93.

For the glass fibre repaired columns, prior to failure, a cracking noise was observed, indicating the full activation of the glass fibre reinforced polymer jacket. The failure was gradual and ended with a sudden and explosive noise. It was characterized by crushing of the concrete followed by rupture of the glass fibre reinforced polymer at the top end of the specimen. It was observed that the glass fibre jacket ruptured at a different location to the overlap position at the top end, as shown in Figs. 94 and 95. For the carbon fibre wrapped columns, the similar cracking sound was observed prior to failure manifesting the activation of the carbon fibre reinforced polymer jacket. The failure of CFRP wrapped columns was notably more violent and explosive than the GFRP wrapped columns. Again, the rupture of the carbon fibre jacket took place other than at the overlap position either at the top or bottom ends, as shown in Figs. 96 and 97. The sudden and explosive nature of the failure indicates the release of an immense amount of energy as a result of the uniform confining stress provided by the fibre jacket. The visual inspection of the damaged columns showed good contact between the jacket and the concrete indicating that no de-bonding took place at any stage during the test.

In the case of the post-heated ferrocement repaired column, a gentle cracking sound was observed approaching the failure load. The failure was initiated close to the bottom end of the column at the corner. The vertical cracks were initiated close to the bottom end and the mortar cover over the wire mesh started to spall. The wire mesh started bulging out with a cracking sound towards the ultimate stage of loading. The failure occurred without a sudden and explosive noise indicating very ductile behaviour of the column. The failure mode of the post-heated column repaired with ferrocement jacket is shown in Fig. 98. The visual inspection of the damaged column indicated that there was no deboning between the concrete core and the ferrocement jacket. The failure mode clearly demonstrated that several wires in the hoop direction were broken while wires in vertical directions were buckled and broken. This could be
due to the fact that under axial compression, the ferrocement jacket was subjected to hoop tension while the concrete core was subjected to radial compression, thereby producing passive confinement pressure [125, 130]. The failure of the column was mainly due to the development of vertical cracks in the jacket and yielding of hoop wires due to hoop tension.

Fig. 91: Crushing failure of un-heated circular columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 92: Crushing failure of post-heated circular columns

Fig. 93: Crushing failure of post-heated seriously spalled epoxy resin mortar repaired circular column
Fig. 94: Crushing failure of Tyfo SEH-51A GFRP wrapped post-heated circular columns

Fig. 95: Crushing failure of post-heated seriously spalled epoxy resin mortar and Tyfo SEH-51A GFRP wrapped circular columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 96: Crushing failure of Weber.tec force C-240 CFRP wrapped circular columns

Fig. 97: Crushing failure of post-heated seriously spalled epoxy resin mortar and Tyfo SCH-41 CFRP wrapped columns
6.3 ANALYSIS OF RESULTS AND DISCUSSIONS

6.3.1 Circular columns (repaired with epoxy resin mortar, FRP jackets, ferrocement jacket and tested under axial loading)

6.3.1.1 Effect of mortar, ferrocement, CFRP and GFRP jackets on ultimate strength of columns

The post-heated circular columns were repaired using a number of techniques comprising epoxy resin mortar, ferrocement, unidirectional glass and carbon fibre reinforced polymer jackets. Fig. 99 shows the average strength of the tested columns. The numbers 1 to 8 in the figure represent:

1) The average of two unheated columns (C1&C2)
2) One post-heated column (C3).

Fig. 98: Crushing failure of post-heated seriously spalled concrete and ferrocement repaired columns
3) One post-heated column wrapped with GFRP (single layer of Tyfo SEH-51A GFRP jacket)(C4)
4) One post-heated column which spalled and was repaired with an epoxy resin mortar(C5)
5) One post-heated column which spalled and was repaired with an epoxy resin mortar and wrapped with GFRP (single layer of Tyfo SEH-51A GFRP jacket)(C7)
6) One post-heated column and wrapped with CFRP (single layer of Weber.tec. force C-240 CFRP jacket)(C6)
7) One post-heated column which spalled and was repaired with an epoxy resin mortar and wrapped with CFRP (single layer of Tyfo SCH-41 CFRP jacket)(C8)
8) One post-heated column which spalled and was repaired with concrete and wrapped with a ferrocement jacket (four layers of wire mesh)(C9)

From Fig. 99 and Table 26, it can be seen that 42% of strength was lost after heating. The post-heated columns repaired with a single layer of unidirectional GFRP or CFRP regained the same strength or even higher than that of the unheated columns. This is likely to the fact that when a post-heated column is subjected to axial load it expands laterally, as shown in Fig. 126, which causes tensile stress in the fibre reinforced polymers jackets and confines the concrete and thus keeps the concrete in a state of three-dimensional stress [220]. Due to this three-dimensional state of stress, the load-carrying capacity of post-heated columns was increased significantly. It is interesting to note that the strength of a post-heated column wrapped with a single layer of Tyfo SEH-51A GFRP jacket (Test 3) was increased 29% higher than an unheated column (Test 1) and 122% higher than the post-heated column (Test 2). This is due to the fact that under axial compression the lateral expansion in a post-heated concrete is greater due to the heat damaged concrete being ‘softer’ thus allowing more tensile stress to be produced in the GFRP. This can be seen by comparing the hoop strains in the unheated columns (Test 1) with the hoop strains in post-heated column (Test 2) in Fig. 102. Therefore for post-heated columns the adequate strength of the GFRP jacket was utilized in increasing the strength. The strength of a post-heated seriously spalled column repaired with epoxy resin mortar
(Test 4) was increased 15% higher than a post-heated column (Test 2). The strength of a seriously spalled post-heated column repaired with both epoxy resin mortar and Tyfo SEH-51A GFRP jacket (Test 5) was found to be 10% higher than an unheated reference column (Test 1) and 65% higher than a mortar repaired reference column (Test 4).

It is interesting to note that the lateral dilation in the mortar repaired column (Test 4) was reduced due to the stiff mortar as compared to post-heated columns without any repair (Test 2), as shown in Fig. 104, resulting in an overall increase in strength. The post-heated column wrapped with a single layer of Weber.tec force C-240 CFRP jacket (Test 6) regained approximately the same strength as the unheated columns (Test 1). The post-heated seriously spalled column repaired with epoxy resin mortar and wrapped with a single layer of Tyfo SCH-41 CFRP jacket (Test 7) regained strength 20% higher than the original strength of the reference unheated columns (Test 1) and 80% higher than epoxy resin repaired spalled column (Test 4). It is noteworthy that the strength regained by the mortar and Tyfo SCH-41 CFRP jacket repaired column (Test 7) was higher than the column repaired with both mortar and Tyfo SEH-51A GFRP jacket (Test 5). This is attributable to the higher modulus of elasticity of the Tyfo SCH-41 CFRP jacket (Table 11). Overall, Fig. 99 shows that the strength of the post-heated column without spalling, and the strength of post-heated seriously spalled columns, could be restored up to the original or even higher than that of unheated columns.

It is worth noting that the post-heated seriously spalled column repaired with both concrete and with a ferrocement jacket (Test 8) also regained approximately the same strength as the unheated columns (Test 1). This increase in load carrying capacity attributes to the following reasons: under axial compression, the ferrocement jacket was subjected to hoop tension while the concrete core was subjected to radial compression, thereby producing passive confinement pressure until failure, resulting in increasing the axial capacity of columns [125, 130]. It also attributes to the
increase of cross sectional size, improvement of dimensional stability and integrity of the material (ferrocement jacket) [130].

The link-bars provide confinement to columns and influence the ultimate strength. It was interesting to note that during testing of the unheated and post-heated columns the link-bars were observed to open at the overlap position (Figs. 91 and 92). The same behaviour was also observed in the column repaired with both mortar and GFRP jacket, however the failure load was higher and defined by the failure of the FRP (Figs. 94 and 95). This suggests that the type of lateral steel tie confinement becomes insignificant when the columns are confined by both steel ties and FRP jackets.

6.3.1.2 Effect of mortar, ferrocement, CFRP and GFRP jackets on stiffness

Fig. 101 shows the effect of heat exposure, mortar repaired, wrapping of single layer of carbon or glass fibre reinforced polymer jackets and wrapping of the four layer of wire mesh ferrocement jacket on the stiffness of columns. The secant stiffness was
calculated by dividing the ultimate measured compressive load by the ultimate deformation measured at the mid-height of columns, as shown in Fig. 100. After heating to 500°C the stiffness of the post-heated column (Test 2) was reduced drastically as compared to unheated columns (Test 1). This is attributed to micro cracking and softening of the concrete after heating, with the concrete becoming more porous due to the evaporation of water.

It is interesting to note that the effect of GFRP or CFRP on the overall stiffness of post-heated columns (Tests 3 and 6) was negligible. This could be due to the fact that the confinement of the GFRP or CFRP jackets had nominal effect in the elastic range of loading [115]. The GFRP or CFRP confinement was fully activated after reaching the maximum strength of post-heated unconfined concrete and applies continuous pressure on the concrete core until the rupture of GFRP or CFRP jackets. Therefore, the behaviour of post-heated GFRP or CFRP wrapped columns was similar to that of the post-heated columns without wrapping, as shown in Figs. 103, 105 to 107 and 109. The stiffness of the seriously spalled column repaired with epoxy resin mortar (Test 4) was similar to the post-heated column (Test 2).

It is worth noting that the stiffness of a post-heated column repaired with ferrocement jacket (Test 8) was increased considerably as compared to the stiffness of a post-heated column (Test 2) but did not approach the level of pre-heated columns (Test 1). The stiffness of a post-heated column wrapped with a ferrocement jacket (Test 8) was improved up to 69% of the un-heated column (Test1), as shown in Fig. 101 and Table 26. The stiffness of a post-heated column wrapped with a ferrocement jacket was increased by 150% of the stiffness of a post-heated column (Test 2). This attributes to an increase of cross-sectional size, improvement of dimensional stability and integrity of the material [129, 130]. Comparing Figs. 99 and 101, it is noteworthy that the reduction in residual stiffness of the heat damaged column was more than the reduction in ultimate load. Therefore consideration should also be given to deformation and stress redistribution within reinforced concrete buildings following a fire. It can be seen that if failure is by crushing, the use of GFRP or CFRP can
increase the compressive strength of post-heated columns tremendously, but their use cannot restore the stiffness of post-heated columns. However, the use of ferrocement jacket improves the strength as well as the stiffness of post-heated columns.

\[ k = \frac{P}{\Delta} \]

Fig. 100: Secant stiffness

Fig. 101: Comparison of secant stiffness of circular columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
<thead>
<tr>
<th>Test No.</th>
<th>No. of columns tested</th>
<th>Column condition</th>
<th>Moisture content (%)</th>
<th>Secant stiffness (refer Fig. 100) (kN/mm)</th>
<th>Failure Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>Without heating</td>
<td>-</td>
<td>2715 2794</td>
<td>1439 1397</td>
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<tr>
<td>2</td>
<td>1</td>
<td>After heating without spalling</td>
<td>3.4</td>
<td>758</td>
<td>826</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>After heating without spalling wrapped with glass fibre jacket (Tyfo SEH-51A GFRP)</td>
<td>3.4</td>
<td>548</td>
<td>1834</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>After heating but serious explosive spalling repaired with epoxy resin mortar</td>
<td>3.6</td>
<td>763</td>
<td>946</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>After heating but with serious spalling repaired with epoxy resin mortar and wrapped with glass fibre jacket (Tyfo SEH-51A GFRP)</td>
<td>3.8</td>
<td>741</td>
<td>1557</td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>After heating without spalling wrapped with carbon fibre jacket (Weber tec force C-240 CFRP)</td>
<td>3.7</td>
<td>522</td>
<td>1356</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
<td>After heating but with serious spalling repaired with epoxy resin mortar and carbon fibre jacket (Tyfo SCH-41 CFRP)</td>
<td>3.8</td>
<td>842</td>
<td>1701</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>After heating but with serious spalling repaired with concrete and ferrocement jacket</td>
<td>3.8</td>
<td>1891</td>
<td>1398</td>
</tr>
</tbody>
</table>

Table 26: Summery of test results of circular columns

6.3.1.3 Effect of mortar, ferrocement, CFRP and GFRP jackets on ultimate stress-strain

The average axial stress plotted against axial and hoop strains for all the test columns are shown in Figs. 102 to 108, where the hoop and axial strains were
measured at the mid-height of all columns. The variations of strains within each specimen are shown in Figs. A.30 to A.33 (Appendix). The axial stress was calculated from the measured axial load divided by the concrete cross-sectional area in which the contribution from the GFRP or CFRP was ignored. However, the contribution of a ferrocement jacket in increasing the size of column was considered in the calculations. In Figs. 102 to 109, the curves C1, C3, C4, C5, C6, C7, C8, C9 refers to stress-strain relationship relating to the test numbers listed in Section 6.3.1.1

Fig. 102 shows the comparison between unheated (Test 1) and post-heated columns (Test 2), where it can be seen that the axial stress was reduced drastically while the axial and hoop strains were increased significantly after exposing the columns to a temperature of 500°C. As previously explained, this can be attributed to the fact that concrete after heating becomes ‘soft’ and porous due to the development of micro-cracking and the removal of water. Fig. 103 compares the stress-strain relationship of a post-heated column wrapped with a single layer of Tyfo SEH-51A GFRP jacket (Test 3) with the unheated (Test 1) and post-heated (Test 2) columns. It can be seen that the axial stress, axial strains and hoop strains were increased tremendously in the GFRP wrapped column (Test 3) as compared to unheated (Test 1) and post-heated (Test 2) columns. The GFRP wrapped post-heated columns achieved a compressive strength higher than that of the unheated columns due to the fact that after heating concrete dilates more laterally under axial loading compared to unheated concrete. This lateral expansion causes higher tensile stress in the fibre reinforced polymers jackets, which provides a greater restraining force.

Fig. 104 compares the stress-strain relationship of unheated (Test 1), post-heated (Test 2) and a post-heated seriously spalled column repaired with epoxy resin mortar (Test 4). It can be seen from Fig. 104 that for a given stress the axial and hoop strains are lower in the post-heated spalled column, repaired with epoxy resin mortar, compared to the post-heated column (Test 2) since the epoxy resin mortar was stronger than the post-heated residual concrete. Fig. 105 compares the stress-strain curves of unheated columns (Test 1), the post-heated seriously spalled columns
repaired with epoxy resin mortar (Test 4), and the post-heated seriously spalled columns repaired with both epoxy resin mortar and Tyfo SEH-51A GFRP jacket (Test 5). The post-heated seriously spalled column repaired with both epoxy resin mortar and Tyfo SEH-51A GFRP jacket (Test 5) withstood higher ultimate stress and strains compared to the unheated columns (Test 1) and post-heated seriously spalled mortar repaired columns (Test 4). Comparing Figs. 103 and 105 it can be seen that the ultimate strength and strains were higher in the post-heated column wrapped with a single layer of Tyfo SEH-51A GFRP jacket (Test 3) compared to a post-heated seriously spalled column repaired with both epoxy resin mortar and Tyfo SEH-51A GFRP jacket (Test 5).

