



## City Research Online

### City, University of London Institutional Repository

---

**Citation:** Weng, Y., Fu, F. & Qian, K. (2023). Punching Shear Resistance of Corroded Slab-Column Connections Subjected to Eccentric Load. *Journal of Structural Engineering*, 149(1), 04022219. doi: 10.1061/(asce)st.1943-541x.0003504

This is the accepted version of the paper.

This version of the publication may differ from the final published version.

---

**Permanent repository link:** <https://openaccess.city.ac.uk/id/eprint/28409/>

**Link to published version:** [https://doi.org/10.1061/\(asce\)st.1943-541x.0003504](https://doi.org/10.1061/(asce)st.1943-541x.0003504)

**Copyright:** City Research Online aims to make research outputs of City, University of London available to a wider audience. Copyright and Moral Rights remain with the author(s) and/or copyright holders. URLs from City Research Online may be freely distributed and linked to.

**Reuse:** Copies of full items can be used for personal research or study, educational, or not-for-profit purposes without prior permission or charge. Provided that the authors, title and full bibliographic details are credited, a hyperlink and/or URL is given for the original metadata page and the content is not changed in any way.

---

---

---

City Research Online:

<http://openaccess.city.ac.uk/>

[publications@city.ac.uk](mailto:publications@city.ac.uk)

---

# 1 **Punching Shear Resistance of Corroded Slab-Column Connections Subjected to Eccentric** 2 **Load**

3 Yun-Hao Weng<sup>1</sup>, Feng Fu<sup>2</sup>, C. Eng., F. ASCE, Kai Qian<sup>3\*</sup>, M. ASCE

## 4 **ABSTRACT**

5 Due to deicing salt, marine and offshore environment may cause rebar corrosion in reinforced concrete  
6 (RC) flat-slab floor system. Therefore, it increases the possibility of punching shear failure of slab-column  
7 connections. However, little research results are available for RC slab-column connections with corroded  
8 rebars under eccentric load, which is very common in realistic loading conditions. To fill the gap, 15 full-  
9 scaled RC flat slab-column connections were fabricated and tested to investigate the performance of  
10 corroded slab-column connections under eccentric load. The design variables include reinforcement ratio,  
11 loading eccentricity, and degree of rebar corrosion. There are two stages for the experimental process  
12 including: 1) accelerated rebar corrosion test; and 2) quasi-static test. It is found from the test results that,  
13 in general, rebar corrosion has detrimental effects on the punching shear strength and stiffness of the  
14 connections. In addition, corrosion of reinforcement may change the failure mode of the slab-column  
15 connection. However, it is unexpected that the energy-dissipating capacity and deformation capacity of the  
16 slab-column connection with high reinforcement ratio and small loading eccentricity increased with  
17 increasing the corroded degree.

18 **Author Keywords:** Punching shear resistance; Reinforced concrete slab-column connection;  
19 Accelerated corrosion test; Eccentric load; Experimental results.

---

21 <sup>1</sup>PhD student, College of Civil Engineering and Architecture at Guangxi University, Nanning, China  
22 530004, Email: [wengyh@st.gxu.edu.cn](mailto:wengyh@st.gxu.edu.cn)

23 <sup>2</sup>Senior Lecturer (Associate Professor) in Structural Engineering, School of Mathematics, Computer  
24 Science and Engineering, City, University of London, U.K., [Feng.Fu.1@city.ac.uk](mailto:Feng.Fu.1@city.ac.uk)

25 <sup>3</sup>Professor, College of Civil Engineering and Architecture at Guilin University of Technology, China,  
26 541004; previous Professor, Guangxi University, Nanning, China 530004, Email: [qiankai@glut.edu.cn](mailto:qiankai@glut.edu.cn)

## 28 INTRODUCTION

29 The reinforced concrete (RC) flat-slab floor system offers an economical and reliable  
30 structural option. Since 20<sup>th</sup> century, it is widely used in apartments, parking lots, bridges,  
31 dormitories, and other places with the advantages of simple formwork, flexible layout, floor height  
32 reduction, and shortened construction time. However, as no down stand beams are designed it  
33 brings a structural problem that higher shear and flexural stresses are concentrated in the slab-  
34 column connections. This can lead to brittle punching failure, leading to catastrophic  
35 consequences (Qian and Li 2015; Liu *et al.* 2015; Xue *et al.* 2018, 2020). The shear and flexural  
36 stresses in the slab-column connections were inevitably caused by the combined action of shear  
37 force and unbalanced moment due to the horizontal load or unsymmetric factors of the structure.  
38 The combined action of unbalanced bending moment and axial force was normally represented by  
39 eccentric load. Meanwhile, the flat-slab floor systems might be under harsh environmental  
40 conditions, such as de-icing salts, coastal environmental, freeze thaw cycles, and wet-dry cycles,  
41 etc., which may cause rebar corrosion. The rebar corrosion is prone to many adverse consequences,  
42 such as reducing effective area, yield strength, and ductility of steel bars (Cairns *et al.* 2005),  
43 which in turn is detrimental to punching shear strength of the connection or flexural strength of  
44 the slab.

45 Many efforts have been devoted to understanding the punching shear resistance of slab-  
46 column connections under either concentric or eccentric load (Jang and Shen 1986; Bazant and  
47 Cao 1987; Durrani *et al.* 1995; Marzouk *et al.* 1998, 2000; Hawkins *et al.* 1989; Tian *et al.* 2008;  
48 Teng *et al.* 2018; Drakatos *et al.* 2018). However, there are few quantitative studies on the impact  
49 of reinforcement corrosion on the shear resistance of slab-column connections, as most of the  
50 available researches have focused on RC slabs, columns, and beams (Okada *et al.* 1988;

51 Almusallam *et al.* 1996; Castel *et al.* 2000a, 2000b; Lee *et al.* 2000; Marano *et al.* 2008; Ikehata  
52 *et al.* 2020).

53 Since the realistic rebar corrosion procedure in RC structures is usually very slow, the  
54 artificial electrified accelerated corrosion method is commonly used to accelerate the corrosion  
55 procedure and produce equivalent corroded RC components in the laboratory (Maaddawy and  
56 Soudki 2003; González *et al.* 1995; Al-Swaidani and Aliyan 2015; Ou *et al.* 2016). In this method,  
57 the electrochemical potential is used to the rebar and external cathode, and the corrosion rate is  
58 controlled by the applied constant current density. The corroded mass loss of rebar was commonly  
59 determined based on Faraday's equation (Maaddawy and Soudki 2003):

$$60 \quad t = \frac{zFm}{MI} \quad (1)$$

61 where  $t$  is corroded time (s);  $z$  is the ionic charge;  $F$  is the constant of Faraday;  $m$  is weight of rusty  
62 steel (g);  $M$  is the atomic weight of steel; and  $I$  is the amperes of current (A).

63 So far, little research has been carried out on RC slab-column connections with corroded  
64 rebars under punching shear forces and unbalanced moments, which could be expressed by  
65 eccentric load. Therefore, to partially fulfill the gap, a series of RC slab-column connections with  
66 different corroded degree and unbalanced moments (or eccentricity), were fabricated and tested in  
67 the present study.

## 68 **EXPERIMENTAL PROGRAM**

### 69 **Specimen design and material properties**

70 A total of 15 RC full scaled slab-column connections are fabricated and tested. The size and  
71 detailed drawing of typical specimens are shown in Fig. 1. The key characteristics of the specimens  
72 are tabulated in Table 1. These specimens were categorized into five series according to loading  
73 eccentricity and reinforcement ratio: C-0.91, C-0.52, E1-0.91, E2-0.91, and E2-0.52. In each series,

74 three different corroded degrees were designed, including 0% (control specimen), 10%, and 20%.  
75 Therefore, the label of each specimen consists of three parts: The first part represents loading  
76 eccentricity: “C” for concentric loading; and “E1” and “E2” for loading eccentricity of 100 mm  
77 and 200 mm, respectively. The second one is a number to denote the flexural reinforcement ratio  
78 of the slabs: 0.52% and 0.91%. The last one is a number to denote the corroded degree (0, 10%,  
79 and 20%). For example, E1-0.91-10 represents a specimen with a reinforcement ratio of 0.91%, a  
80 designated corroded degree of 10%, and a loading eccentricity of 100 mm. As shown in Fig. 1, the  
81 size of the slab is 150×2200×2200 mm while the dimension of the center column is 200 mm. The  
82 column stub extended 150 mm and 300 mm from the top surfaces for the specimens subjected to  
83 concentric load and eccentric load, respectively. And a corbel was cast beside the column stub to  
84 apply eccentric load. The corbel and column stub were reinforced by 8T20, and the transverse  
85 reinforcement was R8@50 mm. Two layers of reinforcement mesh were used to reinforce the slab.  
86 For the top layer reinforcement, T10@260 mm mesh is designed. For the bottom layer, T12@105  
87 mm mesh is designed for specimens with reinforcement ratio of 0.91%. For the specimens with  
88 the reinforcement ratio of 0.52%, T12@190 mm mesh is utilized. “T10”, “T12”, and “T20”  
89 represent deformed rebar with a diameter of 10 mm, 12 mm, and 20 mm, respectively, while “R8”  
90 represents plain rebar with a diameter of 8 mm.

91 Based on cylinder tests, the average compressive strengths on the testing day of the specimens  
92 are shown in Table 1. The yield strength of T10 and T12 is 558 MPa and 532 MPa, respectively,  
93 whereas the ultimate strength of the T10 and T12 is 717 MPa and 695 MPa, respectively. In  
94 addition, the ultimate elongation is 15.0% for T10 and 22.1% for T12 before corrosion.

