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1           **Punching Shear Behavior and Corrosion Resistance of Composite Slab-Column**  
2           **Connections using Hybrid BFRP and Steel Rebar Reinforcement**

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4   **Abstract**

5           The collapse of Champlain Towers in South, Miami in 2021 shows that flat slab structures in  
6 coastal region show high risk of building collapse due to rebar corrosion. However, to date, few  
7 studies have been carried out to avoid these types of tragic events. A new type of hybrid steel bars  
8 and basalt fiber reinforced polymer (BFRP) bars reinforcement will provide a possible solution  
9 due to their excellent erosion resistance features. Thus, in this study, an experimental study of  
10 eight large-scale composite slab-column connections is conducted to investigate the advantages  
11 of partially replacing steel bars using equal-stiffness BFRP bars to resist corrosion. Two normal  
12 reinforced concrete (RC) slab-column connections were first tested as reference specimens,  
13 together with two corroded counterparts having a target corrosion degree of 20%. Then, two  
14 composite connections with hybrid rebars were tested to investigate the advantage of replacing  
15 half of the steel bars with BFRP bars based on equal-stiffness rule. Finally, two composite  
16 connections were reinforced by hybrid rebar and the steel rebar was corroded with a target  
17 corrosion degree of 20% to investigate the effectiveness of hybrid reinforcement to compensate  
18 the decrease in punching shear resistance due to corrosion. The test results demonstrated that the  
19 RC slab-column connections with tensile reinforcement ratio of 0.52% and 0.91% reduced the  
20 punching shear resistance by 19.7% to 24.3% when the real corrosion degree reached 12.8% and  
21 18.9%, respectively. Replacement of steel rebar with BFRP bar following equal-stiffness rule  
22 resulted in slightly greater load resistance but lower ductility energy dissipation capacity. The  
23 punching shear resistance of the corroded composite slab-column connections with hybrid bars  
24 was greater than that of the corroded conventional RC slab-column connections. To incorporate

25 the rebar corrosion effects in design, the accuracy of the equations from prevalent design codes  
26 was re-evaluated, design recommendation was made.

27 **Author keywords:** Punching shear failure; Slab-column connection; Corrosion; Large-scale test;  
28 BFRP bars

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39

## 40 **Introduction**

41 Flat slab/plate system is widely used in parking garages, office, and residential buildings because  
42 the absence of down stand beams enables a clear story height and efficient space usage. The failure  
43 of flat slab/plate system may be due to punching shear failure at the slab-column connections,  
44 which is quite dangerous due to its brittle failure nature. Once a punching shear failure occurs at  
45 one of the connections, the force initially resisted by the column required to redistributed to  
46 surrounding connections and resulted in greater bending moment and shear force at these  
47 connection. Finally, the punching shear failure may also occur at these surrounding connections,  
48 which leads to a progressive collapse of the whole building (Weng et al. 2020). As flat slab/plate  
49 is normally built as basement, parking lot or servicing in coastal environment are exposed to  
50 moisture, chlorides, and dry-wet cycles. In this context, steel bar corrosion is one of the major  
51 concerns that may lead to building collapse, such as the collapse of the Champlain Towers South,  
52 Miami in 2021 and partial collapse of Pipers Row car park, Wolverhampton, UK in  
53 1997. Corrosion deteriorates steel bar and concrete, as well as the bond between steel bar and  
54 concrete, thereby impairing the stiffness and strength of structural members (Cairns et al. 2005).  
55 When the corrosion degree is greater than 2%, the decrease in bond strength, stiffness, and slip is

56 considerable, which can cause a brittle failure (Auyeung et al. 2000). Therefore, it is necessary to  
57 explore the adverse effects of rebar corrosion on the punching shear resistance of slab-column  
58 connections.

59 To date, few tests have been conducted to investigate the effects of corrosion on structural  
60 performance of slab-column connections or two-way slabs. Said and Hussein (2019a and b)  
61 studied the performance of two-way slabs with corrosion degrees of 0%, 15%, 25%, and 50% and  
62 concluded that the increasing corrosion degree resulted in a decrease in the punching shear  
63 resistance and initial stiffness although the energy-absorption capacity and ductility were  
64 increased. Qian et al. (2022a and b) investigated the slab-column connections with target corrosion  
65 degree of 10%, 20%, and 30% and demonstrated that, compared with uncorroded slab-column  
66 connections, the failure mode of the corroded slab-column connections might be changed from  
67 flexural or flexural-shear failure to punching shear failure. Moreover, the horizontal cracks caused  
68 by corrosion significantly decreased the initial stiffness and punching shear resistance of the  
69 connections. On the basis of a critical shear crack theory (Muttoni 2008), a modified model taking  
70 into account the corrosion effects was proposed by Qian et al. (2022a).

71 To mitigate the detrimental influences of corrosion, corrosion-resistant fiber reinforced  
72 polymer (FRP) bars were used to replace steel bars of slab-column connections (Hassan et al.  
73 2013a and b; Long and Marián 2013; Aljazaeri et al. 2020; Huang et al. 2020; Sim and Frosch  
74 2020). However, these studies replaced all steel bars by FRP bars completely based on equal area  
75 rule. It is predictable that, if the steel bars were completely replaced by FRP bars relied on equal  
76 area rule, the slab-column connections may perform poorly than conventional RC slab-column  
77 connections because of the lower elastic modulus of FRP bars (Hassan et al. 2013a and b; Long  
78 and Marián 2013). Therefore, to achieve similar stiffness and load resistance, the equal-stiffness  
79 replacement rule was adopted in the current study. However, because of the low elastic modulus  
80 of basalt fiber reinforced polymer (BFRP) bars, the area of BFRP bars should be several times

81 than that of steel bars to achieve similar stiffness, which significantly increases reinforcement ratio,  
82 resulting in construction congestion and additional costs. Thus, compared with the complete  
83 replacement scheme, partial replacement based on equal-stiffness rule is more practical. However,  
84 to date, little relevant study was reported. In this study, eight large-scale composite slab-column  
85 connections were tested to investigate the efficiency of proposed partial replacement rule and  
86 corrosion effects. It should be noted that glass fiber reinforced polymer (GFRP) bars were not used  
87 as their lower stiffness and strength while carbon fiber reinforced polymer (CFRP) bars were not  
88 used as their much higher cost. BFRP bars were used in this study due to their relatively high  
89 stiffness and strength, but much lower cost.

## 90 **Experimental Study**

### 91 *Specimen and Material Properties*

92 **Fig. 1** shows the geometry and reinforcing details of the specimens. The slab was 150 mm in  
93 thickness and 2,200 mm in both length and width. The cross section of the column stub was 200  
94 mm  $\times$  200 mm. Based on the slab reinforcement ratio, the test specimens were categorized into L-  
95 and H-series, as listed in

96 **Table 1.** The first letter, L or H indicates low or high reinforcement ratio of 0.52% and  
97 0.91%, respectively. Each series included four specimens, the numeral in the specimen notation  
98 represents corrosion degree. The second letter *S* denotes the specimens with equal-stiffness  
99 replacement. As illustrated in **Fig. 1**, in Specimens L-S-0, L-S-21.1, H-S-0, and H-S-22.2, half of  
100 the steel bars in the zone highlighted by the red dashed lines were replaced by BFRP bars following  
101 equal-stiffness rule. The area of BFRP bars  $A_b$  was determined by **Eq. 1** and the equivalent  
102 reinforcement ratio of the hybrid steel and BFRP bars  $\rho_b$  was calculated by **Eq. 2**. For Specimens  
103 L-12.8, L-S-21.1, H-18.9, and H-S-22.2, the steel bars within the corrosion zone (hatched zone)  
104 were corroded to the target corrosion degree of 20% by electrified accelerated corrosion. The

105 dimension of the corroded zone, 800 mm × 800 mm, was designed to ensure the critical shear  
 106 crack (if any) to be within the corroded zone. The bottom steel bars of the slab were designed as  
 107 T10@260 for all specimens. T10 indicates a deformed steel bar with a 10-mm nominal diameter.  
 108 Based on uniaxial compression tests and splitting tests, the compressive and tensile strength of  
 109 concrete cylinders is shown in

110 **Table 1.** The material properties of the steel and BFRP bars are listed in **Table 2** and shown  
 111 in **Fig. 2.**

$$112 \quad A_b = \frac{E_s}{E_b} A_s \quad (1)$$

$$113 \quad \rho_h = \rho_b \frac{E_b}{E_s} + \rho_s \quad (2)$$

#### 114 *Accelerated Corrosion*

115 **Fig. 3** shows the device for electrified accelerated corrosion. A temporary tank was fabricated  
 116 on the bottom surface of the specimens (top surface in actual constructions), which contained 5%  
 117 electrolyte NaCl solution. A stainless steel gauze immersed in the solution and the steel bars within  
 118 the target zone were connected to the cathode and anode of a direct current (DC) power source,  
 119 respectively. The impressed constant current density was 0.6 mA/cm<sup>2</sup>. Based on **Eq. 3**, the  
 120 required corroding time to achieve the target corrosion degree of 20% was 34 days.

$$121 \quad t = \frac{zFm}{MI} \quad (3)$$

122 where  $t$  is corroded time (in s);  $m$  is mass of corroded steel bars (in g);  $M$  is atomic weight of the  
 123 steel equal to 56g;  $I$  is current amperes (in A);  $z$  is ionic charge (2 for Fe→Fe<sup>2+</sup> +2e<sup>-</sup>); and  $F$  is  
 124 Faraday's constant equal to 96,500 A/s.

## 125 *Test Setup and Instrumentation*

126 **Fig. 4** shows the test setup. The specimens were simply supported along four edges. A  
127 concentrated load was applied to the top of the column stub by a hydraulic jack with a loading  
128 capacity of 2,000 kN. Displacement-control loading scheme was implemented. A load cell was  
129 installed below the hydraulic jack to measure the applied load. As shown in **Fig. 5**, six linear  
130 variable differential transformers (LVDTs) were installed to measure slab deflections. Moreover,  
131 strain gauges were installed to the steel bars of uncorroded specimens. Test data were collected at  
132 a sampling frequency of 5 Hz during the entire loading process.