Fig. 106 compares the stress-strain curves of unheated (Test 1), post-heated (Test 2) and a post-heated Weber tec.force C-240 CFRP wrapped column (Test 6) with a post-heated column wrapped with Tyfo SEH-51A GFRP jacket (Test 3). It can be seen from this figure that the Weber tec.force C-240 CFRP wrapped column (Test 6) regained approximately the same strength as the unheated columns (Test 1). However, the ultimate strains were significantly higher than the unheated (Test 1) and post-heated columns (Test 2). It is interesting to note the ultimate strains and strength for the Tyfo SEH-51A GFRP wrapped post-heated column (Test 3) were greater compared to the post-heated column wrapped with Weber.tec force C-240 CFRP jacket. However, the modulus of elasticity and ultimate tensile strength of the Weber.tec force C-240 CFRP jacket were greater than the Tyfo SEH-51A GFRP jacket.

Fig. 107 compares the stress-strain curves of unheated (Test 1), post-heated seriously spalled repaired with both epoxy resin mortar and Tyfo SEH-51A GFRP jacket (Test 5), with the post-heated seriously spalled column repaired with both epoxy resin mortar and Tyfo SCH-41 CFRP jacket (Test 7). It can be seen that the post-heated seriously spalled column repaired with both epoxy resin mortar and Tyfo SCH-41 CFRP jacket (Test 7) withstood lower axial strains and hoop strains compared to the post-heated column repaired with both epoxy resin mortar and Tyfo
SEH-51A GFRP jacket (Test 5). This could be due to the high modulus of elasticity of the Tyfo SCH-41 CFRP jacket (Table 11). The overall efficiency of Tyfo SCH-41 CFRP jacket is greater compared to the Tyfo SEH-51A GFRP and Weber tec. force C-240 CFRP jackets. The efficiency of the Tyfo SEH-51A GFRP jacket (Test 3) is greater compared to the Weber tec. force C-240 CFRP jacket (Test 6).

Fig. 108 compares the stress-strain curves of unheated (Test 1), post-heated (Test 2) and a post-heated wrapped with ferrocement jacket (Test 8). It is interesting to note that the axial and hoop strains were reduced in the post-heated column when wrapped with a ferrocement jacket compared to the post-heated column (Test 2). This is due to the increase in cross-sectional size and the structural behaviour of ferrocement laminate.

Fig. 102: Stress-strain comparison of un-heated and post-heated circular columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 103: Stress-strain comparison of un-heated; post-heated and post-heated circular columns wrapped with Tyfo SEH-51A GFRP jacket

Fig. 104: Stress-strain comparison of un-heated; post-heated and post-heated seriously spalled repaired with high build epoxy resin mortar circular columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig.105: Stress-strain comparison of un-heated; post-heated seriously spalled repaired with high build epoxy resin mortar; post-heated seriously spalled repaired with both high build epoxy resin mortar and Tyfo SEH-51A GFRP jacket

Fig.106: Stress-strain comparison of un-heated; post-heated and post-heated circular columns wrapped with Tyfo SEH-51A GFRP and Weber.tec.force C-240 CFRP jackets
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 107: Stress-strain comparison of un-heated; post-heated epoxy resin mortar repaired and post-heated circular columns repaired with both epoxy resin mortar, Tyfo SEH-51A GFRP and Tyfo SCH-41 CFRP jackets.

Fig. 108: Stress-strain comparison of un-heated; post-heated and post-heated seriously spalled circular columns repaired with both concrete and ferrocement jacket.
6.3.1.4 Effect of mortar, ferrocement, CFRP and GFRP jackets on ductility

The ductility is defined as the ability of a structure or member to support applied loads after the elastic limit without loss of load-carrying capacity prior to failure [6]. The absolute definition of ductility is still a debatable issue and many methods exist to quantify the ductility of a structure [235]. Based on the stress-strain curves obtained from the compressive tests, as shown in Figs. 103, 105, 106 and 107, it is evident that wrapping of GFRP or CFRP jackets enhanced the ductility of columns. The ductility of columns increased significantly after heating. Fig. 104 shows that the ductility of seriously spalled columns repaired with epoxy resin mortar is approximately the same as the post-heated column. However, the ductility of the post-heated column (Test 2) and the post-heated seriously spalled and repaired with epoxy resin mortar column (Test 4) was greater compared to the unheated columns (Test 1).

The ductility of the GFRP or CFRP wrapped columns was higher compared to the unheated and post-heated columns. It was also observed that the enhancement in
ductility was more noticeable than the enhancement in ultimate load for GFRP or CFRP wrapped post-heated columns as compared to that of unwrapped post-heated columns. It is interesting to note that the ductility of post-heated columns wrapped with FRP jackets (Tests.3, 5, 6, 7) increased tremendously compared to the post-heated column repaired with ferrocement laminate (Test 8). This could be due to more stiffness of the material offered by the ferrocement laminate [129, 130].

6.4 TEST OBSERVATIONS AND FAILURE MODE (SQUARE COLUMNS)

6.4.1 Square columns (repaired with FRP, ferrocement jackets and tested under axial compressive loading)

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 17). 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 columns a typical crushing failure mode was observed under axial compression. However, the failure of all columns was initiated at the top or bottom ends of the columns due to the high stress concentration (due to the platen effect of the testing machine) within these regions [11, 35, 128], as shown in Figs. 110 to 114.

For the unheated columns; the failure was sudden and explosive, indicating the typical brittle behaviour of the concrete. In contrast, the 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 close to the ends at the corners, as shown in Figs. 112 to 114. The failure was preceded by a cracking sound, indicating significant stress transfer from the dilated concrete to 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 or bottom ends of the column due to the platen effect of the testing machine [128]. The failure was initiated due to the development of vertical cracks in the jacket. Visible folds generated by lateral dilation
were observed in the transverse direction with increased loading close to the top or bottom ends followed by 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.

It was also observed that the glass and carbon fibre jacket started to rupture within the single layer area close to the top or bottom end, as shown in Figs. 112 to 114. 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 jackets as compared to the glass fibre jackets in the form of a sudden and explosive noise, indicating a more brittle failure in the carbon fibre jackets. This could be due to a higher modulus of elasticity of the carbon fibre reinforced polymer (Table 11). It also indicates more confining pressure was developed by the carbon fibre jackets 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.

For the ferrocement repaired column, a gentle cracking sound was observed approaching to the ultimate stage of loading. The failure was initiated closer to the bottom end of column again indicating the stress concentration (due to the platen effect of the testing machine) [128]. As the failure load was approached, the wire mesh started bulging out with cracking sounds and mortar on the surface of wire mesh started to spall. The failure of the ferrocement jacket took place by rupture of the ferrocement jacket similar to the FRP jacket at the corner close to the bottom end, as shown in Fig. 115. This again indicates the stress in the square jacket tends to be concentrated at the corners. The failure occurred without a sudden and explosive noise indicating very ductile behaviour of the column. The visual inspection of the damaged column showed that no de-bonding took place between the concrete core and ferrocement jacket. The failure of the column clearly showed that the wires of mesh in the horizontal direction were broken while wires in the vertical directions
were buckled and broken at the corner. This could be likely due to the fact that under axial compression, the stress in the square ferrocement jacket tends to be concentrated at the corners in the lateral direction. The failure was initiated due to the development of vertical cracks in the jacket followed by the yielding of the horizontal wires due to transverse tension which took place at the corner of column, accompanied by crushing of the concrete, as shown in Fig. 115

![Crushing failure of un-heated square columns](image)

Fig. 110: Crushing failure of un-heated square columns
Fig. 111: Crushing failure of post-heated square columns

Fig. 112: Crushing failure of Tyfo SEH-51A GFRP wrapped post-heated square columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig.113: Crushing failure of Tyfo SCH-41 CFRP wrapped post-heated square columns

Fig.114: Crushing failure of Weber tec.force C-240 CFRP wrapped post-heated square columns
6.5 ANALYSIS OF RESULTS AND DISCUSSIONS (SQUARE COLUMNS)

6.5.1 Square columns (repaired with FRP, ferrocement jackets and tested under axial compressive loading)

6.5.1.1 Effect of GFRP, CFRP and ferrocement jackets on ultimate strength

Fig. 116 shows the effect of unidirectional glass or carbon fibre reinforced polymers and ferrocement jackets on the strength of post-heated square columns. The numbers on the x-axis represented in Fig. 116 relate to:

1. The average strength of two unheated columns (S1 & S2).
2. The average strength of two post-heated columns (S3 & S4).
3. The average strength of two post-heated square columns wrapped with a single layer of Tyfo SEH-51A GFRP jacket (S5 & S6).

Fig. 115: Crushing failure of ferrocement repaired post-heated square column
4. The average strength of two post-heated columns repaired with a single layer of Weber.tec force C-240 CFRP jacket (S8&S9).

5. The strength of a post-heated square column wrapped with Tyfo SCH-41 CFRP jacket (S7).

6. The strength of a post-heated square column wrapped with ferrocement jacket (four layers of wire mesh) (S10).

From Fig. 116 it can be seen that the strength of columns was reduced significantly after heating uniformly to 500°C. However, considerable strength was recovered when the post-heated square columns were wrapped with a single layer of Tyfo SEH-51A GFRP, Tyfo SCH-41 CFRP, Weber.tec force C-240 CFRP or ferrocement jackets (four layers of wire mesh). This could be attributed to the fact that concrete dilates laterally under axial compression, as shown in Fig. 126 and this dilation causes tensile stresses in the GFRP, CFRP or ferrocement jackets. Due to this tensile stress, the jackets confine the column and keep the column in a three-dimensional state of stress [220]. Subsequently, due to this three-dimensional state of stress, the load carrying capacity of the columns was increased significantly.

Fig. 116 also compares the improvement in the strength of post-heated columns wrapped with different FRPs. It can be seen from Fig. 116 and Table 27 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, Weber.tec force C-240, and Tyfo SCH-41 CFRP jackets respectively. However, the efficiency of the Tyfo SCH-41 CFRP jacket for the improvement of strength is considerably higher than the Tyfo SEH-51A GFRP and Weber.tec force C-240 CFRP jackets. The confinement effect of Tyfo SCH-41 CFRP jacket is higher due to a greater modulus of elasticity and ultimate tensile strength compared to the Tyfo SEH-51A GFRP (Table 11).

The ferrocement jacket (four layers of wire mesh) increased the ultimate axial load capacity of the post-heated column by 38% of the strength of post-heated reference
columns, as shown in Fig. 116. This increase in load carrying capacity is partly due to the greater cross-sectional area offered by the ferrocement jacket (46656 mm$^2$ in case of ferrocement repaired column, 40000 mm$^2$ in case of post-heated column without repairing). The strength of the post-heated square columns was restored approximately up to 85%, 78%, 74% and 71% of the strength of unheated reference columns when wrapped with Tyfo SCH-41 CFRP, ferrocement, Weber.tec force C-240 CFRP and Tyfo SEH-51A GFRP jackets respectively. This indicates the strength of post-heated square columns (exposed to uniform temperature of 500°C) could be restored to some extent, but not up to the original level of the pre-heated reference concrete columns when wrapped with a single layer of GFRP, CFRP or four layers of wire mesh ferrocement jackets. This is due to the fact that in a square section the distribution of lateral confining pressure of the jackets varies from a maximum at the corners to a minimum between the corners due to arching action [11, 32, 33, 38]. Fig. 117 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.

Only a single layer of GFRP or CFRP jackets have been used in this study to validate the effectiveness of this method for the repair of fire damaged concrete square columns. However, increasing the number of layers of the fibre reinforced polymer, or increasing the corner radius, improves the strength of the square columns, as shown from previous work [11, 20, 32, 33, 39]. To restore the original strength more than one layer of glass or carbon fibre reinforced polymer could have been used or a greater corner radius (to avoid stress concentration at corners) could have been provided [11, 20, 32, 33, 39].

6.5.1.2 Effect of CFRP, GFRP and ferrocement jackets on stiffness

Fig. 118 shows the effect of GFRP or CFRP and ferrocement jackets on the stiffness of post-heated columns. In the present study, the secant stiffness of all columns was calculated from the measured ultimate axial load divided by the measured ultimate displacement at the mid-height of the columns as described earlier in Section 6.3.1.2. The secant modulus provides a useful comparison of the effect of GFRP or CFRP
jackets on the stiffness of post-heated columns. The numbers on the x-axis represented in Fig. 118 relate to:

1) The average stiffness of two unheated columns (S1&S2).
2) The average stiffness of two post-heated columns (S3&S4).
3) The average stiffness of two post-heated square columns wrapped with a single layer of Tyfo SEH-51A GFRP jacket (S5&S6).
4) The average stiffness of two post-heated square columns wrapped with a single layer of Weber.tec force C-240 CFRP jacket (S8&S9).
5) The stiffness of a post-heated square column wrapped with a single layer of Tyfo SCH-41 CFRP jacket (S7).
6) The stiffness of a post-heated square column repaired with four layers of wire mesh ferrocement jacket (S10).

It is evident from Fig. 118 that the exposure of columns to a uniform temperature of 500°C results in a significant loss of stiffness. The degradation of the post-heated column 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 has 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 stiffness loss.

Fig. 118 compares the effect of Tyfo SEH-51A GFRP, Tyfo SCH-41 CFRP and Weber.tec force C-240 CFRP jackets on the stiffness of post-heated square columns. It is evident from this figure 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 stage of loading [114, 115] and 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, as shown in Figs. 120 to 123 and 125.
It is worth highlighting that the ferrocement jacket has a considerable effect on the stiffness of post-heated column, as shown in Fig. 118. This is attributed to the increase in the cross-sectional area of the column, improving the dimensional stability of column and due to the integrity of the composite material provided by ferrocement jacket [130]. It is interesting to note that increasing the cross-sectional area of the column by approximately 17%, the ferrocement laminate would cause a 180% increase in the stiffness of post-heated columns and improves the stiffness of post-heated columns up to 47% of un-heated reference columns.

Comparing Figs. 116 and 118, it can be seen that the reduction in residual stiffness is higher than that of ultimate load. It is therefore more important when evaluating fire damaged concrete structures to consider deformation and stress redistribution.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Table 27: Summery of test results of square columns

<table>
<thead>
<tr>
<th>Group</th>
<th>No. of columns</th>
<th>Testing conditions</th>
<th>Moisture content [%]</th>
<th>Failure load [kN]</th>
<th>Secant Stiffness (kN/mm)</th>
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<td>Two</td>
<td>Un-heated</td>
<td>-</td>
<td>1965</td>
<td>3854</td>
</tr>
<tr>
<td>2</td>
<td>Two</td>
<td>Post-heated</td>
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<td>653</td>
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<tr>
<td>3</td>
<td>Two</td>
<td>With heating and repaired with Fyfe GFRP</td>
<td>3.8</td>
<td>1396</td>
<td>859</td>
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<td>Two</td>
<td>With heating and repaired with Weber tec CFRP</td>
<td>3.5</td>
<td>1448</td>
<td>921</td>
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<td></td>
<td>One</td>
<td>With heating and repaired with Fyfe CFRP</td>
<td>3.4</td>
<td>1680</td>
<td>564</td>
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<td></td>
<td>One</td>
<td>Ferrocement</td>
<td>3.4</td>
<td>1532</td>
<td>1826</td>
</tr>
</tbody>
</table>

Fig.117: Arching action of square columns with GFRP or CFRP jackets
6.5.1.3 Effect of GFRP, CFRP and ferrocement jackets on ductility

The ductility of structural members is considered one of the most critical parameters for evaluating its performance. Based on the stress-strain response of unheated, post-heated, and post-heated GFRP, CFRP wrapped columns, as shown in Figs.120 to 123; it is evident that the confinement with GFRP or CFRP improves the ductility of columns. This could be attributed to the fact that the failure of columns was primarily due to tensile lateral strains. In un-heated and post-heated columns, without FRP wrapping, the strain in the lateral direction reached the ultimate tensile strain of concrete at a relatively lower axial strain. However, in wrapped post-heated columns, the fibre reinforced polymer laminate withstood this lateral tension and consequently increased the column’s capacity to carry a much higher value of axial and lateral strain.

