Behavior of a 60-year-old Reinforced Concrete Box Beam Strengthened with Basalt Fiber-reinforced Polymers Using Steel Plate Anchorage

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Abstract

In the past twenty years, fiber-reinforced polymer (FRP) has been utilized broadly to strengthen concrete structures for its superiority. The influence of FRP and end anchorage with FRP U-strips or grooving on the behaviors of reinforced concrete (RC) beams has been investigated. However, investigations on the influence of basalt FRP (BFRP) and steel plates on the behaviors of strengthened RC beams remain lacking, especially under the circumstances that the RC beams are full-scale and cracked. The present study investigated the influence of BFRP and steel plates on the behaviors of full-scale cracked RC beams including failure mode, load-carrying capacity, stress-strain relationship and stiffness. Test results demonstrated that: (1) BFRP improved the behaviors of full-scale cracked RC beams from multiple angles; (2) the steel plates had a better effect on restricting the development of cracks and increasing the load-carrying capacity of full-scale cracked RC beams than FRP U-strips; (3) the calculation method considering the influence of FRP debonding was proved effectively to obtain the theoretical load-carrying capacity of BFRP-strengthened RC beams anchoring with steel plates; (4) the steel plates could postpone the development of BFRP debonding at the initial stage, and delay the further propagation of the debonding.

1. Introduction

In civil engineering, the concrete structures and members often need to be repaired or strengthened due to various reasons, such as earthquakes, overloading, and external erosion (Wu et al. 2012). Nowadays, fiber-reinforced polymer (FRP) is utilized broadly to strengthen concrete structures and members in the actual project (Wu and Wei 2010), and this method is considered as efficient and non-corrosive. This strengthening method has been investigated from multiple angles and directions in the last twenty years, and the corresponding standards have been revised, including CECS-146: 2003 (CECS 2007a) and ACI 440.2R-2017 (ACI 2017). Strengthening concrete members with FRP has little influence on its original weight and member size (Wu et al. 2015). The glass FRP (GFRP) (Teng et al. 2001), carbon FRP (CFRP) (Ng and Lee 2002), and aramid FRP (AFRP) (Kong et al. 2018) are three kinds of common FRP composites. Basalt FRP (BFRP) is a new kind of environmentally friendly FRP composite, which can be naturally degraded (Shen et al. 2021a, 2021b). BFRP has good stability, high temperature resistance, and good corrosion resistance (Talijsten 2003). BFRP is considered a replacement for other FRP due to its good properties (Sim et al. 2005). Ouyang (2013) showed that the yielding load and ultimate load of RC beam improved when the RC beam was strengthened only with BFRP strips. Compared with CFRP, BFRP has a wide range of application and low cost (Sim et al. 2005; Wang et al. 2019). Hou et al. (2020) proposed a new strengthening system made of BFRP bars-reinforced engineered cementitious composite (ECC) matrix for repairing of RC beams, and the new strengthening system could enhance the flexural strengths of the RC beams. Wu and Li (2017) proposed a CFRP-ECC hybrid system for strengthening of the concrete structures, and this hybrid consists of CFRP composite embedded in the ECC matrix. The existing investigations mainly focused on the application and effectiveness of GFRP and CFRP. Meanwhile, the current investigation on FRP-strengthened beams mainly focused on cast-in-place structures and members, which are reduced-scale. However, the old structures and members are full-scale. Strengthening full-scale old structures and members is more difficult and needs higher technical requirements (Waal et al. 2017). Investigations on the application and effectiveness of BFRP on the behaviors of full-scale RC beams which have been in service under normal service condition are still lacking, and it is necessary to further investigate for better understanding the repair efficacy.

The old structures and members have been in service for many years, which leads to the decrease of mechanical properties, load-carrying capacity and stiffness. Meanwhile, the properties of the strengthened old
structures and members are different from those of the ordinary structures and members (Zhang et al. 2014). Compared with the ordinary structures, the old structures usually have obvious initial flexural cracks. The cracks have great effect on the performance and properties of concrete structures and members. The externally bonded FRP composites can be utilized to strengthen RC beams which were corroded, load-induced damaged or cracked (Rogers et al. 2012). However, Meier and Kaiser (1991) found that the shear cracks of structures may cause FRP composites to fall off from tension face. Investigations on the influence of BFRP on the behaviors of full-scale strengthened RC beams with initial flexural cracks are still inadequate, and it deserves further systematic and quantitative investigation.

Steel plates have been utilized widely to strengthen the RC beams in recent years (Garden and Hollaway 1998). Rakgate and Dundu (2018) showed that the externally bonded steel plates could greatly improve the bending stiffness and load-carrying capacity of RC beams, and to decrease the deflection at midspan and crack width of RC beams. Strengthening with FRP strips or steel plates both have certain effect on the behavior of RC beams, but both of them have the limitations. Although the FRP strips have superior mechanical properties and light-weight, the failure of flexural members strengthened with FRP strips is sudden due to its brittleness. Qin et al. (2019) found that the occurrence of cracks will weaken the stiffness of the FRP-strengthened RC beams. The steel plate has good ductility. However, the thickness of the steel plate is limited when the steel plate is used to strengthen the flexural members because of the high density of steel plate. The advantages of two materials can be comprehensively utilized when two kinds of methods are combined. Maaddawy and Soudki (2008) showed that the application of steel plates as well as externally bonded FRP can prevent the premature FRP debonding. The steel plate anchorage refers to the application of steel plates with anchor bolts to fix the FRP composites on the concrete structures and members (Qin et al. 2019). Lu and Zhou (2006) showed that the composite method of FRP strips and steel plate had better effect on the improvement of the bending resistance of beams than single method. Therefore, the application and effectiveness of the combination of BFRP strips and steel plates on the behaviors of full-scale old RC beams with initial flexural cracks needs to be further investigated due to its practical value and broad prospects in the actual project.

