1   **Effects of High Strength Concrete on Progressive Collapse Resistance of**

2   **Reinforced Concrete Frame**

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4   **ABSTRACT**

5   The application of extreme loads such as impact and blast may lead to progressive collapse and the

6   robustness of a structure must be considered in this context. Although extensive studies had been

7   carried out over the past decades to study the load resisting mechanism of reinforced concrete (RC)

8   frames to prevent progressive collapse, the effects of high-strength-concrete (HSC) on progressive

9   collapse resistance capacity is still unclear. Therefore, six tests of RC frames with different span-to-

10   depth ratio and concrete strength were conducted in present study. Among them, three are HSC frames

11   and the remaining are normal strength concrete frames. It was found that the use of HSC could further

12   enhance the compressive arch action (CAA) capacity, especially for those with low span-to-depth ratio.

13   On the other hand, HSC can reduce the tensile catenary action (TCA) capacity at large deformation

14   stage, primarily because of higher bond stress between concrete and rebar, leading to earlier fracture of

15   the rebar. The analytical results from the model were compared with the test results. It was found that

16   the refined CAA model could accurately predict the CAA capacity of NSC frames, but not for HSC

17   frames. Moreover, existing model is hard to predict the CAA capacity of the frames with relatively

18   small span-to-depth ratio (less than 7) accurately.

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20   **CE Database subject heading:** progressive collapse; high strength concrete; compressive arch action;

21   tensile catenary action

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32 **INTRODUCTION**

33 Buildings may subject to initial local damage due to intended or accidental events, such as the loss of

34 one or a couple of columns. However, in ordinary civilian building design, the column missing is not

35 well considered in the past design guidelines. Therefore, these buildings may have high risk to

36 propagate initial local damage disproportionately to a large area of the building or even cause entire

37 collapse. The terminology of progressive collapse is first proposed after the collapse of Ronan Point in

38 1968. The collapse of Murrah Federal Building in 1995 and Twin-Tower of World Trade Center in

39 2001 re-sparkled the interest on progressive collapse in academic and practical engineer's communities.

40 Several design codes or guidelines (BS8110 1997; BSI 2006; GSA 2009; ASCE/SEI 7 2010; DoD

41 2009; ACI-318 2014) were issued for progressive collapse design using so-called explicitly or

42 implicitly design methods. Among them, Alternate Load Path method is commonly accepted for

43 evaluation of the capacity of a building to mitigate progressive collapse due to its threat independent

44 feature.

45 Based on Alternate Load Path method, extensive tests had been carried out in the past decades to

46 understand the capacity of reinforced concrete (RC) frames to resist progressive collapse. These tests

47 could be categorized into three groups: multi-story tests (Yi et al. 2008; Sasani et al. 2011a; Xiao et al.

48 2015; Qian and Li 2017; Qian et al. 2019), single-story beam-column or beam-column-slab

49 subassembly tests (Su et al. 2009; Orton et al. 2009; Qian and Li 2012a; FarhangVesali et al. 2013; Yu

50 and Tan 2013a; Lew et al. 2014; Valipour et al. 2015a; Qian et al. 2016; Ren et al. 2016; Peng et al.

51 2017; Qian et al. 2018), and single-story beam-column connections tests (Qian and Li 2012b; Yu et al.

52 2014). Yi et al. (2008) carried out a 1/3-scaled three-story planar frame test to evaluate the load

53 resisting mechanism of RC frame subjected to the loss of an interior column. Compressive arch action

54 (CAA) and tensile catenary action (TCA) were found to be the primary mechanisms in resisting

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55 progressive collapse at different stages. Sasani et al. (2011a) conducted a real time removal test to

56 study the dynamic response of an 11-story building, which was planned to be demolished, subjected to

57 sudden removal of four adjacent ground columns due to explosives. Flexural beam action and

58 Vierendeel action were identified as the two primary load resisting mechanisms. Xiao et al. (2015)

59 experimentally investigated the dynamic response of a half-scaled three-story RC building, which is

60 deliberately built for progressive collapse study, subjected to different column missing scenarios. The

61 load resisting mechanism shifted from flexural moment resisting to TCA mechanism was observed

62 when two ground exterior columns were removed simultaneously. Qian and Li (2017) tested a series of

63 six three-story frames with or without infilled walls to quantify the effects of masonry infilled walls on

64 load resisting mechanism and capacity of RC frames to resist progressive collapse. It was found that

65 masonry infilled walls enhance the initial stiffness and increase the first peak load significantly.

66 Moreover, the crushing of masonry infilled walls will not jeopardize the development of TCA of the

67 beam at large deformation stage. Qian et al. (2019) also tested another series of five three-story frames

68 to quantify the efficiency of using steel bracings in strengthening RC frames to mitigate progressive

69 collapse. Different configurations of steel bracings were applied. It was found that compressive

70 bracings prone to out-of-plane buckling and have little contribution to the collapse resistance, while

71 tensile bracings may fracture before the development of TCA.

