Experimental Study on Flexural Performance of Partially Precast Steel Reinforced Concrete Beams

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Abstract: An innovative partially precast steel reinforced concrete beam, which is abbreviated here as PPSRC beam, is presented in this paper. An improved type, an innovative hollow-core PPSRC beam, which is abbreviated HPSRC, is also introduced here. These two types of beams both have two parts, the precast outer-part and the cast-in-place inner-part. To investigate the flexural performances of the PPSRC beam and the HPSRC beam, static loading experiments were carried out on seven specimens, including three PPSRC T-beam specimens and three HPSRC T-beam specimens together with a cast-in-place SRC control T-beam specimen. In the tests, the flexural performances of the PPSRC beams and the HPSRC beams were emphasized, and the effect of the compressive strength of concrete in the inner-part was also taken into account. On the basis of the experiments, the failure mode, strain development and ultimate bearing capacity were thoroughly examined. The flexural failure mechanisms of the PPSRC beams and the HPSRC beams were clearly revealed. From comparison of the test results, the flexural performance of the HPSRC beams was similar to that of the PPSRC beams and was even comparable to that of the cast-in-place SRC beam. The calculation method for flexural capacity was also presented in this paper. The analysis of mechanical behavior and the design method can be used for the design of these innovative composite beams and provide a significant foundation for further research.

Keywords: steel reinforced concrete structure; precast steel reinforced concrete beam; flexural performance; flexural capacity; sagging bending moment; experimental study

1. Introduction

Steel reinforced concrete structures (SRCs), as an important part of steel-concrete composite structures, have been widely applied in large-span or high-rise structures due to some favorable characteristics, such as high bearing capacity, great stiffness and outstanding ductility performance [1-4]. However, because of the complexity of the construction process [5, 6], the use of cast-in-place SRC structure has been limited in conventional residential buildings. The application of precast concrete structures has increased because of the efficiency and high quality of these structures. Precast concrete can also provide a better solution to the problem of wasted resources for on-site activities [7, 8].

To solve the problem caused by deficiencies in constructability and to promote the application of SRC structures, some researchers have suggested the benefits of combining SRC members and precast concrete members. For example, the Fujita steel and reinforced precast concrete system (FSRPC) has been widely adopted in Japan [9]. The entire precast SRC structure seems to be a way to reduce the labor cost on site and improve quality control. However, there are still some flaws in the entire precast SRC structure, such as heavy weight and vulnerability in the joint area. Because of the heavy loads and larger dimensions, special large vehicles and cranes have to be employed in the process of transportation, lifting and installation, which is difficult to accomplish and unaffordable for some builders. Furthermore, it is difficult to ensure integrity of the beam-column joint.

As an alternative method that can reduce the weight of precast SRC members, the partially precast or prefabricated steel reinforced concrete (PPSRC) beam method can be used. Hong et al. [10, 11] created a new type of PPSRC structure and named it MHS (modularized hybrid system). In the reference paper [12, 13], the PPSRC method has been proven to be a feasible solution to the problems and shortages of cast-in-place SRC structures through a series of experiments on the mechanical behaviors of PPSRC beams. However, the optimal combination mode and the structural integrity in the partially precast member must be further investigated for better structural capacity and constructability.

On the construction site, for convenience, concrete in beams is cast with the slab at the same time and in the same grade. It is not necessary to apply high-strength concrete in the floor slab; therefore, waste occurs because the beams subjected to heavy loads require high-strength concrete. As an alternative solution to the problem, an innovative partially precast steel reinforced concrete beam is presented in this paper. As illustrated in Fig. 1, this kind of PPSRC beam consists of two main parts, the outer-part and the inner-part. The U-shaped outer-part is composed of high-strength concrete, a steel shape, and reinforcing bars as well as stirrups that can be prefabricated in the factory. After lifting and installing the precast outer-part on site, cast-in-place conventional-strength concrete fills in the web core and concrete flange meaning the inner-part is cast on site. The two parts are then joined together and behave structurally as an integral member. To enhance the bond performance between the steel shape, precast concrete and cast-in-place concrete, some special measures are adopted besides the inherent bond of the interface. First, the flanges of the steel shape and the reinforcing bars as well as the stirrups prevent vertical separation between the outer-part and inner-part. A series of diaphragms are specially arranged in the outer-part that can be made of reinforced concrete plates, steel reinforced concrete plates or steel plates. These diaphragms are large shear connectors employed not only to develop shear stress transfer in the interface between the two parts but also to enhance flexural rigidity during the construction stage. A reusable rubber formwork is employed here to prefabricate the outer-part efficiently.

