Although externally bonded carbon fiber-reinforced polymer (CFRP) sheets have been used more commonly for the strengthening of existing reinforced concrete (RC) structures, the effects of this strengthening method on prestressed concrete (PC) beams have not been well clarified. In this study, an experiment was conducted to investigate the behaviors of pre-tensioned PC beams without shear reinforcement strengthened in shear using externally bonded CFRP sheets with the consideration of CFRP reinforcement ratio, stiffness and prestressing level in strands. The results implied that the shear resistance in the strengthened PC beams depended on maintaining the combination of beam action, arch action, and the bonded sheets. The failures and the increase in shear capacity of the strengthened PC beams were strongly affected by the amount, thickness of the CFRP sheets, and the effective prestress in the strands. The higher effectiveness of strengthening could be obtained in the beams strengthened with a higher CFRP reinforcement ratio of the same thickness, higher elastic modulus sheets or having higher prestressing level. Nevertheless, the increase in the thickness of the sheets did not provide a better performance. Moreover, the high inaccuracy in the calculations using the equations stipulated in the recent design guidelines compared to the increment of the shear strength obtained by the experiment was explained by the shear-resisting mechanisms.

**Key Words:** shear strengthening, mechanism, prestressed concrete, CFRP, arch action

1. INTRODUCTION

Owing to the advantages of high strength and durability of prestressed concrete (PC), the number of PC bridges constructed in Japan and worldwide has been increasing, especially during the 1960s to the 1990s\(^1\). In the recent years, deterioration of PC structures due to aging, which leads to reduced strength and durability, has been reported in many countries. When the deterioration of the shear reinforcement occurs, the shear capacity of PC girders reduces causing a shear failure of the girders, which have been known to be very brittle. To avoid such a catastrophic failure, the shear capacity of the damaged PC girders needs to be improved. In addition, there has been an increasing need to upgrade the shear capacity of the girders due to the increase in the applied load, which is greater than the design values. Thus, to enhance the structural performance and improve the sustainability and the safety for a prolonged use of the existing PC girders/beams, shear strengthening has become an important feature.
In the last decades, the application of carbon fiber-reinforced polymer (CFRP) sheets to strengthening of reinforced concrete (RC) structures has become more common. The CFRP sheets with the advantages of high tensile strength and corrosion resistance are externally bonded to the surface of structures for repair, strengthening, and improving the durability of the existing RC structures. This strengthening method can be implemented within a short time and does not require heavy equipment. Therefore, the application of externally bonded CFRP sheets for strengthening of PC girders and beams can be promisingly effective.

Up to now, numerous studies on the contribution of externally bonded CFRP sheet to shear capacity of RC beams have been carried out. Nevertheless, there has been little discussion on the effects of this strengthening method to shear behavior of pre-tensioned PC beams. In the previous study by Kang and Ary on shear strengthening of two PC beams using externally bonded CFRP sheet, a significant improvement in shear capacity and ductility of strengthened beams was reported. However, the effects of effective prestress had not been studied so far. Because the shear resistance of PC beams is affected by the prestressing level in the prestressing strands, the behaviors of the strengthened PC beams having different prestressing levels need to be clarified. Moreover, although the equations to predict shear increment of RC beams strengthened with externally bonded FRP sheets have been introduced in the design guidelines of many organizations (e.g., ACI and JSCE), the adoption of those equations to PC beams externally bonded with FRP sheets has been found conservative.

In order to investigate the strengthening effects of various parameters (strip spacing, stiffness, prestressing levels in the strands) and shear-resisting mechanism of pre-tensioned PC beams strengthened with externally bonded CFRP sheets, experiment on 12 specimens have been conducted. This paper discusses the enhancement of the stiffness, strength, and shear-resisting mechanisms of pre-tensioned PC beams based on the experimental outcomes. In addition, the increments in shear strength in experimental results were compared to the predictions by the equations of ACI and JSCE guidelines.

2. EXPERIMENTAL PROGRAM

(1) Details of pre-tensioned PC beams

The details of pre-tensioned PC beams are shown in Fig. 1. The specimens were prepared in two types (B0 and B1) in which different initial prestress ($P_i$) values were introduced in prestressing strands. The specimens had rectangular cross-section of 150 mm by 300 mm and the shear-span-to-depth ratio equalled to 3. Two high-strength bar with the diameter of 22 mm (nominal area of 380.1 mm$^2$) and two pre-stressing strands (seven-wire type, SWPR7BL, JIS G 3536) with diameter of 12.7 mm (nominal area of 98.71 mm$^2$) were arranged in the tension zone. In order to obtain the designed effective stress at the loading time, the values of the initial prestress were determined with the consideration of the prestress during the curing time and the CFRP sheet treatment, which was assumed and calculated based on JSCE standard. The prestressing strands were tensioned with the initial stress of 950 N/mm$^2$ for B0 and 1190 N/mm$^2$ for B1. Two 16-mm diameter deformed bars were used in the compression zone. Stirrups were not provided in the shear span. The material properties of pre-tensioned PC beams are given in Table 1 and Table 2.

| Table 1 | Material properties of PC beams. |
|---------|---------------------------------|
| Concrete | Prestressing strands | Steel |
| $f'c$ (N/mm$^2$) | $f_{pu}$ (N/mm$^2$) | $f_{py}$ (N/mm$^2$) | $f_y$ (N/mm$^2$) | $f_{pu}$ (N/mm$^2$) | $f_{py}$ (N/mm$^2$) | $f_y$ (N/mm$^2$) |
| 50 | 1,945 | 1,854 | 373 | 386 | 947 |
| $f'c$: compressive strength of concrete; $f_{pu}$, $f_{py}$: ultimate and yield strength of prestressing strand; $f_y$: yield strength of non prestressing steel |

| Table 2 | Properties of CFRP sheets. |
|---------|----------------------------|
| Types | $t_f$ (mm) | $f_{pu}$ (N/mm$^2$) | $E_f$ (N/mm$^2$) |
| a1 | 0.111 | 3400 | 2.3x10$^5$ |
| a2 | 0.333 | 3400 | 2.3x10$^5$ |
| b | 0.165 | 2900 | 3.9x10$^5$ |
| $t_f$: thickness of sheet; $f_{pu}$: ultimate strength, $E_f$: elastic modulus |
(2) Strengthening schemes

Table 2 shows the properties of unidirectional CFRP sheets used in this experiment. Type “a1” and “a2” sheets had the same properties of tensile strength and elastic modulus while type b indicates higher elastic modulus sheets. The only difference between types “a1” and “a2” was the thickness of sheet.

