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Examining fire response of unilaterally concrete-reinforced web prestressed composite beams with corrugated webs

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Abstract

Prestressed steel-concrete composite beams with concreted and corrugated webs are a novel system that have been attracting attention lately. Unfortunately, there has been little research with regard to the fire performance of such beams. To overcome this knowledge gap and in the hope of exploring attractive solutions to improve the fire resistance of these beams, a testing campaign was conducted to examine the potential of reinforcing the webs of such beams via concrete. In this pursuit, three different prestressed steel-concrete composite beams (PCBCWs) were examined under fire conditions. The first beam was conventional PCBCW, the second PCBCW was unilaterally concreted between flanges and the third PCBCW was bilaterally concreted between flanges. These PCBCWs were exposed to the standard ISO834 fire and mechanical loading. The results of the fire tests revealed that PCBCWs without encased concrete suffer from the buckling of the web, but beams reinforced with concreted webs did not. Moreover, the vertical deformations were significantly influenced by the concreting of webs or the absence of such reinforcement. PCBCW with one-sided concrete between the flanges also underwent horizontal deflection during the test. To complement the conducted fire tests, finite element models (FEM) were developed to further explore the fire response of PCBCWs. The FE simulations show that, for PCBCW with one-sided encased concrete between flanges, there is horizontal deflection due to asymmetrical tension of the cable strands, horizontal temperature gradient, and torque around the longitudinal axis of the beam. Further, the shear center of PCBCW with one-sided encased concrete moves towards the coldest side with temperature rise. Finally, the FE analysis indicates signs of rupture of the cable strands inside the encased concrete triggers the failure and corresponding horizontal deflection of PCBCW with one-sided encased concrete between the flanges at high temperatures.

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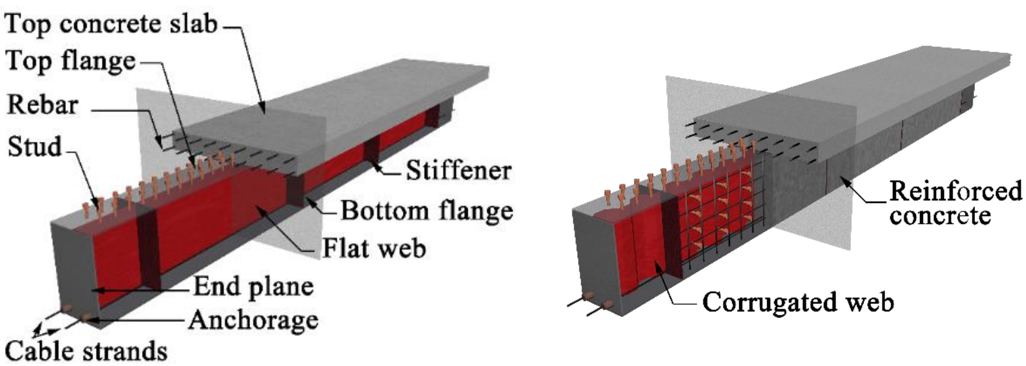
Keywords: Fire resistance; Prestressed; Steel concrete composite beam; Corrugated web; Concrete between flanges.

1. Introduction

Prestressed steel-concrete composite beams with corrugated webs (PCBCWs) are a novel configuration of beams[1] with greater out-of-plane stiffness and ratio of height to the thickness of the web than that of conventional composite beams. This is due to the fact that the greater height of the web allows PCBCWs to carry a larger shear force than that of the composite beams with flat webs[2]. In addition to the above unique features, PCBCWs also inherits the advantages of traditional prestressed steel-concrete composite beams (Fig. 1 (a)), such as high tensile strength of steel, high compressive strength of concrete[3], and the presence of the cable strands enabling PCBCWs to serve for larger spans[4, 5]. Also, PCBCWs overcome the disadvantages of the conventional composite beams with flat webs, such as vulnerability to local instability of the webs under bending moment. The corrugated webs give PCBCWs a greater height-to-thickness ratio of webs and further improve their economic value [6-12]. Furthermore, due to the accordion effect of the corrugated webs, the effect of prestress in the PCBCWs is higher than that of the traditional composite beams with flat webs[13-17]. This accordion effect makes it possible for the prestressed strands to contribute much to the bending stiffness of PCBCWs[18-21].



(a) PCB (b) PCBCW concreted between flanges

Fig. 1 Comparison of traditional prestressed steel-concrete composite beams (PCB) and new PCBCW concreted between flanges

Recent works have examined the mechanical response of PCBCWs theoretically [20, 22-24]. Similarly, the shear performance of PCBCWs was also investigated experimentally and numerically[23, 25-27]. These studies demonstrate that these types of composite beams are likely to fail under an interaction mode, which is a buckling behavior between global and local buckling[26]. An effective approach to mitigate such failure mode is to incorporate a new configuration of PCBCW wherein flanges are concreted. The typical feature of this configuration is that the web is surrounded by

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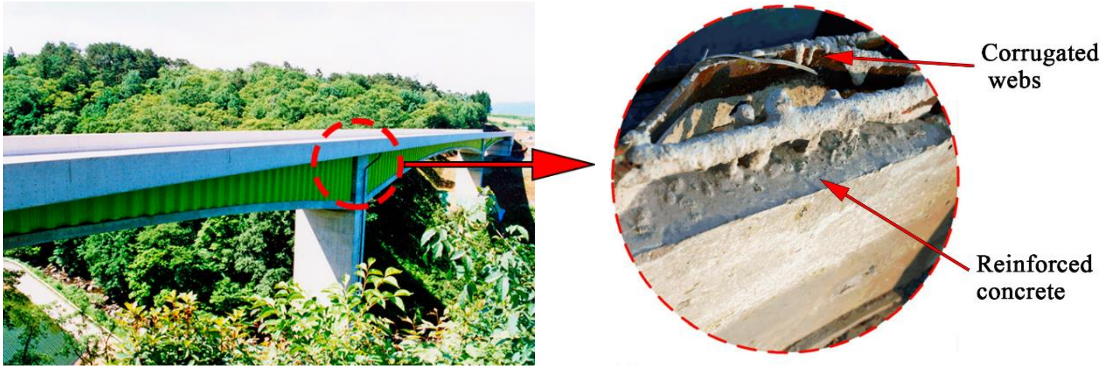
concrete between flanges, which includes two types of configurations, namely, one-sided and two-sided concrete between flanges (Fig. 1 (b)). This configuration of one-sided concrete around the web strengthens the stability of the web especially, which was utilized in the negative moment zone of continuous beams (Fig. 2 (a) and (b)).



(a) Elevation of PCBCWs with concrete-reinforced web at middle support

(b) Details at the middle support

Fig. 2 Prestressed composite girders with concrete-reinforced web corrugated webs[28]

to date, much research has been carried out on the performance of PCBCWs concreted between flanges at ambient conditions. For example, previous studies reveal that composite beams with corrugated webs have greater shear capacity than that conventional composite beams[16]. Other studies demonstrate that composite beams with encased corrugated webs have the advantage of larger bending capacity, greater shear buckling, and ductility over traditional beams without the encased concrete[29-31].

Interestingly, composite beams concreted between flanges on one side of the web have some performance differences from composite beams without encasement of concrete around the web, and composite beams concreted between flanges on two sides of the web because of the movement of the shear center between flanges on one side of the web when exposed to fire. Specifically, the shear center of the cross section moves towards the coolest side due to different deterioration rates of elastic modulus of steel and concrete in the cross section when exposed to fire.

Regrettably, there is only a little amount of research with regard to the fire resistance of PCBCWs with unilateral encased concrete between flanges[32]. Thus, to investigate the fire responses of PCBCWs with unilateral concreted between flanges, three specimens were designed. These specimens included conventional PCBCW, PCBCW with one-sided reinforced concrete between flanges, and PCBCW with two-sided reinforced concrete between flanges, which were investigated experimentally and numerically. The focus was to understand the effect of unilaterally reinforced concrete between flanges on the fire performance of PCBCWs.