Fig. 1. Schematic diagram of the PPSRC beam

To simplify the construction method, an innovative hollow core PPSRC beam, which is based on the PPSRC beam and abbreviated as HPSRC beam, is illustrated in Fig. 1(b). In HPSRC beams, the hollow section can be fabricated by foam formwork, which would stay in the web core after casting the precast outer-part instead of being removed. The foam formwork would be kept in the core of the cross section permanently after casting the inner-part and the adjacent slabs. Because of a weakness in strength and stiffness, the foam formwork cannot bear or transfer any kind of stress, and it is actually a structural hollow core in the cross section of the HPSRC beam.

The PPSRC members can be regarded as the best way to combat the deficiencies of cast-in-place and entire precast SRC members. Weight can be reduced due to the lightweight precast outer-part, and the cost of labor and time on site can be cut down because there is no need to remove the foam formwork and fieldwork, such as reinforcing bar placement and formwork, is eliminated. Fireproofing and heat preservation of the HPSRC beams will be further improved because of the use of insulating foam formwork. The precast outer-part can be made from recycled concrete and lightweight concrete, which would keep pace with sustainable development.

To investigate the flexural behavior of the PPSRC beams and the HPSRC beams, a series of experiments were carried out where both specimens were tested with a four-point test setup. The specimens were all designed as T-beams and sorted into two series, the PPSRC series and the HPSRC series. In this paper, the effects of the construction method, section form and compressive strength of concrete are critically examined. Based on the experimental observations and test results, the analytic model and design method are presented and revised later in this paper.

2. Test program

2.1 Test specimens

Seven composite beam specimens were designed and constructed, including three PPSRC beams, three HPSRC beams and a cast-in-place SRC control specimen. The key parameters are summarized in Table 1. These seven specimen beams were designed to investigate flexural performance and were T-beams with concrete flanges to simulate the slab effect caused by adjacent floor slabs on both sides. These seven specimens were sorted into the PPSRC series and the HPSRC series.

2.1.1 cross sections

These seven T-beam specimens were similar in cross section form and dimensions, except for specimens of the HPSRC series with a hollow core. As illustrated in Fig. 2, the height of the cross section was 300 mm, and the height and the width of the web were both 200 mm. The dimensions of the concrete flange of width and thickness were 880 mm and 100 mm, respectively. In the specimens of the HPSRC series, the width and the height of the hollow core were both 120 mm. The steel shape in all specimens was HN175×90×5×8 of Q235 grade according to the Chinese standards, the total height and the width of the steel shape were 175 mm and 90 mm, respectively, and the thickness of its web and flanges were 5 mm and 8 mm, respectively. All the reinforcing bars and stirrups of the specimens are illustrated in detail in Fig. 2. In the PPSRC specimens, a series of reinforced concrete diaphragms were employed, which are shown in Fig. 1(d). The thickness of the reinforced concrete diaphragms was 150 mm, and the spacing was approximately 500 mm. As a structural component in the outer-part, the reinforced concrete diaphragms were precast at the same time as the prefabricated U-shape outer-part.

(a) The cross section of PPSRC beams

(b) The cross section of HPSRC beams

Fig. 2. Schematic diagram of beam specimens (unit: mm)

2.1.2 construction methods

All the PPSRC specimens were constructed in two steps, which are shown in Fig. 1. In the first step, the outer-part of the specimens was molded and precast in the factory. The outer-part was composed of the steel shape, high-strength concrete, reinforcements and stirrups as well as reinforced concrete diaphragms. A foam formwork was adopted to facilitate the prefabrication of the outer part as illustrated in Fig. 3. The second step was started after the precast concrete hardened, and the inner-part including the web core and concrete flange was cast in this step. During this step, the foam formwork would stay permanently in the specimens of the HPSRC series, while it would be removed in the specimens of the PPSRC series; no other shoring work or formwork was needed. The SRC control specimen was constructed in step two; all concrete in the SRC specimen was cast in place.

