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# Resilience of Prefabricated Concrete Frames using Hybrid Steel-Concrete

## Composite Connections

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### Abstract:

The progressive collapse resistance of prefabricated concrete frames with hybrid steel-concrete composite connections (HSCC) is rarely studied. To fill the gap, five (one reinforced concrete (RC) and four prefabricated concrete) 1/2 scaled beam-column sub-assemblages were designed and tested. In four prefabricated concrete specimens, different HSCC types, top and seat with web angle (TSWA), end plate (EP), top and seat angle (TSA), and web cleat (WC), were employed. Experimental results demonstrated that the failure patterns of prefabricated concrete frames with HSCC are different from that of the RC frame counterpart. The failure of the prefabricated concrete frames is governed by the shear fracture or thread stripping of the bolts in the connections while that of the RC frame is governed by the fracture of beam longitudinal rebar. Due to brittle failure of the connections in prefabricated concrete frames, both ultimate bearing capacity and deformation capacity of prefabricated concrete frames are smaller than the corresponding RC frame. Among them, the prefabricated concrete frame with the EP connection achieves the greatest first peak load or compressive arch action capacity. However, no catenary action could be developed in this specimen.

**Author Keywords:** Disproportionate collapse; Hybrid prefabricated concrete frame; Load resisting mechanism; Beam-column sub-assemblage; Experimental results

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## 23 1. Introduction

24 According to the ASCE/SEI-10 [1], disproportionate collapse is defined as “the spread of an initial  
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325 local failure from element to element eventually leading to the collapse of an entire structure or a  
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626 disproportionately large part of it.” In recent years, with the frequent occurrence of extreme events,  
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927 such as terrorist attacks, fires, and explosions, the likelihood of disproportionate collapse caused by  
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1128 extreme loads was increasing. The disproportionate collapse of structure usually results in substantial  
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1429 economic and life loss. How to evaluate or enhance the ability of structures to resist disproportionate  
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1730 collapse has attracted increasing attention from engineers and policymakers. Several design guidelines  
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2031 [1-4] were issued successively. The alternative load path (ALP) method was proposed and gradually  
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2232 became the most popular method in design or academic investigations as it was independent of the  
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2533 extreme loading [5]. Based on the ALP method, many studies were conducted in the past decades [6-  
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2834 9]. Structures may lose columns after an accidental event, causing a significant increase in shear force  
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3135 and flexural bending moment of the adjacent structural components. The beams adjacent to the lost  
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3336 columns can't resist such a significant amount of bending moment purely relying on their flexural  
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3637 strength. Therefore, exploring the inherent potential mechanisms is necessary. The inherent  
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3938 mechanisms include catenary action (CA) and compressive arch action (CAA) of the beams, and tensile  
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4239 membrane action (TMA) and compressive membrane action (CMA) of the slabs.

4440 With the development of modular building technologies, prefabricated concrete (PC) structures  
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4741 have been widely used in developed countries. Several studies [10-15] were carried out to investigate  
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5042 the disproportionate collapse resistance of conventional PC structures with dry or wet connections.  
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5343 Moreover, as steel-concrete composite joints offer an efficient method to connect precast columns and  
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5544 beams, various hybrid steel-concrete composite (HSCC) structures were developed. Compared to the  
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5845 conventional connection of prefabricated concrete structures, the use of hybrid steel-concrete  
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6146 composite (HSCC) connections offers advantages of both steel and concrete materials. It is possible to

47 reduce the size of both hybrid precast beams and columns through effective interaction between the  
48 two materials. Moreover, the HSCC connections can be connected by using high-strength bolts or  
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349 welds, which required less on-site work. Thus, the HSCC structure is ideal for PC buildings [16]. In  
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50 the past decades, the seismic behavior of PC frames with various types of HSCC connections was  
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951 investigated. Kulkarni et al. [17] and Li et al. [18] conducted experimental and analytical investigations  
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1152 on HSCC connections subjected to quasi-static cyclic loading to reveal their seismic behavior. It was  
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1453 found that the HSCC connections exhibited sufficient ductility under seismic loading. Li et al. [19]  
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16  
1754 tested a new-type of HSCC connection with end plates to investigate its seismic behavior. It was  
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2055 revealed that the HSCC connection performed even better than the conventional RC connection. Zhang  
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2256 et al. [20] developed a new type of HSCC connection using steel fiber concrete. It was found that the  
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24  
2557 proposed HSCC connection had a satisfactory ability to resist seismic loads. In addition, Zhang et al.  
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2858 [21] designed an innovative type of HSCC connection with energy dissipated plates and I-shaped steel  
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3159 connectors. It revealed that the hybrid joints with the energy dissipated plates and steel fiber concrete  
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3360 performed much better.  
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3661 From the above investigations, it can be seen that the HSCC connection has been proved to be an  
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3962 effective type of connection for PC frames to resist seismic loadings. However, the characteristics of  
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4263 disproportionate collapse are quite different from those of a seismic hazard: such as the loading  
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4464 direction, monotonic or cyclic, the main load resisting components, etc. Moreover, the resilience of PC  
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4765 frames with HSCC connection to resist disproportionate collapse was still unclear due to few available  
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5066 studies. Therefore, to fill this gap, four PC beam-column assemblies with HSCC connections and one  
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5367 counterpart sub-assembly using normal RC were designed and tested in this study. The difference  
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5568 between PC frames with HSCC connections and RC frame in terms of load resisting mechanisms were  
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5869 quantified and discussed.  
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## 70 2. Experimental program

### 71 2.1. Specimen design

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372 To investigate the resilience of PC frames with HSCC connections, five 1/2 scaled beam-column  
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673 assemblies were fabricated and tested. Among them, four PC beam-column s assemblies were designed  
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974 with various HSCC connections and the remaining one is a cast-in-place RC sub-assemblage, which  
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1175 was taken as a control specimen for comparison. The prototype frame of the specimen is a 5-story  
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1476 commercial building that is seismically designed by BS 8110-97 [22], following Kulkarni et al. [17].  
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16  
1777 Both longitudinal and transverse spans of the prototype frame are 6 m. The story height is 4.1 m. It was  
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1978 assumed that a middle column on the ground floor was lost due to extreme loading. The sub-assemblage  
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2279 above the lost column was extracted from the prototype frame for the test. Therefore, all specimens  
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2580 consist of a two-span beam, a short middle column stub, two side columns, and two overhanging beams,  
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2881 as shown in Fig. 1. The height of the side column and the length of the overhanging beam is determined  
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3182 by the position of inflection points. It was assumed that the inflection points were in the middle height  
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3383 of the column and 1/5 span of the beam. The cast-in-place RC specimen was named as RC-Con, and  
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3684 its dimensions and rebar details are illustrated in Fig. 2. As can be seen in the figure, the clear span of  
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3985 the beam is 2750 mm. The cross-section of the beam and column is 150 mm × 250 mm and 250 mm ×  
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4286 250 mm, respectively. In this study, a relatively low reinforcement ratio is used to make the effect of  
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4487 CAA obvious. T10 and T13 are used as longitudinal rebar while R6 is used as transverse rebar. The  
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4788 beam transverse rebar in the potential plastic hinge zone and the remaining zones were R6@70mm and  
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5089 R6@140mm, respectively, to follow the seismic design details. “T” and “R” represent deformed rebar  
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5390 and plain rebar, respectively. Detailed specimen properties of RC-Con were shown in Table 1.

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5591 For PC specimens, four different bolted connections were investigated. Based on the connection  
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5892 types, the specimens were named as follows: PC specimen with TSWA connection was named as PC-  
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6193 TSWA, PC specimen with end plate connection was named as PC-EP, PC specimen with top and seat

94 angle connection was named as PC-TSA, and PC specimen with web cleat connection was named as  
95 PC-WC. As shown in Fig. 3, except for the beam-column connection, the dimensions and rebar details  
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396 of the PC specimens are identical to those of the control specimen RC-Con. The detailing of different  
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697 HSCC connections is shown in Fig. 4 and tabulated in Table 2, and the detailed fabrication process was  
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998 illustrated in Fig. 5. For PC-TSWA, the PC beam and column were connected by top and seat angles  
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1199 with double web angles. The angles were connected to the PC beams and columns using M10 size  
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1400 bolts. In terms of PC-EP, the PC beam was connected to the PC column by an end plate. The end plate  
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1701 was welded to the PC beam, and six M10 bolts were applied to fasten the end plate to the PC column.  
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102 Only top and seat angles were used to connect the PC beam and column in PC-TSA, and only double  
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21  
2203 web angles were applied to connect the PC beam and column in PC-WC. It should be noted that the  
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2504 connectors of PC beams and columns were made of I-shaped steels with the section of UB178×  
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2805 101×19 and UC 203×203×46, respectively. The length of I-shaped steel in the PC beam is 125 mm and  
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3106 that in the PC column is 710 mm. The strength grade of the I-shaped steel is S355. The longitudinal  
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3307 rebar of the PC beams and columns are welded to the I-shaped connectors. For PC-TSWA, PC-TSA,  
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3608 and PC-WC, steel angles with a section size of L60×6 mm were used. For PC-EP, the end plates were  
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3909 160×220×10 mm and were welded to the I-shaped connectors in PC columns.

## 41 4210 2.2. *Property of materials*

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4511 According to the compressive and tensile splitting cylinder tests, the average compression strength  
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4712 and splitting tensile strength of the concrete are 34 MPa and 3.3 MPa, respectively. The critical  
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5013 properties of rebar and steel angles are listed in Table 3.

## 51 52 5314 2.3. *Test setup with instrumentation layout*

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5615 In this study, a displacement-controlled pushdown loading regime was applied. The test setup and  
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5916 instrumentation layout are displayed in Fig. 6. This setup was commonly used for pushdown tests of  
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6117 disproportionate collapse resilience of structures [23-27]. As mentioned above, the specimens were

118 extracted from the prototype frame at contraflexure points and 1/2 scaled down. Thus, the side column  
119 was pin supported with a roller installed horizontally at the top of the side column and overhanging  
120 beam end to represent the horizontal restraints from adjacent members. An axial force of  $0.2f'_cA_g$   
121 (where  $f'_c$  is the cylinder compressive strength of concrete while  $A_g$  is the gross area) was applied  
122 on the side column. To assess the bridging capacity of the specimen under the loss of a middle column,  
123 a vertical displacement with the rate of 0.2mm/s was applied to the top of the middle column by a jack  
124 with a 700 mm stroke (Item 1 in Fig. 6). A specially designed steel assembly (Item 3 in Fig. 6) was  
125 installed for preventing the undesired out-of-plane failure.

126 A series of load cells, displacement transducers, and strain gauges were applied before testing.

127 Load pins (Item 6 in Fig. 6) were applied in the pin supports to measure the reaction forces of pin  
128 support. Tensile/compressive both way load cell (Item 5 in Fig. 6) was applied in each horizontal roller  
129 to record its horizontal reaction force. During testing, the applied vertical load was measured by a load  
130 cell (Item 2 in Fig. 6) installed between the hydraulic jack (Item 1 in Fig. 6) and the reaction frame.  
131 Seven displacement transducers (Item 7 and 8 in Fig. 6) were applied along the beam span to record  
132 beam deflection shape. In addition, a series of strain gauges were glued on the rebar at special positions  
133 before casting, as shown in Figs. 2 and 3.

### 134 3. Test results

135 In the present study, one RC and four PC beam-column assemblies were tested by pushdown  
136 loading regime to assess the effects of different HSCC connections on the performance of PC frames  
137 to resist disproportionate collapse. The key results of the test were listed in Table 4 and discussed below.

#### 138 3.1 Global behavior

##### 139 3.1.1 Specimen RC-Con

140 Specimen RC-Con is the cast-in-place RC beam-column sub-assembly. The load-middle joint  
141 deflection (MJD) relationship and failure pattern of RC-Con are displayed in Figs. 7 and 8, respectively.

142 Initially, the first crack formed at the beam near the middle column at the MJD of 12 mm. With the  
143 MJD further increasing to 15 mm, the RC-Con reached its first yield load (FYL), which was defined  
1 as the vertical load corresponding to the first yield of the beam longitudinal rebar, of 25 kN. The actual  
B44 as the vertical load corresponding to the first yield of the beam longitudinal rebar, of 25 kN. The actual  
4  
5  
145 moment applied at the beam ends near the middle column was 16 kN·m, which is 64% of the maximum  
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8  
146 moment resisted at the section. Meanwhile, noticeable cracks were formed near the beam ends. When  
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147 the MJD reached 73 mm, the first peak load (FPL) of 42 kN was measured, which is 168 % of FYL,  
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148 indicating that the load capacity is increased by 68 % due to the mobilization of CAA. The ratio of the  
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16  
149 actual moment applied to the maximum moment resisted at the beam ends near the middle column  
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150 increased to 97 % at this time. When MJD reached 120 mm, the actual moment applied at the beam  
20  
21  
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151 ends near the middle column attained its maximum. With a further increase in MJD, the existing cracks  
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252 became wider, and the load resistance began to decrease due to the concrete crushing. Further  
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253 increasing MJD to 154 mm, fracture was occurred in the rebar near the middle column, resulting in the  
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154 load resistance dropping to 28 kN. Meanwhile, the widest cracks appeared at the location of curtailment  
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32  
33  
155 of the beam longitudinal rebar. When MJD reached 309 mm, rebar fracture was also observed in the  
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35  
356 curtailment region of the longitudinal rebar, leading to a further decrease in the load resistance. Once  
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357 MJD exceeds 448 mm, the load resistance enhanced rapidly since the development of CA. Finally, the  
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41  
158 ultimate load capacity (UL) of 39 kN was recorded at the MJD of 672 mm. As displayed in Fig. 8,  
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44  
159 rebar fracture appeared in the beam ends near the middle column and cut-off points of the longitudinal  
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46  
4760 rebar. Obvious concrete crushing occurred in the beam ends. Many penetrated cracks were formed  
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49  
5161 along the beams due to tensile axial force developed in the CA stage.