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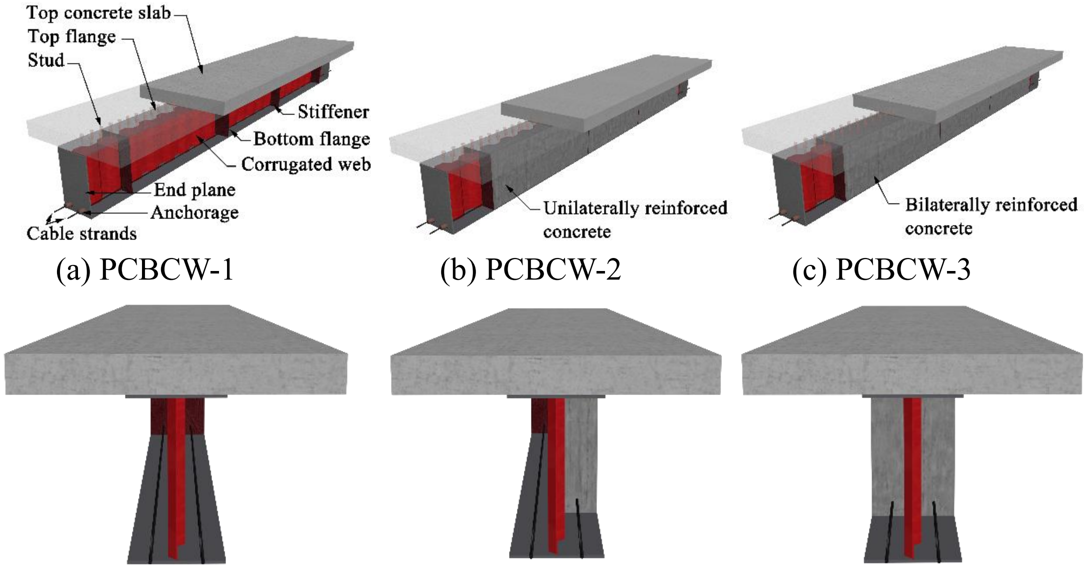
2. Experimental program

2.1 Specimen preparation

To investigate the behavior of PCBCW unilaterally concreted between the flanges at elevated temperatures, three specimens of PCBCW concreted between flanges on one side of the web were fabricated (see Fig. 3 and note thedetails on these specimens are listed in Table 1). In these specimens, PCBCW-1 was not concreted between its flanges, PCBCW-2 was concreted between flanges on one side of the web, and PCBCW-3 was concreted between flanges on both sides of the web. To investigate the worst performance of PCBCWs at high temperatures, all these PCBCWs were not protected by thermal insulation materials.



(a) PCBCW-1 (b) PCBCW-2 (c) PCBCW-3

(d) Cross section of PCBCW-1 (e) Cross section of PCBCW-2 (f) Cross section of PCBCW-3

Fig. 3 Elevation and cross section of tested PCBCWs

Table 1 Details of the tested specimens and their critical time

Specimen	Span (mm)	Rebar	Prestress ratio	Load ratio	Magnitude of load (kN)	Critical time (min)
PCBCW-1	4400	HRB335	0.6	0.3	130	19
PCBCW-2	4400	HRB335	0.6	0.3	265	25
PCBCW-3	4400	HRB335	0.6	0.3	282	43

In Table 1, the load ratio is defined as the ratio of the applied load to the bearing capacity of the composite beam at room temperature. The bearing capacity is obtained through numerical analysis by increasing the applied load step by step until the beam cannot sustain the applied loading. The external load at this state is defined as the bearing capacity of the beam at room temperature. The prestress ratio is defined as the

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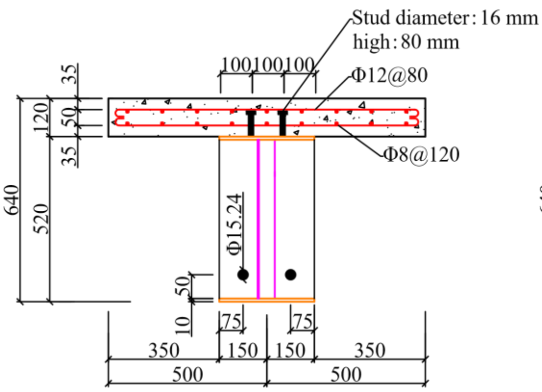
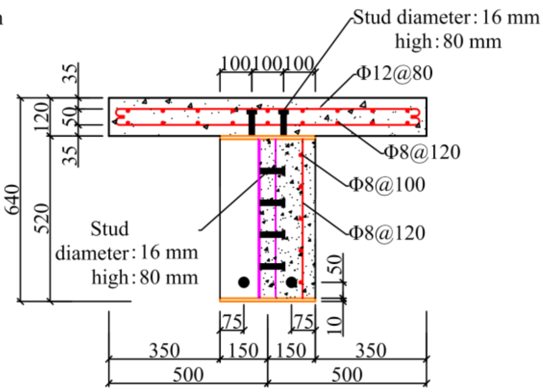
ratio of the actual prestress in the cable strand to the tensile strength of the cable strand at room temperature. The prestress level is determined by referring to the stress level used in practical (in-situ) conditions which also considers the expected safety level of the prestressed cable strands at high temperatures.

The concrete strength in these PCBCWs is C30, as classified per the Chinese code (GB50010-2010)[33], with an average cubic strength of 33.7 MPa. The yield strength of the steel beams is 332.7 MPa for 4 mm thick steel plates and 273.2 MPa for 10 mm thick steel plates which make the webs and flanges, respectively. The yield strength is 342.4 MPa for the steel bars. The cross sections of PCBCWs are shown in Fig. 4 (a)~(c). In these specimens, the corrugated web is of a trapezoidal shape, in which wave length, wave height, and slope of the hypotenuse are 50 mm, 70 mm, and 45°, respectively (see Fig. 5). The prestressed cable strands, with a diameter of 15.24 mm, have a tensile strength of 1860 MPa, and a modulus of elasticity of 1.9×10^6 MPa.

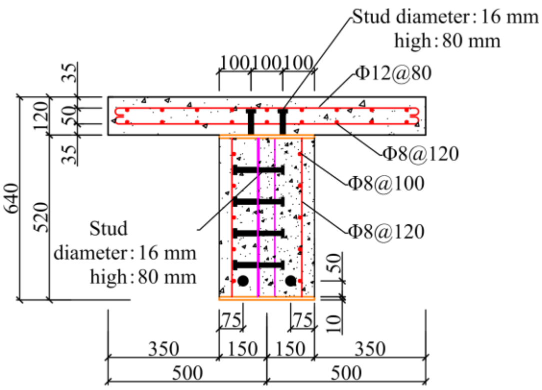
According to Eurocode 4[34], the concrete encasement between the flanges should be attached to the web of the beam. To attach the encased concrete to the web, studs with a diameter of 16 mm and length of 80 mm, were welded to the web by fillet welds as shown in Fig. 4. The load ratio, the prestress ratio, the diameter and length of the studs mentioned above and the width of the top concrete slab are determined based on engineering experiences and former fire tests carried out by the authors.



(a) Cross section of PCBCW-1



(b) Cross section of PCBCW-2



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(c) Cross section of PCBCW-3

(d) Bottom steel beam of PCBCW-2

(e) Top concrete slab pouring completed

Fig. 4 Cross-section details and fabrication of PCBCWs

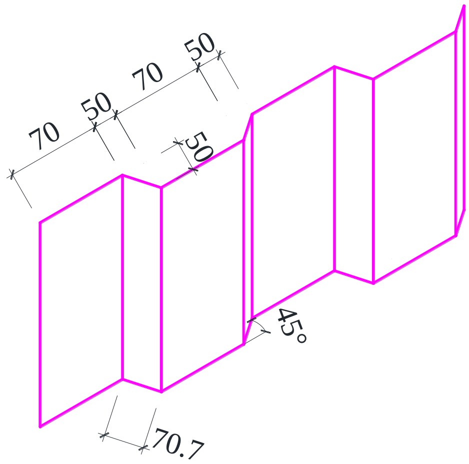

Fig. 5 Details of corrugated web

2.2 Test set-up and loading

The tests were conducted in the key laboratory at the South China University of Technology in Guangzhou, China. In the laboratory, there is a horizontal test furnace, with 1.5 m (height) × 5.0 m (width) × 5.0 m (length) (Fig. 6), which can generate a temperature-time curve similar to that of ISO834, expressed as Eq. (1):

$$T = 345 \log_{10}(8t+1) + 20 \quad (1)$$

Where, T is the gas temperature in the furnace (in °C), t is the temperature rise time (in *min*.).