(a) The outer-part skeleton before casting

(b) The outer-part after casting

Fig. 3. Construction of the PPSRC beam specimens

2.1.3 materials

Steel shapes of all the specimens were the same grade, which was HN175×90×5×8. For the flange of the steel shape, the tested tensile strengths were 273 MPa at yield and 450 MPa at failure. For the web of the steel shape, the tested tensile strengths were 262 MPa at yield and 436 MPa at peak. The tested tensile strengths of the stirrup, $\Phi 6$ bars with a diameter of 6 mm and a grade of HRB335, were 387 MPa at yield and 545 MPa at peak. The tested tensile strengths of the longitudinal reinforcement, $\Phi 18$ bars with a diameter of 18 mm and a grade of HRB335, were 420 MPa at yield and 578 MPa at peak.

The compressive strengths of the concrete in the outer-part and the inner-part of the PPSRC specimens were designed to be different. For the seven specimens, the strength grades of the concrete in the outer-parts were identical, which was C60 graded per the Chinese code, and the tested compressive strength was 54.0 MPa at 28 days. The strength grades of concrete in the inner-parts were designed at three levels as C20, C40 and C60, and the tested compressive strengths at 28 days were 21.7 MPa, 38.1 MPa and 68.0 MPa, respectively. The compressive strength, f_c , of the concrete was calculated as 0.8 times the cubic strength f_{cu} in the following discussion. All of the configurations of the specimens and the condition of the reinforcements are summarized in Table 1.

Table 1.
Matrices of test Specimens

Series No.	Specimen ID	Specimen length l (mm)	Effective depth of section d (mm)	Shear span length a (mm)	Shear span-to-depth ratios a/d	Cubic strength of outer-part concrete $f_{cu,out}$ (MPa)	Cubic strength of inner-part concrete $f_{cu,in}$ (MPa)	Test setup
Control specimen	SRC-1	2200	270	810	3.0	-	68.0	Four-point test
PPSRC	PPSRC-1	2600				54.0	21.7	
	PPSRC-2	2600	270	810	3.0	54.0	38.1	
	PPSRC-3	2600				54.0	68.0	
HPSRC	HPSRC-1	2200				54.0	21.7	
	HPSRC-2	2600	270	810	3.0	54.0	38.1	
	HPSRC-3	2600				54.0	68.0	

2.2 Test setup and instrumentations

In the experiments, all the specimens were tested on an electro hydraulic servo-testing machine with a maximum capacity of 5000 kN. A four-point test procedure was used in which the specimens were subjected to the sagging moment and the concrete flange was placed upward. As illustrated in Fig. 4, the shear span-to-depth ratio was 3.0 for all specimens.

Static loading was applied at the loading point by the electro hydraulic servo-controlled jacks, as shown in Fig. 4. During the test process, all of the deflections at the central point, loading point and two supports were measured using linear variable differential transformers (LVDTs). A considerable number of electrical-resistance strain gauges were arranged on the web and flanges of the steel shape and the reinforcing bars to monitor the strain response. Two sets of strain gauges were vertically set along the height of the cross section on both sides of the specimens at the mid-span point to verify the plane section hypothesis. The layout of the LVDTs is presented in Fig. 4 in detail, and the layout of the strain gauges is illustrated in Fig. 5.

(a) Schematic diagram

(b) Layout of LVDTs

(c) Photo

Fig. 4. Schematic diagram and photo of test setup (unit: mm)

(a) Layout of strain gauges on the steel shape

(b) Layout of strain gauges on the concrete

(c) Layout of strain gauges at the top surface of concrete

Fig. 5. Schematic diagram of the strain gauges of the specimens (unit: mm)

3. Test results

3.1 Failure modes and damage patterns

The failure modes and damage patterns of the specimens are shown in Fig. 6. As illustrated in Fig. 6, all the specimens failed in the typical flexural failure mode, with the yielding in the bottom flange, the web of the steel shape at the tension zone and crushing in the concrete flange. No major difference was observed between the failure modes of the PPSRC specimens and the HPSRC specimens. As attributed to the encased steel shape, all the failure modes were ductile and accompanied with excellent deformation capacity and ductility.

The failure modes of the specimens of the PPSRC series and the HPSRC series were similar to that of the cast-in-place SRC specimen. For example, as illustrated in Fig. 6, specimen SRC-1, specimen PPSRC-3 and specimen HPSRC-3, which had the same dimensions, the same material and the same shear span length, all failed with an almost identical failure mode. Two specimens were cut open after the test, and there was no slippage observed on the interface between the steel shape and the concrete. It was also hard to distinguish the precast outer-part from the cast-in-place part. This indicated that the inner-part and the outer-part were bonded tightly to each other, and the contribution of concrete diaphragms was responsible for the outstanding bond performance. The strain results show that the steel flange and the steel web at the flexural span had almost yielded when the specimen approached its limit. The mechanical performances of both the PPSRC specimens and the HPSRC specimens were proven as expected.