Figure 2 illustrates the attaching layout of the externally bonded CFRP sheets. The experimental cases are listed in Table 3. B0 and B1 denote the non-strengthened specimens with different initial prestresses. The name of strengthened specimens indicates non-strengthened beam, ratio of CFRP, strip (S) or continuous (C) sheet, fully (F) wrapping and types of CFRP sheet (b). It is noted that types “a1” and “a2” were omitted in the specimen names. The last component “w” was added to the specimen name (case 11) in order to differentiate it from the beam that was strengthened with the same CFRP reinforcement ratio but with a different width of the strips.

For B0-strengthened cases, the strips were bonded with the decrease in the spacing in cases 3 and 4. Case 5 had the same strengthening layout with case 4, except the thickness of the sheet was different. In cases 6 and 7, the continuous sheets were bonded at the center of the shear span. The width of the sheets was increased from 150 mm in case 6 to 300 mm in case 7. For case 8, the continuous sheet covered the whole shear span.

The details of bonded strips for strengthening B1 beams are shown in cases 9 to 12. To investigate the effect of prestress levels, the strengthening schemes of cases 9 and 10 were similar to those of cases 5 and 4 for B0. The same CFRP reinforcement ratio was used in cases 9 and 11; nevertheless, in case 11, the width and spacing of strips were doubled. The strips of high elastic modulus sheets were adopted in case 12 and bonded with the same layout of that of case

![Diagram of Strengthening Layout](image)

**Fig.2 Strengthening layout.**

![Diagram of Table 2](image)

**Table 2** showed the properties of unidirectional CFRP sheets used in this experiment. Type “a1” and “a2” sheets had the same properties of tensile strength and elastic modulus while type b indicates higher elastic modulus sheets. The only difference between types “a1” and “a2” was the thickness of sheet.

![Diagram of Table 3](image)

**Table 3** List of experimental cases.

| No. | Name     | \( t_f \) (mm) | \( w_f \) (mm) | \( s_f \) (mm) | \( \rho_f \) (%) |
|-----|----------|----------------|----------------|----------------|----------------|
| 1   | B0       | -              | -              | -              | -              |
| 2   | B1       | -              | -              | -              | -              |
| 3   | B0-1.1SF | 0.333          | 50             | 200            | 0.11           |
| 4   | B0-1.9SF | 0.333          | 50             | 120            | 0.19           |
| 5   | B0-0.6SF | 0.111          | 50             | 120            | 0.06           |
| 6   | B0-1.0CF | 0.111          | 150            | -              | 0.10           |
| 7   | B0-1.5CF | 0.111          | 300            | -              | 0.15           |
| 8   | B0-4.4CF | 0.333          | 680            | -              | 0.44           |
| 9   | B1-0.6SF | 0.111          | 50             | 120            | 0.06           |
| 10  | B1-1.9SF | 0.333          | 50             | 120            | 0.19           |
| 11  | B1-0.6SFw| 0.111          | 100            | 240            | 0.06           |
| 12  | B1-0.9SFB| 0.165          | 100            | 240            | 0.09           |

\( t_f \): thickness of sheet; \( w_f \): width of strip/sheet; \( s_f \): spacing between strips; \( \rho_f \): CFRP reinforcement ratio

11.

The PC beams were fully wrapped in this experiment in order to provide a sufficient anchorage bond length of CFRP sheets; hence, a premature debonding due to an insufficient bond length could be avoided. The CFRP sheets were bonded to the PC beams using wet layup methods by impregnating an epoxy resin. First, the concrete surfaces were slightly ground by a handy grinder to make them smooth. In order to avoid the concentration of stresses at corners of the rectangular section, the corners were rounded with a radius of equal to or greater than 15 mm. In the next step, a primer was painted on the concrete surfaces. Then, a sheet layer was pasted. After the CFRP sheets were pasted, the sheets were coated with a finishing layer of epoxy resin. The amounts of primer and epoxy resin were based on manufacturer’s guidelines. Finally, the strengthened beams were cured in room temperature for at least seven days before the loading test.

(3) Instrumentations and loading method

The strain gauges were attached to prestressing strands, D22 deformed bars, and top fiber of the concrete section at the middle of span as shown in Fig. 1. In addition, in order to obtain tensile behavior in the bonded sheets, the strain gauges were bonded on surface of the CFRP sheets (Fig. 2). The concrete strains were also bonded at several sections along the shear span to measure behaviors of the compressive strains of the concrete during the loading. Four linear variable differential transducers were used to monitor the vertical displacement of specimens. Two transducers were set at the middle of span and the other two were installed at each support.

The specimens were subjected to a static
four-point bending load until failure. During the loading test, all the measurements were recorded by a data logger. Furthermore, two cameras were installed to capture the status of cracks in each shear span with the interval of 10 kN of the applied load. The cracking propagation process was marked with the value of applied load.