72   Actually, majority of existing tests on progressive collapse investigation were focused on beam-

73 column substructures or beam-column-slab substructures. This is because it is easier to replicate the

74 boundary conditions and measure the response. Dynamic effects and dynamic load increase factor of

75 RC frames subjected to sudden column removal scenario were also investigated (Qian and Li 2012b;

76 Yu et al. 2014; Peng et al. 2017). These literatures documented that the failure mode and resistance of

77 the specimens were similar to their counterparts tested in a static test manner. Moreover, the behavior

78 of beam-column connections subjected to different column missing scenarios were evaluated

79 experimentally by Yap and Li (2011) and Qian and Li (2012c), which could provide sufficient

80 evidence for the level of confidence in simplification of the boundary conditions in substructure tests.

81   The load resisting mechanisms of bare RC frames subjected to middle column missing scenario

82   were quantified by pushdown test methods (Su et al. 2009; FarhangVesali et al. 2013; Valipour et al.

83   2015a). Su et al. (2009) concluded that loading rate has little effect on CAA capacity. FarhangVesali et

84   al. (2013) reported that longitudinal reinforcement ratio and stirrup configuration have a minor effect

85   on the CAA. Valipour et al. (2015a) experimentally investigated the effects of concrete strength

86   (ranging from 18 MPa to 67 MPa) on the CAA of RC beam assemblages. The test results demonstrated

87   that the concrete strength has significant influence on the peak load capacity (CAA capacity) of the

88   tested specimens. The stiffness of supports also has significant effects on mobilization of CAA.

89   Valipour et al. (2015b) filled knowledge gap in progressive collapse response of RC frame using steel

90   fiber to replace conventional transverse reinforcements, the test results demonstrated that the

91   replacement had little effects on the development of TCA. The role of slabs, compressive membrane

92   action (CMA) and tensile membrane action (TMA) developed in RC slabs were evaluated (Qian and

93   Li 2012a; Qian et al. 2016; Ren et al. 2016). It was found that, the CMA and TMA bring great benefit

94   to the resistance. The CMA capacity was affected by the stiffness of boundary elements and strength

95   of concrete significantly while the TMA capacity was mainly affected by the amount of slab

96   reinforcement in bottom layer (continual). Moreover, improving CMA of precast concrete slabs to

97   resist wheel loading using additional transverse confining system (i.e., straps, cross-bracing and a

98   combination of straps and cross-bracing) was reported by Valipour et al. (2015c). It was found that the

99   peak load capacity could be enhanced significantly due to considerable restraint provided by the

100   confining system. Furthermore, the effects of seismic design and detailing on behavior of RC moment

101   frames to resist progressive collapse were evaluated (Choi and Kim 2011, Qian and Li 2012c, Kim and

102   Choi 2016, Lu et al. 2017). Choi and Kim (2011) and Kim and Choi (2016) indicated that seismically

103   designed specimens performed much better than the corresponding non-seismically designed

104   specimens as seismically designed specimens had higher reinforcement ratio and transverse

105   reinforcement installed at joint zones, which delayed the failure of exterior joints. Lu et al. (2017)

106   found that for normal strength concrete frames, seismically design could increase the beam

longitudinal reinforcement ratio, which resulted in a much larger resistance in both beam and catenary action. However, the increase of beam depth could improve the resistance of beam action but not the catenary action. Moreover, the results from Kim et al. (2011) indicated that rotational friction damper, which was normally for mitigating seismic or wind load, was also effectively improve the behavior of RC frames to mitigate progressive collapse.

Although above studies had deeply improved the understanding on load resisting mechanisms of RC frames to resist progressive collapse, these studies are mainly focused on normal strength concrete (NSC). As high strength concrete (HSC) has advantages in load resisting capacity enhancement, smaller member size, less self-weight etc, HSC is widely used in high-rise buildings in the past decades. Moreover, the high-rise buildings have higher possibility for terrorism attacks due to their higher social impact caused by attacks. Thus, it is necessary to evaluate the behavior of reinforced HSC frames to resist progressive collapse and to identify the effects of HSC on load resisting mechanism of RC frames. For this purpose, a series of six RC frames, using both HSC and NSC, were designed and tested under pushdown loading regime. The accuracy of existing analytical models in predicting CAA and TCA of HSC frames was also evaluated.