From Figs. 119 to 123 it is evident that the ductility of heat damaged square columns is more than unheated columns. The ductility of post-heated columns wrapped with
Tyfo SCH-41 CFRP jackets is greater compared to post-heated columns wrapped with Tyfo SEH-51A GFRP and Weber.tec force C-240 CFRP jackets. The ductility of post-heated Tyfo SEH-51A GFRP wrapped columns is lower than the ductility of post-heated columns wrapped with Weber.tec force C-240 CFRP jackets. It was also observed that the improvement in ductility due to confinement with GFRP or CFRP is more pronounced than the increase in ultimate load. It is worth noting that the ductility of the post-heated column repaired with ferrocement laminate shown in Fig. 124 is significantly lower than post-heated columns wrapped with FRP (Figs. 120 to 123).

6.5.1.4 Effect of GFRP, CFRP and ferrocement jackets on ultimate strain

Figs. 119 to 124 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 or repaired with four layers of wire mesh ferrocement jackets. The axial and lateral strains were measured at 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 GFRP or CFRP was ignored. However, the increase in cross-sectional size was counted in the calculations in the case of the ferrocement jacket. In Figs. 119 to 124 the curves S1&S2, S3&S4, S5&S6, S7, S8&S9, S10 indicates the average stress-strain relationship relating to the test numbers listed in Section 6.5.1.1. However, the strain variations within each specimen are shown in Figs. A.34 to A.38 (Appendix). It can be seen from Fig. 119 that the values of axial strains at the ultimate loads were significantly lower in the unheated columns as compared to the post-heated columns. This is likely due to the micro-cracking and loss of stiffness of concrete after heating. Figs. 120 to 123 clearly demonstrated that the post-heated columns wrapped with a single layer of Tyfo SCH-41 CFRP, Tyfo SEH-51A GFRP, and Weber.tec force C-240 CFRP jackets withstood significant higher ultimate axial and lateral stains as 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 concrete at a relatively lower axial strain, as
shown in Fig. 119. However, in 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. 120-123. The failure of GFRP or CFRP wrapped post-heated columns took 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 Tyfo SEH-51A GFRP, Tyfo SCH-41 CFRP and Weber.tec force C-240 CFRP wrapped post-heated columns was greater than the increase in transverse strain.

Fig. 123 compares the ultimate axial stress, ultimate lateral and axial strains of post-heated columns wrapped with GFRP or CFRP. The curves ‘S5&S6’ show the stress-strain relationship of post-heated columns wrapped with Tyfo SEH-51A GFRP jackets. The curve ‘S7’ indicates the stress-strain relationship of post-heated columns wrapped with Tyfo SCH-41 CFRP jackets while the curves ‘S8&S9’ indicate the post-heated columns wrapped with Weber.tec force C-240 CFRP jackets. It is interesting to note that the ultimate strains in Tyfo SEH-51A GFRP wrapped post-heated columns were lower compared to Tyfo SCH-41 CFRP and Weber.tec force C-240 CFRP wrapped post-heated columns, although the modulus of elasticity of CFRP is higher compared to GFRP. This indicates that, due to stress concentration at the corners, the strength of the GFRP jacket 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 column were higher than the Weber.tec force C-240 CFRP columns. 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. It can be seen from Fig. 124 that the strains in the post-heated column repaired with a ferrocement jacket were considerably lower as compared to post-heated columns wrapped with FRP. This is due to the increase in cross-sectional area and the structural behaviour of ferrocement laminate.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 119: Comparison of axial and lateral strains in un-heated and post-heated columns

Fig. 120: Comparison of axial and lateral strains in un-heated, post-heated and post-heated columns wrapped with Tyfo SEH-51A GFRP jacket
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 121: Comparison of axial and lateral strains in un-heated, post-heated and post-heated columns wrapped with Tyfo SCH-41 CFRP jacket

Fig. 122: Comparison of axial and lateral strains in un-heated, post-heated and post-heated columns wrapped with Weber.tec force C-240 CFRP jacket
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 123: Comparison of axial and lateral strains in post-heated columns wrapped with Tyfo SCH-41 CFRP, Tyfo SEH-51A GFRP and Weber.tec force C-240 CFRP jackets

Fig. 124: Comparison of axial and lateral strains in un-heated, post-heated and post-heated columns repaired with ferrocement jacket.
6.6 CROSS-SECTIONAL SHAPE EFFECTS ON THE CONFINEMENT OF COMPOSITE JACKETS

6.6.1 Effect of cross-sectional shape on ultimate strength

It can be seen from Figs. 99 and 116 that the circular sections benefited more than square columns with fibre reinforced polymer jackets. This could be attributed to the fact that FRP composite jackets resist axial loads by membrane action, and are more effective for circular cross sections than square cross sections [45]. It is worth highlighting that the strength of post-heated square columns wrapped with a Tyfo SCH-41 CFRP jacket was considerably higher than post-heated square columns wrapped with a Tyfo SEH-51A GFRP jacket. A seriously spalled circular column repaired with both highbuild epoxy resin mortar and Tyfo SCH-41 CFRP also displayed higher strength than the strength of a post-heated seriously spalled circular column repaired with both highbuild epoxy resin mortar and Tyfo SEH-51A.
GFRP jacket. This is likely due to the fact of the high modulus of elasticity of the Tyfo SCH-41 CFRP jackets which ultimately provided more confining pressure than the Tyfo SEH-51A GFRP jacket.

It is evident from Fig. 99 that in the circular sections, the strength of post-heated columns was restored up to, or even greater than, its original pre-heated strength while the strength of post-heated GFRP or CFRP wrapped square columns was recovered to some extent but not to the level of its original pre-heated strength, as shown in Fig. 116. This is due to the fact that square cross sections contain some ineffectively confined concrete regions and intensification of stresses at corners, as shown in Figs. 117 and 127. The post-heated circular columns regained 10-29% higher strength than the un-heated reference circular columns and 65-122% higher strength than the post-heated reference circular columns when wrapped with a single layer of GFRP or CFRP jackets. However, the post-heated square columns recovered 71%-85% strength of the un-heated reference square columns and 26% to 51% higher than the post-heated reference square columns when wrapped with a single layer of GFRP or CFRP jackets.

The corners and long flat sides of square sections cause loss of membrane action of the FRP composites and ultimately reduce the confinement in square columns [45]. Additionally, the lower FRP confinement effectiveness results in softening behaviour for square columns and the FRP composites ruptured prematurely. Therefore, the high strength of FRP composite jackets could not be fully utilized in square columns as compared to circular columns. Based on the results, the overall efficiency of Tyfo SCH-41 CFRP jackets in both square and circular columns was grater as compared to Tyfo SEH-51A GFRP and Weber.tec force C-240 CFRP jackets.

For the circular column repaired with ferrocement (four layers of wire mesh) jacket the column regained the original strength of the un-heated reference circular columns, as shown in Fig. 99. However, a square column repaired with a ferrocement jacket (four layers of wire mesh) recovered considerable strength (78%) but not to the
original level of the pre-heated strength of the square reference columns, as shown in Fig. 116. The lower strength of the square section is probably due to transmission of large confining pressure at the corners as compared to the sides resulting in premature failure of the ferrocement jacket at the corners. However, in circular sections, the core of the concrete was subjected to uniformly radial compression (confinement) and the redistribution of crack propagation resulted in less lateral expansion of the concrete core and increased the load carrying capacity of the circular column [130].

It was observed that in the unheated and post-heated circular columns the lateral ties were opened at the overlap position due to the lack of development length provided by the poorly anchored transverse reinforcement (Figs. 91 and 92). However, no evidence of opening of the lateral ties was found in the square columns (Figs. 110 and 111) due to a good development length provided by the 135° hook at each end, and the 60 mm extension into the concrete cores. The lateral confinement of FRP improves the strength of columns. The fibre wrapped circular post-heated columns exhibited higher strength than the un-heated reference circular columns, while the fibre wrapped post-heated square columns showed less strength compared to the un-heated square columns in which there was no evidence of opening of lateral ties. This indicates that the effectiveness of the lateral steel ties confinement, due to their shape, becomes insignificant in columns when they are confined with both steel ties and FRP jackets.

6.6.2 Effect of cross-sectional shape on stiffness

It is evident from Figs. 101 and 118 that the effect of the cross-sectional shape and FRP confinement on the improvement of the stiffness of post-heated columns was negligible. This could be attributed to the fact that the confinement of the GFRP or CFRP jackets had nominal effect from the beginning to the middle stage of loading [115]. The GFRP or CFRP confinement was fully activated after reaching the maximum strength of post-heated unconfined concrete and applies continuous pressure on the concrete core until the rupture of the GFRP or CFRP jackets.
Therefore, the behaviour of the post-heated fibre reinforced polymer wrapped columns was similar to that of the post-heated columns, as shown in Figs. 103, 105-107,109,120-123 and 125. It is worth noting that the stiffness of post-heated circular and square columns repaired with ferrocement jacket was improved considerably due to an increase in cross-sectional dimensions and the integrity of ferrocement composite material, as shown in Figs. 101 and 118.

6.6.3 Effect of cross-sectional shape on stress-strain response

The stress-strain curves of post-heated circular and post-heated square columns repaired with a single layer of Tyfo SEH-51A glass, Tyfo SCH-41 carbon and Weber tec. force C-240 carbon fibre reinforced polymer jackets are shown in Figs. 103, 105 to 107 and 120 to 123 respectively. In these Figures comparison can be made between un-heated, post-heated and post-heated square and circular columns wrapped with GFRP or CFRP jackets. The stress and strain responses were observed to increase significantly with the confinement of a single layer glass or carbon fibre reinforced polymer jacket. However; this increase was significantly more pronounced in the circular cross-sectional shape than the square cross-section. This is due to the fact that the distribution of hoop stresses in circular sections was uniform and consequently the strain efficiency was increased.

For the FRP wrapped square columns, under axial compressive loading, due to stress concentration at corners, the FRP jacket is in tension while along the sides of the square cross-section, the FRP jacket behaves primarily in flexure [46]. Therefore the strain efficiency of the unidirectional Tyfo SEH-51A glass, Tyfo SCH-41 carbon and Weber.tec force C-240 CFRP square jackets was reduced considerably due to flexural induced strains as compared to a circular jackets (where the jacket is fully engaged in tension uniformly).

It is worth noting that the axial and hoop strains in post-heated seriously spalled circular columns repaired with both epoxy resin mortar and Tyfo SEH-51A glass or Tyfo SCH-41 CFRP jackets were reduced as compared to a circular post-heated column without spalling wrapped with single layer of Tyfo SEH-51A glass jacket only,
as shown in Figs. 103 and 107. This could be due to the stiffness of the post-heated seriously spalled circular columns repaired with epoxy resin mortar being increased due to the high strength of the mortar, as shown in Fig. 104. It can seen from Figs. 103, 105-107, 120-123 that, at the same stress level, the axial strains of the post-heated square and circular columns confined with Tyfo SEH-51A GFRP, Tyfo SCH-41 CFRP and Weber.tec force C-240 CFRP jacket were greater than the lateral strains (in the case of the square columns) and hoop strains (in the case of the circular columns).

6.6.4 Effect of cross-sectional shape on ductility

The ductility of structural members is still a controversial issue and its universal definition is debatable. However, it is well-known that confinement will improve column ductility [11, 31, 44, 47]. The concept of ductility as explained earlier in Section 6.3.1.4 is related to the ability of a structural member to sustain inelastic deformation without loss of load carrying capacity [6]. Therefore, based on the results of stress-strain curves, as shown in Figs. 103, 105 to 107, 120 to 123, it is evident that the confinement of post-heated circular and square columns with a single layer of GFRP or CFRP increases the ultimate axial, hoop and lateral strains which indicate the enhancement of ductility. This is likely due to fact that the GFRP or CFRP provided significant confinement to the micro cracked post-heated columns and thus resulted in more ductile behaviour as compared to un-heated and post-heated columns. The stress-strain curves shown in Figs. 103, 105-107, 120-123 clearly show that circular columns experience greater increase in ductility than square columns. This could be attributed to fact that the circular geometrical configuration of concrete columns allow the FRP jackets to be stressed uniformly, thus providing a highly effective confining pressure for concrete across the entire cross-section. However, in the square columns, the sides of the FRP jackets reduced the confinement effectiveness due to the flexural induced stiffness of the jackets [47].
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Concrete shortens

Concrete dilates

Fig. 126: Lateral dilation of reinforced concrete columns under axial compression
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 127: Function of FRP jacket under axial compression of columns
6.7 PREDICTED CONFINEMENT MODEL STRENGTH VERSUS EXPERIMENTAL CONFINED COMPRESSIVE STRENGTH

Figs. 128,129 and Table 28 compare the predicted confined compressive strengths with the experimental tested confined compressive strengths of the post-heated reinforced concrete circular and square columns wrapped with a single layer of unidirectional carbon or glass fibre reinforced polymer jackets. From Fig. 128 (a)-(b) and Table 28 it can be seen that the ACI Committee 440-2R-02 [238] confinement model predictions are 40% and 69% to 99% lower than the experimental confined compressive strength of post-heated circular columns wrapped with CFRP and GFRP jackets respectively. However, CSA S806-02 [239] confinement model predictions are 51% and 87% to 120% lower than the experimental values for the post-heated circular columns wrapped with CFRP and GFRP jackets respectively.

The Fig. 129 (a) shows the confined compressive strength of the post-heated square columns wrapped with CFRP jacket predicted by ACI Committee 440-2R-02 [238] and CSA S806-02 [239] confinement model is 23% and 61% lower than the experimental values. However the confined compressive strength predicted by ACI Committee 440-2R-02 [238] and CSA S806-02 [239] confinement models for the post-heated square columns wrapped with GFRP is 22 to 30% and 47% to 57% lower than the experimental confined compressive strength respectively. Figs. 128 and 129 clearly shows that both the ACI and CSA confinement models predicted lower confined compressive strengths for post-heated reinforced concrete columns than the actual experimental tested confined compressive strengths. This could be due to the fact that the ACI and CSA confinement models are based on the un-heated concrete. The post-heated concrete displayed more lateral dilation compared to the un-heated concrete. Therefore; the fibre reinforced polymer is more activated in post-heated concrete compared to un-heated concrete. Thus, in case of fire damaged concrete, the current existing international design guidelines are more conservative for the prediction of confined compressive strength of post-heated columns wrapped with FRP.
Fig. 128: (a) CFRP wrapped post-heated circular columns; (b) GFRP wrapped post-heated circular columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 129: (a) CFRP wrapped post-heated square columns; (b) GFRP wrapped post-heated square columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
<thead>
<tr>
<th>Type of FRP</th>
<th>Shape of column</th>
<th>ACI Theoretical load [kN]</th>
<th>ASA Theoretical load [kN]</th>
<th>Experimental load [kN]</th>
<th>Test condition</th>
</tr>
</thead>
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<tr>
<td>Tyfo SEH-51A</td>
<td>Circular</td>
<td>921</td>
<td>834</td>
<td>1834</td>
<td>Post-heated only</td>
</tr>
<tr>
<td>Tyfo SEH-51A</td>
<td>Circular</td>
<td>921</td>
<td>834</td>
<td>1557</td>
<td>Post-heated explosive spalling repaired with epoxy resin mortar</td>
</tr>
<tr>
<td>Tyfo SCH-41</td>
<td>Circular</td>
<td>1214</td>
<td>1129</td>
<td>1701</td>
<td>Post-heated explosive spalling repaired with epoxy resin mortar</td>
</tr>
<tr>
<td>C Sheet 240 (Weber)</td>
<td>Circular</td>
<td>-</td>
<td>-</td>
<td>1356</td>
<td>Post-heated only</td>
</tr>
<tr>
<td>Tyfo SEH-51A</td>
<td>Square</td>
<td>1109</td>
<td>920</td>
<td>1440</td>
<td>Post-heated only</td>
</tr>
<tr>
<td>Tyfo SEH-51A</td>
<td>Square</td>
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<td>920</td>
<td>1350.6</td>
<td>Post-heated only</td>
</tr>
<tr>
<td>Tyfo SCH-41</td>
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<td>1044</td>
<td>1680.5</td>
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</tr>
<tr>
<td>C Sheet 240 (Weber)</td>
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<td>-</td>
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</tr>
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<td>-</td>
<td>1495.5</td>
<td>Post-heated only</td>
</tr>
</tbody>
</table>

Table 28: Predicted confined compressive strength versus experimental confined compressive strength of post-heated reinforced concrete columns
CHAPTER-7

7 ANALYSIS OF RESULTS AND DISCUSSION OF CRITICAL SEISMIC SHEAR TESTED COLUMNS (EXPERIMENTAL PART-2)

7.1 INTRODUCTION

In this experimental part of the study, the test observations along with critical plots are presented. The main variables investigated in this part of the study were the shape of columns, presence of heat damage columns, type of fibres, the orientation of the main fibres and the peak drift ratios. The drift ratio is defined as the lateral displacement divided by the clear height of the column between the point of application of lateral load and the top level of the column footing. Seismic performance of repaired columns in terms of their hysteretic response, shear strength, lateral strength, ductility, energy dissipation and stiffness degradation is evaluated and compared to those of the control original un-heated and post-heated columns.

