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Post-fire Repair of Concrete Structural Members: A Review on Fire Conditions and Recovered Performance

Jin Qiu[1], Liming Jiang[1], and Asif Usmani[1]

[1]*Department of Building Services Engineering, The Hong Kong Polytechnic University, Hong Kong, China*

Abstract

Concrete structures may rarely collapse in fire incidents but fire induced damage to structural members is inevitable as a result of material degradation and thermal expansion. This requires certain repairing measures to be applied to restore the performance of post-fire members. A brief review on investigation of post-fire damage of concrete material and concrete structural members is presented in this paper, followed by a review of post-fire repair research regarding various types of repairing techniques (FRP, steel plate, and concrete section enlargement) and different type of structural members including columns, beams, and slabs. Particularly, the fire scenarios adopted in these studies leading to damage are categorized as three levels according to the duration of gas-phase temperature above 600°C (t_{600}). The repair effectiveness in terms of recovered performance of concrete structural members compared to the initial undamaged performance has been summarized and compared regarding the repairing techniques and fire intensity levels. The complied results have shown that recovering the ultimate strength is achievable but the stiffness recovery is difficult. Moreover, the current fire loading scenarios adopted in the post-fire repair research are mostly idealized as constant heating rates or standard fire curves, which may have produced unrealistic fire damage patterns and the associated repairing techniques may be not practical. For future studies, the realistic fire impact and the system-level structural damage investigation are necessary.

Keywords: Post-fire performance, concrete structures, structural performance, fire loading, post-fire repair

1. Introduction

Concrete members of a building structure are usually considered of better performance in fire incidents compared to bared steel members for the low thermal conductivity of concrete (Ding and Li 2018). A survey conducted by the Concrete Industry (2008) has concluded that concrete buildings rarely collapse due to fire or high temperature shocks, and this conclusion can be once again supported by the Grenfell tower fire in 2017. However, fire induced damage to the concrete structural members vastly exists, which may be caused by the material degradation at elevated temperature, the restraint effect due to thermal expansion/contraction, and the structural load redistribution in fire. The fire damaged structural members thereafter require adequate repairing measures, because the initially designed structural performance may be hardly achieved. The so-called repair is to restore the load-bearing performance of post-fire structural members so that the structures can regain its service ability and resume its service immediately. Various repairing techniques (e.g., section enlargement or jacket) have been proposed in the last two decades for fire damaged structural members and the corresponding repairing effectiveness has been studied by experimental tests. Two review papers of post-fire repair were published (Ma et al. 2019; Zhou and Wang 2019). Both the review papers focus on the details of repairing techniques and the analytical models for the repaired structural members. However, the fire scenarios associated with the fire-induced damage and the repairing effectiveness investigation have not been interpreted. Moreover, the fire scenarios experienced by structural members have great effect on the repairing effectiveness. There is no unified standard at present to classify the fire scenarios adopted in these studies related to repair post-fire structures.

This paper discusses the post-fire performance of concrete material and structural members. Based on the statistical analysis of the duration after the temperature exceeds 600°C (t_{600}), the three-level fire scenario classification is proposed. The repairing effectiveness of various repairing techniques on structural members damaged by different levels of fire scenario is summarized. The purpose of the paper is to review the literature of current works and approaches future research on this subject.

2. Post-fire performance investigation

2.1. Post-fire performance investigation on concrete material

Concrete exposed to fire may undergo mass loss, significant

†Corresponding author: Liming Jiang
Tel: +852-27665844, Fax: +852-27657198
E-mail: liming.jiang@polyu.edu.hk

degradation of mechanical properties, cracking and even spalling (Bastami et al. 2014; Phan et al. 2001). These are due to the physical and chemical changes of cement paste and aggregate of concrete as well as thermally induced incompatibility owing to elevated temperature (Fu et al. 2004; Ichikawa and England 2004). Experimental studies conducted by Sarshar and Khoury (1993) found that the aggregate types of concrete along with the heating and post-heating conditions could significantly affect the post-fire performance of concrete. A slower heating rate (Sarshar and Khoury 1993) and higher peak temperature (Chang et al. 2006; Ergün et al. 2013; Phan et al. 2001; Poon et al. 2001a) result in larger strength loss due to higher temperature distribution achieved in the cross section. Moreover, the mechanical properties of concrete exposed to fire may further deteriorate during the cooling phase. The Eurocode 4 (CEN 2005) suggests a further 10% strength loss after being cooled down from the maximum temperature, while the maximum temperature of concrete is beyond 300ºC. Tests carried out by Li and Franssen (2011) revealed that the additional compressive strength reduction during cooling phase could be as high as 20% of the strength at the maximum temperatures. Studies regarding the cooling rate have shown that higher strength reduction of concrete occurred with faster decreasing temperature, such as cooling concrete by quenching or spraying water (Botte and Caspeele 2017; Lee et al. 2008; Nassif 2006; Peng et al. 2008; Sarshar and Khoury 1993).This was explained as a greater 'thermal shock' to concrete in the cases of higher rate of temperature decreasing (Peng et al. 2008).