The investigations on the influence of BFRP strips and steel plates in strengthening full-scale old RC beams are available and meaningful. However, the effect of initial flexural cracks and steel plates on the behavior of RC beams has rarely received attention and systematic consideration in most existing investigations. Therefore, experimental tests and corresponding analysis on the influence of BFRP and steel plates on the stress-strain relationship, failure mode, ductility and load-carrying capacity of full-scale strengthened RC beams were conducted in the present study.

2. Experimental program

2.1. Description of specimens

Three full-scale RC box beams with initial flexural cracks were utilized as test specimens in the present study, which were removed from a bridge over Nansi Lake in Jining City, Shandong Province, China. All test beams have been in service for 60 years under normal service condition. The test specimens were designated as BEAM-0, BEAM-1 and BEAM-2 and the corresponding average concrete cover depth was 33.6, 31.6 and 30.0 mm, respectively. Figure 1 demonstrates the cross section of these full-scale test specimens. The reinforcements for specimens BEAM-0, BEAM-1 and BEAM-2 were the same.

The CFRP strip has a significant influence on the improvement of flexural capacity of RC beams and could improve the flexural stiffness of RC beams well (Deng 2001). Ouyang (2013) showed that the flexural capacity of continuous beams strengthened with CFRP sheets can be improved better than BFRP sheets. Double-layer CFRP strips were applied from the position of concentrated load to the supports in order to avoid the occurrence of shear failure of specimens because of its high elastic modulus, as demonstrated in Chen et al. (2008). Wang et al. (2015) investigated the influence of anchorages on the flexural behavior of damaged RC beams, and the width of CFRP strips was 100 mm. Combined with the standards (CECS 2007a; ACI 2017), the width of CFRP strips was 100 mm in the present study. Jayaprakash et al. (2008) and Alver et al. (2014) investigated the effect of CFRP-spacing on behavior of CFRP-strengthened RC beam. Zhang and Hu (2008)

![Fig. 1 Cross section of specimens (all units in mm).](image-url)
designed the spacing of CFRP sheets as 100 mm and 150 mm when CFRP strips were utilized to strengthen RC beam.

Combined with the size of specimen and standards (CECS 2007a; ACI 2017), the spacing of CFRP strips in the present study was 100 mm. BEAM-0 was designed as the reference specimen without BFRP strips, as demonstrated in Fig. 2(a). BFRP strips were utilized to strengthen the BEAM-1 and BEAM-2 specimens, which were glued on the tension face of specimens. In the present study, double-layer BFRP strips were utilized because the single-layer BFRP strips were insufficient due to its low modulus, as demonstrated in Shen et al. (2019a, 2019c). The bond length of these BFRP strips was consistent, and the value was 4900 mm. Figure 2 demonstrates the layout of the arrangement of CFRP and BFRP strips of the BEAM-0, BEAM-1 and BEAM-2 specimens. The double-layer CFRP strips were also utilized as FRP U-strips in the BEAM-1 specimen in order to anchor the BFRP strips glued on bottom of the surface, as demonstrated in Fig. 2(b). The placement of FRP U-strips was started at 1050 mm from the centerline of the BEAM-1 specimen. The shear capacity of strengthened beams improves with the decrease of the spacing of CFRP strips (Zhao et al. 2000). Gao et al. (2004) showed that the restraint effect on inclined cracks reduced with the increase of spacing of CFRP strips when the amount of carbon fiber cloth is fixed. Combined with the size of specimen and standards (CECS 2007a; ACI 2017), the spacing between two FRP U-strips was 100 mm in the present study. At the end of the FRP U-strips, the CFRP strips were set horizontally as the trim strips, as demonstrated in Shen et al. (2019b). The steel plates were placed above the BFRP strips of the BEAM-2 specimen as anchorages, as demonstrated in Fig. 2(c). This anchorage started from the end of BFRP strips. Eight steel plates were utilized in the BEAM-2 specimen. Wang et al. (2019) investigated the influence of steel plate anchorage on the behavior of RC beams, and the spacing of steel plate was 300 mm. Xiong et al. (2012) used the steel plates and CFRP sheets to strength the RC beams (2200 mm long), and the spacing of steel plate was 400 mm and 700 mm, respectively. In the present study, the test specimen was...
6300 mm long. Combined with the size of specimen and standards (CECS 2007a; ACI 2017), the spacing of steel plates was 700 mm. The anchorage device included anchor bolts and steel plates, and the size of steel plate was 900 mm×100 mm×10 mm. Combined with the results demonstrated in Wang et al. (2015) and the above-mentioned standards, the width of steel plates was 100 mm. Each steel plate had four reserved holes, the diameter of which was 12 mm, as demonstrated in Fig. 3(a). The BFRP strips behind the steel plates were marked in red, as demonstrated in Fig. 3(b).

The process of anchoring steel plate was mainly divided into four steps, as demonstrated in Figs. 4(a) to 4(d). Figure 4(e) demonstrates the details of BFRP strips and anchor bolt after fixing, and the BFRP strips were marked in red. The FRP U-strips and steel plates were both set symmetrically along the centerline of test specimen. Table 1 demonstrates the mechanical properties of CFRP and BFRP strips. The surface slurry and oil stain on the bonding surfaces of CFRP and BFRP strips were removed in advance through polishing. The alco-

| Property                  | BFRP | CFRP |
|---------------------------|------|------|
| Tensile strength (MPa)    | 2.30×10^3 | 3.50×10^3 |
| Tensile elastic modulus (GPa) | 1.05×10^2 | 2.43×10^2 |
| Elongation (%)             | 2.18  | 1.71  |
| Thickness (mm)             | 0.121 | 0.167 |

Note: BFRP: Basalt fiber-reinforced polymer; CFRP: Carbon fiber-reinforced polymer.