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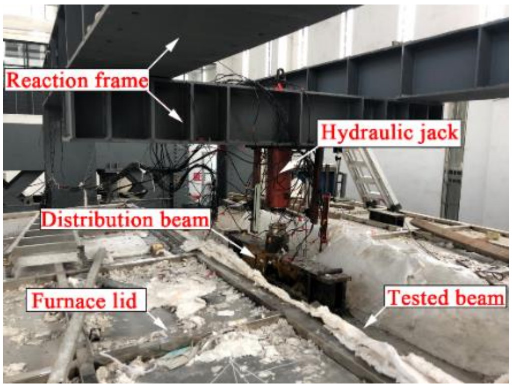
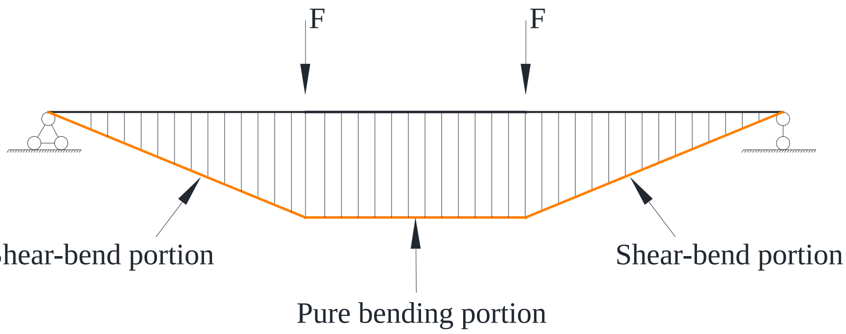
(a) Installation of PCBCW



(b) The experiment in progress



(c) Loading diagram and moment of PCBCW

Fig. 6 Specimen and test set-up

Over the furnace, there is a reaction frame with a maximum load capacity of 2000 kN. During each test, the specimen was simply supported in the furnace wall and loaded by a hydraulic jack installed into the reaction frame through a distribution beam (Fig. 6 (a) and (b)). During the tests, the specimens were subjected to constant mechanical loading and exposed to the standard fire equation (ISO834) on three sides: the web, bottom flange, and the bottom surface of the top flange being explored to fire.

2.3 Instrumentation

During the tests, a series of thermocouples, displacement gauges, and load transducers were arranged to monitor the thermal and mechanical responses of the tested specimens. For example, the temperature of the furnace and across the specimen, the deflections at mid-span in the vertical plane, and the tension in the cable strands were recorded. As one can see in Fig. 7 (a)~(c), three thermocouples labeled *C1*, *C2*, *C3*, were embedded in the different depths of the top concrete slab to monitor the temperature along with the cross-section height. Two thermocouples labeled *GJ-1*, *GJ-2*, were attached to the rebars to measure the temperature of the rebars. Three thermocouples from top to bottom, labeled S_1, S_2, S_3, were attached to the top flange, web and bottom flange to monitor the temperature of steel beams. Especially, two

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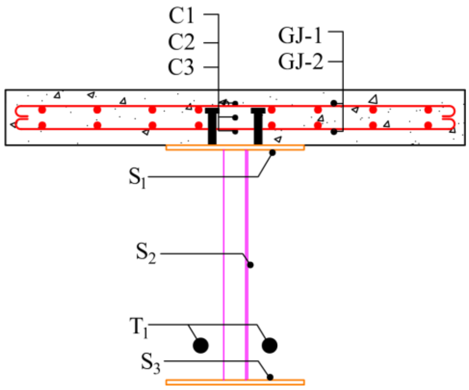
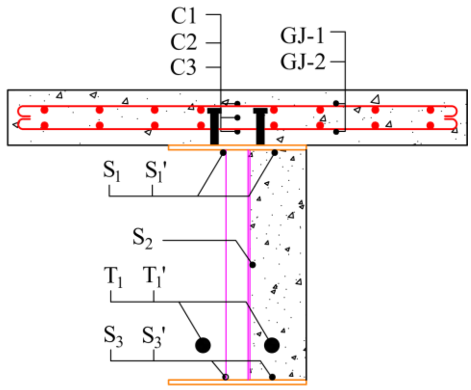
additional thermocouples labeled S_1', S_3', were attached to the top flange and bottom flange inside the encased concrete to monitor the temperature at the right flanges of PCBCW-2. Two thermocouples labeled T_1, T_1', were attached to the prestressed cable strands to monitor the temperature of the cable strands (Fig. 7 (a)~(c)).

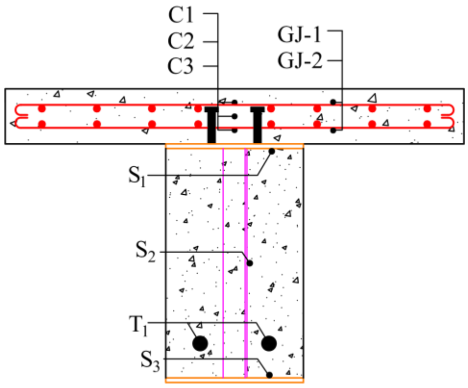
The deflection at mid-span in the vertical plane was also recorded for each specimen using a linear variable displacement transducer (LVDT) installed on the top surface of the top concrete slab labeled *SL-1*. To monitor the lateral deflection of PCBCW-2, LVDT *SL-2* was installed on the top surface of the top concrete slab (Fig. 7 (d)). In addition, during the test, the force transducer (Fig. 7 (e) and (f)) was input into the acquisition system to monitor the tension in the cable strands.



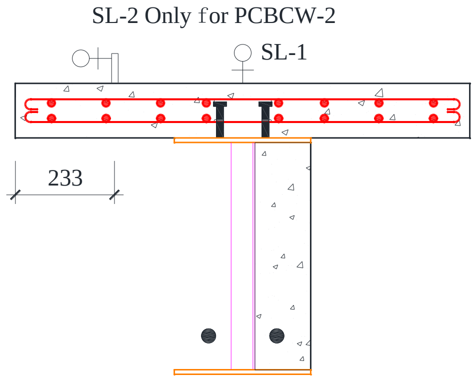
(a) Layout of thermocouples for PCBCW-1



(b) Layout of thermocouples for PCBCW-2



(c) Layout of thermocouples for PCBCW-3



(d) LVDTs for the deflection for PCBCW-1, 2, 3

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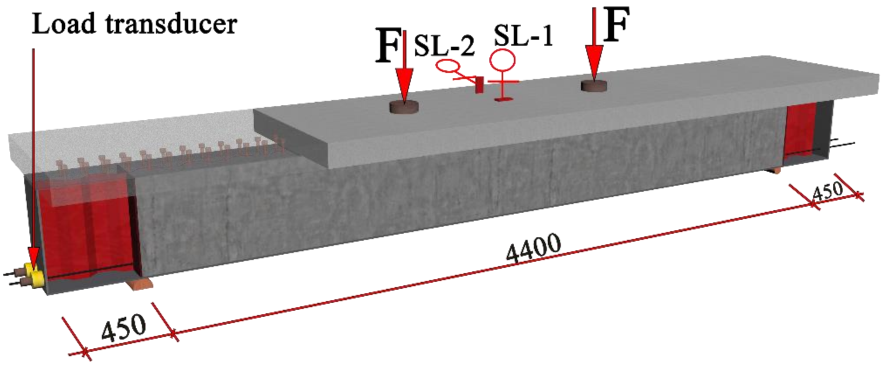
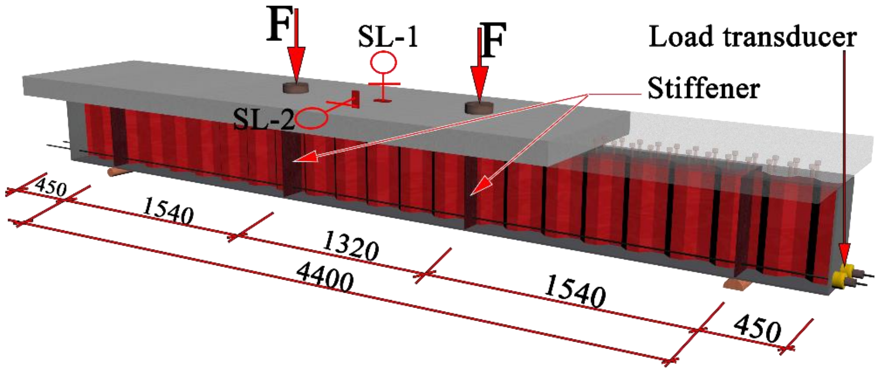


(e) Layout of load transducers in the cable strands for PCBCWs (obverse side)



(f) Layout of load transducers in the cable strands for PCBCWs (reverse side)
Fig. 7 Layout of thermal couples, LVDTs, and load transducers

2.4 Test procedures

The tests were carried out according to the following procedure. Firstly, the specimens were installed on the supports located on the steel wall of the furnace. The instrumentation, such as thermocouples and LVDTs, was connected to the acquisition instrument and tested to ensure their soundness.