Fig. 6. Typical failure mode and damage patterns of specimens

3.2 Bending moment-curvature curves

Fig. 7 shows the bending moment-curvature curves at the mid-span point of the specimens, where the curvature was calculated by dividing the mid-span deflection value by the length of half of the span. From the bending moment-curvature curves, it should be noted that the initial stiffnesses of the PPSRC beams and the HPSRC beams were comparable to that of the SRC beam. The flexural performances of the specimens of the HPSRC series was similar to those of the specimens of the PPSRC series, in which flexural capacities were just slightly higher. From the moment-curvature curves, the ductility ratios of the specimens were calculated as listed in Table 2. The ductility ratio was calculated as the result of the ultimate mid-span deflection Δ_u divided by the yielding mid-span deflection Δ_y , and the ultimate mid-span deflection Δ_u was defined as the deflection of the specimen when its load descended to 85 percent of the peak load during the failing stage. The yielding mid-span deflection Δ_y was calculated from the load-deflection curves of the specimen and based on a recommended method [14]. As illustrated in Fig. 7, a secant was drawn to intersect the load-deflection curve at P_{max} (point A), and point B was the projection of point A onto the load-deflection curve. The line OB was extended to the intersection with a horizontal line corresponding to P_{max} (point C), and the projection of point C onto the load-deflection curve was regarded as the yield point (point D). From the calculated ductility ratios listed in Table 2, all ductility ratios of the specimens are relatively high, therefore the PPSRC beams and the HPSRC beams both exhibit outstanding deformability.

Fig. 7. The moment-curvature curves at mid-span of the specimens

Fig. 8. Definition of yield displacement

3.3 Validation of section assumption

For all the specimens, the flexural failure mode, characterized by crushing of the concrete and yielding of the steel shape at the mid-span, was observed in accordance with its design for flexural critical performance. As a conventional procedure, a set of strain gauges was arranged along the entire height of the section to verify the section assumption. Fig. 9 shows the strain distribution curves of the steel shape in the specimen SRC-1, PPSRC-2 and HPSRC-2, and the result agrees with the theory, in which strain is linearly distributed along the height of the steel shape throughout the experiment. Furthermore, it could be observed that structural steel yielded in most part of the flange and the web.

Fig. 9. Strain distribution curves along the section height

4. Flexural capacity

From the experimental data and the bending moment-curvature curves, the results of the flexural capacities of the specimens were obtained and listed in Table 2. Some crucial factors affecting the flexural capacity are discussed in this section.

4.1 Effect of construction method

As indicated by Table 2, the flexural capacities of specimens PPSRC-3 and HPSRC-3 were 190.28 kN·m and 188.56 kN·m, respectively, while that of specimen SRC-1 was 197.38 kN·m. The flexural capacities of the PPSRC specimen and the HPSRC specimen were approximately 3.60 percent and 4.49 percent lower than that of the entire cast-in-place SRC specimen, which has identical cross sectional dimensions with the PPSRC specimen and the HPSRC specimen. The concrete strengths of the precast outer-part and the cast-in-place inner part in specimens PPSRC-3 and HPSRC-3 were 54.0 MPa and 68.0 MPa, respectively, and the concrete strength of the entire cast-in-place specimen SRC-1 was 68.0 MPa. In other words, the lower material strength of the precast concrete was responsible for the slightly lower peak loads of specimens PPSRC-3 and HPSRC-3. Taking the difference of the concrete strength into account, the difference would be much smaller. From the comparison, the flexural capacities of the PPSRC beam, the HPSRC beam and the SRC beam were comparable and seldom affected by the construction method.