#### 95 **Test apparatus for accelerated corrosion**

96 The steel bar is corroded by electrified accelerated corrosion method. Based on Teng *et al.*

97 (2018), only the rebar within  $1.5d_o$  (where  $d_o$  is slab thickness) distance from the column edge can  
98 effectively resist the punching shear. Thus, to improve the efficiency and reduce the cost, the  
99 corroding area was restricted to  $c + 4d_o$  (800 mm, where  $c$  is the column size) in each direction,  
100 which is larger than the critical region. As shown in Fig. 2, an 800 mm square tank with 5% sodium  
101 chloride solution was placed on the upturned specimen. A wet sponge was used to cover the tank  
102 to decrease water evaporation. A stainless-steel gauze was dipped into the solution and connected  
103 to the cathode of a direct current (DC) power supply, while the reinforcements in the corroded  
104 area were connected to the anode of the DC power supply. The applied constant current density  
105 was set as  $0.6 \text{ mA/cm}^2$ . The predicted required corroding time for specimens with the corroded  
106 degree of 10% and 20% were 17 and 34 days, respectively, according to Eq. (1).

### 107 **Test setup and instrumentations**

108 The punching shear resistances of the specimens were evaluated by a quasi-static loading  
109 method. Fig. 3 shows the typical test setup for the specimens in the E series (eccentric loading  
110 case). As it can be seen in the figure, the specimen was placed on eight height-adjustable simple  
111 supports. The concentrated load is applied on the column stub or corbel using a hydraulic jack. A  
112 steel assembly was particularly designed to guarantee that the load applied was vertical. In addition,  
113 different from the specimens in the C series (concentric loading case), an additional one-way hinge  
114 was designed for the specimens in the E series to allow rotation of the column during testing.

115 A series of Linear Variable Differential Transformers (LVDTs) are installed in target positions  
116 to monitor the deflection of the specimens. For uncorroded specimens, the strain of slab  
117 reinforcements was measured during tests. However, the strain of corroded specimens was not  
118 monitored as the accelerated corrosion procedure may damage the strain gauge. A load cell, which  
119 was installed beneath the hydraulic jack, was utilized to measure applied load. A data logger with

120 60 channels and a sampling frequency of 5 Hz was utilized to record the data of all  
121 instrumentations. In addition, a crack detector was utilized to measure the crack width until failure.

## 122 **TEST RESULTS AND DISCUSSION**

### 123 **Corrosion result**

124 In the accelerated corrosion process, a large amount of corrosion product appeared at the  
125 corrosion zone. As illustrated in Fig. 4, a number of thin cracks parallel to the rebars were observed  
126 on the slab surface after cleaning away the corrosion product. Comparing the specimens with  
127 different corroded degrees, it could be found that the width of cracks increased with increasing  
128 corroded degree. The widths of the initial cracks due to rebar corrosion ranged from 0.04 mm to  
129 0.13 mm.

130 The corroded rebars within the corrosion zone (800 mm square in the center) were removed  
131 after loading test to measure the actual corroded degree. The extracted bars were cut into segments  
132 with 100 mm long. Based on the distance from the column center, the segments were grouped  
133 carefully. Afterward, the rebar segments were then immersed in a 10% hydrochloric acid solution  
134 for 24 hours to remove the corrosion product and then rinsed thoroughly. The rebar segments were  
135 then put into a thermostatic container with 60 °C for 24 hours to dry it fully before weighing. The  
136 actual corroded degree can be defined according to Eq. (2):

$$137 \quad w = \frac{W_0 - W}{W_0} \times 100 \quad (2)$$

138 where  $w$  is corroded degree (%);  $W_0$  is weight of the reinforcement segment in original; and  $W$  is  
139 weight of corroded rebar segment.

140 Table 2 summarizes the measured corrosion of the specimens. As can be seen, the maximum  
141 corroded degree occurred in the rebar around the column edge. Moreover, the maximum corroded  
142 degree could exceed the designated corroded degree. However, the average corroded degrees are

143 always less than the specified values. This may be due to the diffusion of NaCl solution from the  
144 tank to the outside of the tank, resulting in a loss of current.

### 145 **Crack pattern and failure mode**

146 Figs. 5 and 6 show the crack pattern of the specimens in the C and E series, respectively, at  
147 failure. The downward arrows in Fig. 6 represent the directions of unbalanced moments (right-  
148 hand principle). Moreover, the relationship between crack width and load before failure is shown  
149 in Fig.7. In general, for the uncorroded specimens, the first batch of cracks was occurred at the  
150 bottom side of the slab-column interfaces at about 40% of the failure load. For the corroded  
151 specimens, the initial cracks caused by the rebar corrosion developed wider at about 30% of the  
152 failure load. Further increasing the applied load, the cracks were developed towards the slab edge.  
153 Finally, circumferential cracks developed from internal oblique shear cracks were observed when  
154 specimens reaching their punching shear strength. Different from the specimens in the C series,  
155 Fig. 6 shows more damage occurring in the specimens at the side of the eccentric load (right side  
156 of each photo) due to the extra flexural tensile stresses from the moment transfer. Comparing  
157 between E1 and E2 series, the specimens in the E2 series showed more serious asymmetrical  
158 damage. As shown in Fig. 7, wider cracks are measured in the specimens with higher corroded  
159 degrees under the same load level because of the deterioration of the cracked stiffness caused by  
160 rebar corrosion. Furthermore, specimens with higher loading eccentricity have wider cracks under  
161 the same load level due to greater extra tensile stresses transferred from the unbalanced moment.

162 As illustrated in Figs. 5 and 6, all connections exhibited punching shear failure with wide  
163 circumferential cracks.  $R_{cone}$  is called critical failure zone, which is characterized by its radius from  
164 the slab-column interface to the circumferential crack. The  $R_{cone}$  is measured at different positions,  
165 and thus, only the average value was determined and tabulated in Table 3. As can be seen, the  $R_{cone}$

166 of uncorroded specimens are smaller than those of corroded specimens. This could be explained  
167 as the presence of the horizontal crack along the reinforcements due to expansive force after  
168 corrosion. When the internal oblique shear cracks intersect with the initial horizontal crack, the  
169 shear crack may along the horizontal crack before continuing to extend in depth, enlarging the  
170 critical failure zone. Similar findings were concluded in Ikehata *et al.* (2020).

### 171 **Punching shear strength**

172 Table 3 tabulated the measured punching shear strengths of tested specimens. For easier  
173 comparison of the specimens cast in different batches, the punching shear strength was normalized  
174 as below:  $v_c/\sqrt{f'_c}$ . As can be seen in Table 3, comparing to C-0.91-0, the normalized punching  
175 shear strengths of E1-0.91-0 and E2-0.91-0 decreased by 16% and 32%, respectively. For  
176 specimens with a reinforcement ratio of 0.52%, the normalized punching shear strength of E2-  
177 0.52-0 is 80% of C-0.52-0. Therefore, the existence of an unbalanced moment would decrease the  
178 punching shear strengths of the slab-column connections significantly.

179 As shown in Table 3, increasing the corroded degree, the normalized punching shear strength  
180 decreased. For C-0.91 series, the normalized punching shear strength respectively decreased by  
181 14% and 25%, when the average corroded degree was 9.6% and 17.6%. For C-0.52 series, the  
182 normalized punching shear strength decreased by 26% when the corroded degree increased from  
183 0% to 13.0%. For the eccentric loading series of E1-0.91, increasing the corroded degree from 0%  
184 to 16.4%, the normalized punching shear strength decreased by 22%. Similarly, for E2-0.91 series,  
185 when the corroded degree increased from 0% to 17.9%, the normalized punching shear strength  
186 decreased by 23%. For E2-0.52 series, when the corroded degree increased to 15.4%, the  
187 normalized punching shear strength decreased by 24%. Thus, the rebar corrosion may jeopardize  
188 the normalized punching shear strength significantly. The reduction of punching shear strength

189 comes from the decrease of the shear resistance of concrete and reinforcement. The cracks due to  
190 rebar corrosion may be adverse to the shear resistance of concrete slabs by weakening aggregate  
191 interlocking. In addition, due to the reduced cross-section of the corroded steel bars, less dowel  
192 action could be developed. The bond loss between concrete and corroded steel bars further  
193 weakened the shear resistance of the rebar. As the real rebar corrosion is different with the purpose  
194 designed corrosion for tested specimens, based on linear regression, the decreasing of normalized  
195 punching shear strength per 1% corrosion was determined for each corroded specimen. The  
196 decreasing of normalized punching shear strength per 1% corrosion was 1.42%, 1.90%, 1.3%,  
197 1.27%, and 1.56% for C-0.91, C-0.52, E1-0.91, E2-0.91 and E2-0.52 series, respectively. This  
198 indicated that the rebar corrosion has more influence on the punching shear strength of the  
199 connections with lower reinforcement ratio and less loading eccentricity.

