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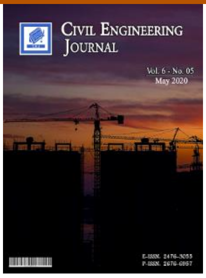

Post-Fire Behavior of Post-Tensioned Segmental Concrete Beams under Monotonic Static Loading

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Abstract

This paper presents a study to investigate the behavior of post-tensioned segmental concrete beams that exposed to high-temperature. The experimental program included fabricating and testing twelve simply supported beams that divided into three groups depending on the number of precasting concrete segments. All specimens were prepared with an identical length of 3150 mm and differed in the number of the incorporated segments of the beam (9, 7, or 5 segments). To simulate the genuine fire disasters, nine out of twelve beams were exposed to a high-temperature flame for one hour. Based on the standard fire curve (ASTM – E119), the temperatures of 300°C (572°F), 500°C (932 °F), and 700°C (1292°F) were adopted. Consequently, the beams that exposed to be cool gradually under the ambient laboratory condition, after that, the beams were loaded till failure to investigate the influence of the heating temperature on the performance during the serviceability and the failure stage. It was observed that, as the temperature increased in the internal layers of concrete, the camber of tested beams increased significantly and attained its peak value at the end of the time interval of the stabilization of the heating temperature. This can be attributed to the extra time that was consumed for the heat energy to migrate across the cross-section and to travel along the span of the beam and deteriorate the texture of the concrete causing microcracking with a larger surface area. Experimental findings showed that the load-carrying capacity of the test specimen, with the same number of incorporated concrete segments, was significantly decreased as the heating temperature increased during the fire event.

Keywords: Segmental Beam; Post-tensioning; Fire Test; Gradual Cooling; Serviceability; Load Capacity.

1. Introduction

Post-tensioned segmental concrete girders have a significant implementation in bridge engineering due to the facilities that offered during the construction process. This method of construction has many advantages such as substantial economical savings due to the possibility of weather-independent segment production and a shorter construction period, simple element assembly at the job site, replacement ability of deteriorated tendons, the concreting and prestressing operations are independent, small light segments, profiling of the main external steel is easier to check, and the friction may be reduced [1]. It is well known that the strength of reinforced concrete and prestressed concrete members decreases after the exposure to a fire disaster. The main fire safety objectives are to protect life and prevent failure. Following a fire, if no collapse happens, there is a possibility of fire-induced damage. It should be noted that the study of the heating history of concrete is very significant to define whether the concrete structure exposed to fire and its components remain intact from the structural aspect. The evaluation of concrete

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structures for fire damage commonly starts with the visual inspection of cracking, discoloration, spalling of concrete, and consequently, determining the residual strength of concrete. The performance of ordinary reinforced concrete and prestressed concrete members exposed to the fire attack was studied by many researchers [2-8]. Chan et al. [2] studied experimentally the pore structure and their distribution in addition to the mechanical properties of the normal-strength (NSC) and high-performance concrete (HPC) after the exposure to a high temperature. The residual compressive strength of the concrete specimens was examined after subjecting concrete to a temperature of 800 °C. It was concluded in this study that after exposure to high temperature, the degradation in strength of HPC was more severe than in NSC.

Phan and Carino [9] presented the data for behavioral variances between the normal-strength concrete (NSC) and the high-strength concrete (HSC) at high temperatures. Also, they reported the material behavior after fire exposure, the code provisions for members that subjected to fire attack, and the analytical modeling of HSC at high temperature. Expanded vermiculite is a significant lightweight aggregate for cementitious materials that are used for fire resistance applications. Koksal et al. [3] examined four different compound mixtures under high temperatures of 300, 600, 900, and 1100 °C for 6 hours in varying amounts of expanded vermiculite. They studied the mechanical and physical properties of concrete including its unit weight, porosity, water absorption, the residual compressive and splitting tensile strengths, and the ultrasonic pulse velocity after fire exposure and consequently air cooling. Zhang et al. [10] demonstrated that the mechanical characteristics of prestressing steel during high temperatures and after cooling are requested for the assessment of the resistance to fire and the residual post-fire load-carrying capacity for the prestressed concrete structures. In that study, mechanical properties of prestressing wires through and after the exposure to the fire were examined by sophisticated equipment to upgrade the reliability and accuracy of the material characteristics database at high temperatures of performance-based design objectives. Accordingly, empirical formulas were proposed for residual strength. Myers and Bailey [5] examined the residual characteristics of uncoated seven-wire, 12.7 mm and 9.5 mm diameters low-relaxation grade 270 (1862 MPa) prestressing strands under the excessive temperature of 260, 427, 538, 649, and 704 °C. The results of that research indicated that there was a loss of tensile strength of the strand could attain 26.0% when the temperature rose from 538 to 649 °C.

Abdelrahman et al. [6] tested a series of statically determinate prestressed concrete beams under fire to investigate the effect of different parameters on the behavior of such structural members including the prestressed index, the concrete compressive strength, and the thickness of the concrete cover. Five specimens were tested in lab conditions while seven beams were loaded up to its working load and exposed to fire for three hours at 600 °C and left to cool gradually at the ambient temperature then tested up to failure. Izzet and Al-Dulffy [8] investigated the effect of fire flame and the rate of loading on the service behavior of partially prestressed concrete beams and the residual strength. Seven pretensioned concrete beams have been fabricated and tested. One beam was considered as a reference beam that tested under static loading without fire exposure. Meanwhile, the other six were exposed initially to fire test and consequently to monotonic static loading, where each pair were subjected to the same heating temperatures of 300, 500 or 700 ^{O}C but cooled in different scenario (i.e., gradually or suddenly). It was concluded that the sudden cooling in comparison to the gradual cooling had a worse impact on the residual load-carrying capacity of the tested specimens. To date, there are several experimental programs that were carried out to investigate the behavior of segmental concrete beams under external load. Sivaleepunth et al. [11] and Nguyen et al. [12] presented the results of the nonlinear finite element analysis on the segmental concrete beams with external tendons. Algorafi et al. [13] investigated the structural behavior of dry joined externally prestressed segmental beams (EPS) under combined stresses (bending, shear and torsional stresses). It was found that the reduction in the load-carrying capacity of such beams can be compensated by the implementation of a higher prestressing force as well as by increasing the stirrup reinforcement area in the joint regions. To ensure serviceability, the joints in the support regions should still be sufficiently prestressed under service load conditions. Yuan et al. [14] studied experimentally the performance of segmental concrete box beams with hybrid tendons. Three scaled-down specimens with different ratios of the number of internal tendons to the number of external tendons were tested up to failure. Test results showed that as more internal tendons were used, higher load-carrying capacity and better ductility were achieved. Therefore, the ratio of these hybrid tendons not less than 1:1 was recommended.

Moubarak et al. [15] investigated the second-order effect in 25 externally prestressed monolithic and segmental concrete beams. The main parameters of this study were the shear span to depth ratio, the effective prestressing level, and the profile of external strands. A suggestion for a solution that improves the system efficiency of the segmental beam with external strands was presented. Thorough analysis based on the mechanics of load transfer, fracture and damage mechanics were proposed. Jiang et al. [16] conducted a series of tests to investigate the effect of using hybrid tendons, load location, and the number of joints on the flexural behavior of post-tensioned segmental concrete beams (PSC) with dry joints. It was noticed that the flexural strength of fully segmental beams with hybrid tendons was 30% less than that of the monolithic beam with hybrid tendons. Due to a high concentration of rotation and deflection at individual joints, the flexural strength of the partially prestressed segmental beam with hybrid tendons was 12.8% less than that of the fully prestressed segmental beam with hybrid tendons.

Despite this interest, no one to the best of our knowledge has studied the post-fire performance of internally post-tensioned segmental concrete beams under monotonic static loading. Accordingly, this paper seeks to address the behavior of such structural concrete beams in an attempt to investigate the effect of the length of the individual segment, the number of joints between individual segments, and the heating temperature to which the member is exposed to on the behavioral performance at serviceability and failure stages.

The structure of this article was designed in such a way to achieve the objectives of the research program that starting with the experimental program, measurement of the important deformability aspects, comparisons and interpretations analysis of experimental results, and highlighting the important conclusions.

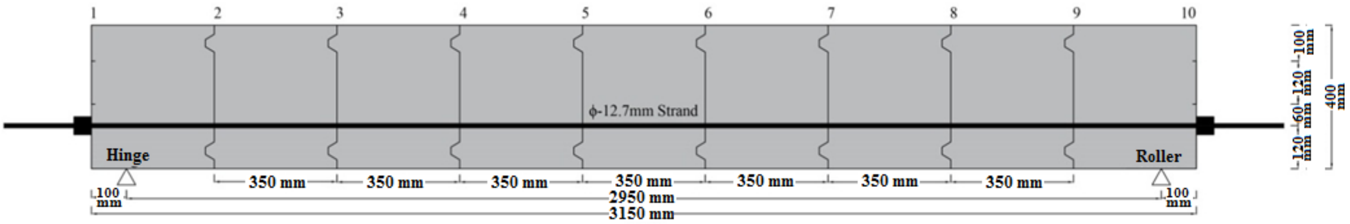
2. Experimental Program

The experimental program consisted of twelve simply-supported post-tensioned segmental concrete beams. All segmental beams were categorized into three groups depending on the number of the incorporated precast concrete segments. All the PSC beams were designed and fabricated with a square cross-sectional configuration of 400 x 400 mm dimensions and 3150 mm overall length. In the first group, the PSC beams consisted of nine segments each of 350 mm length. While the second and third groups included seven and five segments each of 450 mm and 630 mm, respectively.