7.2 TEST OBSERVATIONS AND FAILURE MODE

7.2.1 Circular reinforced concrete columns

Figs. 130 to 133 show the typical failure mode of un-heated, post-heated and GFRP, CFRP wrapped post-heated circular columns tested under similar combined constant axial and lateral reversal cyclic loading. The two un-heated and two post-heated columns were first tested to failure to establish a benchmark testing data.

In the first reference un-heated column (C10), horizontal cracks were observed 150 mm above the steel stub base in the first cycle at a 4% drift ratio (17 mm lateral displacement) at a 47 kN pushing and 43 kN pulling load. In the second un-heated reference column (C11), two horizontal cracks were observed in a similar fashion at the beginning of the first cycle at a 4% drift ratio (17 mm lateral displacement) at a 56
kN pushing and 44 kN pulling load. The horizontal cracks which formed during the first cycle of the 4% drift ratio were observed at a distance of 100 to 200 mm above the face of bottom steel stub. In the later stages of loading, the horizontal cracks stopped becoming any wider and no further horizontal cracks were observed at the end of the third cycle at the 4% drift ratio. The first visible shear crack was observed in both the reference un-heated columns (C10&C11) at the first cycle at the 5% drift ratio, in the pushing part of loading, indicating the formation of a shear failure plane, as shown in Fig.130. The same crack was then also observed during the pulling part of loading in the other diagonal direction. The X-shape cracking pattern became more visible with an increase in the number of cycles indicating the start of shear failure.

The repetition of cyclic loading caused slight crushing of concrete at the column-base joint in the pulling and pushing direction of loading, as shown in the marked circles in Fig. 130. This was due to the stiff edges of the confining steel stub at the base joint restricting any lateral displacement. After 5% drift ratio, the columns attained a higher level of lateral loading during the first cycle than the subsequent second and third cycles at each following drift ratios both in the pulling and pushing part of the loading. At a 6% drift ratio, further opening of the diagonal X-cracking along the shear failure plane was observed. As the lateral load increased, the shear crack propagated upward along the shear failure plane. The shear crack started at the top edge of the confining support then propagated upward to the point of application of the actuator for applying horizontal cyclic shear loading. The maximum lateral strength was attained during the first cycle of the 8% drift ratio in the pushing part of loading at 69 kN and 73 kN lateral loads with 34 mm displacement in the un-heated reference columns C10 and C11 respectively. As the failure load was approached, one of the diagonal shear cracks became significantly wider and longer than the others. The final shear failure took place along this critical shear plane, as shown in Fig. 130. The test was terminated when the lateral loading started to decrease and the column was unsuitable for further lateral loading.
For the post-heated reference columns (C12&C13), the mode of failure observed was again shear failure, as shown in Fig. 131. The first sign of a horizontal crack in the two post-heated specimens was observed at 150 mm above the base column joint at the first cycle of 3% drift ratio (13 mm displacement) during the pushing part of lateral load at 24 kN for column C12 and 31 kN for column C13. As the drift ratio increased, the horizontal crack did not increase in width. The first visible shear crack occurred at the first cycle of the 4% drift ratio (17 mm displacement), in the pushing part of loading at 31 kN for column C12 and 38 kN for column C13. In the pulling part of loading, the diagonal shear crack occurred in the same fashion in both columns but in the other diagonal direction at 25 kN and 31 kN lateral loads with 17 mm displacement respectively.

The trends of the diagonal X-shear cracking were extended diagonally from the base of the column’s footing toward the point of application of lateral loading with increasing drift ratios indicating the shear failure plane. The width of shear cracks was increased with increasing load cycles. After 4% drift ratio, the columns attained a higher level of lateral loading in the first cycle as compared to following second and third cycles at each following drift ratios both in the pushing and pulling part of loading. This indicates further cracking appeared during the first cycle of each subsequent drift ratio. Approaching the failure load, one of the diagonal shear cracks became again wider and longer. The final shear failure took place again along this diagonal critical shear cracking plane.

The maximum lateral strength achieved in post-heated columns C12 and C13 was 46 kN and 52 kN at 7% drift ratio (30 mm displacement) in the pushing part of loading. The crushing of the concrete at the column-base joint was also observed, as shown in Fig. 131. This is due to the maximum stress concentration at the column base region produced by the repetition of lateral cyclic loading. At the inception of the first cycle at 8% drift ratio, the lateral load started to decrease. The test was terminated after completing the third cycle of 8% drift ratio. The cracking patterns and the failure modes of the un-heated (C10 and C11) and posted-heated columns (C12 and C13)
were similar. The only differences between these columns were the magnitude of the lateral load at which the first crack initiated and, as shown in the Table 29, the values of the ultimate failure lateral loads. It is worth noting that the observed width of the critical shear failure crack was significantly greater in the un-heated columns (C10 and C11), compared to the post-heated columns (C12 and C13), as shown in Figs. 130 and 131. Additionally, the failure of the post-heated columns was localized at the bottom critical region of the column after the development of X-shear cracking. This is due to the fact that concrete after being exposed to 500°C, becomes soft and weak compared to un-heated concrete. The shear frictional forces were reduced due to this softening behaviour.

In the unidirectional glass fibre reinforced polymer (GFRP) wrapped post-heated columns (C14 and C15), cracking sounds and a change in colour on the surface of the GFRP jacket, close to base column joint, was observed at the 6% drift ratio in the pushing part of loading. The first visible flexural crack was initiated at 7% drift ratio (30 mm displacement) in the pushing part of the loading at 52 kN for column C14 and 55 kN lateral load for column C15. As the lateral load increased, the existing horizontal cracks extended in width with further cracks developing approximately 50-100 mm above the support, as shown in Fig. 132. In subsequent loading cycles, with further increases in lateral load, the flexural cracks continued to extend around the perimeter of the cross-section at 50-100 mm above the support. In the subsequent displacement cycles, damage was localised in this vicinity. At a 12% drift ratio (51 mm displacement), the flexural cracks extended all around the perimeter at 69 kN lateral load for column C14 and 68 kN lateral load for column C15. In the first cycle of the 13% drift ratio, the lateral strength started to decrease and test was terminated after the third cycle of the 13% drift ratio. No jacket rupture occurred in the GFRP wrapped post-heated columns C14 and C15 even after completing the loading cycles corresponding to a drift ratio 13%. The GFRP wrapping successfully increased both the shear strength and deformation capacity of columns.
In the CFRP wrapped post-heated columns, cracking sounds and a change in colour of the CFRP jacket, at 50-100 mm distance above the base of the column, was noticed at 7% drift ratio in the pushing and pulling part of the loading. The first visible flexural crack was initiated at 8% drift ratio (34 mm displacement) in the pushing part of the loading at 54 kN for both CFRP wrapped post-heated columns (C16 & C17). With further increase in lateral load, the existing horizontal cracks extended, in the same fashion observed in GFRP wrapped post-heated columns, approximately 50-100 mm above the bottom column support. With increasing the displacement cycles, the damage was localised in this region and ultimate failure occurred. The maximum lateral strength achieved in both post-heated CFRP wrapped columns was approximately 69 kN for C16 and 70 kN for C17 at 11% drift ratio corresponding to 47 mm displacement, as shown in Table 30. The CFRP jacket was ruptured at 11% drift ratio, as shown in Fig. 133.

It was found that in the un-heated, post-heated and post-heated columns wrapped with GFRP or CFRP, the level of lateral loading was higher during the first cycle than the subsequent second and third cycles at each drift ratio, both in the pushing and pulling part of loading after the occurrence of the first crack. This could be due to the fact that after the occurrence of the first crack, further cracking was produced at each first cycle with increasing the drift ratio. The strength was further decreased at each first cycle with repeating the same cycles at the same drift ratio in the pushing and pulling direction. The post-heated columns showed relatively larger lateral displacements at the same load-levels compared to the original un-heated columns. The GFRP or CFRP wrapped post-heated columns displayed larger lateral displacements at the failure lateral load levels compared to the un-heated and post-heated columns.

The failure modes of the GFRP and CFRP wrapped circular columns were different compared to the un-heated and post-heated unwrapped columns. Figs. 132 and 133 show the failure of GFRP and CFRP wrapped post-heated columns with main fibres oriented in the circumferential direction.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 130: X-Shear failure of un-heated circular columns

Crushing of concrete

Fig. 131: X-Shear failure of post-heated circular columns

Crushing of concrete
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 132: Flexural failure of post-heated circular columns wrapped with GFRP jackets

Fig. 133: Flexural failure of post-heated circular columns wrapped with CFRP jackets
7.3 ANALYSIS OF TEST RESULTS AND DISCUSSIONS

7.3.1 Circular reinforced concrete columns

7.3.1.1 Hysteretic response

The seismic response of un-heated, post-heated and post-heated circular columns wrapped with GFRP or CFRP jackets was evaluated in terms of their hysteretic response. The hysteretic response provides useful information regarding strength and stiffness degradation, energy dissipation capacity and ductility of all columns tested under similar loading conditions [236]. The recorded data for the hysteretic response in terms of lateral load versus lateral displacement of all columns were plotted with maximum displacement of 60 mm and lateral load of 80 kN for comparison in Figs. A.5 to A.12 in the Appendix. It can be seen from Figs. A.5 to A.8 that the hysteretic loops in post-heated columns (C13 and C14) were small compared to the un-heated columns (C11 and C12). This highlights the weak performance of the post-heated columns under seismic loading due to loss of strength and stiffness.

It is evident from Figs. A.9 to A.12 that the post-heated columns wrapped with GFRP or CFRP (C14&C15 or C16 & C17) displayed larger hysteresis loops compared to post-heated columns (C13&C14). This indicates the improved seismic performance of post-heated columns wrapped with GFRP or CFRP jackets compared to post-heated reference columns in terms of the increase of strength, ductility and energy dissipation. Figs. A.9 to A.12 and Table 30 showed that the post-heated columns wrapped with GFRP or CFRP regained approximately the same lateral strength, with higher displacement, compared to the un-heated columns. This indicates the best seismic performance of GFRP or CFRP wrapped post-heated columns as compared to un-heated columns in terms of energy dissipation and ductility. The bigger loops of post-heated columns wrapped with GFRP or CFRP, shown in Figs. A.9 to A.12 in the Appendix, indicated the slow rate of strength and stiffness degradation of the columns.
7.3.1.2 Lateral load-displacement envelopes

Figs. A.13 through to A.16 (Appendix) show the maximum pushing and pulling loads in each drift ratio plotted against the maximum displacements achieved at the peak of the pushing and pulling loads. The area under the lateral load-displacement envelopes indicates the energy dissipation of the columns [76, 77, 88]. In Fig. A.13, the curves C10 and C11 indicate the lateral strength-displacement envelopes of unheated reference specimen C10 and C11. In Fig. A.14, the curves C12 and C13 shows the lateral strength-displacement envelopes of post-heated reference specimen C12 and C13. In Fig. A.15, the curves C14 and C15 indicates the lateral strength-displacement envelopes of post-heated columns C14 and C15 wrapped with GFRP jackets. In Fig. A.16, the curves C16 and C17 show the lateral strength-displacement envelopes of post-heated columns C16 and C17 wrapped with CFRP jackets. Comparing Fig. A.13 with Fig. A.14, it can be seen that the lateral load capacity of post-heated columns (C12&C13) were significantly reduced as compared to un-heated columns (C10&C11). It is evident from the load-displacement envelopes shown in Figs. A.15 to A.16 that the post-heated columns wrapped with a single layer of GFRP or CFRP regained approximately the same lateral strength with more lateral displacement as that of the un-heated reference columns (C10&C11). This indicates the substantial increase in energy dissipation and ductility, and slower rate of strength and stiffness degradation in GFRP or CFRP wrapped columns as compared to un-heated and post-heated reference columns (discussed in the following sections in detail).

7.3.1.3 Ductility response

The ductility of a structure is one of the most critical parameters for evaluating the seismic performance of structures. The holistic ductile behaviour of the structure depends on the ductility of the main individual structural members. The concept of structural ductility is its ability to undergo large inelastic deformations before its failure [6]. The ductile behaviour of buildings has the ability to oscillate back-and-forth during an earthquake, and to withstand seismic shocks with some damage, but without catastrophic failure [224]. Therefore, the increase of ductility in the columns of fire
damaged concrete structures following repair is generally considered as a realistic way to reduce the risk of catastrophic failure of concrete structures subjected to future strong earthquakes.

Based on the hysteretic response of columns, it can be seen that the GFRP or CFRP wrapped post-heated columns (C14&C15 or C16&C17) showed a significant improvement in the hysteresis loops of lateral load versus displacement, as shown in Figs. A.9 to A.12. This clearly indicates the enhancement of ductility. It is evident from Figs. A.14 to A.16 that the post-heated columns after wrapping with GFRP or CFRP attained higher ultimate lateral loads with higher ultimate displacements than that of post-heated columns under the same loading condition. This indicated that the GFRP or CFRP provided significant confinement to the micro cracked post-heated columns and increased the ultimate strength and deformation, resulting in more ductile behaviour compared to un-heated and post-heated columns.