Secondly, the two cable strands beside the web were passed through force transducers and were tensioned to the target level. During the tensioning of the cable strands, the two cable strands were tensioned alternatively, and the tension in the cable strands was monitored by load transducers (Fig. 7 (e) and (f)). When the tension in the cable strands approached the same level, tensioning of the cable strands was terminated, and the two cable strands were fixed at anchor plates. Subsequently, each specimen was loaded to 5% of the target load to ensure the specimen and instrumentations worked.

Thirdly, the furnace lids were covered and all the gaps between the specimens and the furnace were filled with insulative materials to mitigate any heat loss. At last, the specimen was reloaded to the target load at a rate of 5 kN/min. When the system worked normally, the furnace temperature was elevated to follow the international standard

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curve (ISO834), and the load was kept constant during the test. Once the specimen could not bear the applied load, the test was terminated[35].

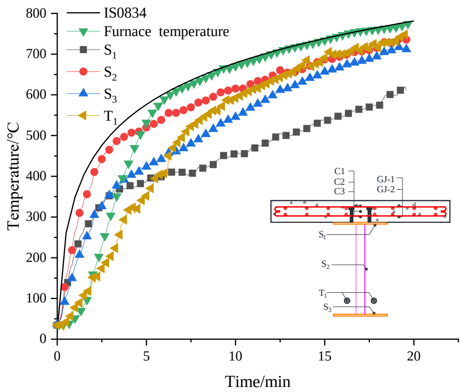
3. Experimental results

3.1 Temperature rise in specimens

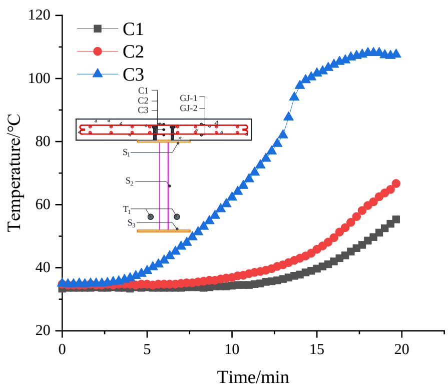
The evolution of temperature rise in the mid-span section of PCBCW-1 to PCBCW-3 is shown in Fig. 8. Comparing these curves indicates that the presence of concrete between flanges has a significant effect on temperature evolution along the cross section of the beams. For example, in the case of PCBCW-1, without encasement of the concrete, the temperature at the steel beam, the cable strands, and the top concrete slab rise at a much faster pace than that of PCBCW-2 and PCBCW-3. This is because the concrete between the flanges absorbs much heat during the test, which slows the temperature rise.

Due to the asymmetry of the section in three specimens, PCBCW-2 is specially taken as an example to analyze the temperature distribution in the section in detail. Of the observed points in the cross section, the temperature at T_1 rose at a rapid pace due to the large shape coefficient of the cable strands and the thermal boundary condition. Comparatively, although both the right cable strand and the left cable strand have the same shape coefficient, the temperature at the right cable strand (Fig. 3 (e)) rose at a much slower pacedue to the thermal shielding effect of the concrete on the right of the web.

In addition, it can be seen that the temperature of the right concrete was lower than that of the web and the left bottom flange due to the large specific heat of the concrete. Therefore, there occurred a distinct temperature gradient along the width of the cross section of PCBCW-2. Due to this gradient, lateral deflection grew. Especially, the movement of the shear center resulted in the fluctuation of the deflection in the horizontal plane during fire exposure.



(a) Temperature at steel beam and cables of PCBCW-1



(b) Temperature at Concrete and steel bars of PCBCW-1

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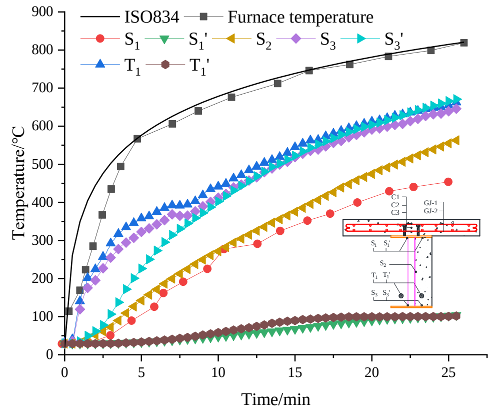
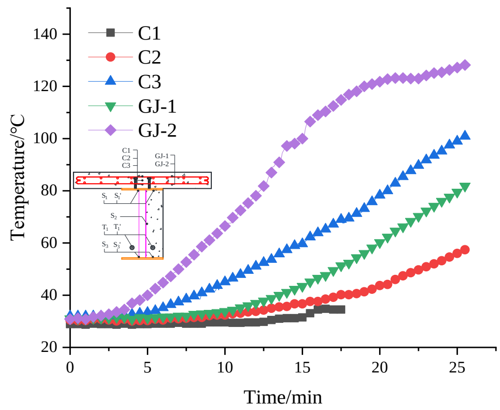
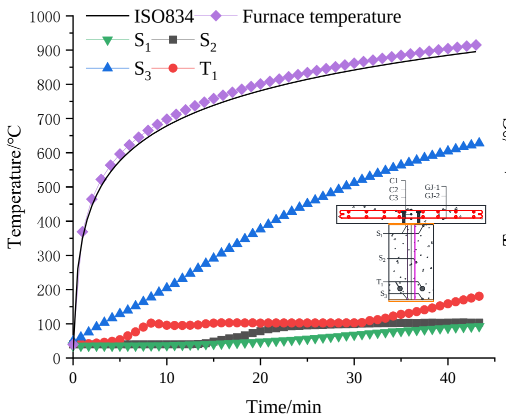


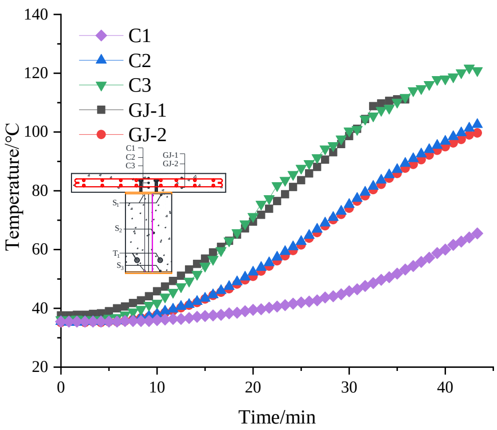
(c) Temperature at steel beam and cables of PCBCW-2



(d) Temperature at Concrete and steel bars of PCBCW-2



(e) Temperature at steel beam and cables of PCBCW-3



(f) Temperature at Concrete and steel bars of PCBCW-3

Fig. 8 Temperature-time curves in the mid-span section of PCBCW [Note: we acknowledge the observation that the temperature rise in PCBCW-1 during the first 2 minutes is slow (seeFig. 8 (a)). This can be explained as follows: the furnace temperature is the average temperature of the furnace. The actual temperature of the furnace is not uniform along the length of PCBCWs in the first few minutes of fire testing, so the temperatures at the observed points can be locally higher than the average furnace temperature.]

3.2 Structural responses

3.2.1 Vertical deflection at mid-span

To investigate the effect of high temperature on the vertical deflection at mid-span in PCBCW-1~PCBCW-3, the evolution of the vertical deflection with temperature is shown in Fig. 9. In these figures, the temperature of the horizontal axis refers to the temperature at S_3 (see Fig. 7), and the abscissas of temperature in all subsequent figures

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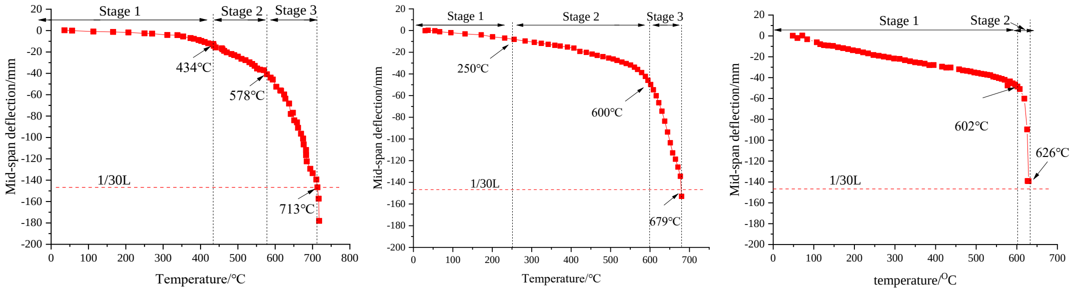
are the same. According to the slope of these curves, the evolutions can be divided into different stages. Taking PCBCW-2 as an example (Fig. 9 (b)), the slopes in the three stages were -0.04, -0.12, and -1.12 mm/℃, respectively. It can be seen that in stage 1 and stage 2, the rise in temperature is mediocre. As such, the mechanical properties of steel and concrete did not seriously deteriorate in these stages and this reflects the smaller deflections obtained. When entering stage 3, the slope was -1.12 mm/℃, which was much larger than that of stage 1 and stage 2. Namely, the bending stiffness decreased significantly in this stage, which resulted in larger deflections.