Figure 1 shows the schematic configuration of the tested PSC beams while Figure 2 illustrates the details of the two ends of the precast concrete segments in their opposite contact surfaces. Four concrete bulges, which play the role of shear keys, and four cavities were fabricated at first and the second end of each segment, respectively. To achieve perfect accommodation for the concrete shear key at the interface section, the dimensions of the cavity and the bulge were adopted identical. The steel cages in each segment included longitudinal and transverse reinforcement of 8 mm diameter deformed bars with yielding and ultimate strengths of 486 and 640 MPa, respectively (Figure 3). Twelve Ø 8 mm longitudinal bars and Ø 8 @ 60 mm c/c steel stirrups were used in each segment. The fabrication of the PSC beams performed in two stages. In the first stage, all the precast concrete segments were cast in special metal forms and continuously moist cured by wet burlap for seven days to achieve a concrete compressive strength of 40 MPa at 28-day for cube samples of 150 x 150 x 150 mm dimensions. Figure 4 shows the grading curves for the fine and coarse aggregate that tested according to the ASTM C33-18 [17]. Consequently, in the second stage, the concrete segments were assembled together according to the assigned groups using one eccentric prestressing force of 120 kN. To create this force, one 12.7 mm diameter low-relaxation seven-wire steel strand (Grade 270, i.e., 1860 MPa) was used through a plastic duct that had been fixed before casting with the steel cage. The distance from the center of the steel strand to the soffit of the beam was considered to be equal to 120 mm in all test specimens. The jacking force was applied from one end, where the adopted value was selected in such a way that to conform to the upper limit recommended by the ACI 318M-14 Code. The test variables included three different structural systems:

- Post-tensioned segmental concrete beams with nine segments (PSC-9);
- Post-tensioned segmental concrete beams with seven segments (PSC-7);
- Post-tensioned segmental concrete beams with five segments (PSC-5).

For each case, three different degrees of heating temperature were selected (300, 500, or 700 °C). Table 1 lists the test variables which have been considered. The abbreviation of each beam takes the format of the post-tensioned segmental concrete beam - the number of precast concrete segments – the heating temperature. Except for the reference beams PSC-9-REF, PSC-7-REF, and PSC-5-REF, the test program was carried out on each PSC beam within two stages, mainly, the first stage during which the experimental beams were exposed directly to fire flame and consequently followed by the second stage during which the beams were subjected to external monotonic static loading up to failure. It is worth to mention that the reference beams were subjected to the monotonic static loading only.



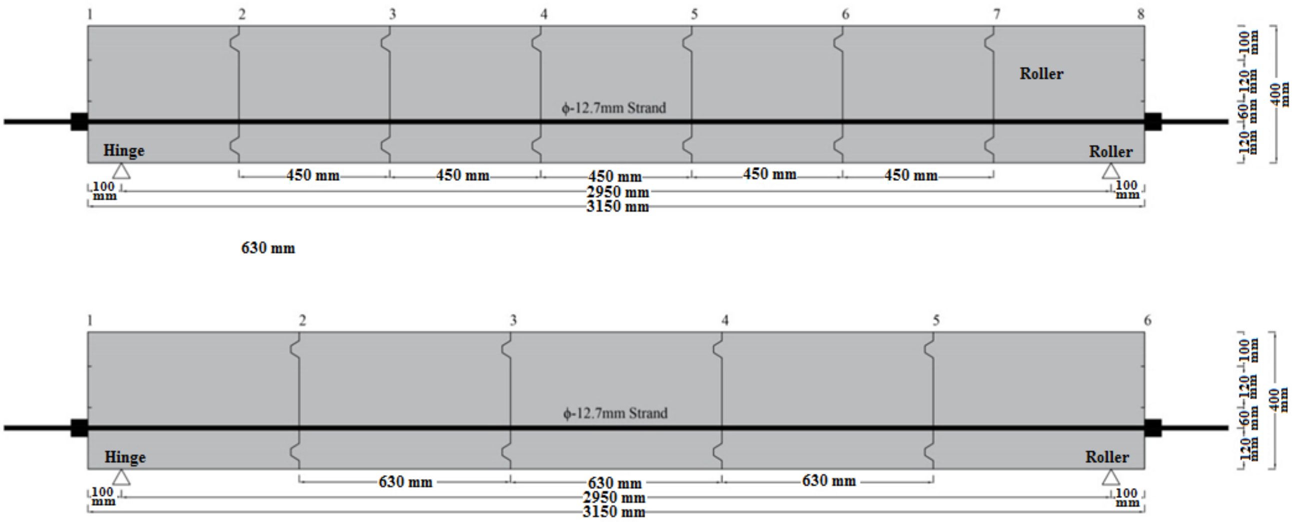
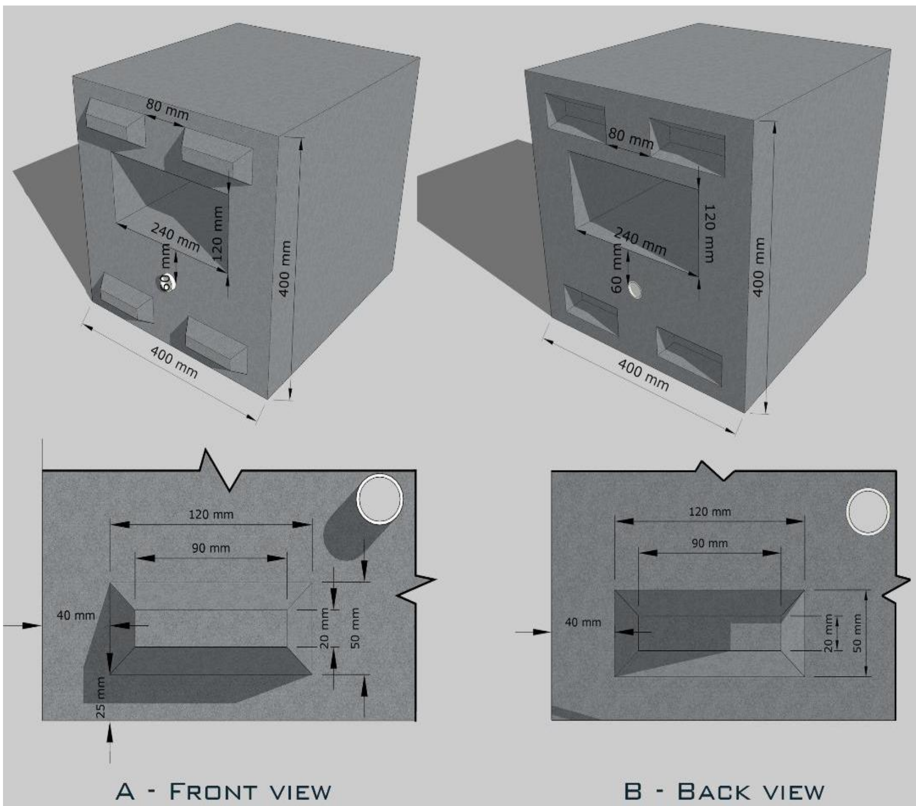


Figure 1. Schematic configuration of the tested PSC beams (All dimensions are in mm)

Table 1. The adopted test variables of the experimental program

Group	Specimen ID	Number of segments	Length of segment, mm	Heating temperature, °C
I	PSC-9-REF	9	350	-
	PSC-9-300			300
	PSC-9-500			500
	PSC-9-700			700
II	PSC-7-REF	7	450	-
	PSC-7-300			300
	PSC-7-500			500
	PSC-7-700			700
III	PSC-5-REF	5	630	-
	PSC-5-300			300
	PSC-5-500			500
	PSC-5-700			700



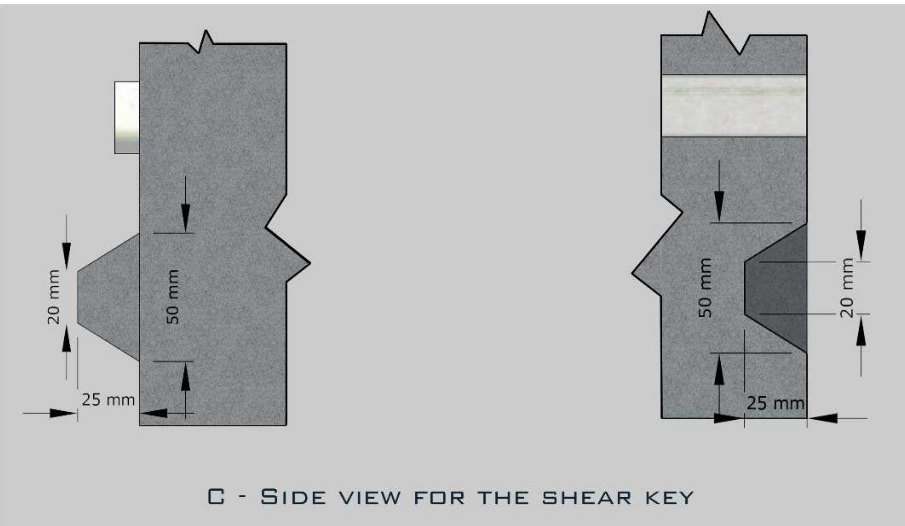


Figure 2. Details of precast concrete segments (All dimensions are in mm)