#### 200 **Strain gauge results**

201 Fig. 8 shows the variation of strain of the rebar in uncorroded specimens at failure. As the  
202 results of the malfunctioned strain gauges were removed, some of the curve is not continual. As  
203 can be seen, different from the concentric loading cases, the strain results of the specimens  
204 subjected to eccentric load were unsymmetrical. The reinforcement strains at the side of the  
205 eccentric load were larger than those of another side, as the unbalanced moment increased the  
206 deflection in the reinforcements. In addition, the specimens with larger loading eccentricity  
207 achieved larger strain at the side of the eccentric. For C-0.91, E1-0.91, and E2-0.91 series, the  
208 yield strength is only measured at the rebar nearby the column face. This confirmed that the  
209 controlled failure mode is punching shear failure. However, for C-0.52 and E2-0.52 series, the  
210 yield strength was reached in the rebar extensively. Therefore, the failure modes of these  
211 specimens were called flexural-punching failure, which is defined as occurring punching shear

212 failure after the flexural reinforcement extensively yielded.

### 213 **Load-displacement relationship**

214 Fig. 9 comparison of the load-displacement relationships of the specimens. It should be noted  
215 that the displacement results were measured by the LVDT installed at the column center.  
216 Specimens with higher corroded degrees obtained lower initial stiffness since wider cracks  
217 occurred in the corrosion process. Moreover, it was found that increasing the corroded degrees,  
218 the post-cracking stiffness decreases significantly as the reduction of the cross-sectional area and  
219 elastic modulus of rebar caused by rebar corrosion.

220 The deformation capacity, which is defined as the displacement in accordance with the failure  
221 load, varied with the increase of the corroded degree. For C-0.91 and E1-0.91 series, increasing  
222 the corroded degree causes an increase in the deformation capacity. It resulted in greater ductility  
223 of the specimens, which could be explained as the effective reinforcement ratio decreased after  
224 rebar corrosion. On the contrary, the higher corroded degree results in a smaller deformation  
225 capacity for E2-0.91 series due to its larger unbalanced moment. For the specimens with a low  
226 reinforcement ratio, the deformation capacity decreased with increasing the corroded degree. This  
227 is because these specimens had attained the flexural strength before the punching shear strength  
228 while the yield strength of reinforcement deteriorated after rebar corrosion.

229 Table 3 tabulated the energy absorbed capacity, which is defined as the area enclosed by the  
230 load-displacement curve. It can be seen from the table, for C-0.91 series, the energy-dissipating  
231 capacity increases with the increase of corroded degree due to greater deformation capacity after  
232 corrosion. However, for C-0.52 series, the energy absorbed capacity kept decreasing with  
233 increasing the degree of rebar corrosion, as both the load resistance and deformation capacity  
234 decreased. Moreover, the E1-0.91 series showed a similar phenomenon to the C-0.91 series, while

235 the E2-0.91 and E2-0.52 series are similar to the C-0.52 series.

## 236 **Deflection shapes**

237 Fig. 10 compares the deflection shapes at the centerlines of different specimens in the  
238 direction of loading eccentricity at failure load. For eccentric loading cases, different from the  
239 symmetrical deflection of the specimens in the C series, the deflection in the side of the eccentric  
240 load is larger than those of other sides as a result of the unbalanced moment. The rotation angle of  
241 the column is  $0.7^\circ$ ,  $0.78^\circ$ ,  $0.86^\circ$ ,  $1.0^\circ$ ,  $1.07^\circ$ ,  $1.14^\circ$ ,  $1.23^\circ$ ,  $1.35^\circ$ , and  $1.54^\circ$  for E1-0.91-0, E1-0.91-  
242 10, E1-0.91-20, E2-0.91-0, E2-0.91-10, E2-0.91-20, E2-0.52-0, E2-0.52-10, and E2-0.52-20,  
243 respectively. Thus, the specimens with higher corroded degrees reach a greater relative rotation at  
244 failure load. It might be attributed to the reduction of the rotational stiffness of the column-slab  
245 connection caused by the rebar corrosion.

## 246 **ANALYTICAL ANALYSIS AND DISCUSSION**

### 247 **Yield-line analysis**

248 The yield-line method is utilized here for prediction of the flexural strength of the specimens.  
249 According to Park and Gamble (1981), the typical yield-line patterns for specimens are shown in  
250 Fig. 11. The yield lines outside the circular line are ignored. As given in Fig. 11(a), the corroded  
251 specimens in C series consisted of corroded and uncorroded areas. Based on the virtual work  
252 principle, the flexural strength  $P_{flex}$  can be calculated as follows:

$$253 \quad P_{flex} = \left[ 4m_u c + 2\pi m_{u,c} r_c + 2\pi m_u (r - r_c) \right] / r \quad (3)$$

254 where  $c$  is the column size;  $r_c$  is the equivalent radius of the corroded area, that is determined by  
255  $r_c = 2(l-c)/\pi = 382$  mm herein ( $l$  is the size of the corroded area);  $r$  is the radius of the positive  
256 moment region (870 mm herein);  $m_u$  and  $m_{u,c}$  are the nominal flexure capacity of the slab for  
257 uncorroded and corroded section, respectively. In accordance with CSA (2014),  $m_u$  and  $m_{u,c}$  can

258 be determined as follows:

$$259 \quad m_u = \rho f_y d^2 (1 - \alpha_1 \rho f_y / f_c') \quad (4)$$

$$260 \quad m_{u,c} = \rho_c f_{y,c} d^2 (1 - \alpha_1 \rho_c f_{y,c} / f_c') \quad (5)$$

$$261 \quad \alpha_1 = 0.85 - 0.0015 f_c' \quad (6)$$

262 where  $\rho_c$  and  $\rho$  are the reinforcement ratio of corroded and uncorroded section, respectively;  $f_{y,c}$   
263 and  $f_y$  are the yield strength for the corroded and uncorroded reinforcement;  $d$  is the effective depth  
264 of the slab; and  $f_c'$  is the compressive strength of concrete.

265 For the corroded section, the yield strength of the reinforcement  $f_{y,c}$  and the effective  
266 reinforcement ratio  $\rho_c$  could be determined by Eq. (7) and (8), respectively.

$$267 \quad f_{y,c} = f_y (1 - \alpha_y (w/100)) \quad (7)$$

$$268 \quad \rho_c = \rho (1 - w/100) \quad (8)$$

269 where  $w$  is the corroded degree;  $\alpha_y$  is an empirical coefficient, according to Lee *et al.* (1998), it is  
270 1.24 for uniform corrosion and 1.98 for pitting corrosion.

271 According to Park and Gamble (1981), for E series, the radius of the positive moment area is  
272 satisfied by  $r = \sqrt{1.5c} \approx 250 \text{ mm}$ , which is smaller than the corroded area in this study. Therefore,  
273 the maximum flexural moment  $M_{flex}$  of all specimens in the E series could be calculated by the  
274 formula proposed by Park and Gamble (1981), as shown in follows:

$$275 \quad M_{flex} = \begin{cases} 9.04m_u c - 0.5V_u c & \text{for uncorroded specimens} \\ 9.04m_{u,c} c - 0.5V_u c & \text{for corroded specimens} \end{cases} \quad (9)$$

276 where  $V_u$  is the applied shear force.

277 The predicted values of the tested specimens based on the yield-line analysis are summarized  
278 in Table 3. The ratio of predicted value to tested value ( $V_u/P_{flex}$  or  $M_u/M_{flex}$ ) is used as an index to  
279 determine the failure modes. If the ratio is greater than or equal to 1, the failure is flexural-  
280 punching, otherwise, the failure is controlled by punching shear. As can be seen in Table 3, the

281 ratios of C-0.91-0, C-0.52-0, E1-0.91-0, E2-0.91-0, and E2-0.52-0 are 0.86, 1.06, 0.50, 0.75, and  
 282 1.22, respectively. Therefore, the failure mode of C-0.52-0 and E2-0.52-0 is flexural-punching  
 283 failure, while that of C-0.91-0, E1-0.91-0, and E2-0.91-0 is punching shear failure, which is  
 284 consistent with the results based on strain gauge results. For corroded specimens, the ratios of C-  
 285 0.52-10, E2-0.91-20, E2-0.52-10, and E2-0.52-20 are greater than or equal to 1, indicating their  
 286 predominant flexural-punching failure, while those failed in punching shear do have ratios less  
 287 than 1. Moreover, increasing the corroded degree, the ratios of the specimens in C series decreased,  
 288 while those of the specimens in E series increased.

### 289 **Design formula in important codes**

290 In this section, the reliability of formulae proposed by ACI 318-19 (2019), GB 50010 (2010),  
 291 Eurocode 2 (2004), BS 8110 (1997), and Model Code 2010 (2012) for prediction of the punching  
 292 shear strength of uncorroded or corroded slab-column connections are evaluated.

#### 293 **ACI 318-19 (2019)**

294 For American Code, ACI 318-19 (2019), the critical section was determined by straight lines  
 295 drawn parallel to and at a distance  $d/2$  from the edges of the columns. For column-slab connection  
 296 under gravity load only, the punching shear stress  $v_c$  can be calculated by Eq. (10).

$$297 \quad v_c = \min \left\{ 0.17 \left( 1 + \frac{2}{\beta} \right) \lambda_s, 0.083 \left( 2 + \frac{\alpha_s d}{b_0} \right), 0.33 \lambda_s \right\} \lambda \sqrt{f'_c} \quad (10)$$

298 where  $\beta$  is a ratio of long side to short side of the column;  $\lambda_s$  is a factor considering size effect,  
 299  $\lambda_s = \sqrt{2/(1+0.004 \cdot d)} \leq 1.0$ ;  $\lambda$  is the density factor of concrete (for normal concrete and semi-  
 300 lightweight concrete, it is 1.0 and 0.85, respectively);  $\alpha_s$  is constant used for column location (for  
 301 interior, edge, and corner columns, it is 40, 30, and 20, respectively);  $b_0$  is the control perimeter;  
 302  $d$  is the effective depth of the slab; and  $f'_c$  is the concrete compressive strength.