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

The experimental results of all cases are summarized in Table 4 in terms of the effective stress in prestressing strands, compressive strength of concrete, ultimate load, calculated flexural capacity, ultimate shear capacity, increment in ultimate shear capacity, shear increment ratio, and failure behavior.

(1) Enhancement on shear strength and post-cracking stiffness

a) Effects of CFRP reinforcement ratio of same sheet thickness

Figure 3 shows the relationship between the normalized shear stress and the displacement of the PC beams strengthened with 0.333-mm thick sheet compared to the control beam (B0). Since the influence of concrete strength on the flexural tension shear capacity or web shear capacity of PC beams is expressed in the existing design equations\(^9\) by the term of the square root of the compressive strength of concrete, the shear force in the experiment was normalized by the square root of the compressive strength of concrete to eliminate the influence of the variation in concrete strengths. Before the diagonal crack occurred, all the specimens showed a similar behavior. After the formation of diagonal crack, the slopes of the curves became steeper in the beams strengthened with a smaller spacing of strips. Particularly, the stiffness of the beam strengthened with continuous sheet in the whole shear span was dramatically enhanced in the post-cracking region. Furthermore, with the same thickness of the sheets, the increase in shear strength was directly proportional to the increase in ratio of CFRP reinforcement.

Table 4 Summary of the experimental results.

| No. | Name       | f_{uc} (N/mm\(^2\)) | f_{c'} (N/mm\(^2\)) | P_u (kN) | P_m (kN) | V_u (kN) | V_{in} (kN) | \(r_{in}\) | Failure behaviors          |
|-----|------------|----------------------|----------------------|----------|----------|----------|------------|-----------|---------------------------|
| 1   | B0         | 785                  | 66.8                 | 320.6    | 526.6    | 160.3    | -          | 1.00      | Diagonal tension            |
| 2   | B1         | 905                  | 68.3                 | 332.8    | 530.3    | 166.4    | -          | 1.04      | Diagonal tension            |
| 3   | B0-1.1SF   | 756                  | 66.8                 | 355.9    | 526.6    | 177.7    | 17.4       | 1.11      | CC at tip of diagonal crack |
| 4   | B0-1.9SF   | 821                  | 64.1                 | 371.2    | 520.4    | 185.6    | 25.3       | 1.16      | CC at tip of diagonal crack |
| 5   | B0-0.6SF   | 809                  | 64.1                 | 369.0    | 520.4    | 184.5    | 24.2       | 1.15      | CC + partial CFRP rupture   |
| 6   | B0-1.0CF   | 815                  | 70.7                 | 476.5    | 534.8    | 223.6    | 63.3       | 1.39      | DT + CFRP rupture            |
| 7   | B0-1.5CF   | 790                  | 70.7                 | 444.2    | 534.8    | 222.1    | 61.8       | 1.39      | CC in middle of span        |
| 8   | B0-4.4CF   | 783                  | 64.1                 | 461.9    | 520.4    | 231.0    | 70.7       | 1.44      | CC in middle of span        |
| 9   | B1-0.6SF   | 1005                 | 68.3                 | 470.0    | 530.0    | 235.5    | 69.1       | 1.42      | CC in middle of span        |
| 10  | B1-1.9SF   | 995                  | 64.1                 | 414.4    | 520.4    | 207.2    | 40.8       | 1.25      | CC in middle of span        |
| 11  | B1-0.6SFw  | 934                  | 70.7                 | 457.2    | 534.8    | 228.6    | 62.2       | 1.37      | CC in middle of span        |
| 12  | B1-0.9SFb  | 870                  | 70.7                 | 476.5    | 534.8    | 228.3    | 71.9       | 1.43      | DT + CFRP rupture            |

For the 0.111-mm thick sheet (Fig. 4), a similar tendency on the enhancement in the shear strength and in the stiffness was obtained when the CFRP reinforcement ratio was increased, except in the case of B0-1.5CF. In this specimen, the localized debonding was observed at a similar load level to that of B0-1.0CF. Moreover, the failure of this specimen was governed by the crushing of concrete in the middle of the span; hence, a further improvement in the stiffness and strength was not obtained.

b) Effects of prestressing level in strands

Figure 5 compares the effects of externally bonded CFRP strips on PC beams having different prestressing levels. The PC beams were bonded with the same layout of CFRP strips and the difference in the thickness of strip was presented in different
CFRP reinforcement ratios.

For non-strengthened PC beams, the beam with higher prestressing level (B1) shows a noticeable enhancement in stiffness in the post-cracking region. Nevertheless, the enhancement in ultimate shear strength was slight compared to that of B0. Interestingly, the effectiveness of strengthening became dramatically higher in the B1 strengthened beams. With the sheet thickness of 0.111 mm, the ultimate shear strength increased 42% in B1-0.6SF compared to 15% gained in B0-0.6SF. For the sheet of 0.333 mm thickness, the increment of ultimate shear strength was 25% in B1-1.9SF, whereas, with the same strengthening scheme, only 16% of shear strength improved for B0-1.9SF. In addition, the failures of the strengthened PC beams having higher prestressing level were less brittle as can be seen in Fig. 5. The results implied that prestressing levels in the strands had significant influences on the effectiveness of strengthening.

c) Effects of stiffness \((t_f \text{ and } E_f)\) of CFRP sheets

Comparing the beams strengthened with the same layout with different thickness of the sheets in Fig. 5, the increase in the thickness of strips did not provide higher ultimate strength regardless of the prestressing levels. In Fig. 6, the behaviors of CFRP strain at the same location in different specimens are plotted. As a given load level, the thicker sheets resulted in the smaller strain. However, when the strain in the bonded sheets at the location, where the diagonal cracks intersected, reached a high value (over 5000\(\mu\varepsilon\)), the debonding propagated. Thus, the bonded sheets were not fully utilized. Except B0-0.6SF, in the other cases, the curves became steeper or suddenly decreased when the tensile strain in the strips rose around 5,000\(\mu\varepsilon\) to 6,000\(\mu\varepsilon\).

