



A Novel Experimental Approach for Evaluating Residual Capacity of Fire Damaged Concrete Members

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Abstract. Reinforced concrete structures exposed to fire retain significant residual capacity due to better fire resistance properties of concrete, and future re-use of the structure is generally possible. Nonetheless, fire induced degradation can result in a permanent loss of strength and serviceability. An accurate evaluation of residual capacity is necessary for taking informed decisions on future use and need for repairs in fire damaged concrete structures. This paper proposes a novel three-stage experimental approach for evaluating residual capacity of fire damaged concrete members. The approach comprises of evaluating response in three sequential stages, namely, during pre-fire exposure condition; during fire exposure comprising of heating and cooling phases of fire, followed by complete cool down of the member to ambient temperature; and then finally during post-fire exposure condition. This approach is applied to evaluate residual capacity of four concrete beams subject to different fire scenarios and load levels. Results indicate that fire damaged concrete beams retain significant residual capacity even after exposure to heating duration lasting beyond their prescriptive fire rating. Furthermore, decay (cooling) rate of fire exposure impacts extent of post-fire residual capacity retained in reinforced concrete beams. Also, increasing load level present prior to, and during fire exposure (including extended cool down) lead to a greater reduction in stiffness than residual capacity of fire damaged concrete beams. Finally, relatively large post-fire deflections occur in fire damaged concrete beams which adversely impact their serviceability limit state. The proposed approach can form the basis to conduct standardized tests for determining residual capacity of fire damaged concrete members.

Keywords: Reinforced concrete beams, Normal strength concrete, Fire damaged beams, Residual capacity

1. Introduction

Structural members are designed to meet code specified fire resistance ratings [1, 2] to ensure life safety by minimizing chances of partial or complete structural collapse during a fire event. Nonetheless, fire induced structural collapse, especially for reinforced concrete (RC) structures, is a rare event [3, 4]. Past experiences

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have shown that RC structures can be opened to re-occupancy with adequate repair and retrofitting measures after most fire incidents [5–7].

During a fire event, RC beams or slabs directly exposed to rising convective currents (hot gases) near the ceiling are particularly susceptible to fire damage [8]. While radiation is the dominant mode of heat transfer in a fully developed fire, convective effects have significant influence on heat flux emitted from the fire to member surface during the growth phase of a fire (when surface temperature of the member is relatively lower) or when localized burning impinges the structural member. Since, horizontal members (beams and slabs) near the ceiling are more susceptible to convective effects and impingement by flames of the fire as compared to vertical members, horizontal members can experience greater temperature induced strength degradation during the complete course of fire exposure. Furthermore, depending on severity of fire exposure, such RC beams (slabs) may undergo varying levels of temperature induced damage, and there is uncertainty regarding extent of load bearing capacity retained in RC members after fire exposure [9–11]. Therefore, it is imperative to assess if adequate residual capacity exists in RC members prior to re-occupancy after the fire incident. Such an assessment also enables development of needed retrofitting (repair) measures.

Current approaches rely on local material (concrete or steel) level testing [5–7] of fire damaged members and may not provide a reliable estimate of global structural damage. Limited structural level tests [12–19] conducted in the past did not monitor conditions present during extended cooling phase of the member, including post-fire residual deformations, which significantly impact post-fire residual response [9, 10]. Moreover, the effect of varying cooling phase conditions during fire exposure, and load level present in the member prior to fire exposure on residual capacity were not considered explicitly in majority of the previous studies. In addition, a load-controlled technique was used to evaluate residual response in majority of previous studies, which may not capture post-peak response (failure) and stiffness of the fire damaged concrete member as accurately as compared to deflection-controlled technique.

To overcome these knowledge gaps, this paper proposes a three-stage experimental approach to evaluate residual capacity of fire damaged concrete members. Complete response of a concrete member from pre-fire exposure conditions to complete cool down of the member, as well as post-fire response are captured in three sequential stages of pre-fire, during fire, and post-fire residual testing. Thus, the proposed approach provides an engineering framework for incorporating effect of conditions present prior to and during fire exposure on residual capacity of fire damaged concrete members. This framework can be adopted for experimental evaluation of residual capacity of fire damaged concrete members, including RC slabs and columns.

The proposed experimental approach is applied to evaluate residual capacity of four concrete beams, after subjecting them to different fire exposure scenarios with distinct cooling phases and loading (stress) level, resulting in varying degrees of fire damage. Data from these tests illustrates significance of each stage of testing, and identifies key parameters influencing residual capacity of fire damaged concrete members.

2. Approach for Evaluating Post-Fire Residual Capacity

In order to capture the realistic residual capacity of the member a three stage evaluation approach comprising of; Stage 1: evaluating the member response at room temperature during service (load) conditions as present prior to fire exposure; Stage 2: evaluating member response during heating and cooling phases as present in a fire incident, and during extended cool down phase of the member to simulate conditions as occurring after fire is extinguished or burnout conditions are attained; and Stage 3: evaluating residual response of the fire damaged member following complete cool down is proposed. A flowchart depicts these three distinct stages of residual capacity assessment through testing (or numerical modeling) in Fig. 1.

Stage 1 of testing involves subjecting the member to load (stress) and boundary conditions, as present during pre-fire conditions. Loads need to be applied in small increments to simulate quasi-static conditions, until service conditions are reached, and maintained constant using load-controlled technique, for a sustained period (typically 1 h to 2 h) until deflections in the member stabilize. During this stage of testing, the load–deflection response is to be monitored to establish initial (pre-fire exposure) stiffness of the structural member, as well as to ensure realistic level of cracking (damage) in the test specimen prior to fire exposure. Level of loading (stress) applied on the structural member during Stage 1 can have a signifi-