4.2 Effect of hollow core

There were few differences between the failure modes of the specimens, which were similar in the process of crack propagation. When concrete members are subjected to a bending moment and suffer typical flexural failure, the tensile strength of the concrete at the tension zone could be relatively small and therefore neglected. In this case, the concrete in the beam web was mostly at the tension zone, so the contribution of this section to the overall flexural capacity could be limited and further neglected even if some concrete in the beam web was at the compression zone. The reason is that the resultant compression force induced by this part was very close to the neutral axis of the cross section, which meant the contribution of this part to the flexural capacity was relatively small, attributed to the short lever arm of the resultant compression force. Therefore, based on this theory, it was an appropriate optimization to subtract or substitute the concrete in this part by foam in the specimens of the HPSRC series. As implied from the test results listed in Table 2 and illustrated in Fig. 10, the flexural capacities of the HPSRC specimens were slightly lower than those of the PPSRC specimens, and the difference would decrease with an increased concrete strength of the inner part. The flexural capacity of specimen HPSRC-1 was 8 percent lower than that of specimen PPSRC-1, and the difference between specimens HPSRC-2 and PPSRC-2 and between specimens HPSRC-3 and PPSRC-3 were 4.1 percent and 0.9 percent, respectively. Therefore, the effect of the hollow core on the flexural performance was relatively small.

Table 2.
Test results of the specimens

Series No.	Specimen ID	Load of the first flexural cracking P_{cr} (kN)	Load of the steel yielding P_y (kN)	Peak load P_u (kN)	Flexural capacity M_u (kN·m)	Yielding deflection Δ_y (mm)	Ultimate deflection Δ_u (mm)	Ductility ratios Δ_u/Δ_y
Control specimen	SRC-1	28.86	390.65	487.36	197.38	13.83	81.09	5.86
PPSRC	PPSRC -1	18.38	310.85	397.36	160.93	22.80	105.51	4.61
	PPSRC -2	21.06	310.85	416.73	168.78	19.09	97.95	5.13
	PPSRC -3	25.35	324.50	469.84	190.28	16.41	81.08	4.94
HPSRC	HPSRC-1	17.16	345.00	365.47	148.02	25.27	121.27	4.80
	HPSRC-2	14.43	308.95	399.51	161.80	15.95	93.00	5.83
	HPSRC-3	23.40	357.75	465.68	188.56	11.81	81.76	6.92

Fig. 10. Differences of the flexural capacities of specimens of two series

4.3 Effect of strength of cast-in-place concrete

Concrete strength plays an important role on the flexural performance of reinforced concrete beams as well as steel reinforced concrete beams. In this case, the compression flange width of the T-beam specimen was relatively large in the compression zone, which led to enough compressive capacity to balance the result of the tension force supplied by the steel shape and the reinforcing bars at the tension zone. The compressive strength of the concrete was supposed to influence the flexural capacity of the specimens. As indicated by Table 2 and Fig. 11, it could be concluded that concrete strength directly affected the flexural capacity, and the flexural capacity increased proportionally to increasing concrete strength but in a mild way with limited room for increase.

Fig. 11. Flexural capacity vs. concrete strength curves

5. Analytical work of flexural capacity

5.1 Basic assumptions

According to the test results, it is worth noting that the specimens of the PPSRC series and the specimens of the HPSRC series both behaved as integral members, which indicated that the specimens were sufficiently composite and that the steel shape worked together well with the precast outer-part concrete and cast-in-place inner-part. The plane section assumption was verified to be valid based on the measured strain results [14].

The measured strains indicated that the bottom flange at the tension zone and the web of the steel shape were almost tensed to yield. Therefore, the plastic stress theory could be applied here to calculate the flexural capacity of the specimens of the PPSRC series and the HPSRC series. In the plastic stress theory, the simple yet rational assumptions made in analyzing these tests can be summarized as follows [15-19]:

- the plane section assumption was satisfied.
- the part of the steel shape and the reinforcing bars at the tension zone were considered as tensed to yield, and the rest of the steel shape and the reinforcing bars at the compression zone were considered as compressed to yield. For simplicity, all the stresses were taken as yield stress.
- the tensile strength of the concrete was relatively small and therefore neglected.
- the stress of the concrete at the compression zone could reach its designed compressive strength, and the rectangle stress block method could be used according to ACI 318 [20].

5.2 Calculation methods

For the composite beams under flexure, the bending capacity varies with the position of the neutral axis. To simplify the design procedure, three different cases can be defined with three different positions of the neutral axis; one case of the neutral axis passing above the steel shape and two cases of the neutral axis passing through the web of the steel shape.