### 52 3.1.2 Specimen PC-TSWA

55 Specimen PC-TSWA is a PC beam-column sub-assembly with HSCC of TSWA connection.  
56  
57  
5864 The vertical load-MJD relationship and failure pattern are displayed in Figs. 7 and 9. Similar to RC-  
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60  
6165 Con, at the MJD of 10 mm, crack first formed at the beam ends near the middle column. When MJD

166 obtained 38 mm, the FYL was measured at 25 kN. The ratio of the actual moment applied to the  
167 maximum moment resisted at the beam ends near the middle column reached 65 %. Further growing  
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2  
368 MJD to 132 mm, the specimen reached FPL of 33 kN, which is 132 % of FYL, indicating that the load  
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669 capacity is increased by 32 % because of the mobilization of CAA. At this time, the ratio of the actual  
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8  
170 moment applied to the maximum moment resisted at the beam ends near the middle column increased  
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10  
171 to 99%. Then, with the increase of MJD, the load resistance of the specimen decreased slowly due to  
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172 concrete crushing occurred. Meanwhile, large openings were observed at the edge of the I-shaped  
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173 connectors near the middle column. The applied moment reached its maximum at MJD of 149 mm.  
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19  
174 When MJD reached 152 mm and 166 mm, the beam rebar fractured at the edge of the I-shaped  
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22 connectors near the middle column. When the MJD exceeded 229 mm, the bolts at the top angle near  
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24  
2576 the side column were sheared off. Then, the bolts at the web angle began to be sheared off at the MJD  
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27  
2877 of 326 mm. The CA began to develop at the MJD of 385 mm. Finally, following the shear fracture of  
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178 the bolts at the bottom angle near the side column, PC-TSWA reached its UL of 19 kN, which was only  
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32  
379 58 % of its FPL. As displayed in Fig. 9, rebar fracture occurred at the edge of the I-shaped connectors  
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35  
380 near the middle column. The failure at the side connection was controlled by bolt fracture in shear.  
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38  
381 Concrete crushing was observed in the beam ends near the middle column while concrete shed occurred  
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182 in the beam ends near the side column. The number of flexural cracks was much less than those in RC-  
42  
43  
44  
183 Con.

### 184 3.1.3 Specimen PC-EP

185 PC-EP is a PC beam-column sub-assembly with HSCC of end plate connection. The vertical  
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52  
5386 load-MJD relationship and failure pattern are shown in Figs. 7 and 10, respectively. Different from RC-  
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187 Con and PC-TSWA, the first crack of PC-EP occurred at the edge of the I-shaped connectors near the  
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588 middle column at the MJD of 12 mm. When MJD increased to 18 mm, the specimen reached its FYL  
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60  
189 of 21 kN. The ratio of the actual moment applied to the maximum moment resisted at the beam ends

190 near the middle column reached 48 %. Further increasing the MJD to 55 mm, the FPL of 35 kN, which  
191 is 167 % of FYL, was measured. This indicated that the mobilization of CAA increases the load  
192 capacity by 67 %. At this time, the ratio of the actual moment applied to the maximum moment resisted  
193 at the beam ends near the middle column increased to 95%. Then, with the increase of MJD, thread  
194 stripping of the bolt occurred at the connections near the side column, leading to the decrease in load  
195 resistance. When the MJD reached 130 mm, the applied moment reached its maximum. At the MJD of  
196 236 mm, the load resistance dropped to zero due to bottom rebar fracture. Finally, further increasing  
197 the MJD, as the side connections failed by thread stripping of all bolts, the beams and side columns  
198 were separated completely. Therefore, CA was not triggered and the load resistance held steady, which  
199 was only about 1 kN. As displayed in Fig. 10, rebar fracture occurred at the edge of the I-shaped  
200 connectors near the middle column. Moreover, the beam lost contact with the side column finally due  
201 to thread stripping of the bolts. Concrete crushing occurred at the edge of the I-shaped connectors near  
202 the middle column. The flexural cracks were much less than those in RC-Con and PC-TSWA. No  
203 penetrated cracks formed along the beam due to no CA mobilized during the test.

#### 3.1.4 Specimen PC-TSA

204 PC-TSA is a PC beam-column sub-assembly with HSCC of top and seat angle connection. The  
205 vertical load-MJD relationship and failure pattern are shown in Figs. 7 and 11, respectively. Similar to  
206 RC-Con and PC-TSWA, the first crack of PC-TSA occurred at the beam with the MJD of 8 mm. As  
207 the beam longitudinal rebar did not yield before FPL, no FYL is shown herein. The FPL of 20 kN was  
208 obtained at the MJD of 87 mm. The ratio of the actual moment applied to the maximum moment  
209 resisted at the beam ends near the middle column was 83% at this time. Then, further increasing MJD,  
210 the load resistance decreased slowly due to concrete crushing. When MJD reached 204 mm and 261  
211 mm, the angle bolts fractured in shear, leading to the load resistance dropping to 0 kN rapidly. The  
212 maximum moment of the beam ends near the middle column was measured at the MJD of 204 mm.

214 Then, with the increase of MJD, the load resistance increased again. Finally, when MJD reached 432  
215 mm, PC-TSA failed by the shear damage of the bolts and achieved its UL of 10 kN. As shown in Fig.  
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316 11, all connections failed by bolt shear fracture. Concrete crushing and spalling appeared in the beam  
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617 ends. Similar to PC-EP, no penetrated cracks formed in the middle zones of the beam due to mild CA  
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8  
9218 developed.

### 11 12 13 3.1.5 Specimen PC-WC

14  
15 PC-WC is a PC beam-column sub-assembly with HSCC of web cleat connection. The vertical  
16  
1721 load-MJD relationship and failure pattern are shown in Figs. 7 and 12. The first crack was measured at  
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19  
2022 beam ends at the MJD of 7 mm. Similar to PC-TSA, there was no FYL because no beam longitudinal  
21  
22  
2323 rebar yielded before FPL. When MJD reached 109 mm, the specimen obtained its FPL of 19 kN. The  
24  
2524 applied moment at the beam ends near the middle column was 14 kN·m, which is 77% of the maximum  
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27  
2825 moment. Then, the load resistance started to drop slowly accompanied by the concrete crushing and  
29  
30  
3126 spalling. After the MJD exceed 230 mm, the bolts began to fracture due to shear force. The maximum  
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33  
3427 moment of the beam ends near the middle column was measured at the MJD of 299 mm. Finally, as all  
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3628 bolts of the connection near the middle column were destroyed completely at the MJD of 436 mm, the  
37  
38  
3929 specimen reached its UL of 12 kN. As shown in Fig. 12, the failure of connections was caused by the  
40  
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4230 shear fracture of bolts. Concrete spalling occurred at beam ends. Similar to PC-EP, PC-TSA, no  
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4531 penetrated cracks occurred in the middle zones of the beam since less tensile axial force developed in  
46  
47  
4832 the beams.

### 49 50 51 3.2 Horizontal reactions

52  
5334 As the specimens' arrangements were all symmetrical, the average value of the horizontal reaction  
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5635 forces obtained at both sides of the specimens was used and discussed. The horizontal reaction force-  
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58  
5936 MJD curves of the specimens are displayed in Fig. 13. The maximum horizontal reaction forces of the  
60  
6137 specimens were tabulated in Table 4. Negative means compression force while positive represents  
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63  
64  
65

238 tension force. As given in Fig. 13(c), no tensile horizontal force was measured in PC-EP, which  
239 confirmed the above conclusion that no CA was developed in the beams due to complete tread stripping  
1  
2  
240 of the bolts at MJD of 337 mm. Different from PC-EP, both tensile and compressive reaction forces  
4  
5  
241 were measured in other specimens. The maximum horizontal compression forces of RC-Con, PC-  
7  
8  
242 TSWA, PC-EP, PC-TSA, and PC-WC are -71 kN, -65 kN, -67 kN, -56 kN, and -56 kN. The maximum  
9  
10  
11  
243 horizontal tension forces of RC-Con, PC-TSWA, PC-TSA, and PC-WC were 95 kN, 47 kN, 20 kN,  
12  
13  
1244 and 24 kN. Therefore, the CAA and CA developed in PC frames with HSCC connection were weaker  
15  
16  
1245 than those in RC-Con. Among the PC specimens, PC-EP achieved a similar compressive reaction force  
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19  
246 as that of PC-TSWA. PC-TSA and PC-WC achieved similar horizontal reaction forces.  
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22  
247 As shown in Fig. 13, for all specimens, in the stage of CAA, the compressive reaction force was  
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24  
2548 primarily from the bottom pin support. In the large deformation stage, the tensile reaction force of RC-  
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27  
249 Con was mainly provided by the horizontal restraint at overhanging beam. However, for PC specimens,  
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30  
250 the tensile reaction force was mostly from the top horizontal restraint and bottom pin support. This is  
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251 because, in the large deformation stage, the development of axial force in beams of PC specimens is  
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252 hindered due to the shear fracture of bolts. Therefore, for PC specimens, the shear forces of the side  
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3253 columns transferred from the beams are small, resulting in a small lateral deformation of the side  
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254 column. Thus, a relatively small force is transmitted to the horizontal restraint at the overhanging beam.  
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### 44 255 3.3 Deformation's shape 45 46

47  
256 The deformation's shape of the beams is monitored by LVDTs along the beam. Fig. 14 displays  
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49  
5257 the deformation's shape of the beams of RC-Con and PC-TSWA at critical status. The dashed straight  
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52  
53  
258 line represented chord rotation. As specified in DoD [4], it is calculated by the ratio of MJD to the  
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55  
259 clean beam span. As shown in Fig. 14(a), for RC-Con, the beams kept almost straight before the FPL.  
56  
57  
58  
260 With the MJD increasing to 250 mm, the rotation of the beam end near the middle column was greater  
59  
60  
6261 than that near the side column. In the last stage, the chord rotation overestimates the rotation of the  
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262 beam end near the side column and underestimates the rotation of the beam end near the middle column.  
263 This is because the plastic hinge formed at the curtailment of the beam longitudinal rebar. Different  
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2  
264 from RC-Con, as displayed in Fig. 14(b), the beams in PC-TSWA kept straight during the whole loading  
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5  
265 history. At the UL stage, the chord rotation could predict the rotation of the beam end near the side  
7  
8  
266 column accurately but overestimated the rotation of the beam end near the middle column due to plastic  
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11  
267 hinges occurring at the edge of the I-shaped connectors. For PC-TSA and PC-WC, the rotation of the  
12  
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268 beam ends was consistent with the chord rotation well. For PC-EP, the deformation of the beam was  
15  
16  
269 similar to that of PC-TSWA.

### 270 3.4 Strain gauge results

271 The location of strain gauges is shown in Figs. 2 and 3. The bottom longitudinal rebar of RC-Con,  
23  
24  
272 PC-TSWA, and PC-EP started to yield at the MJD of 15 mm, 38 mm, and 18 mm. However, no yielding  
26  
27  
273 was measured in PC-TSA during the whole test history while yielding strain was only measured at PC-  
29  
30  
274 WC until the stage of UL. Figs. 15 to 16 show the strain readings along with longitudinal rebar of PC-  
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33  
275 TSWA, and PC-EP, respectively. For PC-TSWA, after reaching FPL, the compression strain of the  
34  
35  
276 rebar near the middle column began to decrease with the increase of MJD, indicating that the CAA  
37  
38  
277 stage was shifted into the CA stage. However, beyond the stage of FPL, the decrease in load resistance  
40  
41  
278 of PC-EP was due to bolt thread stripping instead of concrete crushing. Therefore, different from other  
42  
43  
279 specimens, the compression strain of the rebar near the middle column of PC-EP continued to increase  
45  
46  
280 after the stage of FPL, which can be seen in Fig. 16. In addition, for PC-TSWA, in the large deformation  
48  
49  
281 stage (UL or MJD = 500 mm), most of the longitudinal rebar was in tension, which indicates that CA  
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52  
282 could have developed in these specimens. Conversely, most of the longitudinal rebar strains of PC-EP  
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54  
283 were very low in the large deformation stage, indicating that no CA was developed, which is consistent  
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56  
284 with the results of vertical loads and horizontal reactions. The development of the rebar strain of RC-  
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59  
285 con, PC-TSA, and PC-WC is similar to PC-TSWA.