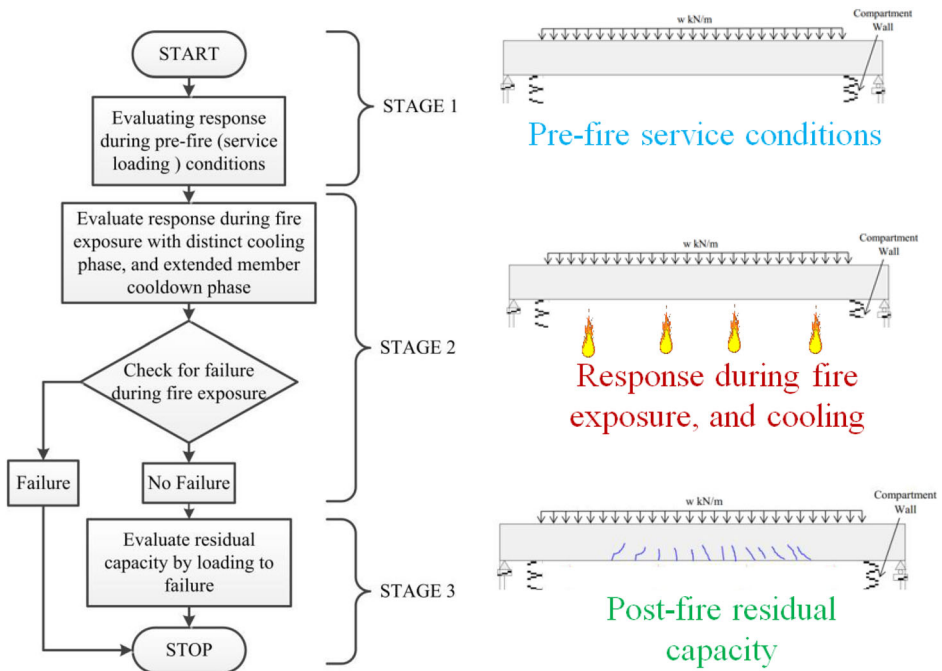


Figure 1. Flow chart illustrating proposed three stage approach for residual capacity assessment of RC structural members.

ificant influence on its performance during fire exposure in Stage 2 of testing. Also, response of the beams during Stage 1 of testing can be directly compared with structural response during Stage 3 of testing to gauge extent of fire damage.

In Stage 2 of testing, extent of fire damage in the structural member, as occurring in a real fire incident, needs to be simulated. Thus, the loaded structural member is to be exposed to a realistic fire exposure scenario having a distinct cooling phase. It is also important to note that cross-sectional temperatures within the structural member may remain significantly high for prolonged time following burnout conditions or after fire is extinguished, owing to high thermal inertia of concrete. In fact, cross-sectional temperatures within the structural member may not cool down to ambient conditions until 24 h to 72 h after fire has been extinguished, depending on size (thermal mass) of the member. Thus, loads on the member are to be maintained during complete duration of fire exposure, and until cross-sectional temperatures in the member revert back to ambient conditions. The load level maintained during this extended cooling phase of the member can also affect extent of recovery in the member. Thus, key parameters to be monitored during Stage 2 of testing include cross-sectional temperatures, deflections, temperature induced spalling, any restraint forces, and load level maintained during extended cool down of the member. Once the member cools down to ambient conditions, the member is unloaded to record recovery in post-fire deformations, and measure extent of irrecoverable (plastic) residual deformations.

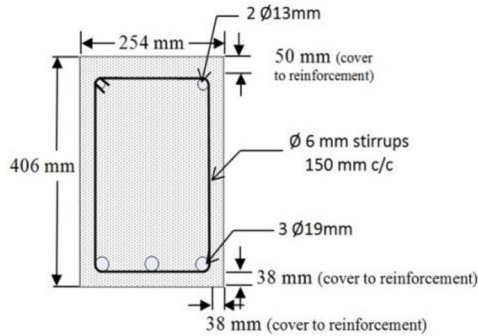
Following complete cool down of the structural member to room temperature, and if there is no failure in Stage 2 of testing, residual response of the fire damaged member is traced in Stage 3 of testing. The post-fire residual response of the member is to be traced by incrementally loading the member to failure through a displacement-controlled loading technique. This ensures that the post-peak response, including softening state, of the tested member till actual failure to be captured accurately. During Stage 3 of testing, the load–deflection response, and modes of failure in the structural member are to be recorded.

3. Residual Capacity Tests

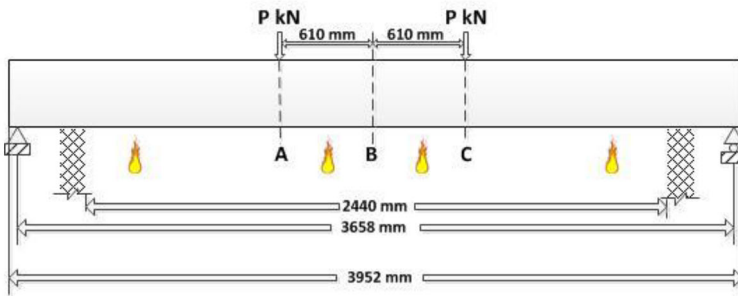
The proposed three-stage experimental approach was applied to evaluate residual capacity of four identical reinforced concrete (RC) beams subject to varying fire exposure scenarios and load level. The experiments were conducted to highlight each stage (pre-fire, fire, and post-fire) of testing of the proposed approach and show the influence of varying service and fire exposure conditions on residual capacity. It can be seen through results that concrete beams can retain significant residual capacity or fail (no residual capacity) when these conditions are varied. The design and fabrication of specimens, as well as test setup and procedure, are discussed in this section below.

3.1. Design and Fabrication of Concrete Beams

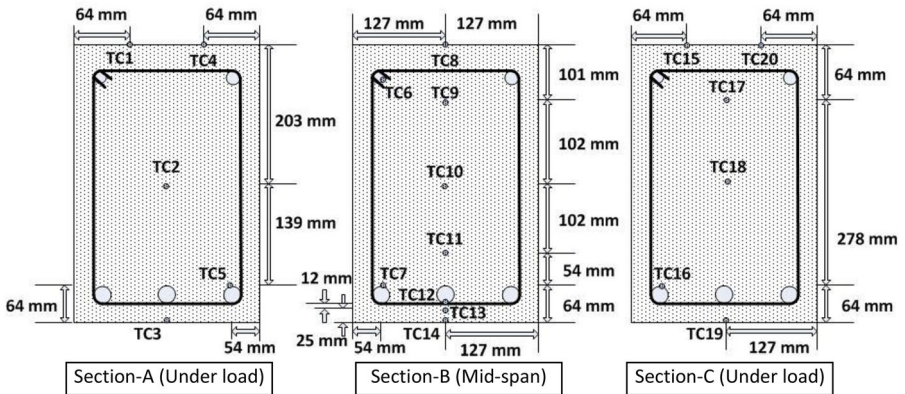
Four rectangular reinforced concrete beams (see Fig. 2), designated as B1–B4, were designed as per ACI 318 specifications [20], and then fabricated for testing.



