Post-fire serviceability and residual strength of composite post-tensioned concrete T-beams

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Abstract

In this study, the response of ten composite post-tensioned concrete beams topped by a reinforced concrete deck with adequate reinforcing shear connectors is investigated. Depending on the concrete compressive strength of the deck slab (20, 30, and 40 MPa), beams are grouped into three categories. Seven of these beams are exposed to a fire attack of 700 and 800 °C temperature simultaneously with or without the presence of a uniformly distributed sustained static loading. After cooling back to ambient temperature, these composite beams are loaded up to failure, using a force control module, by monotonic static loading in a four-point-bending setup with two symmetrical concentrated loads applied in the middle third of the effective span. The objectives of this study include investigating the behavior of the composite prestressed concrete beams under and after the exposure to a direct fire flame, as well as finding their residual load-carrying capacity. Tests demonstrate significant deteriorations caused by exposure to high temperatures associated with the increase of the member's camber. The increase of the midspan camber after heating exposure reached approximately 200%. On the other hand, the 1-h steady-state exposure of test specimens to temperatures of 700 and 800 °C led to reduce the load-carrying capacity of the heat-deteriorated beams up to 45% and 54%, respectively.

Keywords Fire scenario · Composite section · Post-tensioned concrete beam · Camber · Structural integrity

1 Introduction and research significance

Reinforced concrete is considered as the most versatile construction material used in structural engineering applications. At the same time, it is common knowledge that fire is one of the most severe environmental disasters that a concrete structure may experience during its service lifetime, where the ultimate strength of structural concrete members dramatically influenced due to the degradation in both stiffness and strength of the component materials. However, the main effect of fire on the degradation of the stiffness and the strength of the structural concrete members depends on the peak degree of the temperature attained during the fire events and the duration of exposure to the fire.

Composite post-tensioned concrete T-beams have been widely implemented in civil and industrial concrete structures, where their fire resistance is highly dependent on their configuration, the constituent of materials used in their fabrication, the type, and intensity of the applied load, and the characteristics of the fire itself. These composite systems consist of two components of different initial stress state, different concrete strengths, and different ages. The two components were connected together by shear connectors to enable performing under the applied load as one unit.

It is worth mentioning that high strength concrete (HSC) is commonly used in the fabrication of such structural concrete members, where this type of concrete typically has higher compressive strength than normal-strength concrete (NSC) and less porous content. The

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lower porosity of (HSC) may lead to more spalling due to the high pressure that created inside the concrete structure [16, 15]. However, other researches have been proved that (HSC) is showing higher resistance against spalling due to its improved tensile strength [2, 22].

Aslani [4] suggested empirical equations to predict parameters influencing the performance of unconfined prestressed normal-strength and high-strength concrete at elevated temperatures, mainly, the residual concrete strength in compression, strain at peak stress, initial Young's modulus, thermal strain, and the uniaxial stress-strain relationship in compression.

In terms of the effect on the loss of concrete strength, Chan et al. [10] categorized high temperatures into three ranges, namely, 20–400 °C, 400–800 °C and above 800 °C. In the range, 20–400 °C, HSC by and large, unlike NSC, maintained its original strength. Between 400 and 800 °C, both HSC and NSC lost most of their original strength, especially at temperatures above 600 °C. Above 800 °C, only a small portion of the original strength was left for concretes.

The performance of reinforced concrete beams under fire attack depends on many factors. These include a degradation of the physical and mechanical properties of the constituent materials resulted from fire and the heat distribution through the member.

Kigha et al. [23] emphasized experimentally that the steel tensile strength is significantly influenced by heating exposure if the temperature exceeds 700 °C. Accordingly, exposures to heating temperatures of 700 and 800 °C were considered to cause significant in the load-carrying capacity and stiffness of the heated beams associated with the highest temperatures [18, 19].

Thongchom et al. [35] carried out a test program on five simply supported reinforced concrete T-beams to investigate the effect of sustained service loading of 31.6 kN at elevated temperatures on the residual flexural response after exposure to heating temperatures of 700 and 900 °C for three hours and then cooled in ambient condition. After the fire test, all beams were tested under static four-point bending up to failure. The authors concluded that the sustained service loading of 31.6 kN has a detrimental effect on the post-fire flexural behavior of RC beams. The effect was more pronounced on the post-fire stiffness and ductility than on strength.

Kodur and Agrawal [26, 27] proposed a nonlinear analysis model using ABAQUS finite element program for reinforced concrete beams. The proposed model incorporated the steel and concrete properties during and post-fire exposure (during a transient-state heating phase, a steady-state heating phase, and a cooling phase in the ambient air). The investigation results indicated

that significant plastic deformations could occur in reinforced concrete beams after fire attack.

The behavior of composite post-tensioned concrete beams during and post-fire exposure has not been fully addressed. There is only a limited number of experimental and numerical studies that exist at the moment that investigate the behavior of the fire exposed composite post-tensioned concrete beams at the serviceability stage and their bearing capacity at the ultimate stage. Meantime, it is well known that the residual load-carrying capacity of a fire-damaged concrete structural member depends on the distribution pattern of the temperature which has happened within its cross-section, the duration of exposure, and the peak temperature attained during the fire.

Today, there is an increasing need to provide extensive studies for the evaluation of the post-fire performance and the structural integrity of such structural concrete members to enhance their structural safety in case the decision was taken about their retrofitting and reinstatement.

To ensure in future safe exploitation of the composite concrete members, and to propose an effective and adequate strengthening system for fire deteriorated prestressed concrete members before re-occupancy, the assessment of the residual load-carrying capacity is essential. These required data in serviceability and ultimate stages of performance justify the objectives of this study.

The present article is organized as follows. An introduction, literature review of the previous studies, and the objectives of the recent investigation is mentioned this section. The preparation of tested beams, their dimensions and reinforcement details are outlined in Sect. 2. The instrumentation and testing scenarios of the experimental beams are given in Sect. 3. Experimental results and their discussion are described in Sect. 4, including the mechanical properties of concrete and steel after fire exposure, deformability of test beams during thermal exposure and static load application, cracking and spalling of test specimens during thermal exposure and external loading application, concrete strain during static loading stage, and the failure mode of tested specimens. In Sect. 5, the finite element analysis of tested beams after fire exposure, using ABAQUS software, is explained. In Sect. 6, finally, the main conclusions are summarized.

2 Preparation of experimental beams

In the present study, an experimental investigation has been undertaken to understand during- and post-fire performance of ten simply-supported composite post-tensioned concrete beams that are exposed to realistic fire scenarios followed by monotonic static loading to evaluate their residual strength.