286 **4. Analytical and discussion**

287 *4.1 Effects of HSCC connection types*

288 As shown in Table 4, the FPL of PC-TSWA, PC-EP, PC-TSA, and PC-WC is 79 %, 83 %, 48 %,  
289 and 45 % of that of RC-Con, respectively. Thus, the PC frames using the proposed four HSCC  
290 connections could not achieve equivalent behavior as cast-in-place RC frames regarding FPL.  
291 Moreover, the measured deformation capacity of RC-Con, PC-TSWA, PC-TSA, and PC-WC  
292 corresponding to their UL is 672 mm, 413 mm, 236 mm, 432 mm, and 436 mm, respectively. Therefore,  
293 the deformation capacity of PC-TSWA, PC-EP, PC-TSA, and PC-WC is 61 %, 35 %, 64 %, and 65 %  
294 of that of RC-Con, respectively. Furthermore, the UL of PC-TSWA, PC-EP, PC-TSA, and PC-WC is  
295 49 %, 36 %, 26 %, and 31 % of that of RC-Con, respectively. It should be noted that the UL is defined  
296 as the peak load in the re-ascending phase of the load history. Therefore, regarding the deformation  
297 capacity and UL, the PC frames with the proposed four HSCC connections still could not be equivalent  
298 to the cast-in-place frame. The lower FPL is due to the lower strength of the HSCC connection and  
299 weaker CAA. On the other hand, the lower deformation capacity is because the failure patterns of all  
300 PC specimens are controlled by shear fracture or thread stripping failure of the bolts, which are brittle  
301 failures and thus, reduce the ductility of the specimens. In consideration that the failure patterns of PC  
302 specimens may be changed if the greater size bolts or weaker steel plates are designed in HSCC  
303 connection. Therefore, more studies are urgently in need to further investigate the robustness of PC  
304 frames with HSCC connections to resist disproportionate collapse in the future.

305 For PC specimens, the FPL of PC-EP is larger than that of PC-TSWA. However, due to thread  
306 stripping failure at the MJD of 337 mm, which prevents the development of CA and thus, the  
307 deformation capacity and UL of PC-EP is much lower than that of PC-TSWA.

308 *4.2 Dynamic ultimate bearing capacity*

309 It should be noted that disproportionate collapse is generally a dynamic process. Therefore, it is

310 needed to assess their dynamic ultimate bearing capacity. Relied on the quasi-static pushdown load-  
 311 displacement curve, a capacity curve method [28] was adopted to calculate the dynamic ultimate  
 312 bearing capacity of the specimens. Previous studies [29] had been confirmed the feasibility of this  
 313 method. The mathematical equation of the capacity curve method is given in Eq. 1:

$$P_d(u_d) = \frac{1}{u_d} \int_0^{u_d} P_s(u) du \quad (1)$$

314 where  $P_d(u_d)$  and  $P_s(u)$  represent the capacity function and the nonlinear static loading estimated  
 315 at the displacement demand  $u$ , respectively.

316 Fig. 17 compares the dynamic load-MJD curves of the test specimens based on the capacity curve  
 317 method. As displayed in the figure, the dynamic ultimate bearing capacity of PC-TSWA, PC-EP, PC-  
 318 TSA, and PC-WC is 71 %, 73 %, 47 %, and 42 % of that of RC-Con, respectively. Thus, the quasi-  
 319 static pushdown curve is a good alternate method for the disproportionate collapse study. The dynamic  
 320 load increase factor is defined as the ratio of static ultimate bearing capacity to dynamic load-resisting  
 321 capacity [27]. Based on the test results and analytical results, the dynamic load increase factors of RC-  
 322 Con, PC-TSWA, PC-EP, PC-TSA, and PC-WC were 1.11, 1.22, 1.25, 1.11, and 1.19. Among them, the  
 323 dynamic load increase factor of the PC specimens is greater than that of the RC specimen due to the  
 324 brittle failure that occurred there.

#### 326 4.3 Effects of steel angles

327 As shown in Table 4 and Fig. 7, the FPL of PC-TSWA is 165 % and 174 % of that of PC-TSA and  
 328 PC-WC, respectively. Therefore, the web angles and top and seat angles increase the FPL by 65 % and  
 329 74 %, respectively. For UL, PC-TSWA, PC-TSA, and PC-WC are 19 kN, 10 kN, and 12 kN. The  
 330 corresponding MJD is 413 mm, 432 mm, and 436 mm. The results show that the additional web angle  
 331 or top and seat angle has little effect on the deformation capacity. However, the web angle and top and  
 332 seat angle could increase the UL by 90 % and 58 %, respectively. Analytical results indicated that the  
 333 web angle and top and seat angle enhanced the dynamic ultimate bearing capacity by 50 % and 69 %.

#### 334 4.4 De-composition of vertical load resistance

335 According to the force equilibrium of the deformed beam in Fig. 18, the vertical load resistance  
1  
2  
336 at the middle column could be divided into:

$$337 \quad P = \sum_{j=1}^2 (N_j \sin \theta_j + V_j \cos \theta_j) \quad (2)$$

338

339 where  $P$  = the applied load;  $N_j$  and  $V_j$  = the axial force and shear force;  $\theta_j$  = the rotation of the beam  
10 section at one of the joint interfaces.

1340 Based on Eq. (2), the contribution of the axial force (related to CA) and shear force (related to  
15 flexural action) could be calculated. The de-composition of load resistance is shown in Fig. 19. PC-  
17 TSWA, PC-TSA, and PC-WC have a similar response as RC-Con in terms of de-composition of load  
19 resistance. Before CA is triggered, the contribution of axial force is negative, and the vertical load was  
21 mainly provided by the flexural action. After the fracture of the rebars or bolts, the contribution of  
23 flexural action dropped significantly. In the CA stage, the contribution of CA is positive, while the  
25 contribution of flexural action kept decreasing. When the bottom rebar or bolts of the connection near  
27 the middle column fractured entirely, the contribution of flexural action transferred into negative. For  
29 PC-EP, as no CA developed, the vertical load is mainly provided by the flexural action while the  
31 contribution of CA is positive during the test.

## 350 5. Conclusions

351 To examine the resilience of PC frames using innovative hybrid steel-concrete composite (HSCC)  
352 connections to resist disproportionate collapse, a series of four PC beam-column assemblies with  
353 various HSCC connections as well as an RC sub-assembly were tested in this study. Relied on the  
354 experimental and analytical results, the main conclusions were given:

- 355 1. The failure pattern of PC frames with HSCC connections is different from RC frames. The failure  
356 of PC frames with HSCC connection is mainly controlled by the shear failure or thread stripping  
357 of the bolts. However, the failure of the RC frame is normally controlled by the fracture of top

358 longitudinal rebar at cut-off points or bottom longitudinal rebar at beam ends near the middle  
359 column. Moreover, the beams in PC frames with HSCC connections kept straight during the test,  
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360 which means rotation mainly concentrated into the HSCC connection.  
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361 2. RC frame has larger FPL and UL than those of PC frames with HSCC. Together with horizontal  
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362 reaction force results, considerable CAA and catenary action capacity are only measured in PC-  
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363 TSWA. Almost no catenary action is mobilized in the remaining PC frames with EP, TSA, or WC  
12  
13  
364 connections. Thus, the load resisting mechanism of PC frames with HSCC is highly dependent on  
15  
16  
365 the type of HSCC connection and different from that in RC frames.  
18  
19  
366 3. Comparing the results of PC-TSWA, PC-TSA, and PC-WC, the web angles and top and seat angles  
20  
21  
22  
367 enhanced the first peak load of PC frames by 65 % and 74 %, respectively. In addition, the  
23  
24  
25  
368 additional web angles or top and seat angles have little effect on the deformation capacity, but  
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27  
28  
369 could increase the UL by 90 % and 58 %.  
29  
30  
370 4. De-composition of the vertical load resistance indicated that the load contribution in PC-TSWA,  
31  
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33  
371 PC-TSA, PC-WC, and RC-Con is similar. In the small deformation stage, the vertical load mainly  
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36  
372 comes from flexural action. Nevertheless, the vertical load is mainly from the development of  
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39  
373 catenary action in the large deformation stage. However, for PC-EP, the vertical load is mainly  
40  
41  
42  
374 from the flexural action during the test.  
43

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**Table 1-Specimen Properties of Specimen RC-Con**

Specimen ID	Span (mm)	Beam section (mm×mm)	Column section (mm×mm)	Position of curtailment (mm)	Longitudinal Reinforcements				
					A-A section		B-B section		C-C section
					Top	Bottom	Top	Bottom	
RC-Con	2750	150×250	250×250	900	2T10 (0.48%)	2T10 (0.48%)	1T13+2T10 (0.88%)	2T10 (0.48%)	8T13

Note: The concrete cover thickness is 15 mm; T10 and T13 represent deformed rebar with a diameter of 10 mm and 13 mm, respectively;

Reinforcement ratio in brackets is calculated by  $A_s = b \times h_0$ , where  $b = 150$  mm and  $h_0 = 220$  mm; the A-A, B-B, and C-C sections are given in Fig. 2.

**Table 2-Summary of Beam-Column Joint of Precast Specimens**

Test ID	Connections	Beam	Column	End plate /Angle	Bolt
PC-TSWA	Top and seat with web angles	UB178×101×19 S355	UC203×203×46 S355	L60×6	Grade 8.8 M10
PC-EP	end plate	UB178×101×19 S355	UC203×203×46 S355	160×220×10	Grade 8.8 M10
PC-TSA	Top and seat angle	UB178×101×19 S355	UC203×203×46 S355	L60×6	Grade 8.8 M10
PC-WC	Web cleat	UB178×101×19 S355	UC203×203×46 S355	L60×6	Grade 8.8 M10

**Table 3-Material Properties**

Item	Diameter (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Elongation (%)
R6	6	404	575	22.1
T10	10	483	582	11.5
T13	13	533	604	13.2
Angle	N/A	345	488	31.0

**Table 4-Main Results Comparison**

Test ID	MJD at FYL (mm)	MJD at FPL (mm)	MJD at UL (mm)	MJD at the start of CA (mm)	FYL (kN)	FPL (kN)	UL (kN)	MHTF (kN)	MHCF (kN)
RC-Con	15	73	672	448	25	42	39	95	-71
PC-TSWA	38	132	413	385	25	33	19	47	-65
PC-EP	18	55	236	N/A	21	35	14	N/A	-67
PC-TSA	N/A	87	432	386	N/A	20	10	20	-56
PC-WC	N/A	109	436	393	N/A	19	12	24	-56

Note: MJD means middle joint displacement; CA represents catenary action; FYL, FPL, and UL represent first yield load, first peak load, and ultimate load, respectively; MHTF means the maximum horizontal tension force while MHCF represents maximum horizontal compression force.

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502 (e) PC-WC  
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503 **Fig. 14.** Deformation shape of double-span beam: (a) RC-Con; (b) PC-TSWA  
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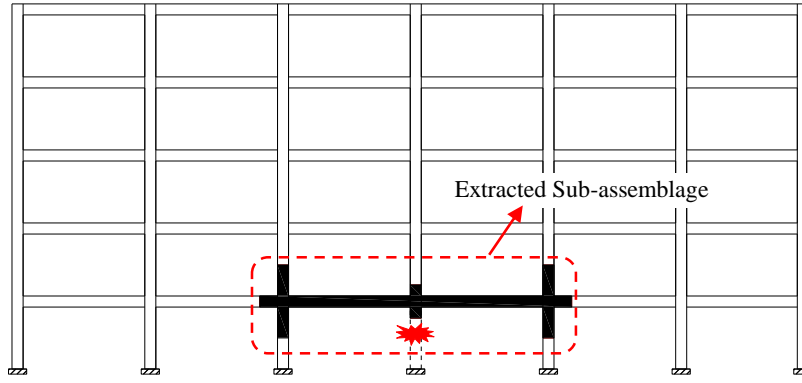
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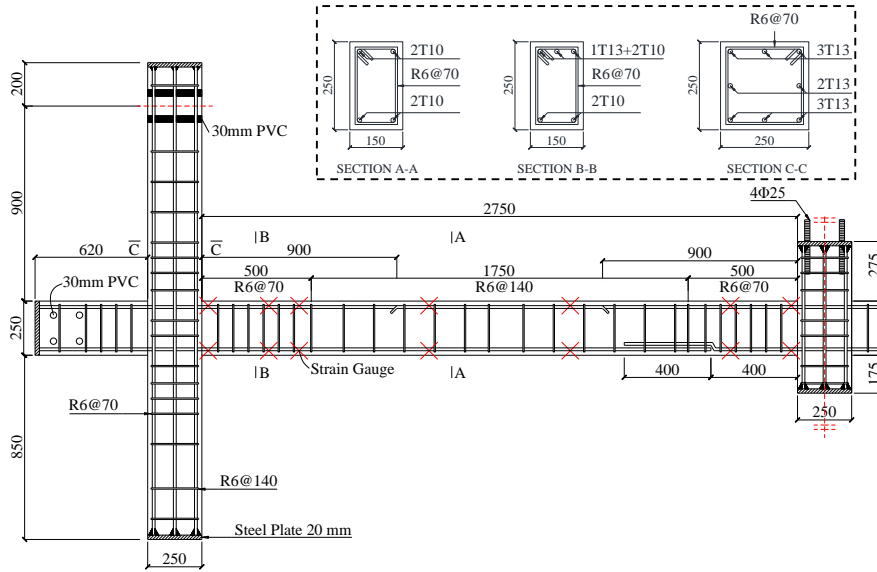
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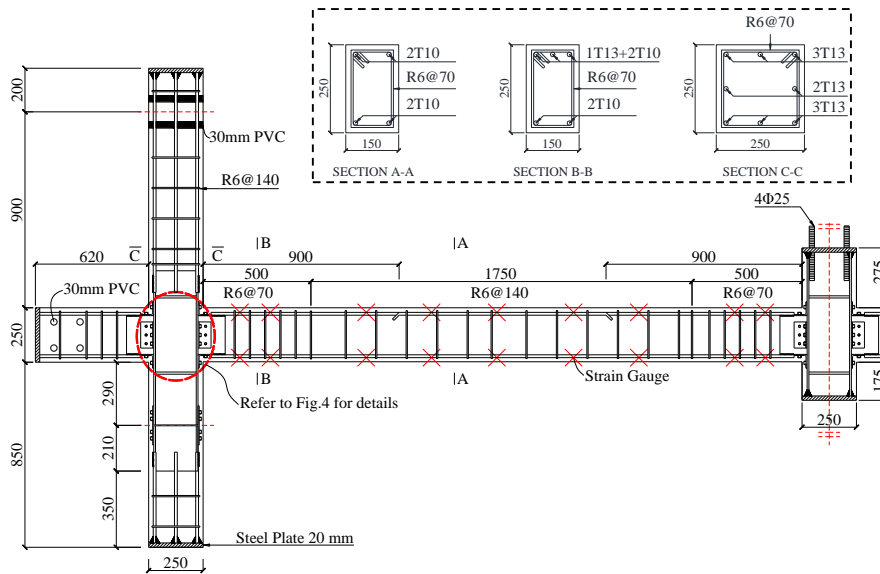
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**Fig. 1.** Elevation view of the prototype building