For the post-fire curing conditions, water/humid post-fire curing was applied to recover the strength of concrete as the strength recovery is dependent on the rehydration process of dehydrated concrete and water plays crucial rolel (Park et al. 2015; Poon et al. 2001b;). Test results showed that the residual compressive strength of high-strength concrete exposed to 600ºC could be recovered to up to 93% of the original compressive strength after being kept in water-curing conditions for 56 days (Poon et al. 2001b). This is owing to the C-S-H gel starts to decompose at about 550ºC. Strength recovery, however, is a slow and long-term process and post-fire concrete can even experience a further strength degradation after being cooled down before they begin to recover strength (Lin et al. 2011; Papayianni and Valiasis 1991; Poon et al. 2001b; Yang et al. 2009). On a material level, these research efforts have suggested that the fire development (characterized by heating rate, peak temperatures and cooling rate), post-fire curing condition and duration can all affect the residual strength of concrete.

2.2. Post-fire performance investigation on structural members

As a result of fire induced damage to concrete material and other heating impact, the post-fire performance of reinforced concrete (RC) structural members may be weakened. As shown in Table 1, various studies on RC columns, beams and slabs were conducted and reported, which primarily focused on the reduced load bearing capacity of the RC members after fire exposure. The representative findings from these studies have been summarized in the table, along with the member types, specimen types, aggregate types, heating conditions, load ratios under fire and the referencing source of research. For columns, the effective cross section reduced due to the spalling of concrete (Kodur et al 2013), whereas the steel ratio could significantly increase the residual load bearing capacity (Li et al. 2019b). The effect of steel rebars was also observed in RC beams, and the flexural capacities of beams were affected by the steel rebar temperature (Kodur and Agrawal 2016) and the post-fire conditions (Akca and Özyurt 2020). For post-fire slabs, cracks were found in the loaded slabs during the fire exposure usually imposed as furnace heating, while the residual load bearing capacity can be reduced (Wang et al. 2018). These experimental studies were usually conducted on full-size structural members, providing an insight into the fire induced damage at a member level. However, the limitation owing to the individual member test should be noticed, since the damage due to global load redistribution mechanism and restraint effect of adjacent structure cannot be reproduced. In general, the reduced capacity of RC structural members poses various levels of threat to the structural safety, which requires performance recovery by means of repairing measures.

3. Repair of post-fire structural members

To recover the structural performance of post-fire RC structural members, specific repairing measures are necessary. Figure 1 illustrates different repairing techniques for post-fire structural members that have been investigated in recent years. Section enlargement (Figure 1a) is a straightforward approach to repair the fire damaged members, which is easy-to-apply and cost-effective. However, the drawbacks such as increasing the structural self-weight and reducing indoor space are recognized. Fiber-Reinforced Polymer (FRP) becomes an alternative measure for its outstanding strength-to-weight ratio, great corrosion resistance, easy installation and excellent compatibility to various structural geometry (Balaguru et al. 2008). As shown in Figure 1b,1c, and 1d, FRP can be applied as FRP jackets, externally-bonded (EB) FRP sheets and near-surface-mounted (NSM) FRP rods/sheets to improve the structural performance. Compared to applying FRP sheets externally, NSM FRP inserts FRP rods/sheets to the concrete substrate to achieve a better bonding performance between FRP and original structural members. Due to temperature sensitivity of epoxy resins, insulation should be applied to the FRP repaired structural members and the fire resistance can be

Table 1. Summary of experimental studies on post-fire performance of structural members

Structural members	Specimen type[1]	Heating condition	Load ratio	Findings	Source
Columns	RC (S)	ISO 834 (2 / 4h)	0.2	The strength reductions were 27% and 38% after heating for 2 and 4 h, respectively. The strength recovery of steel bars after cooling slowed greater degradation of strength induced by longer fire exposure time.	(Chen et al. 2009)
	RC (C)	ISO 834 (1.5 h)	None	The effect of post-fire duration on the residual stiffness was greater than that on the residual lateral load capacity.	(Demir et al. 2020)
	SRC (C); CSFT (S)	ISO 834 (0.5 h); ISO 834 (0.9~2.4h)	0.25 / 0.5; 0.3~0.5	The change of axial deformation in cooling stage could be 12 or 36 times of that in heating stage, thus the residual deformation of columns was mainly formed during cooling stage.	(Han et al. 2016; Zhou and Han 2018)
	RC (C)	ASTM E119 (1.3 / 3h)	0.4 / 0.6	The spalling area could reach 40% of total cross-sectional area of steel reinforced concrete columns, leading to lower residual strength.	(Kodur et al. 2013)
	RC (S)	ASTM E119 (1 / 2h)	0.25	When the RC columns were still subjected to constant load after exposure to fire, the creep deformation could be up to 30mm.	(Mostafaei et al. 2009)
	CFDSTs (SCC)	ISO 834 (1 / 2h)	None	The residual strength and energy dissipation capacity increased significantly with the increase of the nominal steel ratio for steel reinforced columns.	(Li et al. 2019b)
Beams	RC (/)	Self-defined (2.3h)	/	The residual shear capacity of frame beams decreased with the increase of shear span ratio.	(Jiang et al. 2018)
	RC (S)	Self-defined (cyclic loading)	None	When the peak temperature of thermal cycles was lower than 200°C, the residual shear capacity of RC beams increased slightly.	(Khan et al. 2010)
	RC (S)	ISO 834 (1 / 2h)	None	Water re-curing helped recover the stiffness of RC beams more significantly than the strength. The residual stiffness of beams heated for 2h under ISO 834 fire increased by 16% after 28 days of water re-curing.	(Akca and Özyurt 2020)
	RC (C)	Self-defined (1 / 2h)	0.53 / 0.63	Due to the additional capacity present in beams, residual flexural strength of post-fire beams may still satisfy the room temperature design limit states.	(Agrawal and Kodur 2019)
	RC (/)	ISO 834 (700 / 800°C)	0.2	Constant load increased the creep deflection at elevated temperatures especially when the temperature of rebar exceeded 500°C.	(Thongchom et al. 2019)
Slabs	RC (/)	ISO 834 (3h)	None	The residual strength of RC slabs was about 80% of that of slabs with fibres because explosive spalling did not occur in slabs with fibres.	(Chung et al. 2013)
	PC (/); RC (/)	ISO 834 (1 / 1.5h); ISO 834 (0.5 ~ 1.6h)	0.33~0.58; /	The uneven stiffness distribution due to cracks induced by fire may cause the internal force redistribution of prestressed concrete continuous slabs.	(Hou and Zheng 2010; Xiang et al. 2009)
	RC (/)	Self-defined (3h)	0.02	The horizontal displacements of post-fire slab were smaller than that of undamaged slabs due to lower in-plane stiffness caused by cracks.	(Wang et al. 2018)