(a) Cross sectional details of tested beams B1 through B4



(b) Elevation of beams B1 through B4 during fire tests, and subsequent residual capacity tests after cooldown



(c) Thermocouple locations at different cross sections of tested beams B1 through B4

Figure 2. Geometric and instrumentation details of tested RC beams.

All four beams had three $\phi 19$ mm bars as tensile reinforcement and two $\phi 13$ mm bars as compressive reinforcement. The shear reinforcement in these beams comprised of $\phi 6$ mm stirrups spaced at 150 mm along the length of the beam. The steel of the main reinforcing bars and stirrups had specified yield strength of

420 MPa and 280 MPa, respectively. Figure 2 shows the elevation and cross-sectional details of fabricated beams, together with the locations of the stirrups.

The concrete used to fabricate these beams was made with general-purpose Type I Portland cement and carbonate aggregate (see Table 1). The fabricated beams were sealed in their forms for 7 days for curing. All the specimens were then removed from the forms and stored in air maintained at about 25°C and 40% relative humidity for at least 1 year prior to testing. The long curing periods ensured that these beams attained realistic moisture condition as observed in actual buildings. The average compressive (cylinder) strength of concrete measured at 28 days and on test day was 49 MPa and 62 MPa respectively. Noticeable compressive strength gain occurred beyond 28 days due to continued hydration of cementitious products [21].

Also, after fire exposure and subsequent residual capacity tests on beams, cores were drilled from the unaffected (unexposed) support zones in each of the beams and tested for compressive strength. The compressive strength of drilled concrete cores from undamaged support zones of beams B1, B2, B3, and B4 prior to residual capacity testing were measured to be 61 MPa, 66 MPa, 62 MPa, and 63 MPa.

The moisture condition (relative humidity) was measured on the day of the fire test as per ASTM E119 [2] recommendations. Also, the moisture content in the beams as a percentage of their weight was computed based on the measured relative humidity [22] and porosity of the concrete estimated based on concrete mix proportions [23]. The moisture content was similar in all four beams with minor differences due to variation in ambient humidity at the time of testing [23].

The tested beams were instrumented with thermocouples and displacement transducers. Type-K chromel–alumel thermocouples (0.91 mm thick) were installed during fabrication at three different cross-sections in each beam for measuring concrete and rebar temperatures [2, 16, 24]. Five thermocouples were also installed at the top side (unexposed side) of the beam. The location and numbering of the

Table 1
Concrete Batch Mix Proportions Used in Fabrication of RC Beams

Mix design	NSC
Total cement (kg/m ³)	389.9
Water (kg/m ³)	156.4
Coarse aggregate	1036.9
Fine aggregate (kg/m ³)	830.1
Water reducing agent (kg/m ³)	1.9
Slump (mm)	100
Water/cement ratio	0.4
Air content %	1.7
Unit weight (kg/m ³)	2415
Design compressive strength: MPa	42
28-day compressive strength: MPa	49.2 ± 0.3
Test day compressive strength: MPa	62

thermocouples (labelled TC1 to TC20) in three cross-sections along the span of the beam are shown in Fig. 2. The deflection of each beam was measured by placing displacement transducers at mid-span, as well as at the location of one of the two point loads.

3.2. Test-Setup and Procedure

Each stage of testing was carried out in a specially designed test setup. The application of loads at room temperature (Stage 1), and subsequent fire exposure tests (Stage 2) on the four beams were carried out in the fire test furnace. Subsequently, upon complete cool down of the member residual capacity tests were conducted in a displacement controlled flexural loading test setup.

3.2.1. Testing Prior to Fire Exposure In Stage 1 of testing, each of the beams were setup in the loading frame-cum-heating furnace, and two point loads were applied incrementally through a load controlled procedure on the top face at 1.4 m from support ends. For all beams, except beam B4, each of the two point loads was 50 kN generating a bending moment of approximately 53% of the beam capacity according ACI 318 [20], similar to load (stress) level typically present during pre-fire exposure conditions. To study the effect of load level on residual capacity, beam B4 was subjected to a relatively higher load level of about 65% of design capacity, with each of the point loads being 60 kN. All four beams were tested under simply supported end conditions. The load was maintained on the beams through a load-controlled technique, till deflections stabilized, and then during Stage 2 of testing. During Stage 1, the load–deflection response and crack propagation in the beam were monitored.

3.2.2. Testing During Fire Exposure Conditions In Stage 2 of testing, the loaded beams were subjected to fire exposure in a test furnace [16]. The bottom and two side surfaces of each beam were exposed to fire, while a 50 mm layer of insulation (ceramic fiberfrax material) was utilized provided on the top surface of the beam. Such three-sided exposure is similar to the conditions encountered in practice wherein a concrete slab is present on the top side of the beam prevents heat penetration from the top. Also, only middle 2.44 m length of the beam, of the total 3.96 m span was exposed to fire. Such exposure of the middle span of the beam is typical of conditions present in a building compartment where support regions of the beam are not exposed to fire directly. However, the ratio of the exposed to unexposed length is primarily due to limitations of fire test furnace, where the maximum fire exposed span cannot be more than 2.44 m.

The fire exposure on the beams comprised of a heating phase followed by a distinct cooling phase, as shown through time–temperature profile in Fig. 3. The heating phase in all beams was as per ASTM E119 [2] standard fire exposure; with 90 min heating for beams B1 and B2 (indicated as Short Fires SF1 and SF2 in Fig. 3) and 120 min of heating for beams B3 and B4 (indicated as Long Fires LF1 and LF2 in Fig. 3) respectively. The duration of heating phase was equal to or greater than the prescriptive fire rating of the beams determined to be 90 min as