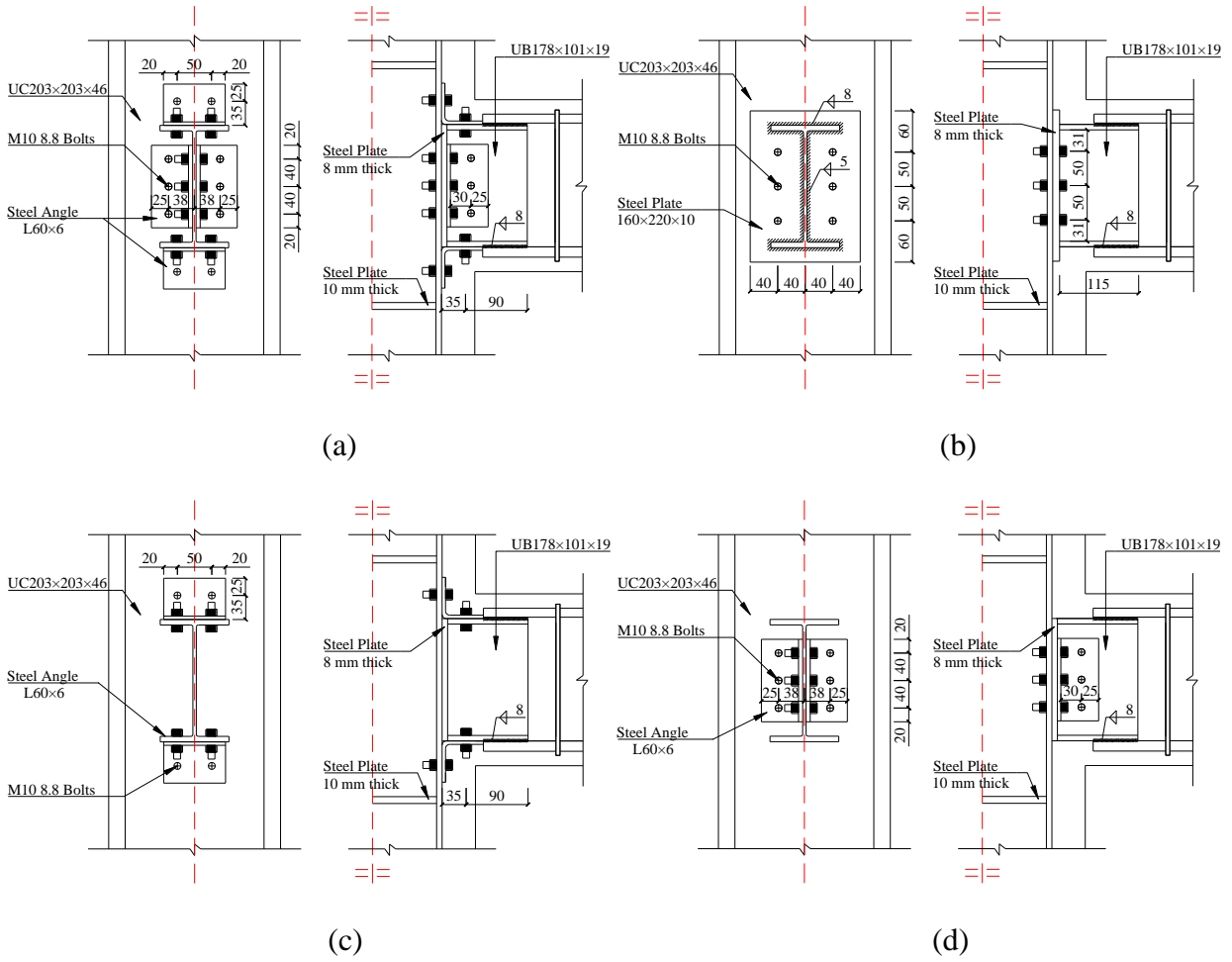


**Fig. 2.** Dimensions and reinforcement details of Specimen RC-Con

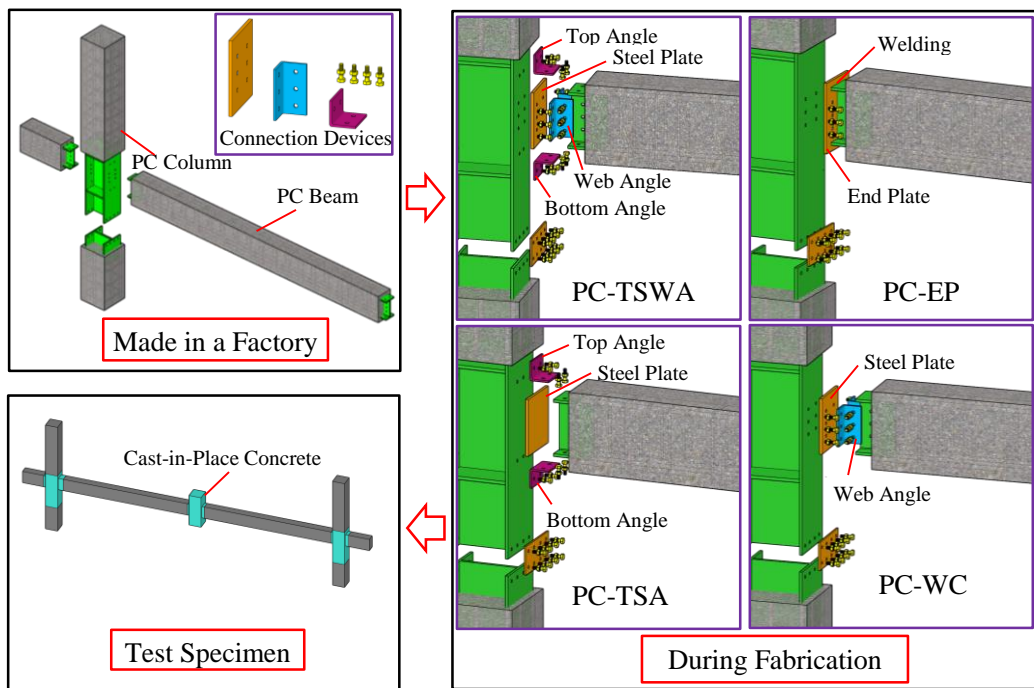


**Fig. 3.** Dimensions and reinforcement details of precast specimens

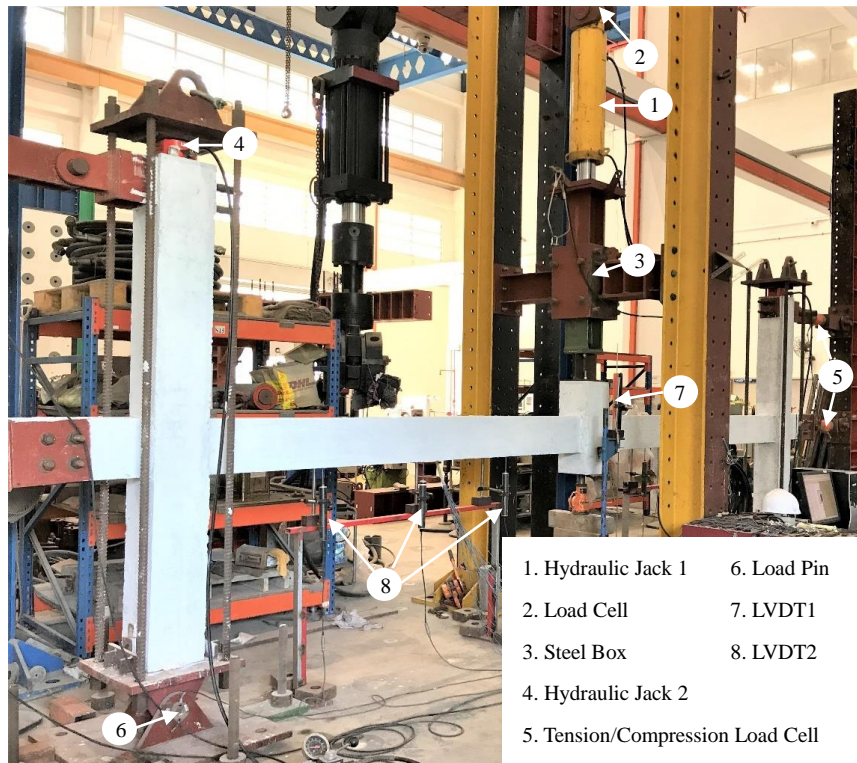
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**Fig. 4.** Connection details: (a) PC-TSWA; (b) PC-EP; (c) PC-TSA; (d) PC-WC



**Fig. 5.** Fabrication process of the PC specimens



- 1. Hydraulic Jack 1
- 2. Load Cell
- 3. Steel Box
- 4. Hydraulic Jack 2
- 5. Tension/Compression Load Cell
- 6. Load Pin
- 7. LVDT1
- 8. LVDT2