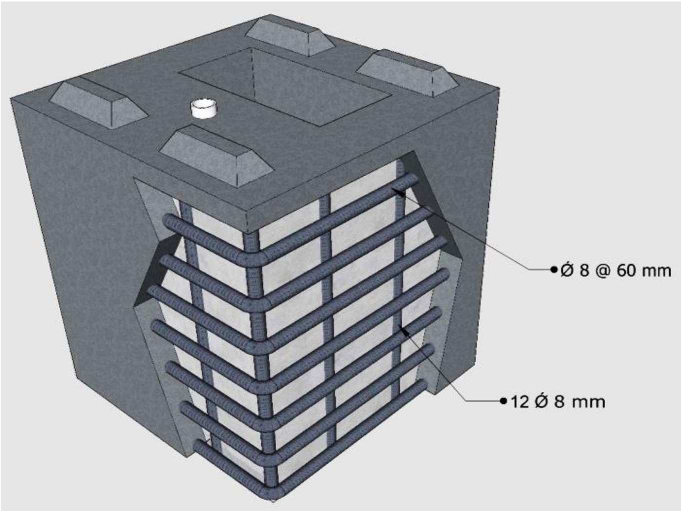


Figure 3. Reinforcement details of precast concrete segments (All dimensions are in mm)

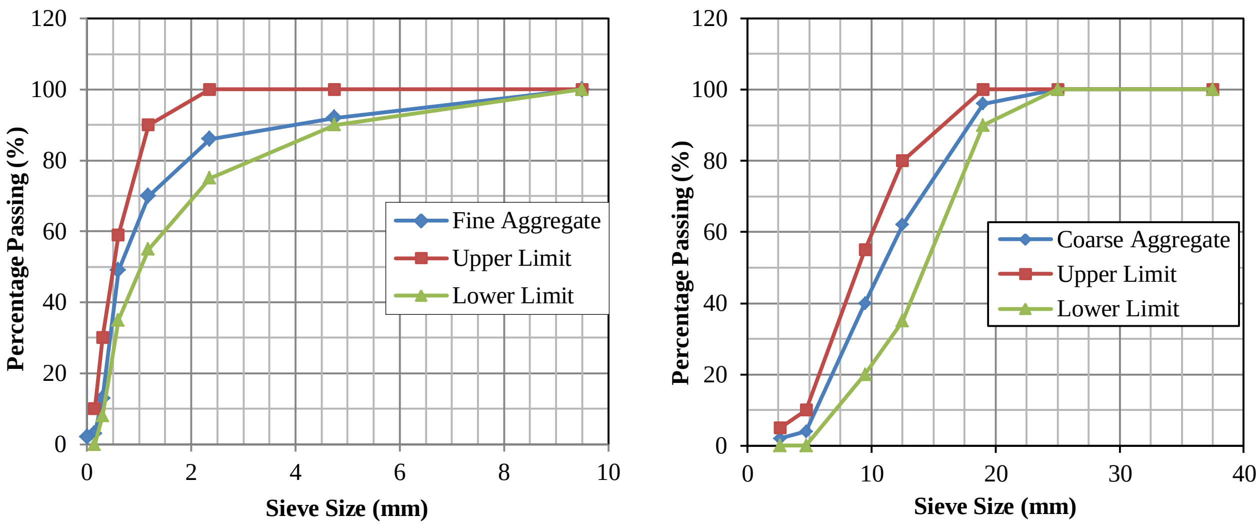
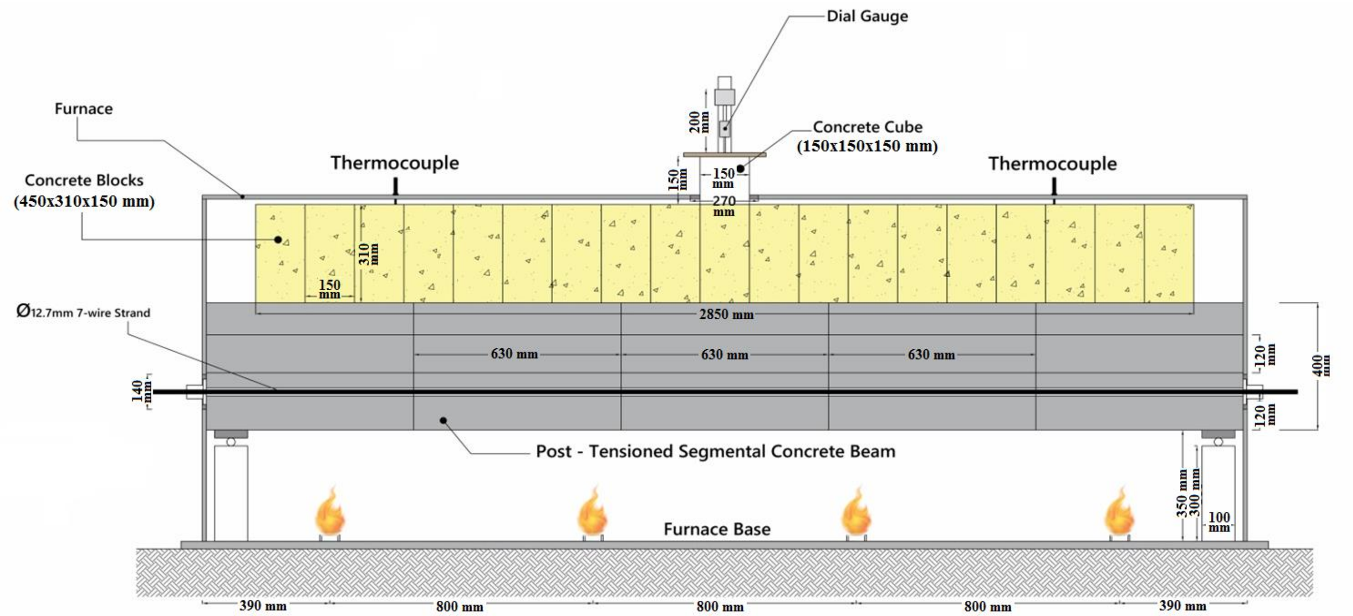


Figure 4. Grading curves for fine and coarse aggregate tested according to the ASTM C33-18 [17]

2.1. Stage I – Fire Test under Direct Flame Exposure

Three PSC beams in each group were exposed to a high temperature of (300, 500, or 700 °C) and uniformly distributed loading of 3.22 kN/m, which simulated the dead load on the tested member, using 19 concrete blocks each of 50 kg. This predetermined superimposed loading was applied before starting the fire test, which comprised a heating and cooling phases. This load was maintained for the entire fire test. It is important to note that the fire chamber was designed in such a way to allow expose the test beams to fire from three sides because a fire on three sides is more common for beams (Figures 5 and 6). The heating temperature was controlled by changing the amount of the supplied methane gas. The temperature readings from three thermocouples, Type K (Nickel-Chromium / Nickel - Alumel) which can be used at temperatures up to 1100°C, was recorded during heating and cooling phases by a digital thermometer reader. Thermocouples were installed in three locations at the center, the right and the left ends of the fire chamber. The required time in minutes to attain the target temperature of 300, 500 and 700°C was 5, 7, and 12, respectively. The heating phase of the fire test was continued for 60 minutes after the target temperature was achieved. After that, the heating phase was terminated and the fire chamber allowed cooling through a cooling phase, which performed gradually in the ambient air condition by removing the top cover of the chamber. It is worth to mention that the heating phase followed the time-temperature curve provided in ASTM-E119 [18]. Accordingly, the test segmental beams were exposed directly to fire flame in the presence of the prestressing force and the sustained superimposed load. During the heating and cooling phases of the fire test, the mid-span section displacement due to the constant dead load, the applied prestressing force, and heating temperature of the fire test was measured using a dial gauge of 0.002 mm/div. sensitivity.

Figure 5. Close-up view of the test under direct fire exposure



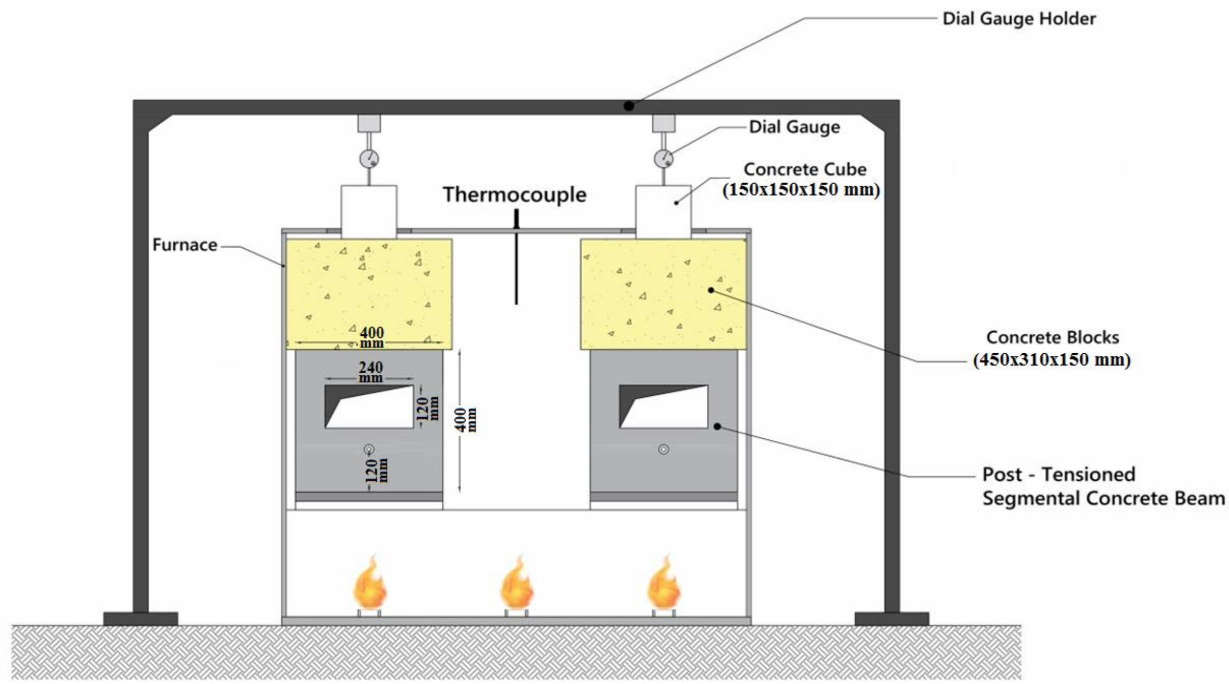


Figure 6. Setup of the fire test under direct flame exposure (All dimensions are in mm)

2.2. Stage II – Test under Monotonic Static Loading

To study the behavior and the residual ultimate strength for PSC beams after the cooling phase of the fire test, all specimens including the reference beams at the ambient temperature were loaded to failure using a force control module with a loading step of 2.5 kN in four-point bending using two symmetrical concentrated loads applied at middle third of the span length (Figure 7).