Each composite concrete beam is composed of an individual post-tensioned concrete beam, of 3300 mm in length with a rectangular cross-sectional configuration of (300 mm deep $\times$ 170 mm wide), and a reinforced concrete deck slab of 400 mm wide and 50 mm deep Fig. 1. All individual post-tensioned beams were reinforced by nonprestressed steel bars of two 10 mm in diameter (with yield and ultimate strengths, respectively, 570 and 678 MPa) in the tension zone and two in 8 mm diameter (with yield and ultimate strengths of 497 and 650 MPa, respectively) in the compression zone of the cross-section. Post-tensioned beams were cast using concrete of target cylinder compressive strength of 40 MPa at 28 days of age. Then at age 28 days, a prestressing force of 120 kN was applied from one end to a low-relaxation 7-wire steel strand with 1862 MPa ultimate strength (Grade 270), 98.7 mm² cross-sectional area, and 12.7 mm in diameter using a portable prestressing jack integrated to the dynamometrical system which manages the hydraulic working of the oil pump. This force was selected so that it satisfies the limits of the ACI 318 M-19. The recorded upward midspan displacement (camber) was 1.1 mm at the prestressing transfer stage. After the transfer of the prestressing force (i.e., at 28 days age), the deck slabs of composite beams were cast. Depending on the concrete compressive strength of the deck slab, the composite beams were divided into three groups. Deck slabs were fabricated with 20, 30, or 40 MPa concrete compressive strength for composite beams in groups I, II, and III, respectively. All deck slabs were reinforced by four 8 mm in diameter mild steel bars with the same mechanical properties mentioned above for the nonprestressed steel in the compression zone of the post-tensioned beam.

In each group, one composite beam was kept as a control specimen exposed to static loading to failure. While

the rest seven composite beams were heated experimentally up to 700 °C or 800 °C following the ASTM E119-10 [5] fire scenario for one hour to simulate the genuine fire events and then being tested under the same loading setup as that used for the three control beams. Three out of seven composite beams were exposed to 700 °C heating temperature only before applying the static loading test. While the other four composite beams were experienced the mentioned above temperatures (i.e., two to 700 °C and two to 800 °C) and a uniformly distributed loading, which simulated the dead load on the tested member. The fire test comprised three phases, mainly, transient-state heating phase, steady-state heating phase, and cooling phase.

Table 1 shows the considered beam test variables, where the identity code “B1-F20”, “B2-F30”, “B3-F40” refers to the composite concrete specimen with an individual post-tensioned beam of 40 MPa target cylinder concrete compressive strength and 20, 30, and 40 MPa, respectively, target cylinder concrete compressive strength for deck slab.

While the identity code “R” refers to the control unheated beam, “700” refers to beams exposed to a heating temperature of 700 °C, and “800” refers to beams exposed to 800 °C. The symbol “U” refers to a uniformly distributed load, which was applied on the top surface of the beam’s deck slab during the fire test stage only and removed during the stage of application of the monotonic static test.

3 Instrumentation and testing of experimental specimens

Except for the reference composite post-tensioned concrete T-beams B1-F20-R, B2-F30-R, and B3-F40-R, the test program was carried out on each composite beam

Fig. 1 Beam layout, dimensions, and reinforcement details

Table 1 Composite post-tensioned concrete beam test variables

Specimen ID		Design cylinder concrete compressive strength at 28 days age (MPa)		Actual cylinder concrete compressive strength at 28 days age (MPa)		Details and parameters considered during fire exposure stage	
		Post-tensioned beam	Deck slab	Post-tensioned beam	Deck slab	Applied superimposed load (kN/m)	Applied heating temperature (°C)
Group I	B1-F20-R	40	20	42	20.5	Control	
	B1-F20-700	40	20	42	20.5	0.0	700
Group II	B2-F30-R	40	30	42	31.0	Control	
	B2-F30-700	40	30	42	31.0	0.0	700
	B2-F30-700U	40	30	42	31.0	1.1	700
	B2-F30-800U	40	30	42	31.0	1.1	800
Group III	B3-F40-R	40	40	42	41.5	Control	
	B3-F40-700	40	40	42	41.5	0.0	700
	B3-F40-700U	40	40	42	41.5	1.1	700
	B3-F40-800U	40	40	42	41.5	1.1	800

following two different testing stages. During the first stage, the experimental beams were exposed to a transient-state heating phase with a variable period to reach the target temperature, a steady-state heating phase with a constant period for one hour beyond attaining the target temperature, and a cooling phase in the ambient air. During the second stage, the test beams were subjected to external monotonic static loading up to failure. Meantime, the composite beams B1-F20-R, B2-F30-R, and B3-F40-R were exposed only to monotonic static loading to consider them as control specimens.

First stage—direct exposure to fire effect: a horizontal fire chamber was designed and manufactured for this study with 750 × 500 × 3500 mm dimensions to subject the test specimen to direct fire flame from four fire sources positioned at the bottom of the fire chamber to simulate underneath fire disaster on the soffit of the tested member Fig. 2. At age of 28 days for the deck slab (i.e., 56 days for the post-tensioned concrete beam), all test specimens were secured in the fire chamber using suitable supports. Four thermocouples type K (Nickel–Chromium/Nickel–Alumel), with 2 mm outer diameter, were used to measure the temperature variation, two inside the test specimen while the other two within the fire chamber outside the specimen. These thermocouples can measure heating temperatures up to 1400 °C. The first two thermocouples were inserted inside holes of 6 mm depth bored at mid-depth of the beam's flange at midspan and the north end sections of the test specimen and covered by a thin layer of gypsum material for fixation and protection purposes, while the other two thermocouples were inserted inside the fire chamber through its top cover and clamped in the chamber's top cover at the same section positions of the first two thermocouples. The heating temperature

was controlled by changing the amount of the supplied methane gas.

The adopted heating rates of the fire test were based on the time–temperature scenario of the ASTM-E119. While the gradual cooling regime was selected to carry natural character until attaining the ambient room temperature. Accordingly, one composite post-tensioned concrete T-beam from each group was exposed to a high temperature of 700 °C, mainly, specimens B1-F20-700, B2-F30-700, and B3-F40-700. While the other two beams in each of the second and the third groups, (B2-F30-700U, B2-F30-800U, B3-F40-700U, B3-F40-800U), were exposed to a high temperature of (700 or 800 °C) and uniformly distributed loading of 1.1 kN/m, which simulated the dead load on the tested member, using 10 concrete blocks each of 32.5 kg mass and 450 × 300 × 100 mm dimensions.

The uniformly distributed loading was applied before the commencement of the heating phases and maintained constant for the entire fire test period. The required time to reach the target heating temperatures of 700 and 800 °C inside the fire chamber was 15 and 25 min, respectively, and inside the composite post-tensioned concrete beam was 55 and 65 min, respectively.

The heating temperature was kept constant at the target value for 60 min and then cooled gradually to room temperature after stopping the heating sources and consequently removing the top cover of the fire chamber. During the heating and cooling phases, the monitoring devices used included a mechanical strain indicator (dial gauge) with 0.01 mm/div. sensitivity used at the top surface of the tested specimen at mid-span section to measure the absolute central displacement due to the applied dead load and the heating temperature and a digital

Fig. 2 Schematic shape of the fire chamber used in the present study

thermometer reader to record temperature readings from thermocouples at one second time intervals.

By the end of this stage and on the same day, the superimposed dead load was removed from the tested specimen, strain gauges were attached to the concrete surface to observe the concrete behavior during loading, and consequently, the composite beam was shifted to the testing rig which supplied with 1000 kN maximum capacity electrical hydraulic jack to start the second stage of testing on the next day.

Second stage—application of external static loading: to investigate the service performance and the residual load-carrying capacity of the test specimens, all beams including the references were exposed to monotonic

static loading using a force control module in a four-point-bending setup with two symmetrical concentrated loads applied at the middle third of the effective span at room temperature Fig. 3. The applied load was controlled by a 300 kN load cell with a digital load reader. The test program for each specimen started by many cycles of small loading steps, about 5 kN, to get rid of any slack in the test setup and measuring devices.