## 133 *Test Results and Discussions*

### 134 *Corrosion Measurement*

135 As shown in **Fig. 6**, the corroded zone was divided into four regions according to the distance  
136 of their outer boundaries to the column center at an interval of 100 mm. After testing, four 400-  
137 mm-long steel bars, oriented in the four different directions, were extracted. In other words, a total  
138 of 16 samples were collected from each corroded specimen. Each bar was then cut into four 100  
139 mm-long pieces, each corresponding to a corroded region. The corrosion degree  $w$  for each piece  
140 was then determined in accordance with **Eq. 4**. As shown in **Fig. 6**, the measured  $w$  in the four  
141 regions of L-12.8 and H-18.9 were generally lower than the target value. This might be because  
142 the current loss owing to the unavoidable solution leakage from the bottom of the tank. However,  
143 the measured  $w$  of L-S-21.1 and H-S-22.2 was close to the target one. Finally, the average  $w$  over  
144 regions 2 and 3 was used to represent the corrosion degree of the specimens because punching  
145 shear failure was expected to occur in these two regions. As listed in

146 Table 1. the corrosion degrees of L-12.8, H-18.9, L-S-21.1, and H-S-22.2 determined in this  
147 manner were 12.8%, 18.9%, 21.1%, and 22.2%, respectively.

$$148 \quad w = \frac{W_0 - W}{W_0} \times 100 (\%) \quad (4)$$

149 where  $W_0$  and  $W$  are the masses of a steel rebar before and after the corrosion, respectively.

### 150 *Crack Pattern and Failure Mode*

151 **Fig. 7** shows the crack pattern and failure mode of the specimens viewed from slab bottom  
152 (tension face). For corroded specimens, due to corrosion expansion, initial cracks along the  
153 corroded steel bars were observed before testing. The initial cracks widened when the applied load  
154 reached about 30% of the ultimate load given in **Table 3**. For the uncorroded specimens, the first  
155 crack occurred at approximately 35% of the ultimate load. These cracks extended to the edges of  
156 the specimens with increasing load. In the final, the critical shear crack and circumferential cracks  
157 widened and resulted in the sudden drop in load resistance. The measured radius of concrete cover  
158 spalling  $R_s$ , which is defined as the average radius measured in the bottom surface of a specimen,  
159 is summarized in **Table 3** and shown in **Fig. 7**. In general, the  $R_s$  of corroded specimens was larger  
160 than that of the corresponding uncorroded specimens, which could be attributed to the initial  
161 horizontal cracks along the steel bars. After testing, the specimens reinforced with hybrid bars  
162 were vertically cut along a column face to expose the inclined shear cracks that caused the failure.  
163 As shown in **Fig. 8**, for the uncorroded Specimens L-S-0 and H-S-0, the diagonal cracks penetrated  
164 the whole slab thickness directly. Differently, for corroded specimens L-S-21.1 and H-S-22.2, the  
165 diagonal cracks penetrated the effective slab thickness and intersected with the initial horizontal  
166 cracks, and then developed along the horizontal cracks. The formation of the horizontal cracks  
167 was mainly attributed to the corrosion-induced radial expansive stress of the steel bars (Weng et  
168 al. 2023). The radius of the punching shear core, which is defined as the distance from the  
169 intersections of the inclined cracks and the tension rebars to the column center, of L-S-0, H-S-0,  
170 L-S-21.1, and H-S-22.2 observed in **Fig. 8** was about 275 mm, 500 mm, 412 mm, and 525 mm,  
171 respectively. The radius increased with increasing reinforcement ratio. In theory, the radius should  
172 be decreased when the rebars were corroded, however, vice versa was observed in **Fig. 8**. This

173 may be attributed to randomness in tests because the specimens were only cut along one column  
174 face.

175 **Fig. 9** shows the response of crack width vs applied load. In general, the crack width  
176 increased linearly with the applied loads. Initial cracks were found in the slabs of the corroded  
177 specimens after corrosion and removal of corrosion products. The initial crack width of L-12.8,  
178 L-S-21.1, H-18.9, and H-S-22.2 was 0.12 mm, 0.13 mm, 0.13 mm, and 0.10 mm, respectively.  
179 The cracks of the corroded specimens were wider than those of the uncorroded specimens under  
180 the same applied load.

### 181 *Load-Displacement Curves*

182 **Fig. 10** shows the load-displacement response of the specimens. The critical values are  
183 summarized in **Table 3**. Comparing **Fig. 10** (a) and (b) found that the specimens with a tension  
184 reinforcement ratio of 0.91% achieved much higher initial stiffness than those with a tension  
185 reinforcement ratio of 0.52%. Decreasing the tension reinforcement ratio from 0.91% to 0.52%  
186 led to a 25.5% decrease in the punching shear resistance for uncorroded RC specimens. When the  
187 rebars were corroded with corrosion degree of 12.8% and 18.9%, the punching shear resistance of  
188 L-0 and H-0 decreased by 24.3% and 19.7%, respectively. This is because corrosion decreased the  
189 area of steel bars and degraded the bond between concrete and steel bars. The punching shear  
190 resistance of L-S-0 and H-S-0 was 5.0% and 2.7% greater than that of L-0 and H-0, respectively.  
191 Therefore, adopting the equal-stiffness replacement rule slightly increased the punching shear  
192 resistance of the specimens completely reinforced with steel bars. The initial stiffness of L-S-0  
193 and H-S-0 was similar to that of L-0 and H-0, respectively. However, with the increasing  
194 displacement, the load resistance of L-S-0 and H-S-0 was greater than that of L-0 and H-0,  
195 respectively, under the same displacement. This is because, before yielding of steel bar, the  
196 stresses of steel and BFRP bars were similar under the same displacement due to equal-stiffness.  
197 However, the stiffness of steel bars significantly decreased once yielding occurred while the

198 stiffness of BFRP did not change.. In this case, the increase of stress of steel bars is much slower  
199 than that of BFRP bars. Therefore, the increased load resistance is mainly ascribed to the greater  
200 tensile stress of BFRP bars than that of yielded steel bars. However, because the actual corrosion  
201 degree was much lower than 20%, the load resistance of L-12.8 was greater than that of L-S-21.1  
202 until the displacement reached 14.1 mm. The punching shear resistance of L-S-21.1 and H-S-22.2  
203 was only 10.0% and 2.4% lower than that of L-0 and H-0, respectively, and was 18.9% and 21.5%  
204 higher than that of L-12.8 and H-18.9, respectively. It should be noted that the actual corrosion  
205 degrees of L-S-21.1 and H-S-22.2 was higher than that of L-12.8 and H-18.9. Therefore, the  
206 effectiveness of the equal-stiffness replacement to resist corrosion should be better under the same  
207 corrosion degree, indicating that the hybrid reinforcement under equal-stiffness replacement  
208 scheme could effectively moderate the negative effects of rebar corrosion on the punching shear  
209 resistance of slab-column connections due to the corrosion-avoidance capacity of the BFRP bars.  
210 As shown in **Fig. 10**, steel bar corrosion decreased the deformation capacity of L-0 but increased  
211 the deformation capacity of H-0, and so does the energy dissipation capacity which is defined as  
212 the area under the load-displacement curve from beginning to failure, as shown in **Table 3**. The  
213 hybrid reinforcement with equal-stiffness replacement scheme resulted in both lower deformation  
214 capacity and energy dissipation capacity. Moreover, although L-S-21.1 and H-S-22.2 had similar  
215 punching shear resistance as L-0 and H-0, the deformation capacity and energy dissipation  
216 capacity of L-S-21.1 and H-S-22.2 were much lower than that of L-0 and H-0, respectively.

### 217 *Deflection of Specimens*

218 **Fig. 11** shows the deflection shape of Specimens L-0 and L-12.8. At the initial loading stage,  
219 the deflection of the slab almost linearly decreased with increasing distance to the column center.  
220 However, at the ultimate stage, the deflection of the slab center was increased sharply and  
221 nonlinear compared with surrounding measuring positions, demonstrating that punching shear  
222 failure occurred. Similar results were measured for other specimens.

223 **Analytical Study**

224 **Identification of Failure Mode**

225 Because the strain of the corroded steel bars was difficult to measure, it was necessary to  
 226 identify the failure mode of the specimens analytically. Therefore, the yield line method was used  
 227 to determine the nominal flexural strength of the specimens ( $F_{pre}$ ), and then the flexural strength  
 228  $F_{pre}$  was compared with the measured punching shear resistance to judge the failure mode of  
 229 specimens. As illustrated in **Fig. 12**, only positive yield lines were assumed because the specimens  
 230 were simply supported. The assumed positive yield lines consist of the yield lines developed along  
 231 the column edges and the radial yield lines within the polar axis. To simplify the calculation, the  
 232 rectangular positive yield lines along the column edges were converted to circular positive yield  
 233 lines with the same perimeter. In this context, the rotation of the circular fan was consistent.  
 234 Moreover, the rectangular hybrid bar zone and corroded zone in the slab center were converted to  
 235 circular zones with identical areas. Based on the virtual work principle, **Eqs. 5a to 5d** were  
 236 obtained for L-0&H-0, L-12.8&H-18.9, L-S-0&H-S-0, and L-S-21.1&H-S-22.2, respectively, for  
 237 a given displacement  $\delta$

$$F_{pre}\delta = \begin{cases} (2\pi r_1 m_u + 2\pi(R-r_1)m_u) \frac{\delta}{R-r_1}, & (5a) \\ (2\pi r_1 m_{u,c} + 2\pi(r_2-r_1)m_{u,c} + 2\pi(R-r_2)m_u) \frac{\delta}{R-r_1}, & (5b) \\ (2\pi r_1 m_{u,h} + 2\pi(r_2-r_1)m_{u,h} + 4\alpha(R-r_2)m_u + (2\pi-4\alpha)(R-r_2)m_{u,h}) \frac{\delta}{R-r_1}, & (5c) \\ (2\pi r_1 m_{u,h,c} + 2\pi(r_2-r_1)m_{u,h,c} + 4\alpha(R-r_2)m_u + (2\pi-4\alpha)(R-r_2)m_{u,h}) \frac{\delta}{R-r_1}, & (5d) \end{cases}$$

239 where  $F_{pre}$  is the virtual load;  $r_1$  and  $r_2$  are the radii of the converted circle, as illustrated in **Fig.**  
 240 **12**;  $m_u$ ,  $m_{u,c}$ ,  $m_{u,h}$ , and  $m_{u,h,c}$  are the nominal flexure strength of slab with uncorroded steel bars,  
 241 slab with corroded steel bars, slab with uncorroded hybrid bars, and slab with corroded hybrid  
 242 bars, respectively;  $\alpha$  is the angle of the four corners of L-S- and H-S-series specimens with steel  
 243 bars only, as the parts highlighted by yellow in **Fig. 12b**, which was assumed to be  $30^\circ$  (0.52 rad).