DESCRIPTION OF TEST PROGRAM

Experimental specimens

Six half-scaled beam-column sub-assemblages were designed and constructed to evaluate the effects of HSC on behavior of RC frames to resist progressive collapse. These specimens include three HSC specimens (HSC-13, HSC-11, and HSC-8) and three NSC specimens (NSC-13, NSC-11, and NSC-8). The specimens are denoted flows below conventions:

1. 'HSC' represents specimens using HSC and 'NSC' represents specimens using NSC;

2. Number after hyphen denotes span/depth ratio, which is defined by the ratio of clear beam span to its depth.

5

131   Fig. 1 shows the dimension and reinforcement details of specimen NSC-11 while Table 1 lists the key

132   properties of the specimens. As shown in Fig. 1, Specimen NSC-11 was non-seismically designed in

133   accordance with ACI 318-14 (2014) with clear span of 2750 mm and beam cross-section of 250

134   mm×150 mm. The bottom rebar is continuous 2T12 reinforcement, while curtailment is considered for

135   top rebar. The beam transverse reinforcement is R6@100 mm throughout the whole beam without

136   transverse reinforcements in the joint zone. The clear cover of the concrete for beam and column are

137   both 15 mm. T12 and R6 herein represent deformed reinforcement with diameter of 12 mm and plain

138   reinforcement with diameter of 6 mm, respectively. Two beams, one middle column stub, and two

139   enlarged side column stubs were casted. The enlarged side column has dimension of 400 mm×400 mm

140   to replicate fixed boundary conditions following previous studies (Orton et al. 2009; Su et al. 2009; Yu

141   and Tan 2013a).

142   As tabulated in Table 1, Specimens NSC-13 and NSC-8 have similar reinforcement ratio and

143   beam cross-section to Specimen NSC-11 but clear span of 3250 mm and 2000 mm, respectively.

144   Specimens HSC-13, HSC-11, and HSC-8 have identical dimensions and reinforcement details to NSC

145   counterparts but high strength concrete is used. According to cylindrical compression tests, at the day

146   of test, the recorded concrete compressive strength of NSC-13, NSC-11, NSC-8, HSC-13, HSC-11,

147   and HSC-8 are 30.5 MPa, 31.1 MPa, and 31.7 MPa, 59.3 MPa, 61.2 MPa, and 60.5 MPa, respectively.

148   Based on tensile splitting tests, the tensile strength of the concrete of NSC-13, NSC-11, NSC-8, HSC-

149   13, HSC-11, and HSC-8 are 2.9 MPa, 3.0 MPa, 2.9 MPa, 6.0 MPa, 6.1 MPa, and 6.1 MPa, respectively.

150   Moreover, the properties of reinforcement are tabulated in Table 2.

151   **Test Setup and instrumentations**

152   Similar to previous studies (Orton et al. 2009; Su et al. 2009; Yu and Tan 2013a), as shown in Fig. 2a,

153   fixed boundary condition was replicated at the side column by using two rollers and one bottom pin.

154   To eliminate the redundant horizontal restraints from the bottom pin, a series of steel rollers were

155   placed below the pin support. Therefore, the side columns were statically determinate and the

horizontal and vertical reaction force could be measured directly. It is intentionally designed with no middle column at ground level due to desired element removal before applying vertical load. The column removal effect is implemented through a hydraulic jack with a downward stroke of 700 mm. Displacement-controlled method was adopted with a rate of 0.5 mm/s throughout the tests. To prevent out-of-plane failure, a specially designed steel assembly was installed below the hydraulic jack. As illustrated in Fig. 2b, two load cells were installed above and below the hydraulic jack to measure the vertical load (average value was used for final test results records hereafter). In addition, load cell was installed below each pin support to monitor the load redistribution of the columns. Tension/compression load cell (Item 5 in Fig. 2b) was installed in each horizontal roller to measure the horizontal reaction force. A series of linear variable displacement transducers (LVDTs) were installed along the beam (D1 to D7) to monitor the deformation shape during test. LVDTs (H1 and H2) were also installed horizontally at the side columns to determine the stiffness of the horizontal restraints as gap allowance was inevitable when installation of the appliance. Strain gauges were mounted along the length of beam longitudinal reinforcements before casting.

EXPERIMENTAL RESULTS

General behavior

NSC-series: Fig. 3a shows the vertical load-displacement curve of NSC-series specimens and Fig. 4a shows the development of crack pattern of NSC-11. For NSC-11, first crack occurred at the beam ends when the middle joint displacement (MJD) reached 9 mm. When the MJD reached 36 mm, the yield load of 37 kN was obtained. However, the calculated yield strength due to pure bending resistance was 35 kN, which was less than the measured one. This was mainly because of the inherent compressive axial force in reality is not taken into consideration in the analytical model. Further increasing the MJD, the CAA capacity of 52 kN was observed at an MJD of 90 mm, which is called peak displacement in this study. As shown in Fig. 4a, at this loading stage, concrete crushing was observed at the beam ends. The ratio of CAA capacity to yield load is about 1.41, which is due to

7

181 strain hardening of reinforcements and the mobilization of CAA. After that, the load resistance began

182 to drop gradually due to concrete crushing and second-order effects. However, the load resistance

183 began to re-ascend when the MJD reached 288 mm (about $0.1l_n$) due to the start of TCA. As shown in

184 Fig. 4a, penetrated cracks occurred at this stage. Further increasing displacement, more penetrated

185 cracks were observed which were uniformly distributed along the beam length. The drop of load

186 resistance was due to fracture of bottom rebar in the region of the beam-middle column interface. The

187 TCA capacity of 94 kN was obtained at an MJD of 712 mm. After that, the load resistance suddenly

188 dropped significantly because the complete fracture of the top rebar near the beam-middle column

189 joint. Fig. 5 shows the failure mode of NSC-11. As shown in the figure, severe concrete crushing

190 occurred at the beam ends while rebar fracture occurred primarily at beam end near middle joint region.