Afterward, the load was applied progressively with 10 kN loading steps up to failure. The monitoring devices used during this stage included load cell, mechanical dial gauges, and electrical resistance strain gauges. Two mechanical dial gauges were used; the first one, with 0.01 mm/div. sensitivity was located at the midspan

Fig. 3 Set-up and instrumentation of test beams during loading exposure

section underneath the soffit of the tested specimen to record the absolute central displacement due to the progressively applied loading. While the other dial gauge, with 0.001 mm/div. sensitivity was fixed horizontally at the north end of the tested beam to detect any slip that may occur between the lower post-tensioned concrete beam and the upper reinforced concrete deck slab. Uniaxial electrical resistance strain gauges, type (PL-60-11-5L) of (60) mm base length, denoted S_1 to S_5 were used to monitor the longitudinal strain in concrete at different locations across the midspan section, see Fig. 3.

4 Experimental results and discussion

4.1 Mechanical properties of materials after fire exposure

Some of the parameters that control concrete performance are compressive strength, tensile strength, and modulus of elasticity, which are nonlinear functions of temperature [4]. To determine the average compressive strength, modulus of rupture, and modulus of elasticity of the concrete at room temperature (25 °C) at 28 days age, three standard specimens per each type of testing per each design strength (i.e., for 20, 30, and 40 MPa) were tested. The average concrete compressive strength and the modulus of elasticity were obtained according

to ASTM C39/C39M-20 [6] and ASTM C469/C469M-14 [7], respectively, using cylindrical specimens of dimensions 150 × 300 mm (diameter × height).

While the average concrete modulus of rupture was determined according to ASTM C78/C78M-18 by testing concrete prisms of dimensions 400 × 100 × 100 mm.

To investigate the change of the mechanical properties of the above mentioned concretes after heating and cooling phases of fire tests of 300, 500, 700 or 800 °C temperatures, following the ASTM E119 curve, 72 concrete cylindrical specimens of dimensions 150 × 300 mm (diameter × height) and 36 concrete prismatic specimens of dimensions (400 × 100 × 100 mm) were heated and then cooled gradually to room temperature after terminating the heating sources to consider the average value of three specimens per each test. Consequently, on the same day, the concrete specimens were tested following the procedures mentioned in ASTM C39/C39-20 [6], ASTM C469/C469M-14 [7], and ASTM C78/C78M-18 [8], respectively. The tested concrete cylinders and prisms were exposed to heating temperatures which were kept constant at the levels of 300, 500, 700, or 800 °C for 60 min before starting the cooling phase.

The knowledge of the high-temperature mechanical properties of concrete is a critical issue for the assessment of its fire resistance. It is worth to mention that, all tests on the mechanical properties of concrete were performed on the same day as the fire tests for the composite post-tensioned concrete beams. The obtained data were very

important for finite element modeling, considering the effect of fire exposure on the residual strength of concrete. Tables 2, 3 and 4 show the residual compressive strength, modulus of rupture, and initial modulus of elasticity for all types of concrete used in the tested beams.

Also, Fig. 4 demonstrates the degradation of the most important mechanical properties of concrete due to the exposure to elevated temperatures. Accordingly, as the concrete of 40 MPa compressive strength was heated to 300 °C, the loss of its original compressive strength, modulus of rupture, and modulus of elasticity attained 6%, 13.2%, and 12.7%, respectively. Meanwhile, as this concrete was exposed to temperatures of 700 °C and 800 °C, the reduction of the mentioned parameters reached (63.9–78.3%), (71–81.6%), and (63–78.9%), respectively. The concrete of 30 MPa design compressive strength loss 9.7%, 12.1%, and 12.8%, respectively, of its original compressive strength, modulus of rupture, and modulus of elasticity when heated to 300 °C and (61.3–74.2%), (69.7–78.8%), and (59.9–75.2%), respectively, at heating temperatures of (700–800 °C). While the concrete of 20 MPa design compressive strength loss 12.2%, 10.7%, and 12.9% of its original compressive strength, modulus of rupture, and modulus of elasticity, respectively, when heated to 300 °C and (51.2–70.7%), (67.9–78.6%), and (58.2–70%), respectively, at heating temperatures of (700–800 °C).

Table 2 Residual concrete compressive strength after heating and cooling phases of fire test

Heating temperature (°C)	Parameter	Residual cylinder concrete compressive strength (MPa)		
		Design concrete compressive strength at 28 days age f'_c (MPa)		
		40	30	20
Room temp., 25	$f'_c @ 25$	41.5	31.0	20.5
300	$f'_c @ 300$	39.0	28.0	18.0
	$(f'_c @ 300 / f'_c @ 25) \times 100\%$	94.0	90.3	87.8
500	$f'_c @ 500$	27.0	21.0	14.0
	$(f'_c @ 500 / f'_c @ 25) \times 100\%$	65.0	67.7	68.3
700	$f'_c @ 700$	15.0	12.0	10.0
	$(f'_c @ 700 / f'_c @ 25) \times 100\%$	36.1	38.7	48.8
800	$f'_c @ 800$	9.0	8.0	6.0
	$(f'_c @ 800 / f'_c @ 25) \times 100\%$	21.7	25.8	29.3

Each value is the average from testing three concrete cylinders of dimensions 150 × 300 mm

Table 3 Residual concrete modulus of rupture after heating and cooling phases of fire test

Heating temperature, °C	Parameter	Residual concrete modulus of rupture, (MPa)		
		Design cylinder concrete compressive strength at 28 days age f'_c , (MPa)		
		40	30	20
	$f'_c @ 25$	41.5	31	20.5
Room temp., 25 300	$f_r @ 25$	3.8	3.3	2.8
	$f_r @ 300$	3.3	2.9	2.5
500	$(f_r @ 300 / f_r @ 25) \times 100\%$	86.8	87.9	89.3
	$f_r @ 500$	2.2	2.1	2.0
700	$(f_r @ 500 / f_r @ 25) \times 100\%$	58.0	63.6	71.4
	$f_r @ 700$	1.1	1.0	0.9
800	$(f_r @ 700 / f_r @ 25) \times 100\%$	29.0	30.3	32.1
	$f_r @ 800$	0.7	0.7	0.6
	$(f_r @ 800 / f_r @ 25) \times 100\%$	18.4	21.2	21.4

Each value is the average from testing three concrete prisms of dimensions 400 × 100 × 100 mm

It should be mentioned that the higher rates of original strength loss, as much as (60–80%), were observed for all types of concrete at temperatures 700–800 °C. This observation interprets by three reasons, which significantly affected by the level of the heating temperature, mainly; the irreversible transformation of the physical and chemical properties of the concrete constituent materials (i.e., cement, fine, and coarse aggregates), the structural damage of bonding between the constituents of the concrete matrix which had been subjected to elevated temperatures, and the thermal cracking intensity.