This result possibly was due to the fact that at a high strain value, the width of a diagonal crack would be at the level that caused a loss of aggregate inter-lock\(^7\), which is one of the shear-resisting mechanism in a beam. It should be noted that in the previous studies, Khalifa et al.\(^7\) proposed the limits of tensile strain in the bonded sheets in order to maintain the shear integrity of concrete as 4,000\(\mu\varepsilon\) to 5000\(\mu\varepsilon\) for the low modulus CFRP sheets and Triantafillou et al.\(^9\) took the value of 5,000\(\mu\varepsilon\). Once the debonding occurred in the thicker strip, the loss of the shear-carrying capacity was more significant, then, the failures were accelerated.

Figure 7 compares the ultimate shear strengths of PC beams strengthened by increasing the ratio of CFRP sheets with different sheet thicknesses. It is apparent that the increase in the CFRP reinforcement ratio by increasing the thickness of the sheets did not result in the enhancement in ultimate shear strength. The higher ultimate shear strength was obtained in the specimens strengthened with the thinner sheets even though a smaller CFRP reinforcement ratio bonded in these specimens. When the thinner sheets were used, at a given CFRP reinforcement ratio, the length of diagonal crack covered by the bonded sheets became greater compared to those of the thicker sheets. Thus, the thinner sheets probably more highly utilized and effectively restrained the widening of the cracks. The distributions of tensile strains in the CFRP sheets along the diagonal crack in different specimens are illustrated in Fig. 8. As can be seen, the strains in B0-1.5CF were smaller than that in B0-1.9SF indicating that the diagonal crack was better restrained in B0-1.5CF.

Figure 9 presents the comparison of CFRP strain development at two different locations in B1-0.6SFw and B1-0.9SFb. The two specimens were bonded with the same strengthening layout. The type a1 strips were used in B1-0.6SFw, whereas, the type b strips were used in B1-0.9SFb. The strains in both locations were smaller in the beam strengthened with the sheets of high elastic modulus. That indicated that
the opening of crack was better restrained. Furthermore, as the drops of the strains show the occurrence of progressive debonding, the debonding occurred at a smaller load in B1-0.6SFw. Therefore, the sheet with high elastic modulus effectively restrained the opening of the diagonal crack. That delayed the debonding process and a higher ultimate strength was achieved in B1-0.9SFb as shown in Fig. 10.

(2) Shear-resisting mechanisms and failure behaviors

a) Failure behavior of non-strengthened beams

Figure 11 illustrates the typical failure in the non-strengthened PC beam (B0). After several flexural cracks were formed in the shear span, a diagonal crack extended from a flexural crack (flexure-shear crack) at the applied load of about 180 kN. The crack diagonally propagated towards the loading point and the support. Figure 12 shows the crack patterns and the distributions of compressive strains in the horizontal direction in concrete along the shear span in B0 at two loading stages: before the presence of the flexure-shear crack and just before the ultimate load. At the measured sections, the concrete strain gauges were bonded on the surface of the concrete and parallel to the beam axis. As can be seen in the figure, the compressive strains rapidly increased in the vicinity above the flexure-shear crack. It indicates that the compression arch was generated in the shear span after the flexure-shear crack was formed and possibly transmitted a portion of the shear force from the loading point to the support. As a result, the load-carrying capacity still increased even though the shear reinforcement was not provided in the specimens. The beam failed when the diagonal crack widened and the aggregate interlock was lost. It led to an abrupt splitting of the concrete in the shear span as shown in Fig. 11. For the case of B1, except for the flexure-shear crack taking place at a higher load, the failure occurred in the same manner as that of B0.

b) Shear-resisting mechanisms and failure behaviors of strengthened PC beams

Figure 13 plots the behaviors of the strains in concrete at location (9) (see Fig. 12) in the strengthened PC beams compared to that of non-strengthened beams. Similar to that in B0, in the strengthened beams, the strains above the diagonal crack turned into compression and rapidly increased after the formation of the flexure-shear cracks indicating the presence of the compression arch in the strengthened specimens. As a portion of the applied shear force was transmitted by the compression arch, the shear portion carried by the bonded sheets was
In general, the shear resistance in the strengthened PC beams can be described in two stages: before the presence of diagonal crack and after the presence of diagonal crack. Before the occurrence of diagonal crack, the shear force is resisted by concrete with the influence of the prestressing level in the strands. The influence of prestress in the strands is shown in Fig. 14. Considering a concrete element at the centroid of the sections, the stresses acting on the element include shear stress and compressive stress induced by the prestress. Due to the combination of these stresses, the principal tensile stress becomes smaller with a higher prestress in the strands. As a result, a higher load is needed for propagating a diagonal crack.

After the formation of the diagonal crack, the shear force is resisted by the beam and arch actions and the bonded sheets as illustrated in Fig. 15. The shear force carried by the beam action, which is similar to that in RC beam, consists of shear resisted by concrete in compression zone ($V_c$), shear transmitted along the diagonal crack through aggregate interlock ($V_a$), and dowel action of the longitudinal steels ($V_d$).

The failure of the strengthened beams, therefore, depended on the behavior of each component. According to the experimental results, the failure behaviors of the strengthened beams were divided into three groups: concrete crushing at the tip of flexure-shear crack, diagonal tension failure together with rupture of CFRP sheet, and concrete crushing in the middle of span (Fig 16).

First, the failure of concrete crushing at the tip of the flexure-shear crack is demonstrated in Fig. 16 a. This failure took place in the specimens in which the flexure-shear crack propagated deeply into the compression zone, close to the loading point. Consequently, the concrete portion above the tip of the crack was reduced. Thus, the concrete in this portion became insufficient to resist the shear force and crushed in the vicinity of the loading point. The damage of concrete above the tip of flexure-shear crack concurrently terminated the compressive path of the arch action. Therefore, the specimens could not resist a further loading and the failure occurred in a brittle manner.