303 For the interior connection with square column under eccentric load, the  $v_u$  is estimated by

304 Eq. (11).

$$305 \quad v_u = v_{uv} + \frac{\gamma_v M_{sc}(c+d)/2}{J_c} \quad (11)$$

306 where  $v_{uv}$  is the shear stress on the slab critical section;  $M_{sc}$  is the moment resisted by the column;  
307  $\gamma_v$  factor used to quantify the unbalanced moment transferred by the eccentricity of shear at slab-  
308 column connections (0.4 for square column);  $J_c$  is property of assumed critical section analogous  
309 to the polar moment of inertia; and  $c$  is the size of the square column.

### 310 **GB 50010 (2010)**

311 Similar to American Code, the assumed critical section of Chinese Code, GB 50010 (2010),  
312 is also defined as at a distance  $d/2$  from the column edge. For concentric loading cases, the  
313 punching shear strength  $V_l$  of the specimens excluded shear reinforcement can be calculated as  
314 follows:

$$315 \quad V_l \leq 0.7 \beta_h f_t \eta u_m d \quad (12)$$

$$316 \quad \eta = \min \left\{ 0.4 + \frac{1.2}{\beta_s}, 0.5 + \frac{\alpha_s d}{4u_m} \right\} \quad (13)$$

317 where  $\beta_h$  is the influence coefficient of slab thickness, 1.0 for slab thickness  $d_0 \leq 800$  mm and 0.9  
318 for  $d_0 \geq 2000$  mm;  $f_t$  is the axial tensile strength of concrete;  $u_m$  is the critical shear perimeter;  $d$  is  
319 the effective depth;  $\beta_s$  is the ratio of long to short sides of the column; and  $\alpha_s$  is an adjustment  
320 factor (40, 30, and 20 for interior, edge, and corner columns, respectively).

321 For interior connections subjected to eccentric load, the equivalent punching shear strength  
322  $V_{l,eq}$  could be determined by Eq. (14).

$$323 \quad V_{l,eq} = V_l + \frac{\alpha_0 M_{unb}(c+d)/2}{I_c} u_m d \quad (14)$$

324 where  $\alpha_0$  is the shear stress-moment transfer coefficient at the critical section (0.4 for square  
325 column);  $M_{unb}$  is the unbalanced moment at the gravity axis of the control perimeter;  $c$  is the size

326 of the square column; and  $I_c$  is the property of assumed critical section analogous to the polar  
 327 moment of inertia.

328 **Eurocode 2 (2004)**

329 Different from ACI 318-19 (2019), the Eurocode 2 (2004) specifies the critical section at a  
 330 distance  $2d$  from the column face. The punching shear stress  $v_{Rd,c}$  of the slab-column connections  
 331 under concentric loading cases can be calculated by the following expression:

$$332 \quad v_{Rd,c} = 0.18k(100\rho f'_c)^{1/3} \geq v_{\min} = 0.035k^{3/2}\sqrt{f'_c} \quad (15)$$

333 where  $k$  is the coefficient considering size effect ( $k = 1 + \sqrt{200/d} \leq 2.0$ );  $\rho$  is the average flexural  
 334 reinforcement ratio ( $\rho = \sqrt{\rho_x \rho_y} \leq 0.02$ );  $d$  is the effective depth; and  $f'_c$  is the concrete  
 335 compressive strength.

336 For eccentric loading cases, the maximum shear stress  $v_{Ed}$  should be taken as:

$$337 \quad v_{Ed} = \frac{V_{Ed}}{u_1 d} \left( 1 + \frac{3}{5} \frac{M_{Ed}}{V_{Ed}} \cdot \frac{u_1}{W_1} \right) \leq v_{Rd,c} \quad (16)$$

338 where  $V_{Ed}$  is the applied shear force;  $u_1$  is the critical shear perimeter;  $M_{Ed}$  is the applied internal  
 339 bending moment; and  $W_1$  is calculated for the basic control perimeter  $u_1$ . For an internal square  
 340 column,  $W_1$  is taken as:

$$341 \quad W_1 = \frac{c^2}{2} + c^2 + 4cd + 16d^2 + 2\pi dc \quad (17)$$

342 where  $c$  is the size of the square column.

343 **BS 8110 (1997)**

344 For the BS8110 (1997), the critical section at a distance  $1.5d$  from the column face is assumed.

345 For concentric load, the punching shear strength  $V_c$  can be calculated as follow:

$$346 \quad V_c = 0.79(100\rho)^{1/3} \left( \frac{400}{d} \right)^{1/4} \left( \frac{f'_c / 0.78}{25} \right)^{1/3} b_0 d \quad (18)$$

347 where  $\rho$  is the reinforcement ratio;  $b_0$  is the control perimeter;  $d$  is the effective depth; and  $f_c'$  is  
 348 the concrete compressive strength.

349 For eccentric loading, the punching shear strength  $V_{eff}$  is taken as follow:

$$350 \quad V_{eff} = V_t \left( 1 + \frac{1.5M_t}{V_t x} \right) \quad (19)$$

351 where  $V_t$  is the design shear force;  $x$  is the length of the perimeter side parallel to the axis of  
 352 bending; and  $M_t$  is the design moment transmitted from the slab to the column.

### 353 **Model Code 2010 (2012)**

354 The critical section of the Model Code 2010 (2012) is defined as at a distance  $d_v/2$  from the  
 355 column face. The punching shear resistance  $V_{Rd,c}$  can be calculated as:

$$356 \quad V_{Rd,c} = k_{\psi} \sqrt{f_{ck}} b_0 d_v \quad (20)$$

$$357 \quad k_{\psi} = \frac{1}{1.5 + 0.9 k_{dg} \psi d} \leq 0.6 \quad (21)$$

358 where  $f_{ck}$  is the characteristic value of compressive strength of concrete;  $b_0$  is the control perimeter;  
 359  $d_v$  is the shear-resisting effective depth of the slab;  $k_{dg}$  is the aggregate size influence parameter,  
 360  $k_{dg} = 32 / (16 + d_g) \geq 0.75$  for the maximum aggregate size  $d_g < 16$  mm and  $k_{dg} = 1.0$  for  
 361  $d_g \geq 16$  mm ;  $d$  is the effective depth of the slab; For Level II of approximation, the slab rotation  
 362  $\psi$  can be calculated as:

$$363 \quad \psi = 1.5 \cdot \frac{r_s}{d} \cdot \frac{f_{yd}}{E_s} \cdot \left( \frac{m_{Ed}}{m_{Rd}} \right)^{1.5} \quad (22)$$

364 where  $r_s$  is the position where the radial bending moment is zero with respect to the support axis;  
 365  $E_s$  is the modulus of elasticity of reinforcing steel;  $f_{yd}$  is the yield strength of reinforcing steel;  
 366  $m_{Ed}$  is the average moment per unit length for calculation of the flexural reinforcement in the  
 367 support strip;  $m_{Rd}$  is the average flexural strength per unit length in the support strip.

368 For inner columns,  $m_{Ed}$  can be calculated as:

369 
$$m_{Ed} = V_{Ed} \left( \frac{1}{8} + \frac{|e_{u,i}|}{2 \cdot b_s} \right) \quad (23)$$

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

373 
$$m_{Rd} = \rho \cdot f_{yd} \cdot d^2 \cdot \left( 1 - \frac{\rho \cdot f_{yd}}{2 \cdot f_{ck}} \right) \quad (24)$$

374 where  $\rho$  is the reinforcement ratio.

375 The effect of eccentric loading can approximately be included by multiplying the length of  
 376 the reduced basic control perimeter  $b_{1,red}$  by the coefficient of eccentricity  $k_e$ :

377 
$$b_0 = k_e \cdot b_{1,red} \quad (25)$$

378 
$$k_e = \frac{1}{1 + e_u / b_u} \quad (26)$$

379 where  $e_u$  is the eccentricity of the resultant of shear forces with respect to the centroid of the  
 380 basic control perimeter; and  $b_u$  is the diameter of a circle with the same surface as the region  
 381 inside the basic control perimeter.

382 It should be noted that the parameters used in the above formulae are the experimental ones.  
 383 Table 4 summarizes the predicted punching shear strength of the tested specimens according to  
 384 these design codes. It should be noted that the results of Eurocode 2 (2004), BS 8110 (1997), and  
 385 Model Code 2010 (2012) for the corroded specimens had already taken into account the reduction  
 386 in reinforcement ratio caused by corrosion. As the reinforcement ratio is not considered in GB  
 387 50010 (2010) and ACI 318-19 (2019) explicitly, the influence of reinforcement corrosion on  
 388 punching shear strength of slab-column connection is not considered in both codes. Therefore, the  
 389 predicted value would be unchangeable after rebar corrosion, although the test value decreased  
 390 with the increase of the corroded degree, which would result in an unconservative predicted value

391 ( $V_{test}/V_{th} < 1.0$ ) for the specimens with high corroded degree. These unsafe predictions may be  
392 caused by the change of failure mechanism. For Eurocode 2 (2004) and BS 8110 (1997), the result  
393 of  $V_{test}/V_{th}$  is far less than 1.0 for the specimens with relatively high corroded degree, such as  
394 Specimens C-0.52-20, E2-0.91-20, and E2-0.52-20, indicating that the formulae proposed by  
395 Eurocode 2 (2004) and BS 8110 (1997) could not predict the punching shear strength of the  
396 corroded slab-column connections well, although both codes had considered the reduction of  
397 reinforcement ratio caused by rebar corrosion. This is because the reinforcement corrosion not  
398 only reduces the reinforcement ratio, but also weakens interlocking action of the aggregate and  
399 dowel action of the rebar, which are ignored in BS 8110 (1997) and Eurocode 2 (2004). Model  
400 Code 2010 (2012) has a relatively high conservative estimation for all the uncorroded specimens.  
401 Although the value  $V_{test}/V_{th}$  of Model Code 2010 (2012) decreases with increasing the corroded  
402 degree, it is still able to maintain a conservative estimation for a relatively high corroded degree.