To study the change of the yield and ultimate strengths of the steel bars and strands after heating and cooling to room circumstance phases of the adopted fire tests of 300, 500, 700, or 800 °C temperatures, following the ASTM E119 curve, three specimens for each steel type per each level of these elevated temperatures were tested in a steady-state condition (i.e., the test was conducted under a constant room temperature after cooling and constant load rise). Table 5 shows the data of these strengths for the steel bars and strands of 10 mm and 12.7 mm in diameter, respectively. The ultimate strength of steel strands was more sensitive than the yield strength and the ultimate strength of reinforcing steel bars at identical heating temperatures, also, the strands dramatically lose its ductility. It is worth to mention that the initial modulus of

Table 4 Residual concrete initial modulus of elasticity after heating and cooling phases of fire test

Heating temperature, °C	Parameter	Residual concrete modulus of elasticity, (MPa)		
		Design cylinder concrete compressive strength at 28 days age f'_c , (MPa)		
	$f'_c @ 25$	40	30	20
		41.5	31	20.5
Room temp., 25	$E_c @ 25$	30,459	26,168	21,019
300	$E_c @ 300$	26,600	22,810	18,301
	$(E_c @ 300 / E_c @ 25) \times 100\%$	87.3	87.2	87.1
500	$E_c @ 500$	18,892	17,010	14,292
	$(E_c @ 500 / E_c @ 25) \times 100\%$	62.0	65.0	68.0
700	$E_c @ 700$	11,270	10,500	8790
	$(E_c @ 700 / E_c @ 25) \times 100\%$	37.0	40.1	41.8
800	$E_c @ 800$	7740	6500	6305
	$(E_c @ 800 / E_c @ 25) \times 100\%$	21.1	24.8	30.0

* Each value is the average from testing three concrete cylinders of dimensions 150 × 300 mm

Fig. 4 Degradation of concrete mechanical properties after heating and cooling phases of fire test

Table 5 Residual steel strength after heating and cooling phases of fire test

Heating temperature (°C)	Residual steel strength					
	Steel bars Ø10 mm				Steel strand Ø12.7 mm	
	Yield strength		Ultimate strength		Ultimate strength	
	f_{yr} (MPa)	$\left\{ \frac{f_y}{f_{y@25}} \right\} \times 100$ (%)	f_{sur} (MPa)	$\left\{ \frac{f_{su}}{f_{su@25}} \right\} \times 100$ (%)	f_{pur} (MPa)	$\left\{ \frac{f_{pu}}{f_{pu@25}} \right\} \times 100$ (%)
Room temp., 25	570	100	678	100	1860	100
300	570	100	678	100	1860	100
500	542	95.1	610	90.0	1395	75.0
700	445	78.1	465	68.6	706	38.0
800	422	74.0	440	64.9	558	30.0

Each value is the average from testing three samples

elasticity for both types of steel was not affected by heating and cooling for elevated temperatures.

4.2 Deformability of test specimens during thermal exposure stage

The deformability of the experimental composite post-tensioned concrete beams was assessed by investigating the midspan camber—heating time diagrams at different heating temperatures, see Figs. 5–6. Overall, the camber–heating time curves show a nonlinear response in all time intervals for all test beams. The camber–time response exhibited three stages of behavior.

The first stage of behavior, which corresponded to the transient-state heating phase, characterized by the first ascending branch slightly deviated from the linear response because the flexural stiffness was very close to the initial stiffness before fire exposure. During this stage, the camber increase occurred at a slow rate mainly due to the thermal strains generated by high thermal gradients in the early stage of fire exposure. To reach the target heating temperatures of 700 and 800 °C in the test specimen, the transient-state heating phase required 55 and 65 min, respectively, and 15 and 25 min, respectively, to attain these temperatures in the fire chamber.

The second stage of behavior, which corresponded to the steady-state heating phase, was characterized by the second ascending branch which expressed the pronounced nonlinear response due to the degradation of the flexural stiffness and the strength of concrete of the structural member after the fire exposure. During this stage, the heating temperatures were kept constant for 60 min. Accordingly, the camber rise occurred at a higher rate mainly due to the generated thermal strains, and the accelerated creep strains resulted from high temperatures in concrete which composed of different materials. It is worth

mentioning that the progressive increase of the nonlinear camber with time was continued beyond the 60-min constant temperature period for the test specimens that were exposed to the effect of the applied prestressing force only (i.e., specimens B1-F20-700, B2-F30-700, and B3-F40-700). While the second ascending branch was terminated at the peak camber by the end of the 60-min constant temperature period for the test beams that experienced the effect of the applied prestressing force and the sustained superimposed dead load together (i.e., specimens B2-F30-700U, B2-F30-800U, B3-F40-700U, and B3-F40-800U).

The third stage of behavior, which corresponded to the cooling phase, characterized either by continuing the progressive increase of the nonlinear response up to the peak camber then followed by the descending branch (i.e., post-peak nonlinear response) or by the immediate initiation of the post-peak nonlinear behavior depending on the existing loading condition on the test beam. The descending branch expressed the nonlinear response of the reduction of the camber with time due to the steady recovery of the concrete strength. To reach the target room temperatures after the exposure to 700 and 800 °C, the natural cooling regime phase for the test specimens required in maximum 510 and 555 min, respectively, with a rate of 1.35–1.60 °C/min.

The midspan camber results for each of the test beams are summarized in Table 6.

It was observed from Figs. 6 and 7 that the midspan camber of the test specimens through the heating and cooling phases affected, at different levels, by the magnitude of heating temperature, the presence of the superimposed dead load, and the concrete compressive strength of the reinforced concrete flange of the section. Although the two considered magnitudes of heating temperature (700 and 800 °C) are close to each other, with a slight difference, the residual midspan camber and behavior of the

Fig. 5 Camber-heating time diagram for all specimens after heating and cooling phases of fire test

heated specimen were completely different, especially the post-peak nonlinear response during the cooling phase. Accordingly, the increase of the midspan camber after heating exposure to 700 and 800 °C reached (200, 181.8, 227.3, and 190.9%) and (190.9 and 200%), respectively. These changes in camber magnitude were considered significant for composite post-tensioned concrete beams exposed to 700 and 800 °C temperatures and eccentric prestressing force due to the fact that the stiffness is reduced depending on the strength degradation level of the concrete and steel reinforcement associated with the highest temperatures.

It was emphasized experimentally that the presence of the sustained superimposed dead load simultaneously with the exposure to elevated temperatures led to limit the excessive increase of the midspan camber of the test specimen. This evidence attributes to the fact that the applied dead load in nature counteracts the effect of the moment from the prestressing force. Accordingly, test results showed that specimens B2-F30-700U, B2-F30-800U, B3-F40-700U, and B3-F40-800U that subjected to the simulated superimposed dead load in addition to the beam self-weight during fire exposure experienced larger creep midspan deflection. Thus, under 700 °C exposure, the residual camber after the fire test decreased by 9.7% and

12.5% for specimens B2-F30-700U and B3-F40-700U compared to B2-F30-700 and B3-F40-700, respectively. This impact may be reflected with significant benefit on the post-fire performance of the simply supported composite post-tensioned concrete beams due to the restraining of the deterioration in the top concrete fibers, especially if these fibers exist at the pre-fire stage in tension due to the prestressing force, as the tensile strength of concrete is greatly affected by heating if the temperature is more than 700 °C.

It was revealed in Fig. 6 that, at the same heating temperature and loading circumstance, increasing the concrete compressive strength of the top flange led to slightly increasing the residual midspan camber.