### 7.3.1.4 Energy dissipation

The energy dissipation ability of a structure is one of the most important parameters to evaluate its seismic performance. The area under the lateral load displacement envelopes could be defined as the energy dissipation by the columns [76, 77, 88]. The larger the area occupied, the larger will be the dissipation of energy and the larger will be the damping effect [236]. The energy dissipation in un-heated, post-heated and post-heated columns wrapped with GFRP or CFRP was calculated from the area under the ultimate lateral load displacement envelopes, as shown in Figs. A.13 to A.16. Fig. 132 compares the energy dissipation in un-heated, post-heated and post-heated columns wrapped with a single layer of GFRP or CFRP tested under similar loading conditions. The numbers 1 to 4 on x-axis in Fig. 134 represent:

1) Average of the energy dissipation of two un-heated reference circular columns (C10&C11)
2) Average of the energy dissipation of two post-heated reference circular columns (C12&C13)
3) Average of the energy dissipation of two post-heated circular columns wrapped with Tyfo SEH-51A GFRP jacket (C14&C15)
4) Average of the energy dissipation of two post-heated circular columns wrapped with Tyfo SCH-41 CFRP jacket (C16&C17)

It can be seen from Fig. 134 and Table 29 that the energy dissipated by un-heated circular columns (C10&C11) was 47% higher than those of post-heated columns (C12&C13). This is also clear from the hysteresis hoops seen in Figs. A.5 to A.8. It is evident from Table 29 and Fig. 134 that the energy dissipation capacity of post-heated columns wrapped with GFRP or CFRP jackets was increased substantially compared with that of reference post-heated columns (C12&C13).

The comparison can be made From Fig. 134 that among un-heated, post-heated and post-heated columns wrapped with GFRP or CFRP jackets, the best energy dissipation showed by post-heated GFRP or CFRP wrapped columns. This is likely to be due to the fact that GFRP or CFRP wrapped post-heated columns attained higher strengths and displacements as compared to post-heated columns (C12&C13) and approximately the same strength with higher displacements when compared to un-heated columns (C10&C11). This is clear from the large hysteresis loops seen in Figs. A.5 through to A.12. The GFRP and CFRP wrapped post-heated columns showed 163% and 123% increase in energy dissipation compared to the post-heated reference columns (C12&C13). The GFRP and CFRP wrapped post-heated columns indicated 40% and 19% increase in energy dissipation when compared to un-heated reference columns (C10&C11), as shown in Table 29. The normalised energy dissipation was calculated for comparison purposes considering un-heated columns (C10&C11) as reference columns and is shown in Table 29.
7.3.1.5 Stiffness degradation

Fig. 136 compares the secant stiffness degradation of un-heated, post-heated and post-heated GFRP or CFRP wrapped columns tested under constant axial and lateral reversal cyclic loading. The secant stiffness was calculated from the measured maximum lateral load reached within each cycle divided by the displacement reached at the peak of the each load cycle in the pushing and pulling direction, as shown in Fig. 135. The overall secant stiffness of each cycle was then calculated by taking the average of the secant stiffness in the pushing and the pulling part of loading following the method used by Shannag [76, 77, 88] and Saadatmanesh [92]. The final stiffness was then calculated by taking the average of three cycles of each drift ratio in the pushing and pulling part of the loading. The calculated stiffness was plotted in Fig. 136 versus the drift ratio for un-heated, post-heated and post-heated GFRP or CFRP wrapped columns. The Fig. 136 clearly demonstrates that the initial stiffness values
for post-heated columns were substantially lower than the un-heated columns. The loss of initial stiffness of post-heated columns was mainly caused by the softening of concrete after heating, to a uniform temperature of 500°C, due to micro-cracking.

It is worth noting that the glass or carbon fibre reinforced polymer wrapping influenced the rate of stiffness degradation, as shown in Table 31. The post-heated columns wrapped with a single layer of GFRP or CFRP composite showed a slower rate of stiffness degradation than the un-heated and post-heated columns, as shown in Fig. 136. The slower rate of stiffness degradation is a very worthwhile property of the structure during earthquakes as many reinforced structures have collapsed during past earthquakes due to a sudden loss of stiffness degradation [49]. The un-heated and post-heated columns showed a sudden drop in stiffness up to a drift ratio of approximately 3.0%. The loss of stiffness for un-heated and post-heated columns was approximately 65% and 53% respectively of their initial stiffness at the end of the test. The stiffness of post-heated GFRP and CFRP wrapped columns was 34% and 28% respectively more than the stiffness at the end of post-heated columns test.

\[ K_{avg} = \frac{K_{pushing} + K_{pulling}}{2} \]

**Fig. 135: Secant stiffness**
7.3.1.6 Lateral strength

Fig. 137 compares the lateral load carrying capacities of un-heated, post-heated and post-heated GFRP or CFRP wrapped columns tested under similar combined constant axial and reversal lateral load. The number 1 to 8 in Fig. 137 represents:

1) Un-heated circular column (C10)
2) Un-heated circular column (C11)
3) Post-heated circular column (C12)
4) Post-heated circular column (C13)
5) Post-heated circular column wrapped with Tyfo SEH-51A GFRP jacket (C14)
6) Post-heated circular column wrapped with Tyfo SEH-51A GFRP jacket (C15)
7) Post-heated circular column wrapped with Tyfo SCH-41 CFRP jacket (C16)
8) Post-heated seriously spalled circular column repaired with concrete and wrapped with Tyfo SCH-41 CFRP jacket (C17).

Fig. 136: Degradation of secant stiffness of circular columns with increasing drift ratios
It can be seen from Fig. 137 that the lateral strength of the post-heated columns (C12 & C13) was reduced significantly as compared to the un-heated columns (C10 & C11). The lateral load capacity of post-heated columns (C12 & C13) was reduced approximately 30% of the un-heated columns (C10 & C11). The lateral strength of post-heated columns (C12 & C13) was restored up to approximately 98% of the original strength of the un-heated columns (C10 & C11) when wrapped with a single layer of unidirectional glass or carbon fibre reinforced polymers (C14 & C15 or C16 & C17). The lateral strength of post-heated columns wrapped with GFRP or CFRP jackets was increased by 40% compared to the heat damaged columns (C12 & C13).

The main fibres placed in the horizontal or hoop direction improves the shear capacity of post-heated columns. The tensile stresses in the fibre reinforced polymers oriented in the hoop or horizontal direction under seismic forces, contributed to the overall shear resistance of the columns. This is due to the fact that shear cracks occurred at an angle of 45° to the direction of main fibres. The tensile stresses in the fibres reduce or minimize the opening of the shear cracks. It is worth highlighting that all columns gained more strength in pushing as compared to the pulling part of the loading, as shown in Fig. 137 and Table 30. It could be due to the fact that the first crack occurred in the pushing part of loading cycle. It is evident from Table 30 that the un-heated circular columns sustained lateral load up to 8% drift ratio, post-heated circular columns had taken up to 7% drift ratio while the GFRP and CFRP wrapped post-heated circular columns sustained up to 12% and 11% drift ratios respectively. This indicates that the rate of strength degradation was slower in the GFRP or CFRP wrapped post-heated columns (C14 & C15 or C16 & C17) compared to the un-heated (C10 & C11) and post-heated columns (C12 & C13) with increasing drift ratios.

It is interesting to note that approximately the same lateral strength of the un-heated columns (C10 & C11) was achieved when post-heated columns were wrapped with a single layer of GFRP or CFRP jackets with higher level of displacement as compared to the un-heated columns. It was noted that at the same drift ratio the level of lateral load is slightly higher in the GFRP wrapped post-heated columns than the CFRP
wrapped post-heated columns even though the tensile modulus and tensile strength of the CFRP jacket is greater compared to the GFRP jacket (Table 11). This is probably due to the fact in the GFRP jacket there were some additional fibres in the longitudinal direction (Table 13) which provided additional strength to the GFRP wrapped post-heated columns.

7.3.1.7 Cracking pattern of columns

Figs. 130 to 133 show the cracking pattern of un-heated, post-heated and GFRP or CFRP wrapped post-heated columns. It can be seen from Fig. 130 that un-heated columns failed drastically in the shear mode. The Fig. 131 shows the failure of post-heated columns. The post-heated columns again showed a similar shear failure mode. However, the shear cracks were not as wide compared to the un-heated columns. Figs. 132 and 133 show the failure mode of the GFRP or CFRP wrapped columns. It is interesting to note that the failure mechanism of GFRP or CFRP wrapped post-heated columns was shifted from column shear failure to a flexural failure mode. The uni-directional glass or carbon fibres wrapped in the transverse direction increased the shear strength of the post-heated columns to such an extent that at 12% and 11% drift ratios, the column shear strength still remained within its capacity. The flexural failure mode is a desirable property of reinforced concrete columns during earthquakes. This is because flexural failure is generally less destructive compared to a shear failure mode and no catastrophic collapse of a structure has occurred due to this kind of failure. Additionally, during flexural failure, the stresses are redistributed and the columns are continuing to support the super-structure which gives warning to the occupants to leave the buildings prior to failure.

It is evident from Figs. 132 and 133 that the GFRP or CFRP confinement improved the shear capacity of post-heated columns. It is interesting to note that in all columns, no crack was observed in the upper portion of the columns where the horizontal load was applied. This indicates that in shear strengthening of columns, the fibre should be wrapped up to the level of application of lateral loading. As in the GFRP or CFRP wrapped columns the main fibres were oriented in the circumferential direction and therefore they contributed to increasing the shear strength only. To increase the
flexural and shear strength both a combination of unidirectional fibres in the circumferential and longitudinal directions should be used or bidirectional fibres should be recommended.

7.3.1.8 Restraining effect of bottom steel stub

The columns were tested in vertical cantilever positions confined at the bottom with a stiff steel pipe up to 200 mm height. The maximum moment during pushing and pulling was supposed to occur at the interface of the columns and the top edge of stiff steel stub. However, it was observed that the most damage was initiated at a shorter distance away from the top edge of confining bottom stiff steel stub, as shown in Figs. 132 and 133. This could be probably due to the fact that the bottom stiff steel stub provided an additional restraining effect to the column section near to the of top edge of the confining steel stub. This restraining effect reduced the tendency of lateral expansion at the interface of the column and top edge of the steel stub. Consequently, the moment capacity of the column in the vicinity close to top edge of steel stub was increased and the failure was shifted to a shorter distance away from the top edge of steel stub. This observation is consistent with the previous findings of Shiek [84, 237] where columns were cast monolithically with strong concrete stubs.

<table>
<thead>
<tr>
<th>Test conditions</th>
<th>Energy dissipation [kN.mm]</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Pushing part of loading</td>
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<tr>
<td>Un-heated columns</td>
<td>1577.9</td>
</tr>
<tr>
<td>Post-heated columns</td>
<td>880.1</td>
</tr>
<tr>
<td>Post-heated columns wrapped with GFRP jackets</td>
<td>2258.2</td>
</tr>
<tr>
<td>Post-heated columns wrapped with CFRP jackets</td>
<td>1867.3</td>
</tr>
</tbody>
</table>

Table 29: Energy dissipation in circular columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 137: (a) Maximum lateral strength of circular columns in pushing

Fig. 137: (b) Maximum lateral strength of circular columns in pulling
<table>
<thead>
<tr>
<th>Test conditions</th>
<th>Drift ratio [%]</th>
<th>Maximum horizontal load in pushing [kN]</th>
<th>Maximum displacement in pushing [mm]</th>
<th>Maximum horizontal load in pulling [kN]</th>
<th>Maximum displacement in pulling [mm]</th>
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<tbody>
<tr>
<td>Un-heated column-1</td>
<td>8 (1st cycle)</td>
<td>68.6</td>
<td>33.7</td>
<td>68.4</td>
<td>33.9</td>
</tr>
<tr>
<td>Un-heated column-2</td>
<td>8 (1st cycle)</td>
<td>72.7</td>
<td>29.7</td>
<td>62.7</td>
<td>33.9</td>
</tr>
<tr>
<td>Post-heated column-1</td>
<td>7 (1st cycle)</td>
<td>46.2</td>
<td>29.6</td>
<td>42.3</td>
<td>29.8</td>
</tr>
<tr>
<td>Post-heated column-2</td>
<td>7 (1st cycle)</td>
<td>52.2</td>
<td>29.8</td>
<td>46.4</td>
<td>29.7</td>
</tr>
<tr>
<td>Post-heated GFRP wrapped column-1</td>
<td>12-(1st cycle)</td>
<td>69.1</td>
<td>50.8</td>
<td>58</td>
<td>51</td>
</tr>
<tr>
<td>Post-heated GFRP wrapped column-2</td>
<td>12(1st cycle)</td>
<td>68.4</td>
<td>51</td>
<td>66.1</td>
<td>51</td>
</tr>
<tr>
<td>Post-heated CFRP wrapped column-1</td>
<td>11(1st cycle)</td>
<td>68.6</td>
<td>46.7</td>
<td>67.2</td>
<td>46.8</td>
</tr>
<tr>
<td>Post-heated CFRP wrapped column-2</td>
<td>11(1st cycle)</td>
<td>69.9</td>
<td>46.75</td>
<td>68</td>
<td>46.75</td>
</tr>
</tbody>
</table>

Table 30: Maximum lateral loads and lateral displacements in circular columns
## Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

### Table 31: Stiffness degradation in circular columns

<table>
<thead>
<tr>
<th>Drift ratio [%]</th>
<th>Degradation of secant stiffness [kN/mm]</th>
<th></th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Un-heated columns</td>
<td>Post-heated columns</td>
</tr>
<tr>
<td>0.5</td>
<td>4.357</td>
<td>2.604</td>
</tr>
<tr>
<td>1</td>
<td>4.317</td>
<td>2.449</td>
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<tr>
<td>2</td>
<td>3.437</td>
<td>2.125</td>
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<tr>
<td>3</td>
<td>3.050</td>
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<td>4</td>
<td>2.808</td>
<td>1.797</td>
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<td>1.733</td>
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<tr>
<td>6</td>
<td>2.424</td>
<td>1.658</td>
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<td>7</td>
<td>2.186</td>
<td>1.486</td>
</tr>
<tr>
<td>8</td>
<td>1.933</td>
<td>1.233</td>
</tr>
<tr>
<td>9</td>
<td>1.598</td>
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<tr>
<td>10</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>-</td>
<td>-</td>
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<tr>
<td>12</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>13</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Failure mode: Shear, Shear, Flexural, Flexural

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7.4 TEST OBSERVATIONS AND FAILURE MODE

7.4.1 Square reinforced concrete columns

The typical damage which occurred in the un-heated, post-heated and post-heated concrete square columns following repair with glass or carbon fibre reinforced polymers is shown in Figs.138 to 145. In the first reference un-heated column (S11) horizontal cracks were observed during first cycle of the 4% drift ratio (17 mm displacement) at 65 kN pushing and 63 kN pulling loads. These horizontal cracks were observed at a distance equal to about 150 mm above the interface of the column support, as shown in Fig. 138.

The horizontal cracks transverse to the pulling and pushing part of the loading were also observed closer to the face of the confining steel plate, as shown in Fig. 138. These horizontal cracks were approximately 15 mm above the bottom confined steel plate. In the un-heated reference column (S12), the horizontal cracks were also observed during the first cycle of the 4% drift ratio (17 mm lateral displacement) at 65 kN pushing and 64 kN pulling load. These horizontal cracks were developed approximately at a distance of 200 mm above the column support. With increasing drift ratios, the horizontal cracks stopped becoming wide and no further permanent horizontal cracks were observed at the end of the third cycle of the 4% drift ratio in both the un-heated reference columns. Many X-shear cracks were developed in the later stages of the cyclic loading which were almost symmetrical on the two faces of the un-heated columns (S11&S12) transverse to the direction of loading, as shown in Fig. 138.