403 In summary, all these formulae in codes could not accurately predict the punching shear  
404 strength of corroded slab-column connections. Therefore, it is necessary to further study the  
405 mechanism of punching shear resistance of corroded slab-column connection. According to the  
406 critical shear crack theory (CSCT) from Muttoni (2008), the punching shear failure is caused by  
407 the opening of a critical shear crack. Based on the above assumption, Muttoni (2008) proposed a  
408 failure criterion for punching shear. This failure criterion can predict the punching shear strength  
409 of the uncorroded RC slab without transverse reinforcement accurately. After rebar corrosion, the  
410 opening of the critical shear crack would be larger than that of the uncorroded case under the same  
411 load. Therefore, a magnification factor of the crack width could be introduced in the failure  
412 criterion to consider the influence of rebar corrosion. However, as the number of tests was limited,  
413 the data measured was not enough to establish the relationship between corroded degree and

414 punching shear strength. Thus, it is suggested to conduct more tests to fill this gap.

## 415 **Conclusions**

416 The punching shear resistance of corroded slab-column connections under eccentric load was  
417 investigated in this study. The following conclusions were drawn based on the experimental and  
418 analytical results:

- 419 1. Comparing to the slab-column connections subjected to concentric load, the slab-column  
420 connections subjected to eccentric load show more damage, greater reinforcement strain,  
421 wider crack width, and larger deflection at the side of eccentric load due to extra stress that  
422 developed from the unbalanced moment transfer. Moreover, the existence of unbalanced  
423 moment has a great influence on the punching shear resistance of the slab-column connections.  
424 When increasing the loading eccentricity from zero to one time the column dimension, the  
425 punching shear strength decreases by 32% and 20% for the slab-column connections with  
426 reinforcement ratios of 0.91% and 0.52%, respectively.
- 427 2. The strain gauge results together with the yield-line results demonstrated that the rebar  
428 corrosion may change the failure mode of slab-column connections. C-0.52-0 and C-0.52-10  
429 failed in flexure first, while C-0.52-20 failed in pure punching shear failure. In addition, E2-  
430 0.91-0 and E2-0.91-10 failed in pure punching shear failure, while E2-0.91-20 failed in  
431 flexure first.
- 432 3. With increasing the corroded degree, the crack widths of the slab-column connections  
433 decrease under the same load level due to the detrimental of the cracked stiffness caused by  
434 rebar corrosion. In addition, the critical failure zone of the corroded slab-column connections  
435 is larger than that of uncorroded connections, because the critical shear cracks may extend  
436 along the existing horizontal cracks, which were along the corroded reinforcement.

- 437 4. Rebar corrosion reduces the punching shear resistance of the slab-column connection. With  
438 increasing the corroded degree to about 20%, the punching shear strength of C1-0.91-20, C-  
439 0.52-20, E1-0.91-20, E2-0.92-20, and E2-0.52-20 decreased by 24.5%, 25.9%, 21.7%, 23.2%,  
440 and 24.0%, respectively, comparing to their controlled specimens. This reduction can be  
441 attributed to weakened interlocking of aggregate, dowel action of rebar, and bond strength  
442 between concrete and reinforcement caused by rebar corrosion. In addition, the slab-column  
443 connection with lower reinforcement ratio or larger loading eccentricity is more susceptible  
444 to the rebar corrosion effects.
- 445 5. Rebar corrosion may change the deformation capacity of the slab-column connections. The  
446 deformation capacity of the slab-column connections with high reinforcement ratio and small  
447 loading eccentricity may increase with increasing the corroded degree owing to lowered  
448 effective reinforcement ratio due to rebar corrosion, and consequently may lead to an increase  
449 in the energy-dissipating capacity. However, for the slab-column connections with a low  
450 reinforcement ratio and high loading eccentricity, the deformation capacity and energy-  
451 dissipating capacity decrease with increasing the corroded degree.

#### 452 **Data Availability**

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

#### 455 **Acknowledgements**

456 This research was supported by a research grant provided by the National Natural Science  
457 Foundation of China (Nos. 52022024, 52168028). Natural Science Foundation of Guangxi  
458 (No.2021GXNSFFA196001). Any opinions, findings and conclusions expressed in this paper are  
459 those of the writers and do not necessarily reflect the view of National Natural Science Foundation

460 of China.

461 **REFERENCES**

- 462 American Concrete Institute (ACI). 2019. Building code requirements for structural concrete  
463 (ACI 318-19) and commentary. ACI Committee 318. Farmington Hills, MI: ACI.
- 464 Almusallam, A. A., Al-Gahtani, A. S., Aziz, A. R., *et al.* (1996). “Effect of reinforcement  
465 corrosion on flexural behaviour of concrete slabs.” *Journal of Materials in Civil Engineering*,  
466 8(3): 123-127.
- 467 Al-Swaidani, A. M., and Aliyan, S. D. (2015). “Effect of adding scoria as cement replacement on  
468 durability-related properties.” *International Journal of Concrete Structures and Materials*,  
469 9(2): 241-254.
- 470 Bazant, Z. P., and Cao, Z. P. (1987). “Size effect in punching shear failure of slabs.” *ACI Structural*  
471 *Journal*, 84(6): 44-53.
- 472 BS 8110. (1997). “Structural use of concrete. Part I: Code of practice for design and construction.”  
473 Br. Stand. Institution, UK.
- 474 Cairns, J., Plizzari, G. A., Du, Y., Law, D. W., and Franzoni, C. (2005). “Mechanical properties of  
475 corrosion-damaged reinforcement.” *ACI Structural Journal*, 102(4): 256-264.
- 476 Castel, A., Francois, R., and Arliguie, G. (2000a). “Mechanical behaviour of corroded reinforced  
477 concrete beams-part 1: experimental study of corroded beams.” *Materials and structures*,  
478 33(9): 539-544.
- 479 Castel, A., Francois, R., and Arliguie, G. (2000b). “Mechanical behaviour of corroded reinforced  
480 concrete beams-Part 2: Bond and notch effects.” *Materials and structures*, 33(9): 545-551.
- 481 CSA (Canadian Standards Association). (2014). “Design of concrete structures.” *CAN/CSA A23.3-*  
482 *14*, Mississauga, ON, Canada.

483 Drakatos, I. S., Muttoni, A., and Beyer, K. (2018). “Mechanical model for drift-induced punching  
484 of slab-column connections without transverse reinforcement.” *ACI Structural Journal*, 115(2):  
485 463-474.

486 Durrani, A. J., Du, Y., and Luo, Y. H. (1995). “Seismic resistance of nonductile slab-column  
487 connections in existing flat-slab buildings.” *ACI Structural Journal*, 92(4): 479-487.

488 European Committee for Standardization (CEN). (2004). “Design of concrete structures—Part 1-  
489 1: General rules and rules for buildings.” Eurocode 2, Brussels, Belgium.

490 Fédération International du Béton (fib). (2012). Model Code 2010 final completed draft, Bulletins  
491 d'Informations 65 and 66. Fédération International du Béton (fib), Lusanne, Switzerland.

492 GB 50010. (2010). “Code for Design of Concrete Structures”. Ministry of House and Urban-Rural  
493 Development of People’s Republic of China, China.

494 González, J. A., Andrade, C., Alonso, C., and Feliu, S. (1995). “Comparison of rates of general  
495 corrosion and maximum pitting penetration on concrete embedded steel reinforcement.”  
496 *Cement and Concrete Research*, 25(2): 257-264.

497 Hawkins, N. M., Bao, A., and Yamazaki, J. (1989). “Moment transfer from concrete slabs to  
498 columns.” *ACI Structural Journal*, 86(70): 705-716.

499 Ikehata, S., Ishiguro, H., Nakano, T., and Nakamura, H. (2020). “Experimental evaluation of  
500 punching shear capacity of reinforced concrete slabs with horizontal crack due to compression  
501 rebar corrosion.” *Structural Concrete*, 21(3).

502 Jang, D. H., and Shen, J. H. (1986). “Strength of concrete slabs in punching shear.” *Journal of*  
503 *Structural Engineering*, 112(13): 2578-2591.

504 Lee, C., Bonacci, J. F., Thomas, M. D., Maalej, M., Khajehpour, S., Hearn, N., *et al.* (2000).  
505 “Accelerated corrosion and repair of reinforced concrete columns using carbon fibre

506 reinforced polymer sheets.” *Canadian Journal of Civil Engineering*, 27(5): 941-948.

507 Lee, H., Noguchi, T., and Tomosawa, F. (1998). “FEM analysis for structural performance of  
508 deteriorated RC structures due to rebar corrosion.” *In Proceedings of the second international  
509 conference on concrete under severe conditions*, Tromso, Norway, 327-336.