Accordingly, the increase of the midspan camber after heating exposure to 700 °C reached 200, 209.1, and 227.3% in specimens with a top flange of 20, 30, and 40 MPa design compressive concrete strength, respectively. Whereas this increase after heating exposure of 700 °C and 800 °C in the presence of the superimposed loading attained (181.8 and 190.9%) and (190.9 and 200%) in beams with a concrete flange of 30 and 40 MPa, respectively. This evidence attributes to the increased deteriorations that occur in the concrete of the top flange with higher compressive strengths that are caused by the increase in density and

Fig. 6 Effect of flange concrete compressive strength on the behavior of test specimens under fire

the decrease in porosity that affects the thermal conductivity of the concrete.

4.3 Cracking and spalling of test specimens during thermal exposure stage

Generally, spalling under fire exposure is considered one of the major problems which result in the rapid loss of the bearing capacity of all concrete types. Spalling is defined as the breaking up of surface layers (pieces) of concrete from the structural concrete member when it is exposed to high and rapidly rising temperatures, such as those encountered in fires, and the explosive spalling occurs more suddenly and violently [24, 33].

Three types could account for the spalling phenomenon of concrete, mainly, the thermal-mechanical spalling, thermal-hydro spalling, and thermal-chemical spalling [20, 24, 28, 29, 31, 32]. Most researchers consider that thermal-hydro spalling, appearing at the early heating stage, is the most critical among the others [36]. Many researchers believe that due to the low permeability of concrete, resulted from the low porosity, the extremely high water vapor pressure, generated during fire exposure and restrained inside the concrete body could generate the thermal-hydro spalling phenomenon

when the pore pressure gradually grows-up, attains the saturation vapor pressure and exceeds the concrete tensile strength [13, 14, 17, 22, 29–32, 36]. It should be mentioned that at 300 °C, the vapor pressure could approximately reach the saturated case of 8 MPa [24, 29]. Such internal pressure is comparably too high to be resisted the concrete which has a limited tensile strength of about 5 MPa. The spalling can occur immediately after exposure to rapid heating and can be accompanied by explosions or it may happen during later stages of exposure when concrete has become so weak after heating such that, when cracks develop, pieces of concrete fall off from the surface of concrete member [25]. The spalling exposes the concrete core of the structural member to fire temperatures, thereby increasing the rate of transmission of heat to the inner layers of the concrete core and the reinforcement. The spalling may lead to early loss of integrity and stability.

It was noticed that all test beams which exposed to fire showed a distinct network of minor surface cracks, no major cracks were fixed during testing. While thermal-hydro spalling type was reported in some tested specimens, the other beams were showing insignificant or no obvious spalling. The reason behind this conflicting performance on the appearance of this type of spalling

Table 6 Change of midspan camber at the end of each period of burning and cooling stage for all CPC beams

Specimen ID	Camber due to pre-stressing Δ_{pr} , (mm)	Midspan camber during thermal exposure stage						Change of camber $\left\{ \frac{\Delta_{co}-\Delta_{pr}}{\Delta_{pr}} \right\} \times 100$ (%)	Recovery of camber $\left\{ \frac{\Delta_{pr}-\Delta_{co}}{\Delta_{co}-\Delta_{pr}} \right\} \times 100$ (%)
		Transient-state heating phase		Steady-state heating phase		Cooling phase			
		Period length, (minute)	Accumulative camber Δ_{tr} (mm)	Period length, (minute)	Accumulative camber Δ_{sr} (mm)	Period length, (minute)	Accumulative camber Δ_{co} (mm)		
Group I	B1-F20-R	– 1.1	–	–	–	–	–	–	–
	B1-F20-700	– 1.1	– 2.4	60	– 4.2	465	– 3.3	200.0	40.9
Group II	B2-F30-R	– 1.1	–	–	–	–	–	–	–
	B2-F30-700	– 1.1	– 2.5	60	– 4.3	500	– 3.4	209.1	39.1
	B2-F30-700U	– 1.1	– 2.3	60	– 4.2	510	– 3.1	181.8	55.0
	B2-F30-800U	– 1.1	– 2.7	60	– 4.4	535	– 3.2	190.9	57.1
Group III	B3-F40-R	– 1.1	–	–	–	–	–	–	–
	B3-F40-700	– 1.1	– 2.6	60	– 4.4	435	– 3.6	227.3	32.0
	B3-F40-700U	– 1.1	– 2.3	60	– 4.1	445	– 3.2	190.9	42.9
	B3-F40-800U	– 1.1	– 2.6	60	– 4.3	560	– 3.3	200.0	45.5

may attribute to the huge number of parameters that affect spalling and their interdependency. In some test beams, the thermal-hydro spalling type was detected at an early stage of heating, at about 100 °C, approximately 10 min later after the commencement of the heating phases which was accompanied by fierce explosions. This evidence may attribute to the fact that these specimens were not fully dry and the liquid water inside the body of concrete became vapor that led to the gradual build-up pressure inside pores. The induced pore pressure attained its peak value, which was larger than the tensile strength of concrete, and as a result, the spalling occurred because concrete could not resist the mentioned pressure.

It was observed in all test specimens that at later stages of fire exposure, a thermal-chemical spalling of sloughing-off spalling type has occurred at extremely high temperatures of (700 °C and 800 °C). After cooling, a network of minor cracks in specimens had increased and a thermal-chemical spalling of post-cooling spalling type was reported. These two types of thermal-chemical spalling were occurred due to the break-down of aggregate cement bonds, such as calcium silicate hydroxide and calcium hydroxide [3, 11, 12, 34, 37].

It should be mentioned that more deterioration, which led to spalling, was monitored at the soffit of the web due to the fact that the concrete of the web was generally greater than that in the flange, except the specimens of the third group, and the fire sources were directly located under the web.

Figure 7 shows the surface hairline cracking and spalling of some test specimens.

4.4 Deformability of test specimens during static loading exposure stage

The load-midspan deflection behavior for the three control beams B1-F20-R, B2-F30-R, and B3-F40-R shows a nearly linear response until the appearance of the first crack in tension zone at an average applied load of 60 kN. The curves then start progressively deviate from linearity up to the peak load. At an average loading stage of 115 kN, indications of the tension steel yielding were observed as the test specimens experienced excessive deformability with a slight increase in the failure load. After the yielding of tension steel, the midspan deflection increased in a steady manner which indicating the ductile behavior of the test specimens. As indicated in Figs. 8 and 9, the load-midspan deflection behavior after the decompression stage, (i.e., the loading stage which causes a zero stress at the precompressed fiber of the midspan section), characterized by three stages of response. In the first stage, the flexural stiffness was not affected by the applied initial loading steps because there was no cracking

Fig. 7 Surface hairline cracking and spalling at 700 °C and 800 °C

in concrete. As the applied load exceed the cracking load, cracks tended to open with increasing applied load and the second stage of response was observed to initiate with reduced flexural stiffness, followed by the third stage of behavior that corresponded to the yielding of the tension nonprestressed reinforcement. The control composite post-tensioned concrete beams B1-F20-R, B2-F30-R and B3-F40-R achieved failure load of 150 kN, 155 kN, and 160 kN and a maximum increment of midspan deflection at the failure of 26 mm, 26 mm, and 25 mm, respectively, regardless of the residual camber from the previous fire test stage (i.e., the net maximum midspan displacement from that position reached after fire test was 24.9 mm, 24.9 mm, and 23.9 mm, respectively). It is important to note that the increase of the compressive strength of the concrete flange insignificantly affected the load-carrying capacity and the maximum midspan deflection at failure due to the fact that the failure of the control beams attained due to the yielding of the main nonprestressed steel, (i.e., typical

tension-controlled failure mode), followed by the crushing of concrete in the compression zone.