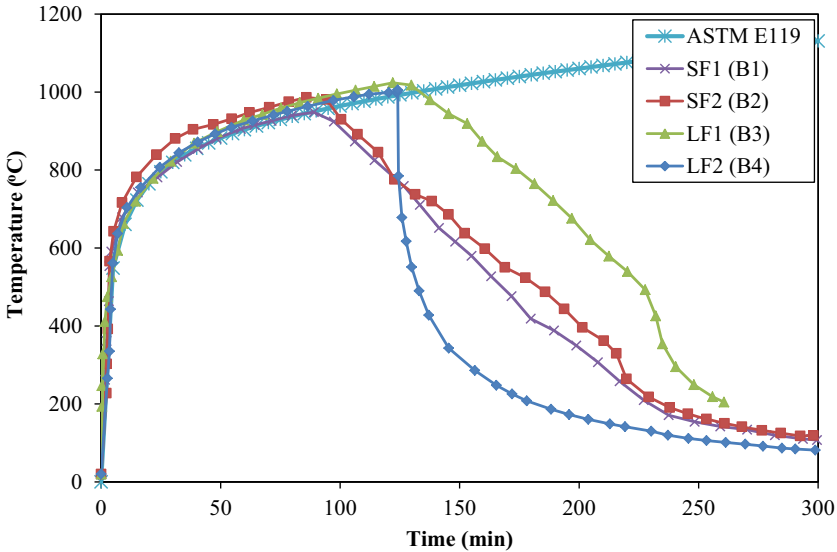


Figure 3. Time-temperature curves depicting heating phase and cooling phase for fire scenarios adopted for simulating fire damage in tested RC beams.

per ACI 216 [25]. To study the effect of distinct cooling phase, a linear decay rate of approximately -6°C per minute during cooling phase of fire exposure was adopted for beams B1, B2, and B3, representing conditions when there is no fire-fighting intervention and the fire is allowed to decay on its own. Beam B4 however was allowed to cool at a rapid non-linear rate of approximately -80°C per minute, to simulate conditions when there is fire-fighting intervention and fire temperatures decay much more rapidly. These decay rates qualitatively highlight the influence of cooling phase on both compartment temperatures and residual response of RC beams. The chosen decay rates were governed by upper-bound and lower-bound limits achievable using the available test furnace, than a detailed study on compartment temperatures during actual fires with and without fire-fighter intervention. It is expected that cooling rates may differ in cases of water cooling, and other variations in cooling conditions. Further studies are needed for quantifying the effect of decay rates on residual response of fire damaged concrete members. It should be noted that the fire exposure scenarios adopted for beams B1 and B2 were identical as they were tested under different load level. Also, its noteworthy to mention here that these fire test conditions do not account for factors such as axial restraint (and line of thrust), continuity, moment redistribution, varying dimensions of the member etc. which are known to significantly influence the fire resistance of the member. The influence of these factors on residual capacity of fire damaged concrete members will be addressed as part of future studies.

During fire tests, observations were made at every 5 min through the view ports in the furnace to record any noticeable fire induced spalling. It should be noted

that these visual observations were qualitative in nature, and focused on recording time and location at which spalling occurred (if any). The beam would be considered to have failed in case the hydraulic jack, with a stroke of 250 mm, could no longer maintain the load or the peak deflection in the beam exceeded $L/20$ as per ASTM E119 standard failure criterion [2].

Following end of fire exposure, representing extinguishing of fire in a building, the recovery of cross-sectional temperatures and deflections was monitored 24 h after fire exposure to gauge extent of recovery in the strength and stiffness properties of the beams during extended cooling phase. After complete cool down, post-fire inspection was carried out to record fire induced spalling, rupture of reinforcement, and cracking of concrete in the tested beams. Also, a quantitative measurement of temperature induced spalling was carried out once beams were removed from the furnace after fire tests. Spalling measurements were carried out 1 week after fire exposure in order to record the upper bound magnitude of the overall volume of spalling.

3.3. Testing During Post-Fire Residual Conditions

The fire damaged beams were stored for 7 days following fire exposure after temperatures revert back to ambient conditions, to well as allow for any short term reduction in compressive strength of concrete during post-cooling storage [26]. Subsequently, each beam was tested for residual capacity in a four-point loading set-up under flexural loading (see Fig. 2). Two point loads at 1.4 m from the supports were applied on the top face of the beam through a displacement controlled actuator (with a capacity of 1500 kN). A displacement-controlled technique at 2 mm per min was adopted for loading the beam during residual strength test, and the load–deflection response of the beams was recorded to evaluate residual capacity.

4. Test Results

Data generated through tests is utilized to illustrate the significance of each stage of testing in evaluating residual capacity of fire damaged concrete beams. Beams B1, B2, and B4 did not fail during fire exposure and were tested for residual capacity, while beam B3 failed during fire exposure, and hence was not tested for residual capacity. The varying test parameters together with summary of test results are provided in Tables 2 and 3.

4.1. Response Prior To Fire Exposure (Stage 1)

The measured mid-span deflection in Stage 1 is plotted as a function of applied loading in Fig. 4 for all four tested beams. As expected, deflection in all beams increases monotonically with increasing load. Furthermore, the response in all beams is almost linear until the onset of tensile cracking. This tensile cracking is primarily confined to the critical region between the two point loads, subject to a constant bending moment (flexural stresses). Pre-fire secant stiffness, calculated as

Table 2
Test Parameters Varied in Residual Capacity Tests

Beam designation	Fire exposure	Support conditions	Applied load: kN (% of capacity)	Relative humidity %	Moisture content (% by weight)	Measured fire resistance (min)	Spalled volume ratio (%)
B1	SF1	Simply	50 (53%)	91.8	2.6	No failure	0.1
B2	SF2	supported	60 (65%)	87.5	2.5	No failure	0.1
B3	LF1		50 (53%)	86.6	2.5	230	0.5
B4	LF2		50 (53%)	89.8	2.6	No failure	0.1

Table 3
Summary of Measured Response Parameters for Tested RC Beams

Beam designation	Peak rebar temperature: °C	Peak mid-span deflection: mm	Stabilized cool-down deformation: mm	Residual deformation: mm	Residual capacity: kN (% ACI capacity)	Secant stiffness: kN/mm	
						Pre-fire	Post-fire
B1 (SF1)	521	37	32	20	91 (98%)	8.8	3.8
B2 (SF2)	510	63	56	34	99 (106%)	7.22	2.9
B3 (LF1)	580	122	–	–	–	9.8	–
B4 (LF2)	500	38	28	16	93 (100%)	9.1	4.5

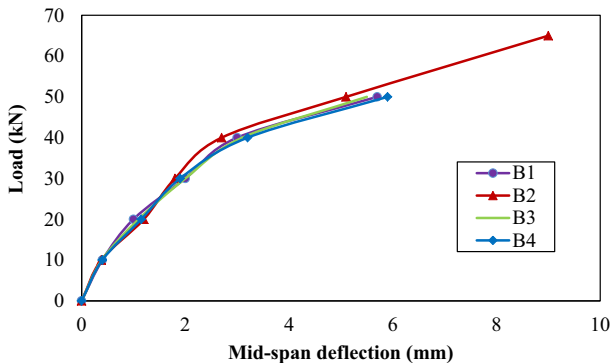


Figure 4. Response of RC beams during Stage-1 loading at room temperature. Mid-span deflection is assumed to be increasingly positive with increasing load.

the ratio of peak load over final deflection for each beam B1, B2, B3, and B4 was calculated to be 8.8 kN/mm, 9.8 kN/mm, 9.1 kN/mm, and 7.2 kN/mm respectively (see Table 3). Noticeable reduction in secant stiffness of beam B2 can be

attributed to greater applied load resulting in tensile cracking (softening) response. The calculated secant stiffness during Stage 1 of testing can be compared with post-fire stiffness to gauge extent of fire damage.