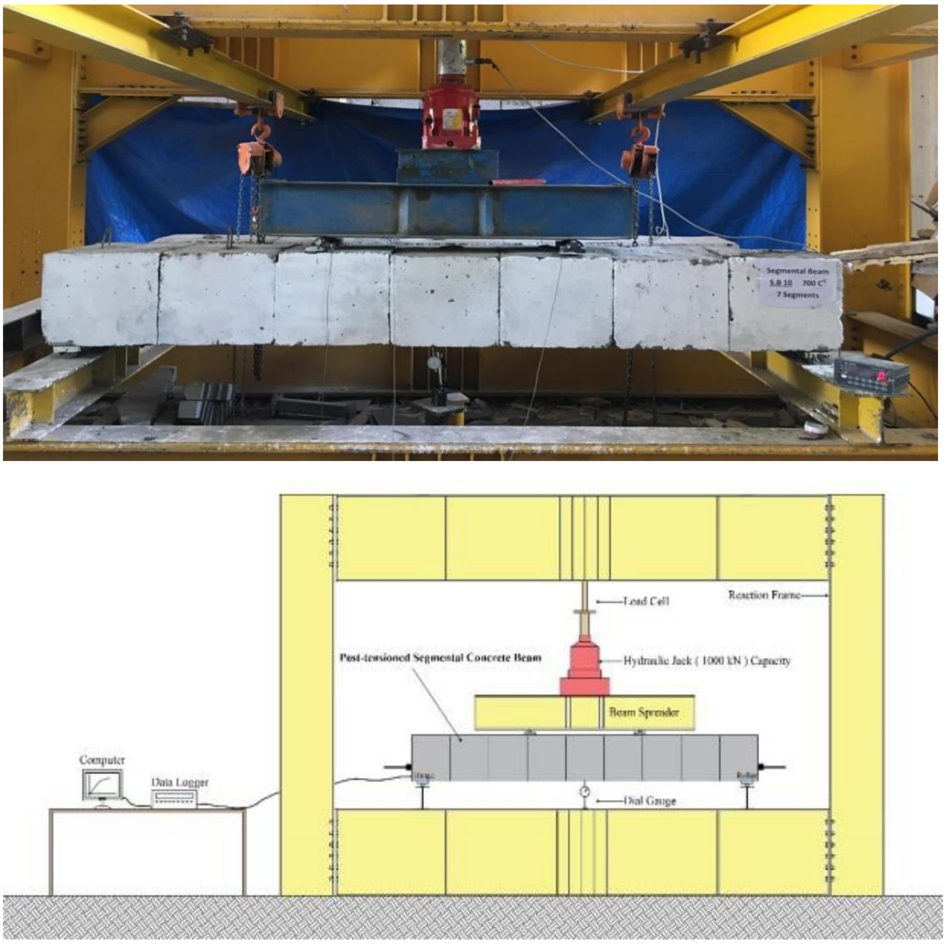


Figure 7. Close-up and schematic view of the test under the monotonic static loading

3. Experimental Results and Discussion

A summary of the test results including the test condition, the applied load at the collapse of the test beam, the camber at the midspan section, the deflection of the midspan section due to the superimposed load, and the failure mode are given for each beam. A detailed discussion for the test scenario is given in the following subsections.

3.1. Displacement of the Test Beams Prior to Fire Test

The combined effect of the applied prestressing force together with the beam self-weight caused upward deflection (camber) at the midspan section before the application of the superimposed dead and the live load (Table 2). The PSC beam's midspan camber was measured by a mechanical dial gauge of 0.002 mm/division. Table 2 shows a significant reduction in the midspan camber with the decreasing of the number of the incorporated segments of the member. It should be noted that as the number of the incorporated segments decreased from nine (i.e., in Group I) to seven (i.e., in Group II) or five (i.e., in Group III), the camber was decreased by 21% or 38%, respectively. The reason behind this evidence is that the eccentric prestressing force which generates prestressing moment that causes more concentration of rotations at the interface section between different segments due to the degradation of the flexural stiffness at these sections. Accordingly, as more segments will be incorporated in the structural member as more concentration of rotations and consequently displacements will occur. It should be noted that prior to exposing the test beams to the fire test, a uniformly distributed load was applied, which simulated the superimposed load of the member itself. Each specimen was load by 19 concrete blocks acting over the compression face of the member sections along its main axis. This applied load sustained acting over the test beam during the first and second stages of the testing program until failure. The measured midspan deflection due to the superimposed load is listed in Table 2. The same observation was recorded for the values of midspan deflection which highly dependent on the number of the incorporated concrete segments in the test beams.

Table 2. Deformability of the tested PSC beams due to prestressing force, self-weight, and superimposed load

	Specimen	Camber due to prestressing force and self-weight			Relative camber value for group δ_i, %	Deflection due to superimposed load, mm	Net camber before fire test Δ_{bf}, mm
		Camber for member Δ, mm	Average camber for group Δ_i, mm		$\delta_i = \left(\frac{\Delta_i}{\Delta_I}\right) \times 100$		
Group I	PSC-9-REF	-2.9	Δ_I	-2.9	100	-	-2.90
	PSC-9-300	-2.8				+0.36	-2.44
	PSC-9-500	-3.1				+0.36	-2.74
	PSC-9-700	-3.0				+0.36	-2.64
Group II	PSC-7-REF	-2.3	Δ_{II}	-2.3	79	-	-2.30
	PSC-7-300	-2.2				+0.27	-1.93
	PSC-7-500	-2.4				+0.27	-2.13
	PSC-7-700	-2.2				+0.27	-1.93
Group III	PSC-5-REF	-1.8	Δ_{III}	-1.8	62	-	-1.80
	PSC-5-300	-1.6				+0.20	-1.40
	PSC-5-500	-1.8				+0.20	-1.60
	PSC-5-700	-1.9				+0.20	-1.70

3.2. Displacement of the Test Beams During the Fire Test

During the heating and cooling phases of the fire test, the midspan section displacement was monitored systematically for all beams. Results during these phases for all beams are summarized in Table 3. Camber versus time history for the PSC beams in Groups I, II and III, respectively are shown in Figures 8 to 10. From these figures, it can be seen that the increasing temperature during the heating phase led to a steep increase in the beam camber. Meanwhile, a gradual decreasing of the achieved camber was observed during the cooling phase. In other words, exposing a concrete beam to high temperature under sustaining eccentric load (i.e., prestressing force) led to a progressive increase of the camber as a result of the degradation of the flexural stiffness of the member. The structural response of the segmental beams during the heating phase of the fire test characterized by two-time intervals. In the first interval (i.e., the progress of heating temperature interval), the increase of the camber values resulted essentially from the generation of thermal strains caused by high thermal gradients. In the second time interval (i.e., the stabilization of the heating temperature interval), as the temperature increased in the internal layers of concrete, camber increased significantly and attained the peak value at the end of this time interval due to the reduction of the thermal concrete strength due to the formation of the internal microcracks. The interference of the thermal effect on different internal layers attributed to the degradation of the strength and the modulus of elasticity of concrete more than that of the steel strand then camber increases at a high pace. In addition, camber increment is essentially attributed to the high creep strains subsequent from high temperatures in concrete which compose of different materials and strands. It is interesting from Figures 8, 9 and 10 to note that during the cooling phase of the fire test, the camber-time diagram starts to descend. The length and the slope of the descending branch depend on the heating temperature that the structural member was experienced and the number of incorporated concrete segments that the member was composed of. The main reasons behind the appearance of the descending branch of the camber-time diagram are the progressive increase of the prestressing losses due to the temperature difference (i.e., the temperature difference between the prestressing steel in the heating zone and the device that receives the prestressing force (anchorages) during concrete heating) and the creep of concrete that highly depend on the value of the heating temperature.