![Fig. 3 Details of steel plate anchorage: (a) Diagram; (b) Actual photos (all units in mm).](image-url)
hol was utilized to clean up the surfaces of CFRP and BFRP strips after polishing to ensure that bonding between concrete and FRP strips was effective. The type of adhesive utilized in the present study was epoxy resin, and the properties of epoxy resin were shown in Table 2. The CFRP and BFRP strips utilized in strengthening test specimens were all impregnated with epoxy resin in advance, and this epoxy resin was also utilized as adhesive between CFRP and BFRP strips and concrete.

### 2.2 Instrumentation layout and test setup

The instrumentation utilized in the tests included three kinds of sensors, which were utilized to obtain the value of load, strains and deflection, respectively. The linear variable differential transformer (LVDT) gauges were utilized to obtain the deflections of test specimens. Figure 5(a) demonstrates that fourteen LVDT gauges were placed along the span of test specimens. Figure 5(b) demonstrates that the 80 mm uniaxial strain gauges placed at midspan were utilized to measure the strain of longitudinal direction of beams on the concrete surface. Figure 5(c) demonstrates that the 3 mm uniaxial strain gauges set on the steel reinforcement at midspan of beams were utilized to obtain the strains in longitudinal direction.

| Property                        | Epoxy resin |
|---------------------------------|-------------|
| Tensile strength (MPa)          | 41.3        |
| Tensile elastic modulus (GPa)   | 2.7         |
| Elongation (%)                  | 1.7         |
| Flexural strength (MPa)         | 66.3        |
| Compressive strength (MPa)      | 93.8        |
| Tensile bonding strength with concrete (MPa) | 3.8        |

Fig. 4 The process of steel plate anchorage: (a) Drill holes at the bottom; (b) Clean hole; (c) Fix anchor bolt; (d) Fix steel plate; (e) The details of BFRP strips and anchor bolt.
steel reinforcement. Similar arrangement of strain gauges for the BEAM-0 and BEAM-1 specimens are demonstrated in Shen et al. (2019d).

Figure 6(a) demonstrates that the 5 mm uniaxial strain gauges were utilized to obtain the strains at various points along the length of BFRP strips in the longitudinal direction. The strain in the steel plates was measured by the same kind of 5 mm gauges, as demonstrated in Fig. 6(b). The model of the static loading test utilized in the present study was four-point bending, which is the same as the test model demonstrated in Rogers et al. (2012). The loading system was mainly equipped with spreader beams, reaction frame, steel beams, supports and jack, as demonstrated in Fig. 7(a). Figure 7(b) shows the actual photo of loading device utilized in the present study.

The load was divided into two parts with equal size by the steel spreader beam. The deflection of the test specimens was applied in three stages during the test. The first stage was from 0 mm to 4.0 mm with the interval of 0.4 mm; the second stage was from 4.0 mm to 30.0 mm with the interval of 2.0 mm; and the final stage was from 30.0 mm until failure with the interval of 5.0 mm.

2.3 Material behavior
The actual compressive strength and elastic modulus of concrete could be obtained by testing the concrete cylinder samples. Different sizes of samples of test specimens were taken in the present study to obtain above mechanical properties of concrete when the loading tests were finished. In line with Chinese Standard CECS 03-2007 (CECS 2007b), three samples removed from the compression area of test specimens were processed into cylinders, the diameter and height of which were both 100 mm. In line with Chinese Standard JGJ/T 384 (MHURD 2016) and the above standard (CECS 2007b), the compressive strength of concrete cylinders (100×100 mm) can be calculated by Eqs. (1), (2) and (3) below.

\[ f_{cu,e} = f_{cu,cor} - k_s S_{cor} \]  
\[ S_{cor} = \sqrt{\frac{\sum_{i=1}^{n}(f_{cu,cor,i} - f_{cu,cor,m})^2}{n-1}} \]  
\[ f_{cu,cor,m} = \frac{\sum_{i=1}^{n}f_{cu,cor,i}}{n_i} \]

where \( S_{cor} \) means the standard deviation of compressive strength of test samples, in MPa; \( f_{cu,cor} \) and \( f_{cu,cor,m} \) mean the mean value and single value of compressive strength of test samples, respectively, in MPa; \( n_i \) means the number of test samples; and \( k_s = 1.222 \). The compressive strength of concrete cubes (150 mm), \( f_{cu,k} \), and concrete cylinders (100 mm×100 mm), \( f_{cu,e} \), could be considered as equivalent, as demonstrated in Figure 5.

**Fig. 5 Instrumentation layout (all units in mm).**
Chinese standard JGJ/T 384 (MHURD 2016). Equation (4) could be utilized to obtain the compressive strength of concrete cylinders, \( f'_c \), the diameter and height of which were 150 and 300 mm, respectively, as demonstrated in Eurocode 0 (CEN 1999).

\[
f'_c = 0.79 f_{cu,k}
\]  

(4)

Equation (5) can be utilized to obtain elastic modulus of concrete, \( E_c \), in line with the Chinese Standard GB/T 50010 (SAC 2010a), as follows.

\[
E_c = \frac{10^5}{2.2 + \frac{34.7}{f_{cu,k}}}
\]  

(5)

Fig. 6 Strain gauge positions in BEAM-2 specimen: (a) Positions on BFRP strips; (b) Positions on steel plates (all units in mm).
In line with Chinese Standard GB/T 228.1 (SAC 2010b), the mechanical properties test was conducted to obtain the properties of steel reinforcement through a U.S. servo hydraulic testing machine. The results of tensile yield strength of steel reinforcements were obtained by testing three samples in tension. Table 3 demonstrates the amount and properties of steel reinforcement for three test specimens, respectively.