The applied load was maintained for at least 45 min after reaching peak load for each of the four tested beams to allow for the deflections to stabilize, and subjecting the beams to fire testing in Stage 2.

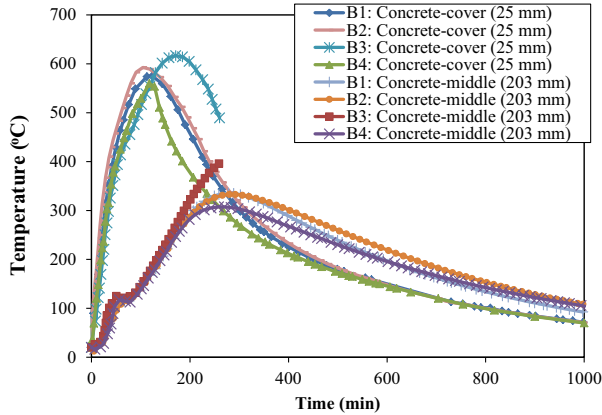
4.2. Response During Fire Exposure Conditions (Stage 2)

During Stage 2 of testing (fire exposure), thermal and structural response of the beams was evaluated until the end of fire exposure, and the extended cooling period i.e. period covering between fire being extinguished to the time taken for entire cross-section of the structural member to revert back to room temperature. Response parameters monitored during this stage included cross-sectional temperatures, deflections, spalling, and post-fire residual deformation.

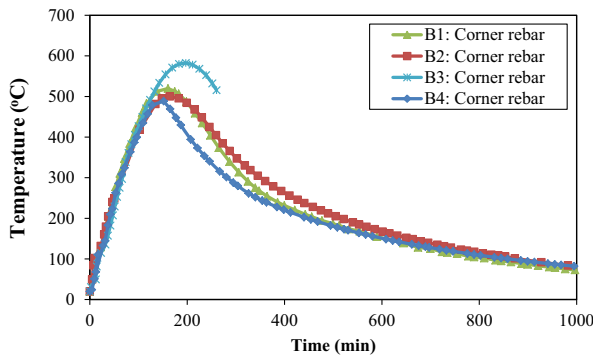
4.2.1. Thermal Response Thermal response of the tested beams (B1 through B4) during fire exposure are presented in Fig. 5a, b by plotting rebar and concrete temperatures at different locations along the cross section, as a function of fire exposure time. Unlike fire temperatures that rose rapidly in the first few minutes, sectional temperatures within the beam remained fairly low during initial phase of fire exposure. Temperatures within the cross section of all beams started to rise at about 10 min to 15 min into fire exposure, when fire temperatures were already in excess of 700°C. Furthermore, a temperature plateau is seen at about 110°C in all four beams, at various measured locations within the beam cross-section (see Fig. 5a). This temperature plateau results from latent heat absorbed by moisture (free capillary water) present in the beam, as it undergoes phase change from liquid to vapor. Once majority of this moisture evaporates cross sectional temperatures increase monotonically with fire (gas) temperatures. Furthermore, it can be seen that measured temperatures in concrete increase less rapidly towards inner zones of the concrete core, owing to low thermal conductivity and high thermal capacity of concrete which slows down heat penetration to inner concrete layers.

The rate of temperature rise in corner rebar is similar for all four beams, as shown in Fig. 5b. Peak rebar temperatures at the end of heating phase of 90 min for beams B1 and B2 were similar and measured to be 390°C and 385°C respectively. For beams B3 and B4 however, peak rebar temperatures at the end of heating phase lasting for 120 min were relatively higher, and measured to be 480°C, and 465°C respectively. Thus rebar (and concrete) temperatures recorded in beams B3 and B4 were higher by almost 100°C, when compared with the measured temperatures in beam B1 and B2, owing to longer burning duration (heating phase) of the applied fire exposure in the former case.

Cross-sectional temperatures in beams were monitored for up to 24 h after completion of fire exposure as the beams cooled down to room temperature, except for beam B3 which experienced failure at 230 min during fire exposure. It can be seen that cross-sectional temperatures in all four beams continue to increase even as fire (gas) temperature begins to decay (see Fig. 5a, b), owing to



(a) Temperatures in concrete beams B1 through B4



(b) Temperatures in rebar for beams B1 through B4

Figure 5. Measured rebar and concrete temperatures as a function of fire exposure time in tested RC beams (heating and cooling phases), as well as extended cool down (Stage 2).

high thermal inertia of concrete. Cross-sectional temperatures attain peak values during the cooling (decay) phase of fire in all four tested beams. Also, effect of thermal inertia is more significant on temperature lag within inner layers of concrete (mid-depth of concrete cross-section) as compared to the outer layers. Consequently, temperatures measured in the outer layer (at 25 mm concrete depth) for all four beams begin to decrease soon after fire exposure temperature enters decay phase (see Fig. 5a). Measured temperatures at concrete mid-depth however, start to decay at a significantly later duration into fire exposure. In particular, cross-sectional temperatures at concrete mid depth of beam B3 which failed during fire exposure, show an increasing trend even after fire (gas) temperature reduces to almost half its peak value.

Peak temperatures at concrete mid-depth reach 335°C and 334°C at about 297 min and 300 min into the fire exposure in beams B1 and B2 respectively, exposed to relatively shorter heating phase. For beams B3 and B4, exposed to rel-

atively longer fire exposure, temperatures at concrete mid-depth attain peak values of 384°C and 310°C at about 230 min and 260 min, well after peak fire temperatures have subsided (see Fig. 5a).