The second type of failure was severe because the splitting of concrete and the rupture of bonded sheets took place simultaneously (Fig. 16 b). This failure occurred because (i) the tensile strain induced in the bonded sheets reached the rupture strain of the sheets (B0-0.6SF (Fig. 8) and B1-0.9SFb), or (ii) the loss of aggregate interlock along the flexure-shear crack caused a concrete splitting and rupture of CFRP as in B0-1.0CF. The failure was sudden and followed by an excessive drop in the load-carrying capacity.

Different from the brittle manners in the previous two cases, the failure in the concrete crushing in the
The compression zone at the middle of span was more ductile. In this case, the concrete in the middle of span reached its ultimate strain and started crushing. Nevertheless, the applied load still increased until the concrete in the compression zone was completely destroyed as shown in Fig. 16c. It was supposed that in these specimens, the combination of shear-resisting actions was maintained. As the applied load was increased, the portion of shear force transmitted by compression arch also increased, which induced additional compressive stress in the compression zone. At the middle of span, the concrete in compression zone resisted not only the stress caused by the bending moment, but also an additional stress resulted from the arch action. Hence, the concrete crushed when its ultimate strength was reached at the top fiber. Because the flexural strength of the section was still higher than the applied moment, a further load was gained until the concrete in this region was inadequate to resist the flexural bending or insufficient for the shear force transmitted.

The influences of different experimental parameters on the failures of the strengthened PC beams, which strongly affected the shear capacity, are discussed in the following section.

(3) Factors affecting failure behaviors of strengthened PC beams
a) Debonding in the bonded sheets
In order to clarify the behavior of bonded sheets, the strain gauges along the height of each strip were examined. Figures 17 and 18 illustrate the typical behaviors of CFRP strains when debonding occurred. In general, the strains in the bonded sheets developed rapidly at the location where the flexure-shear crack intersected. When a high stress was induced, the interfacial micro cracks started spreading in the vicinity of the crack; then, a progressive debonding occurred. Figure 17 describes the debonding between two locations of the strain gauges as the values of the strain gauges became approximately equal.

On the other hand, in Fig. 18, a localized debonding took place when the strain at a location gradually reduced while the applied load still increased. In this study, to determine the debonding status, the behavior of strains at each measured location and the gradient of the two adjacent strains were checked. The debonding load is defined as the load where the gradient of the strains between two locations was approximately zero for the case of the debonding between two locations, or the load where the strain at a location started to reduce for the localized debonding. It should be noted that the debonding position and process were different in the specimens depending on the locations where the strips/sheet intersected with the diagonal cracks.

b) CFRP reinforcement ratio of same sheet thickness
Figures 19 and 20 present the normalized shear stresses at different stages (debonding, concrete crushing, and ultimate) and average CFRP strains.
along the diagonal crack at the applied load of 369 kN in PC beams strengthened with various CFRP reinforcement ratios. The average strain was the average of the strains on CFRP strips/sheets along the diagonal cracks. For each strip, the value of the strain gauge, which located near the intersection of the strips with the diagonal crack, was taken. With the same thickness of the sheets, the failures of CFRP rupture or crushing of concrete at the crack tip took place in the specimens strengthened by a smaller amount of CFRP sheets. In the PC beams strengthened by smaller amount of CFRP sheets, the higher strains were induced in the sheets. As the crack opened widely, the shear force transmitted through aggregate interlock was reduced gradually. Once the debonding of the sheet took place, the remaining amount of CFRP sheets and the other shear-resisting mechanisms became insufficient to carry the shear force. That led to the failure by crushing of concrete above the tip of the diagonal crack. Thus, when the small amount of CFRP sheets was bonded, it was insufficient to restrain the widening of the diagonal crack. Therefore, the debonding of CFRP sheets had predominant influence on the failure of the specimen.

When the larger amount of CFRP sheets was bonded, the average strain induced in the bonded CFRP sheets became smaller at a given load level. Since the widening of the crack was restrained, the loss of aggregate interlock was postponed. Even though localized debondings were observed, the remaining bonded sheets and other shear-resisting mechanisms still effectively carried the increasing shear force. Thus, the higher shear strength could be achieved. In these cases, the strengthened beams failed due to the damage of concrete in the compression zone at the middle of span.

For the cases of B0-1.0CF and B0-1.5CF (Fig. 19), the progressive debonding was observed at the edges of the sheet at a similar load level. The narrow width of CFRP sheet in B0-1.0CF became insufficient to restrain the widening of the flexure-shear crack that caused the rapid loss of aggregate interlock, which accelerated the crushing of concrete in the middle of span. The beam, finally, failed by an intermediate splitting of concrete and rupture of CFRP as shown in Fig. 21. With the increase in the width of the sheet in B0-1.5CF, the brittle failure was prevented. Therefore, the PC beams strengthened with a sufficient amount of CFRP sheets could obtain higher ultimate shear strength as a result of maintaining the combination of the shear-resisting components.

c) Prestressing level

The comparison of the strain development in strips of the PC beams having different prestressing levels (Fig. 6) implied that the diagonal crack occurred at a higher load level in the beams having high prestressing level. Furthermore, the strips started to carry stress at a higher load. As a result, the smaller strains were induced at the same load level in the beam with higher prestress and the loss of aggregate interlock was delayed. The higher prestress also led to a lower rate of crack propagation. Figure 22 shows the different crack propagations in B0 and B1 at the same applied load of 220 kN. Thus, in the PC beams having the higher prestressing level, the propagation of flexure-shear crack was effectively postponed and the strain induced in the bonded sheets was reduced. As a result, the combination of the shear-resisting mechanisms remained. Meanwhile, the failures due to crushing of concrete at the tip of the diagonal crack or rupture of CFRP sheets were prevented in the PC beams having higher prestressing levels and the PC beams failed when the concrete crushed in the constant moment region. This failure was less brittle. Consequently, with the same strengthening layout, the higher effectiveness of strengthening can be obtained in the PC beams hav-
ing higher prestressing level (Fig. 23).