1.specimen type (aggregate type). SRC: steel-reinforced concrete; CFST: concrete-filled steel tube; CFDSTs: concrete-filled double-skin steel tubes; PC: prestressed concrete; (S): siliceous aggregate; (C): calcareous aggregate; (SCC): self-consolidating concrete.

thereafter achieved (Kodur et al. 2004). Other techniques such as the bolted-side plating (BSP) as shown in Figure 1e is to install steel plates on both sides of the structural members using anchor bolts, which can provide additional resistance but creating damage to the existing structure (Li et al. 2019a). In studying the feasibility of various repairing techniques, different fire tests were performed to structural members and the performance of

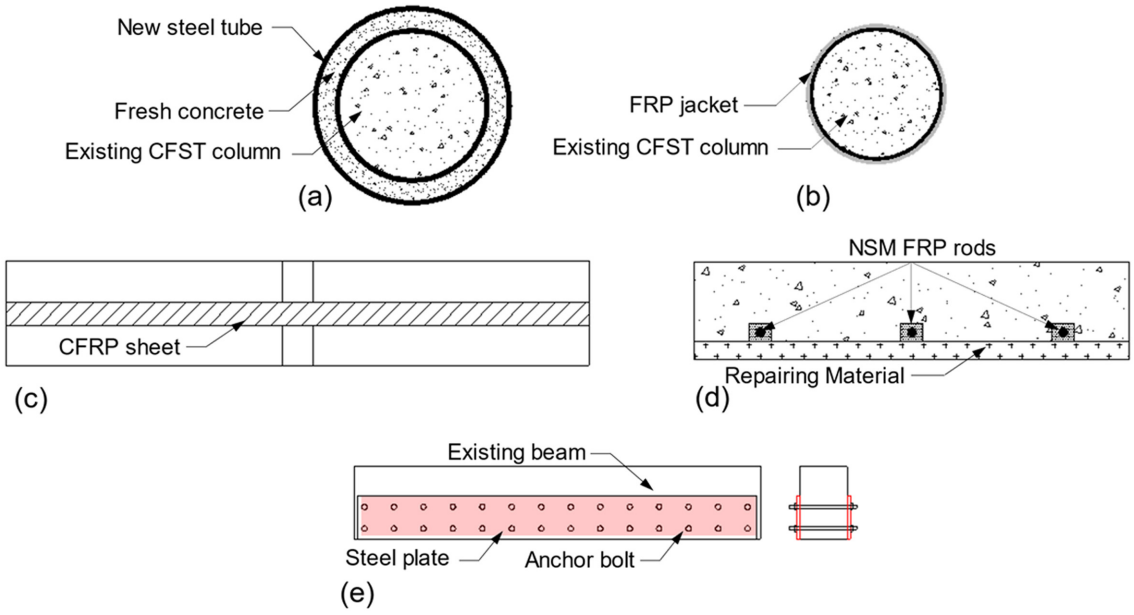


Figure 1. Diagram of repairing techniques: (a) section enlargement (Han et al. 2006); (b) FRP jacket (Tao and Han 2007); (c) EB-CFRP sheets (Xu et al. 2019b); (d) NSM FRP rods (Thi et al. 2015); (e) BSP technique (Jiang et al. 2017).