510 Liu, J. R., Tian, Y., Orton, S. L., Said, A. M. (2015). “Resistance of flat-plate buildings against  
511 progressive collapse. 1: modelling of slab-column connections.” *Journal of Structural  
512 Engineering*, 141(12).

513 Maaddawy, T., and Soudki, K. A. (2003). “Effectiveness of impressed current technique to  
514 simulate corrosion of steel reinforcement in concrete.” *Journal of Materials in Civil  
515 Engineering*, 15(1): 41-47.

516 Marano, G. C., Quaranta G., Mezzina M. (2008). “Fuzzy time-dependent reliability analysis of  
517 RC beams subject to pitting corrosion.” *Journal of Materials in Civil Engineering*, 20(9): 578-  
518 587.

519 Marzouk, H., Emam, M., and Hilal, M. S. (1998). “Effect of high-strength concrete slab on the  
520 behaviour of slab-column connections.” *ACI Structural Journal*, 95(3): 227-237.

521 Marzouk, H., Osman, M., and Helmy, S. (2000). “Behavior of high-strength lightweight aggregate  
522 concrete slabs under column load and unbalanced moment.” *ACI Structural Journal*, 97(3):  
523 860-866.

524 Muttoni, A. (2008). “Punching shear strength of reinforced concrete slabs without transverse  
525 reinforcement”, *ACI Structural Journal*, 105(4): 440-450.

526 Okada, K., Kobayashi, K., and Miyagawa, T. (1988). “Influence of longitudinal cracking due to  
527 reinforcement corrosion on characteristics of reinforced concrete members.” *Journal of  
528 Structural Engineering*, 85(2): 134-140.

529 Ou, Y., Susanto, Y. T. T., and Roh, H. (2016). "Tensile behavior of naturally and artificially  
530 corroded steel bars." *Construction and Building Materials*, 103: 93-104.

531 Park, R., and Gamble, W. L. (1981). "Reinforced Concrete Slabs." Wiley.

532 Qian, K., and Li, B. (2015). "Strengthening of multibay reinforced concrete flat slabs to mitigate  
533 progressive collapse." *Journal of Structural Engineering*, 141(6).

534 Xue, H. Z., Gilbert, B. P., Guan, H., and Lu, X. Z. (2018). "Load transfer and collapse resistance  
535 of RC flat plates under interior column removal scenario." *Journal of Structural Engineering*,  
536 144(7).

537 Xue, H. Z., Guan, H., Gilbert, B. P., and Lu, X. Z. (2020). "Comparative and parametric studies  
538 on behavior of RC-flat plates subjected to interior-column loss" *Journal of Structural*  
539 *Engineering*, 146(9).

540 Teng, S., Chanthabouala, K., Lim, D. T. Y., and Hidayat, R. (2018). "Punching shear strength of  
541 slabs and influence of low reinforcement ratio." *ACI Structural Journal*, 115(6):1815-1815.

542 Tian, Y., Jirsa, J. O., Widiyanto, O. B., and Argudo, J. F. (2008). "Behavior of slab-column  
543 connections of existing flat-plate structures." *ACI Structural Journal*, 105(5):561-569.

544  
545 **FIGURE CAPTIONS**

546 **Fig. 1.** Dimensions and reinforcement details

547 **Fig. 2.** Accelerated corrosion test setup

548 **Fig. 3.** Test setup

549 **Fig. 4.** Surface condition of E1-0.91-20 after the corrosion process

550 **Fig. 5.** Crack patterns and critical failure zones of specimens subjected to concentric load

551 **Fig. 6.** Crack patterns and critical failure zones of specimens subjected to eccentric load

552 **Fig. 7.** Relationship between crack width and load: (a) C-0.91 and E1-0.91 series; (b) E2-0.91 Fig.

553 **Fig. 8.** Reinforcement strain profile of uncorroded specimens at the failure load

554 **Fig. 9.** Load-displacement curves of the specimens: (a) C-0.91 and E1-0.91 series; (b) E2-0.91  
555 series; (c) C-0.52 and E2-0.52 series

556 **Fig. 10.** Deflection profiles of the specimens at failure load: (a) C-0.91 and C-0.52 series; (b) E1-  
557 0.91 and E2-0.91 series

558 **Fig. 11.** Typical yield-line patterns: (a) C series; (b) E series

559

560

**Table 1.** Characteristics of Specimens

	Specimen ID	Slab dimensions (m)	Column size (mm)	$e$ (mm)	$f'_c$ (MPa)	Bottom reinforcement (mm)	$\rho$ (%)	Target corroded degree (%)
C-0.91 Series	C-0.91-0				36.3			0
	C-0.91-10	2.2×2.2×0.15	200	0	39.3	T12@105	0.91	10
	C-0.91-20				41.1			20
C-0.52 Series	C-0.52-0				43.7			0
	C-0.52-10	2.2×2.2×0.15	200	0	44.1	T12@190	0.52	10
	C-0.52-20				45.6			20
E1-0.91 Series	E1-0.91-0				38.0			0
	E1-0.91-10	2.2×2.2×0.15	200	100	36.5	T12@105	0.91	10
	E1-0.91-20				36.5			20
E2-0.91 Series	E2-0.91-0				40.1			0
	E2-0.91-10	2.2×2.2×0.15	200	200	39.3	T12@105	0.91	10
	E2-0.91-20				39.3			20
E2-0.52 Series	E2-0.52-0				40.5			0
	E2-0.52-10	2.2×2.2×0.15	200	200	36.5	T12@190	0.52	10
	E2-0.52-20				39.3			20

561 Note: The clear cover of concrete is 20 mm; The maximum size of aggregate is 20mm;  $e$  is the loading eccentricity;  $f'_c$  is the  
562 compressive strength of concrete;  $\rho$  is average flexural reinforcement ratio.

563

564

565

566

567

568

569

570

571

572

573  
574  
575

**Table 2.** Corrosion result of the specimens

Specimen ID	Designated corroded degree (%)	Distribution of corroded degree from column center				Average corroded degree (%)
		0-100 mm	100-200 mm	200-300 mm	300-400 mm	
		(%)	(%)	(%)	(%)	
C-0.91-10	10	11.8	14.2	7.0	5.4	9.6
C-0.91-20	20	20.9	28.2	9.5	11.8	17.6
C-0.52-10	10	11.6	7.8	7.3	5.9	8.1
C-0.52-20	20	10.5	16.6	8.9	16.1	13.0
E1-0.91-10	10	7.1	7.7	6.4	4.6	6.4
E1-0.91-20	20	18.2	18.8	13.8	14.6	16.4
E2-0.91-10	10	9.6	10.3	6.6	5.6	8.0
E2-0.91-20	20	21.0	22.5	15.7	12.7	17.9
E2-0.52-10	10	6.4	7.5	7.0	4.2	6.3
E-0.52-20	20	31.0	18.8	7.4	4.5	15.4

576  
577  
578  
579  
580

**Table 3.** Summary of Test Results

	Specimen ID	$V_u$ (kN)	$M_u$ (kN·m)	$v_u/\sqrt{f'_c}$	$R_{cone}$	Energy dissipation (kN·mm)	$P_{flex}$ (kN)	$M_{flex}$ (kN·m)	$V_u/P_{flex}$ or $M_u/M_{flex}$	Failure mode
C-0.91 Series	C-0.91-0	376	-	62.4	$2.6d_0$	3115	436	-	0.86	P
	C-0.91-10	335	-	53.4	$4.0d_0$	3385	386	-	0.87	P
	C-0.91-20	302	-	47.1	$3.4d_0$	5096	366	-	0.83	P
C-0.52 Series	C-0.52-0	280	-	42.4	$3.3d_0$	6669	265	-	1.06	F
	C-0.52-10	244	-	36.7	$4.2d_0$	4504	241	-	1.01	F
	C-0.52-20	212	-	31.4	$3.7d_0$	3889	230	-	0.92	P
E1-0.91 Series	E1-0.91-0	324	32.4	52.6	$3.2d_0$	2327	-	64.4	0.50	P
	E1-0.91-10	294	29.4	48.7	$3.3d_0$	2370	-	53.6	0.55	P
	E1-0.91-20	249	24.9	41.2	$4.9d_0$	2713	-	42.0	0.59	P
E2-0.91 Series	E2-0.91-0	267	53.4	42.2	$3.6d_0$	1457	-	70.7	0.75	P
	E2-0.91-10	241	48.2	38.4	$4.7d_0$	1250	-	56.3	0.85	P
	E2-0.91-20	203	40.6	32.4	$3.7d_0$	937	-	40.7	1.00	F
E2-0.52 Series	E2-0.52-0	215	43.0	33.8	$2.7d_0$	2519	-	35.1	1.22	F
	E2-0.52-10	184	36.8	30.5	$2.8d_0$	1951	-	29.9	1.23	F
	E2-0.52-20	161	32.2	25.7	$4.0d_0$	1366	-	21.5	1.49	F

581 Note:  $V_u$  is the failure load;  $R_{cone}$  is the radius of the critical failure zone;  $d_0$  is the slab thickness;  $P_{flex}$  and  $M_{flex}$  are the flexural  
582 strength calculated by Eq. (3) and Eq.(9), respectively; P and F represent the failure mode of punching shear failure and flexural-  
583 punching failure, respectively.