The load-carrying capacity of RC box beams can be evaluated based on the equivalent reinforcing ratio, $\rho_{eq}$, as demonstrated in Eq. (6).

$$\rho_{eq} = \frac{1}{f_y} \left( \rho_s f_y + \rho_f f_{fu} \right)$$

(6)

where $\rho_s$ and $\rho_f$ mean the longitudinal and FRP reinforcement ratio, respectively, defined as $\rho_s = A_s / bh_0$ and $\rho_f = A_f / bh_0$; $h_0$ and $b$ mean the effective depth and web width of section of box beam, respectively, in mm; $f_{fu}$ means the ultimate tensile strength of BFRP strips, in MPa; $A_f$ means the area of FRP external reinforcement, in mm$^2$; and $f_y$ means the yield strength of longitudinal reinforcement, in MPa.

Table 4 demonstrates the properties of concrete and the reinforcing ratios of three test specimens, respectively.

### 2.4 Modal tests

Investigating the damage identification of concrete structures based on the experimental modal analysis has been widespread in recent years because this major technique has much superiority. In the present study, the modal test was utilized to investigate the natural frequency of two BFRP-strengthened specimens with cracks. All three full-scale RC box beams had initial

| Specimen Type of steel reinforcement | Yielding tensile strength (MPa) | Ultimate tensile strength (MPa) |
|-------------------------------------|-------------------------------|-------------------------------|
| BEAM-0, BEAM-1 and BEAM-2           | 284.5                         | 423.5                         |
| 4 A 12*                             | 357.5                         | 451.5                         |
| 3 C 16***                           | 447.0                         | 611.5                         |

*4 A 12 means steel reinforcement with design yielding strength of 300 MPa, and a diameter of 12 mm;  
**B 12 means steel reinforcement with design yielding strength of 335 MPa, and a diameter of 12 mm;  
***C 16 means steel reinforcement with design yielding strength of 400 MPa, and a diameter of 16 mm.

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Fig. 7 Layout of test setup: (a) Loading device; (b) Actual photo of loading device.
flexural cracks on the surface, as demonstrated in Fig. 8. The cracks patterns of specimens at ultimate load were related to the initial flexural cracks, especially the crack spacing. At the initial stage of loading, the closed cracks at the bottom of the specimen with initial cracks begin to open, and the initial cracks continue to expand upward along the original direction with the increase of load (Dai et al. 2014). Before the loading test, all three test specimens were inspected in detail in order to find out the flexural cracks on them. Figure 8 demonstrates that the initial flexural cracks in the BEAM-0, BEAM-1 and BEAM-2 specimens were different, but the crack

| Specimen   | $f_{cu,k}$ (MPa) | $f'_{c}$ (MPa) | $E_{c}$ (GPa) | $\rho_s$ | $\rho_{eq}$ |
|------------|------------------|----------------|---------------|----------|-------------|
| BEAM-0     | 52.0             | 41.1           | 34.9          | 1.15%    | -           |
| BEAM-1     | 48.0             | 37.9           | 34.2          | 1.15%    | 2.34%       |
| BEAM-2     | 48.0             | 37.9           | 34.2          | 1.15%    | 2.34%       |

Fig. 8 Initial crack patterns of specimens: (a) BEAM-0; (b) BEAM-1; (c) BEAM-2 (all units in mm).
spacing of initial flexural cracks were similar on the both the sides for the BEAM-0 and BEAM-1 specimens.

The modal test for the BEAM-0 specimen was divided into two stages, which was conducted before and after loading test, respectively. The modal test for the BEAM-1 and BEAM-2 specimens was divided into three stages, which was conducted before reinforcement, after reinforcement, and after loading, respectively. Seven magneto electric velocity sensors were set along the top surface of test specimens, the specific positions of which were demonstrated in Fig. 5(a). The data acquisition system was utilized to obtain the sensor data. The ideal test results were obtained based on the combination of the natural vibration and appropriate hammering in the present study.

3 Results and discussion

3.1 Influence of BFRP and steel plates on the cracking characteristics

The failure mode of the BEAM-0 specimen was the flexural failure. Under the load of 90.0 kN, the initial cracks were widened and extended upward, and new flexural cracks were found on the specimen at this time. The maximum crack width was 1.5 mm when the load further increased to 208.6 kN. The ultimate load of the BEAM-0 specimen was 287.4 kN. Figure 9(a) demonstrates the actual photos of the BEAM-0 specimen after failure.

For the BEAM-1 specimen, FRP U-strips were utilized to anchor BFRP strips. The BEAM-1 specimen failed when CFRP and BFRP strips in end zone of test specimen debonded. Under the load of 68.3 kN, new flexural cracks occurred on the specimen. The previous
flexural cracks propagated when the load further increased, and more flexural cracks occurred in pure bending zone of test specimen at this moment. The maximum crack width increased from 0.1 mm to 1.5 mm when the corresponding load increased from 49.7 kN to 331.7 kN. When the load increased to 342.1 kN, FRP U-strips started to debond from the side surface of the specimen. At this moment, debonding at the BFRP strips and adhesive interface occurred in pure bending zone of test specimen. The ultimate load of the BEAM-1 specimen was 375.9 kN. Figure 9(b) demonstrates the actual photos of this specimen after failure.