244 Based on the works of Park and Paulay (2000) and Wight and MacGregor (2011), the nominal  
 245 flexure strength per unit width of the slab can be calculated by **Eq. 6**

$$\begin{cases}
 m_u = \rho f_y d^2 (1 - 0.59 \rho f_y / f_c) \\
 m_{u,c} = \rho_c f_{y,c} d^2 (1 - 0.59 \rho_c f_{y,c} / f_c) \\
 m_{u,h} = \rho_h f_{yh} d^2 (1 - 0.59 \rho_h f_{yh} / f_c) \\
 m_{u,h,c} = \rho_{h,c} f_{yh,c} d^2 (1 - 0.59 \rho_{h,c} f_{yh,c} / f_c)
 \end{cases} \quad (6)$$

247 where  $\rho$ ,  $\rho_c$ ,  $\rho_h$ , and  $\rho_{e,c}$  is the tension steel bar ratio, corroded tension steel bar ratio, equivalent  
 248 tension bar ratio, and equivalent corroded tension bar ratio, respectively;  $f_y$ ,  $f_{y,c}$ ,  $f_{yh}$ , and  $f_{yh,c}$  is the  
 249 yield strength of uncorroded steel bar, corroded steel bar, equivalent yield strength of the hybrid  
 250 bar, and equivalent yield strength of the corroded hybrid bar, respectively,  $f_{y,c}$  is calculated based  
 251 on the suggestion of Weng et al. (2023);  $d$  is the effective section depth;  $f_c$  is the cylinder  
 252 compressive strength of concrete.

$$\begin{cases}
 \rho_c = \rho (1 - w/100) \\
 \rho_h = \rho_b E_b / E_s + \rho \\
 \rho_{h,c} = \rho_b E_b / E_s + \rho_c
 \end{cases} \quad (7)$$

$$\begin{cases}
 f_{y,c} = f_y (1 - 1.24 (w/100)) \\
 f_{yh} = (f_{yb} \rho_b E_b / E_s + f_y \rho) / \rho_h \\
 f_{yh,c} = (f_{yb} \rho_b E_b / E_s + f_{y,c} \rho_c) / \rho_{h,c}
 \end{cases} \quad (8)$$

255 where  $E_s$  and  $E_b$  indicate the elastic modulus of steel bars and BFRP bars, respectively.

256 **Table 3** lists the calculated  $F_{pre}$ . It was found that the ratio of the measured punching shear  
 257 resistance to the predicted flexural resistance ( $V_u/F_{pre}$ ) of L-0, L-12.8, L-S-0, L-S-21.1, H-0, H-  
 258 18.9, H-S-0, and H-S-22.2 was 1.04, 0.90, 0.71, 0.65, 0.83, 0.80, 0.58, and 0.58, respectively. The  
 259  $V_u/F_p$  of L-0 was greater than 1.0, indicating that this specimen reached its nominal flexural  
 260 strength before punching shear. In other words, the failure mode of L-0 was flexure-punching  
 261 shear failure while the rest of specimens failed by pure punching shear failure. This can be proved  
 262 by the strain gauge results, as shown in **Fig. 13**. The strains of steel bars in the slab center were

263 much greater than that of steel bars in the slab edge, and yielding of steel bars occurred in all  
 264 specimens except H-S-0. As shown in **Fig. 13a**, for L-0, the strains of steel bar at monitor points  
 265 SX2 to SX3 yielded, and the strain of steel bar at SX4 reached 1564  $\mu\epsilon$ . In comparison, as shown  
 266 in **Fig. 13(b to d)**, the strain of steel bar at SX3 of H-0, L-S-0, and H-S-0 did not yield and the  
 267 strain of steel bar at SX4 was quite small.

### 268 *Comparison of Test Results with the Predictions of Code Equations*

269 The measured punching shear resistance of the specimens was compared with the design  
 270 formula from Chinese code, American code, model code, and European code to evaluate the  
 271 accuracy of these codes to predict the punching shear resistance of slab-column connections with  
 272 corroded steel bars, hybrid bars, and corroded hybrid bars. Notable that the Chinese code,  
 273 American code, and model code define the critical sections as at a distance  $d/2$  from the column  
 274 edges while the European code defines the critical section as the section with  $2d$  away from the  
 275 column edges.

### 276 *Chinese Code*

277 According to the Chinese code, GB 50010 (2015), the punching shear resistance  $V_{GB}$  of the  
 278 critical section can be calculated with **Eq. 9**.

$$279 \quad V_{GB} \leq 0.7\beta_h f_t \eta u_m d \quad (9)$$

$$280 \quad \eta = \min \left\{ 0.4 + \frac{1.2}{\beta_s}, 0.5 + \frac{\alpha_s d}{4u_m} \right\} \quad (10)$$

281 where  $\beta_h$  is a factor associated with slab thickness, 1.0 for slab thickness  $h \leq 800$  mm and 0.9 for  
 282  $h \geq 2,000$  mm;  $f_t$  is the axial tensile strength of concrete;  $u_m$  is the critical shear perimeter;  $h_0$  is the  
 283 effective slab depth;  $\beta_s$  is the aspect ratio the column section ( $\geq 1$ ); and  $\alpha_s$  is a factor associated  
 284 with column position, take 40, 30, and 20 for interior, edge, and corner column, respectively.

285 *American Code*

286 Based on the American Code, ACI 318-19 (2019), the punching shear stress  $v_{ACI}$  can be  
 287 calculated by **Eq. 11**.

$$288 \quad v_{ACI} = \min \left\{ 0.17 \left( 1 + \frac{2}{\beta_s} \right) \lambda_s, 0.083 \left( 2 + \frac{\alpha_s d}{u_m} \right), 0.33 \lambda_s \right\} \lambda \sqrt{f_c} \quad (11)$$

289  $\lambda_s$  is the size effect factor,  $\lambda_s = \sqrt{2/(1+0.004 \cdot d)} \leq 1.0$ ;  $\lambda$  is the concrete density factor, take 1.0 for  
 290 normal concrete.

291 *Model Code*

292 The Model Code 2010 (fib 2012) suggests calculating the punching shear resistance  $V_{fib}$  by  
 293 **Eq. 12**.

$$294 \quad V_{Rd,c} = k_\psi \sqrt{f_c} b_0 d \quad (12)$$

$$295 \quad k_\psi = \frac{1}{1.5 + 0.9 k_{dg} \psi d} \leq 0.6 \quad (13)$$

296 where  $k_{dg}$  is the aggregate size influence parameter,  $k_{dg} = 32/(16 + d_g) \geq 0.75$  for the maximum  
 297 aggregate size  $d_g < 16$  mm and  $k_{dg} = 1.0$  for  $d_g \geq 16$  mm; the slab rotation  $\psi$  is calculated as:

$$298 \quad \psi = 1.5 \cdot \frac{r_s}{d} \cdot \frac{f_y}{E_s} \cdot \left( \frac{m_{Ed}}{m_{Rd}} \right)^{1.5} \quad (14)$$

299 where  $r_s$  is the location where the radial bending moment equals zero with respect to the support  
 300 axis;  $m_{Ed}$  is the average moment per unit length in the support strip;  $m_{Rd}$  is the average flexural  
 301 strength per unit length in the support strip. For internal column,  $m_{Ed}$  can be calculated as:

$$302 \quad m_{Ed} = V_{Ed} \left( \frac{1}{8} + \frac{|e_{u,i}|}{2 \cdot b_s} \right) \quad (15)$$

303 where  $e_{u,i}$  is the eccentricity of the resultant of shear forces with respect to the centroid of the basic  
 304 control perimeter;  $V_{Ed}$  is the shear force at punching;  $b_s$  is the width of the support strip,  
 305  $b_s = 1.5 \cdot \sqrt{r_{s,x} \cdot r_{s,y}}$ ;  $m_{Rd}$  is calculated as

$$306 \quad m_{Rd} = \rho \cdot f_y \cdot d^2 \cdot \left(1 - \frac{\rho \cdot f_y}{2 \cdot f_c}\right) \quad (16)$$

### 307 *European Code*

308 As suggested by the European code, Eurocode 2 (CEN 2004), the punching shear stress  $v_{CNE}$   
 309 of can be calculated by **Eq. 17**.

$$310 \quad v_{CEN} = 0.18k(100\rho_s f_c)^{1/3} \geq v_{min} = 0.035k^{3/2}\sqrt{f_c} \quad (17)$$

311 where  $k$  is the factor considering size effect,  $k = 1 + \sqrt{200/d} \leq 2.0$ ;  $\rho_s$  is the tension reinforcement  
 312 ratio,  $\rho = \sqrt{\rho_x \rho_y} \leq 0.02$ .