The first diagonal visible shear crack was developed during the first cycle of the 5% drift ratio (21 mm displacement) at approximately 75 kN pushing loads in the bottom critical regions of both the un-heated reference columns (S11 &S12). In the pulling part the diagonal shear crack was developed in the same manner but in the opposite direction. These diagonal shear cracks were developed during the first cycle of the 5% drift ratio (21 mm lateral displacement) at approximately 64 kN lateral load. The
repeated lateral cyclic loading also caused crushing of the concrete at the column-base interface in the pushing and pulling direction of loading, as indicated in the marked circles in Fig. 139. This attributes to the stiff edges of the steel plate of the stub restricting the lateral displacement. After 5% drift ratio (21 mm displacement), the columns attained a higher level of lateral loading in the first cycle as compared to the following second and third cycles at each drift ratios both in the pushing and pulling part of loading. This is probably due to the fact that with further increasing drift ratios, the cracks were always developed at the first cycle of each drift ratios. Additionally, the repetition of lateral cyclic loading caused the opening and closing of formed cracks and the lateral strength was reduced further during the second and third cycle of each drift ratios.

The shear cracks were widened with increasing load cycles and one of the diagonal shear cracks became obviously wider and longer than the others similar to circular columns described previously in Section 7.2.1. This diagonal shear crack formed close to the column critical bottom region and extended upwards towards the point of application of the actuator pin for applying horizontal cyclic shear loading. This is due to the diagonal tension caused by the more shear stress at the point of application of lateral loading. This diagonal shear crack defines the critical shear failure plane for both un-heated columns (S11&S12). The maximum lateral strength was reached at the first cycle of the 10% drift ratio (42 mm lateral displacement) at 97 kN for column S11 and 98 kN for column S12 in the pushing part of loading. Approaching the failure load, the wider diagonal shear crack was further expanded and finally both the un-heated columns failed drastically in a brittle shear failure mode along the critical shear failure plane, as shown in Figs. 139 and 140. The test was terminated when the lateral loading started to decrease and the column was unsuitable for further lateral loading.

For the post-heated columns, the failure trend was similar as observed in the un-heated columns. Figs. 141 and 142 show the cracking patterns of post-heated columns. Very minor horizontal cracks were observed at the first cycle of the 3% drift
ratio (12.7 mm displacement) at 45 kN pushing lateral load in both post-heated reference columns (S13 & S14). The trend of the horizontal cracks was changed into diagonal cracks when the drift ratio exceeded 3% in the pushing and pulling part of loading in both post-heated columns (S13 & S14). The first visible shear crack appeared when the drift ratio reached the first cycle of the 4% drift ratio (17 mm lateral displacement) in the pushing part of loading at 53 kN lateral load in both post-heated reference columns (S13 & S14). After 4% drift ratio, the columns attained a higher level of lateral loading during the first cycle than subsequent similar second and third cycles both in the pulling and pushing part of the loading.

With increasing drift ratios, many shear cracks were also observed in the post-heated columns. The shear cracking area of post-heated columns was not as large as it was observed in the un-heated columns, as shown in Figs. 141 and 142. It was found that at the same level of displacements, the post-heated columns (S13 & S14) attained lower lateral loads as compared to the un-heated columns (S11 & S12). In the post-heated reference columns (S13 & S14), the trend of the initial X-shear cracks was extended from the top edge of the bottom confining steel stub diagonally towards the point of application of the lateral loads, with increasing drift ratios, indicating the shear failure plane, as shown in Fig. 141. The crushing of concrete at the column-base interface was also observed, as shown in the marked circles in Fig. 141. This could be due to the maximum stress concentrations at the column base critical region produced by the repetition of reversed horizontal loading and the stiff edges of the confining steel stub. Approaching the failure load, the critical diagonal shear crack was further widened and finally the post-heated columns failed in a shear failure mode, as shown in Fig. 142.

It is worth highlighting that in post-heated columns in the subsequent displacement cycles, the shear X-cracking damage due to cyclic loading was localised in the bottom critical region with increasing drift ratios instead of extending upward towards to point of the application of the actuator for applying horizontal reversal shear loading, as shown in Fig. 142. This is likely due to the fact that concrete being
exposed to 500°C becomes weak and the lateral cyclic loading caused the opening and closing of formed cracks. The opening and closing of cracks caused further softening of concrete. The shear frictional forces were reduced due to this softening behaviour. Consequently, the shear X-cracking damage was localised in the bottom critical region.

The shear failure of post-heated columns was gentle as compared to un-heated columns. The maximum lateral strength was achieved at the first cycle of the 10% drift ratio (42 mm displacement) in pushing at 75 kN lateral load for post-heated column S13 and at 78 kN lateral load for post-heated square column S14. The test was terminated when the lateral strength of the post-heated columns started to decrease with further increasing drift ratios. The cracking patterns and the failure modes of the posted-heated and un-heated columns were similar. Approaching the failure load, the diagonal shear crack, called the critical shear crack, became obviously wider in the un-heated columns than the post-heated columns.

For the post-heated GFRP wrapped column (S15) with main fibres in the circumferential direction, the failure mechanism is shown in Figs. 143 and 144. In the early stages of cyclic loading up to 5% drift capacity, no crack was observed. At 6% drift capacity white patches with cracking sound were observed during the pushing part of loading. The first visible flexural crack was observed during the first cycle of the 7% drift ratio (30 mm displacement) at 82 kN lateral pushing load. With increasing the lateral loading, several flexural cracks were developed with increasing cracking sounds, as shown in Fig. 143. These cracks were developed in the range of 60 mm to 200 mm above the interface of column support. The flexural cracks in the bottom critical region were localized with further increasing drift ratios.

Approaching the 12% drift ratio (51 mm lateral displacement) the critical flexural crack was extended all around the perimeter of the column cross-section at approximately 60mm above the top edge of bottom confining steel plate similar to circular columns. At 13% drift ratio the localized flexural crack was substantially
expanded and consequently caused the failure of the column by dropping the lateral strength. The maximum lateral strength was achieved during the first cycle of the 12% drift ratio (51 mm displacement) at 95 kN lateral load.

Fig. 144 shows the failure of post-heated column (S16) wrapped with a single layer of GFRP with the main fibres oriented in the longitudinal direction. At 7% drift capacity, white patches appeared at the interface of the confining steel plate and column. These patches were increased with increasing drift ratios and fracture of the GFRP jacket was started at the 8% drift ratio. The GFRP jacket fractured at the interface of the top edge of the bottom confining steel plate due to stress concentration when the drift capacity exceeded 12%, as shown in Fig. 144.

The maximum lateral strength achieved in the GFRP wrapped post-heated column (S16) was 98.5 kN during the first cycle of the 11% drift ratio (46.7 mm displacement). It is interesting to note that no shear or flexural cracks were observed in the GFRP wrapped column (S16) with the main fibres oriented in the longitudinal direction throughout the testing of the column. It was noted that the post-heated column (S16) wrapped with a single layer of GFRP with main fibres in the longitudinal direction attained slightly higher lateral strength than the post-heated column (S15) wrapped with GFRP with main fibres oriented in the transverse direction.

Fig. 145 shows the failure pattern of post-heated columns wrapped with a single layer of unidirectional carbon fibre reinforced polymer (CFRP) with main fibres in the transverse direction. In the early stages of lateral cyclic loading up to 6% drift ratio there was no evidence of any cracking in both CFRP wrapped post-heated columns. Approaching to 7% drift ratio white patches with cracking sounds were observed at a distance in the range of 30 mm to 200 mm above the top edge of column support in the pushing and pulling part of the loading in both CFRP wrapped post-heated columns.
The first visible flexural crack was observed during the first cycle of the 8% drift ratio (34 mm displacement) at approximately 80 kN pushing load in the CFRP wrapped columns (S17 and S18). In the subsequent displacement cycles, two flexural cracks were developed with increasing cracking sounds. These cracks were developed in the range of 40 mm to 150 mm above the interface of the column support. The flexural cracks formed close to the column critical bottom region and were localized with further increasing drift ratios and extended all around the perimeter of the column. In the following load cycles, one of the flexural cracks extended all around the perimeter of the column defining the critical failure plane, as shown in Fig. 145.

When the failure load is reached, white patches also appeared on the surface of carbon fibre at the corners of both columns indicating the stress concentration at these regions. At 12% drift ratio the critical flexural crack was extended all around the perimeter of the column at approximately 40 mm above the column support. The maximum lateral strength attained was 93.8 kN for column ‘S17’ and 94.48 kN for column ‘S18’ at the first cycle of the 12% drift ratio with 51 mm displacement in the pushing part of loading.

It was found that among GFRP and CFRP wrapped post-heated square columns with main fibres oriented in the transverse direction, the level of the lateral load was slightly higher in the GFRP wrapped post-heated columns than the CFRP wrapped post-heated columns at the same drift ratios. This is primarily due to the presence of additional fibres $90^\circ$ to the transverse direction in the GFRP jacket (Table 13). The GFRP jacket with main fibres oriented in the longitudinal direction contributed for both improving the flexural and shear strength of post-heated columns.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. 138: X- shear cracking pattern of un-heated square columns

Fig. 139: Concrete crushing failure of un-heated square columns
Fig. 140: Shear failure of un-heated square columns

Fig. 141: X-shear cracking pattern and concrete crushing failure in post-heated square columns
Fig. 142: Localised X-shear failure of post-heated square columns

Fig. 143: Flexural failure of post-heated square column wrapped with GFRP jacket with main fibres oriented in the transverse direction
Fig. 144: Fracture failure of GFRP jacket with main fibres oriented in the longitudinal direction

Fig. 145: Flexural failure of post-heated column wrapped with CFRP jacket with main fibres oriented in the transverse direction
7.5 ANALYSIS OF TEST RESULTS AND DISCUSSIONS

7.5.1 Square reinforced concrete columns

7.5.1.1 Hysteretic response

The hysteretic response in terms of lateral load versus lateral displacement of un-heated, post-heated and post-heated square columns wrapped with a single layer of GFRP or CFRP is shown in Figs. A.17 through to A.24 (in Appendix-A). For comparison, in all Figs. A.17 to A.24, the lateral loads versus lateral displacements were plotted with maximum displacement of 60 mm and lateral load of 100 kN in both the pushing and pulling directions. It can be seen from Figs. A.17 to A.20 that the loops in the post-heated columns (S13 & S14) were small compared to the un-heated columns (S11 & S12). This indicates the loss of strength and stiffness of the post-heated square columns. It is evident from Figs. A.21 to A.24 that the post-heated columns wrapped with GFRP or CFRP (S15 & S16 or S17 & S18) showed bigger loops as compared to the post-heated columns. This indicates the increase of strength, ductility and energy dissipation. It can be seen from Fig. A.22 that the loops in the pulling part of the loading are smaller as compared to the pushing part of the loading. This is primarily due to the fracture of the GFRP jacket. The bigger loops of post-heated columns wrapped with GFRP or CFRP jackets indicated the slower rate of strength and stiffness degradation.

7.5.1.2 Lateral load-displacement envelopes

In Fig. A.25, the curves ‘S11’ and ‘S12’ indicate the strength-displacement-envelopes of un-heated reference columns. In Fig. A.26, the curves ‘S13’ and ‘S14’ show the lateral strength-displacement envelops of post-heated reference columns. In Fig. A.27, the curve ‘S15’ indicates the lateral strength-displacement envelopes of post-heated column wrapped with a single layer of GFRP with main fibres oriented in the transverse direction. The curve ‘S16’ in Fig. A.28 indicates the load-displacement envelop of a post-heated column wrapped with a single layer of GFRP with main fibres oriented in the longitudinal direction. In Fig. A.29, the curves ‘S17’ and ‘S18’ show the load-displacement envelopes of post-heated columns wrapped with a single layer of CFRP jackets with main fibres oriented in the transverse direction.
Comparing Fig. A.25 with Fig. A.26 it can be seen that the lateral strength and stiffness of post-heated columns were significantly reduced after heating to 500°C. However, it is evident from the lateral load-displacement envelopes, shown in Figs. A.27 to A.29 that post-heated columns regained approximately the same lateral strength as that of the un-heated reference columns with greater lateral displacements (Table 32). Comparing Figs. A.25 to A.26 with Figs. A.27 to A.29, it can be seen that a single layer of GFRP or CFRP jacket substantially increased the lateral strength, energy dissipation, ductility and delayed the rate of stiffness and strength degradation of post-heated columns (described in the following sections in detail).

7.5.1.3 Lateral strength

Fig. 146 compares the lateral strength of un-heated, post-heated and post-heated columns wrapped with a single layer of unidirectional GFRP or CFRP jackets. The numbers 1 to 8 on x-axis represent:

1) Un-heated square reference column (S11)
2) Un-heated square reference column (S12)
3) Post-heated square reference column (S13)
4) Post-heated square reference column (S14)
5) Post-heated square column wrapped with Tyfo SEH-51A GFRP jacket with main fibres oriented in the transverse direction (S15)
6) Post-heated square column wrapped with Tyfo SEH-51A GFRP jacket with main fibres oriented in the longitudinal direction (S16)
7) Post-heated square column wrapped with Tyfo SCH-41 CFRP jacket with main fibres oriented in the transverse direction (S17)
8) Post-heated square column wrapped with Tyfo SCH-41 CFRP jacket with main fibres oriented in the transverse direction (S18)

Fig. 146 shows that the lateral strength of post-heated columns was significantly reduced after being exposed to 500°C. This strength was reduced to approximately 22% of the un-heated columns. The post-heated columns regained approximately the same original level of lateral strength of un-heated columns when wrapped with a
single layer of unidirectional glass or carbon fibre reinforced polymer. The lateral strength of post-heated columns wrapped with GFRP and CFRP jackets was increased by 27% and 23% of the post-heated columns respectively.

It is worth highlighting that all columns attained slightly more lateral strength in the pushing than in the pulling part of loading, as shown in Table 32 and Fig.146. It could be due to the occurrence of the first crack in the pushing part of the loading cycles. It was noted that the un-heated and post-heated columns sustained lateral load up to 10% drift ratios while GFRP or CFRP wrapped post-heated columns with main fibres oriented in the lateral direction had taken up to 12% drift ratios, as shown in Table 32. This indicates that the strength degradation was smaller in GFRP or CFRP wrapped post-heated columns than un-heated and post-heated columns with increasing drift ratios.

Fig.146 (a): Maximum lateral strength of square columns in pushing
7.5.1.4 Stiffness degradation

Fig. 147 and Table 33 shows the stiffness degradation of the un-heated, post-heated and post-heated square columns wrapped with a single layer of GFRP or CFRP jackets tested under similar condition of constant axial and lateral cyclic loading. The secant stiffness of the test specimens was calculated as described earlier in Section 7.3.1.5. This equals the slope of the line joining the peak of positive (pushing) and negative (pulling) loads at a given cycle, as shown in Fig. 135. The overall stiffness for each cycle was then calculated by taking the average of the stiffness in the pushing and the pulling directions. The secant stiffness provides a useful comparison of the overall response of the un-heated, post-heated and post-heated columns wrapped with a single layer of GFRP or CFRP jacket. The calculated secant stiffness of all columns was plotted against drift ratios, as shown in Fig. 147.
Fig. 147 provides the qualitative measure of the stiffness degradation of all tested columns. It can be seen from Fig. 147 that the initial stiffness values for post-heated columns (S13&S14) were substantially lower than the un-heated columns (S11&S12). This is likely due to the fact that the concrete after being exposed to 500°C becomes porous and soft due to micro cracking. It is worth highlighting that the un-heated (S11&S12) and post-heated columns (S13&S14) displayed a higher rate of stiffness degradation than the post-heated columns (S15&S16 or S17&S18) wrapped with a single layer of GFRP or CFRP jacket with increasing drift ratios.