Fig. 6. Test setup

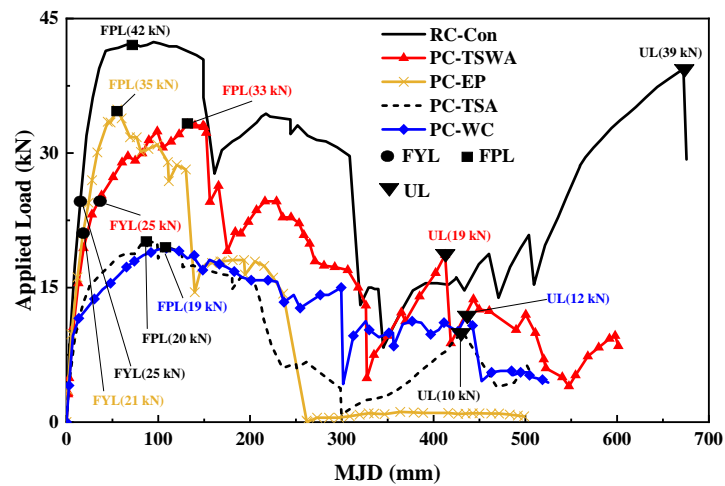


Fig. 7. Vertical load-MJD relationships of specimens

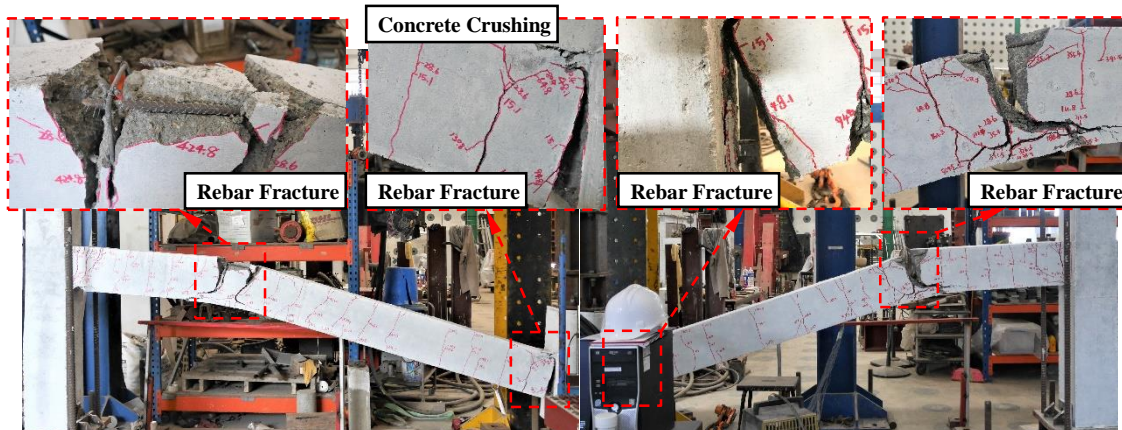


Fig. 8. Failure pattern of RC-Con

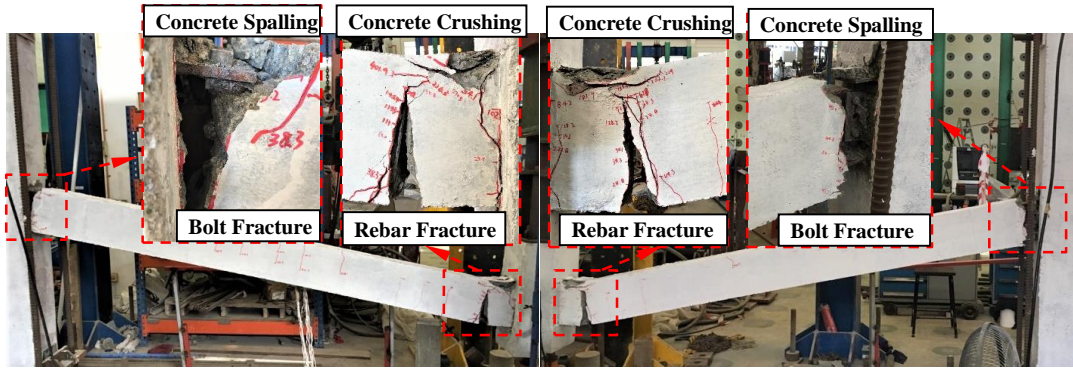


Fig. 9. Failure pattern of PC-TSWA

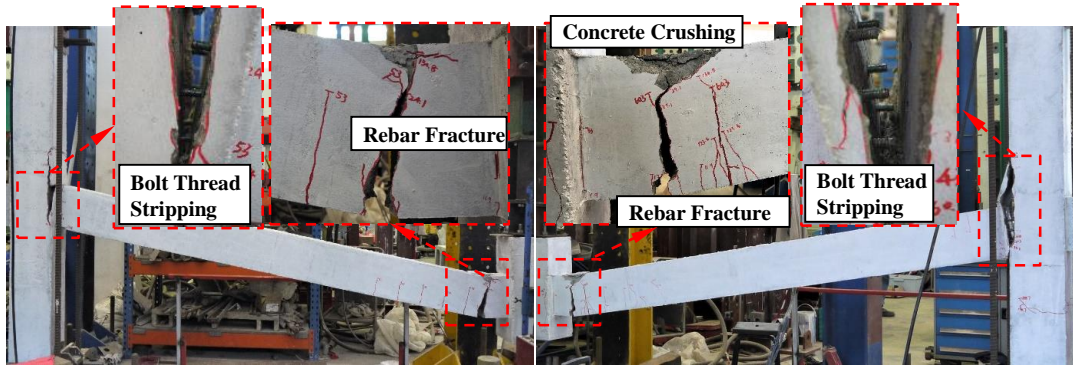


Fig. 10. Failure pattern of PC-EP

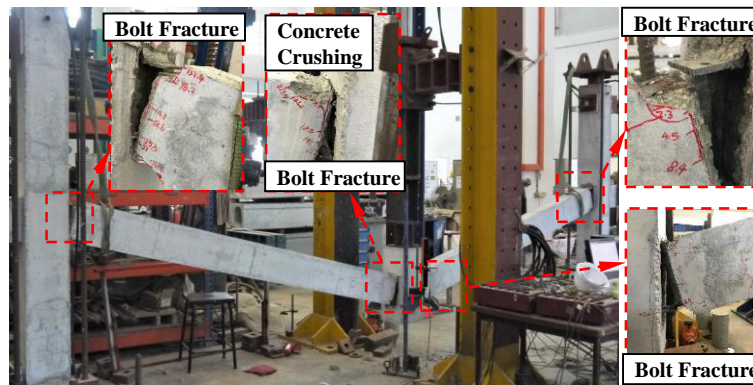


Fig. 11. Failure pattern of PC-TSA

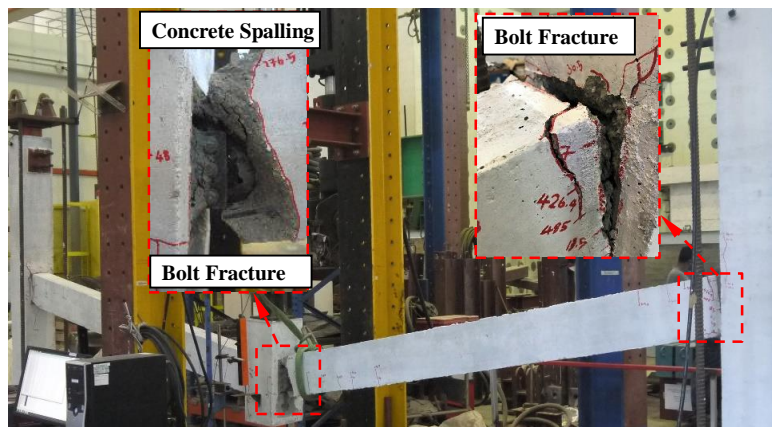
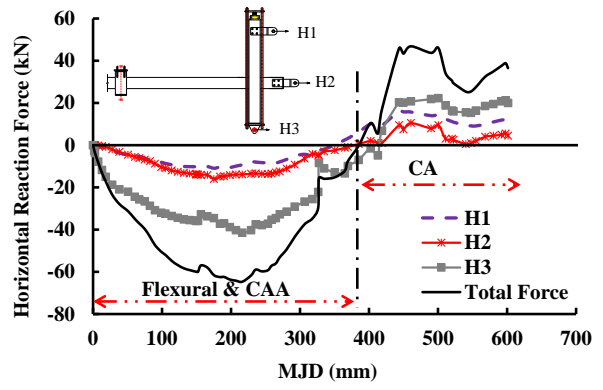
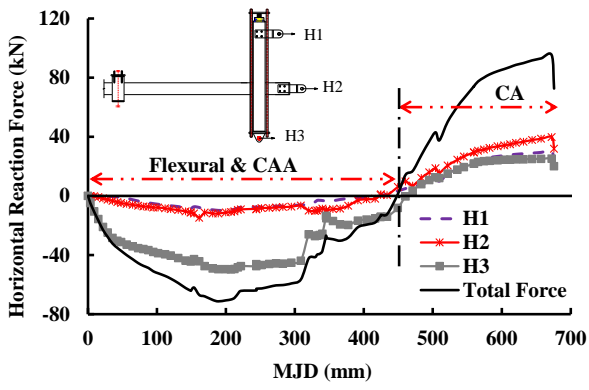
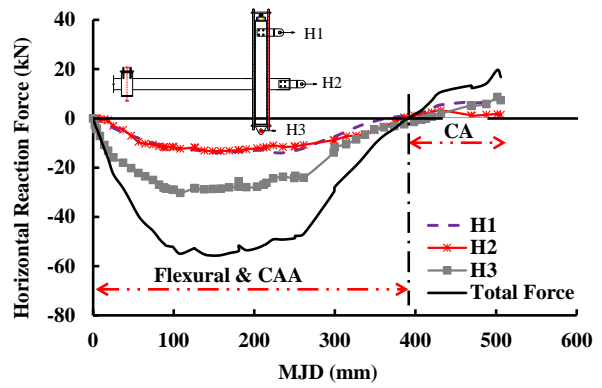
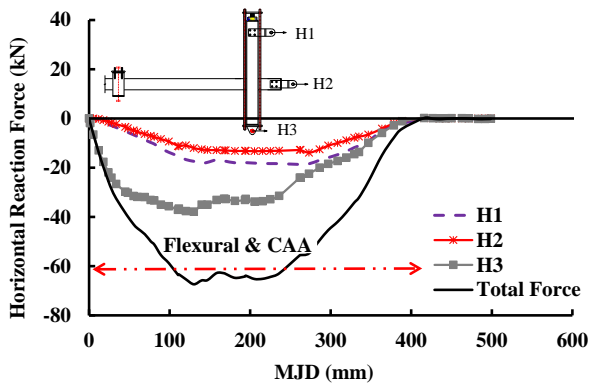


Fig. 12. Failure pattern of PC-WC



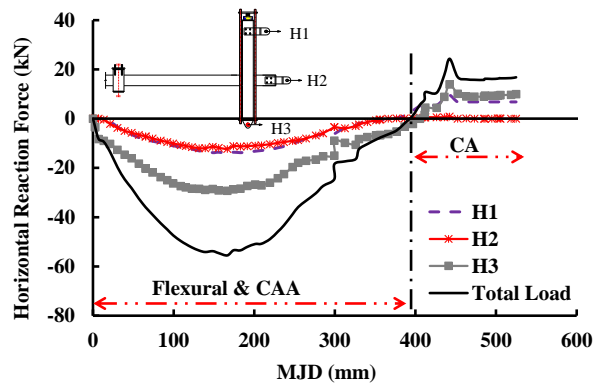
(a)

(b)



(c)

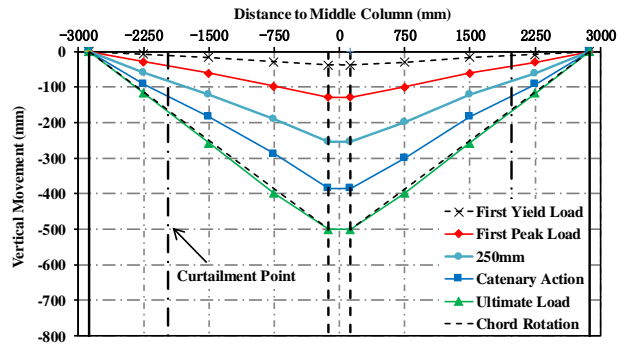
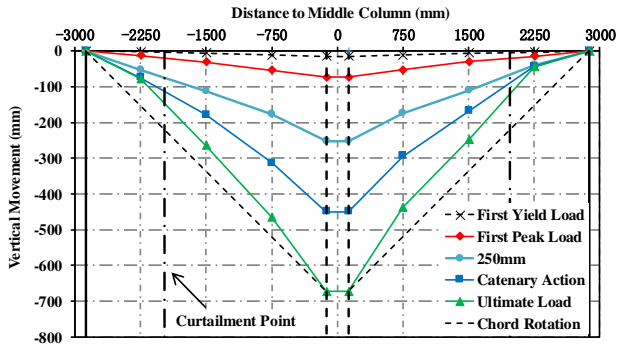
(d)



(e)

**Fig. 13.** Horizontal reaction force versus MJD: (a) RC-Con; (b) PC-TSWA; (c) PC-EP; (d) PC-TSA;

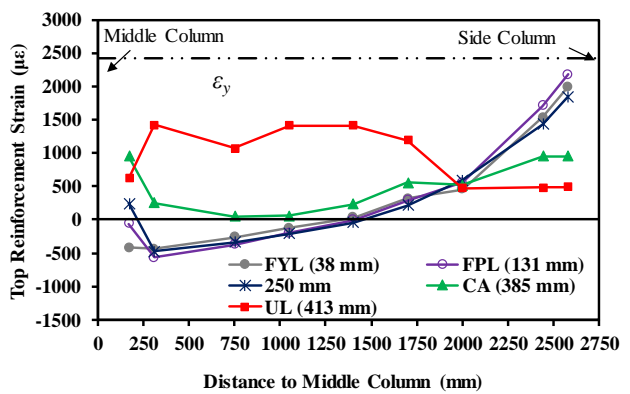
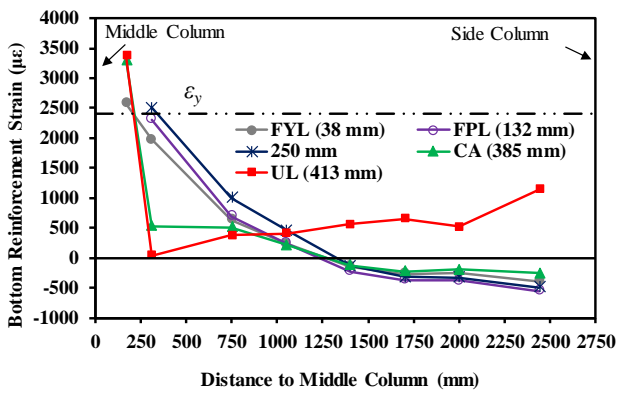
(e) PC-WC



(a)

(b)

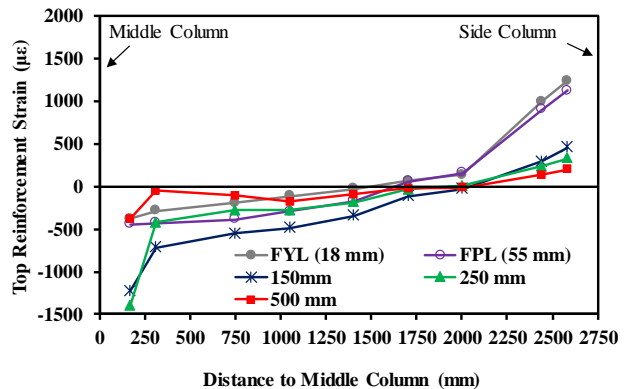
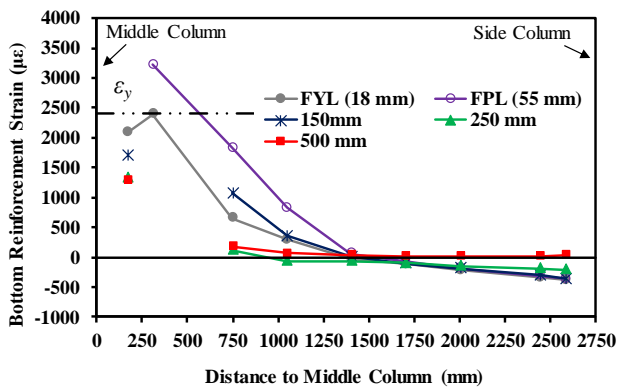
**Fig. 14.** Deformation shape of double-span beam: (a) RC-Con; (b) PC-TSWA