### 313 *Comparison of Calculated Load Resistance with Test Results*

314 The predicted punching shear resistance according to the above code equations is  
 315 summarized in **Table 4** and **Fig. 14**. The average ratio of the measured value to the predicted value  
 316 was 0.99, 0.97, 1.34, and 1.06 for GB 50010 (2015), ACI 318 (ACI 2019), Eurocode 2 (CEN  
 317 2004), and Model code (fib 2012), respectively. Therefore, GB 50010 (2015), ACI 318 (ACI 2019),  
 318 and Model code (fib 2012) could well predict the punching shear resistance of the specimens while  
 319 the predictions of Eurocode 2 (CEN 2004) were conservative. It is worthwhile noting that the  
 320 standard deviation (SD) and coefficient of variation (CV) of GB 50010 (2015) and ACI 318 (ACI  
 321 2019) were large, i.e., 0.23 for both. This was ascribed to the exclusion of the contribution of  
 322 reinforcement and therefore, the effects of variation of reinforcement could not be considered. If  
 323 only focus on specimens with corroded steel bars, the average ratio of the measured value to the  
 324 predicted value was 0.82, 0.79, 1.17, and 0.84 for GB 50010 (2015), ACI 318 (ACI 2019),

325 Eurocode 2 (CEN 2004), and Model code (fib 2012), respectively. In addition, if only focus on  
 326 specimens with hybrid corroded steel bars and BFRP bars, the average ratio of the measured value  
 327 to the predicted value was 0.98, 0.97, 1.32, and 1.15 for GB 50010 (2015), ACI 318 (ACI 2019),  
 328 Eurocode 2 (CEN 2004), and Model code (fib 2012), respectively. Therefore, only Eurocode 2  
 329 (CEN 2004) produced safe predictions for specimens with corrosion although it is conservative.  
 330 As shown in **Fig. 14**, generally, the calculated results were located in the domain with a maximum  
 331 error of 35%. All the predicted results of Eurocode 2 (CEN 2004) were lower than the measured  
 332 ones regardless of corrosion of steel bars is considered or not.

### 333 ***Punching Shear Resistance - Critical Shear Crack Theory***

334 The critical shear crack theory (CSCT) proposed by Muttoni (2008) was also adopted here to  
 335 calculate the punching shear resistance of uncorroded slab-column connections, and the modified  
 336 CSCT modified by Qian et al. (2022a) was used to predict the punching shear resistance of  
 337 corroded slab-column connections. In the CSCT, both the load-rotation relationship and the failure  
 338 criteria are required to judge the ultimate stage of the slab-column connections, the punching shear  
 339 resistance  $V_R$  is obtained at the intersection of the two curves, as shown in **Fig. 15**.

### 340 ***Load-Rotation Relationship***

341 As suggested by Muttoni (2008), the load-rotation relationship of slab-column connections  
 342 without shear reinforcement can be defined by **Eq. 18**

$$343 \quad V = \frac{2\pi}{r_q - r_c} \cdot \left( \begin{array}{l} -m_r \cdot r_0 + m_R \cdot \langle r_y - r_0 \rangle + EI_1 \cdot \psi \cdot \langle \ln(r_1) - \ln(r_y) \rangle + \\ EI_1 \cdot \chi_{TS} \cdot \langle r_1 - r_y \rangle + m_{cr} \cdot \langle r_{cr} - r_1 \rangle + EI_0 \cdot \psi \cdot \langle \ln(r_s) - \ln(r_{cr}) \rangle \end{array} \right) \quad (18)$$

344 The meaning of the parameters in **Eq. 18** and the corresponding calculated method can be  
 345 found in Muttoni (2008), which is not presented herein for the sake of brief.

## 346 *Failure Criteria*

347 The semi-empirical failure criterion proposed by Muttoni (2008) is shown in **Eq. 19**, which  
348 was modified by Qian et al. (2022a) to account for the effects of corrosion by introducing a factor  
349  $k$  associated with corrosion degree  $w$ , as shown in **Eq. 20**.

$$350 \quad \frac{V_R}{b_0 d \sqrt{f_c}} = \frac{3/4}{1 + 15 \frac{k \psi d}{d_{g0} + d_g}} \quad (19)$$

$$351 \quad k = \begin{cases} 1, & \text{for } w=0 \\ e^{0.016w}, & \text{for } 0 \leq w \leq 30\% \end{cases} \quad (20)$$

352 where  $V_R$  is the punching shear resistance;  $b_0$  is the control shear perimeter with  $d/2$  from the  
353 column edge;  $d_{g0}$  is a referential size equal to 16 mm; and  $d_g$  is the maximum aggregate size;  $\psi$  is  
354 rotation of slab.

355 **Fig. 16** compares the predicted punching shear resistance of the CSCT with measured ones,  
356 the punching shear resistance of the uncorroded and corroded slab-column connections was well  
357 predicted by the CSCT (Muttoni 2008) and modified CSCT (Qian et al. 2022a), respectively. The  
358 maximum error was less than 10%. Therefore, the CSCT (Muttoni 2008) and modified CSCT  
359 (Qian et al. 2022a) were reliable methods to predict the punching shear resistance of uncorroded  
360 and corroded slab-column connections, respectively, regardless of hybrid bars were considered or  
361 not.

## 362 **Conclusion**

363 An experimental study of eight large-scale slab-column connections is conducted to  
364 investigate the efficiency of partially replacing steel bars by BFRP bars based on equal-stiffness  
365 rule to resist corrosion. Moreover, an analytical study is performed to quantitatively identify the  
366 failure mode of the specimens and to evaluate the accuracy of the equations of prevalent design  
367 codes and the critical shear crack theory to predict the punching shear resistance of slab-column

368 connections. Based on the experimental and analytical studies results, the main conclusions are  
369 drawn below:

- 370 1. Test results demonstrated that the punching shear resistance of the RC slab-column  
371 connections with tension reinforcement ratios of 0.52% and 0.91% decreased by 19.7%  
372 to 24.3% under the corrosion degrees of 13.0% and 17.6%, respectively. Following equal-  
373 stiffness replacement rule, replacing partial of steel rebar by BFRP bars may increase the  
374 load resistance but decrease the deformation capacity and energy dissipation capacity  
375 slightly. The corroded composite slab-column connections reinforced by hybrid bars  
376 achieved greater load resistance but lower ductility than the corroded RC slab-column  
377 connections.
- 378 2. Steel bar corrosion decreased the deformation capacity of the slab-column connections  
379 with low reinforcement ratio but increased the deformation capacity of the slab-column  
380 connection with high reinforcement ratio. The equal-stiffness replacement rule resulted  
381 in lower deformation capacity and energy dissipation capacity. In addition, whilst  
382 corroded composite slab-column connections reinforced hybrid bars had similar  
383 punching shear resistance as uncorroded RC slab-column connections, both the  
384 deformation capacity and energy dissipation capacity of corroded composite slab-column  
385 connections reinforced hybrid bars were lower than that of uncorroded RC slab-column  
386 connections.
- 387 3. Analytical work indicated that GB 50010 (2015), ACI 318 (ACI 2019), and Model code  
388 (fib 2012) may overestimate the punching shear resistance of corroded specimens.  
389 However, Eurocode 2 (CEN 2004) could obtain conservative results regardless of  
390 whether corrosion of steel bars was considered. The CSCT (Muttoni 2008) was reliable  
391 to predict the punching shear resistance of uncorroded slab-column connections  
392 regardless of they were reinforced hybrid bars or not. The modified CSCT (Qian et al.

393 2022a) was reliable to predict the punching shear resistance of corroded slab-column  
394 connections regardless of they were reinforced hybrid bars or not.

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### 400 **Data Availability**

401 Some or all data, models, or code that support the findings of this study are available from  
402 the corresponding author upon reasonable request.

### 403 **Reference**

- 404 American Concrete Institute. 2019. *Building code requirements for structural concrete and*  
405 *commentary*. ACI 318-19. Farmington Hills, MI: ACI.
- 406 Aljazaeri, Z., Alghazali, H. H., and Myers, J. J. 2020. "Effectiveness of using carbon fiber grid  
407 systems in reinforced two-way concrete slab system." *ACI Structural Journal*, 117(2): 81-89.  
408 <https://doi.org/10.14359/51720198>.
- 409 Auyeung, Y., Balaguru, P., and Chung, L. 2000. "Bond behavior of corroded reinforcement bars."  
410 *ACI Structural Journal*, 97(2): 214-220. <https://doi.org/10.14359/826>.
- 411 Cairns, J., Plizzari, G. A., Du, Y., Law, D. W., and Franzoni, C. 2005. "Mechanical properties of  
412 corrosion-damaged reinforcement." *ACI Materials Journal*, 102(4): 256-264.  
413 <https://doi.org/10.14359/14619>.
- 414 CEN (European Committee for Standardization). 2004. *Design of concrete structures—Part 1-1:*  
415 *General rules and rules for buildings*. Eurocode 2. Brussels, Belgium: CEN.

416 fib (Fédération International du Béton). 2012. *Model Code 2010*. Bulletins d'Informations 65 and  
417 66. Lusanne, Lusanne, Switzerland: fib.

418 Hassan, M., Ahmed, E., and Benmokrane, B. 2013a. "Punching-shear strength of normal and  
419 high-strength two-way concrete slabs reinforced with GFRP bars." *Journal of Composites for*  
420 *Construction*, 17(6): 04013003. [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000424](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000424).

421 Hassan, M., Ahmed, E. A., and Benmokrane, B. 2013b. "Punching shear strength of glass fiber-  
422 reinforced polymer reinforced concrete flat slabs." *Canadian Journal of Civil Engineering*,  
423 40(10): 951-960. <https://doi.org/10.1139/cjce-2012-0177>.

424 Huang, Z., Zhao, Y., Zhang, J., and Wu, Y. 2020. "Punching shear behaviour of concrete slabs  
425 reinforced with CFRP grids." *Structures*, 26: 617-625.  
426 <https://doi.org/10.1016/j.istruc.2020.04.047>.

427 Long, N. M., and Marián, R. 2013. "Punching shear resistance of interior GFRP reinforced slab-  
428 column connections." *Journal of Composites for Construction*, 17(1): 2-13.  
429 [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000324](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000324).