584  
585  
586  
587  
588

589

590

591

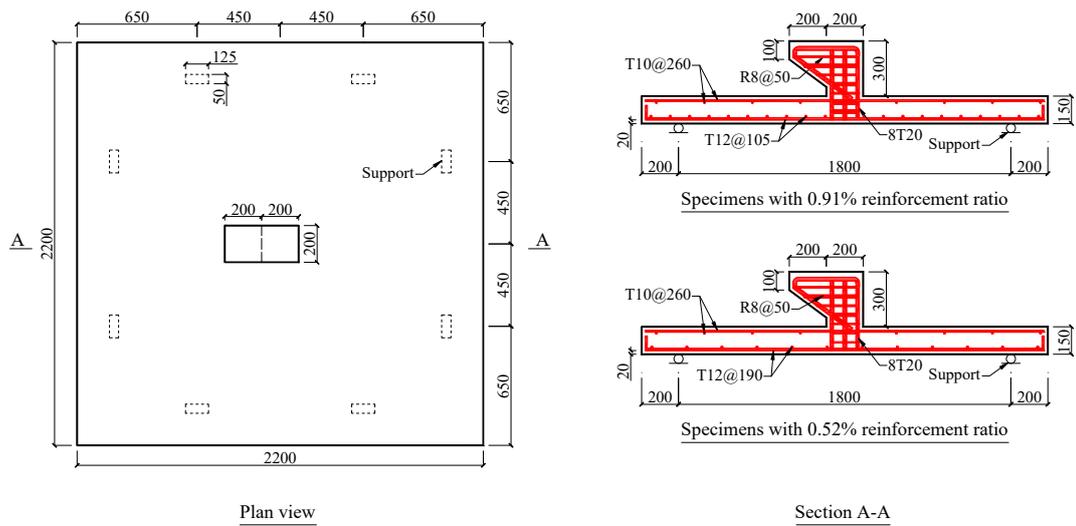
**Table 4.** Comparison between tests results and predictions

Specimen ID	ACI 318-19 (kN)		GB 50010 (kN)		Eurocode 2 (kN)		BS 8110 (kN)		Model Code (kN)	
	$V_{th}$	$\frac{V_{test}}{V_{th}}$	$V_{th}$	$\frac{V_{test}}{V_{th}}$	$V_{th}$	$\frac{V_{test}}{V_{th}}$	$V_{th}$	$\frac{V_{test}}{V_{th}}$	$V_{th}$	$\frac{V_{test}}{V_{th}}$
C-0.91-0	298	1.26	284	1.33	275	1.21	334	1.13	285	1.32
C-0.91-10	310	1.08	294	1.14	308	1.09	331	1.01	275	1.22
C-0.91-20	317	0.95	305	0.99	304	0.99	326	0.92	263	1.15
C-0.52-0	327	0.86	315	0.89	275	1.02	295	0.95	234	1.20
C-0.52-10	329	0.74	315	0.77	268	0.91	288	0.85	221	1.10
C-0.52-20	334	0.63	326	0.65	265	0.80	285	0.74	215	0.99
E1-0.91-0	224	1.45	212	1.53	285	1.14	293	1.11	228	1.42
E1-0.91-10	219	1.34	207	1.42	275	1.07	282	1.04	217	1.35
E1-0.91-20	219	1.14	207	1.20	268	0.93	272	0.92	203	1.23
E2-0.91-0	181	1.47	171	1.35	235	1.14	298	0.90	193	1.38
E2-0.91-10	180	1.34	169	1.24	227	1.06	288	0.84	182	1.32
E2-0.91-20	173	1.17	169	1.04	218	0.93	277	0.73	170	1.19
E2-0.52-0	182	1.18	174	1.23	195	1.10	246	0.87	151	1.42
E2-0.52-10	173	1.06	164	1.12	185	0.99	232	0.79	140	1.31
E2-0.52-20	179	0.90	164	0.98	183	0.88	230	0.70	134	1.20
Average		1.10		1.13		1.02		0.90		1.25
SD		0.24		0.23		0.11		0.13		0.12
COV		0.21		0.21		0.11		0.14		0.09

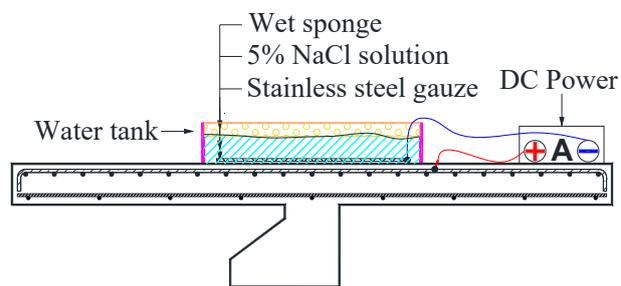
592

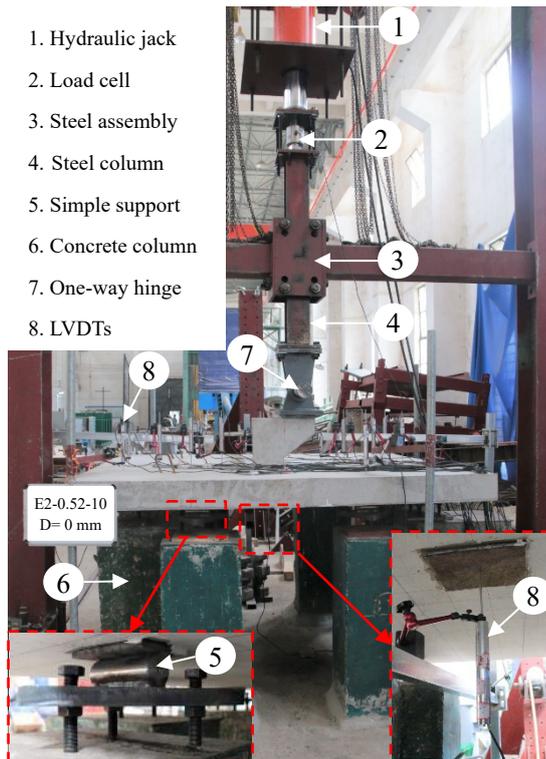
Note:  $V_{test}$  is the measured punching shear strength;  $V_{th}$  is the punching shear strength predicted by the codes; SD is the standard deviation; COV is the coefficient of variation.

593

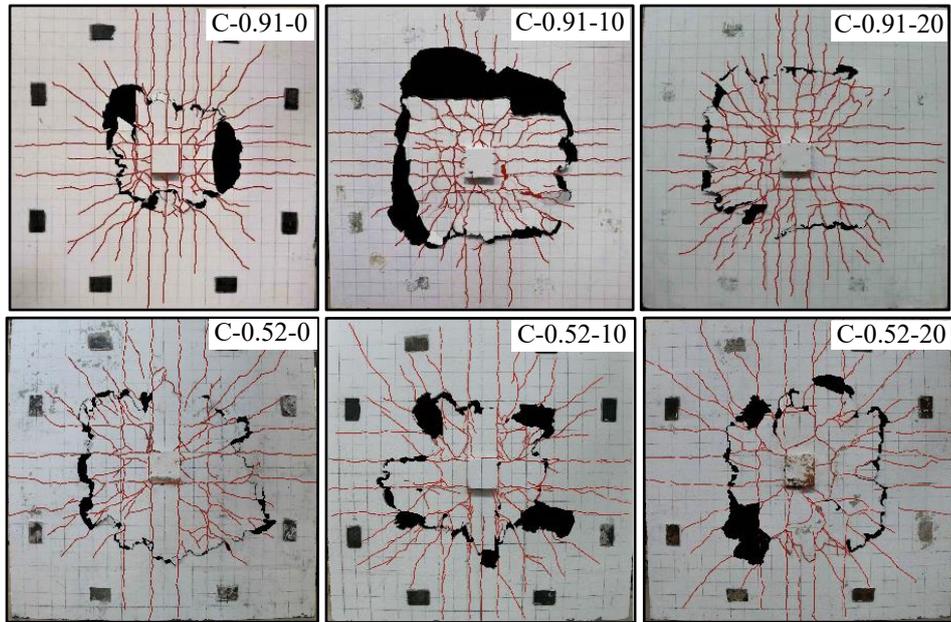


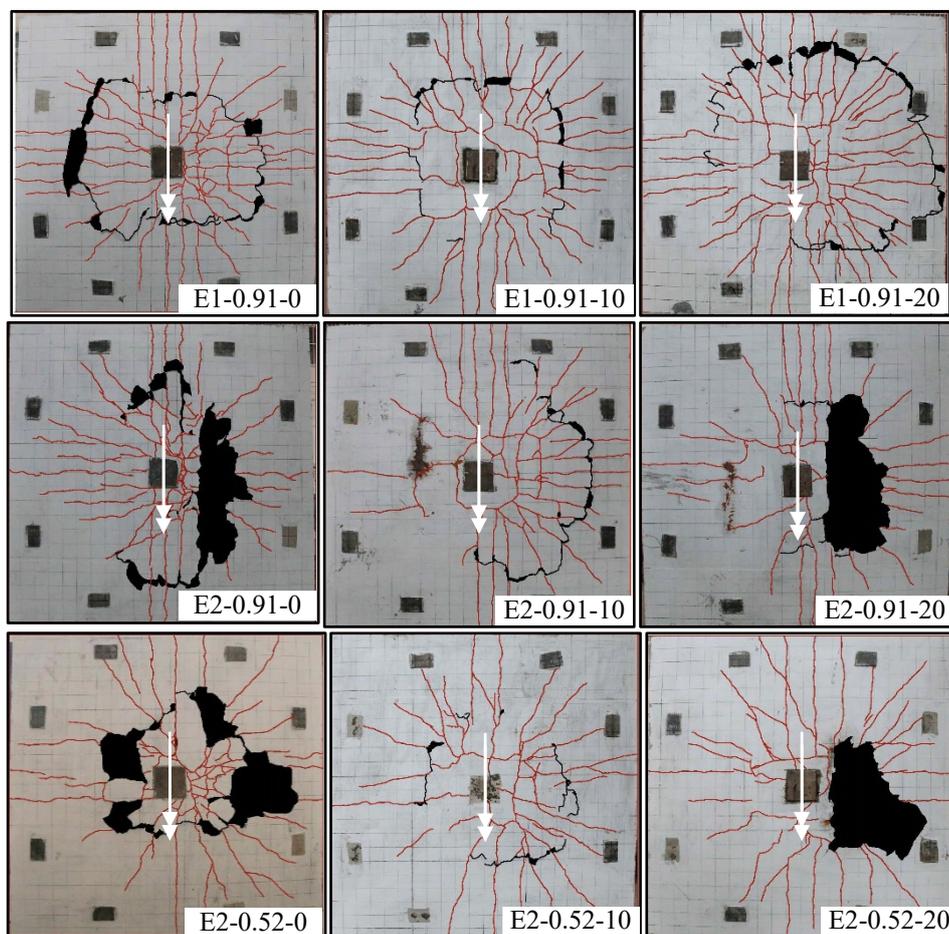
**Fig. 1.** Dimensions and reinforcement details