d) Stiffness \( (f_t \text{ and } E_t) \) of CFRP sheets

The shear strength at different stages and the average tensile strain along the diagonal crack at the applied load of 369 kN of the PC beams strengthened with the same layout of strips are presented in Fig. 23. The 0.111-mm thick sheets were used for the 0.6SF cases, whereas in the cases of 1.9SF, the sheet thickness was 0.333 mm. Although, the average CFRP strains induced in the strips were smaller for both the strengthened B0 and B1 beams, the completely debonding loads were smaller in the thicker strips. For the bonded FRP sheets, the bond stresses developed within a certain length known as the effective bond length.\(^{22}\) The previous studies\(^{23}, 24\) have reported that the increase in the stiffness of the sheets resulted in the greater effective bond length.\(^{22}\) When the debonding of the sheets initiated, the effective bond length shifted to the nearby inactive bond zone until the debonding progressed completely.\(^{20}, 25\) In the strengthened beams, the high strains were induced where the strips were intersected by the diagonal crack. It is supposed that at a certain level of the strain, the progressive debonding started (3 (1)c)), and the active bond zone shifted. Because longer effective bond length is required for thicker sheets, the debonding between two certain locations was completed sooner.

The debonded thicker sheets caused a greater reduction of the shear-carrying capacity of the bonded sheets that led to an increase in the portion of the shear force transmitted by the compression arch. Figure 24 compares the strain development in concrete at the top fiber of the midspan section in two specimens. After the debonding of strip in B1-1.9SF, the slope of the curve changed, indicating an increase in the compressive stress intensity in the concrete at the middle of the span. Thus, the crushing of concrete in the middle of span was accelerated and occurred at a smaller load in this specimen, which was strengthened by the thicker sheets.

For the PC beam bonded with the strips of high elastic modulus (B1-0.9SFb), as discussed in 3 (1)c), the stiffness of the beam remained and higher ultimate strength was attained. Because the high elastic modulus sheet having a smaller ultimate strength (Table 2) was adopted in the experiment, the tensile strain in the strip reached the rupture strain at the

![Concrete crushing](image1)

![Widen crack](image2)

![Debonding](image3)

**Fig.21** Failure in B0-1.0CF.

![Crack patterns in B0 and B1](image4)

**Fig.22** Crack patterns in B0 and B1 at P = 220 kN.

![Effects of prestress on shear resistance and average strain along diagonal crack.](image5)

**Fig.23** Effects of prestress on shear resistance and average strain along diagonal crack.

![Compressive strain in concrete at top fiber.](image6)

**Fig.24** Compressive strain in concrete at top fiber.
ultimate load. As a result, the rupture of CFRP sheets occurred.

It can be concluded that the increment of the ultimate shear strength in the strengthened PC beams resulted from the enhancement of the shear resistance by beam action, arch action, and the bonded sheets, which are strongly affected by the amount and thickness of the CFRP sheets and the prestressing level in strands. Furthermore, as the composite action is maintained, the failure is governed by the properties of concrete in compression or the ultimate strain of the bonded sheets.

4. COMPARISON OF EXPERIMENTAL RESULTS AND PREDICTIONS BY DESIGN GUIDELINES

(1) Review of equations in recent design guidelines

In the recent design guidelines of ACI\(^{16}\) and the recommendations of JSCE\(^{17}\), the shear capacity of the RC beams strengthened in shear by externally bonded FRP system is computed by simply adding a shear carried by the bonded sheet to the shear capacity of the original beams. A truss analogy model is adopted to calculate the contribution of the bonded sheets to the shear capacity. The angle of diagonal crack is assumed to be equal to 45 degrees, and all the strips intersected by the diagonal crack contribute to the average stress level.

The contribution to shear capacity of the bonded FRP sheets is calculated based on the following equations given by the ACI guidelines\(^{16}\):

\[
V_f = \psi f_{x,f} \left( \sin \alpha + \cos \alpha \right) \frac{d_f}{s_f}
\]

where

- \(A_f\): cross-section area of FRP in spacing \(s_f\) (mm\(^2\))
- \(\psi\): reduction factor (0.95 for fully wrapped sections)
- \(\alpha\): angle formed by FRP sheet and member axis (degree)
- \(d_f\): effective depth of FRP shear reinforcement (mm)

The effective stress \(f_{x,f}\) is proportional to the strain developed in the bonded FRP at nominal strength.

\[
f_{x,f} = \varepsilon_{u,f} E_f
\]

where

- \(\varepsilon_{u,f}\): effective strain level in FRP reinforcement attained at failure (mm/mm)
- \(E_f\): modulus of elasticity of FRP (N/mm\(^2\))

For the fully wrapped sheets, based on the test observation, the guidelines of ACI recommend the limit of the effective strain in order to maintain the aggregate interlock of concrete:

\[
\varepsilon_{u,f} = 0.004 \leq 0.75 \varepsilon_{u,f}
\]

\(\varepsilon_{u,f}\): rupture strain of FRP (mm/mm)

It can be seen that the properties of concrete are not presented in the equations of ACI guidelines for the fully wrapped sheets and the contribution of the bonded sheets to shear capacity is associated with the increase in the amount of FRP reinforcement and the elastic modulus of the sheets.