430 Ministry of Housing and Urban-Rural Development of the People's Republic of China. 2015. *Code*  
431 *for design of concrete structures*. GB 50010-15. Beijing: China, Architecture & Building Press.

432 Muttoni, A. 2008. "Punching shear strength of reinforced concrete slabs without transverse  
433 reinforcement." *ACI Structural Journal*, 105(4): 440-450. <https://doi.org/10.14359/19858>.

434 Park, R., and Paulay, T. (2000). "CH4-Strength of Members with Flexure." Reinforced concrete  
435 structures, John Wiley & Sons.

436 Qian, K., Li, J. S., Huang, T., Weng, Y. H., and Deng, X. F. 2022a. "Punching shear strength of  
437 corroded reinforced concrete slab-column connections." *Journal of Building Engineering*, 45:  
438 103489. <https://doi.org/10.1016/j.jobbe.2021.103489>.

439 Qian, K., Liu, J. G., Yu, X. H., and Weng, Y. H. 2022b. "Experimental and numerical  
440 investigation of punching shear capacity of corroded reinforced concrete slab-column  
441 connections." *Structures*, 43: 1548-1557. <https://doi.org/10.1016/j.istruc.2022.07.065>.

442 Said, M. E., and Hussein, A. A. 2019a. "Effect of bandwidth reinforcement corrosion on the  
443 response of two way slabs." *Construction and Building Materials*, 216: 137-148.  
444 <https://doi.org/10.1016/j.conbuildmat.2019.04.034>.

445 Said, M. E., and Hussein, A. A. 2019b. "Structural behavior of two-way slabs with large corroded  
446 area." *Engineering Structures*, 199: 109556. <https://doi.org/10.1016/j.engstruct.2019.109556>.

447 Sim, C., and Frosch, R. J. 2020. "Cracking behavior of slabs with corrosion-resistant and high-  
448 strength reinforcing bars." *ACI Structural Journal*, 117(5): 245-257.  
449 <https://doi.org/10.14359/51724684>.

450 Weng, Y. H., Fu, F., and Qian, K. 2023. "Punching shear resistance of corroded slab-column  
451 connections subjected to eccentric load." *Journal of Structural Engineering*, 149(1): 04022219.  
452 [https://doi.org/10.1061/\(ASCE\)st.1943-541x.0003504](https://doi.org/10.1061/(ASCE)st.1943-541x.0003504).

453 Weng, Y. H., Qian, K., Fu, F., and Fang, Q. 2020. "Numerical investigation on load redistribution  
454 capacity of flat slab substructures to resist progressive collapse." *Journal of Building  
455 Engineering*, 29: 101109. <https://doi.org/10.1016/j.jobe.2019.101109>.

456 Wight, J. K., and MacGregor, J. G. 2011. *Reinforced concrete mechanics and design*, NJ: Prentice-  
457 Hall.

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**Table 1. Specimen details**

Specimen	Size (m)	Concrete strength		Slab tension reinforcement			Corrosion degree		
		$f_c$ (MPa)	$f_t$ (MPa)	Reinforced scheme	$\rho_s$ (%)	$\rho_b$ (%)	$P_h$ (%)	Target (%)	Measured (%)
L-0	2.2×2.2×0.15	43.7	3.0	Steel bar	0.52	\	0.52	\	/
L-12.8		45.6	3.1	Steel bar	0.52	\	0.52	20	12.8
L-S-0		42.0	3.1	Hybrid bar	0.26	1.04	0.52	\	/
L-S-21.1		40.5	3.0	Hybrid bar	0.26	1.04	0.52	20	21.1
H-0		36.3	2.7	Steel bar	0.91	\	0.91	\	/
H-18.9		41.1	2.9	Steel bar	0.91	\	0.91	20	18.9
H-S-0		41.9	3.0	Hybrid bar	0.46	1.84	0.91	\	/
H-S-22.2		41.5	3.0	Hybrid bar	0.46	1.84	0.91	20	22.2

Note:  $f_c$  and  $f_t$  denote cylinder compressive strength and tensile strength of concrete, respectively;  $\rho_s$ ,  $\rho_b$ , and  $\rho_h$  indicate steel bar ratio, BFRP bar ratio, and hybrid bar ratio, respectively. The  $\rho_h$  is obtained by converting BFRP bar to steel bar of the same stiffness.

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**Table 2** Material test results

Item	Diameter (mm)	Elastic modulus (MPa)	Yield strength (MPa)	Tensile strength (MPa)	Elongation (%)
Steel bar	10	216,000	567	717	15.0
	12	201,000	532	695	22.1
BFRP bar	12	47,000		1257	2.7
	17	48,000		1337	2.9

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**Table 3** Test results

Specimens	Critical loads (kN)			$V_u / F_{pre}$	$R_s$	Dissipated energy (kN·mm)	Failure mode
	$V_{cr}$	$V_u$	$F_{pre}$				
L-0	101	280	270	1.04	3.3 <i>d</i>	6,670	FP
L-12.8	/	212	238	0.90	3.7 <i>d</i>	3,889	P
L-S-0	107	294	412	0.71	4.4 <i>d</i>	3,544	P
L-S-21.1	/	252	387	0.65	7.3 <i>d</i>	5,278	P
H-0	128	376	452	0.83	2.6 <i>d</i>	3,102	P
H-18.9	/	302	380	0.80	3.4 <i>d</i>	5,096	P
H-S-0	135	386	670	0.58	4.3 <i>d</i>	2,614	P
H-S-22.2	/	367	631	0.58	7.3 <i>d</i>	3,043	P

Note:  $V_{cr}$  and  $V_u$  denote cracking strength and punching shear strength, respectively;  $F_{pre}$  indicates predicted nominal flexural strength; FP and P indicate flexure-punching failure and punching shear failure, respectively;  $d$  is the effective section depth.

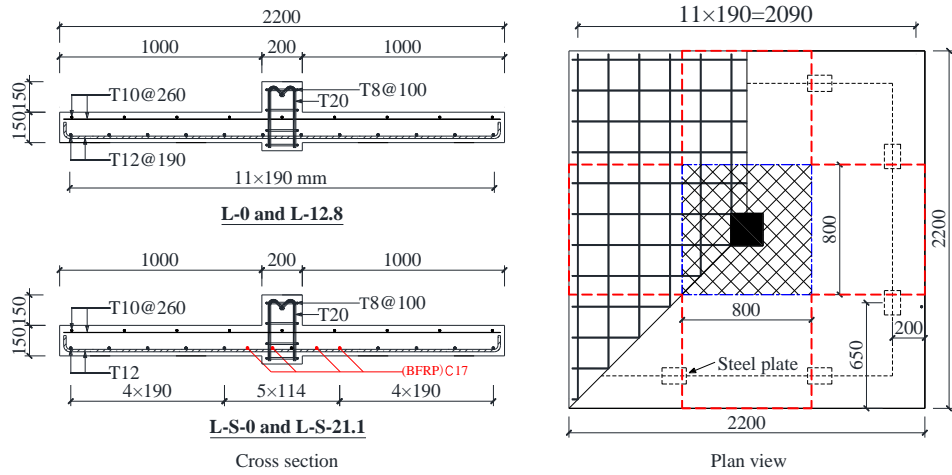
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**Table 4** Comparison of calculated load resistance with measured ones

Specimen ID	GB 50010		ACI 318		Eurocode 2		Model Code	
	$V_{GB}$	$\frac{V_u}{V_{GB}}$	$V_{ACI}$	$\frac{V_u}{V_{ACI}}$	$V_{CEN}$	$\frac{V_u}{V_{CEN}}$	$V_{fib}$	$\frac{V_u}{V_{fib}}$
	(kN)		(kN)		(kN)		(kN)	
L-0	315	0.89	327	0.86	210	1.33	295	0.95
L-20	326	0.65	334	0.63	203	1.05	285	0.74
L-S-0	320	0.92	321	0.92	221	1.33	248	1.19
L-S-20	315	0.80	315	0.80	209	1.21	233	1.08
H-0	284	1.33	298	1.26	238	1.58	334	1.13
H-20	305	0.99	318	0.95	232	1.30	326	0.93
H-S-0	318	1.21	321	1.20	261	1.48	313	1.23
H-S-20	315	1.16	319	1.15	257	1.43	299	1.23
Mean		0.99		0.97		1.34		1.06
SD		0.23		0.22		0.17		0.17
CV		0.23		0.22		0.12		0.16

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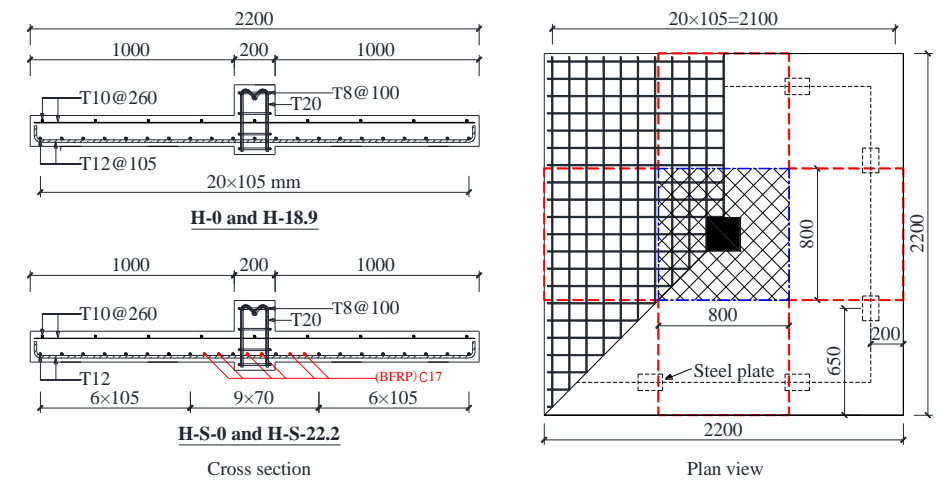
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(a)