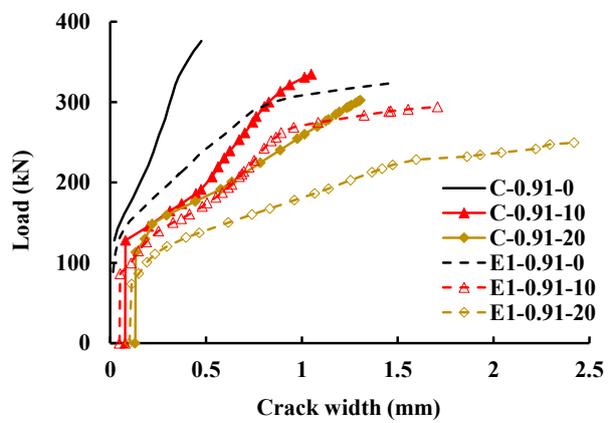




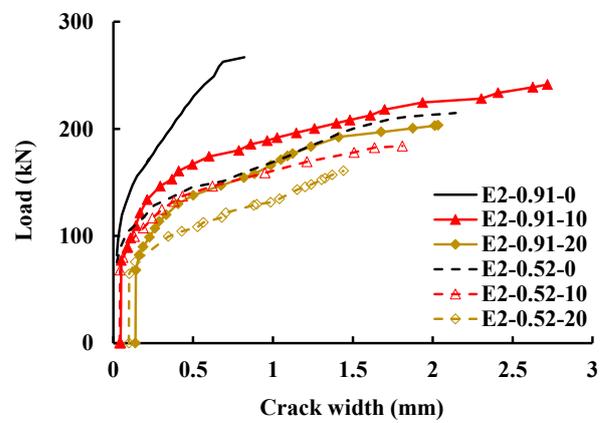




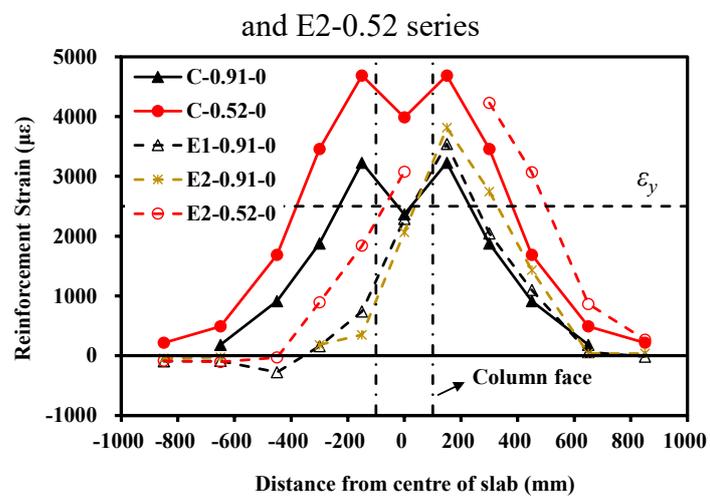


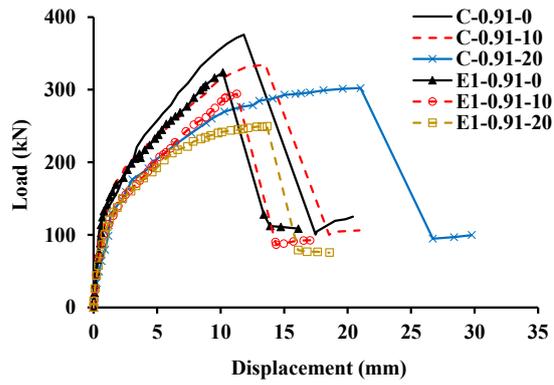


(a)

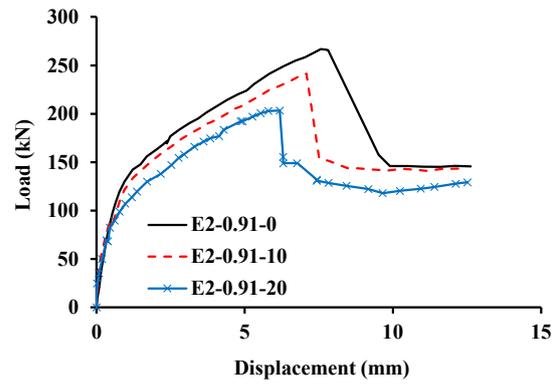


(b)

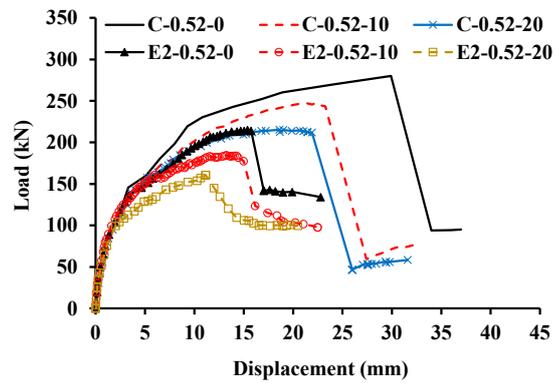




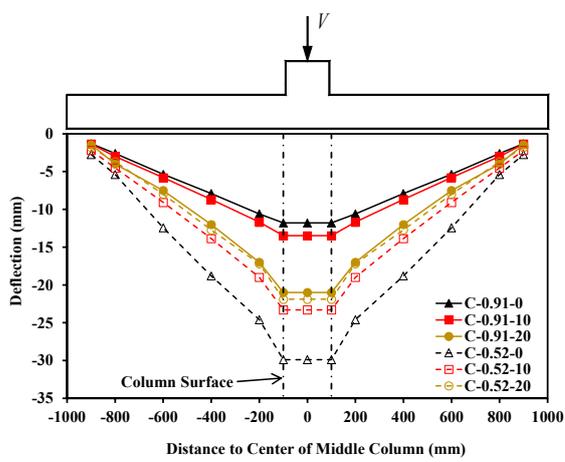
(a)



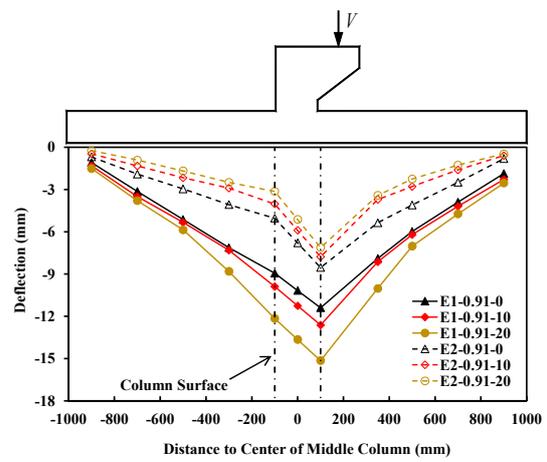
(b)



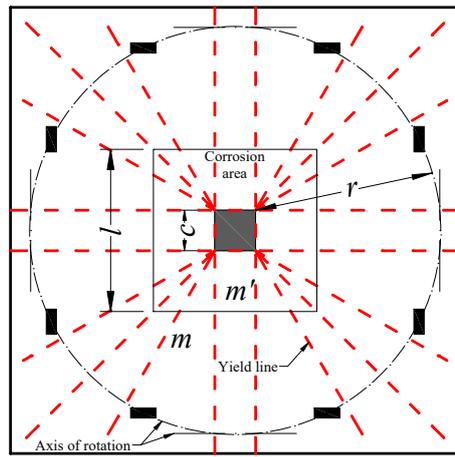
(c)



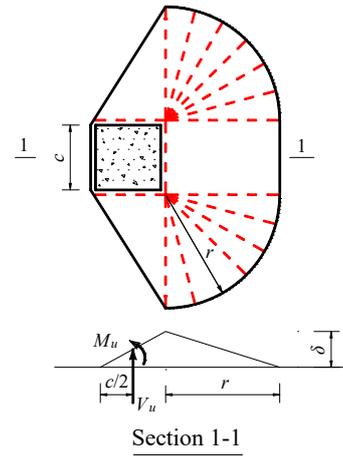
(a)



(b)



(a)



(b)