Measured temperatures at corner rebar (shown in Fig. 5b) show a similar trend as seen in concrete temperatures during decay phase of fire exposure. Rebar temperatures continue to increase well into the decay phase of fire exposure. For beams B1 and B2, exposed to fire scenario SF1 and SF2 having a similar heating phase of 90 min, tensile rebar temperatures attain a similar peak value of 521°C at 155 min and 510°C at 150 min into fire exposure respectively. In case of beams B3 and B4, which were heated for 120 min under LF1 and LF2 fire scenarios, peak tensile rebar temperatures of 583°C and 500°C are experienced at 190 min, and 146 min since beginning of fire exposure in each case. Thus, the effect of thermal inertia is more pronounced in case of slower cooling rates (representing conditions when the fire is allowed to decay on its own) adopted in fire exposure scenarios SF1, SF2, and LF2, as opposed to the rapid cooling rate adopted in case of LF1, representing conditions when there is fire-fighting intervention and fire temperatures decay much more rapidly.

In addition, time taken by each beam to revert back to ambient temperature is significantly greater than duration of heating. As a comparison, time for temperatures at the mid depth of concrete to drop from peak temperature of 344°C to room temperature in beam B2 was more than 1400 min (or 24 h), when the duration of the heating phase was only 90 min (1.5 h). This can be attributed to relatively high thermal inertia of concrete, as well as micro-cracking in concrete during fire exposure, which results in increased porosity and hence reduced thermal conductivity, and leads to prolonged retention of heat within the beam.

4.2.2. Structural Response Structural response of beams, B1 through B4, during fire exposure can be gauged by tracing mid-span deflection as a function of fire exposure time (see Fig. 6). Similar trend in deflection progression in all beams can be seen during early stages of fire exposure, due to almost identical rise in cross-sectional temperatures in the first 20 min. During this early stage of fire exposure, deflection rise is mainly governed by level of applied loading and thermal gradients that develop within the beam cross-section. As fire exposure progresses further, cross-sectional temperature within the beam begin to rise and thermal gradients decrease along the depth. Mid-span deflection in beams continues to increase, but at a relatively slower rate, owing to gradual reduction in mechanical properties, especially elastic modulus of reinforcing steel.

In case of beams B1 and B2 subject to identical fire exposure scenarios, a higher mid span deflection of about 25 mm occurs in beam B2 as compared to 19 mm in case of beam B1 owing to higher load level in the former case after 90 min of heating. For beam B3 and B4 heated for 120 min as per ASTM E119 standard fire [2], almost identical mid span deflection of 25 mm and 27 mm was measured at the end of heating phase.

Beam B3 failed at 230 min into fire exposure due to excessive deflections (see Fig. 6), during cooling (decay) phase when fire (furnace) temperature dropped to 500°C (almost half its peak value). Rebar temperatures also decay by almost 60°C

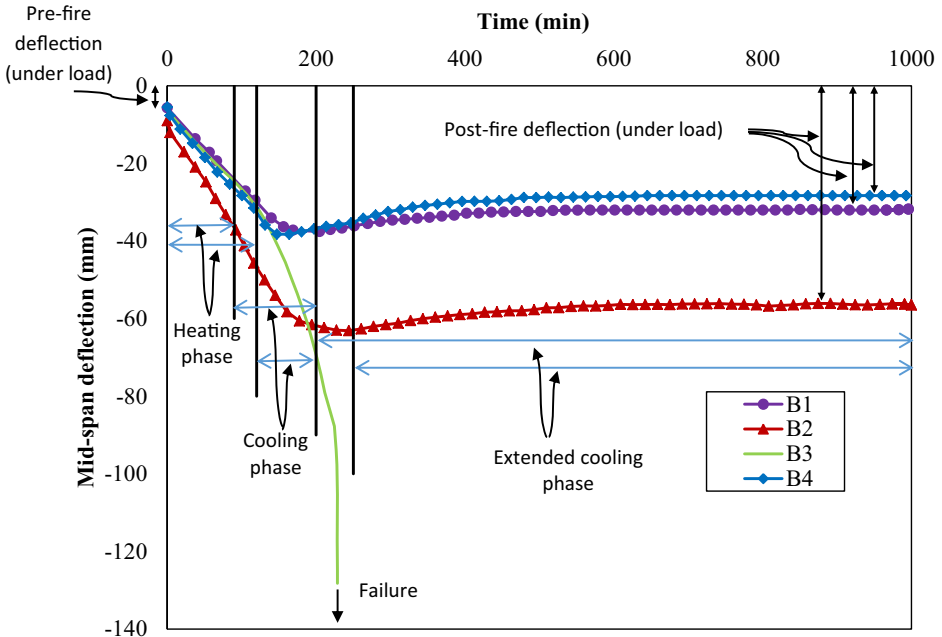


Figure 6. Measured mid-span deflections in tested RC beams as a function of time during fire exposure (heating and cooling phases), as well as extended cool down (Stage 2). Mid-span deflections are assumed to be increasingly negative with increasing time (temperature).

from their peak value but remain in excess of 500°C when failure occurred. In fact, rebar temperatures remained in excess of 500°C for almost 130 min during fire exposure due to slower rate of cooling adopted in fire scenario LF1. Upon examination of the beam prior to failure, two of the three tensile rebars in the beam were found to have ruptured as well. These observations indicate that failure in the beam occurred due to yielding of steel reinforcement, and increased mechanical and creep strains owing to a drastic decrease in yield strength occurs in reinforcing steel at temperatures in excess of 500°C . Thus, no recovery was seen in the beam during cooling phase of fire exposure and it was not tested for residual capacity.

Beams B1, B2 and B4 did not fail during fire exposure, and their mid-span deflection reverted during cooling phase is also plotted in Fig. 6. Relating deflection progression with cross-sectional temperatures in Fig. 5a–c, it can be inferred that level of mid-span deflection is governed by peak temperatures experienced in the tensile rebars and load level. Mid-span deflection in beams B1 and B2, heated for 90 min, continue to increase until about 180 min into fire exposure, as long as rebar temperatures remain above 350°C . Similarly, mid-span deflections for beam B4 heated for 120 min continue to increase for almost 200 min into fire exposure. Deflection during cooling stage of fire in these three beams, when temperatures do

not rise significantly (but are maintained above a threshold of 350°C), can be attributed to an almost 25% reduction in elastic modulus [1], as well as effects of high temperature creep typically observed in later stages of heating phase during fire exposure in concrete beams [16]. In addition, the influence of load level on peak mid-span deflections can be seen in deflection response of beams B1 and B2. An increased load level of about 20% present in beam B2, as compared to the load on beam B1, resulted in peak mid-span deflection to increase by almost 70% and this is mainly due to early yielding of steel reinforcement and increased mechanical and creep strains due to higher stress level present in beam B2. Furthermore, the rate of cooling adopted during fire exposure also affects level of recovery seen in the fire exposed RC beams, as seen in beams B1 and B4. Although beam B1 experiences heating for 90 min, as compared to beam B4 heated for 120 min, recovery in mid-span deflections in beam B1 is relatively lower due to slower cooling, representing conditions when the fire is allowed to decay on its own, adopted during fire exposure in the latter case.