Table 3. Deformability of the PSC beams under fire tests

Specimen	Net camber before fire test Δ_{bf}, mm	Heating phase: Progress of heating temp.		Heating phase: Stabilization of heating temp.		Cooling phase		Net camber after fire test Δ_{af}, mm	Relative residual camber value $((\Delta_{af}-\Delta_{bf})/\Delta_{bf})\times 100$ %	Camber change relative to reference member, %	Camber change relative to nine-segment member, %
		Time period, min	Camber variation, mm	Time period, min	Camber variation, mm	Time period, min	Displacement variation, mm				
PSC-9-REF	-2.90	-	-	-	-	-	-	-2.90	-	-	-
PSC-9-300	-2.44	0-5	-0.2	5-65	-2.0	65-245	+0.9	-3.74	+53	+29	-
PSC-9-500	-2.74	0-7	-0.5	7-67	-3.3	67-380	+1.9	-4.64	+69	+60	-
PSC-9-700	-2.64	0-12	-1.0	12-72	-6.7	72-520	+4.1	-6.24	+136	+115	-
PSC-7-REF	-2.30	-	-	-	-	-	-	-2.30	-	-	-
PSC-7-300	-1.93	0-5	-0.1	5-65	-1.6	65-245	+0.7	-2.93	+52	+27	-22
PSC-7-500	-2.13	0-7	-0.3	7-67	-2.9	67-380	+1.6	-3.73	+75	+62	-20
PSC-7-700	-1.93	0-12	-0.9	12-72	-6.2	72-520	+3.9	-5.13	+166	+123	-18
PSC-5-REF	-1.80	-	-	-	-	-	-	-1.80	-	-	-
PSC-5-300	-1.4	0-5	-0.07	5-65	-1.27	65-245	+0.67	-2.07	+48	+15	-45
PSC-5-500	-1.6	0-7	-0.2	7-67	-2.3	67-380	+1.2	-2.90	+81	+61	-38
PSC-5-700	-1.7	0-12	-0.8	12-72	-5.2	72-520	+3.4	-4.30	+153	+139	-31

In Table 3, the accumulative value for the midspan section displacement including the net camber before the fire test Δ_{bf}, the camber variation during the heating phase and the displacement variation during the cooling phase is called the net camber after the fire test Δ_{af}. The difference between Δ_{af} and Δ_{bf} indicates the size of the residual deformation that the test member was experienced due to the temperature exposure. Depending on the number of the incorporated concrete segments in the test beams and the heating temperature of the fire test, the relative residual midspan camber consisted (48 – 53%), (69 – 81%), and (136 – 166%) for heating temperature of 300, 500, and 700 °C, respectively. Obviously after the heating and cooling phases of the fire test, with the increasing of the heating temperature, the net value for camber was increased in all test beams. At the end of the fire test, the net camber values for beams of Group I that exposed to different heating temperatures were increased by 29, 60, and 115% compared to the net camber value of the reference beam PSC-9-REF for heating temperatures of 300, 500, and 700 °C, respectively. Meanwhile, this increase in the net camber value was attained 27, 62, and 123% for specimens of Group II and 15, 21, and 139% for specimens of Group III for the same mentioned above temperatures compared to their net camber values for reference beams, respectively, PSC-7-REF and PSC-5-REF. From Table 3 and Figures 8 to 10, it should be noticed that the deformability of the test member depends on the heating temperature that the member was exposed to and the length of the time interval of the fire test. The camber was increased during the heating period and reached its maximum value at the end of the stabilization period of the heating temperature. Table 3 and Figures 11 to 13 illustrate the comparison of the camber – time history during the fire test for different test beams depending on the number of the incorporated concrete segments. Obviously at the same heating temperature of the fire test, as the number of the incorporated concrete segments decreased the net camber value at the end of the fire test was decreased. In comparison to the test beams with nine concrete segments, it can be noted that at a heating temperature of 300 °C the net camber value decreased by 22 and 45% for beams with seven and five concrete segments, respectively. However, at 500 °C, the reduction of the net camber value attained 20 and 38% for test beams with seven and five concrete segments, respectively. The same behavior was monitored at the fire test of 700 °C heating temperature, whereas the reduction of the net camber value reached 18 to 31% for the same mentioned above specimens, respectively. This behavior of deformability can be interpreted by the fact that, as the number of dry joints increased the rotational displacement between different concrete segments due to the creep of concrete under the dual effect of prestressing force and fire exposure also increased which in turn achieved higher camber values accordingly. Additionally, the comparisons illustrated in Figures 8 to 10, also in Figures 11 to 13 show that for the same number of segments, as the heating temperature increased the camber of the tested member was increased. Meanwhile for the same heating temperature, as the number of concrete segments increased the camber of the tested specimen was also increased.

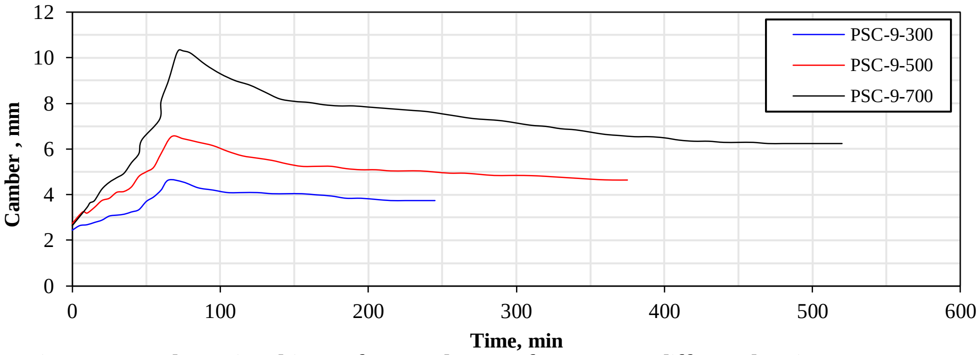


Figure 8. Camber - time history for PSC beams of Group I at different heating temperatures

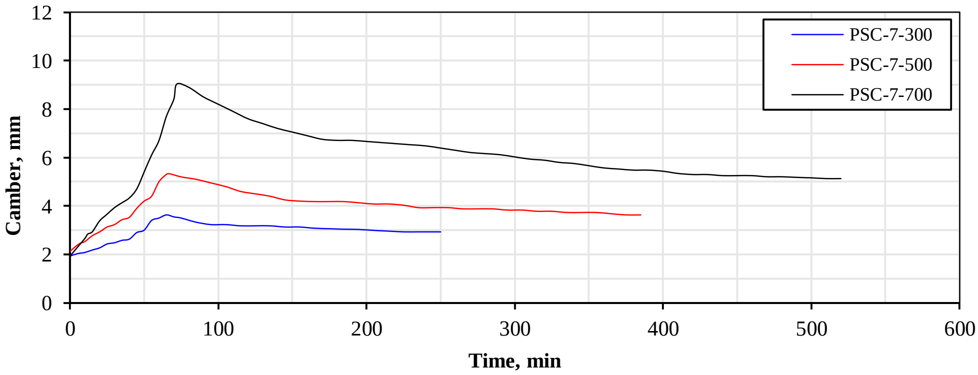


Figure 9. Camber - time history for PSC beams of Group II at different heating temperatures

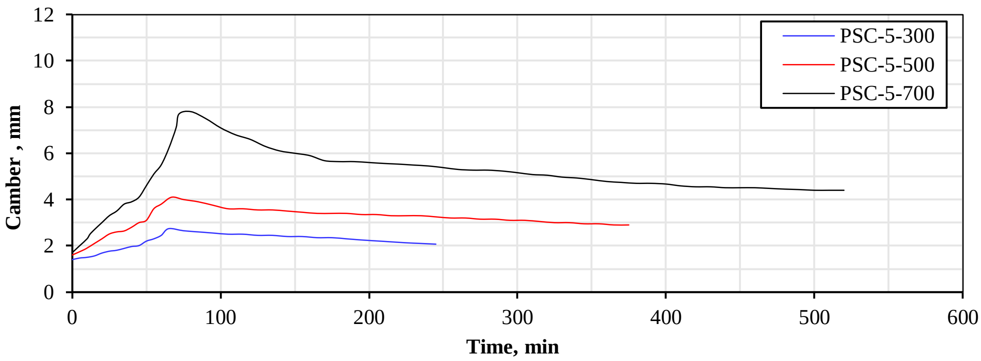


Figure 10. Camber - time history for PSC beams of Group III at different heating temperatures

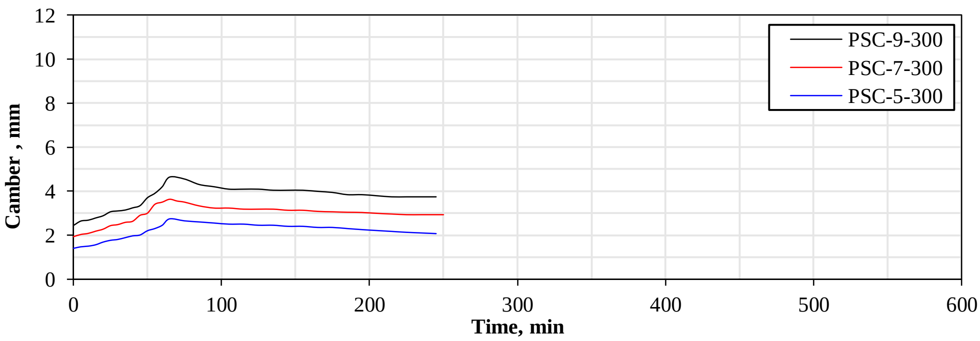


Figure 11. Camber-time history of PSC beams depending on the number of incorporated segments at fire test of 300˚C

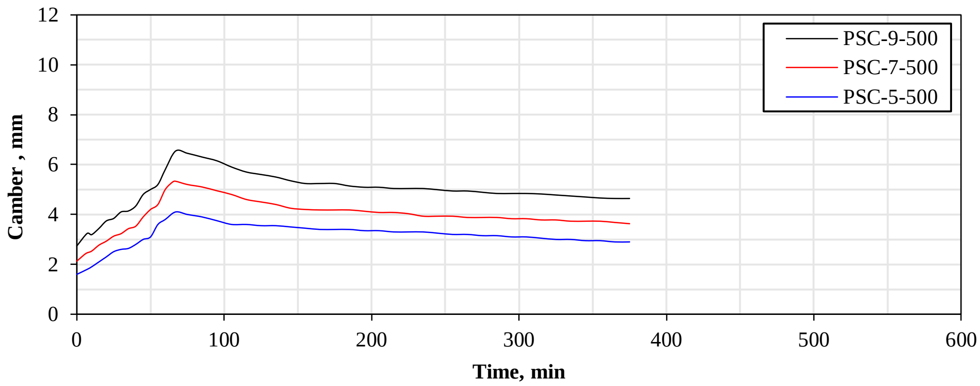


Figure 12. Camber-time history of PSC beams depending on the number of incorporated segments at fire test of 500°C