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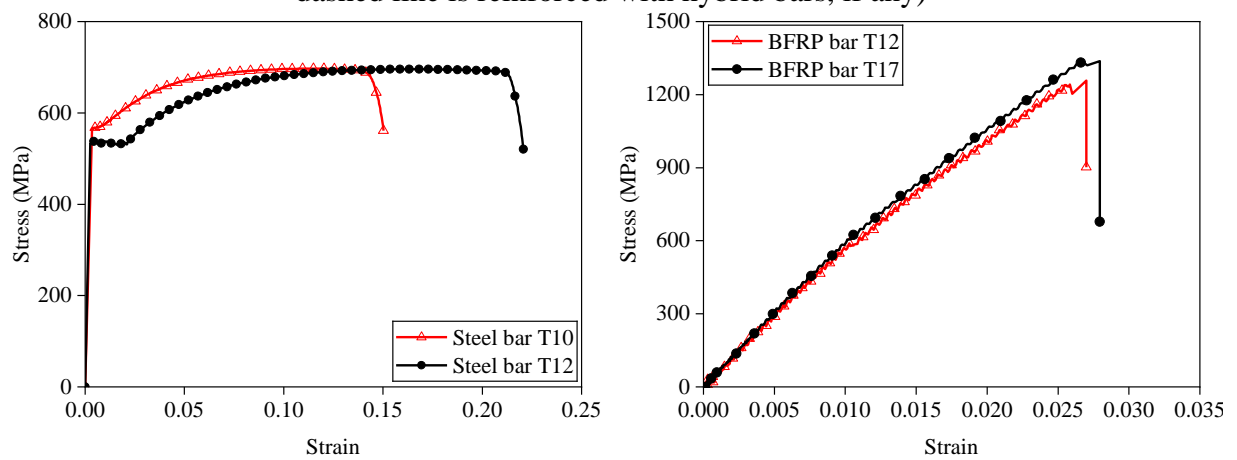
(b)

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**Fig. 1** Details of the specimens: (a) specimens with slab tension reinforcement ratio of 0.52%; (b) specimens with slab tension reinforcement ratio of 0.91% (the zone enveloped by the red dashed line is reinforced with hybrid bars, if any)

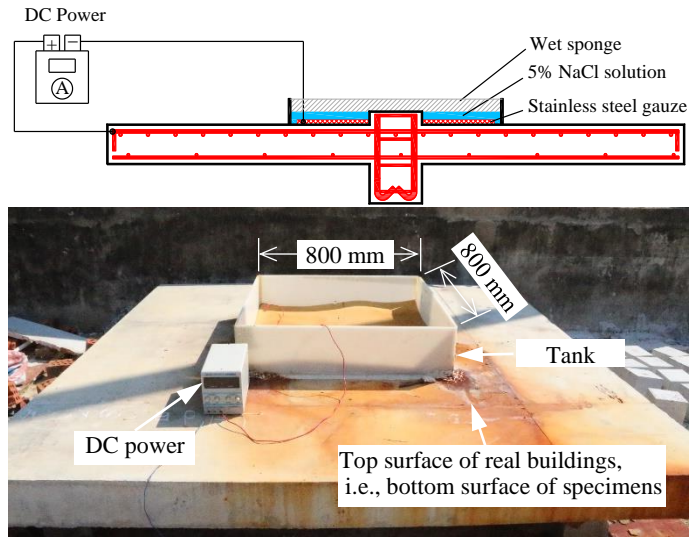


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**Fig. 2** Tensile stress-strain curve of steel bars and bars



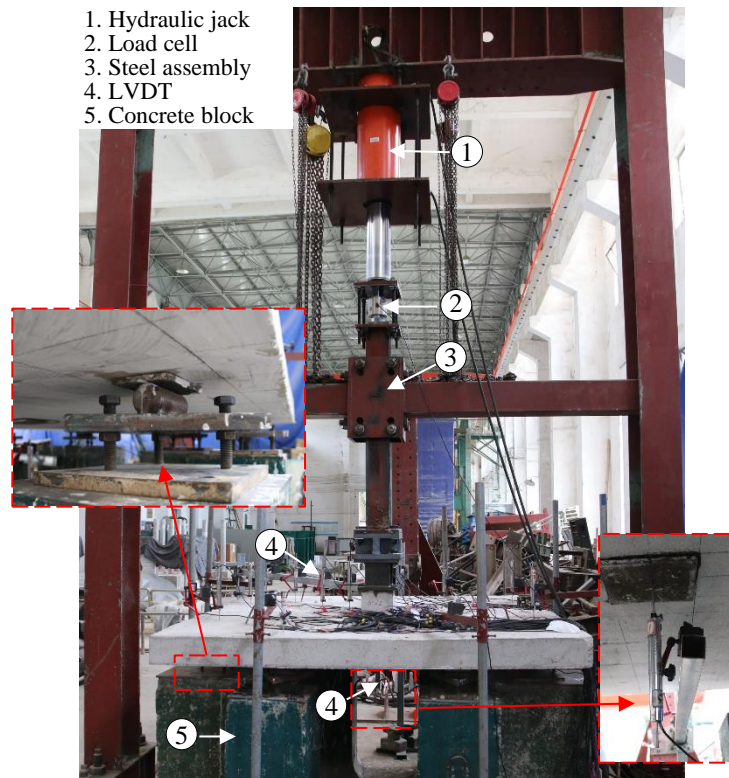
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**Fig. 3** Setup of electrified accelerated corrosion



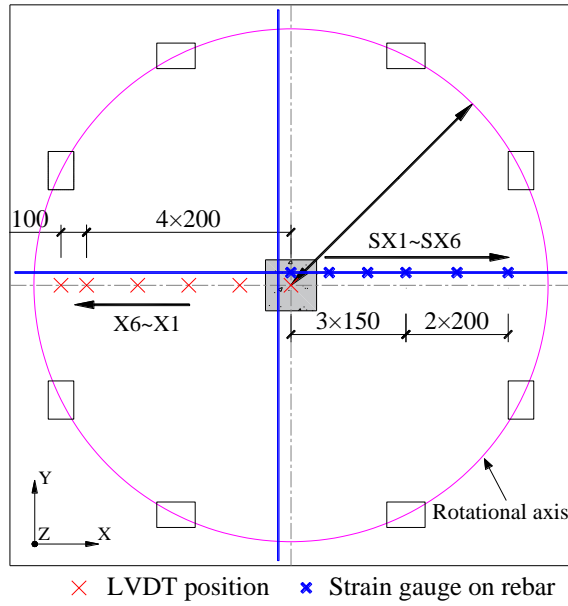
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**Fig. 4** Test setup

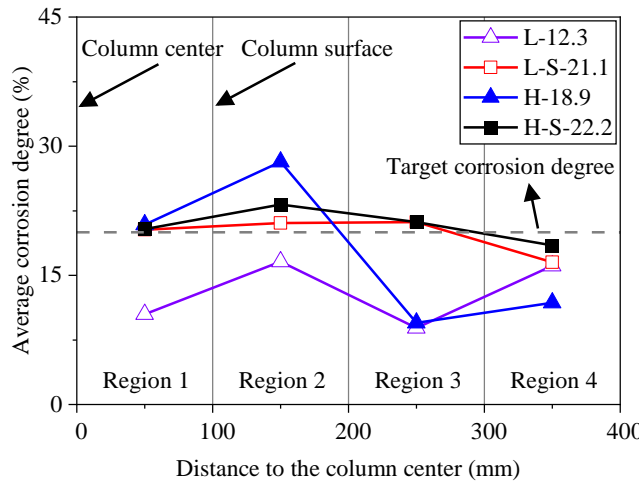


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**Fig. 5** Layout of LVDTs and strain gauges



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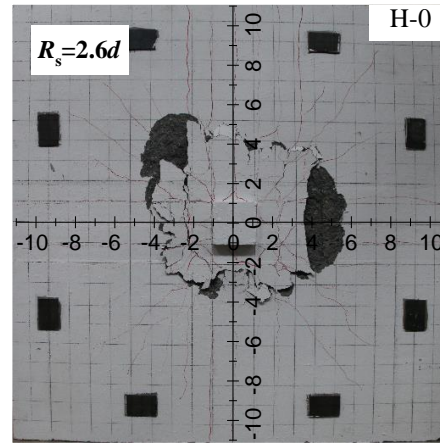
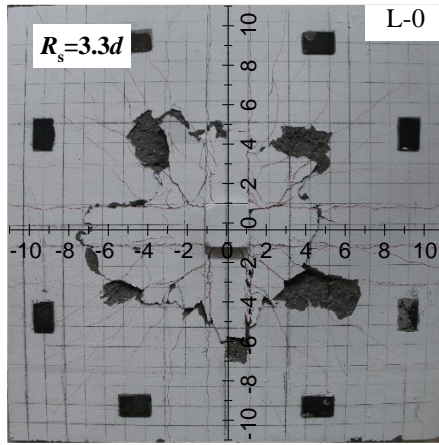
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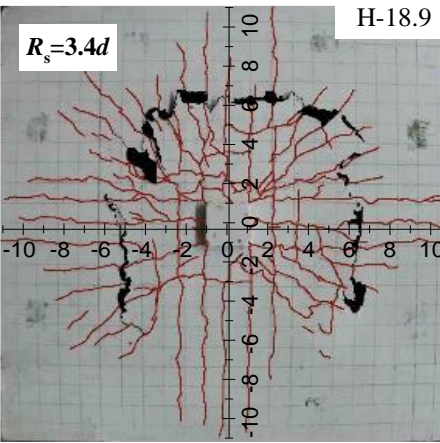
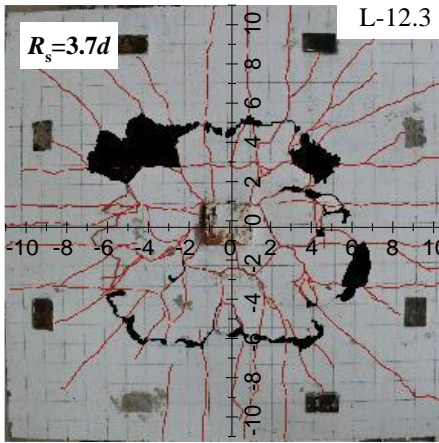
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**Fig. 6** Corrosion degree distribution