(a) Deflection of PCBCW-1 (b) Deflection of PCBCW-2 (c) Deflection of PCBCW-3

Fig. 9 Vertical deflections at the mid-span of PCBCW with temperature

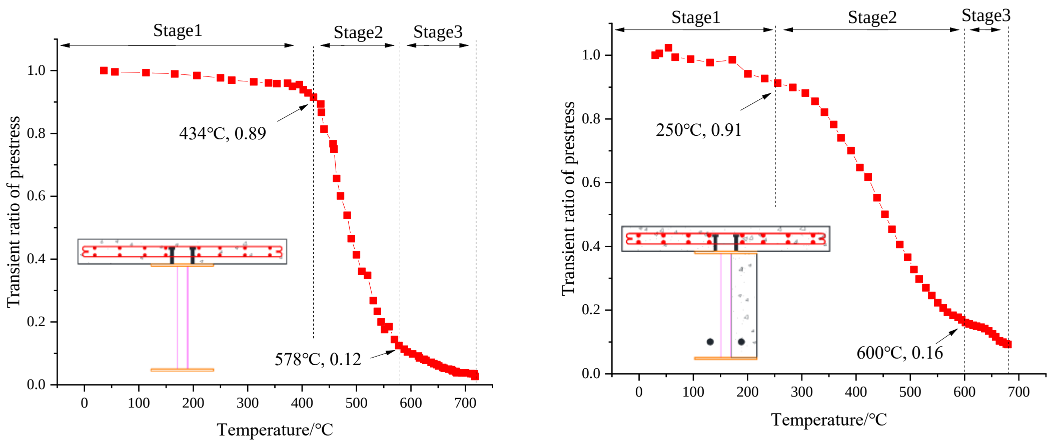
In these three composite beams, only the deflection curve of PCBCW-3 experienced a sudden and significant rise once reached its critical state at 602 °C[35]. This is due to the rupture of the two cable strands inside the lateral concrete after reaching their tensile strength. In contrast, the deflection of PCBCW-1 and PCBCW-2 developed gradually.

As one can see, among these tested composite beams, PCBCW-1 has the largest deflection, while PCBCW-3 has the least deflection at their critical states which reflects the effectiveness of the thermal insulation provided by the concrete

3.2.2 Tension in the cable strand

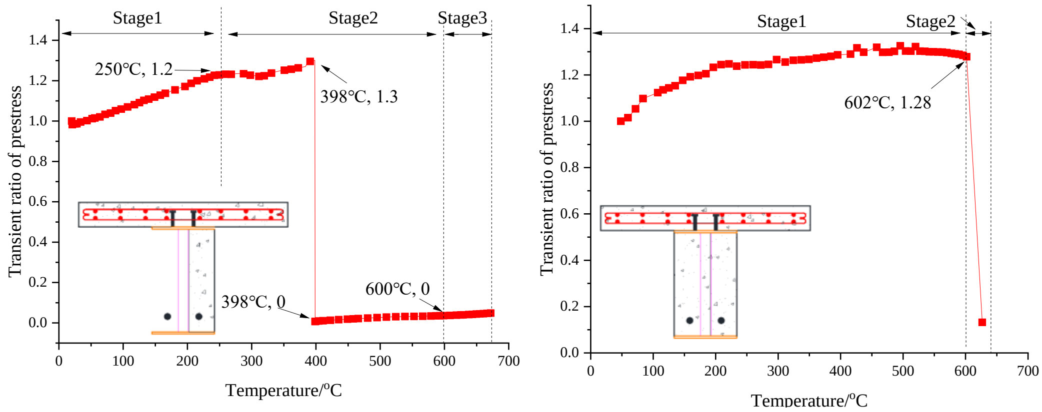
The variation of prestress in PCBCW-1~PCBCW-3 as a function of fire exposure is shown in Fig. 10. It can be seen there are two typical trends for the prestress variation in PCBCW-1~PCBCW-3. The first is the prestress in cable strands without the encased concrete falls smoothly until it decreases to zero at the critical stage (Fig. 10 (a) and (b)). The second is the prestress in cable strands encased by the lateral concrete increases with temperature slightly at the beginning stage and then decreases suddenly to zero due to the cable strands rupture in the following stage (Fig. 10 (c) and (d)).

This variation of the tension in the cable strands results as follows. Firstly, with the rise in temperature, the thermal expansion of the cable strands leads to a gradual decrease of prestress in the cable strands. Secondly, with the rise in temperature, the elastic modulus of the cable strands decreases, which lowers the stress in the cable strands under the same strain. Thirdly, with the continued rise in temperature, the coordinated deformation between the cable strands and the composite beam produces a large deflection, which excessively stretches the cable strands and causes their stress tends to rise correspondingly. Fourthly, as the temperatures further rise, deterioration of the axial and flexural stiffness of the composite beam as a whole gradually leads to the decrease of the stress in the cable strands.



(a) Stress in the cable strands of PCBCW-1

(b) Stress in the left cable strands of PCBCW-2



(c) Stress in the right cable strands of PCBCW-2

(d) Stress in the cable strands of PCBCW-3

Fig. 10 Prestress evolution with temperature for PCBCWs

The different tendencies of the stress shown in Fig. 10 (a), (b) and Fig. 10 (c), (d) are mainly due to the different rise rates of temperature. For example, the temperature at cable strands in PCBCW-1 rises fast due to the lack of insulative concrete as in PCBCW-2 and PCBCW-3 (Fig. 8), which leads the thermal expansion strain to be larger and the mechanical properties to deteriorate faster than that in PCBCW-2, and PCBCW-3. This is the reason why the stress in Fig. 10 (a) and (b) decreases continually,

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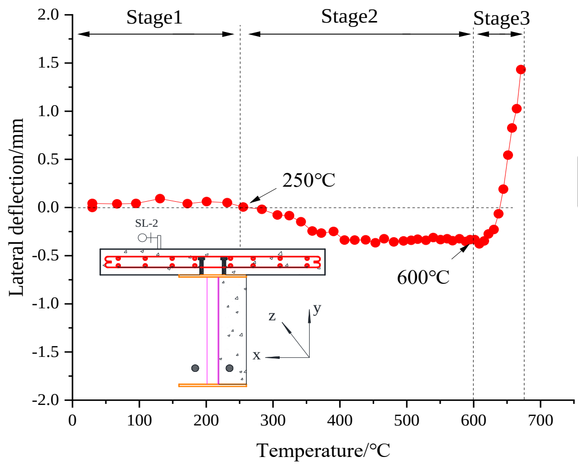
while the prestress in PCBCW-2 and PCBCW-3 experienced rising stress in the beginning stage and suddenly falling at the same temperature.

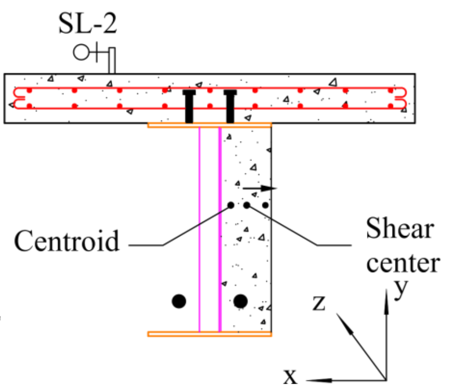
3.2.3 Lateral deflection of PCBCW-2 in the horizontal plane

Among the three composite beams, the primary difference between PCBCW-2 from PCBCW-1 and PCBCW-3 is in its asymmetric cross section. In the cross section of PCBCW-2, the unilateral concrete is attached to its web, which makes results in noncoincidencing of the shear center and centroid of the cross section, leading to the generation of a torque around its longitudinal axis. In addition, the shear center moves towards the lateral concrete when subjected to a temperature gradient along the width of the section. Noting that the higher the temperature, the lower the shear strength of the material; thus, the shear center of the section moves towards the lateral concrete, whose strength undergoes less deterioration than that of the web when exposed to fire.