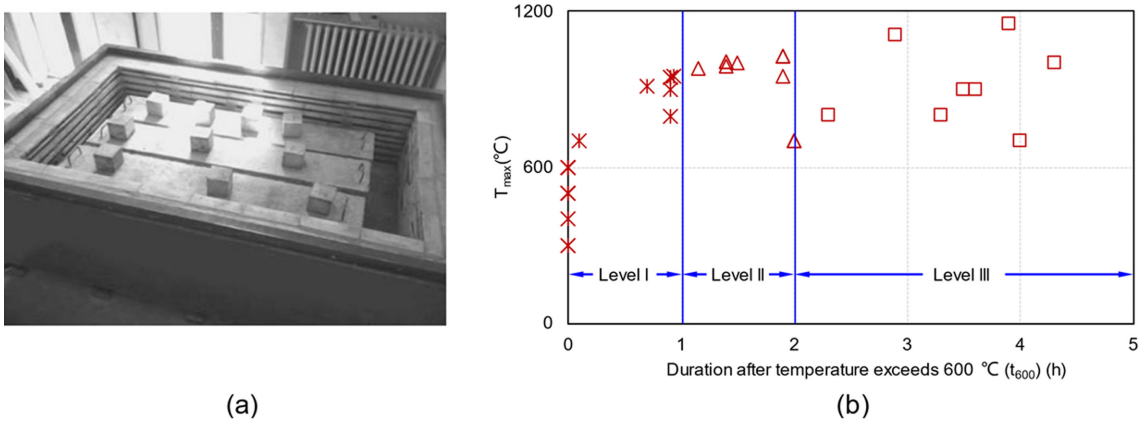


Figure 2. Fire scenarios adopted in these studies on repair: (a) typical furnace test (Haddad et al. 2008); (b) classification of fire scenarios.

these members after repair was experimentally investigated. This paper first summarizes these fire conditions and categorizes them according to the duration of gas phase temperature higher than 600°C. The repairing effectiveness in terms of the recovered capacity to the initial capacity is retrieved from the literatures and compiled as figures, which illustrates the dependency of fire intensities and demonstrates the importance of identifying realistic fire scenarios in the post-fire repair research.

3.1. Fire scenarios adopted in post-fire repair research

As listed in Table 2, various fire scenarios were adopted in studying the repair of post-fire structural members. Several studies employed constant heating rates (150°C/h or 600°C/h) or a random heating procedure until reaching a designated peak temperature, whereas most of studies adopted standard time-temperature heating curves (i.e., ISO 834 (ISO 1975) and ASTM E119 (ASTM 2007)). Natural cooling or water cooling was applied following the heating stage of fire scenarios. When the heated structural members were cooled to ambient temperatures, they were removed from the furnace and usually kept in the room environment until repairing and further testing. These scenarios were imposed to consider the post-flashover temperature distribution in a compartment of uniformly distributed gas temperature. As listed in Table 2, a wide range of fire scenarios have been employed in producing damage to the concrete structural members. This paper aims to categorize these different heating conditions in multiple levels and then the investigation of

Table 2. Heating conditions and post-fire curing adopted in current literature

Heating Condition				Post-fire curing	Reference
Heating rate	Duration	T_{max} (°C)	Cooling rate		
150°C/h	--	500	Natural	Room	(Yaqub and Bailey 2011a, 2011b, 2012; Yaqub et al. 2011, 2013)
600°C/h	Lasting for 3h at T_{max}	300, 600 or 900	Natural or Water Quenching	--	(Roy et al. 2014)
ISO-834	Lasting for 1, 2, 3 or 4h at T_{max}	300, 500, 600, 700, 800, 900 or 1000	Natural	--	(Bisby et al. 2011; Jadooe et al. 2017, 2018; Kumar et al. 2009; Yang et al. 2019)
	Heating for 1, 1.25, 1.5, 2, 3 or 4h	--	Natural	Room	(Gao et al. 2018; Han et al. 2006; Jiang et al. 2017; Li et al. 2017; Tao and Han 2007; Tao et al. 2007, 2008; Wang and Su 2014; Xiang et al. 2011; Xu et al. 2019b, 2019a)
	--	500,600, 950 or 913	--	--	(Chinthapalli et al. 2020; Irshidat and Al-Saleh 2017; Xu et al. 2005a)
	--	--	--	--	(Zhang and Xu 2009)
ASTM E119	Heating for 1.5h	1000	--	--	(Thi et al. 2015)
--	Lasting for 3h at T_{max}	500	Natural	Wet	(Al-Nimry et al. 2013)
--	Lasting for 2, 2.5 or 3h at T_{max}	400, 500 or 600	Natural	--	(Abdullah et al. 2019; Al-Nimry and Ghanem 2017; Haddad et al. 2007, 2008, 2011; Irshidat and Al-Saleh 2017)

repairing measures can be discussed accordingly.

The state-of-art of the post-fire damage has not included a criterion to classify these fire scenarios regarding the fire impact. Obviously, different levels of fire intensity could lead to different degrees of damage, which thereafter requires different repairing measures. Considering the heat transfer from fire to concrete section, the peak temperature and the duration of high temperature determine the temperature evolution at various section depths. As previously mentioned, the C-S-H gel of concrete decomposes when its temperature reaches 550°C. When the maximum temperature of fire environment is lower than 600°C, Poon et al. (2001) suggested that the original strength can be regained without repair if proper curing condition is applied. After analysing the fire scenarios listed in this paper proposes to categorize these fire scenarios into three levels (Level I, II, III), which are according to the duration of fires sustaining 600°C or higher gas-phase temperature (denoted as t_{600}). As shown in Figure 2, the fire scenarios corresponding to the above-mentioned experimental studies have been denoted as datapoints (t_{600}, T_{max}). The classification of fire scenarios is currently recommended by authors after reviewing the test data as shown in Figure 2. The data points under this criteria are evenly distributed in different categories, which are also corresponding to the commonly adopted fire rating in prescriptive fire safety design. For the Level I fires, the duration of gas phase temperature exceeding 600°C is less than 1hour, which naturally include the fire scenarios of no incident gas temperature higher than 600°C. The Level II fires are represented by the fire scenarios of t_{600} = 1~2 hour, while the Level III fires (extreme cases) hold at least 2 hours of high gas-phase temperature exceeding 600°C. In Figure 2, different markers have been used to distinguish the different levels of fire scenarios, and these markers will be used consistently in the following figures.