(a) Case 1: the neutral axis passes over the steel shape

(b) Case 2: the neutral axis passes through the web of the steel shape and the flange of the T-beam

(c) Case 3: the neutral axis passes through both the web of the steel shape and the web of the T-beam

Fig. 12. Calculation sketch for bending capacity

• Case 1, the neutral axis passes over the steel shape

When the height of the compression zone x is smaller than the thickness of the concrete cover of the steel flange h_1 , the neutral axis is supposed to pass above the steel shape. In this case, the whole steel shape is in the tension zone and tensed to yield. The stress distribution of the cross section can be simplified as shown in Fig. 12(a). Based on the basic assumptions, the height of the compression zone of concrete x and the bending capacity M_u , which can be calculated by Eq. (1) and Eq. (2), are determined from the equilibrium conditions of internal force in the cross section.

$$x = \frac{(A_s - A'_s) f_y + (A_{uf} + A_a + A_{bf}) f_a}{\beta b_c f_{c,in}} \leq h_1 \quad (1)$$

$$M_u = A_s f_y (h_0 - x) + A_{bf} f_a (h_w + h_1 + \frac{t_f}{2} - x) + A_a f_a (\frac{h_w}{2} + h_1 - x) + A_{uf} f_a (h_1 - \frac{t_f}{2} - x) + A'_s f_y (x - a'_s) + \beta b_c x f_{c,in} (x - \frac{\beta x}{2}) \quad (2)$$

• Case 2, the neutral axis passes through the web of the steel shape and the flange of the T-beam.

When the height of the compression zone x is larger than h_1 , the neutral axis is supposed to pass through the web of the steel shape. If the height of the compression zone is smaller than the thickness of the flange of the T-beam, which is labeled h_c , the neutral axis could be regarded as passing through the flange of the T-beam and the web of the steel shape as illustrated in Fig. 12(b). Based on the basic assumptions, the height of the compression zone of concrete x and the bending capacity M_u , which can be calculated by Eq. (3) and Eq. (4), are obtained from the equilibrium conditions of internal force in the cross section.

$$h_1 < x = \frac{(A_a - 2A'_a + A_{bf} - A_{uf}) f_a + (A_s - A'_s) f_y}{\beta b_c f_{c,in}} \leq h_c \quad (3)$$

$$M_u = A_s f_y (h_0 - x) + A_{bf} \left(h_w + h_1 - x + \frac{t_f}{2} \right) + (A_a - A'_a) f_a \frac{(h_w + h_1 - x)}{2} + A'_a f_a \frac{(x - h_1)}{2} + A_{uf} \left(x - h_1 + \frac{t_f}{2} \right) + \beta b_c x f_{c,in} (x - \frac{\beta x}{2}) + A_s f_y (x - a'_s) \quad (4)$$

• Case 3, the neutral axis passes through both the web of the steel shape and the web of the T-beam.

When the height of the compression zone x is larger than h_c , the neutral axis is supposed to pass through both the web of the steel shape and the web of the T-beam as illustrated in Fig. 12(c). In this case, the concrete in the compression zone consists of two parts, one part is the compression flange of the T-beam and the other part is a portion of the web of the T-beam. The height of the compression zone of concrete x and the bending capacity M_u , which are calculated by Eq. (5) and Eq. (6), are obtained from the equilibrium conditions of internal force in the cross section based on the basic assumptions.

$$h_c < x = \frac{(A_a - 2A_a' + A_{bf} - A_{uf})f_a + (A_s - A_s')f_y - b_c h_c f_{c,in}}{\beta[(b - 2c)f_{c,in} + 2cf_{c,out}]} + h_c \leq h_1 + h_w \quad (5)$$

$$M_u = A_s f_y (h_0 - x) + A_{bf} \left(h_w + h_1 - x + \frac{t_f}{2} \right) + (A_a - A_a') f_a \frac{(h_w + h_1 - x)}{2} \\ + A_a' f_a \frac{(x - h_1)}{2} + A_{uf} \left(x - h_1 + \frac{t_f}{2} \right) + b_c h_c f_c \left(x - \frac{h_c}{2} \right) + A_s f_y (x - a_s') \\ + \left(\beta - \frac{\beta^2}{2} \right) (x - h_c)^2 [(b - 2c)f_{c,in} + 2cf_{c,out}] \quad (6)$$

All the index and variables in the above equations are defined as follows.