The evolution of the lateral deflection of PCBCW-2 with temperature is shown in Fig. 11. It can be seen that the lateral deflection has different directions during the test, and the deflection curve can be divided into three stages, namely 1, 2, and 3. As seen in Fig. 11 (a), from ambient temperature to 250°C, the horizontal deflection at the mid-span is very small.



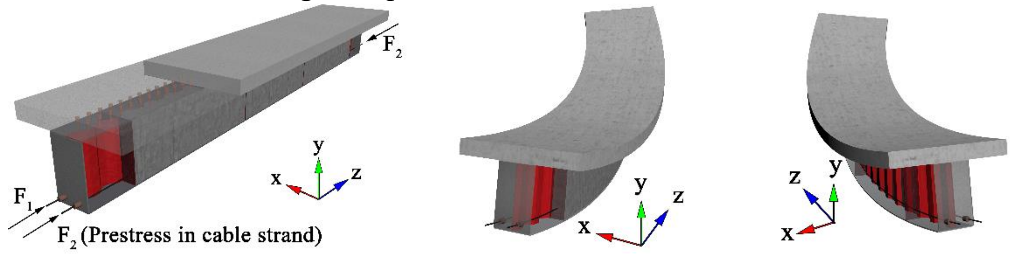
(a) Lateral deflection of PCBCW-2



(b) Centroid and shear center of the cross section of PCBCW-2

Fig. 11 Horizontal lateral deflection of PCBCW-2 at mid-span

At 250-600°C, the horizontal deflection increases with temperature and is convex to the negative *x*-axis (Fig. 11 (a) and (b)). This is due to the following facts: Firstly, the temperature of the left flanges and the web of the steel beam rises faster than that of the right side (reinforced concrete), which reflects the better insulation properties of concrete. This results in the nonuniform temperature distribution of the cross section. Consequently, a significant thermal gradient occurs along the width of the cross section, which makes the beam deflected to the right (Fig. 12 (b)). Secondly, the pretension in the left and right cable strands produces different deflection effects in the horizontal plane. Namely, the tension in the left cable strand causes the convex of the PCBCW to the right, while the tension in the right cable strand curves the beam to the left at room temperature (Fig. 12 (b) and (c)). This convex effect of the cable strand disappears when the cable strand breaks at high temperatures.



(a) prestress in two cables (b) convex to the right (c) convex to the left

Fig. 12 Effect of prestress in the cable strands on two sides of PCBCW-2

After 600°C, the deflection continues to decrease and, finally, moves towards the positive direction of the *x*-axis (Fig. 11 (a) and Fig. 12 (c)). This can be explained by the following observations. Firstly, the temperature of the web rises faster than that of the encased concrete due to its different thermal properties. This makes the shear strength of the web decay more rapidly than that of the encased concrete at high temperatures, therefore, which makes the shear center of the cross section moves towards the encased concrete, and the external load generates greater torque with respect to the shear center (Fig. 11 (b)). This torque around the longitudinal axis of the PCBCW results in a larger horizontal deflection. In addition, the bending moment generated from the left cable strand can be ignored due to the little prestress in the cable strand (Fig. 10 (b)).

In sum, the combined effects of thermal and mechanical loading affescts the shear center movement of the cross section and the horizontal temperature gradient results in the fluctuation of horizontal deflection during the fire tests.

3.3 Comparison of failure patterns of PCBCW-2 with other types of PCBCWs

The failure patterns of PCBCW-2 are compared agansit PCBCW-1 and PCBCW-3 in Fig. 13. It can be seen that there are many cracks on the side of the lateral concrete of PCBCW-2, while there is minimal cracking on the side of the lateral create of PCBCW-3. This is due to the existing horizontal moment in PCBCW-2, which makes the web concrete slab crack in the weak cross section with the transverse stiffeners (see Fig. 7 (f)), while there is not a horizontal moment in PCBCW-3 as discussed in Section 3.3.

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On the other hand, there exists buckling on the web of PCBCW-1. This is because the web of PCBCW-1 was not supported by the vertical concrete slab, while the webs of PCBCW-2 and PCBCW-3 were supported by laterally reinforced concrete. In addition, with the faster rise of temperature than that of PCBCW-2 and PCBCW-3, the elastic modulus of the web deteriorated, which adversely affected the stability of the beam. These two combined effects above cause the buckling of the web of PCBCW-1 in the shear-bend portion at high temperatures (Fig. 6 (c)).

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(a) Failure model of PCBCW-2 (obverse side)

(b) Failure model of PCBCW-2 (reverse side)

(c) Failure model of PCBCW-1

(d) Failure model of PCBCW-3

Fig. 13 Comparison of failure modes of PCBCWs

4. Numerical simulations of PCBCW-2

PCBCW-2 shows a different mechanical performance from that of PCBCW-1 and PCBCW-3 at high temperatures. To further investigate the uniqueness of this specimen, a series of new numerical models created in ABAQUS were developed to analyze the variation of temperature, vertical and horizontal deflection, and the change of prestress in the cable strands with temperature.

4.1 FE model

To mimic the performance of PCBCW-2, sequential coupled models, namely, thermal models and mechanical models, are developed in ABAQUS, in which the thermal output is input to the structural model to calculate the responses of the tested beams at high temperature.

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The details of the numerical model are as follows. In the thermal analysis, the upper concrete slab, the steel beam, and the cable strands are modeled by DC3D8, DS4, DC1D2[36], respectively. The interpolating function, the integration methods, and the number of DOF for each finite element can be seen in reference[36]. The elements of the concrete slab and the elements of the upper flange of the steel are tied in ABAQUS models. In the thermal analysis, the thermal model is exposed to fire on three sides, namely, the bottom flange, the bottom surface of the concrete slab, and two sides of the web. The furnace temperature is applied and follows the curve of the actual average temperature in the presented tests. In the thermal analysis, the convection coefficient for exposed surfaces and unexposed surfaces is 25 W/(m·℃) and 4 W/(m·℃), respectively. Stefan-Boltzmann's radiation coefficient and emissivity for both the steel beam and the slab are 5.669×10^{-8} W/(m^2·K^4) and 0.7, respectively.

In the consequent structural analysis, the upper concrete slab, the steel beam, and the cable strands are modeled by C3D8R, S4R, T3D2[36], respectively. The corresponding elements in the structural and the thermal models have identical dimensions (i.e., mesh) to avoid temperature interpolation. The interface between the bottom surface of the top concrete slab and the upper flange is connected via contact elements CONN3D2[36] along the axis of the beam. Normal to the interface, the elements of the top concrete slab and the elements of the upper flange remain in hard contact. Namely, there is not any relative normal displacement between the top concrete slab and the upper flange. The shear-slip relationship is expressed by the Cartesian-Align method[37]. Similarly, the interface between the concrete slab and the web was dealt with using the same method mentioned above.

The prestress in the cable strands is applied by using the cooling temperature method. In this method, the temperature of the cable strands can change according to the target stress in the cable strands, and the rest of the composite beam is kept constant. The cooling temperature of the cable strands was determined based on a rule of equivalent stress. Namely, the shrinkage stress in the cable strands is equal to the target prestress in the specimens. When the prestress in the cable strand reaches the targeted stress, the temperature of the entire model of the PCBCW is restored to ambient temperature to avoid influencing actual mechanical analysis by ABAQUS at high temperature. On the other hand, the nodes of the cable strands inside the concrete are tied to the adjacent nodes of the concrete in the thermal analysis. In the mechanical models, temperature-dependent constitutions of concrete and steel from the code[34, 38] are utilized. During the analysis, when the cable strands fracture, the restart function is used to continue the mechanical analysis after the elements corresponding to the fractured cable strands are dealt with.