For the BEAM-2 specimen, steel plates were utilized to anchor BFRP strips. It failed when the BFRP strips in pure bending zone of test specimen debonded. Under the load of 49.3 kN, the change of initial crack width was negligible, and no new cracks occurred at that moment. When the load further increased to 127.3 kN, new flexural cracks occurred, the maximum width of which was 0.05 mm. The maximum crack width was 1.5 mm when the load further increased to 376.9 kN. The BFRP strips were straightened out at this time, which made a great sound as a result of the slip failure. However, FRP debonding was not observed at this moment. This phenomenon may be due to that the FRP debonding had already occurred in fact, but the steel plates anchored the BFRP strips on the adhesive interface in pure bending zone of test specimen. Under the load of 385.1 kN, debonding at the BFRP strips and adhesive interface occurred in pure bending zone of the test RC box beam, the position of which was near the cracks. This failure mechanism was considered as a kind of ‘intermediate crack-induced debonding’, as demonstrated in Teng et al. (2003). The tensile stress released by cracked concrete was transferred to the BFRP strips because of the occurrence of new flexural cracks in concrete. This phenomenon induced the high level of local interfacial stresses at the interface of BFRP strips and concrete surface.

The tensile stress developed in the strips as the load increased, and the interfacial stress increased simultaneously. When the values of these stresses reached critical values, debonding occurred near the cracks, and then spread to other parts of BFRP strips. The ultimate load of the BEAM-2 specimen was 421.5 kN. Figure 9(c) demonstrates the actual photos of this specimen after failure. Figure 10 demonstrates the crack patterns in the pure bending zone of three test specimens under ultimate load, respectively. The amount of main flexural cracks of the BEAM-2 specimen was fewer than that of BEAM-1, as demonstrated in Fig. 10. The crack depth was obtained by the crack depth tester, and the actual photo was demonstrated in Fig. 11. The average crack depth was the average value of the main cracks on both sides of the bending zone for the BEAM-0, BEAM-1, and BEAM-2 specimens. Table 5 demonstrates characteristics of cracks under ultimate load for three test specimens, respectively.

The average crack width, average crack depth and average crack spacing of the BEAM-2 specimen under ultimate load were 21.4%, 25.8% and 30.0% lower than those of the BEAM-0 specimen, and 9.3%, 6.9%, and 16.2% lower than those of the BEAM-1 specimens, respectively. Compared with the BEAM-0 specimen, the development of cracks was postponed effectively by the application of BFRP for the BEAM-1 and BEAM-2 specimens. This phenomenon indicated that the BFRP strips had an evident influence on the control of cracks. The reason may be that FRP composites could carry the load throughout the loading stage, which could significantly reduce the development of cracks.

Figure 12 demonstrates the relationship between maximum crack width and midspan moment of three test specimens. Compared with BEAM-0 and BEAM-1, the maximum crack width of the BEAM-2 specimen was the lowest at different levels of load. The crack width of the BEAM-1 specimen increased more quickly than that of BEAM-2 when the moment exceeded 150 kN·m. The performance of the BEAM-2 specimen in the confinement of cracks was better than that of BEAM-1, and the steel plates utilized in BEAM-2 could restrain the development of cracks more effectively than FRP U-strips.

### 3.2 Influence of BFRP and steel plates on the ductility

The deflection ductility and curvature ductility are two indices of the ductility (Shen et al. 2019d; Dong et al. 2020). The physical behaviors of test specimens were integrated into the definitions of structural ductility, and the relationships were demonstrated in Figs. 13 and 14. Figure 13 demonstrates the relationships between load and deflection at the midspan section of three test specimens, respectively. The strains obtained from the whole depth of test specimens could be utilized to calculate the curvature at midspan. The relationship between the curvature and the midspan moment of test specimens was demonstrated in Fig. 14. Equations (7) and (8) can be utilized to define the deflection ductility and curvature ductility in the present study, respectively, and ductility indices could be quantified on the basis of the data demonstrated in Figs. 13 and 14.

| Specimen | Maximum spacing | Average spacing | Maximum depth | Average depth | Maximum width | Average width |
|----------|-----------------|-----------------|---------------|---------------|---------------|---------------|
| BEAM-0   | 178             | 140             | 462           | 400           | 1.39          | 1.12          |
| BEAM-1   | 155             | 117             | 378           | 319           | 1.26          | 0.97          |
| BEAM-2   | 124             | 98              | 336           | 297           | 1.03          | 0.88          |
\[
\mu_\Delta = \frac{\Delta_\mu - \Delta}{\Delta_\mu}
\]
(7)

\[
\mu_\phi = \frac{\phi_\mu - \phi}{\phi_\mu}
\]
(8)

where \(\Delta_\mu\) and \(\phi_\mu\) mean the midspan deflection and the midspan curvature at ultimate failure, respectively, in mm; and \(\Delta\) and \(\phi\) mean midspan deflection and the midspan curvature at yielding of steel reinforcement, respectively, in m\(^{-1}\).

Figure 13 demonstrates that the load-deflection curves of test specimens were divided into two stages.

The deflections of the BEAM-1 and BEAM-2 specimens were small at the elastic stage (first stage) because the effect of BFRP strips was not fully exerted. As the load continued to increase, the cracks occurred in the tensile zone of test specimens. The beams were at the elastoplastic stage (second stage) at this moment, the deflections of the BEAM-1 and BEAM-2 specimens changed greatly. The BFRP strips fully exerted its effect, which effectively inhibited the cracking of concrete and delayed the development of the cracks. The load-deflection curves of these two specimens showed a good linear relationship at the first stage, which indicated that the BFRP strips and steel plates were uniformly loaded at this moment. The midspan deflection