(a)

(b)

**Fig. 15.** Strain distribution along beam rebar of PC-TSWA: (a) bottom rebar; (b) top rebar



(a)

(b)

**Fig. 16.** Strain distribution along beam rebar of PC-EP: (a) bottom rebar; (b) top rebar

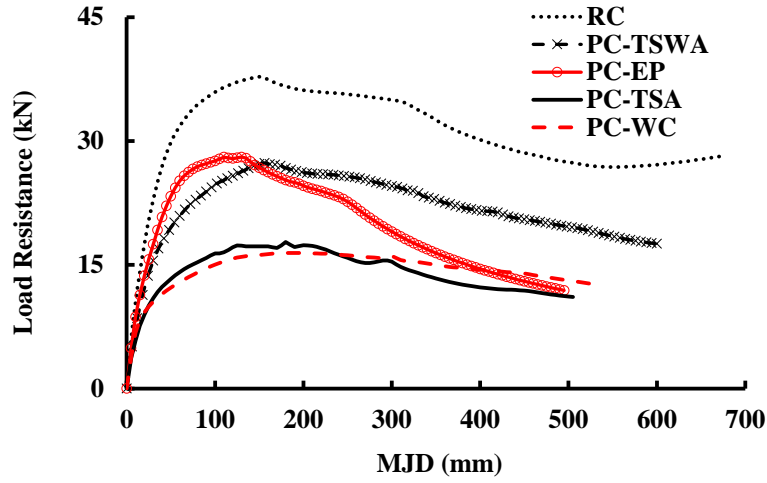


Fig. 17. Dynamic resistance of tested specimens

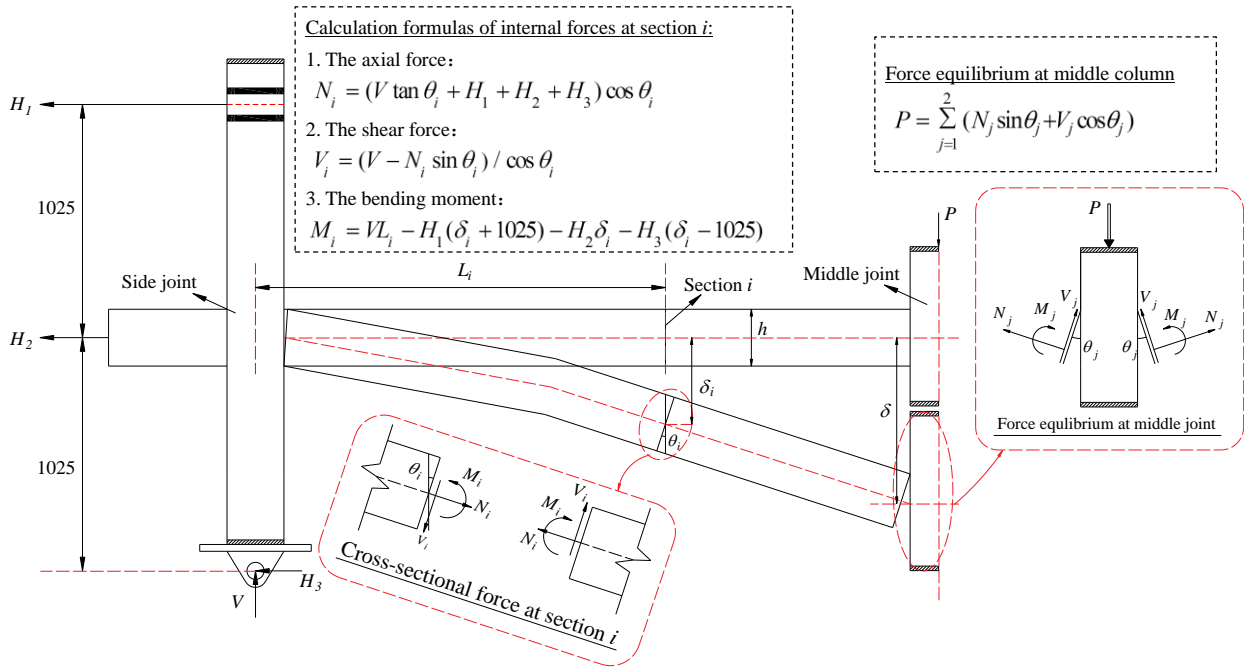


Fig. 18. Calculation of internal forces

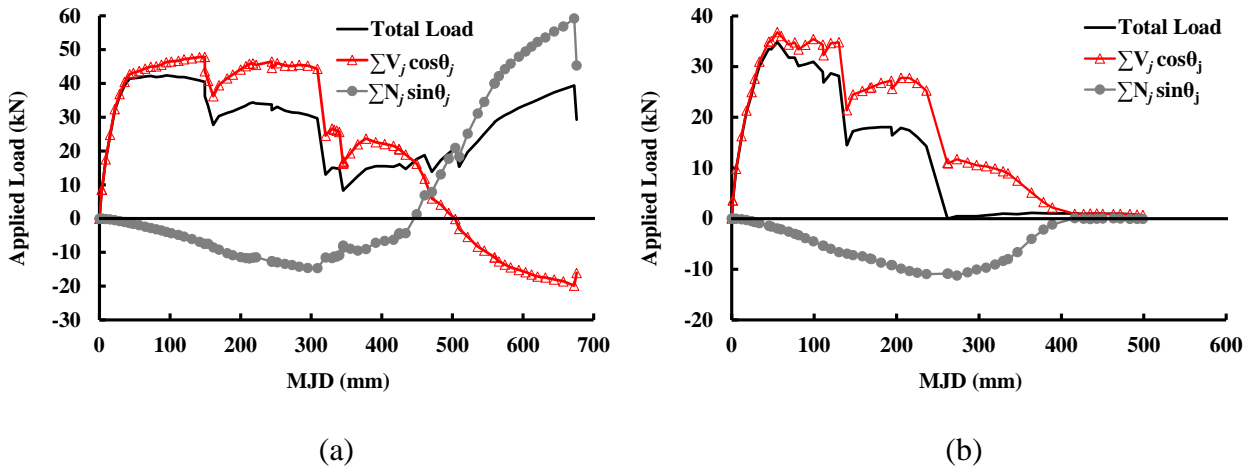


Fig. 19. De-composition of the vertical resistance: (a) RC-Con; (b) PC-EP

### **Conflict of interest**

The authors declare that there is no conflict of interest regarding the publication of this paper.

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## Author Statements

**Xiao-Fang Deng:** Supervision. **Teng-Fang Dong:** Methodology and Formal Analysis. **Feng Fu:** Resources. **Yun-Hao Weng:** Writing-Original Draft Preparation.

# Resilience of Prefabricated Concrete Frames using Hybrid Steel-Concrete Composite Connections

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## Abstract:

The progressive collapse resistance of prefabricated concrete frames with hybrid steel-concrete composite connections (HSCC) is rarely studied. To fill the gap, five (one reinforced concrete (RC) and four prefabricated concrete) 1/2 scaled beam-column sub-assemblages were designed and tested. In four prefabricated concrete specimens, different HSCC types, top and seat with web angle (TSWA), end plate (EP), top and seat angle (TSA), and web cleat (WC), were employed. Experimental results demonstrated that the failure patterns of prefabricated concrete frames with HSCC are different from that of the RC frame counterpart. The failure of the prefabricated concrete frames is governed by the shear fracture or thread stripping of the bolts in the connections while that of the RC frame is governed by the fracture of beam longitudinal rebar. Due to brittle failure of the connections in prefabricated concrete frames, both ultimate bearing capacity and deformation capacity of prefabricated concrete frames are smaller than the corresponding RC frame. Among them, the prefabricated concrete frame with the EP connection achieves the greatest first peak load or compressive arch action capacity. However, no catenary action could be developed in this specimen.

**Author Keywords:** Disproportionate collapse; Hybrid prefabricated concrete frame; Load resisting mechanism; Beam-column sub-assemblage; Experimental results

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## 23 **1. Introduction**

24 According to the ASCE/SEI-10 [1], disproportionate collapse is defined as “the spread of an initial  
25 local failure from element to element eventually leading to the collapse of an entire structure or a  
26 disproportionately large part of it.” In recent years, with the frequent occurrence of extreme events,  
27 such as terrorist attacks, fires, and explosions, the likelihood of disproportionate collapse caused by  
28 extreme loads was increasing. The disproportionate collapse of structure usually results in substantial  
29 economic and life loss. How to evaluate or enhance the ability of structures to resist disproportionate  
30 collapse has attracted increasing attention from engineers and policymakers. Several design guidelines  
31 [1-4] were issued successively. The alternative load path (ALP) method was proposed and gradually  
32 became the most popular method in design or academic investigations as it was independent of the  
33 extreme loading [5]. Based on the ALP method, many studies were conducted in the past decades [6-  
34 9]. Structures may lose columns after an accidental event, causing a significant increase in shear force  
35 and flexural bending moment of the adjacent structural components. The beams adjacent to the lost  
36 columns can't resist such a significant amount of bending moment purely relying on their flexural  
37 strength. Therefore, exploring the inherent potential mechanisms is necessary. The inherent  
38 mechanisms include catenary action (CA) and compressive arch action (CAA) of the beams, and tensile  
39 membrane action (TMA) and compressive membrane action (CMA) of the slabs.

40 With the development of modular building technologies, prefabricated concrete (PC) structures  
41 have been widely used in developed countries. Several studies [10-15] were carried out to investigate  
42 the disproportionate collapse resistance of conventional PC structures with dry or wet connections.  
43 Moreover, as steel-concrete composite joints offer an efficient method to connect precast columns and  
44 beams, various hybrid steel-concrete composite (HSCC) structures were developed. **Compared to the  
45 conventional connection of prefabricated concrete structures, the use of hybrid steel-concrete  
46 composite (HSCC) connections offers advantages of both steel and concrete materials. It is possible to**

47 reduce the size of both hybrid precast beams and columns through effective interaction between the  
48 two materials. Moreover, the HSCC connections can be connected by using high-strength bolts or  
49 welds, which required less on-site work. Thus, the HSCC structure is ideal for PC buildings [16]. In  
50 the past decades, the seismic behavior of PC frames with various types of HSCC connections was  
51 investigated. Kulkarni et al. [17] and Li et al. [18] conducted experimental and analytical investigations  
52 on HSCC connections subjected to quasi-static cyclic loading to reveal their seismic behavior. It was  
53 found that the HSCC connections exhibited sufficient ductility under seismic loading. Li et al. [19]  
54 tested a new-type of HSCC connection with end plates to investigate its seismic behavior. It was  
55 revealed that the HSCC connection performed even better than the conventional RC connection. Zhang  
56 et al. [20] developed a new type of HSCC connection using steel fiber concrete. It was found that the  
57 proposed HSCC connection had a satisfactory ability to resist seismic loads. In addition, Zhang et al.  
58 [21] designed an innovative type of HSCC connection with energy dissipated plates and I-shaped steel  
59 connectors. It revealed that the hybrid joints with the energy dissipated plates and steel fiber concrete  
60 performed much better.

61 From the above investigations, it can be seen that the HSCC connection has been proved to be an  
62 effective type of connection for PC frames to resist seismic loadings. However, the characteristics of  
63 disproportionate collapse are quite different from those of a seismic hazard: such as the loading  
64 direction, monotonic or cyclic, the main load resisting components, etc. Moreover, the resilience of PC  
65 frames with HSCC connection to resist disproportionate collapse was still unclear due to few available  
66 studies. Therefore, to fill this gap, four PC beam-column assemblies with HSCC connections and one  
67 counterpart sub-assembly using normal RC were designed and tested in this study. The difference  
68 between PC frames with HSCC connections and RC frame in terms of load resisting mechanisms were  
69 quantified and discussed.