The behavior of composite post-tensioned concrete beams depends on the level of damage to concrete caused by elevated heating temperatures. The heat-deteriorated specimens were able to support the imposed load for an extended time. It is apparent from Table 7 that the load-carrying capacity of the heat-damaged beams reduced after a one-hour of steady-state exposure to 700 and 800 °C by (36.7–45.2%) and (48.4–53.7%), respectively. This reduction is related to the degradation of the most important mechanical properties of concrete and steel, see Tables 2–5 and Fig. 4. It is apparent that at 800 °C, the concrete experienced more defects than at 700 °C that led to more degradation of the stiffness and strength of the test beams. This effect was more pronounced on the post-fire residual strength than the flexural stiffness and ductility.

Figures 8 and 9 demonstrate the recorded load–deflection curves for all test beams of Groups I, II, and III

Fig. 8 Load-midspan deflection diagram for all test specimens

throughout the application of the external load. The exposure of test specimens to elevated temperature deteriorated their post-fire stiffness, ductility, and load-carrying capacity. In contrast to control beams, there was no obvious cracking load in all fire-damaged specimens due to the presence of pre-existing cracks from fire damage and there was no initial linear portion in the load–deflection response (i.e., the entire behavior was nonlinear). The load-midspan deflection diagrams are characterized by two stages of response. The first stage was initiated with reduced flexural stiffness and cracks tended to reopen with increasing applied load. As the applied load exceed the yielding of the tension nonprestressed or prestressed steel, the second stage of performance was started with significant degradation of the flexural stiffness as the test specimens experienced excessive deflection with a slight increase in the applied load.

However the applied superimposed dead load had a limited magnitude of 1.1 kN/m, its presence during the fire test only deteriorated the post-fire performance of the test specimens during the application of the external load. This is due to the fact that the applied superimposed dead load promoted the creep deflection during the fire test and consequently it had a detrimental effect

on the post-fire stiffness and ductility. Accordingly, the increase of the midspan deflection of specimens B2-F30-700U and B3-F40-700U in comparison to specimens B2-F30-700 and B3-F40-700 achieved 21.5 and 5.3%, respectively. The main parameter due to which this difference is attributed is the compressive strength of the concrete flange. Thus for beams exposed to the same elevated temperature, as the concrete compressive strength of the top flange increased the difference of the midspan deflection at the ultimate stage between the loaded and unloaded by superimposed dead load specimens decreased.

Figure 9 illustrates the effect of the flange concrete compressive strength on the behavior of test specimens under the applied loading. Obviously, at the same heating temperature and loading circumstance, increasing the compressive strength of the concrete flange led to slightly increasing the midspan deflection at failure. This observation attributes to the increased deteriorations that occur in the concrete of the top flange with higher compressive strengths during fire exposure that is related mainly to the losses in the mechanical properties of concrete.

Fig. 9 Effect of flange concrete compressive strength on the behavior of test specimens under the applied load

Table 7 summarizes the flexural response of all test composite beams including the cracking load, yielding load, failure load, and their corresponding, also, the ductility index.

The ductility index is defined as the ratio between the deflection at the ultimate load to the deflection at the yielding load.

For the specimen of Group I (B1-F20-700), the residual yielding and failure loads were 55 and 63.3% of the control beam (B1-F20-R), respectively. In Group II, without the superimposed dead load during the fire test, the residual yielding and maximum loads for the specimen (B2-F30-700) were 55 and 59.4%, respectively, of the control specimen (B2-F30-R). With the presence of superimposed dead load during a fire event, specimens (B2-F30-700U) and (B2-F30-800U) in comparison to their control beam experienced residual yielding and ultimate loads of (50 and 54.8%) and (50 and 51.6%), respectively.

For the specimen of Group III (B3-F40-700) which was tested without the presence of the superimposed dead load during fire exposure, the residual yielding and the failure loads were 55 and 56.3% of the control beam (B3-F40-R), respectively. While specimens (B2-F30-700U)

and (B2-F30-800U), in comparison to the reference beam (B3-F40-R), achieved residual yielding and maximum loads of (50 and 50.6%) and (50 and 46.3%), respectively.

In comparison to the unheated reference beams, a significant increase in the ductility index was observed in all test specimens which were exposed to an elevated temperature of 700 °C. Meanwhile, only a slight increase in the ductility index was recorded in specimens which were exposed to 800 °C.

4.5 Progress of concrete strain during the static loading stage

Figure 10 elucidates the load-strain response of the extreme top fibers of concrete in the compression zone at the midspan section for all groups of test specimens. The effect of elevated temperatures on the progress of the concrete strain was highly pronounced. The increase in concrete strain in the failure stage is a function of the level of the heated temperature and in turn the level of degradation of the stiffness of the test beam and the residual strength of the concrete and steel. Unlike the concrete strain of the unheated specimens B1-F20-R, B2-F30-R, and

Table 7 Post-fire test results during static loading application

Specimen ID	Residual camber from previous stage, (mm)	Cracking load, (kN)	Deflection at cracking load, (mm)	Yield load, (kN)	Deflection at yield load, (mm)	Failure load, (kN)	Deflection at failure load, (mm)	Ductility index column (8)/column (6)	Residual load capacity, (%)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Group I									
B1-F20-R	-1.1	60	+1.25	100	+6.4	150	+24.9	3.9	100
B1-F20-700	-3.3	-	-	55	+3.7	95	+24.7	6.7	63.3
Group II									
B2-F30-R	-1.1	60	+1.2	100	+5.9	155	+24.9	4.2	100
B2-F30-700	-3.4	-	-	55	+3.1	92	+24.6	7.9	59.4
B2-F30-700U	-3.1	-	-	50	+3.9	85	+29.9	7.7	54.8
B2-F30-800U	-3.2	-	-	50	+7.3	80	+28.8	3.9	51.6
Group III									
B3-F40-R	-1.1	60	+1.1	100	+5.4	160	+23.9	4.4	100
B3-F40-700	-3.6	-	-	55	+2.4	90	+26.4	11.0	56.3
B3-F40-700U	-3.2	-	-	50	+3.1	81	+27.8	9.0	50.6
B3-F40-800U	-3.3	-	-	50	+6.7	74	+30.7	4.6	46.3

B3-F40-R, the progress of the post-fire strain in concrete increased at a higher rate up to failure. It was due to the degradation of the stiffness, resulted from the breakdown of aggregate cement bond and the formation of a network of minor surface cracks at elevated temperatures, the formation of flexure cracks in the pure bending moment span, and consequently the flexure-shear cracks in the shear spans. It was apparent that as the exposure temperature increased the post-fire strain increased at a higher rate with the progress of the load. Accordingly, composite beams B2-F30-800U and B3-F40-800U experienced the highest rate increase of the concrete strain in comparison to other test specimens. Unlike the concrete strain of the unheated beams, the concrete strain at the midspan section of all specimens exposed to elevated temperatures did not show a significant increase at failure load. However, the failure strain was not very significantly different for various concrete types in the flange of the cross-section. It was due to the fact that the failure in all heated beams induced with the commencement of the rupture of the tension steel. The test specimens after fire exposure of 700 and 800 °C attained an average maximum concrete compressive strains at midspan section of 1500 microstrain in B1-F20-700, while in specimens of Group II reached 880, 950, and 980 microstrains in B2-F30-700, B2-F30-700U, and B2-F30-800U, respectively, using the foil strain gauges (S_5) on the top surface of concrete, see Fig. 3. Meanwhile, specimens of group III achieved peak concrete compressive strain of 950, 1000, and 1000 microstrains for beams B3-F40-700, B3-F40-700U, and B3-F40-800U, respectively.