3.2. Repairing effectiveness on post-fire concrete structural members

The effectiveness of post-fire repair refers to a ratio of recovered load bearing capacity to the initial capacity of structural members. The repairing effectiveness is dependent on the degree of structural damage induced by various fire impact and the repairing technique itself. To reveal these effects, studies on the repairing of post-fire structural members have been summarized and the following aspects of each study are highlighted: 1) the type of structural members; 2) repair technique and 3) the level of fire intensity experienced by structural members.

3.2.1. Columns

A variety of studies have been conducted on repairing the post-fire columns. The mechanical performance of fire-damaged RC columns using a constant heat rate of 150°C/h and then repaired with FRP jackets were investigated in the University of Manchester between 2011 and 2013 (Yaqub and Bailey 2011a, 2011b, 2012;

Yaqub et al. 2011, 2013). These tests systematically investigated the influences of cross-section shape and repairing materials on the effectiveness regarding axial compression and seismic performance. After heated by ISO 834 fire (ISO 1975), the concrete-filled steel tubular (CFST) columns repaired using FRP jackets were investigated with respect to flexural bending, axial compression and cyclic loading performance by Tao and Han et al (2007; Tao et al. 2007, 2008). In these studies, the effect of the number of FRP layers on repairing the post-fire columns was investigated. In order to achieve better repairing effectiveness, Al-Nimry et al. (2013),

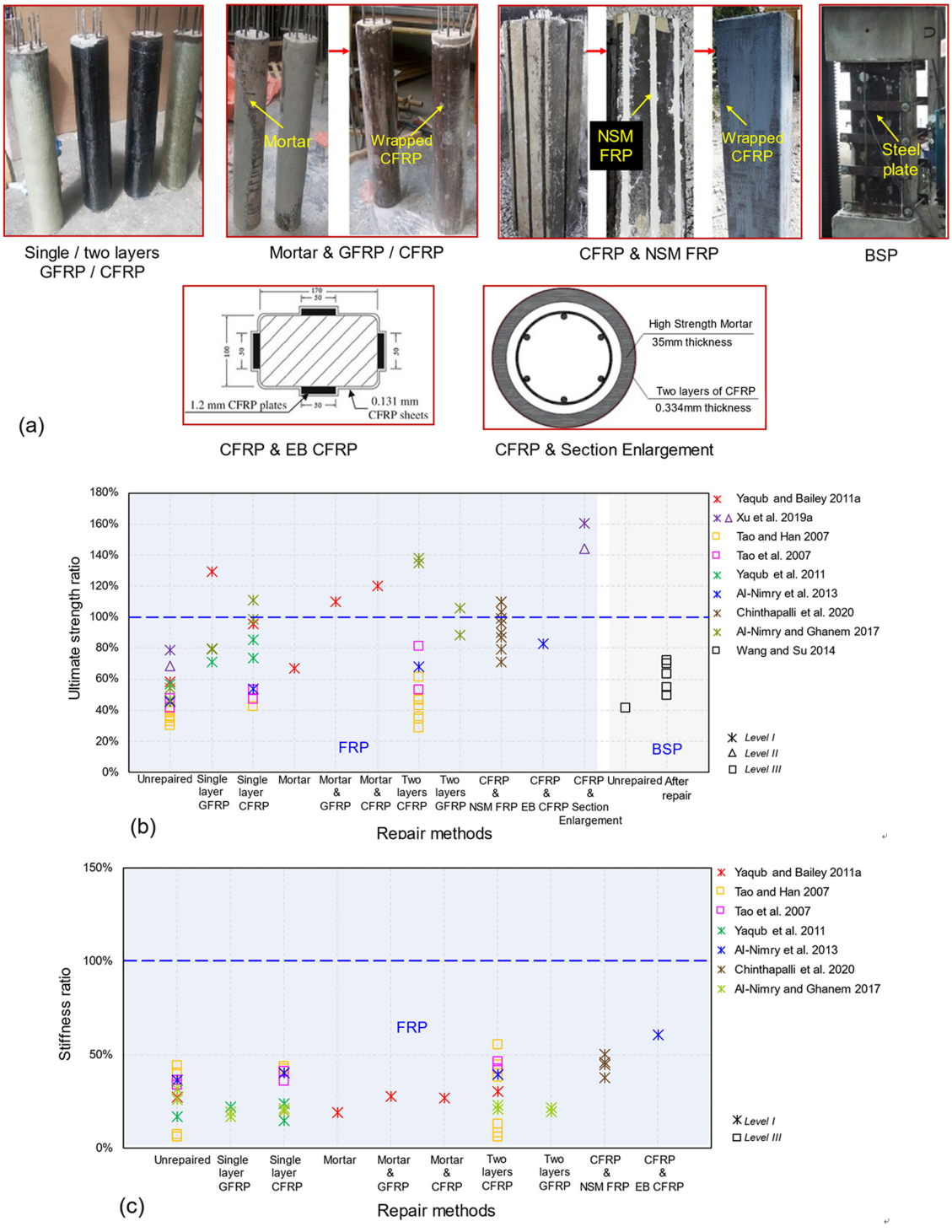


Figure 3. Repairing effectiveness of various repairing techniques on post-fire columns: (a) illustration of repair methods; (b) ultimate compressive strength; (c) stiffness.