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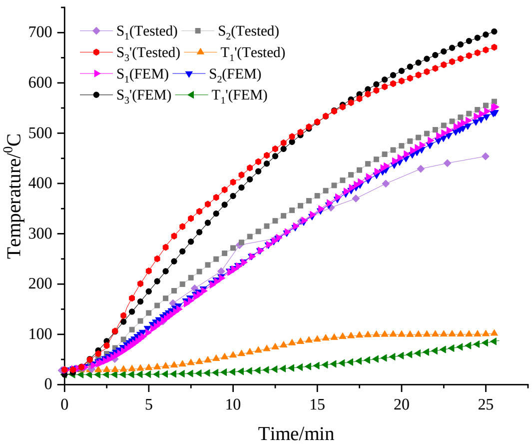
4.2 FE simulation

4.2.1 Thermal and mechanical responses

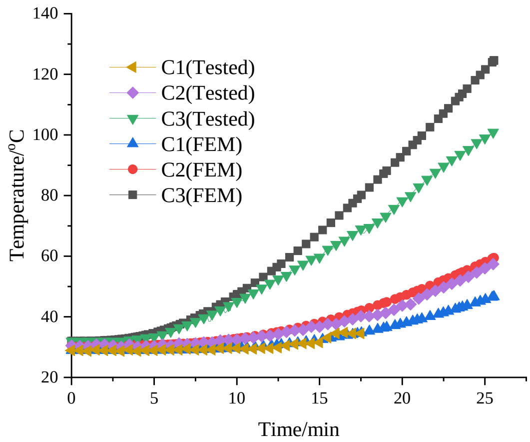
In this section, the thermal and mechanical evolutions of PCBCW-2 are presented to analyze its performance.

The simulated temperatures at the steel beam and the top concrete slab of PCBCW-2 versus time are presented in Fig. 14 (a) and Fig. 14 (b). Comparing the tested temperature and the simulated temperature shows that there are slight differences between the simulated temperatures and the measured points. This is due to the fact that the furnace temperature in the numerical model is taken as the average furnace temperature along the length of PCBCW-2, while the real furnace temperature is not uniform along the length of the beam. This difference in the input of the furnace temperature leads to a larger difference in the final stage. To facilitate understanding the distribution of the temperature, the typical cross-sectional temperature contour is presented in Fig. 14 (c). It can be seen that there exist two typical temperature gradients along the width and the height of the cross section.

A comparison of the evolutions of the numerical and tested deflection at the mid-span of PCBCW-2 with temperature is presented in Fig. 14 (d). It can be seen that there is a slight difference between the numerical deflection and the tested deflection before 600°C. When the temperature exceeds 600°C, the difference in these two deflections increases slightly. This may be related to the increasing temperature difference between the test temperature and the numerical simulation temperature of the composite beam, as discussed above. Meanwhile, in the numerical deflection, there is a jump point at 400°C due to the fracture of the cable strands encased in the concrete between flanges at this temperature (Fig. 14 (e)), which leads to the sudden variation of the flexural stiffness of PCBCW-2 at 400°C. In general, the numerical deflection is close to the measured deflection.



(a) Temperature at the steel beam



(b) Temperature at the top concrete slab

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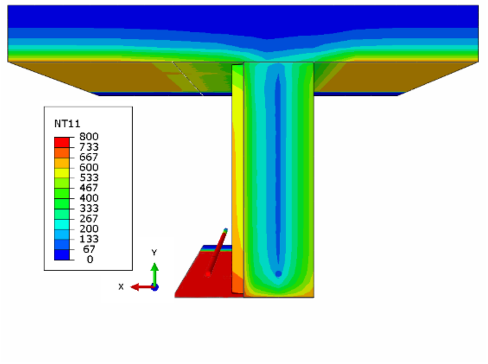
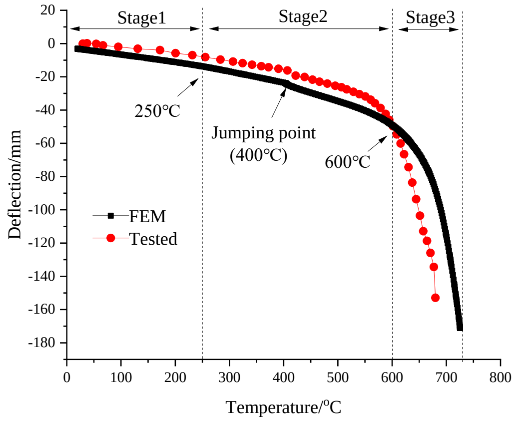
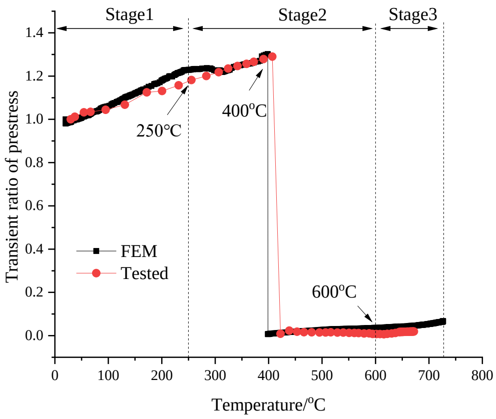
and the cable strands



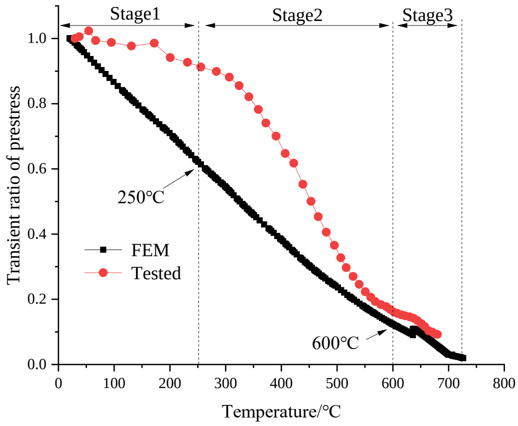
(c) Typical contour of cross-section temperature



(d) Vertical deflections at the mid-span



(e) Stress in the right cable strands



(f) Stress in the left cable strands

Fig. 14 Numerical and tested responses of PCBCW-2

The evolutions of the stress in the right cable strands in PCBCW-2 are shown in Fig. 14 (e). It can be seen that the stress in the cable strand increases with temperature continuously, and the cable strand fractured once the stress reached the tension strength at 400°C. Fig. 14 (f) shows the numerical and tested stress in the cable strands on the left of the web (Fig. 3 (e)), which demonstrates the FEM and measured stress decrease with the rise of the temperature. However, the two stresses redcuced at different rates, although they almost coincide at the critical temperature. This may be due to the fact that the actual temperature of the furnace is not uniform along the longitudinal axis of PCBCW-2 during the test, while in the FE model the furnace temperature is processed according to to completely uniform temperature distribution along the longitudinal axis of PCBCW-2. Therefore, the actual difference of temperature along the axis of PCBCW-2 results in the difference between the simulated and tested stress.

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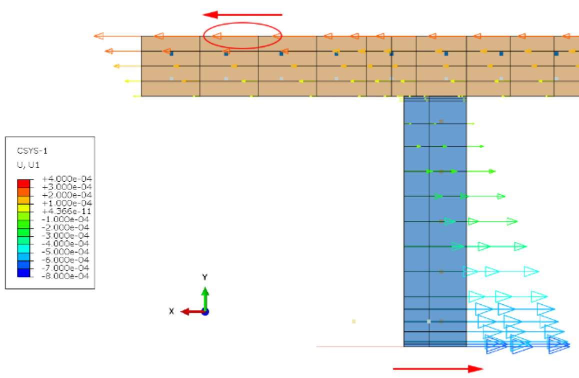
4.2.2 Mechanism analysis of horizontal deflection

To investigate the evolution mechanism of PCBCW-2 at high temperatures, the deflection vectors in the horizontal plane are extracted at different temperatures. Typical responses, such as lateral deflection (in *x* direction) vector diagram of PCBCW-2 at different temperatures, are shown in Fig. 15. As seen from the figure, there is lateral deflection at mid-span, and the direction and magnitude of the lateral deflection change constantly at different temperatures. In Fig. 15 (b), the arrow in the top concrete slab shows a relative deflection to Fig. 15 (a).

This evolution of the horizontal deflection can be explained as follows. Firstly, due to the asymmetry of the cross section, the sectional shear center of PCBCW-2 is on the right side of its web, while the external load (see Fig. 7 (e)) is applied along the middle line of the top concrete slab, not passing through the shear center of the section and resulting in torsion around the shear center (Fig. 11 (b)). Secondly, due to the asymmetry of the cross section and the heat absorption of the concrete on the right side, the temperature of the concrete on the right side of the web is lower, and the temperature of the web is higher. These factors cause PCBCW-2 to deflect on the side with the lower temperature (Fig. 16 (a)). In addition, due to the asymmetry about the *y*-axis of PCBCW-2, the prestressed efficiency of the cable strands on both sides of the web is different[8]. This difference results in horizontal deflection towards the side of the cable strand with higher prestress.