However, the drop in stiffness degradation was more in the un-heated columns (S11&S12) as compared to post-heated columns (S13&S14). The decrease in stiffness of un-heated and post-heated columns at the end of the test was found to be approximately 72% and 69% respectively of their initial stiffness values. At the end of the un-heated and post-heated test, the GFRP or CFRP wrapped post-heated columns showed approximately 13% and 30% higher stiffness than the un-heated and post-heated columns respectively. The slower rate of stiffness degradation of post-heated columns wrapped with a single layer of GFRP or CFRP is an excellent property of columns during earthquakes as many reinforced concrete structures collapsed during past earthquakes due to a sudden loss of stiffness [49]. It is worth noting that the GFRP or CFRP jackets delayed the stiffness degradation of post-heated columns. However, the initial stiffness of post-heated columns was not improved by wrapping with a single layer of GFRP or CFRP jackets. Therefore to increase the initial stiffness of post-heated columns, other methods should be adopted or more than one layer of FRP may be tried to see the effect.
7.5.1.5 Ductility response

The main objective of repairing post-heated reinforced concrete square columns was to achieve a sufficient level of deformation ductility to dissipate seismic energy before failure. It can be seen in Figs. A.21 to A.24 that significant improvement in the hysteretic response of lateral load versus displacement was achieved when post-heated columns were wrapped with a single layer of GFRP or CFRP jacket. This clearly indicates the enhancement in the ductility of post-heated columns. It is also evident from Figs. A.27 to A.29 (Table 32) that the post-heated columns regained approximately the same level of lateral failure load as that of the un-heated reference columns with higher displacement when wrapped with a single layer of GFRP or CFRP jacket. This could be due to fact that GFRP or CFRP provided significant confinement to the micro cracked post-heated columns and thus resulted in more ductile behaviour compared to the un-heated and post-heated columns.

Fig. 147: Stiffness degradation of square columns with increasing drift ratios
7.5.1.6 Energy dissipation

The ability of a reinforced concrete structure to resist earthquake loading is evaluated in terms of energy dissipation. The evaluation of energy dissipation depends upon the hysteretic response of the structure. The area under the load-displacement envelopes at an ultimate load could be defined as the energy dissipated by the system [76, 77, 88]. The larger the area under full load displacement envelopes, the larger would be the dissipation of energy and the higher would be the damping effect [236]. Fig. 148 compares the energy dissipation in un-heated, post-heated and post-heated columns wrapped with a single layer of GFRP or CFRP tested under simulated earthquake loading. The numbers 1 to 5 on x-axis in Fig. 148 represent:

1) The average of the energy dissipation of two un-heated square columns (S11&S12)
2) The average of the energy dissipation of two post-heated square columns (S13&S14)
3) Energy dissipation in a post-heated square column wrapped with Tyfo SEH-51A GFRP jacket with main fibres oriented in the transverse direction (S15)
4) Energy dissipation in a post-heated square column wrapped with Tyfo SEH-51A GFRP jacket with main fibres oriented in the longitudinal direction (S16)
5) The average of the energy dissipation of two post-heated square columns wrapped with Tyfo SCH-41 CFRP jacket with main fibres oriented in the transverse direction (S17&S18)

It can be seen from Fig.148 and Table 34 that the greatest energy dissipation was for the post-heated GFRP or CFRP wrapped square columns. It is worth noting that the energy dissipated in a post-heated square column wrapped with a GFRP jacket with main fibres oriented in the longitudinal direction was more than the post-heated columns wrapped with GFRP or CFRP jackets with main fibres oriented in the transverse direction in pushing part of loading. However, the energy dissipation in a post-heated square column wrapped with a GFRP jacket with main fibres oriented in longitudinal direction was slightly less as compared to columns wrapped with GFRP or CFRP with the main fibres oriented in the transverse direction in the pulling part of
loading, as shown in Table 34. This is due to the fact that unfortunately a GFRP jacket with main fibres oriented in the longitudinal direction was fractured during pulling due to stiff edge of steel stub, as shown in Fig. 144.

It is evident from Fig. 148 and Table 34 that among the post-heated columns wrapped with a GFRP or CFRP jacket with main fibres oriented in the transverse direction, columns wrapped with a GFRP jacket showed more energy dissipation than columns wrapped with a CFRP jacket. This is likely due to the fact that the GFRP jacket had additional strength 90 degrees to the primary fibres, as shown in Table 13. It can be seen from Figs. A.25 to A.29 that the post-heated columns wrapped with GFRP or CFRP jackets attained the same failure lateral strength as that of un-heated columns but with higher displacement. This indicates that the GFRP or CFRP wrapped post-heated columns showed a better response in terms of energy dissipation and ductility than the un-heated and post-heated columns. It can be seen from Table 31 that the un-heated and post-heated columns sustained lateral load up to 10% drift ratios while GFRP or CFRP wrapped post-heated columns with main fibres oriented in the lateral direction had taken up to 12% drift ratios. Therefore the inelastic deformations of GFRP or CFRP wrapped post-heated columns were more as compared to un-heated and post-heated columns. Normalised energy dissipation was also considered for comparison, as shown in Table 34 by taking un-heated columns as the reference columns.

Based on the test results it is therefore confirmed that the energy dissipation of fire damaged reinforced concrete square columns can be enhanced remarkably by wrapping with a single layer of glass or carbon fibre reinforced polymers to respond to future earthquakes. Additionally, the fibres oriented in the longitudinal direction or bi-directional fibres could provide more energy dissipation than unidirectional fibres oriented in the transverse direction.

### 7.5.1.7 Effect of fibre orientation on cracking pattern and strength of columns

The cracking pattern of un-heated, post-heated and post-heated wrapped with a single layer of GFRP or CFRP square columns provided useful information regarding
the distribution of stresses and the failure mechanism. It can be seen from Figs.138 to 145 that un-heated square columns failed drastically in a brittle shear failure mode. The post-heated square columns showed a similar shear failure mode as observed in the un-heated columns. The shear X-cracking was localized in the bottom critical region of the post-heated columns instead of moving up-wards to the point of application of lateral loading as observed in the un-heated square columns. The failure mechanism of post-heated columns wrapped with a single layer of unidirectional glass or unidirectional carbon fibre reinforced polymer with main fibres in the transverse direction was shifted from column shear failure mode to a flexural failure mode, as shown in Figs.142 and 145. This is primarily due to the fact that unidirectional glass or carbon fibres in the transverse direction increase the shear strength of the post-heated columns to such an extent that at 12% drift ratio the column shear strength still remains within its capacity. This causes the shifting of column shear failure mode to flexural failure.

When post-heated square columns were wrapped with a single layer of GFRP or CFRP with main fibres oriented in the transverse direction, the flexural cracks were developed close to the column bottom critical region. These flexural cracks were localized within the bottom critical region and extended all around the perimeter of the square column, as shown in Figs.142 and 145. In un-wrapped columns it was observed that the shear crack was initiated at the bottom critical region and moved upward diagonally to the point of application of lateral loading. No evidence of cracks was observed above the point of application of the lateral loading. Therefore for the shear strengthening of reinforced concrete columns, the fibre reinforced polymer jacket should be wrapped over the height of the column up to the level of application of lateral cyclic loading.

The tensile stresses in the fibre reinforced polymers oriented in the transverse direction under seismic forces significantly contributed to the overall shear resistance of the columns. This could be attributed to the fact that shear cracks occurred at an angle of 45° to the direction of fibres, as shown in Figs. 138 to 141 and 149 (a), 149
(b). The tensile stresses in the fibres reduce or minimize the opening of the shear cracks and excellently impart shear strength to the columns. This admirable contribution of fibres in the seismic shear resistance causes the shifting of column shear failure mode to flexural failure.

The fibres oriented in the transverse direction also contributed in improving the axial load carrying capacity of post-heated columns. The post-heated concrete after being exposed to higher temperature expands more laterally under axial compression. This expansion causes tensile stress to develop in the fibres oriented in the hoop or transverse direction. The tensile stresses in the fibres resist the opening of vertical cracks developed under axial compression, as shown in 149(b) and increase the axial strength of the columns leading to a more ductile and stronger structural member during earthquakes. It can be seen from Figs.132, 133,143,145,149 (b) that under lateral cyclic loading, the flexural cracks were developed parallel to the directions of the main fibres oriented in the hoop or transverse direction. Therefore, the unidirectional fibres oriented in the hoop or transverse direction will not contribute to improving the flexural strength of post-heated columns.

It is evident from Fig. 144 that no shear or flexural cracks were observed when main fibres were oriented in the longitudinal direction. This indicates that the fibre reinforced polymer jackets with main fibres oriented in the longitudinal direction were very effective in increasing both the flexural and shear capacity of columns. This is due to the fact that the flexural cracks occurred at an angle of 90° to the direction of the main fibres at the bottom critical region of the columns under reversed lateral cyclic loading, as shown in Fig. 149 (a). The tensile stresses in the fibres reduce or minimize the opening of the flexural cracks, as shown in Fig. 144, and consequently the flexural strength of column was improved considerably. Additionally, when the main fibres oriented in the longitudinal direction, the shear crack occurred at an angle of 45° to the direction of fibres, as shown in Fig. 149 (a). Therefore, the tensile stresses in the fibres oriented longitudinally contributed significantly to improving both the shear strength and flexural capacity of columns, as shown in Fig. 144.
However, under axial compression, the unidirectional fibres oriented longitudinally will not contribute to preventing the lateral dilation of the concrete because the vertical crack occurs parallel to the direction of the fibres, as shown in Fig.149 (a). It is concluded that only the lateral or only the longitudinal confinement of columns with unidirectional fibres is not sufficient to increase the overall capacity of columns. Therefore it is recommended that to increase the shear, flexural and axial capacity of columns, the columns should be wrapped with combined lateral or hoop and longitudinal fibres of unidirectional glass or carbon fibre reinforced polymer sheets or bi-directional fibre reinforced polymers should be used to increase axial, shear and flexural capacity of post-heated columns.

![Energy Dissipation in Square Columns](image_url)

*Fig.148: Energy dissipation in square columns*
7.5.1.8 Restraining effect of bottom steel stub

All square columns were tested in the upright cantilever position confined at the bottom from two sides with a stiff steel plate up to 200 mm height in the pushing and pulling loading direction. The maximum moment during pushing and pulling is considered at the interface of the column and the top edge of the bottom stiff steel plate being the most critical section. However, it was found that the failure was initiated at a shorter distance away from the critical top edge of the confining bottom stiff steel plate, as shown in Figs. 143 and 145 similar to the FRP wrapped circular columns shown in Figs. 132 and 133. This is likely due to the fact that the bottom stiff steel plate offered an additional restraining effect to the column section near to the interface of the top edge of the bottom steel plate and column. This restraining effect of the stiff steel plate reduced the tendency of lateral expansion. Therefore, the moment capacity of the column in the locality close to the top edge of steel stub was increased and consequently the failure was shifted to a shorter distance away from the critical region of the top edge of the steel plate. This observation supports the
previous findings of Sheik.S.A,Memon and Lacobucci [84, 86, 102, 106, 237] during the investigation of carbon or glass fibre reinforced polymer confined concrete square columns monolithically cast with strong concrete stub.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
<thead>
<tr>
<th>Test conditions</th>
<th>Drift ratio [%]</th>
<th>Maximum horizontal load in pushing [kN]</th>
<th>Maximum displacement in pushing [mm]</th>
<th>Maximum horizontal load in pulling [kN]</th>
<th>Maximum displacement in pulling [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Un-heated column-1</td>
<td>10 (1st cycle)</td>
<td>96.8</td>
<td>42.3</td>
<td>93.4</td>
<td>42.4</td>
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<tr>
<td>Un-heated column-2</td>
<td>10 (1st cycle)</td>
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<td>42.4</td>
<td>93.7</td>
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<tr>
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<td>42.3</td>
<td>71.1</td>
<td>42.5</td>
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<tr>
<td>Post-heated column-2</td>
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<td>42.4</td>
<td>73.4</td>
<td>42.5</td>
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<tr>
<td>Post-heated GFRP wrapped column-1</td>
<td>12-(1st cycle)</td>
<td>95.4</td>
<td>50.8</td>
<td>87.3</td>
<td>51</td>
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<tr>
<td>(Transverse direction)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Post-heated GFRP wrapped column-2</td>
<td>11(1st cycle)</td>
<td>98.5</td>
<td>46.6</td>
<td>87.4</td>
<td>46.7</td>
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<tr>
<td>(Longitudinal direction)</td>
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<td></td>
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<tr>
<td>Post-heated CFRP wrapped column-1</td>
<td>12(1st cycle)</td>
<td>93.8</td>
<td>50.7</td>
<td>89.2</td>
<td>50.9</td>
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<tr>
<td>(Transverse direction)</td>
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<tr>
<td>Post-heated CFRP wrapped column-2</td>
<td>12(1st cycle)</td>
<td>94.5</td>
<td>50.9</td>
<td>91</td>
<td>51</td>
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<tr>
<td>(Transverse direction)</td>
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Table 32: Maximum lateral loads and displacements in square columns
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

<table>
<thead>
<tr>
<th>Drift ratio [%]</th>
<th>Secant stiffness [kN/mm]</th>
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<tr>
<td></td>
<td>Un-heated columns</td>
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<tr>
<td>0.5</td>
<td>5.556</td>
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<tr>
<td>1</td>
<td>5.880</td>
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<tr>
<td>2</td>
<td>4.733</td>
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<tr>
<td>3</td>
<td>4.174</td>
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<tr>
<td>4</td>
<td>3.759</td>
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<td>5</td>
<td>3.214</td>
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<td>6</td>
<td>3.006</td>
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<td>7</td>
<td>2.797</td>
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<tr>
<td>8</td>
<td>2.606</td>
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<tr>
<td>9</td>
<td>2.377</td>
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<td>10</td>
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<td>1.867</td>
</tr>
<tr>
<td>12</td>
<td>1.530</td>
</tr>
<tr>
<td>13</td>
<td>-</td>
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</table>

Failure mode: Shear failure, Shear failure, Flexural failure, Flexural failure

Table 33: Stiffness degradation in square columns
<table>
<thead>
<tr>
<th>Test conditions</th>
<th>Energy dissipation [kN.mm]</th>
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<tbody>
<tr>
<td></td>
<td>Pushing part of loading</td>
<td>Pulling part of loading</td>
<td>Total</td>
<td>Normalised</td>
</tr>
<tr>
<td>Un-heated columns</td>
<td>1515.56</td>
<td>1441.24</td>
<td>2956.8</td>
<td>1</td>
</tr>
<tr>
<td>Post-heated columns</td>
<td>1231.12</td>
<td>1019.46</td>
<td>2250.57</td>
<td>0.76</td>
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<tr>
<td>Post-heated columns wrapped with GFRP(Transverse direction)</td>
<td>1752.58</td>
<td>1651.05</td>
<td>3403.63</td>
<td>1.15</td>
</tr>
<tr>
<td>Post-heated columns wrapped with GFRP(Longitudinal direction)</td>
<td>1769.88</td>
<td>1426.54</td>
<td>3196.42</td>
<td>1.08</td>
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<tr>
<td>Post-heated columns wrapped with CFRP(Transverse direction)</td>
<td>1591.48</td>
<td>1587.35</td>
<td>3178.84</td>
<td>1.07</td>
</tr>
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</table>

Table 34: Energy dissipation in square columns
CHAPTER-8

8 CONCLUSIONS AND FUTURE RECOMMENDATIONS

8.1 INTRODUCTION

This chapter concludes all the research work and provides an insight into the effectiveness of fibre reinforced polymer or ferrocement for the repairing of post-heated reinforced concrete columns. A total of thirty-five (seventeen circular and eighteen square) reinforced concrete columns were tested in this study. Thirteen circular and fourteen square columns were initially damaged by heating up to a uniform temperature of 500°C. The post-heated columns were subsequently repaired using epoxy resin mortar, cement sand mortar, concrete, ferrocement, unidirectional carbon and glass fibre reinforced polymers. The experimental research programme was conducted in two parts. In experimental part-1 ten square and nine circular columns were tested under axial loading only. In experimental part-2 eight square and eight circular columns, with a shear span-to-depth ratio of 2.5, were tested under combined constant axial and reversed lateral cyclic loading in order to simulate the gravity and earthquake loading. The test specimens in each part of the experimental programme were divided into main three categories, un-heated, post-heated and post-heated repaired columns. The results of unheated and post-heated reference columns were compared with post-heated repaired columns. This reported research work has significant importance for the construction industry regarding the repairing of fire damaged reinforced concrete columns.