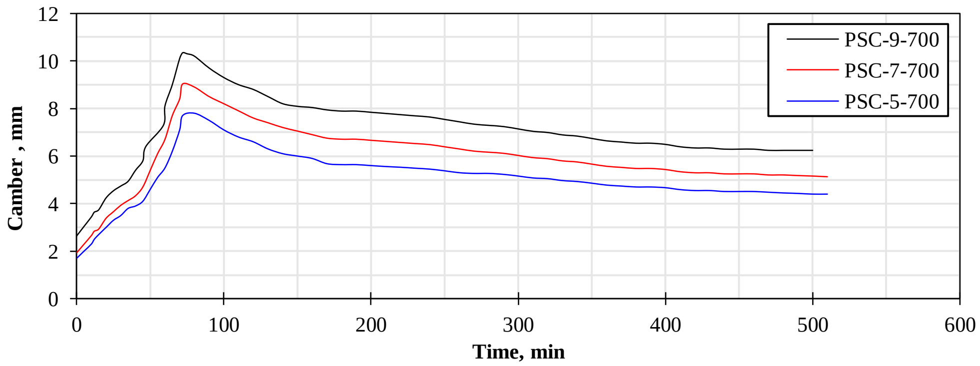


Figure 13. Camber-time history of PSC beams depending on the number of incorporated segments at fire test of 700°C

3.3. Displacement of the Test Beams under Monotonic Static Loading

To investigate the structural behavior of the segmental beams after fire exposure and to evaluate their post-fire resistance, all test beams were subjected to monotonic static loading under the effect of two concentrated loads that were applied at a distance of 975 mm from the nearest support for each.

The load was applied gradually up to failure, where each load increment consisted of 2.5 kN. It is worth to mention that after fire testing microcracks with different intensity were observed on the surfaces of the test specimens. The intensity and distribution of these cracks depended on the heating temperature that the structural member was exposed. The load-deflection curves for PSC beams of Groups I, II and III, respectively, are shown in Figures 14 to 16. Based on these figures it can be observed that, for specimens with the same number of incorporated concrete segments, as the heating temperature increased the load capacity of the test specimen was significantly decreased. This fact can be attributed to the considerable microcracking that formed during the exposure to elevated temperatures. Accordingly, as the heating temperature increases the microcracking process progressively increases. The formed microcracks affected the bond between the aggregate and the cement paste that caused a reduction of the effective section area and the concrete modulus of elasticity which in turn led to significant degradation of the overall stiffness of the test member. Figures 14 to 16 show also that, the effect of the heating temperature of 300 °C was very limited on changing the load-deflection behavior of segmental beams and on the reduction of their load-carrying capacity. Meanwhile, as the heating temperature during the fire test was increased to 500 or 700 °C, the load-deflection behavior became flattered and the reduction of the load capacity became more than the reduction at 300 °C due to the same reasons that mentioned above. It is worth mentioning that in segmental concrete beams the lower concrete chord does not contribute to the flexural resistance of the member due to the dry joints created between different concrete segments. As a result, the externally applied load in this system of construction (i.e., post-tensioned segmental concrete beams) resisted by the internal couple represented by the lower tension in the prestressing steel and the upper compression forces resultant in the top concrete chord. From the flexural resistance point of view, the concrete in the lower chord is playing just the role of a protector layer from fire and environmental attacks. The applied load and the corresponding midspan deflection for all PSC beams at different stages of loading are listed in Table 4. In this table, three different loads were adopted to compare the performance of all test beams, mainly, the applied load of 27.3 kN which represents 65% of the minimal failure load among all test beams, the service load of the nine-segment members which considered equal to 65% of the failure load of the corresponding nine-segmental beam, and the failure load of the corresponding beam. As shown in this table and illustrated in Figures 14 to 16, the deflection due to the applied load was increased with the increasing of the heating temperature that the structural member was exposed during the fire test. At the applied load of 27.3 kN, the deflection increase relative to the reference member (i.e., member which was not exposed to fire test), for PSC beams of different numbers of incorporated concrete segments 9, 7, and 5 that exposed to heating temperature of 300 ºC, was 44, 31, and 20%, respectively. This increase of the midspan deflection for specimens that exposed to 500 ºC attained 134, 105, and 80%, respectively. It is important to note that the worst case was for the segmental beams that exposed during the fire test to the heating temperature of 700 ºC. For these beams, the overall stiffness was highly affected by this range of temperature and the relatively long time period of exposure. The midspan deflection increase reached 480, 405, and 310 % for the specimens of 9, 7, and 5 concrete segments, respectively, in comparison to their reference beams.

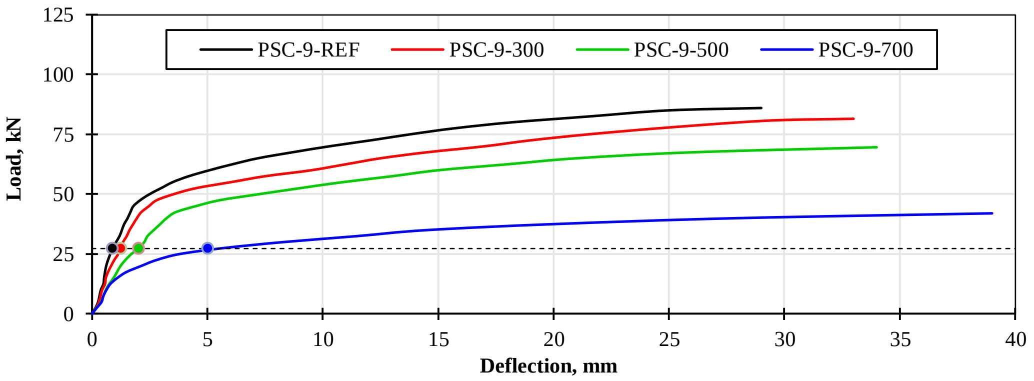


Figure 14. Load-deflection curve of Group I

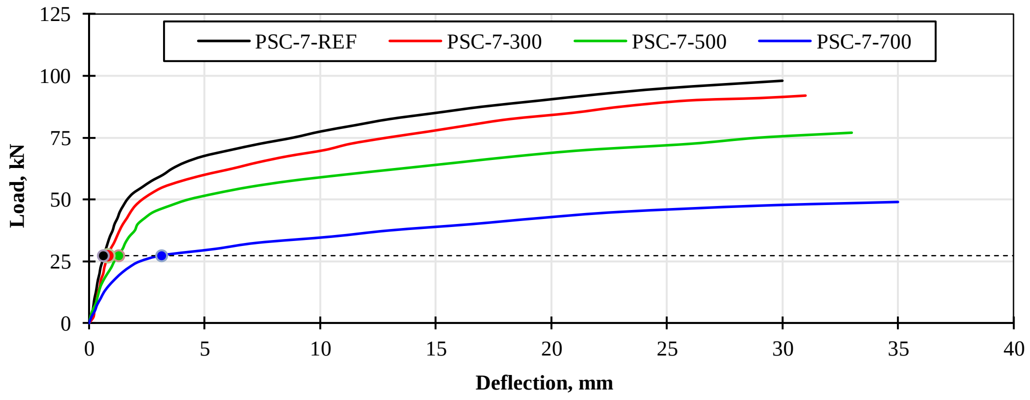


Figure 15. Load-deflection curve of Group II

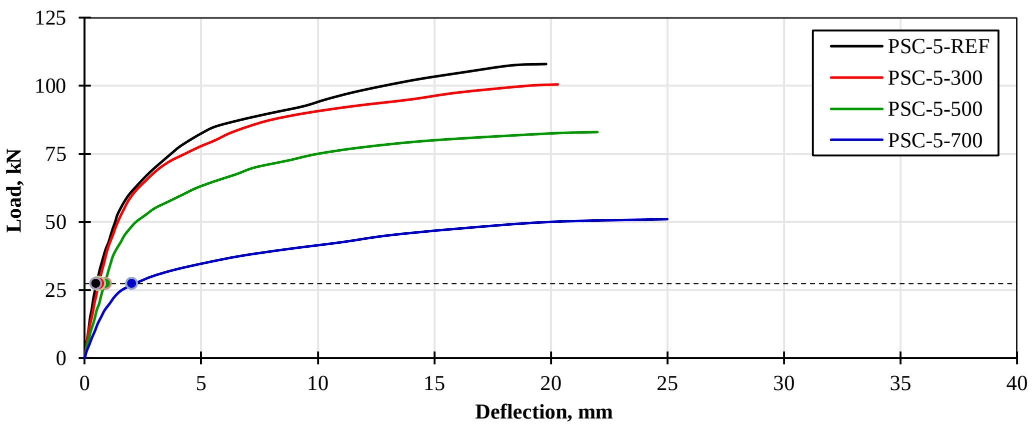


Figure 16. Load-deflection curve of Group III

The effect of the number of the incorporated concrete segments on the behavior of the test specimens was demonstrated through the load-midspan deflection diagrams during the monotonic static loading test, see Table 4 and Figures 17 to 20. Obviously, the overall stiffness of the reference beams and the beams that exposed to the fire test of different heating temperatures was highly depending on the number of the incorporated concrete segments. Accordingly, as this number increased the overall stiffness decreased and in turn the midspan deflection increased. Needless to say that the reason behind this fact is as the number of the incorporated in the structural member concrete segments increases, the possibility of the rotational displacement in the created interface sections (i.e., joints) between different segments also increases which leads to excessive translational displacement. Accordingly, at a load equals to the service load of nine-segment beams the midspan deflections for seven-segment and five-segment beams were averagely decreased by about 37 and 63%, respectively, at different heating temperatures, see Table 4. It is very interesting to note that the difference in performance, under the applied monotonic static loading depending on the number of the incorporated concrete segments, was decreased as the heating temperature of the fire test was increased. The reason behind this observation was the huge deterioration and microcracking intensity that occurred in concrete composition at high temperatures (i.e., 500 and 700 ºC) regardless of the number of the incorporated segments. This fact led to minimizing the difference between the load-midspan deflection behavior for the members that exposed to the same heating temperature (i.e., 500 or 700 ºC) but consisted of different numbers of concrete segments (i.e., 9, 7 or 5).