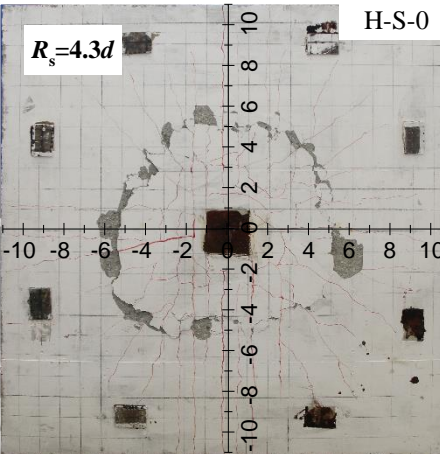
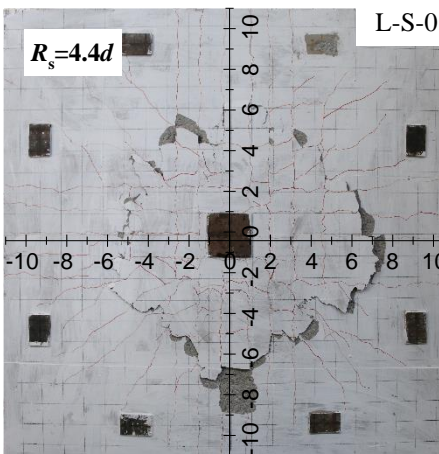
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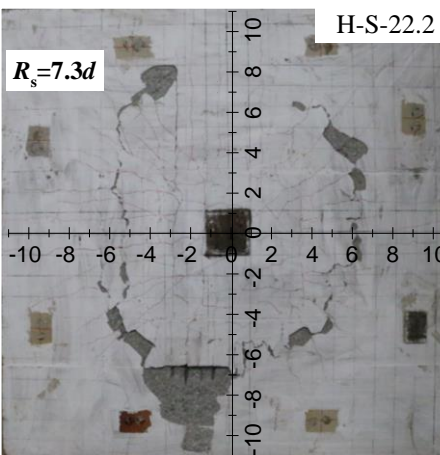
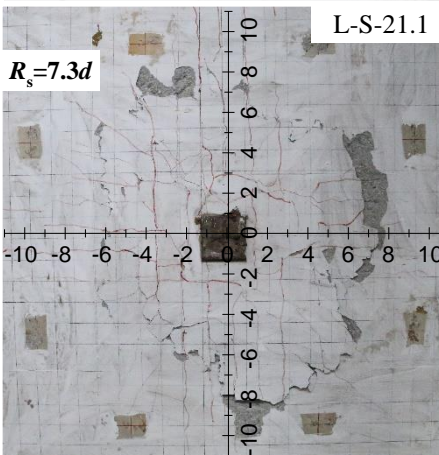
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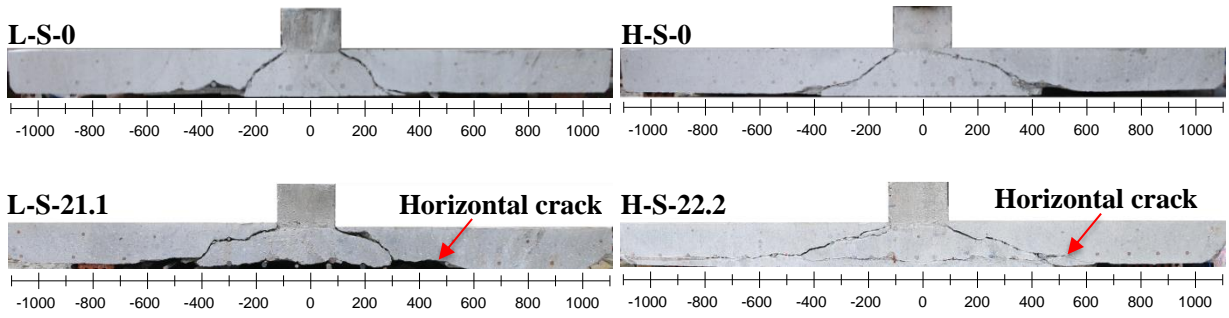
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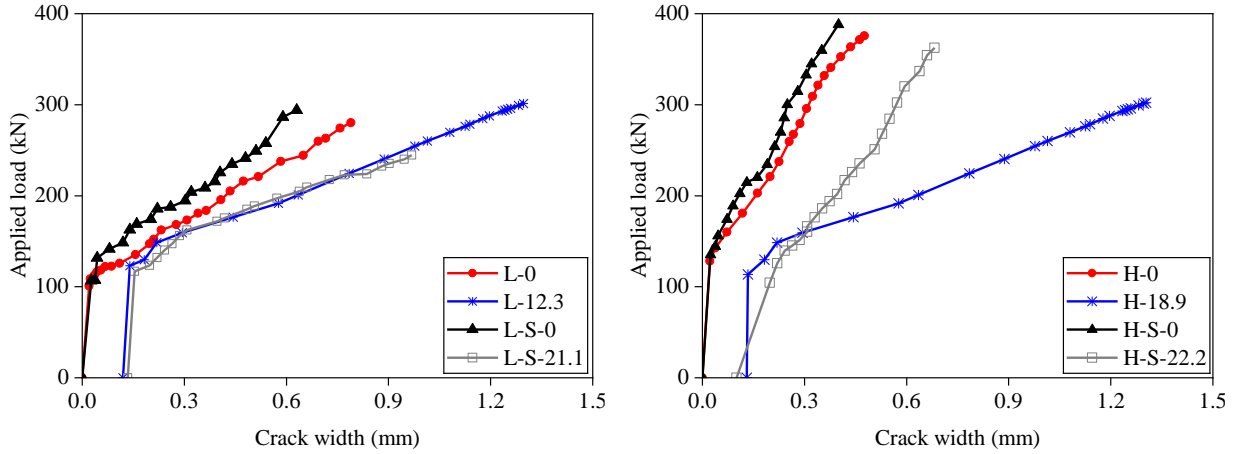
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**Fig. 7** Crack patterns and failure modes of the specimens (unit in decimeter)

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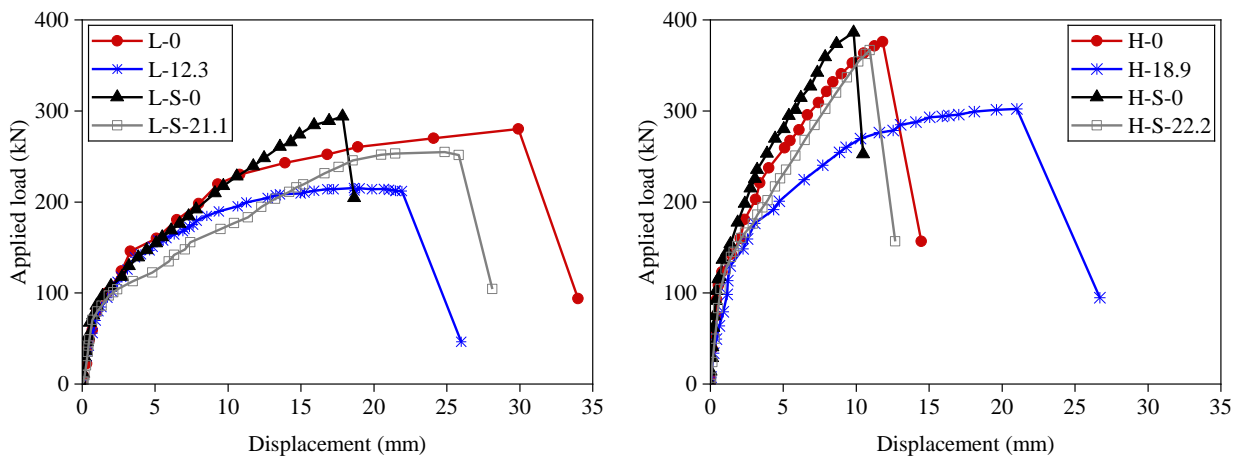
503 **Fig. 8** Punching shear cone of the specimens (unit in mm)



(a)

(b)

506 **Fig. 9** Relationship between crack width and applied load: (a) specimens with low tension  
507 reinforcement ratio; (b) specimens with high tension reinforcement ratio



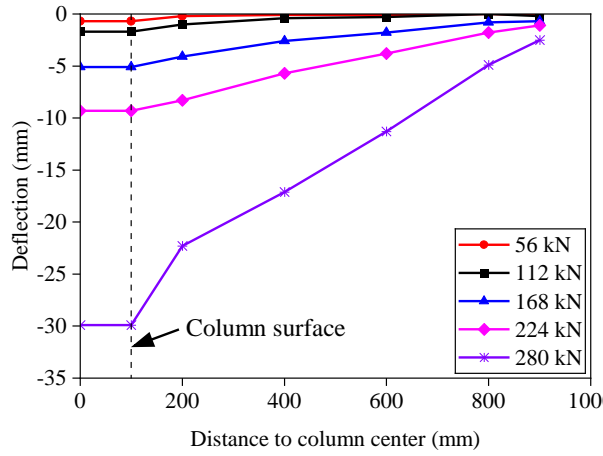
(a)

(b)

511 **Fig. 10** Load-displacement curves: (a) specimens with a tension reinforcement ratio of 0.52%;  
512 (b) specimens with a tension reinforcement ratio of 0.91%