191 Penetrated cracks were uniformly distributed along the beam.

192 For NSC-13 and NSC-8, similar crack pattern and global behavior were observed. The yield load

193 of NSC-13 and NSC-8 was 33 kN and 53 kN, respectively. The calculated yield load of NSC-13 and

194 NSC-8 was 30 kN and 48 kN, respectively based on the analytical model. Similarly, the calculated

195 yield load is less than the measured one, which is primarily due to ignorance of compressive axial

196 force. For NSC-13, the CAA capacity of 43 kN was measured at an MJD of 108 mm. However, for

197 NSC-8, the CAA capacity was 77 kN, which was about 179 % and 148 % of that of NSC-13 and NSC-

198 11, respectively. Moreover, the TCA capacity of NSC-13 and NSC-8 was 81 kN and 88 kN,

199 respectively whereas the deformation capacity of NSC-13 and NSC-8 was 731 mm and 581 mm,

200 respectively. Although the TCA capacity of NSC-13 was less than that of NSC-11 and similar

201 deformation capacity was measured for them as shown in Fig. 3a. The test of NSC-13 was forced to

202 stop due to limited stroke capacity of the jack, rather than the failure of the specimen. If the jack had

203 larger stroke capacity, the deformation capacity and TCA capacity of NSC-13 would have been larger.

204 Figs. 6 and 7 show the failure modes of NSC-13 and NSC-8. In general, the failure mode of NSC-13

205 was similar to that of NSC-11. However, different to NSC-11 and NSC-13, the diagonal shear cracks

along the beams of NSC-8 were observed, rather than flexural cracks perpendicular to the beam axis. This indicated the shear failure in this test.

HSC-series: Fig. 3b shows the vertical load-middle joint displacement curve of HSC-series specimens. For HSC-11, first cracks occurred at the beam ends when the MJD reached 15 mm. At an MJD of 28 mm, yield load of 42 kN, which was 114 % of that of NSC-11, was obtained. Further increasing MJD to 74 mm, the CAA capacity of 60 kN, which was 115 % of that of NSC-11, was achieved. The TCA capacity and deformation capacity of HSC-11 were 80 kN and 663 mm respectively, less than these of NSC-11. The smaller deformation capacity in HSC-11 is mainly due to high strength concrete resulted in high bond strength between reinforcement and concrete, which led to stress concentration and rebar fracture in the tests. Fig. 4b shows the crack pattern of HSC-11. Compared to NSC-11, it can be found that the high strength concrete has little effects on crack development. The failure mode of HSC-11 is shown in Fig. 8, which is similar to that of NSC-11. As shown in Fig. 3b and Table 3, due to larger span/depth ratio, HSC-13 only achieved yield load and CAA capacity about 86 % and 80 % of these of HSC-11. Similar to normal strength concrete, HSC-13 experienced larger deformation. The lower TCA capacity of HSC-13 was caused by the insufficient stroke capacity during the tests. On the contrast, for HSC-8, its yield load capacity and CAA capacity were 133 % and 152 % of these of HSC-11. Different to rest specimens, the TCA capacity of HSC-8 is less than its CAA capacity, which will be further discussed in analytical section of this paper. The failure mode of HSC-13 and HSC-8 are illustrated in Figs. 9 and 10, respectively. For HSC-13, only bottom rebar near the middle joint were fractured. For HSC-8, both bottom and top rebar near the middle joint were fractured.

Horizontal reaction

The horizontal reaction force v.s. middle joint displacement curves are shown in Fig. 11. As shown in Fig. 11a, the horizontal compressive force increased with the increase of vertical displacement. For NSC-11, the horizontal compressive force was -70 kN at yield displacement, which explains the reason

231   that the calculated yield load is less than the measured one. The maximum horizontal compressive

232   force was -178 kN at an MJD of 180 mm, which was greater than the corresponding peak displacement.

233   Then, the horizontal compressive force began to decline with further increase of the displacement. The

234   horizontal compressive force transferred to horizontal tensile force after the MJD of 356 mm. The

235   maximum horizontal tensile force of 154 kN was measured at the MJD of 699 mm. Similar behavior

236   was observed for NSC-13 and NSC-8. The maximum horizontal compressive force of NSC-13 and

237   NSC-8 were -153 kN and -202 kN, respectively. Thus, when span/depth ratio reduced from 11 to 8, the

238   maximum horizontal compressive force increased by 13.4 %. Conversely, increasing the span/depth

239   ratio from 11 to 13, the maximum horizontal compressive force decreased by over 14.0 %. Moreover,

240   the maximum horizontal tensile force of NSC-13 and NSC-8 were 148 kN and 147 kN, respectively.