Table 4. Deflections at different loading stages of test beams

Specimen ID	At a load of 27.3 kN		At service load of nine-segment members which adopted 65% of failure load			At failure load		
	Deflection, mm	Deflection increase relative to reference member, %	Service load of nine-segment specimen, kN	Deflection, mm	Deflection change relatie to nine-segment member, %	Failure load, kN	Deflection, mm	Deflection increase relative to reference member, %
PSC-9-REF	0.86	-	-	-	-	86	29	-
PSC-9-300	1.24	44	53.3	4.57	-	82	33	14
PSC-9-500	2.01	134	45.5	4.71	-	70	34	17
PSC-9-700	4.99	480	27.3	4.99	-	42	39	34
PSC-7-REF	0.62	-	-	-	-	98	30	-
PSC-7-300	0.81	31	53.3	2.84	-38	92	31	3
PSC-7-500	1.27	105	45.5	2.95	-37	77	33	10
PSC-7-700	3.13	405	27.3	3.13	-37	47	35	17
PSC-5-REF	0.49	-	-	-	-	108	20	-
PSC-5-300	0.59	20	53.3	1.61	-65	101	20	0
PSC-5-500	0.88	80	45.5	1.77	-63	83	22	10
PSC-5-700	2.01	310	27.3	2.01	-60	51	25	25

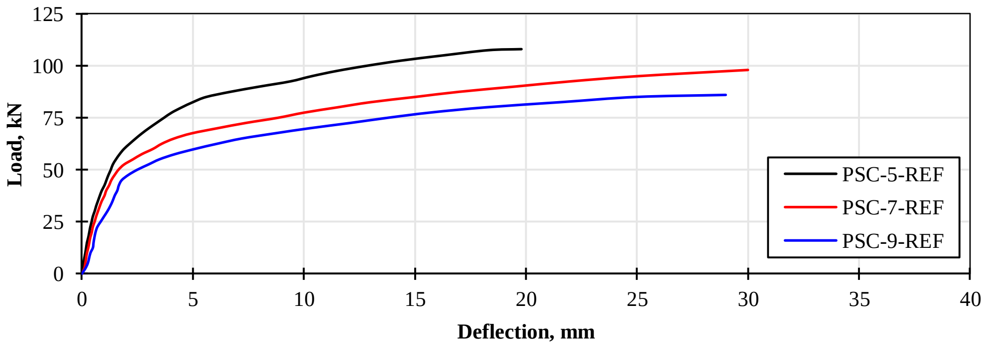


Figure 17. Load-midspan deflection curves for reference PSC beams depending on the number of the incorporated concrete segments

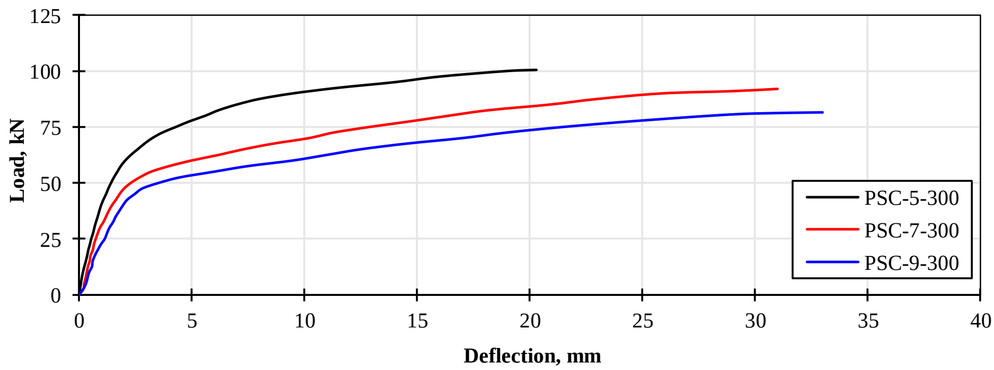


Figure 18. Load-midspan deflection curves for PSC beams, which exposed during fire test to 300 ˚C heating temperature, depending on the number of the incorporated concrete segments

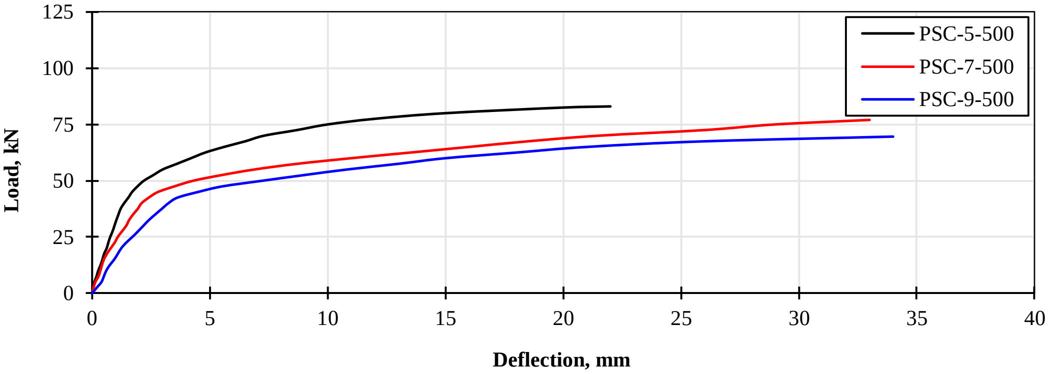


Figure 19. Load-midspan deflection curves for PSC beams, which exposed during fire test to 500 ˚C heating temperature, depending on the number of the incorporated concrete segments

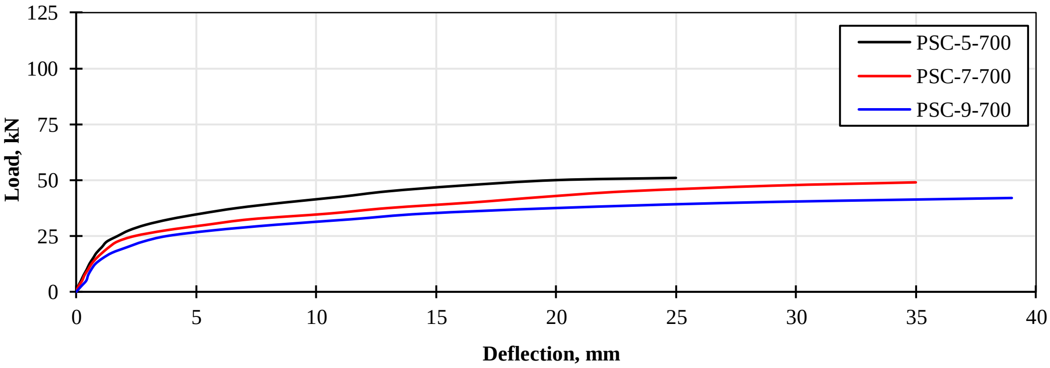


Figure 20. Load-midspan deflection curves for PSC beams, which exposed during fire test to 500 ˚C heating temperature, depending on the number of the incorporated concrete segments

3.4. Load-carrying Capacity and Failure Modes of the Test Beams

Table 5 shows the outcome data for the failure load of all test specimens. Obviously, the exposure of the post-tensioned segmental concrete beams to high heating temperatures during the fire test stage affected seriously the post-fire performance and the load-carrying capacity of the mentioned structural members during their exposure to a monotonic static loading stage. Test results revealed that two parameters were significantly influenced the load capacity of the investigated structural members, mainly, the heating temperature value that the member was exposed during the fire test stage and the number of the incorporated concrete segments that the member was consisted of.

Increasing each or both of these parameters resulted in a reduction of the load-carrying capacity due to the reasons discussed above. In Group I of nine-segment beams, the residual load-carrying capacity was 95, 81 and 49% of the failure load of the reference beam PSC-9-REF for members exposed during fire test to heating temperatures of 300, 500, and 700 ºC, respectively. The reduction of the failure load for this group in comparison to the reference beam was ranged between 5 to 51%. Whereas in Group II of seven-segment beams and in Group III of five-segment beams, the failure load was 94, 79, and 48 % of the load capacity of the reference beam PSC-7-REF and 94, 77, and 47% of the failure load of the reference beam PSC-5-REF for specimens that experienced during fire test stage to temperatures of 300, 500, and 700 ºC, respectively. Accordingly, the reduction of the load capacity for Groups II and III in comparison to their reference beams was ranged between 6 to 52% and 6 to 53%, respectively.