Fig. 10 Crack patterns of specimens at ultimate load: (a) BEAM-0; (b) BEAM-1; (c) BEAM-2 (all units in mm).
of the BEAM-1 specimen was smaller than that of BEAM-2 at the same load, as demonstrated in Fig. 13. The slopes of load-deflection curves changed abruptly at the moment when the steel reinforcement yielded. Therefore, the load at the yielding of steel reinforcement could be determined by the sudden change of slopes of load-deflection curves. The midspan deflections at ultimate failure or yielding of steel reinforcement of the BEAM-0, BEAM-1 and BEAM-2 specimens were 83.4, 58.1, and 49.1 mm or 9.1, 10.4, and 11.9 mm, and the corresponding loads were 287.4, 375.9, and 421.5 kN or 241.6, 301.7, and 334.6 kN, respectively. The curvature at ultimate failure or yielding of steel reinforcement for the BEAM-0, BEAM-1 and BEAM-2 specimens were 11.7×10⁻³, 9.3×10⁻³, and 8.5×10⁻³ m⁻¹ or 1.4×10⁻³, 1.9×10⁻³, and 2.4×10⁻³ m⁻¹, respectively. The ductility indices of three test specimens were calculated, and the corresponding results were demonstrated in Table 6.

Compared with the BEAM-0 specimen, the deflection ductility or curvature ductility of BEAM-1 and BEAM-2 decreased by 39.1% and 55.4% or 41.7% and 58.3%, respectively. A distinct loss in ductility occurred in both in BEAM-1 and BEAM-2. This phenomenon may be due to the existence of bond slip between the BFRP strips and concrete substrate. The deflection ductility and curvature ductility of the BEAM-2 speci-

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**Table 6 Ductility indices of all specimens.**

| Specimen | Deflection ductility (mm) | Deflection ductility ratio | Curvature ductility (mm) | Curvature ductility ratio |
|----------|--------------------------|---------------------------|--------------------------|--------------------------|
| BEAM-0   | 9.2                      | 1.000                      | 8.4                      | 1.000                    |
| BEAM-1   | 5.6                      | 0.609                      | 4.9                      | 0.583                    |
| BEAM-2   | 4.1                      | 0.446                      | 3.5                      | 0.417                    |
men were 26.8% and 28.6% lower than the ductility of BEAM-1, respectively. This meant that the BEAM-2 specimen sacrificed more ductility to improve its load-carrying capacity.

3.3 Influence of BFRP and steel plates on the load-carrying capacity

The load-carrying capacity of FRP-strengthened RC beams, \( M_{cr} \), could be calculated through the method raised in ACI 440.2R-2017 (2017), as demonstrated in Eq. (9).

\[
M_{cr} = A_f f_f (d - \frac{\beta x}{2}) + \psi_f A_f f_\mu (d - \frac{\beta x}{2}) \tag{9}
\]

where \( x \) means the length between the extreme compression fiber and the neutral axis, in mm; \( A_f \) means the area of FRP external reinforcement, in mm\(^2\); \( d \) means the effective depth of FRP flexural reinforcement, in mm; \( \beta \) means the length between the extreme compression fiber and the centroid of steel reinforcement, in mm; \( \psi_f \) means the reduction factor of FRP strength, \( \psi_f = 0.85 \) (ACI 2017); and \( f_\mu \) means the stress of non-prestressed steel reinforcement, in MPa.

Equation (10) can be utilized to calculate the debonding strain of externally bonded FRP strips, \( \varepsilon_{\mu d} \), in line with ACI 440.2R-2017 (2017), as follows.

\[
\varepsilon_{\mu d} = 0.41 \frac{\int f'_f \sqrt{\frac{1}{n_1 E_f t_f}} \leq 0.9 \varepsilon_{\mu c} \tag{10}
\]

where \( t_f \) means the nominal thickness of one layer of FRP strips, in mm; \( n_1 \) means the number of layers of FRP strips; \( \varepsilon_{\mu c} \) means the design rupture strain of FRP strips, in \( \varepsilon \); and \( E_f \) means the tensile elastic modulus of FRP strips, in MPa.

Considering the self-weight of simple support beam, the initial substrate strain, \( \varepsilon_{si} \), is obtained through Eq. (11).

\[
\varepsilon_{si} = \frac{M_{self} (d - x)}{I_n \varepsilon_c} \tag{11}
\]

where \( I_n \) means the inertia moment of cracked section transformed to concrete, in mm\(^4\); and \( M_{self} \) means the maximum moment of self-weight, in kN·m.

Equation (12) can be utilized to calculate the effective strain of FRP strips obtained at failure, \( \varepsilon_{\mu f} \) (ACI 2017), and is demonstrated as follows:

\[
\varepsilon_{\mu f} = \varepsilon_{cu} \left( \frac{d - x}{x} \right) - \varepsilon_{si} \leq \varepsilon_{\mu d} \tag{12}
\]

The strain of non-prestressed steel reinforcement, \( \varepsilon_s \), is obtained through Eq. (13).

\[
\varepsilon_s = \varepsilon_{cu} \left( \frac{d - x}{d_f - x} \right) \tag{13}
\]

When a perfect behavior is assumed, the effective stress of FRP reinforcement, \( f_{\mu f} \), is obtained through Eq. (14).

\[
f_{\mu f} = E_f \varepsilon_{\mu f} \tag{14}
\]

The stress of steel reinforcement, \( f_s \), can be obtained based on the value of \( \varepsilon_s \) through Eq. (15).

\[
f_s = E_s \varepsilon_s \leq f_f \tag{15}
\]

where \( E_s \) means the elastic modulus of steel reinforcement, in MPa.

The neutral axis depth can be obtained through Eq. (16) when the values of \( f_\mu \) and \( f_s \) are obtained based on the assumed internal force equilibrium.

\[
x = \frac{A_f f_f + A_f f_\mu}{\alpha_f f_s \beta_f b} \tag{16}
\]

where \( b \) means the web width of section, in mm; and \( \alpha_f \) means the multiplier of \( f'_f \) to obtain the intensity of an equivalent rectangular stress distribution for concrete.