- x - height of compression zone of concrete;
- c - thickness of U-shaped precast concrete;
- b - web width of composite beams;
- b_c - slab width of composite beams;
- h_0 - distance from extreme compression fiber to centroid of longitudinal tension reinforcement;
- h_c - slab height of composite beams;
- h_1 - distance from extreme compression fiber to bottom edge of upper flange of steel shape;
- a_s' - distance from extreme compression fiber to centroid of longitudinal compression reinforcement;
- t_f - flange thickness of steel shape;
- t_w - thickness of steel shape web;
- h_w - height of steel shape web;
- A_s - area of longitudinal tension reinforcement;
- A_s' - area of compression reinforcement;
- A_{uf} - area of top flange of steel shape;
- A_{bf} - area of bottom flange of steel shape;
- A_a - area of steel web;
- A_a' - area of web of steel shape above neutral axis;
- f_a - yield strength of steel shape;
- f_y - yield strength of reinforcing bar;
- $f_{c,out}$ - compressive strength of outer-part concrete;
- $f_{c,in}$ - compressive strength of inner-part concrete;
- M_u - calculated flexural capacity.
- β - factor relating depth of equivalent rectangle compressive stress block to neutral axis depth.

5.3 Validation

Based on the calculation methods and equations established above, the flexural capacities of the specimens of the PPSRC series and the HPSRC series were calculated as listed in Table 3. The flexural capacities of the specimens were meanwhile determined based on the AIJ-SRC [21] by Eq. (7), where M_a and M_{rc} are the flexural capacity of the steel beam and the flexural capacity of the reinforced concrete beam, respectively, and the results are listed in Table 3. As indicated by Table 3, the calculated results based on the method proposed in this paper are much more reasonable than those based on the method recommended by the Japanese specification, which is a little simpler but excessively conservative. Therefore, the proposed formulas provide a reliable means of predicting the flexural capacities of these innovative composite beams in practice, with the consideration of safety and conservation.

$$M_u = M_a + M_{rc} \quad (7)$$

Table 3.

Comparisons of the calculated results of the specimens

Series No.	Specimen ID	Tested flexural capacity M_u^e (kN·m)	Calculated results by methods suggested M_u^{c-I} (kN·m)	M_u^{c-I}/M_u^e	Calculated results by methods of Japanese specification M_u^{c-J} (kN·m)	M_u^{c-J}/M_u^e
Control specimen	SRC-1	197.38	154.13	0.78		0.45
PPSRC	PPSRC -1	160.93	128.93	0.80	88.76	0.55
	PPSRC -2	168.78	136.65	0.81		0.53
	PPSRC -3	190.28	154.13	0.81		0.47
HPSRC	HPSRC-1	148.02	128.93	0.87		0.60
	HPSRC-2	161.80	136.65	0.85		0.55
	HPSRC-3	188.56	154.13	0.82		0.47
Average value				0.82		0.52
Coefficient of variation				0.04		0.10

6. Conclusions

To reduce weight and enhance the structural integrity of large PSRC beams, an innovative partially precast steel reinforced concrete beam and a further improved type, a hollow core partially precast steel reinforced concrete beam, are proposed in this paper. A series of experiments on PPSRC T-beam specimens and HPSRC T-beam specimens thoroughly investigated the flexural performances of both the PPSRC beam and the HPSRC beam. The main conclusions drawn from this paper are as follows:

The mechanical behaviors of both the PPSRC beam specimens and the HPSRC beam specimens were similar to that of the entire cast-in-place SRC specimen. All the specimens failed in typical flexural failure modes. Due to the contribution of the encased steel shape, the failure mode was ductile. The flexural performance of the HPSRC beam was similar to that of the PPSRC beam and was even comparable to that of the cast-in-place SRC beam.

The flexural capacities of the PPSRC beam specimen and the HPSRC beam specimen were 3.60 percent and 4.49 percent lower than that of the cast-in-place SRC beam specimen, which led to the conclusion that the flexural capacities of the PPSRC beam, the HPSRC beam and the SRC beam were similar and seldom affected by the construction method.

The flexural capacities of the PPSRC specimens were just slightly higher than those of the HPSRC specimens, and the difference would decrease with an increase in the concrete strength of the inner part. The effect of the hollow core on flexural performance was relatively small.

The concrete strength directly affected the flexural capacity, and the flexural capacity of the specimen increased proportionally to increases of the concrete strength but in a mild way with limited room for increase.

A set of calculation methods for the flexural capacities of the PPSRC beam and the HPSRC beam was introduced, in which three different cases were suggested taking different positions of the neutral axis into account. By comparing the results calculated by the proposed model to the experimental results, the proposed method was verified to be valid and more reasonable than the AIJ method.

7.Acknowledgment

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