For the three groups when the heating temperature increased up to 300 ºC, the average residual strength decreased by a rate of 6% compared to the reference beams. Increasing during the fire test stage the heating temperature up to 500 ºC led to a decrease in the average residual strength by 21%. Meanwhile, the average residual strength decreased by a rate of 52% when the temperature reached 700 ºC compared to the reference beams. In comparison to the test beams with nine concrete segments, it can be noted that at a heating temperature of 300 °C the failure load value increased by 12 and 23% for beams with seven and five concrete segments, respectively. However, at 500 °C, the increase of the failure load value attained 10 and 19% for test beams with seven and five concrete segments, respectively. The same behavior was monitored for the members that exposed to a fire test of 700 °C heating temperature, whereas the increase of the load-carrying capacity reached 12 to 21% for the same mentioned above specimens, respectively. Figure 21 shows the comparison of the load-carrying capacity between different test beams. Figure 22 demonstrates the cracking distribution and the failure mode of the test specimens. It should be mentioned that in all test specimens the mode of failure included the crushing of the concrete in the compression zone of the central concrete segment due to the excessive rotation that occurred during the application of the external loading.

Also, the joint-opening width at failure stage was highly depended on the heating temperature that the post-tensioned segmental concrete member was exposed to.

Table 5. The load-carrying capacity of test PSC beams

Specimen	Failure Load, (kN)	Change of failure load relative to a reference member, %	Change of failure load relative to nine-segment member, %
PSC-9-REF	86	-	-
PSC-9-300	82	-5	-
PSC-9-500	70	-19	-
PSC-9-700	42	-51	-
PSC-7-REF	98	-	+14
PSC-7-300	92	-6	+12
PSC-7-500	77	-21	+10
PSC-7-700	47	-52	+12
PSC-5-REF	108	-	+26
PSC-5-300	101	-6	+23
PSC-5-500	83	-23	+19
PSC-5-700	51	-53	+21

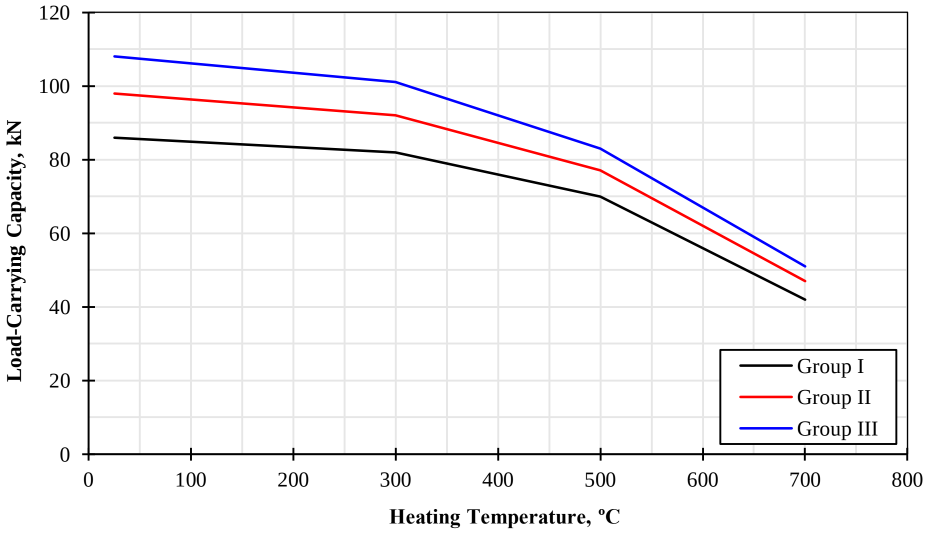
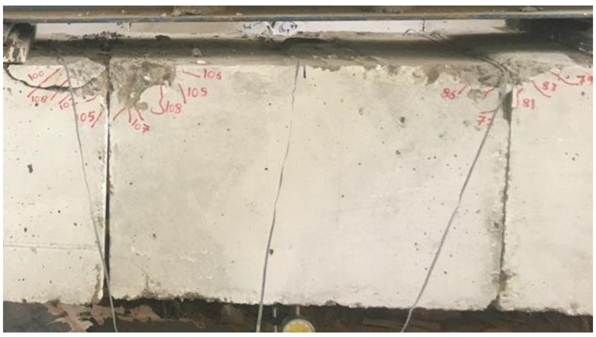
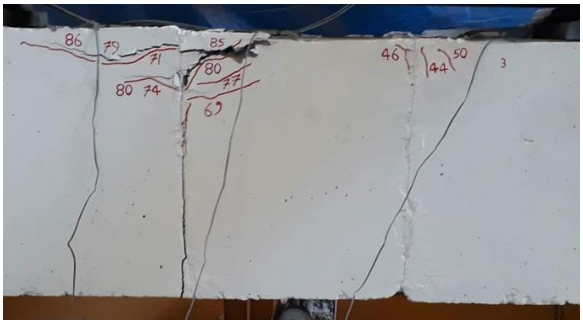


Figure 21. Comparison of the load-carrying capacity between different groups

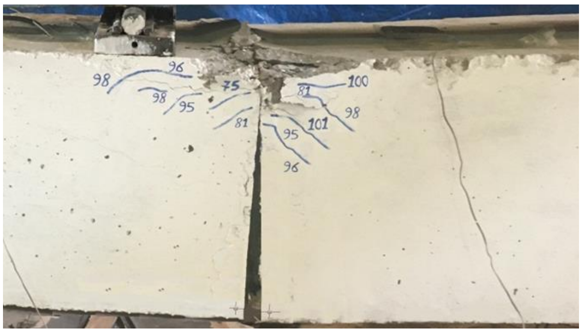


PSC-5-REF

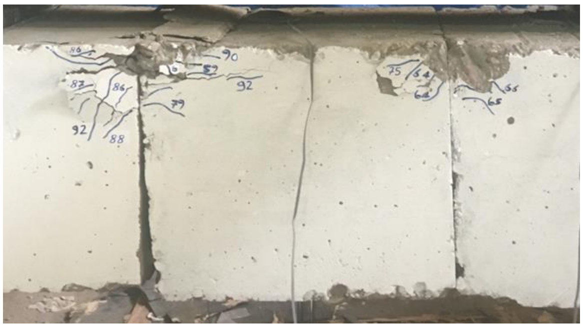
PSC-7-REF



PSC-9-REF



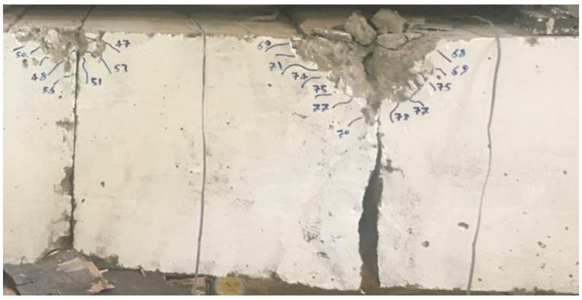
PSC-5-300



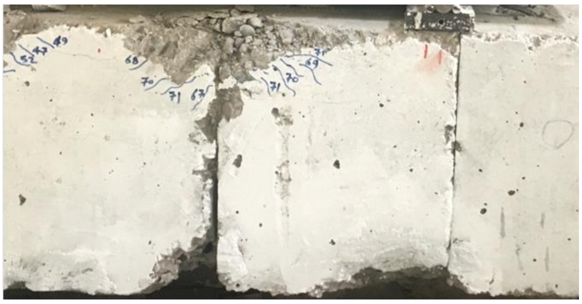
PSC-7-300

PSC-9-300

PSC-5-500



PSC-7-500



PSC-9-500

PSC-5-700

PSC-7-700

PSC-9-700

Figure 22. The failure mode of test beams

4. Conclusion

It was observed that decreasing the number of the incorporated segments of the beam led to a significant reduction in the midspan camber (i.e., decreasing the number of segments from nine to seven or five, which led to a decrease in the camber by 21% or 38%, respectively). The structural response of the segmental beams during the heating phase was characterized by two-time intervals. In the first interval, the increase of the camber resulted from the thermal strains caused by high thermal gradients. While the camber was progressively increased and attained its peak value at the end of the second interval as the temperature increased in the internal layers of concrete due to the migration of the heat energy across the member that deteriorated the texture of the concrete and caused microcracking of larger surface areas. Under the cooling event, the camber-time diagram descended where the length and the slope of the descending branch depend on the heating temperature and the number of incorporated concrete segments. The main reasons behind the appearance of the descending branch are the progressive increase of the prestressing losses due to the temperature difference and the creep of concrete that highly depends on the value of the heating temperature.

The relative residual midspan camber at the end of the cooling phase consisted (48–53%), (69–81%), and (136–166%) for heating temperature of 300, 500, and 700 °C, respectively, depending on the number of the incorporated concrete segments in the test beams and the heating temperature of the fire test. For beams with the same number of incorporated concrete segments, as the heating temperature increased during the fire event, the load-carrying capacity of the test specimen was significantly decreased. The reduction of the failure load for the nine-segment beams in comparison to the reference beam was ranged between (5-51%), whereas in the seven-segment beams and the five-segment beams, the reduction attained (5 –52%) and (6–53%), respectively. As the heating temperature increased up to 300 ºC, the average residual strength decreased by a rate of 6% compared to the reference beams. Increasing during the fire test stages the heating temperature up to 500 ºC led to a decrease in the average residual strength by 21%. Meanwhile, the average residual strength decreased by a rate of 52% when the temperature reached 700 ºC compared to the reference beams. The mode of failure included the crushing of the concrete in the compression zone of the central concrete segment.

5. Conflicts of Interest

The authors declare no conflict of interest.

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