In comparison to the heat-damaged beams, the performance of the benchmark beams was significantly different. At failure, the extreme top fibers of concrete in the compression zone at the midspan section attained 3300, 3100, and 3100 microstrains in B1-F20-R, B2-F30-R, and B3-F40-R, respectively. However, these strains at failure were not very significantly different for different types of concrete in the flange of the cross-section.

Figure 11 illustrates the effect of the flange concrete compressive strength on the increment of flexural strain at extreme top concrete fibers. It is clear that the concrete compressive strength of the flange did not show a significant effect on the compressive concrete peak strain at failure. However, three observations could be mentioned, mainly, as the compressive strength of the concrete flange increased the failure strain in concrete in compression decreased, also, as the level of elevated temperature increased the post-fire strain in concrete in compression increased, and a minor effect was pronounced on the concrete failure strain related to the presence of the superimposed dead load during the exposure of the test specimens to the fire test.

Fig. 10 Load-increment of compressive strain at extreme top concrete fibers for all test specimens

Based on the above-mentioned observations, obviously, the residual strength of the main tension-steel played a major role in limiting the load-carrying capacity of the heat-damaged beams. Therefore, by right, unlike the concrete in compression flange, the longitudinal steel reinforcement in the tension zone is considered a key parameter that determines the length of the exploitation period of the composite post-tensioned concrete members subjected to fire exposure and consequently to static loading.

4.6 Failure mode and cracking pattern during the static loading stage

It was observed that a typical ductile flexural failure was characterized by the behavior of the reference beams at ultimate. However, sudden failure occurred under the applied static load in heat-damaged composite post-tensioned concrete beams when exposed to the fire of 700 and 800 °C.

In all test specimens, flexural cracks initiated occurring, increased, and propagated in the pure bending region with the progressed applied load when the bending tensile stress exceeded the concrete tensile strength. Almost, flexural cracks were predominant and widespread in the

post-tensioned beam with a larger spacing of distribution and significant widths of crack opening.

It is worth to mention that the yielding of the tension steel reinforcement was pronounced first before attaining the maximum applied load.

The typical failure mode of the benchmark beams B1-F20-R, B2-F30-R, and B3-F40-R was the yielding of the longitudinal nonprestressed steel in the tension zone, followed by the crushing of concrete in the compression zone due to the migration of a considerable-width flexural crack toward the top concrete fibers, as shown in Fig. 12.

For composite post-tensioned concrete beams exposed to 700 or 800 °C before experiencing the applied static load, the cracking was restricted throughout the region of pure bending moment. However, only limited flexural-shear cracks were observed throughout the two shear spans at high levels of loading. In comparison to reference beams, the number of the monitored flexural-shear cracks in the heat-damaged beams was almost less. As the applied load progressed, cracks in the flexural zone appeared with larger widths compared to those in the unheated beams.

The typical failure mode of the beams heated to 700 or 800 °C was the sudden collapse due to two effects; mainly, the rupture of the tension steel (the nonprestressed steel

Fig. 11 Effect of flange concrete compressive strength on the increment of flexural strain at extreme top concrete fibers

or/and the prestressing strand) and the degradation of the concrete mechanical properties at these temperatures. The degradation of the strength of concrete can be attributed to the excessive thermal stresses generated during fire exposure and the physical and chemical reforming in the concrete microstructure [9], see Fig. 12.

Accordingly, the failure of the beams B1-F20-700, B2-F30-700, and B2-F30-700U was the crushing of the concrete in compression, followed by the rupture of the longitudinal nonprestressed steel in the tension zone. While, the failure of beams B3-F40-700 was the crushing of the concrete in compression, followed by the rupture of the prestressed steel in the tension zone. In specimens B3-F40-700U, B2-F30-800U, and B3-F40-800U the failure started with the rupture of both the prestressed and nonprestressed steel in the tension zone, followed by the crushing of concrete in the compression zone.

It is well known that the transfer of internal forces across the interface between the tension steel and the surrounding concrete through bond plays a very important role in the performance during serviceability and in the resistance during the ultimate stage of the structural concrete member. Meantime, the fire exposure weakens the bond strength between the gravel particles and the surrounding cement paste. Accordingly, the weakness of the mentioned bond led to split up the concrete cover

during the exposure of all the heat-damaged test beams to the applied static loading. Therefore, as the applied load approached the failure capacity, concrete cover separation started at the midspan section and propagated toward the ends of the pure bending span.

5 Finite element analysis of test beams after fire exposure

All composite post-tensioned concrete beams in this investigation were modeled using ABAQUS V6.13/2016 software to shed further light on their behavior and residual strength after fire exposure. To achieve this purpose, a nonlinear finite element analysis was carried out to replicate the experimental outcomes and to obtain a better understanding of the associated post-fire performance of heat-damaged composite concrete beams.

The finite element modeling was started with the modeling of the benchmark beams (unheated) which enabled the development of the heat-damaged beam model later. Therefore, calibrated FE models were implemented to simulate the actual specimen components of a post-tensioned concrete member, prestressing strand, shear connector bars, reinforced concrete deck slab, and

Fig. 12 Crack and failure patterns of composite post-tensioned concrete T-beams

the steel plate used for restraining the beam in the force control test frame.

The concrete material was modeled using a damage plasticity model that capable to capture the general two failure mechanisms of the concrete; the cracking in tensile and the crushing in compression. The model is assumed that the concrete uniaxial tensile and compressive response is characterized by damaged plasticity. To avoid stress concentration at supports, loading points, and prestressing force transfer points, steel plates were used to obtain the most representative test results and to closely replicate the experimental setup and were modeled as a linear elastic material. For simulation purposes, the concrete of the post-tensioned member and the steel plates were modeled with the three-dimensional C3D8R linear brick element having eight nodes with three degrees of freedom at each node (that is, translations in x, y, and z directions). The S8R6 quadrilateral shell element having eight nodes with six degrees of freedom per node (that are, translations and rotations in x, y, and z directions), was selected to model the reinforced concrete deck slab. It is capable of providing sufficient degrees of freedom to explicitly model deformations, as well as its capability in modeling reinforcement in concrete slab without element distortion which will occur in solid element due to the small relative element thickness with the other two dimensions when modeling RC slab cover.

The nonprestressed steel and stirrups in the post-tensioned concrete member were modeled analytically as elastic-perfectly plastic material. While the steel strand was modeled as elastic linear-strain hardening plastic material. The steel reinforcements were modeled with the T3D2 linear truss element which was a uniaxial tension and compression element having two nodes with three degrees of freedom at each node (that is, translations in x-, y-, and z-directions). While the deck slab reinforcement was modeled as smeared layers (solid rebar layers) with a constant thickness in shell elements.

Figure 13 illustrates the adopted in the numerical model stress–strain relationships for concrete and steel at different temperature exposures.