241   Thus, span/depth ratio will not affect the development of horizontal tensile force.

242   As shown in Fig. 11b, the maximum horizontal compressive force of HSC-11, HSC-13, and HSC-

243   8 were -259 kN, -233 kN, and -321 kN, respectively. Thus, when span/depth ratio decreased from 11

244   to 8, the maximum horizontal compressive force was increased by 23.9 %, which was greater than that

245   of the NSC specimens. For the maximum horizontal tensile force, similar to NSC specimens, the

246   span/depth ratio will not affect it significantly.

247   **Deflection shape of beams**

248   Fig. 12 shows the beam deflection shape of NSC-11 in accordance with different critical stages: yield

249   load capacity, CAA capacity, onset of TCA, fracture of rebar, and ultimate deformation. As shown in

250   the figure, from the beginning of the test, the beams exhibit double-curvature deflection shape. Before

251   fracture of the first rebar near the middle joint, the beams' deformation was almost symmetric. Then,

252   the middle joint continued to rotate and the damage prone to be concentrated in the left side of the

253   middle joint due to the weld failure between the top of the middle stub and the steel column, which

254   released the rotational restraints at the middle joint. Moreover, at the final stage of test, the chord

255   rotation, which is defined as ratio of MJD to beam span, was compared with the beam deformation

256　shape. It can be seen that the chord rotation will over-estimate the actual end rotation of the beam end

257　near the side columns while it could estimate the rotation of the beam end near the middle joint well.

258　For other specimens, similar results were observed.

259　**Strain gauge results**

260　Figs. 13a and b show the variation of strain gauge readings along beam top and bottom longitudinal

261　reinforcements of NSC-11, respectively. As shown in the figure, the bottom reinforcement near the

262　middle joint was first yielded. At CAA stage, plastic hinges were formed at both beam ends. However,

263　the compressive strain in both top and bottom rebar began to decline after onset of the TCA stage. At

264　ultimate load stage, no compressive strain was measured at both top and bottom beam longitudinal

265　reinforcement. As shown in Fig. 14, the strain variation of HSC-11 was quite similar to that of NSC-11.

266　However, as shown in Fig. 15, at ultimate load stage, considerable compressive strain was still

267　measured at bottom reinforcement of HSC-8. This could be explained as the high bond between

268　concrete and rebar as well as low span-depth ratio resulted in earlier fracture of longitudinal rebar and

269　delayed the development of tensile strain in rebar.

270　**ANALYSIS AND DISCUSSIONS**

271　**Dynamic response of tested specimens**

272　As progressive collapse is a dynamic event due to the sudden column removal, it was worthwhile to

273　evaluate the dynamic capacity of test specimens. Based on the investigation from Qian and Li (2015a,

274　b) and Tsai (2010), an energy-based simplified single-degree-of-freedom (SDOF) model, first

275　proposed by Izzuddin et al. (2008), is accurate for dynamic assessment. Thus, in this study, the energy-

276　based model was utilized to assess the dynamic capacity of specimens based on the measured quasi-

277　static load-displacement curves from the tests. The mathematic equations were expressed as:

278
$$P_d(u_d) = \frac{1}{u_d} \int_0^{u_d} P_{NS}(u)\,du \qquad (1)$$

279 where $P_d(u)$ and $P_{NS}(u)$ are the dynamic capacity and the nonlinear static loading estimated at the

280 displacement demand u, respectively.

281   Fig. 16 shows the dynamic response curves of tested specimens. As shown in the figure, the

282 dynamic ultimate capacity of NSC-13, HSC-13, NSC-11, HSC-11, NSC-8, and HSC-8 were 44 kN, 43

283 kN, 53 kN, 53 kN, 64 kN, and 78 kN, respectively. Thus, the higher strength concrete has little effects

284 on dynamic ultimate capacity of the specimens with moderate or large span/depth ratio. This is

285 primarily because TCA governs the failure. However, for specimens with small span/depth ratio, high

286 strength concrete could increase the dynamic ultimate capacity significantly as CAA governs the load.

287 **De-composition of the load resistance contribution from axial force and bending moment**

288 To de-composite the resistance contribution from the axial force and bending moments, a series of

289 analyses were carried out. As shown in Fig. 17, only left bay was extracted for analysis due to

290 symmetry. The load resistance P could be determined as the summation of the vertical components of

291 the shear force (V) and axial force (N) at the middle joint when the MJD was δ.

292 $$P = (N sin\theta + V \cos\theta) \tag{2}$$

293 where θ is the rotation of the beam end near the middle joint and can be determined by the vertical

294 displacements ($\theta = \arctan\left(\dfrac{4(D_4 - D_3)}{l}\right)$); D_3 is the vertical displacement measured at the position

295 with $l/4$ from the middle joint, and D_4 is the MJD; l is beam span length.