The moments caused by self-weight for the BEAM-0, BEAM-1 and BEAM-2 specimens were 40.2, 45.2, and 43.8 kN·m, respectively. When the moments caused by self-weight was included, the experimental load-carrying capacity for BEAM-0 was 342.0 kN·m, while those of BEAM-1 and BEAM-2 were 434.9 kN·m (27.2% increase) and 486.4 kN·m (42.2% increase), respectively. This phenomenon can be explained by the reason that the steel plates could effectively prevent the stress concentration at the edge of the anchorage when compared with the traditional FRP U-strips. The theoretical load-carrying capacity of the BEAM-0, BEAM-1, and BEAM-2 specimens calculated by Eq. (9) were 330.3, 519.4, and 519.4 kN·m, respectively. Table 7 demonstrates the results of experimental and theoretical load-carrying capacity for all three specimens. The experimental values of the BEAM-1 and BEAM-2 specimens were 19.4%, and 6.8% lower than the corresponding theoretical values, respectively. This phenomenon may be due to the following reasons: the existence of initial cracks would decrease the load-carrying capacity of RC box beam, and the effect of cracks was not considered in the current standard ACI 440.2R-2017 (2017). Meanwhile, the loss of capacity resulted from the debonding of BFRP strips and the effect of the anchorage on the increase of capitivity were not considered. Therefore, the current calculation method raised in ACI 440.2R-2017 (2017), namely Eq. (9), was not suitable for the cracked beams using anchorage.

The load-carrying capacity of FRP-strengthened RC beams using different kinds of anchorages can be obtained through the calculation method proposed by...
Table 7 Load-carrying capacities of all specimens.

| Specimen | Experimental | Calculated (ACI) | Calculated (Tomas) | \( M_{\text{exp}} / M_{\text{ACI}} \) | \( M_{\text{exp}} / M_{\text{Tomas}} \) |
|----------|--------------|-----------------|-------------------|-------------------------------|----------------------------------|
| BEAM-0   | 342.0        | 330.3           | -                 | 1.035                         | -                                |
| BEAM-1   | 434.9        | 519.4           | 444.3             | 0.837                         | 0.979                            |
| BEAM-2   | 486.4        | 519.4           | 507.8             | 0.936                         | 0.958                            |

Note: \( M_{\text{exp}} \) means the load-carrying capacities obtained from experimental tests; \( M_{\text{ACI}} \) means the load-carrying capacities calculated by ACI 440.2R-08; \( M_{\text{Tomas}} \) means the load-carrying capacities calculated by Tomas’ method.

Equation (23) can be utilized to calculate the value of \( I_{c,\text{eff}} \) (Skuturna and Valivonis 2014).

\[
I_{c,\text{eff}} = \frac{b \cdot x^2}{3} \tag{23}
\]

Equations (24) and (25) can be utilized to calculate the value of \( G_{w,\text{eff}} \) on the basis of \( k_i \), which is obtained experimentally (Rzhansitsyn 1986).

\[
G_{w,\text{eff}} = \frac{3k_i\beta}{E_fA_f} \tag{24}
\]

\[
\beta = \frac{2.5 - b_f/b_c}{1.5 + b_f/b_c} \tag{25}
\]

where \( b_f \) means the width of the external BFRP strips, in mm; \( \beta \) means coefficient which is utilized to evaluate the effect of the width of external reinforcement on the operation of strengthened RC members; and \( k_i \) means coefficient which is utilized to evaluate the kind of anchorage: \( k_i = 1 \) when FRP is not anchored, \( k_i = 1.5 \) when the steel plates are utilized as the anchorage, and \( k_i = 2.0 \) when FRP wraps or interlocking grooves are utilized as the anchorage (Slaitas et al. 2018).

Considering the bond performance, the load-carrying capacities of the BEAM-1 and BEAM-2 specimens calculated by Eqs. (17) and (18) were 444.3 and 507.8 kN·m, respectively. The calculated values showed that steel plates had better performance on the increase of the load-carrying capacity than FRP U-strips. Compared with the experimental values, the deviations between experimental values and calculated values based on Eq. (17) for BEAM-1 and BEAM-2 were 2.1% and 4.2%, respectively. The theoretical load-carrying capacity calculated by the calculation method proposed by Tomas demonstrated good accuracy with the experimental values. The nature of semi-theoretical and semi-experimental calculation determined that the method proposed by Tomas was effective and accurate.

3.4 Influence of BFRP and steel plates on the stiffness and natural frequency

A procedure based on the modal analysis and tests was proposed in the present study to evaluate the stiffness variation induced by the cracks and strengthening of RC beams. The modal tests were divided into three stages for the BEAM-1 and BEAM-2 specimens, which in-
cluded before reinforcement, after reinforcement, and after loading, respectively. The natural frequency was obtained through the modal tests, and the curve of time-dependent velocity was determined based on the data of modal tests. The curve of amplitude-frequency was determined through the fast Fourier transform (FFT) method in the present study, as demonstrated in Shen et al. (2015). Figures 15 and 16 demonstrate the curves of time-history and amplitude-frequency for BEAM-0 and BEAM-2, respectively.

Equation (26) can be utilized to calculate the theoretical natural frequency of test specimens before reinforcement, $f_n$ (Maalej et al. 2010):

$$f_n = \frac{n^2 \pi}{2l^2} \sqrt{\frac{EI}{m}}$$  \hspace{1cm} (26)

where $m$ means the mass per unit length, in kg/m; $n$ means the vibration mode, $n=1$; and $I$ means the inertia moment of the cross-section, in m$^4$.