## 70 2. Experimental program

### 71 2.1. Specimen design

72 To investigate the resilience of PC frames with HSCC connections, five 1/2 scaled beam-column  
73 assemblies were fabricated and tested. Among them, four PC beam-column s assemblies were designed  
74 with various HSCC connections and the remaining one is a cast-in-place RC sub-assembly, which  
75 was taken as a control specimen for comparison. The prototype frame of the specimen is a 5-story  
76 commercial building that is seismically designed by BS 8110-97 [22], following Kulkarni et al. [17].  
77 Both longitudinal and transverse spans of the prototype frame are 6 m. The story height is 4.1 m. It was  
78 assumed that a middle column on the ground floor was lost due to extreme loading. The sub-assembly  
79 above the lost column was extracted from the prototype frame for the test. Therefore, all specimens  
80 consist of a two-span beam, a short middle column stub, two side columns, and two overhanging beams,  
81 as shown in Fig. 1. The height of the side column and the length of the overhanging beam is determined  
82 by the position of inflection points. It was assumed that the inflection points were in the middle height  
83 of the column and 1/5 span of the beam. The cast-in-place RC specimen was named as RC-Con, and  
84 its dimensions and rebar details are illustrated in Fig. 2. As can be seen in the figure, the clear span of  
85 the beam is 2750 mm. The cross-section of the beam and column is 150 mm × 250 mm and 250 mm ×  
86 250 mm, respectively. **In this study, a relatively low reinforcement ratio is used to make the effect of**  
87 **CAA obvious.** T10 and T13 are used as longitudinal rebar while R6 is used as transverse rebar. The  
88 beam transverse rebar in the potential plastic hinge zone and the remaining zones were R6@70mm and  
89 R6@140mm, respectively, to follow the seismic design details. “T” and “R” represent deformed rebar  
90 and plain rebar, respectively. Detailed specimen properties of RC-Con were shown in Table 1.

91 For PC specimens, four different bolted connections were investigated. Based on the connection  
92 types, the specimens were named as follows: PC specimen with TSWA connection was named as PC-  
93 TSWA, PC specimen with end plate connection was named as PC-EP, PC specimen with top and seat

94 angle connection was named as PC-TSA, and PC specimen with web cleat connection was named as  
95 PC-WC. As shown in Fig. 3, except for the beam-column connection, the dimensions and rebar details  
96 of the PC specimens are identical to those of the control specimen RC-Con. The detailing of different  
97 HSCC connections is shown in Fig. 4 and tabulated in Table 2, and the detailed fabrication process was  
98 illustrated in Fig. 5. For PC-TSWA, the PC beam and column were connected by top and seat angles  
99 with double web angles. The angles were connected to the PC beams and columns using M10 size  
100 bolts. In terms of PC-EP, the PC beam was connected to the PC column by an end plate. The end plate  
101 was welded to the PC beam, and six M10 bolts were applied to fasten the end plate to the PC column.  
102 Only top and seat angles were used to connect the PC beam and column in PC-TSA, and only double  
103 web angles were applied to connect the PC beam and column in PC-WC. It should be noted that the  
104 connectors of PC beams and columns were made of I-shaped steels with the section of UB178×  
105 101×19 and UC 203×203×46, respectively. The length of I-shaped steel in the PC beam is 125 mm and  
106 that in the PC column is 710 mm. The strength grade of the I-shaped steel is S355. The longitudinal  
107 rebar of the PC beams and columns are welded to the I-shaped connectors. For PC-TSWA, PC-TSA,  
108 and PC-WC, steel angles with a section size of L60×6 mm were used. For PC-EP, the end plates were  
109 160×220×10 mm and were welded to the I-shaped connectors in PC columns.

## 110 2.2. Property of materials

111 According to the compressive and tensile splitting cylinder tests, the average compression strength  
112 and splitting tensile strength of the concrete are 34 MPa and 3.3 MPa, respectively. The critical  
113 properties of rebar and steel angles are listed in Table 3.

## 114 2.3. Test setup with instrumentation layout

115 In this study, a displacement-controlled pushdown loading regime was applied. The test setup and  
116 instrumentation layout are displayed in Fig. 6. This setup was commonly used for pushdown tests of  
117 disproportionate collapse resilience of structures [23-27]. As mentioned above, the specimens were

118 extracted from the prototype frame at contraflexure points and 1/2 scaled down. Thus, the side column  
119 was pin supported with a roller installed horizontally at the top of the side column and overhanging  
120 beam end to represent the horizontal restraints from adjacent members. An axial force of  $0.2f'_cA_g$   
121 (where  $f'_c$  is the cylinder compressive strength of concrete while  $A_g$  is the gross area) was applied  
122 on the side column. To assess the bridging capacity of the specimen under the loss of a middle column,  
123 a vertical displacement with the rate of 0.2mm/s was applied to the top of the middle column by a jack  
124 with a 700 mm stroke (Item 1 in Fig. 6). A specially designed steel assembly (Item 3 in Fig. 6) was  
125 installed for preventing the undesired out-of-plane failure.

126 A series of load cells, displacement transducers, and strain gauges were applied before testing.  
127 Load pins (Item 6 in Fig. 6) were applied in the pin supports to measure the reaction forces of pin  
128 support. Tensile/compressive both way load cell (Item 5 in Fig. 6) was applied in each horizontal roller  
129 to record its horizontal reaction force. During testing, the applied vertical load was measured by a load  
130 cell (Item 2 in Fig. 6) installed between the hydraulic jack (Item 1 in Fig. 6) and the reaction frame.  
131 Seven displacement transducers (Item 7 and 8 in Fig. 6) were applied along the beam span to record  
132 beam deflection shape. In addition, a series of strain gauges were glued on the rebar at special positions  
133 before casting, as shown in Figs. 2 and 3.

### 134 3. Test results

135 In the present study, one RC and four PC beam-column assemblies were tested by pushdown  
136 loading regime to assess the effects of different HSCC connections on the performance of PC frames  
137 to resist disproportionate collapse. The key results of the test were listed in Table 4 and discussed below.

#### 138 3.1 Global behavior

##### 139 3.1.1 Specimen RC-Con

140 Specimen RC-Con is the cast-in-place RC beam-column sub-assembly. The load-middle joint  
141 deflection (MJD) relationship and failure pattern of RC-Con are displayed in Figs. 7 and 8, respectively.

142 Initially, the first crack formed at the beam near the middle column at the MJD of 12 mm. With the  
143 MJD further increasing to 15 mm, the RC-Con reached its first yield load (FYL), which was defined  
144 as the vertical load corresponding to the first yield of the beam longitudinal rebar, of 25 kN. **The actual**  
145 **moment applied at the beam ends near the middle column was 16 kN·m, which is 64% of the maximum**  
146 **moment resisted at the section.** Meanwhile, noticeable cracks were formed near the beam ends. When  
147 the MJD reached 73 mm, the first peak load (FPL) of 42 kN was measured, which is 168 % of FYL,  
148 **indicating that the load capacity is increased by 68 % due to the mobilization of CAA. The ratio of the**  
149 **actual moment applied to the maximum moment resisted at the beam ends near the middle column**  
150 **increased to 97 % at this time. When MJD reached 120 mm, the actual moment applied at the beam**  
151 **ends near the middle column attained its maximum.** With a further increase in MJD, the existing cracks  
152 became wider, and the load resistance began to decrease due to the concrete crushing. Further  
153 increasing MJD to 154 mm, fracture was occurred in the rebar near the middle column, resulting in the  
154 load resistance dropping to 28 kN. Meanwhile, the widest cracks appeared at the location of curtailment  
155 of the beam longitudinal rebar. When MJD reached 309 mm, rebar fracture was also observed in the  
156 curtailment region of the longitudinal rebar, leading to a further decrease in the load resistance. Once  
157 MJD exceeds 448 mm, the load resistance enhanced rapidly since the development of CA. Finally, the  
158 ultimate load capacity (UL) of 39 kN was recorded at the MJD of 672 mm. As displayed in Fig. 8,  
159 rebar fracture appeared in the beam ends near the middle column and cut-off points of the longitudinal  
160 rebar. Obvious concrete crushing occurred in the beam ends. Many penetrated cracks were formed  
161 along the beams due to tensile axial force developed in the CA stage.

### 162 *3.1.2 Specimen PC-TSWA*

163 Specimen PC-TSWA is a PC beam-column sub-assembly with HSCC of TSWA connection.  
164 The vertical load-MJD relationship and failure pattern are displayed in Figs. 7 and 9. Similar to RC-  
165 Con, at the MJD of 10 mm, crack first formed at the beam ends near the middle column. When MJD

166 obtained 38 mm, the FYL was measured at 25 kN. The ratio of the actual moment applied to the  
167 maximum moment resisted at the beam ends near the middle column reached 65 %. Further growing  
168 MJD to 132 mm, the specimen reached FPL of 33 kN, which is 132 % of FYL, indicating that the load  
169 capacity is increased by 32 % because of the mobilization of CAA. At this time, the ratio of the actual  
170 moment applied to the maximum moment resisted at the beam ends near the middle column increased  
171 to 99%. Then, with the increase of MJD, the load resistance of the specimen decreased slowly due to  
172 concrete crushing occurred. Meanwhile, large openings were observed at the edge of the I-shaped  
173 connectors near the middle column. The applied moment reached its maximum at MJD of 149 mm.  
174 When MJD reached 152 mm and 166 mm, the beam rebar fractured at the edge of the I-shaped  
175 connectors near the middle column. When the MJD exceeded 229 mm, the bolts at the top angle near  
176 the side column were sheared off. Then, the bolts at the web angle began to be sheared off at the MJD  
177 of 326 mm. The CA began to develop at the MJD of 385 mm. Finally, following the shear fracture of  
178 the bolts at the bottom angle near the side column, PC-TSWA reached its UL of 19 kN, which was only  
179 58 % of its FPL. As displayed in Fig. 9, rebar fracture occurred at the edge of the I-shaped connectors  
180 near the middle column. The failure at the side connection was controlled by bolt fracture in shear.  
181 Concrete crushing was observed in the beam ends near the middle column while concrete shed occurred  
182 in the beam ends near the side column. The number of flexural cracks was much less than those in RC-  
183 Con.

### 184 3.1.3 Specimen PC-EP

185 PC-EP is a PC beam-column sub-assembly with HSCC of end plate connection. The vertical  
186 load-MJD relationship and failure pattern are shown in Figs. 7 and 10, respectively. Different from RC-  
187 Con and PC-TSWA, the first crack of PC-EP occurred at the edge of the I-shaped connectors near the  
188 middle column at the MJD of 12 mm. When MJD increased to 18 mm, the specimen reached its FYL  
189 of 21 kN. The ratio of the actual moment applied to the maximum moment resisted at the beam ends

190 near the middle column reached 48 %. Further increasing the MJD to 55 mm, the FPL of 35 kN, which  
191 is 167 % of FYL, was measured. This indicated that the mobilization of CAA increases the load  
192 capacity by 67 %. At this time, the ratio of the actual moment applied to the maximum moment resisted  
193 at the beam ends near the middle column increased to 95%. Then, with the increase of MJD, thread  
194 stripping of the bolt occurred at the connections near the side column, leading to the decrease in load  
195 resistance. When the MJD reached 130 mm, the applied moment reached its maximum. At the MJD of  
196 236 mm, the load resistance dropped to zero due to bottom rebar fracture. Finally, further increasing  
197 the MJD, as the side connections failed by thread stripping of all bolts, the beams and side columns  
198 were separated completely. Therefore, CA was not triggered and the load resistance held steady, which  
199 was only about 1 kN. As displayed in Fig. 10, rebar fracture occurred at the edge of the I-shaped  
200 connectors near the middle column. Moreover, the beam lost contact with the side column finally due  
201 to thread stripping of the bolts. Concrete crushing occurred at the edge of the I-shaped connectors near  
202 the middle column. The flexural cracks were much less than those in RC-Con and PC-TSWA. No  
203 penetrated cracks formed along the beam due to no CA mobilized during the test.

#### 204 3.1.4 Specimen PC-TSA

205 PC-TSA is a PC beam-column sub-assembly with HSCC of top and seat angle connection. The  
206 vertical load-MJD relationship and failure pattern are shown in Figs. 7 and 11, respectively. Similar to  
207 RC-Con and PC-TSWA, the first crack of PC-TSA occurred at the beam with the MJD of 8 mm. As  
208 the beam longitudinal rebar did not yield before FPL, no FYL is shown herein. The FPL of 20 kN was  
209 obtained at the MJD of 87 mm. The ratio of the actual moment applied to the maximum moment  
210 resisted at the beam ends near the middle column was 83% at this time. Then, further increasing MJD,  
211 the load resistance decreased slowly due to concrete crushing. When MJD reached 204 mm and 261  
212 mm, the angle bolts fractured in shear, leading to the load resistance dropping to 0 kN rapidly. The  
213 maximum moment of the beam ends near the middle column was measured at the MJD of 204 mm.

214 Then, with the increase of MJD, the load resistance increased again. Finally, when MJD reached 432  
215 mm, PC-TSA failed by the shear damage of the bolts and achieved its UL of 10 kN. As shown in Fig.  
216 11, all connections failed by bolt shear fracture. Concrete crushing and spalling appeared in the beam  
217 ends. Similar to PC-EP, no penetrated cracks formed in the middle zones of the beam due to mild CA  
218 developed.

### 219 *3.1.5 Specimen PC-WC*

220 PC-WC is a PC beam-column sub-assembly with HSCC of web cleat connection. The vertical  
221 load-MJD relationship and failure pattern are shown in Figs. 7 and 12. The first crack was measured at  
222 beam ends at the MJD of 7 mm. Similar to PC-TSA, there was no FYL because no beam longitudinal  
223 rebar yielded before FPL. When MJD reached 109 mm, the specimen obtained its FPL of 19 kN. **The**  
224 **applied moment at the beam ends near the middle column was 14 kN·m, which is 77% of the maximum**  
225 **moment.** Then, the load resistance started to drop slowly accompanied by the concrete crushing and  
226 spalling. After the MJD exceed 230 mm, the bolts began to fracture due to shear force. **The maximum**  
227 **moment of the beam ends near the middle column was measured at the MJD of 299 mm.** Finally, as all  
228 bolts of the connection near the middle column were destroyed completely at the MJD of 436 mm, the  
229 specimen reached its UL of 12 kN. As shown in Fig. 12, the failure of connections was caused by the  
230 shear fracture of bolts. Concrete spalling occurred at beam ends. Similar to PC-EP, PC-TSA, no  
231 penetrated cracks occurred in the middle zones of the beam since less tensile axial force developed in  
232 the beams.