Mid-span deflections further recover in beams B1, B2, and B4, once rebar and concrete temperatures revert back to ambient temperatures. This can be primarily attributed to the recovery in strength and modulus properties in reinforcing steel once the beam enters cooling phase. Recovery in mid-span deflection during the cooling phase of fire exposure is governed by sectional temperatures, especially rebar temperatures. All three beams- B1, B2 and B4, attain a steady state mid-span deflection as rebar temperatures cool down below 150°C. In addition, noticeable residual deformation is left over in each of the fire exposed beams and the beams do not revert back to their pre-fire configuration after applied loading is removed. This is primarily due to irreversible temperature induced damage in concrete, which does not recover all of its strength and stiffness properties upon cool down to ambient conditions, as well as residual plastic strains in steel reinforcement and concrete even after cool down to room temperature. These residual deformations, with no load acting on the beams, were measured to be 20 mm, 34 mm, and 16 mm in beams B1, B2, and B4 respectively. The effect of cooling on the extent of residual deformations can be gauged by comparing response of beams B1, B2, and B4 which were cooled at different rates representing no intervention (beams B1 and B2), and with fire-fighter intervention (beam B4). Lower residual deformations result in beam B4 as compared to beam B1 despite similar peak rebar temperatures, due to the faster cooling adopted in the latter case. Besides rate of cooling, load level present during cooling phase also affects the magnitude of post-fire residual deformations. Higher load level of about 20% present on beam B2 which had an almost identical temperature history as beam B1, led to almost doubling the post-fire residual deflections in beam B2 as compared to that in beam B1. Furthermore, the post-fire stiffness calculated as the ratio of applied load to post-fire elastic deflection (under load), for each fire damaged beam B1, B2, and B4 was calculated to be 3.8 kN/mm, 2.9 kN/mm, and 4.5 kN/mm respectively (see Table 3). Such reduction in stiffness, and residual deformations adversely affect future serviceability of the fire damaged concrete structure, as the fire damaged beam undergoes significantly larger deformations than pre-fire (room temperature) deflection in the member. Although the present study pro-

vides novel data for cooling phase failure of a flexural member, further studies are needed for developing design recommendations with respect to cooling phase. This aspect will be studied as part of future studies.

Upon post-fire inspection of beam B3, spalling of the concrete in the tensile region was seen with rebar being exposed. The spalling in beam B3 occurred during cooling phase of fire exposure (not heating phase of fire exposure), primarily due to a combination of flexural cracks and longitudinal cracking along rebar [27] that developed during extended cool down of the member. In addition, since this type of spalling occurs during later stages of fire exposure, concrete on the bottom tensile (exposed) face is already significantly weakened and spalls as soon as tensile stresses exceed the reduced tensile capacity of fire damaged concrete when tensile rebar ruptured suddenly. No early stage explosive spalling was seen in any of the four beams B1, B2, B3, and B4 during fire exposure due to relatively higher permeability of NSC which prevents build-up of excessive pore pressure within the concrete layers [28–30]. Nonetheless, upon examination of beams B1, B2, and B4 that did not fail during fire exposure 24 h after fire exposure as they cooled down to ambient conditions, it was seen that these three beams underwent some level of spalling during cooling phase. The three fire damaged beams underwent minor spalling confined to fire exposed (bottom) convex corners of the beam. Such spalling in beams B1 (similar to beam B2) and B4 is shown in Fig. 8a, b. This type of late stage spalling, also known as corner spalling or sloughing-off, can be attributed to thermal cracking, combined with thermo-mechanical stresses in the surface giving rise to a crack pattern, where cover concrete in corners fall-off due to self-weight [30]. Majority of this corner spalling in the beams was seen to occur while they were stored under ambient conditions (after fire exposure) for a period of 1 week before testing for residual capacity.

Overall, beams B1, B2, and B4 did not fail during fire exposure as per applicable failure limit states according to ASTM E119 [2], and were tested for residual capacity after complete cool down to ambient conditions.

4.3. Residual Response of Fire Damaged Beams

Beam B3 failed during fire exposure due to excessive deflections and hence there was no need to test this beam for residual capacity. Residual capacity of other three beams B1, B2, and B4, which did not fail during fire exposure, was evaluated by incrementally loading them to failure. The load–deflection response, progression of cracking, and failure mode were monitored during these residual capacity tests.

Mid-span deflection for each tested beam was measured through LVDTs installed on the beams. The measured load–deflection response for beams B1, B2, and B4 is plotted in Fig. 7. All three fire damaged beams exhibit three key phases in deflection progression i.e., linear response (marked as A–B in Fig. 7), onset of yielding in steel reinforcement (marked as B in Fig. 7), and plastic deformation with strain softening until failure (marked as B–C in Fig. 7).