Chinthapalli et al. (2020) and Xu et al. (2019a) proposed to repair post-fire RC columns using hybrid FRPs, which were a combined form of two or multiple FRP repairing techniques.

The repair methods and corresponding repairing effectiveness reported in the literatures have been summarized as shown in Figure 3. In Figure 3b, the repairing effectiveness of FRPs on axial compressive strength refers to the ultimate strength ratios, which are calculated by dividing the compressive strength of repaired post-fire columns with that of the unheated columns. As shown in the figure, FRP repairing was primarily implemented and Level I fires were mostly used in the fire tests. It is achievable for columns subjected to Level I and Level II fires to restore its initial capacity or even higher compared to un-heated columns, which are unlike the cases of Level III fire heated columns, as the recovered capacity remains below 60% of the initial value. This may be due to the much higher degree of damage in columns exposed to Level III fires, while the deeper layer of section suffers high temperature elevation. The ultimate strength ratios of unheated columns shown in the figure have indicated a much higher degree of strength loss in Level III fire cases compared to the columns heated by Level I&II fires.

Furthermore, the stiffness ratios of repaired post-fire columns have been similarly given in Figure 3c. The secant stiffness was differently calculated in literatures. For example, Yaqub et al. (Yaqub and Bailey 2011a; Yaqub et al. 2011) defined the secant stiffness as the slope between the origin and the maximum axial load point of the load-deformation curve of the column specimen, whereas the secant stiffness was defined between the origin and the 30%, 50% and 60% of the ultimate load points by Chinthapalli et al. (2020), Al-Nimry et al. (Al-Nimry and Ghanem 2017; Al-Nimry et al. 2013) and Tao et al. (Tao and Han 2007; Tao et al. 2007), respectively. Therefore, the stiffness ratios are more representative in comparing the repairing effectiveness regarding stiffness, which is herein defined as the reported stiffness of repaired post-fire columns to the stiffness of unheated columns in each paper. Surprisingly, the stiffnesses of post-fire columns after repair are only recovered to a half or even lower of the initial value, which is regardless of the fire intensity levels and the calculation methods. While comparing to the strength reduction induced by different levels of fires, the fire intensity is less significant on the stiffness reduction of post-fire columns as no clear dependency between them can be found.

Based on the current research, FRPs were found effective in restoring the compressive strength of post-fire columns rather than restoring the stiffness. This can be explained as: 1) greater stiffness loss of columns occurs due to the damage of concrete and the strength loss is mitigated by the steel rebars (Chen et al. 2009); 2) the FRP strengthening measures contribute more to the ultimate strength of columns but not the stiffness because of much higher strain allowed in FRP sheets (Yaqub and Bailey 2011a). Therefore, hybrid repairing techniques may be better in terms of the stiffness and strength, and the effect of repair and fire induced damage should be analysed in the context of whole-structure performance.

3.2.2. Beams

For post-fire beams, repairing materials can be applied to the bottom or the side surfaces to restore its mechanical performance, which primarily refers to the flexural capacity and the shear resistance of beams. Studies revealed that the repaired mechanical performance of post-fire RC beams did not increase linearly with the increasing number of CFRP layers, whilst the failure mode could vary from ductile failure to brittle failure if excessive layers of CFRP sheets were used (Xu et al. 2005b; Zhang and Xu 2009). In order to overcome the peeling problem of FRP sheets on the EB FRPs repaired RC beams, Irshidat and Al-Saleh (2017) proposed to improve the bonding property of adhesive by adding carbon nanotube (CNT). The observed experimental phenomenon indicated that the CNT-modified epoxy resin delayed the initiation and propagation of the FRP sheet deboning. Jadooe et al. (2017, 2018) tested the effect of adhesive on the repairing effectiveness and concluded that RC beams repaired with NSM CFRP sheets and epoxy adhesive exhibited higher bearing capacity comparing to the cement-based adhesive. Experimental study carried out by Jiang et al. (2017) revealed that the repairing effectiveness of post-fire RC beams repaired with BSP technique could be significantly enhanced by increasing the steel plate depth and decreasing the spacing of bolts.

The effectiveness of different repairing techniques on fire-damaged RC beams has been presented in Figure 4. Similarly, the ratios of bending strength and stiffnesses are calculated according to the reported data in literatures. As shown in Figure 4a, Level I and Level II fires were commonly adopted as fire loading, whereas only Jadooe et al. (2018) used the Level III fire to heat the beam. NSM CFRP and BSP measures can successfully restore the strength of RC beams to the initial performance or even higher. EB FRPs and section enlargement failed to achieve this specifically in Level I fire cases. Regarding the stiffness recovery, the available data as shown in Figure 4b indicates that the section enlargement technique adopting high-strength fibre reinforced concrete exhibits the highest repairing effectiveness. On the contrary, the stiffness recovery ratio is below 100% for post-fire beams repaired with other techniques, which include EB FRP, BSP, and normal mortar enlargement used in scaled beam specimens. Moreover, it is difficult for existing one technique to restore both bending strength and stiffness of post-fire RC beams exposed to Level I and Level II fire intensity to the level before heated by fire.