296   As shown in Fig. 17, N and V could be determined by the measured horizontal and vertical

297 reaction force at the supports:

298 $$N = (F_L \tan\theta + H_t + H_b)\cos\theta \tag{3}$$

299 $$V = (F_L - N \sin\theta)/\cos\theta \tag{4}$$

Therefore, the bending moment at the beam end near the middle column (M_M) and the one near the side column (M_S) could be expressed as:

$$M_M = F_L l - H_t(\delta + 0.35) - H_b(\delta - 0.35) \qquad (5)$$

$$M_S = 0.2 F_L - 0.35 H_t + 0.35 H_b \qquad (6)$$

where H_t and H_b are the horizontal reaction force at the upper roller and bottom roller, respectively; F_L is the vertical reaction force at the left side column.

NSC-11, HSC-11, NSC-8 and HSC-8 were selected to show the de-composition of load resistance, as shown in Fig. 18. As shown in the figure, initially the contribution from axial force is negative due to development of compressive force at CAA stage. At this stage, the load resistance mainly attributed to the shear force. When tensile force mobilized at TCA stage, the contribution of axial force increased with increase of MJD. However, as shown in the figure, the contribution from shear force is still significant although the contribution from shear force is decreasing with the increase of MJD. Thus, it is not correct to assume the load resistance purely provided from tension force of reinforcement at TCA stage.

Figs. 19a, b, c and d show the variation of bending moment of NSC-11, HSC-11, NSC-8 and HSC-8, respectively. As shown in the figure, the bending moments were much larger than expected pure flexural induced bending moments due to compressive force developed in the beams. Moreover, the maximum bending moment and maximum compressive axial force achieved at the same MJD. For NSC-11, the maximum bending moments near the middle column and near the side column were 46.7 kN·m and 48.7 kN·m, respectively. Compared with NSC-11, the maximum bending moments of HSC-11 were increased by 36.2 % and 38.8 %, respectively. Similarly, compared to NSC-8, the bending moment near the middle column and side column of HSC-8 were increased by 34.3 % and 12.2 %, respectively.

323   The measured M-N curves of NSC-11, NSC-8, HSC-11, and HSC-8 were presented in Figs. 20,

324   21, 22, and 23, respectively. It was found that the M-N curves were similar to the theoretical M-N

325   curves for both NSC and HSC specimens. When the bending moment reached its maximum value, the

326   bending moment began to decrease as the axial force changed from compression to tension at large

327   deformation stage. As shown in the figure, even the axial force in tension (catenary action kicked in),

328   the M-N curves agreed with the theoretical ones well.

329   **Assessment of the accuracy of existing CAA models**

330   As a favorable alternate load path to resist progressive collapse due to its low demand in

331   deformation, CAA has been widely studied theoretically. Based on plastic theory, Park and Gamble

332   (2000) proposed a classical model to calculate the CMA in RC slabs. The Park and Gamble (2000)'s

333   model can be further used to predict CAA capacity in RC beam as the CMA and CAA share similar

334   merits. As shown in Fig. 24, the CAA capacity P can be given as:

335

$$P_{CAA} = \frac{2(M_s + M_m - N\delta)}{\beta L} \qquad (7)$$

336   where M_s and M_m are the bending moments at the beam-column interface; N is the axis force in beam;

337   L is the total span of the double-bay beam; β is the ratio of the net span to the total span L, which is 0.5

338   here; δ is the vertical displacement in the middle column stub. After stringent derivation, which can be

339   found in Park and Gamble (2000) in detail, P can be expressed as:

340

$$\begin{aligned}
P_{CAA} = \frac{2}{\beta L}\Bigg\{ &0.85 f_c' \beta_1 hb \left[\frac{h}{2}\left(1 - \frac{\beta_1}{2}\right) + \frac{\delta}{4}(\beta_1 - 3) + \frac{\beta L^2}{4\delta}(\beta_1 - 1)\left(\varepsilon + \frac{2t}{L}\right)\right. \\
&\left. + \frac{\delta^2}{8h}\left(2 - \frac{\beta_1}{2}\right) + \frac{\beta L^2}{4h}\left(1 - \frac{\beta_1}{2}\right)\left(\varepsilon + \frac{2t}{L}\right) - \frac{\beta_1 \beta^2 L^4}{16h\delta^2}\left(\varepsilon + \frac{2t}{L}\right)^2 \right] \\
&- \frac{1}{3.4 f_c'}(T_s - T_m - T_s' + T_m')^2 + (T_s' + T_m')\left(\frac{h}{2} - a_s - \frac{\delta}{2}\right) \\
&+ (T_s + T_m)\left(h_0 - \frac{h}{2} + \frac{\delta}{2}\right)\Bigg\}
\end{aligned} \qquad (8)$$

341   and

342
$$\varepsilon + \frac{2t}{L} = \frac{\left(\frac{1}{hE_c b} + \frac{2}{LK}\right)\left[0.85 f_c' \beta_1 b\left(\frac{h}{2} - \frac{\delta}{4} - \frac{T_s - T_m - T_s' + T_m'}{1.7 f_c' \beta_1 b}\right) + T_m' - T_m\right]}{1 + 0.2125 \dfrac{f_c' \beta_1 \beta L^2 b}{\delta}\left(\dfrac{1}{hE_c b} + \dfrac{2}{LK}\right)} \qquad (9)$$