In the first phase (A–B in Fig. 7), load–deflection response of fire damaged beams follow a linear trend as seen in a cracked section, until the onset of yielding

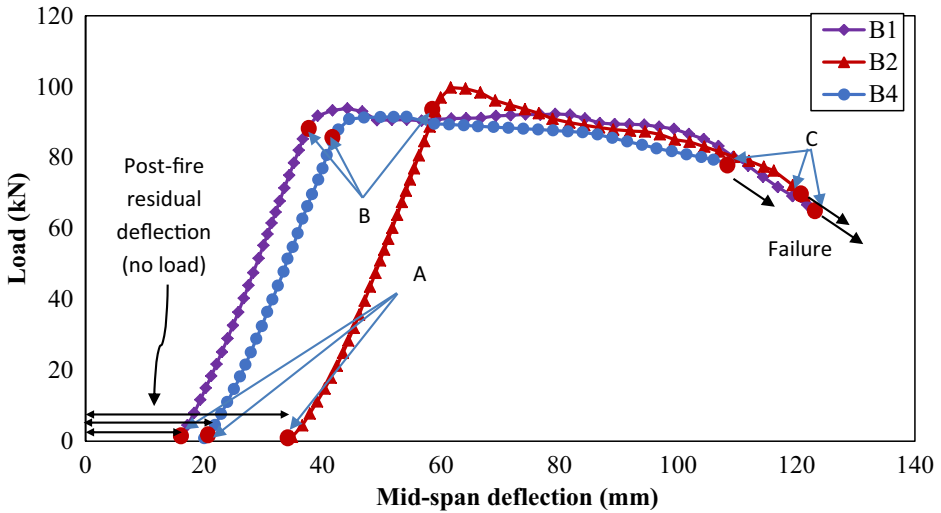


Figure 7. Load-deflection response of fire damaged RC beams during residual capacity (Stage 3) testing. Mid-span deflection is assumed to be increasingly positive with increasing load.

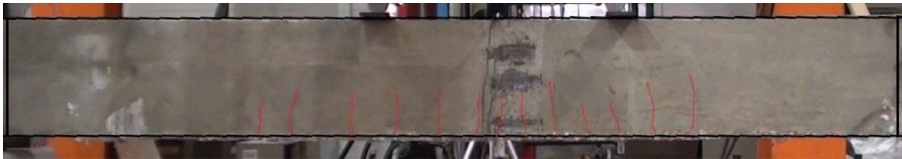
in steel reinforcement. This can be attributed to extensive tensile cracking and temperature induced material degradation that occur in the beams during fire exposure. The post-fire stiffness of beams B1 and B4 are similar as they attain similar peak rebar temperatures. However, the stiffness of beam B1 is almost 10% higher than beam B2 which was subjected to a higher load level. This is despite the fact that compressive strength of cores drilled from unaffected support zones of beam B2 was almost 8% higher compared to those from beam B1. This indicates that higher load level during fire exposure has a detrimental effect on stiffness of fire damaged RC beams. Thus, stiffness of the fire exposed RC beams is governed by peak temperatures experienced in the cross-section, and level of loading present on beams during fire exposure.

The second phase of load-deflection response is characterized by onset of yielding in tensile steel reinforcement. Beams B1, B2, and B4, which experienced peak rebar temperatures of 521°C, 510°C and 500°C respectively, yielding of rebars occurred at a load of 91 kN, 99 kN, and 93 kN respectively. Counterintuitively, beam B2 which was subject to greater load level during fire exposure yielded at a higher load by about 10% as compared to beam B1 exposed to identical fire exposure. This can partly be attributed to higher compressive strength of concrete in beam B2, as well as relatively lower peak rebar temperatures experienced in the beam. Thus, peak rebar temperature attained during fire exposure and concrete strength are critical in determining yielding load of fire damaged concrete beams. However, the effect of load level on yielding load of fire damaged RC beams was found to be very limited.

Once yielding occurs, fire damaged beams B1, B2 and B4 exhibit a strain softening response after yielding (see B-C in Fig. 7). In fact, compression rebars in

NSC beams experienced buckling soon after yielding of tensile steel reinforcement, as concrete under compression fails through crushing. The cracking pattern at failure is shown for beams B1 and B4 after residual capacity testing in Fig. 8c, d. The orientation and extent of cracking around the mid-span of the fire damaged beams are clearly indicative of flexural failure for all three tested beams. Since the end (support) zones of the beam are not exposed to fire, the shear capacity of the beams (at supports) does not reduce significantly as compared to flexural capacity (at mid-span). Also, the large deflection of the beam, with respect to its span indicates flexural (ductile) failure.

The peak load-carrying capacity in beams B1, B2, and B4 was measured to be 91 kN, 99 kN, and 93 kN respectively. The measured residual capacity in all three



(a) Beam B1 at the beginning of residual tests



(b) Beam B4 at the beginning of residual tests



(c) Critical mid-span for beam B1 at failure



(d) Critical mid-span for beam B4 at failure

Figure 8. Crack patterns observed post-cool down prior to residual capacity testing, and failure mode at the end of residual capacity test photographed in tested beams B1 and B4 highlighting flexural cracks, and flexural failure.

beams is comparable to the room temperature design capacity of 92.7 kN computed as per ACI 318 [20], in spite of significant temperature (fire)-induced damage in these beams. It should be noted that ACI 318 [20] design equations are not intended to capture ultimate capacity of flexural members as measured through tests, but provided a lower bound expected capacity. This comparison illustrates that fire damaged concrete beams can retain load carrying capacity comparable to load levels they were designed to sustain prior to fire exposure. Thus, following a fire incident, fire damaged beams may satisfy design limit state from strength consideration but need to be retrofitted to provide comparable level of safety (capacity) which existed prior to the fire incident.

5. Conclusions

A three-stage testing approach comprising of evaluating pre-fire response (Stage 1), response during fire exposure including extended cool down of the member (Stage 3), and finally post-fire (after cooldown) residual response is needed for evaluating residual capacity of fire damaged members. Both fire severity (especially cooling phase) and stress (load) level experienced by the member during a fire incident influence its post-fire residual response. Abrupt failure of RC member (beam) can occur during cooling phase of fire exposure when the cooling (phase) is at a slower rate (about -6°C per minute) representing no fire-fighting intervention, following rapid heating conditions in a severe fire. Also, rapid cooling results in greater level of strength and stiffness recovery in the RC member as it cools down to ambient conditions. Load (stress) level prior to and during fire exposure has limited influence on residual load carrying capacity, but adversely impacts post-fire stiffness of fire damaged concrete members. An increase in load level by about 20% can result in a 30% greater reduction in post-fire stiffness of a fire damaged concrete beams subject to similar fire exposures. Furthermore, irrecoverable residual deformations in fire damaged concrete members (beams) are significantly larger than pre-fire deflections, and hence negatively impact the serviceability of member. The level of residual deformation is directly proportional to peak rebar temperatures experienced within the concrete beam if load level is maintained constant. For similar peak rebar temperatures, presence of higher (by 20%) load level during fire exposure leads to significant increase (by 100%) in post-fire residual deformations. Thus, following a fire incident, fire damaged beams may satisfy design limit state from strength consideration, but may need some level of retrofitting to provide comparable level of safety (capacity) and serviceability which existed prior to the fire incident.

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