Assuming that all test beams utilized in the present study were uncracked, the theoretical natural frequencies of three specimens calculated by Eq. (26) were 40.0, 38.9, and 38.9 Hz, respectively. Table 8 demonstrates the test results of experimental natural frequency at three different stages and the calculated values for the BEAM-0, BEAM-1 and BEAM-2 specimens. Considering that the natural frequency was proportional to the stiffness, the stiffness could be determined from the natural frequency. After 60 years of service, the stiffness of BEAM-0, BEAM-1 and BEAM-2 decreased by 41.5%, 37.3%, and 36.8%, respectively. The stiffness of BEAM-1 and BEAM-2 increased by 8.2% and 8.6% after reinforcement, respectively. This phenomenon indicated that the application of BFRP increased the stiffness of the cracked RC beams. Compared with the stiffness after reinforcement, the stiffness after loading test for the BEAM-1 and BEAM-2 specimens decreased by 33.3% and 30.8% due to the severe damage to the section of the beams. Maalej et al. (2010) showed that the cracking induced by loading decreased the natural frequency because the value of $EI$ decreases. The values of natural frequency of BEAM-1 and BEAM-2 were almost the same after reinforcement. However, the stiffness after loading test of BEAM-2 was 0.8 Hz larger than that of BEAM-1. This phenomenon could be ex-
plained that the steel plates started to function earlier than FRP U-strips, and the stiffness of steel plates was larger than that of FRP U-strips.

3.5 Influence of BFRP and steel plates on the Relationship between load and strain

The stress-strain relationship is significant parameter of the structural characteristics (Zanaedo et al. 2006). For the BEAM-1 and BEAM-2 specimens, the value of $\varepsilon_{f'}$ was 2079 $\mu$ε, and the design value of $\varepsilon_{d'}$ was 1585 $\mu$ε. The theoretical failure mode of the beam is determined from the relationship between $\varepsilon_{f'}$ and $\varepsilon_{d'}$. The calculated value of $\varepsilon_{f'}$ was larger than design value of $\varepsilon_{d'}$, which determined that the theoretical failure mode of BEAM-1 and BEAM-2 was FRP bonding. The difference was that the damage of the BEAM-1 specimen was premature because of the occurrence of FRP debonding in the end zone of beam. When the strains in BFRP strips reached the debonding value, the corresponding load of the BEAM-2 specimen was 13.6% higher than that of BEAM-1. Therefore, BFRP-strengthened beam using steel plate anchorage had better performance on the bond behavior than that using FRP U-strips anchorage.

**Figure 17** demonstrates the relationship between the load and the strains in BFRP strips and steel reinforcements. The strain in steel plates was less than that in FRP U-strips, which indicated that the steel plate anchorage could effectively prevent the BFRP cloth from slipping. The anchor bolts would not be pulled out when
the test beam was damaged, so that the BFRP strips can fully exert its tensile resistance. At the initial stage, the strains in BFRP strips and steel reinforcements were in linear changes, which indicated that BFRP strips and steel reinforcements were both in the elastic stage at this time, as demonstrated in Fig. 17. The strains in BFRP strips and steel reinforcements of two test specimens were almost the same before the load reached the yielding value. Due to the different reinforcement methods, the strain of steel reinforcement for the BEAM-1 and BEAM-2 specimen was different under yielding load. For BEAM-1, the yielding load was 294.5 kN. The strains in BFRP strips and steel reinforcements for this specimen were 1152 and 1130 με, respectively, under the yielding load of 294.5 kN. For BEAM-2, the yielding load was 366.1 kN. The strain in BFRP strips and steel reinforcements for this specimen were 1477 and 1446 με, respectively, under the yielding load of 366.1 kN. This phenomenon indicated that the BFRP strips and steel reinforcements bonded well and worked together to provide the load-carrying capacity before yielding load, and no debonding phenomenon occurred at this moment. The strain in BFRP strips was slightly larger than that in steel reinforcements, which indicated that there is no slippage between BFRP strips and concrete cover at this stage. When the load exceeded the yielding value, the steel reinforcements yielded, and the strains in BFRP developed more quickly. The phenomenon could be explained by the reason that the steel reinforcement started to debond from concrete, and BFRP strips carried more load. The BFRP strips of the BEAM-2 specimen remained coordinated with steel reinforcements at a higher load level than BEAM-1. This phenomenon indicated that the steel plate anchorage was more effective to postpone BFRP bonding of BFRP-strengthened beams than FRP U-strips.

Conclusions
In the present study, experimental tests and analysis were conducted to study the influence of BFRP and steel plates on the behaviors of full-scale cracked RC box beams and the following conclusions could be drawn by analyzing the experimental results.

1. The development of cracks was postponed effectively by the application of BFRP, and the steel plates had a better effect on restricting the development of cracks of beams than FRP U-strips.
2. The application of BFRP increased the experimental load-carrying capacity of RC box beams effectively. The steel plates had a better effect on improving the load-carrying capacity of beams than FRP U-strips because the steel plate anchorage could effectively prevent the stress concentration at the edge of the anchorage.
3. The calculation method considering the influence of FRP debonding was proved effective. For the BEAM-1 and BEAM-2 specimens, the deviation between the load-carrying capacity calculated by this method and the experimental value was 2.1% and 4.2%, respectively.
4. The application of steel plates had little influence on the stiffness of beams after reinforcement. However, the stiffness after loading test of the BEAM-2 specimen was larger than that of the BEAM-1 because the steel plates started to function earlier than FRP U-strips, and the stiffness of steel plates was larger than that of FRP strips.
5. The development of BFRP debonding at the initial stage could be postponed effectively by the application of steel plate anchorage, and the further propagation of the debonding could be delayed.

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