343   where h and b are the beam depth and beam width, respectively; β_1 is the ratio of the depth of the

344   concrete equivalent rectangular stress block to the depth of neutral-axis; ε is the axial strain of the

345   double-bay beam; t is the axial movement of the side column stub; T_s and T_m are the steel tensile forces

346   at side beam-column interface and middle beam-column interface, respectively; T_s' and T_m' are the

347   steel compressive forces at side beam-column interface and middle beam-column interface,

348   respectively; f_c' is the concrete cylinder compression strength; h_0 is the effective depth of the beam; a_s

349   is the distance from the centroid of compressive steel to the concrete compression surface; E_c is the

350   concrete elastic modulus; K is the lateral stiffness.

351   To evaluate the accuracy of the model, 45 specimens from existing tests (Su et al.2009, Choi and

352   Kim 2011, Sasani et al. 2011b, FarhangVesali et al. 2013, Valipour et al. 2015a, Yu and Tan 2013b,

353   Yu and Tan 2014, Qian et al. 2015, Alogla et al. 2016, Ren et al. 2016) were used for assessment.

354   Table 4 presents the key parameters and analytical results. As shown in Fig. 25a, the mean value and

355   standard deviation of the ratio of measured CAA capacity to the calculated one based on Park and

356   Gamble (2000) were 1.37 and 0.38, respectively. Thus, Park and Gamble (2000)'s model may

357   underestimate the CAA significantly. Similar conclusions were found by Lu et al. (2018). To reveal

358   the reasons for this underestimation, the measured peak displacements (corresponding CAA capacity)

359   were substituted into the model. As shown in Fig. 25b, the mean value and standard deviation of the

360   ratio of the measured CAA to the calculated one were 1.10 and 0.23, respectively. Therefore, the

361   underestimation of Park's model was mainly due to improperly assumption of the peak displacement

362   as a constant value (δ=0.5h). To improve the accuracy of Park and Gamble (2000)'s model, Lu et al.

363   (2018) conducted comprehensive parametric studies based on validated finite element model (FEM). A

364   regression model of δ=0.0005L^2/h was proposed by Lu et al. (2018). The calculated peak

365   displacements of the specimens are compared with the measured ones in Table 4. As shown in the

366   table, in general, the measured displacements are larger than the calculated ones, especially for

367   specimens with span-to-depth ratio less than 7, which could be explained as the regression model was

368   mainly calculated based on specimens with larger span-to-depth ratio. As shown in Fig. 25c, relied on

369   Lu et al. (2018)'s model, the mean value and standard deviation of the ratio of the measured CAA to

370   the analytical one was 1.04 and 0.23, respectively. If only look at the specimens with span-to-depth

371   ratio less than 7, the mean value was 1.16. Therefore, the regressed equation is more favorable for

372   specimens with relatively larger span-to-depth ratio (greater than 7). Moreover, if we only look at

373   HSC-series specimens in this study, the calculated CAA capacity of HSC-8, HSC-11, and HSC-13 was

374   112 %, 114 %, and 114 % of the measured one, respectively. Thus, Lu et al. (2018)'s model may

375   considerably overestimate the CAA capacity for the frames with high strength concrete as the

376   regression model (δ=0.0005L^2/h) did not include the parameter of concrete strength.

377   **Assessment of the accuracy of existing TCA models**

378   As the last line of defense in resisting progressive collapse, TCA is undoubtedly the most

379   important mechanism to provide alternate load path. To effectively predict TCA capacity, Yi et al.

380   (2008), Su et al. (2009), and Yu and Tan (2013b) proposed simplified TCA models. In their models,

381   progressive collapse was assumed to be resisted by the tensile force in beam rebar. However, the

382   contribution of beam rebar for TCA capacity is different in different models. In Yi et al. (2008)'s

383   model, both the top and bottom rebar of beam are deemed to provide resistance. However, in Su et al.

384   (2009)'s model, only the bottom rebars are considered to provide resistance. Conversely, Yu and Tan

385   (2013b) assumed that the TCA capacity is purely provided by the top rebars. In this evaluation study,

386   the deformation capacity of each specimen is assumed to be 10% of the total span of the double-bay

387   beam, in accordance to DoD (2009). The TCA model of Yi et al. (2008), Su et al. (2009) and Yu and

388   Tan (2013b) can be expressed as Eqs. 10 -12, respectively.

389
$$P_{TCA} = 2\psi(A_{st}f_y + A_{sb}f_y^{'})\sin\alpha \qquad (10)$$

$$P_{TCA} = 2A_{sb}f_y' \sin\varphi \tag{11}$$

$$P_{TCA} = 2A_{st}f_y \sin\alpha \tag{12}$$

where A_{st} and A_{sb} are the area of top and bottom rebars, respectively; f_y and f_y' are the yield strength of top and bottom rebars, respectively; ψ is a strain adjustment coefficient, and $\psi=0.85$; α is the chord rotation of beam; φ is the angle between the connection of top rebar at the side column stub and bottom rebar at the middle column stub and the horizontal line.