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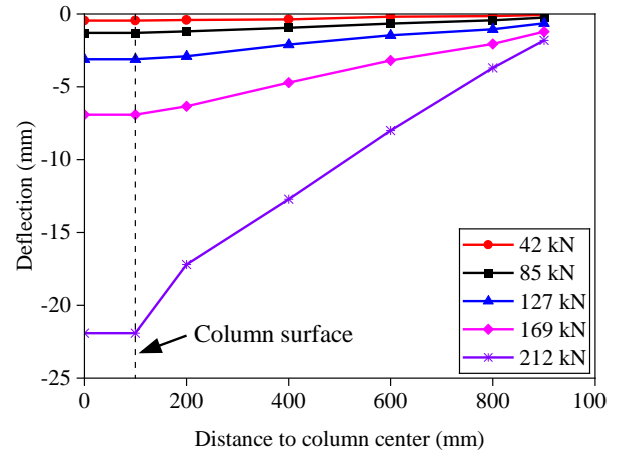


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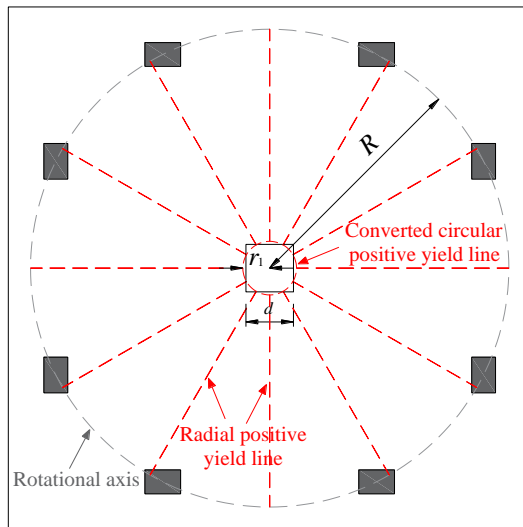
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(a)

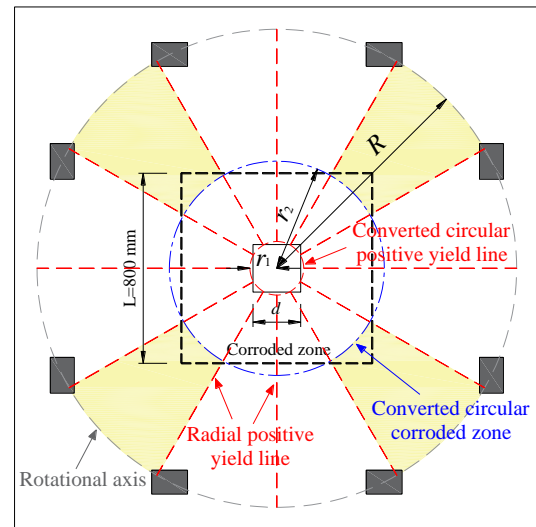
(b)

**Fig. 11** Deflection of specimens: (a) L-0; (b) L-12.8



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(a)

(b)

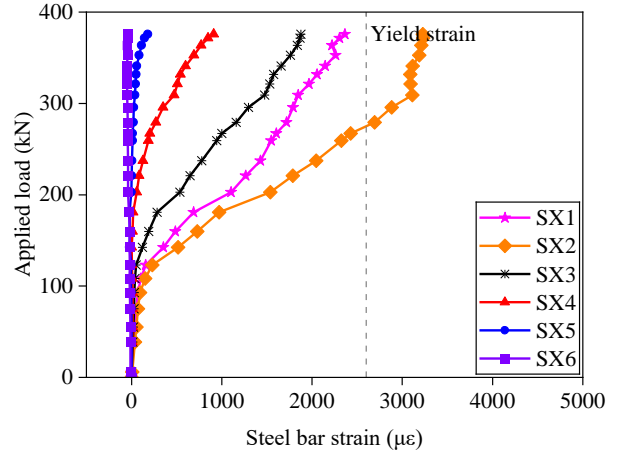
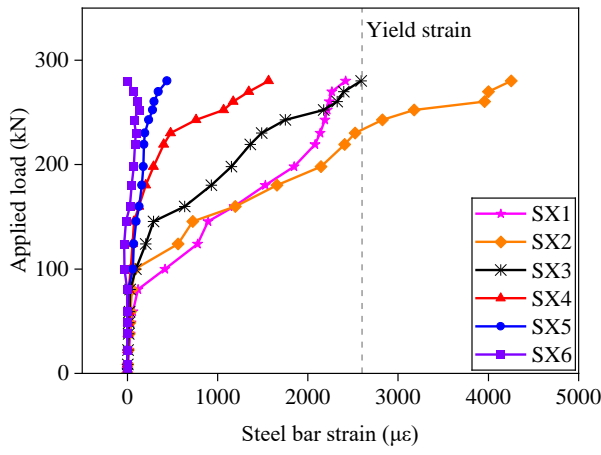
**Fig. 12** Assumed yield line pattern: (a) RC specimens; (b) corroded RC specimens and specimens with hybrid bars

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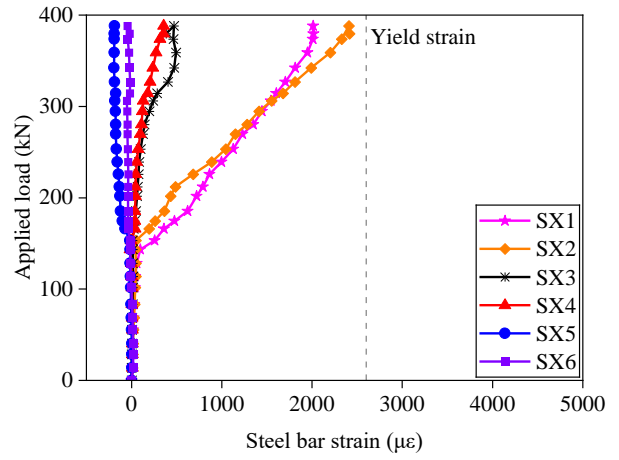
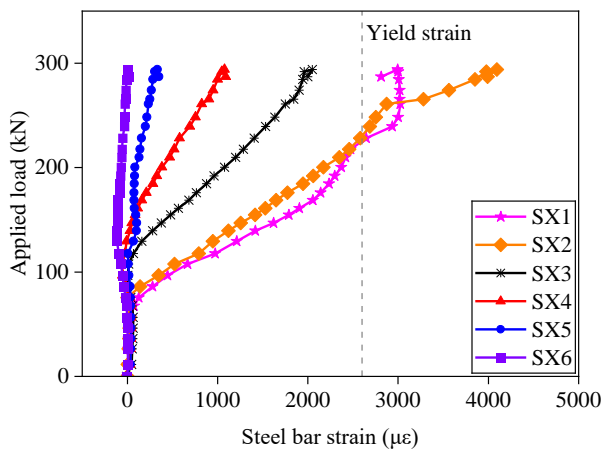


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(a)

(b)



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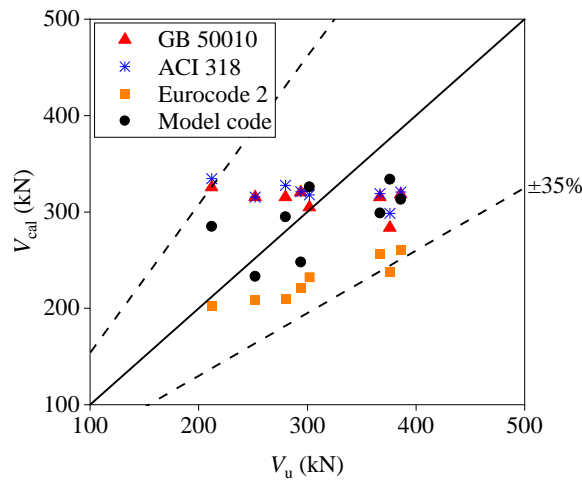
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(c)

(d)

**Fig. 13** Strain of steel bar: (a) L-0; (b) H-0; (c) L-S-0; (d) H-S-0

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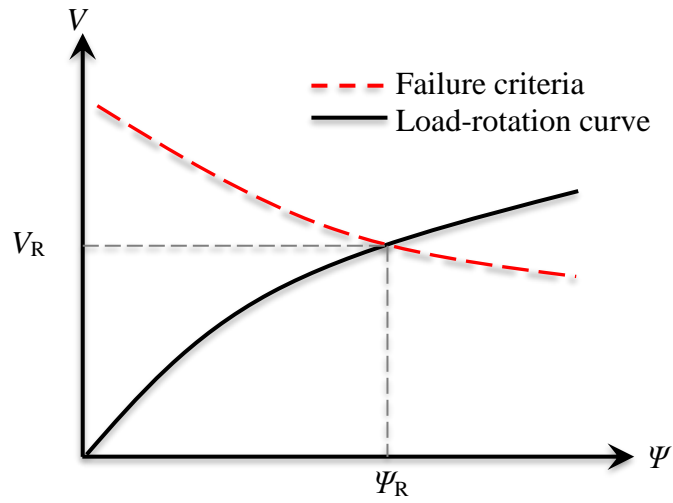


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**Fig. 14** Comparison of calculated punching shear strength of code equations with measured ones

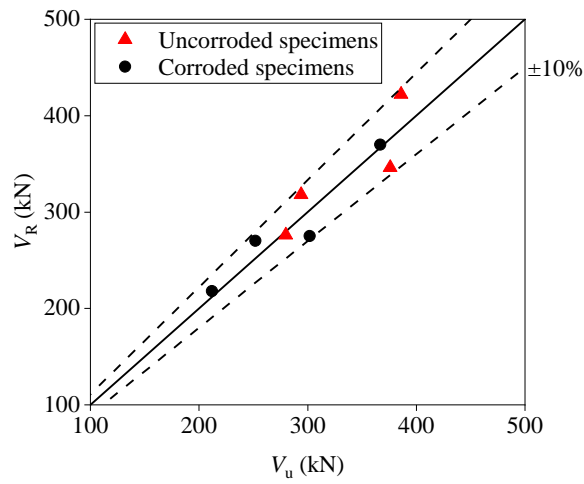


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**Fig. 15** Load-rotation relationship and failure criteria of the CSCT



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**Fig. 16** Comparison of predicted punching shear strength of the CSCT with measured ones