### 233 *3.2 Horizontal reactions*

234 As the specimens' arrangements were all symmetrical, the average value of the horizontal reaction  
235 forces obtained at both sides of the specimens was used and discussed. The horizontal reaction force-  
236 MJD curves of the specimens are displayed in Fig. 13. The maximum horizontal reaction forces of the  
237 specimens were tabulated in Table 4. Negative means compression force while positive represents

238 tension force. As given in Fig. 13(c), no tensile horizontal force was measured in PC-EP, which  
239 confirmed the above conclusion that no CA was developed in the beams due to complete tread stripping  
240 of the bolts at MJD of 337 mm. Different from PC-EP, both tensile and compressive reaction forces  
241 were measured in other specimens. The maximum horizontal compression forces of RC-Con, PC-  
242 TSWA, PC-EP, PC-TSA, and PC-WC are -71 kN, -65 kN, -67 kN, -56 kN, and -56 kN. The maximum  
243 horizontal tension forces of RC-Con, PC-TSWA, PC-TSA, and PC-WC were 95 kN, 47 kN, 20 kN,  
244 and 24 kN. Therefore, the CAA and CA developed in PC frames with HSCC connection were weaker  
245 than those in RC-Con. Among the PC specimens, PC-EP achieved a similar compressive reaction force  
246 as that of PC-TSWA. PC-TSA and PC-WC achieved similar horizontal reaction forces.

247 As shown in Fig. 13, for all specimens, in the stage of CAA, the compressive reaction force was  
248 primarily from the bottom pin support. In the large deformation stage, the tensile reaction force of RC-  
249 Con was mainly provided by the horizontal restraint at overhanging beam. However, for PC specimens,  
250 the tensile reaction force was mostly from the top horizontal restraint and bottom pin support. **This is**  
251 **because, in the large deformation stage, the development of axial force in beams of PC specimens is**  
252 **hindered due to the shear fracture of bolts. Therefore, for PC specimens, the shear forces of the side**  
253 **columns transferred from the beams are small, resulting in a small lateral deformation of the side**  
254 **column. Thus, a relatively small force is transmitted to the horizontal restraint at the overhanging beam.**

### 255 3.3 Deformation's shape

256 The deformation's shape of the beams is monitored by LVDTs along the beam. Fig. 14 displays  
257 the deformation's shape of the beams of RC-Con and PC-TSWA at critical status. The dashed straight  
258 line represented chord rotation. As specified in DoD [4], it is calculated by the ratio of MJD to the  
259 clean beam span. As shown in Fig. 14(a), for RC-Con, the beams kept almost straight before the FPL.  
260 With the MJD increasing to 250 mm, the rotation of the beam end near the middle column was greater  
261 than that near the side column. In the last stage, the chord rotation overestimates the rotation of the

262 beam end near the side column and underestimates the rotation of the beam end near the middle column.  
263 This is because the plastic hinge formed at the curtailment of the beam longitudinal rebar. Different  
264 from RC-Con, as displayed in Fig. 14(b), the beams in PC-TSWA kept straight during the whole loading  
265 history. At the UL stage, the chord rotation could predict the rotation of the beam end near the side  
266 column accurately but overestimated the rotation of the beam end near the middle column due to plastic  
267 hinges occurring at the edge of the I-shaped connectors. For PC-TSA and PC-WC, the rotation of the  
268 beam ends was consistent with the chord rotation well. For PC-EP, the deformation of the beam was  
269 similar to that of PC-TSWA.

### 270 *3.4 Strain gauge results*

271 The location of strain gauges is shown in Figs. 2 and 3. The bottom longitudinal rebar of RC-Con,  
272 PC-TSWA, and PC-EP started to yield at the MJD of 15 mm, 38 mm, and 18 mm. However, no yielding  
273 was measured in PC-TSA during the whole test history while yielding strain was only measured at PC-  
274 WC until the stage of UL. Figs. 15 to 16 show the strain readings along with longitudinal rebar of PC-  
275 TSWA, and PC-EP, respectively. For PC-TSWA, after reaching FPL, the compression strain of the  
276 rebar near the middle column began to decrease with the increase of MJD, indicating that the CAA  
277 stage was shifted into the CA stage. However, beyond the stage of FPL, the decrease in load resistance  
278 of PC-EP was due to bolt thread stripping instead of concrete crushing. Therefore, different from other  
279 specimens, the compression strain of the rebar near the middle column of PC-EP continued to increase  
280 after the stage of FPL, which can be seen in Fig. 16. In addition, for PC-TSWA, in the large deformation  
281 stage (UL or MJD = 500 mm), most of the longitudinal rebar was in tension, which indicates that CA  
282 could have developed in these specimens. Conversely, most of the longitudinal rebar strains of PC-EP  
283 were very low in the large deformation stage, indicating that no CA was developed, which is consistent  
284 with the results of vertical loads and horizontal reactions. The development of the rebar strain of RC-  
285 con, PC-TSA, and PC-WC is similar to PC-TSWA.

## 286 **4. Analytical and discussion**

### 287 *4.1 Effects of HSCC connection types*

288 As shown in Table 4, the FPL of PC-TSWA, PC-EP, PC-TSA, and PC-WC is 79 %, 83 %, 48 %,  
289 and 45 % of that of RC-Con, respectively. Thus, the PC frames using the proposed four HSCC  
290 connections could not achieve equivalent behavior as cast-in-place RC frames regarding FPL.  
291 Moreover, the measured deformation capacity of RC-Con, PC-TSWA, PC-TSA, and PC-WC  
292 corresponding to their UL is 672 mm, 413 mm, 236 mm, 432 mm, and 436 mm, respectively. Therefore,  
293 the deformation capacity of PC-TSWA, PC-EP, PC-TSA, and PC-WC is 61 %, 35 %, 64 %, and 65 %  
294 of that of RC-Con, respectively. Furthermore, the UL of PC-TSWA, PC-EP, PC-TSA, and PC-WC is  
295 49 %, 36 %, 26 %, and 31 % of that of RC-Con, respectively. It should be noted that the UL is defined  
296 as the peak load in the re-ascending phase of the load history. Therefore, regarding the deformation  
297 capacity and UL, the PC frames with the proposed four HSCC connections still could not be equivalent  
298 to the cast-in-place frame. The lower FPL is due to the lower strength of the HSCC connection and  
299 weaker CAA. On the other hand, the lower deformation capacity is because the failure patterns of all  
300 PC specimens are controlled by shear fracture or thread stripping failure of the bolts, which are brittle  
301 failures and thus, reduce the ductility of the specimens. In consideration that the failure patterns of PC  
302 specimens may be changed if the greater size bolts or weaker steel plates are designed in HSCC  
303 connection. Therefore, more studies are urgently in need to further investigate the robustness of PC  
304 frames with HSCC connections to resist disproportionate collapse in the future.

305 For PC specimens, the FPL of PC-EP is larger than that of PC-TSWA. However, due to thread  
306 stripping failure at the MJD of 337 mm, which prevents the development of CA and thus, the  
307 deformation capacity and UL of PC-EP is much lower than that of PC-TSWA.

### 308 *4.2 Dynamic ultimate bearing capacity*

309 It should be noted that disproportionate collapse is generally a dynamic process. Therefore, it is

310 needed to assess their dynamic ultimate bearing capacity. Relied on the quasi-static pushdown load-  
311 displacement curve, a capacity curve method [28] was adopted to calculate the dynamic ultimate  
312 bearing capacity of the specimens. Previous studies [29] had been confirmed the feasibility of this  
313 method. The mathematical equation of the capacity curve method is given in Eq. 1:

$$314 \quad P_d(u_d) = \frac{1}{u_d} \int_0^{u_d} P_s(u) du \quad (1)$$

315 where  $P_d(u_d)$  and  $P_s(u)$  represent the capacity function and the nonlinear static loading estimated  
316 at the displacement demand  $u$ , respectively.

317 Fig. 17 compares the dynamic load-MJD curves of the test specimens based on the capacity curve  
318 method. As displayed in the figure, the dynamic ultimate bearing capacity of PC-TSWA, PC-EP, PC-  
319 TSA, and PC-WC is 71 %, 73 %, 47 %, and 42 % of that of RC-Con, respectively. Thus, the quasi-  
320 static pushdown curve is a good alternate method for the disproportionate collapse study. The dynamic  
321 load increase factor is defined as the ratio of static ultimate bearing capacity to dynamic load-resisting  
322 capacity [27]. Based on the test results and analytical results, the dynamic load increase factors of RC-  
323 Con, PC-TSWA, PC-EP, PC-TSA, and PC-WC were 1.11, 1.22, 1.25, 1.11, and 1.19. Among them, the  
324 dynamic load increase factor of the PC specimens is greater than that of the RC specimen due to the  
325 brittle failure that occurred there.

#### 326 *4.3 Effects of steel angles*

327 As shown in Table 4 and Fig. 7, the FPL of PC-TSWA is 165 % and 174 % of that of PC-TSA and  
328 PC-WC, respectively. Therefore, the web angles and top and seat angles increase the FPL by 65 % and  
329 74 %, respectively. For UL, PC-TSWA, PC-TSA, and PC-WC are 19 kN, 10 kN, and 12 kN. The  
330 corresponding MJD is 413 mm, 432 mm, and 436 mm. The results show that the additional web angle  
331 or top and seat angle has little effect on the deformation capacity. However, the web angle and top and  
332 seat angle could increase the UL by 90 % and 58 %, respectively. Analytical results indicated that the  
333 web angle and top and seat angle enhanced the dynamic ultimate bearing capacity by 50 % and 69 %.

#### 334 4.4 De-composition of vertical load resistance

335 According to the force equilibrium of the deformed beam in Fig. 18, the vertical load resistance  
336 at the middle column could be divided into:

$$337 \quad P = \sum_{j=1}^2 (N_j \sin \theta_j + V_j \cos \theta_j) \quad (2)$$

338 where  $P$  = the applied load;  $N_j$  and  $V_j$  = the axial force and shear force;  $\theta_j$  = the rotation of the beam  
339 section at one of the joint interfaces.

340 Based on Eq. (2), the contribution of the axial force (related to CA) and shear force (related to  
341 flexural action) could be calculated. The de-composition of load resistance is shown in Fig. 19. PC-  
342 TSWA, PC-TSA, and PC-WC have a similar response as RC-Con in terms of de-composition of load  
343 resistance. Before CA is triggered, the contribution of axial force is negative, and the vertical load was  
344 mainly provided by the flexural action. After the fracture of the rebars or bolts, the contribution of  
345 flexural action dropped significantly. In the CA stage, the contribution of CA is positive, while the  
346 contribution of flexural action kept decreasing. When the bottom rebar or bolts of the connection near  
347 the middle column fractured entirely, the contribution of flexural action transferred into negative. For  
348 PC-EP, as no CA developed, the vertical load is mainly provided by the flexural action while the  
349 contribution of CA is positive during the test.

#### 350 5. Conclusions

351 To examine the resilience of PC frames using innovative hybrid steel-concrete composite (HSCC)  
352 connections to resist disproportionate collapse, a series of four PC beam-column assemblies with  
353 various HSCC connections as well as an RC sub-assembly were tested in this study. Relied on the  
354 experimental and analytical results, the main conclusions were given:

- 355 1. The failure pattern of PC frames with HSCC connections is different from RC frames. The failure  
356 of PC frames with HSCC connection is mainly controlled by the shear failure or thread stripping  
357 of the bolts. However, the failure of the RC frame is normally controlled by the fracture of top

358 longitudinal rebar at cut-off points or bottom longitudinal rebar at beam ends near the middle  
359 column. Moreover, the beams in PC frames with HSCC connections kept straight during the test,  
360 which means rotation mainly concentrated into the HSCC connection.

361 2. RC frame has larger FPL and UL than those of PC frames with HSCC. Together with horizontal  
362 reaction force results, considerable CAA and catenary action capacity are only measured in PC-  
363 TSWA. Almost no catenary action is mobilized in the remaining PC frames with EP, TSA, or WC  
364 connections. Thus, the load resisting mechanism of PC frames with HSCC is highly dependent on  
365 the type of HSCC connection and different from that in RC frames.

366 3. Comparing the results of PC-TSWA, PC-TSA, and PC-WC, the web angles and top and seat angles  
367 enhanced the first peak load of PC frames by 65 % and 74 %, respectively. In addition, the  
368 additional web angles or top and seat angles have little effect on the deformation capacity, but  
369 could increase the UL by 90 % and 58 %.

370 4. De-composition of the vertical load resistance indicated that the load contribution in PC-TSWA,  
371 PC-TSA, PC-WC, and RC-Con is similar. In the small deformation stage, the vertical load mainly  
372 comes from flexural action. Nevertheless, the vertical load is mainly from the development of  
373 catenary action in the large deformation stage. However, for PC-EP, the vertical load is mainly  
374 from the flexural action during the test.

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