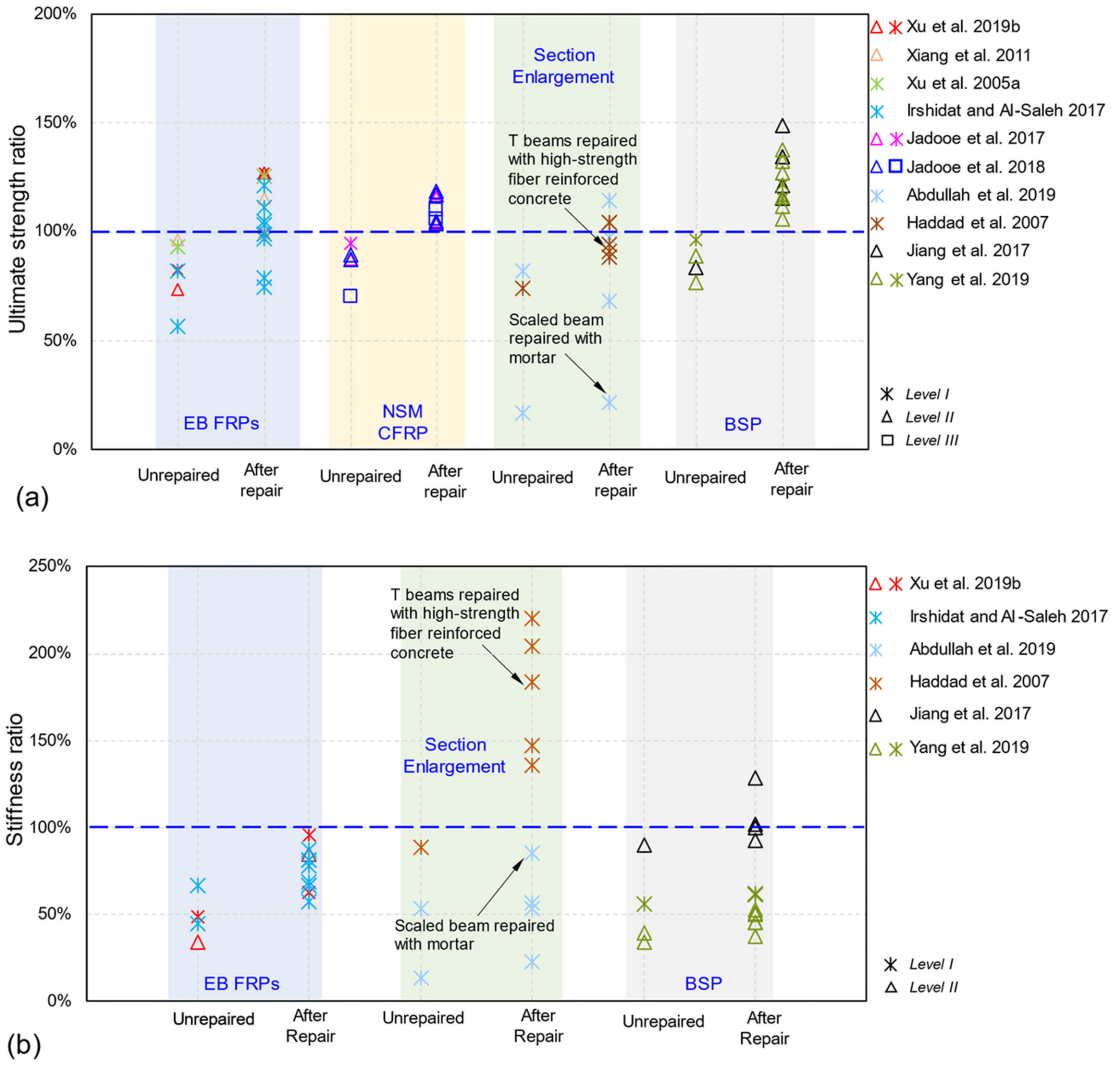


Figure 4. Repairing effectiveness of various repairing techniques on post-fire beams: (a) ultimate bending strength; (b) stiffness.

3.2.3. Slabs

The test results of repaired post-fire RC slabs in terms of ultimate strength and deflection at ultimate strength using different techniques are shown in Figure 5a and Figure 5b, respectively. Note that the deflection ratios are obtained by dividing the deflection at ultimate loads of slabs to that of unheated slabs. The smaller deflection ratio represents stronger section rigidity of the slab. To investigate the repair of post-fire slabs, tests carried out by Thi et al. (2015) suggested that the location of CFRP bars was important as the repaired RC slabs using NSM CFRP with CFRP bars embedded in repairing material exhibited higher repairing effectiveness. In order to reduce the self-weight of repaired RC slabs using section enlargement technique, Gao et al. (2018) proposed to adopt basalt fabric-reinforced shotcrete as repair material due to basalt fibres have great durability to avoid using external corrosion protection layer. Each repair technique adopted in these studies can significantly improve the load capacity of slabs damaged by Level I and Level II fires, accompanied by smaller mid-span deflection compared with unrepaired slabs. The ultimate strength of repaired RC slabs adopting NSM CFRP repair technique can almost double that of the original level and the EB FRPs can reduce the deflection of repaired slabs to half of the original level. As for the slabs repaired with section enlargement, as shown in Figure 5a, some difference in repaired strength can be seen. This may be attributed to the different repair material adopted in these studies.