8.2 CONCLUSIONS

Based on the analysis of the experimental results reported in the thesis, the residual compressive strength of concrete on the 7th day of cooling at ambient after exposing to 200°C, 300°C, 450°C, 500°C and 550°C was found to be 80%, 76%, 60%, 47% and 30% of original un-heated value respectively. The ultrasonic pulse velocity values measured in post-heated cubes on the 7th day of cooling in ambient air after exposing to 200°C, 300°C, 450°C, 500°C and 550°C were found to be 4.6 km/s, 4.1...
km/s, 3.9 km/s, 3 km/s, 2.3 km/s and 1.1 km/s respectively. The ultrasonic test is very useful for estimating the uniformity and residual strength of fire damaged concrete. However, the development of extensive cracks in post-heated concrete after exposing to 550°C prevented the propagation of ultrasonic pulse velocity resulting in abnormal values.

The axial compressive strengths of square and circular columns were reduced by 44% and 42% respectively after exposing to a uniform temperature of 500°C. The cross-sectional shape has a very important role in increasing the strength and ductility of post-heated columns confined with GFRP or CFRP. In this study, the post-heated circular cross-section wrapped with GFRP or CFRP jackets was found to be more efficient compared to the square cross-section in terms of increasing the axial strength and ductility. This is attributed to the fact that square cross-sections contain some ineffectively confined concrete regions and intensification of stresses at their corners while circular cross-sections contain fully effective confined concrete regions when wrapped with GFRP or CFRP jackets. For the circular post-heated columns, the axial strength was restored to, or was increased above, the original level of the unheated circular columns while the square cross-sections regained significant axial strength but not to the level of their original pre-heated strength, when wrapped with a single layer of GFRP or CFRP jackets after heating to 500°C.

The shear capacity of post-heated square and circular columns was reduced by 22% and 30% of the un-heated columns when tested under combined constant axial and reversal lateral loading. The unheated and post heated square and circular columns failed in a dramatic shear failure mode. The GFRP or CFRP wrapped post-heated square and circular columns attained the same level of lateral strength as that of unheated columns with higher displacement and failed in a flexural failure mode instead of shear failure. This indicates the excellent seismic performance of post-heated columns following repair with GFRP or CFRP jackets. The use of a ferrocement jacket for the repairing of post-heated square and circular columns enhanced the axial stiffness and ultimate load carrying capacity of columns significantly.
The most important conclusions drawn from this research work are:

- The GFRP or CFRP is very effective in improving the compressive strength of fire damaged concrete square and circular reinforced concrete columns. This is due to the post-heated columns becoming ‘soft’ after heating and displaying more lateral dilation compared to unheated columns. However, the confinement effect of the GFRP or CFRP was greater in post-heated circular sections compared to the square columns.

- The concrete become porous after heating and the stiffness of post heated columns was reduced significantly. The use of a single layer of GFRP or CFRP jacket had negligible effect on the stiffness of both the square and circular post-heated reinforced concrete columns. This could be due to the little activation of FRP jackets from start of compressive loading up to the maximum strength of post-heated columns. The GFRP or CFRP confinement was activated significantly after reaching the maximum strength of post-heated columns. 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. Thereafter, the behaviour of post-heated columns wrapped with GFRP or CFRP jackets was improved in terms of strength due to significant activation of FRP with more deformation rather than the stiffness.

- GFRP or CFRP jackets performed in an excellent way for increasing the shear capacity, ductility, energy dissipation and slowing the rate of strength and stiffness degradation of fire damaged reinforced concrete square and circular columns subjected to future earthquakes. Therefore, fibre reinforced polymer can be used effectively for the repairing of fire damaged concrete structures existing in the seismic zones. However the use of a single layer of glass or carbon fibre reinforced polymer could not recover the initial stiffness of post-heated columns up to the level of un-heated columns.
The current existing international FRP confinement models such as American Concrete Institute (ACI Committee 440.2R-02), Canadian Standard Association (CSA-S806-02) and Concrete Society Technical Report No.55 are based on un-heated concrete. Currently no design guidelines are available for the prediction of confined compressive strength of fire damaged concrete columns. The existing international FRP confinement model equations given in ACI Committee 440.2R-02 and CSA-S806-02 are highly conservative for the evaluation of confined compressive strength of fire damaged concrete columns wrapped with fibre reinforced polymer. The confinement models predict the confined compressive strength of circular post-heated columns more conservatively compared to the post-heated square reinforced concrete columns.

This thesis provides the present knowledge and experimental evidence to the research community for use the fibre reinforced polymer in fire damaged reinforced concrete columns. The existing models for the prediction of confined compressive strength of columns are based on un-heated concrete. This thesis provides the information for the confined compressive strength of post-heated reinforced concrete columns wrapped with fibre reinforced polymers. Many professional published technical reports regarding the repairing of fire damaged concrete structures such as Concrete Society Technical Report 68 suggested the fibre reinforced polymer for the repairing of fire damaged concrete structures. However, still there is no published data to clearly understand the behaviour of fire damaged concrete structure repaired with fibre reinforced polymer. This thesis provides the confidence and awareness to the practising engineers who are involved in the field of repairing of fire damaged concrete structures using advanced composite smart materials.

8.3 RECOMMENDATIONS FOR FUTURE WORK

Through the course of this experimental research, the following areas have been identified for a broader scope of further research.

1) In this research work small scale columns were heated to a uniform temperature of and 500°C tested to investigate the effectiveness of fibre reinforced polymer
jackets for axial compressive and seismic shear performance of post-heated columns. In real fires, the temperatures greater than 900°C are frequent within buildings. However, in a concrete member, only the temperature of the outside layers is drastically increased, and the temperature of the internal concrete may be comparatively low due to the low thermal conductivity of concrete. Therefore, further research needs to be conducted on full scale columns exposed to greater than 900°C external temperature to further verify the effectiveness of fibre reinforced polymer or ferrocement jacket for the repair of post-heated columns.

2) In the present study, the exposure of columns to a uniform temperature of 500°C results in a significant loss of axial stiffness. The degradation of post-heated columns stiffness is mainly caused by the softening of concrete after heating due to loss of moisture and due to the development of internal micro-cracking. The use of a single layer of GFRP or CFRP jacket had negligible effect on the stiffness of both square and circular post-heated columns. Therefore, further research is required to investigate the effect of more than one layer of FRP on the stiffness of post-heated columns or other methods together with FRP should be investigated.

3) In this study, for the circular post-heated columns, the axial strength was restored to, or was increased above, the original level of the unheated circular columns while the square cross-sections regained significant axial strength but not to the level of its original pre-heated strength, when wrapped with a single layer of GFRP or CFRP jacket after heating to a uniform temperature of 500°C. Further study is needed to investigate the number of FRP layers required to restore the original strength of post-heated square columns.
References


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## APPENDIX-A

### Strain gauge properties

<table>
<thead>
<tr>
<th>Gauge Type</th>
<th>PFL-30-11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gauge Length</td>
<td>30mm</td>
</tr>
<tr>
<td>Gauge Width</td>
<td>2.3mm</td>
</tr>
<tr>
<td>Gauge Resistance</td>
<td>120 Ohms</td>
</tr>
<tr>
<td>Gauge Factor</td>
<td>2.13</td>
</tr>
<tr>
<td>Adhesive</td>
<td>Polyester resin (P-2)</td>
</tr>
</tbody>
</table>

Table A.1: Strain gauge properties

<table>
<thead>
<tr>
<th>S.NO</th>
<th>size of cube (mm)</th>
<th>Weight kg</th>
<th>28 days cube compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100x101x100</td>
<td>2.330</td>
<td>42.55</td>
</tr>
<tr>
<td>2</td>
<td>100x101x100</td>
<td>2.335</td>
<td>41.43</td>
</tr>
<tr>
<td>3</td>
<td>101x101x101</td>
<td>2.350</td>
<td>41.96</td>
</tr>
</tbody>
</table>

Average Compressive Strength 42

<table>
<thead>
<tr>
<th>Test condition</th>
<th>150 mm size cubes</th>
<th>100 mm size cubes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Compressive strength (MPa)</td>
<td>Pulse velocity(km/s)</td>
</tr>
<tr>
<td></td>
<td>Cube-1</td>
<td>Cube-2</td>
</tr>
<tr>
<td>20°C</td>
<td>51.60</td>
<td>53.55</td>
</tr>
<tr>
<td>200°C</td>
<td>44.15</td>
<td>43.03</td>
</tr>
<tr>
<td>300°C</td>
<td>41.08</td>
<td>42.42</td>
</tr>
<tr>
<td>450°C</td>
<td>34.24</td>
<td>30.14</td>
</tr>
<tr>
<td>500°C</td>
<td>25.36</td>
<td>24.98</td>
</tr>
<tr>
<td>550°C</td>
<td>16.11</td>
<td>15.58</td>
</tr>
</tbody>
</table>

Table A.2: 28 days compressive strength

Table A.3: Measured Compressive Strength and Pulse Velocity in 150 mm cubes

Table A.4: Measured Compressive Strength and Pulse Velocity in 100 mm cubes
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. A.1: Stress-strain curves of 6 mm diameter reinforcing steel tested at ambient temperature

Fig. A.2: Stress-strain curves of 10 mm diameter reinforcing steel tested at ambient temperature
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

![Stress-strain curves](image1.png)

**Fig.A.3:** Stress-strain curves of 10 mm diameter reinforcing steel heated to 500°C and tested at ambient temperature.

![Stress-strain curves](image2.png)

**Fig.A.4:** Stress-strain curves of 6 mm diameter reinforcing steel heated to 500°C and tested at ambient temperature.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. A.5: Hysteretic response of un-heated circular column C10

Fig. A.6: Hysteretic response of un-heated circular column C11
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig.A.7: Hysteretic response of post-heated circular column C12

Fig.A.8: Hysteretic response of post-heated circular column C13
Fig. A.9: Hysteretic response of post-heated GFRP wrapped circular column C14

Fig. A.10: Hysteretic response of post-heated GFRP wrapped circular column C15
Fig.A.11: Hysteretic response of post-heated CFRP wrapped circular column C16

Fig.A.12: Hysteretic response of post-heated CFRP wrapped circular column C17
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. A.13: Lateral load-displacement envelopes of un-heated circular columns C10&C11

Fig. A.14: Lateral load-displacement envelopes of post-heated circular columns C12&C13
Fig. A.15: Lateral load-displacement envelopes of post-heated GFRP wrapped circular columns C14 & C15

Fig. A.16: Lateral load-displacement envelopes of post-heated CFRP wrapped circular columns C16 & C17
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

**Fig. A.17**: Hysteretic response of un-heated square column S11

**Fig. A.18**: Hysteretic response of un-heated square column S12
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. A.19: Hysteretic response of post-heated square column S13

Fig. A.20: Hysteretic response of post-heated square column S14
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. A.21: Hysteretic response of post-heated GFRP wrapped square column S15 with main fibres oriented in transverse direction

Fig. A.22: Hysteretic response of post-heated GFRP wrapped square column S16 with main fibres oriented in longitudinal direction
Fig. A.23: Hysteretic response of post-heated CFRP wrapped square column S17 with main fibres oriented in transverse direction.

Fig. A.24: Hysteretic response of post-heated CFRP wrapped square column S18 with main fibres oriented in transverse direction.
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. A.25: Lateral load-displacement envelopes of un-heated square columns S11 & S12
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. A.26: Lateral load-displacement envelopes of post-heated square columns S13 & S14

Max. Pushing loads & displacements
Column-S13: 74.6kN; 42.3mm
Column-S14: 78.2kN; 42.4mm

Max. Pulling loads & displacements
Column-S13: 71.1kN; 42.5mm
Column-S14: 73.4kN; 42.5mm

Strength-displacement envelops of post-heated columns S13 & S14

Fig. A.27: Lateral load-displacement envelopes of post-heated GFRP wrapped square column S15 with main fibres oriented in transverse direction

Max. Pushing loads & displacements
Column-S15: 95.4kN; 50.8mm

Max. Pulling loads & displacements
Column-S15: 87.3kN; 51mm

Displacement [mm]
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig. A.28: Lateral load-displacement envelopes of post-heated GFRP wrapped square column S16 with main fibres oriented in longitudinal direction

- Max. Pushing loads & displacements
  - Column-S16: 98.5 kN; 46.6 mm

- Max. Pulling loads & displacements
  - Column-S16: 87.4 kN; 46.7 mm

Strength-Displacement envelop of post-heated column-S16 wrapped with GFRP jacket in the longitudinal direction

Fig. A.29: Lateral load-displacement envelopes of post-heated CFRP wrapped square columns S17&S18 with main fibres oriented in the transverse direction

- Max. Pushing loads & displacements
  - Column-S17: 93.8 kN; 50.7 mm
  - Column-S18: 94.4 kN; 50.9 mm

- Max. Pulling loads & displacements
  - Column-S17: 89.4 kN; 50.9 mm
  - Column-S18: 91.1 kN; 51 mm

Displacement [mm]

- Lateral Load [kN]
Fig.A.30: Strain gauge variations (a) un-heated circular column C1 (b) un-heated circular column C2

Fig.A.31: Strain gauge variations (a) post-heated circular column C3 without spalling (b) post-heated seriously spalled circular column C5 repaired with epoxy resin mortar
Fig. A.32: Strain gauge variations (a) Tyfo SEH-51A GFRP wrapped post-heated circular column C4 without spalling (b) Tyfo SEH-51A GFRP wrapped epoxy resin mortar repaired post-heated circular column C7

Fig. A.33: Strain gauge variations (a) Weber.tec force C-240 wrapped post-heated circular column C6 (b) Tyfo SCH-41 CFRP wrapped epoxy resin mortar repaired post-heated circular column C8
Axial compressive and seismic shear performance of post-heated columns repaired with composite materials

Fig.A.34: Strain gauge variations (a) un-heated square column S1 (b) un-heated square column S2

Fig.A.35: Strain gauge variations (a) Post-heated square column S3 (b) Post-heated square column S4
Fig. A.36: Strain gauge variations (a) Tyfo SEH-51A GFRP wrapped post-heated square column S5 (b) Tyfo SEH-51A GFRP wrapped post-heated square column S6

Fig. A.37: Strain gauge variations Tyfo SCH-41 CFRP wrapped post-heated square column S7
Fig.A.38: Strain gauge variations (a) Weber.tec force C-240 wrapped post-heated square column S8 (b) Weber.tec force C-240 wrapped post-heated square column S9