(a) PCBCW-2, T=20℃

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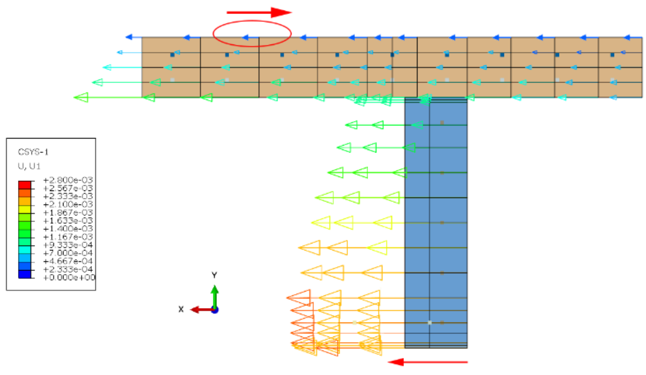
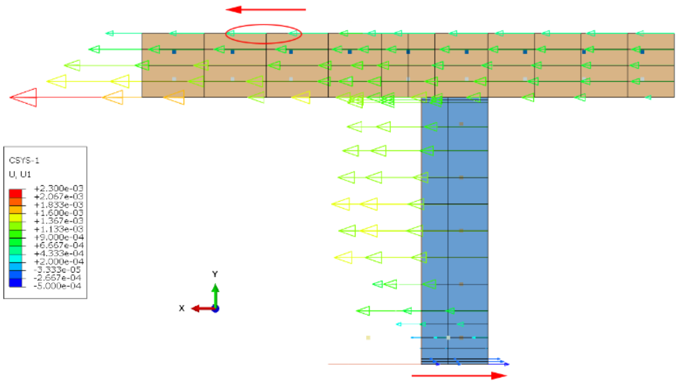


(b) PCBCW-2, T=400℃



(c) PCBCW-2, T=700℃

Fig. 15 The vector diagram of lateral deflection at mid-span

Next, the mechanism of Fig. 15 (a)~(c) is discussed in details. For Fig. 15 (a), at 20℃, the upper concrete plate of the composite beam has a left deflection, and the steel beam has a right deflection, which is mainly caused by the counterclockwise torque generated by the external load not passing through the shear center. At 400℃, the upper concrete slab moves to the right, and the steel beam moves to the left relative to the state at 20℃. This is mainly because the tension in the right cable gradually increases with the rise in temperature (Fig. 14 (e)), while the tension in the left cable gradually decreases (Fig. 14 (f) and Fig. 16 (b)). As a result, such unbalanced tension makes the composite beam shift to the left (Fig. 15 (b)). At 400-700℃ (Fig. 15 (c)), the tension in the cable strand on the left side of the web decreases gradually, while the breaking tension in the cable strand on the right side of the web decreases to zero at 400℃. This variation of the tension alleviates the unbalance tension of the cable on the left and right sides of the web, so the resulting horizontal deflection decreases (Fig. 16 (c)). On the other hand, with the rise in temperature, the temperature gradient of the cross section along the width of the cross section gradually decreases, which also reduces the horizontal deflection (Fig. 15 (c)).

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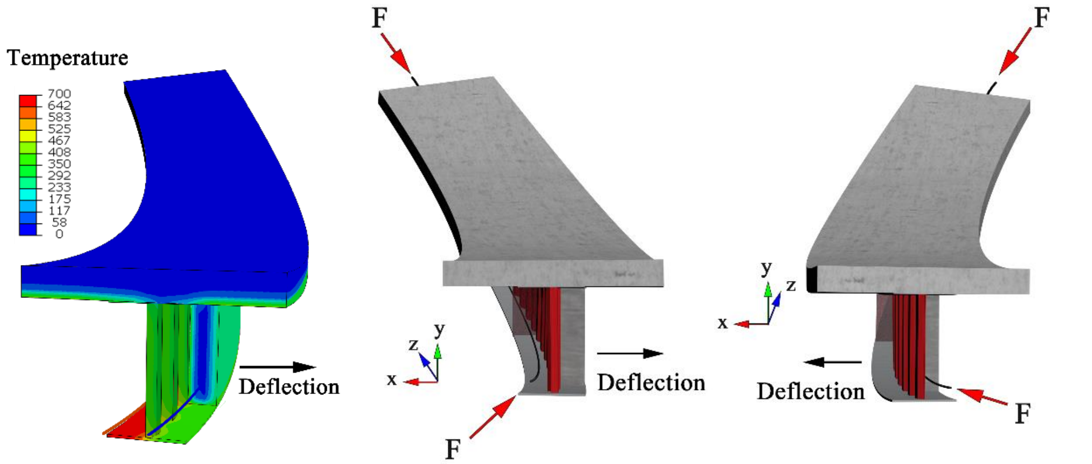

(a) Lateral deflection by horizontal temperature gradient (b) Lateral deflection by left tension (c) Lateral deflection by right tension

Fig. 16 Horizontal deflection by temperature gradient and prestress

The following sums our findings. For composite beams unilaterally concreted between flanges, there is torsion around the longitudinal axis of PCBCWs because the applied load does not pass through the shear center of the section. Due to the temperature gradient along the width of the section and the different efficiency of prestress on the two sides of the cable strands, the section has deflection in the horizontal plane, and the deflection changes with increasing temperature.

The numerically and experimentally relative deflection-temperature curves of PCBCW-2 are shown in Fig. 17, in which the relative horizontal deflection takes the position as the reference point at ignition time rather than the real deflection generated after the application of external load so as to be consistent with the deflection obtained by the test operation.

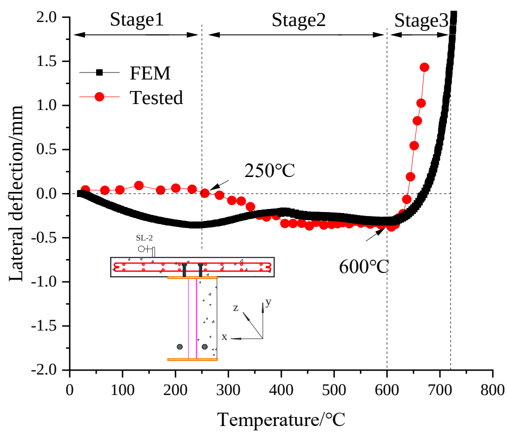

Fig. 17 Numerical and tested lateral deflection of PCBCW-2

It can be seen that the lateral deflection curves of numerical simulation and measurement are basically consistent. Only at 20-250℃, there is little difference between the simulated curve and the tested curve. This is mainly because in the test, the cable strand on the right is tensioned first, and then the cable strand on the left is tensioned, which causes PCBCW-2 to first produce the lateral deflection in x direction. Relatively, in the numerical model, prestressing is applied to both sides of the cable simultaneously, while the prestressing efficiency of the left cable is higher[8], which makes PCBCW-2 produce negative deflection in $-x$ direction.

From Fig. 17, it can be seen that horizontal lateral deflection develops slowly until 650℃, which is related to the gradual development of temperature gradient and the development of the relative tension of the cable on both sides. Also, after 650℃, the lateral movement develops rapidly. This is mainly due to the increasing temperature gradient along the width of the section (Fig. 8 and Fig. 14).

5. Conclusions

The performance of prestressed steel-concrete composite beams with corrugated webs concreted between flanges under standard fire conditions has been investigated experimentally and numerically. The conducted tests and analysis show that PCBCWs without encased concrete, with one-side encased concrete, and with two-side encased concrete have different failure patterns. The following general conclusions can also be drawn:

(1) At the ultimate state, the three types of tested PCBCW-1~PCBCW-3 beams, namely, PCBCWs with encased concrete or without encased concrete around the webs, undergo different vertical deflections. PCBCW-1 has the largest deflection, while PCBCW-3 has the least deflection at the critical states.

(2) For the PCBCW-2 (with one-side encased concrete between flanges), there is significant horizontal deflection, and the direction of this deflection changes with the rise of temperature due to asymmetric tension of the cable strands, the distinct temperature gradient in the horizontal plane, and the generation of torque around the axis of the beam.

(3) The cross-section shear center of PCBCW-2 moves to the coldest side, during the heating process.

(4) The fracture of the cable strands inside the encased concrete led to the failure of the beams and the reverse of the horizontal deflection of PCBCW-2.

(5) PCBCW-1, without encased concrete between flanges, reaches its critical state in the form of buckling of its web. In contrast, the failure modes of PCBCW-2 and PCBCW-3 fail without the instability in the web.

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