4. Concluding remarks

This paper reviews the post-fire damage investigation on concrete and structural members, and particularly focuses on summarizing the state-of-the-art research on repair of post-fire concrete structural members. The post-

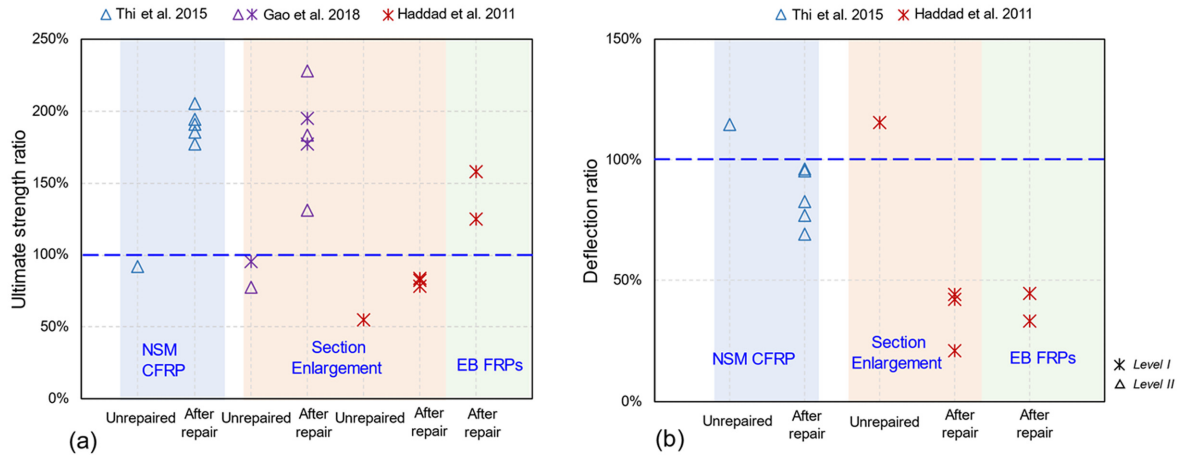


Figure 5. Repairing effectiveness of various repairing techniques on post-fire slabs: (a) ultimate strength; (b) deflection.

fire performance of concrete is affected by material types of concrete, heating rate, peak temperature of fire, cooling rate and post-fire curing conditions. Studies on post-fire performance of structural members reveal that cracks, spalling, creep deformation and reinforcing steel can also affect the residual mechanical properties. The research on repair of post-fire structural members adopted various repairing techniques to recover the structural members exposed to specific fire conditions, which have been categorized in this paper as a Level I, or II, or III fire according to the duration of fire sustaining its gas phase temperature beyond 600°C (t_{600}). The different levels of fire indicate the intensity of fire impact and directly affect the degree of damage to the structural members. The repairing effectiveness referring to a ratio of the recovered structural performance to the initial undamaged performance of structural members has been summarized following a review over the existing studies.

Currently, the fire loading scenarios used in the post-fire repair research were mostly idealized, which were implemented as furnace temperature following constant heat rates or following standard fire curves. The different fire intensity in terms of the duration t_{600} can produce various damage patterns, which challenges the repairing measures aiming to recover the structural performance to the initial state. Regardless of which level of fire intensity, the uniform temperature distribution has been commonly applied, which may not be adequate for the Level I&II fires as such a moderate fire exhibits localised heating behaviour. For Level III fires, it is questionable to apply extreme fires such as a standard fire lasting for 3 hours considering the fuel load limit. For future studies of repairing post-fire structural members, it may be necessary to include localised fire scenarios into the Level I fires, and compartment fires into the level II fires. As extreme cases, post-flashover fires considering compartment configuration may be included in the category of Level III fires. Moreover, it might be necessary to consider spread fire as an extreme fire scenario of Level III.

The existing experimental research on repairing fire damaged structural members commonly adopted single member test, while the fire induced damage is primarily due to the degradation of concrete, cracking, and spalling. However, in the context of a structural system, the load redistribution mechanism among the structural members may lead to different failure or damage patterns in the fire exposed structural members. Especially for the spread fire as above mentioned, multiple compartment fire or multiple floor fire could cause unknown damage modes, which would require different evaluation scheme in repairing the fire affected structure.

FRP sheet, steel plate, concrete or mortar section enlargement are common measures in post-fire repair. The repairing effectiveness of these techniques for post-fire columns, beams, and slabs have been summarized and compared in this paper. The effectiveness of using FRP in repairing concrete members is reasonably good in recovering the ultimate strength but the stiffness. Limited studies have been performed on repairing structural members exposed to Level III fires, and the repairing effectiveness in terms of both strength and stiffness recovery is hardly achieved. More experimental studies are expected in this area to explore suitable measures to repair structural members suffering various levels of fires. Meanwhile, numerical tools equipped with adequate models are needed in the design practice of post-fire repair. The numerical models should be ideally able to address the complex heating conditions, the post-fire cooling and curing, the post-fire material behaviour of concrete and steel reinforcement, the repairing interface models, and the structural system interaction. Therefore, a considerable amount of efforts are needed in developing this type of tools to advance the post-fire repair research.

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