On the other hand, JSCE\(^{17}\) provides a coefficient expressing the shear-reinforcing efficiency of the continuous fiber sheets (\(K\)) in its equation:

\[
V_f = K \left[ A_f f_{x,f} \left( \sin \alpha + \cos \alpha \right) \right] \frac{d_f}{s_f}
\]

\[
K = 1.68 - 0.67 R; \quad 0.4 \leq K \leq 0.8
\]

\[
R = \left( \rho_f E_f \right)^{1/4} \left( f_{x,f} / E_f \right)^{1/3} \left( f_{f,c} / f_{u,c} \right)^{1/3}
\]

with \(0.5 \leq R \leq 2.0\)

where

- \(f_{u,c}\): tensile strength of FRP (N/mm\(^2\))
- \(f_{f,c}\): compressive strength of concrete (N/mm\(^2\))
- \(E_f\): modulus of elasticity of FRP (kN/mm\(^2\))
- \(\rho_f\): FRP reinforcement ratio
- \(z\): lever arm length (mm)

In the equations of JSCE, the effects of the properties of the sheets and compressive strength of concrete are considered as shown in Eq. (6). With the higher tensile strength, smaller stiffness of the sheets, and smaller compressive strength of concrete, the value of \(K\) becomes smaller.

(2) Comparison of experimental results and calculations

The increment of ultimate shear force \((V_n)\) in the experiment, which is computed by subtracting the ultimate shear capacity of the non-strengthened PC beams (B0 and B1) from the ultimate shear force of the strengthened beams \((V_u)\), is given in Table 4. Figure 25 shows the ratio of the shear increment in the experiment by the contribution to shear capacity of the bonded sheets, which was calculated based on the equations of ACI and JSCE. There is a large scatter between the calculations and the experimental values. The shear strength increments in PC beams having higher prestressing level were highly underestimated by the equations. In contrast, all the cases of the PC beams strengthened with the sheets of 0.333 mm thickness were overestimated. Furthermore, the calculations by JSCE equations show...
smaller ratios to the experimental results in comparison to those calculated by the equations of ACI.

There were numerous causes of the significant inconsistency between the predictions and the experimental results. First, the coefficients in both design guidelines have been determined based on the data of strengthening RC beams. Therefore, the effects of prestressing on the propagation of the crack and reduction of tensile strains in the bonded sheets have not been taken into account. Second, in the design models, the increment in the shear capacity of the strengthened beams is determined by the shear capacity carried by the bonded sheets. Nevertheless, as discussed in 3 (2), the shear resistance in the strengthened PC beams is the combination of beam action, arch action, and the bonded sheets. The increment of the ultimate shear strength in strengthened beams resulted from the enhancement in combination with shear-resisting mechanisms, which are strongly affected by the ratio of CFRP sheets, the thickness of the sheets and particularly, the prestressing levels. The effect of prestress presented by the compressive stress induced at the centroid of concrete section \(\sigma_{ctg}\) on the increment of ultimate shear strength in the PC beams strengthened with different thickness sheets is illustrated in Fig.26. Moreover, the effect of the longitudinal prestress in the concrete as shown in Fig. 14, does not only reduce the principal tensile stress of the element but also leads to a flatter angle of diagonal crack\(^{2,3}\). In the experiment, the angles of flexure-shear crack varied from 26 to 31 degrees, which were much smaller in comparison to the assumed 45 degrees in the calculations. As a result, a larger amount of CFRP sheets intersected with the diagonal crack and contributed to resist the shear force.

Figure 27 plots the values of coefficient \(K\) in the experiment versus \(R\) based on Eq. (6) of JSCE. The values of \(K^{exp}\) is computed from the following equations:

\[
K^{exp} = V_{\infty}/V_0
\]  

with

\[
V_\infty = \sqrt{\frac{A_j f_s (\sin \alpha + \cos \alpha)}{s_j}}
\]  

\[V_{\infty}: \text{increment of ultimate shear capacity in the experiment}
\]

It is noted that the experimental values of \(K\) exceeded the limitation of JSCE’s equations. There was a tendency of decrease in the values of \(K\) corresponding to the increase in \(R\) values. Nevertheless, more data are necessary to evaluate the influence of the prestress and other factors on the strengthened PC beams.

The comparison of the experimental results and the existing design models of ACI and JSCE has clarified that the equations of the existing models were not appropriate for predicting the shear capacity of PC beams strengthened by bonded CFRP sheets. It is suggested that the influence of arch action on the increment of the shear capacity of the PC beams strengthened by externally bonded FRP sheets needs to be further investigated.

5. CONCLUSIONS

The experimental results of pre-tensioned PC beams without shear reinforcement strengthened in shear using externally bonded CFRP sheets are presented in this study. The results implied that the enhancement in the ultimate strength and the stiffness in the post-cracking region in the strengthened PC beams resulted from the combination of the shear-resisting mechanisms (beam action, arch action, and the bonded sheets).

The increase in the CFRP reinforcement ratio of the same sheet thickness and prestressing level in the strands improves the performance of these actions; hence, a higher effectiveness of the strengthening can be obtained. The smaller amount of the sheets or smaller prestress accelerates the debonding of the
bonded sheets and loss of aggregate interlock, which results in the failures of concrete crushing at the tip of the diagonal crack or rupture of CFRP sheets. When the composite actions are maintained, the failures depend on the ultimate strain of the bonded sheets or the concrete in compression.

The increase in the thickness of CFRP sheets leads to the reduction of the covered area of the diagonal crack, which is less effective in restraining the widening of the diagonal crack. Consequently, the debonding of the sheets and the failure of the strengthened beams are accelerated.