The steel reinforcing elements (main reinforcement and stirrups) were connected to the concrete beam element by using embedded constraint. Accordingly, the steel reinforcing element is considered as an embedded element while the concrete beam was served as a host element. The response of the host elements is to constrain the translational degrees of freedom of the embedded nodes (i.e., nodes of embedded elements). A surface-to-surface contact was used to simulate the interface between the reinforced concrete deck slab and the post-tensioned concrete member. The steel load

plates surface was connected to the deck slab top surface by using the tie constraint method. The load plate surface is designed as a master surface and the deck slab top surface is used as a slave element.

Most of the material model input parameters, such as concrete compressive and tensile strengths, yield and tensile strengths of steel, their modulus of elasticity were obtained from the experimental data Tables 2–5, Fig. 13. All boundary conditions used to the simulated models follow those used in the actual experimental test configuration. Figure 14 illustrates the finite element modeling for the test specimens.

To perform the static analysis of the heat-damaged specimens, it should transfer the material mechanical properties from ambient temperature to elevated

temperatures of 700 or 800 °C, because of the difference in the material properties of steel reinforcement and concrete at the two different temperatures, see Tables 2–5. Table 8 elucidates the post-fire residual strength for all test specimens.

It is evident from Table 8 that the maximum load obtained experimentally for the control beams are lower than those determined with the nonlinear finite element analysis by 10%, 9%, and 8% for B1-F20-R, B2-F30-R, and B3-F40-R, respectively. This discrepancy may attribute to the adopted assumptions, mainly, the perfect-bond assumption between the concrete and the nonprestressed steel, and the no-slip assumption between the post-tensioned concrete member and the deck slab.

Fig. 13 Material modeling adopted in the finite element analysis of tested beams

Fig. 14 Modeling, loading, and boundary conditions for composite post-tensioned concrete T-beams

Also, microcracks which resulted from the drying shrinkage and curing in concrete were not simulated and not considered by the finite element analysis. Therefore, the overall stiffness of the test specimen can be lower than that expected by the finite element analysis [21].

For specimens exposed to 700 or 800 °C, it is clear that the load-carrying capacity obtained from the finite

element analysis is higher than those recorded from the experiment in the range (2.2–17.3%). Obviously, there is an acceptable agreement between the results of the load-carrying capacity being predicted with reasonable accuracy, where the average value of the ratio of the predicted maximum load, according to the finite analysis, to the

Table 8 Experimental and predicted ultimate load capacity and mode of failure of composite beams

Specimen ID		Exp. maximum load $P_{u,exp}$ (kN)	Finite element analysis		ACI 318-19		Failure mode
			Maximum load $P_{u,est}$ (kN)	$\frac{P_{u,est}}{P_{u,exp}}$	Maximum load $P_{u,est}$ (kN)	$\frac{P_{u,est}}{P_{u,exp}}$	
Group I	B1-F20-R	150	165.00	1.100	136.55	0.910	Yielding and concrete crushing
	B1-F20-700	95	93.00	0.979	77.26	0.813	Concrete crushing then rupture of reinforcing steel bar
Group II	B2-F30-R	155	169.00	1.090	140.00	0.903	Yielding and concrete crushing
	B2-F30-700	92	94.00	1.022	78.58	0.854	Concrete crushing then rupture of reinforcing steel bar
	B2-F30-700U	85	94.00	1.106	78.58	0.924	Concrete crushing then rupture of reinforcing steel bar
	B2-F30-800U	80	85.00	1.063	66.04	0.826	Rupture of prestressing strand and reinforcing steel bar then crushing of concrete
Group III	B3-F40-R	160	173.00	1.081	141.2	0.883	Yielding and concrete crushing
	B3-F40-700	90	95.00	1.056	79.86	0.887	Concrete crushing then rupture of prestressing strand
	B3-F40-700U	81	95.00	1.173	79.86	0.986	Rupture of prestressing strand and reinforcing steel bar then crushing of concrete
	B3-F40-800U	74	86.00	1.162	67.84	0.917	Rupture of prestressing strand and reinforcing steel bar then crushing of concrete
Average of ($P_{u,est}/P_{u,exp}$)				1.083		0.890	
Standard of deviation (σ)				0.059		0.051	
Coefficient of variation (CV)				0.054		0.057	

experimental result was 1.083 with a standard deviation of 0.059 and a coefficient of variation of 0.054, see Table 8.

Figure 15 illustrates the ABAQUS finite element modeling results at ultimate for the test specimens including the deflection of member, the concrete strain, and the steel strain.

In addition to the nonlinear finite element analysis, the ACI 318 [1] section analysis method was adopted for assessing the post-fire flexural carrying capacity of composite post-tensioned concrete sections. Reduced mechanical properties of concrete and steel reinforcement were considered depending on the data available from testing of these materials under elevated temperatures, see Tables 2–5. It was assumed that there is no slip between steel and concrete, the concrete in tension is ignored, and the post-fire strength of concrete and steel in tension and compression depends on the peak temperature experienced during fire exposure. It is evident that the ACI 318 method underestimated the post-fire flexural capacity of composite post-tensioned concrete beams. However, the average value of the predicted by ACI 318 maximum load to the experimental load was 0.890 with a standard deviation of 0.051 and a coefficient of variation of 0.057, see Table 8.

6 Conclusions

This study presents the post-fire behavior and residual strength of composite post-tensioned concrete T-beams. It showed that the higher loss rates of the original mechanical properties, as much as (60–80%), were observed for all types of concrete at temperatures 700–800 °C, also, the presence of the sustained superimposed dead load simultaneously with the exposure to elevated temperatures led to restraining the excessive increase of the midspan camber of the test specimen due to the fact that this load promoted the creep deflection during the fire test. Meanwhile, increasing the concrete compressive strength of the top flange led to slightly increasing the residual midspan camber. This attributes to the increased deteriorations that occur in the concrete of the top flange with higher compressive strengths that caused by the increase in density and the decrease in porosity that affects the thermal conductivity of the concrete. The exposure of test specimens to elevated temperature affected their post-fire stiffness, ductility, and load capacity.

It was found that the residual strength of the main tension-steel played a major role in limiting the load-carrying capacity of the heat-damaged beams. Therefore, the longitudinal steel reinforcement in the tension zone is considered a key parameter that determines the length of the exploitation period of the composite post-tensioned concrete members subjected to fire exposure and consequently to static loading. The load-carrying capacity of

Fig. 15 Finite element analysis results

the heat-damaged beams reduced after a one-hour of steady-state exposure to 700 and 800 °C by (36.7–45.2%) and (48.4–53.8%), respectively.

Meanwhile, for specimens exposed to 700 or 800 °C, the load-carrying capacity obtained from the finite element analysis was higher than those recorded from the experiment in the range (2.2–17.3%), where the average value of the ratio of the predicted maximum load to the experimental result was 1.083 with a standard deviation of 0.059 and a coefficient of variation of 0.054. While for the same specimens, the load-carrying capacity determined according to ACI 318 [1] section analysis method was lower than those monitored from the experiment in the range (1.4–18.7%), where the average value of the ratio of the predicted maximum load to the experimental result was 0.890 with a standard deviation of 0.051 and a coefficient of variation of 0.057.

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Compliance with ethical standards

Conflict of interest The authors declare that there are no potential conflicts of interest with respect to research, authorship, and publication of this article.

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