The comparison of the experimental results and the calculations based on ACI and JSCE guidelines demonstrated that the adoption of the existing design guidelines for strengthened PC beams with externally bonded sheet was highly inconsistent. Since the equations have been developed based on the test data of strengthened RC beams, the effects of prestress and the shear-resisting mechanisms in strengthened PC beams have not been included. Further experimental and analytical works are necessary to propose a rational design method for the strengthened pre-tensioned PC beams. The influences of the other factors such as shear-span-to-depth ratio and shear reinforcement ratio need to be addressed. In the real structures, for many cases, it may be impossible to provide the fully wraps for PC girders. Therefore, depending on the actual size of girders and the types of the bonded sheets, a sufficient anchorage bond length needs to be considered in the design.

REFERENCES
1) Mutuayoshi, H., Hai, N. D. and Kasuga, A. : Recent technology of prestressed concrete bridges in Japan, IAB-SE-JSCE Joint Conference on Advances in Bridge Engineering-II, pp. 46-55, 2010.
2) Nilson, A. H. : Design of Prestressed Concrete (Second edition), pp. 206-218, 1987.
3) Nawy, A. G. : Prestressed Concrete: A Fundamental Approach (Fourth edition), pp. 223-238, 2003.
4) Motavalli, M. and Cazderski, C. : FRP composites for retrofitting of existing civil structures in Europe: State-of-the-art review, Composites & Polymers 2007, pp. 1-10, 2007.
5) Kobayashi, A., Hidekuma, Y. and Saito, M. : Application of FRP for construction as high-durability materials in Japan, Proceedings of US-Japan Workshop on Life Cycle Assessment of Sustainable Infrastructure Materials, pp. 1-9, 2009.
6) Triantafillou, T. C. : Shear strengthening of reinforced concrete beams using epoxy-bonded FRP composites, ACI Structural Journal, Vol. 95, No. 2, pp. 107-115, 1998.
7) Khalifa, A., Gold, W. J., Nanni, A. and Abdel, A. A. : Contribution of externally bonded FRP to shear capacity of RC flexural members, Journal of Composites for Construction, pp. 195-202, 1998.
8) Triantafillou, T. C. and Antonopoulos, C. P. : Design of concrete flexural members strengthened in shear with FRP, Journal of Composites for Construction, Vol. 4, No. 4, pp. 198-205, 2000.
9) Chen, J. F. and Teng, J. G. : Shear capacity of fiber-reinforced polymer-strengthened reinforced concrete beams: Fiber-reinforced polymer rupture, Journal of Structural Engineering, Vol. 129, No. 5, pp. 615-625, 2003.
10) Modifi, A. and Chaallal, O. : Shear strengthening of RC beams with EB FRP: Influence factors and conceptual debonding models, Journal of Composites for Construction, Vol. 15, No. 1, pp. 62-74, 2011.
11) Cao, S. Y., Chen, J. F., Teng, J. G., Hao, Z. and Chen, J. : Debonding in RC beams shear strengthened with complete FRP wraps, Journal of Composites for Construction, Vol. 9, No. 5, pp. 417-428, 2005.
12) Chen, J. F. and Teng, J. G. : Shear capacity of FRP-strengthened RC beams: FRP debonding, Construction and Building Materials, Vol. 17, pp. 27-41, 2003.
13) Sas, G., Taljisten, B., Barros, J., Lima, J. and Carolin, A. : Are available models reliable for predicting the FRP contribution to the shear resistance of RC beams?, Journal of Composites for Construction, Vol. 13, No. 6, pp. 514-534, 2009.
14) Modifi, A. and Chaallal, O. : Shear strengthening of RC beams with externally bonded FRP composites: Effects of strip-width-to-strip-spacing ratio, Journal of Composites for Construction, Vol. 15, No. 5, pp. 732-742, 2011.
15) Kang, T. H. K. and Ary, M. I. : Shear-strengthening of reinforced & prestressed concrete beams using FRP: Part II – Experimental investigation, International Journal of Concrete Structures and Materials, Vol. 6, No. 1, pp. 49-57, 2012.
16) American Concrete Institute (ACI) Committee 440 : ACI 440.2R-08: Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures, 2008.
17) Japan Society of Civil Engineers (JSCE) : Recommendations for upgrading of concrete structures with use of continuous fiber sheets, 2001.
18) Murphy, M., Belbari, A. and Bae, S. W. : Behavior of prestressed concrete I-girders strengthened in shear with externally bonded fiber-reinforced-polymer sheets, PCI Journal, Summer 2012, pp. 63-82, 2012.
19) Japan Society of Civil Engineers (JSCE) : Standard specifications for concrete structures, 2012 (in Japanese).
20) Nguyen, T. T. D., Matsumoto, K., Yamada, M. and Niwa, J. : Behaviors of pretensioned PC beams strengthened in shear using externally bonded CFRP sheets, Proceedings of JCI, Vol. 37, No. 2, pp. 1183-1188, 2015.
21) American Concrete Institute (ACI) Committee 318 : ACI 318-11: Building code requirements for structural concrete, 2011.
22) Ouezdou, M. B., Belbari, A. and Bae S. W. : Effective bond length of FRP sheets externally bonded to concrete, International Journal of Concrete Structures and Materials, Vol. 3, No. 2, pp. 127-131, 2009.
23) Sato, Y., Asano, Y. and Ueda, T. : Fundamental study on bond mechanism of carbon fiber sheet, Concrete Library of JSCE, No. 37, pp. 97-115, 2001.
24) Ueda, T. and Dai, J. : Interface bond between FRP sheets and concrete substrates: properties, numerical modeling and roles in member behavior, Progress in Structural Engineering and Materials, Vol. 7, No. 1, pp. 27-43, 2005.
25) Nakaba, K., Kanakubo, T., Furuta, T. and Yoshizawa, H. : Bond behavior between fiber-reinforced polymer laminates and concrete, ACI Structural Journal, Vol. 98, No. 3, pp. 1-9, 2001.

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