A database consists of 30 specimens including the tests from literatures (Su et al. 2009, Yu and Tan 2013b, Yu and Tan 2014, Qian et al. 2015, Alogla et al. 2016, Ren et al. 2016) and tested specimens in this study was utilized to validate the reliability of the TCA models mentioned above. Fig. 26 shows the comparison of the measured TCA capacity with the calculated one. As shown in the figure, the mean ratio of the measured TCA capacity to the calculated one based on the models of Yi et al. (2008), Su et al. (2009), and Yu and Tan (2013b) was 1.06, 1.43 and 1.60, respectively. The standard deviation was 0.28, 0.42 and 0.53, respectively. Thus, among them, the model of Yi et al. (2008) gives the best prediction. The model of Su et al. (2009) neglected the contribution from top rebars resulted in conservative prediction. However, as the model of Yu et al. (2013b) assuming the bottom rebar was completely fractured, which is not in reality, the model may also underestimate the resistance of TCA significantly.

For HSC-series specimens, the mean value of the ratio of measured TCA capacity to calculate one from the models of Yi et al. (2008), Su et al. (2009), and Yu and Tan (2013b) was 0.94, 1.48 and 1.33, respectively. Therefore, different to the specimens using NSC, Yi et al. (2008)'s model overestimates the TCA capacity of the specimens using HSC slightly. However, as the test data collected from HSC specimens are very few and it is necessary to carry out further tests on HSC RC frames to further support the conclusions.

CONCLUSIONS

Based on the results of the experimental and analytical investigation presented in this paper, the following conclusions are drawn:

1. Test results indicated that for normal strength concrete frames, the CAA capacity and TCA capacity increase by 79.0 % and 8.6 %, respectively, when the span-to-depth ratio decreased from 13 to 8. For the frames with high strength concrete, the CAA capacity and TCA capacity, increase by 89.5 % and 13.9 % respectively, when the span-to-depth ratio decreased from 13 to 8. Therefore, the span-depth-ratio has significant effect on CAA capacity but not for TCA capacity.

2. Based on the test results, high strength concrete could increase the CAA capacity of the frame with span-to-depth ratio of 8, 11, and 13 by 18. 2 %, 15.4 %, and 11.6 %, respectively. Thus, high strength concrete is beneficial to enhance CAA capacity, especially for the frames with low span-to-depth ratio. However, the TCA capacity of specimen HSC-8, HSC-11, and HSC-13 only achieved 93.2 %, 87.2 %, and 88.9 % of that of NSC-8, NSC-11, and NSC-13, respectively. Thus, the specimens with high strength concrete may detriment the TCA capacity due to high bond strength between reinforcements and concrete, which prone to premature the fracture of reinforcements. However, it should be noted that non-seismically designed specimens were tested. For seismically designed and detailed specimens, more tests should be carried out on evaluation of the HSC effects.

3. Analytical evaluation indicated that Park's model will underestimate the CAA capacity significantly due to improperly assumption of the peak displacement as $0.5h$. However, the agreements could be improved for both NSC and HSC specimens significantly when the peak displacement assumes to be $0.0005L^2/h$, in accordance to the study of Lu et al. (2018). However, the model proposed by Lu et al. (2018) is more suit for RC frames with relatively larger span-to-depth ratio (larger than 7). And Lu et al. (2018)'s model may overestimate the CAA capacity of HSC-series specimens significantly due to the regression model did not included the effects of concrete strength.

18

440   4. Although Yi et al. (2008)'s model produced the best prediction for TCA capacity, it slightly

441   underestimates the TCA capacity of NSC frames but overestimate that of HSC frames.

442   Although the models proposed by Yu and Tan (2013b) and Su et al. (2009) underestimate the

443   TCA capacity, the reason was different. For Su et al. (2009), the contribution of top

444   reinforcement is ignored, which disagrees with the test observation. However, for Yu and Tan

445   (2013b), the contribution of bottom reinforcements is neglected, which is over-conservative. In

446   reality, the bottom reinforcement may not fracture completely when the deformation reached

447   10 % of the total length of the double-span beams, which is proposed by the guideline of DoD

448   (2009).

449   **FUTURE RESEARCH**

450   Based on the test results and conclusions, the future research needed was highlighted. The effects of

451   HSC on seismically designed specimens should be evaluated in the future as the conclusions from non-

452   seismically designed specimens may not be suitable for seismically designed ones. Moreover, the

453   effects of different boundary conditions (different column missing scenarios) should be quantified.

454   Furthermore, the effects of HSC on dynamic response of RC moment frame subjected to suddenly

455   column removal should be investigated.

456   **DATA AVAILABILITY**

457   Some or all data, models, or code generated or used during the study are available from the

458   corresponding author by request (data related in the measured curves, photos, etc.).

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571
572 **FIGURE CAPTIONS**
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574 **Fig. 1.** Dimension and reinforcement details of Specimen NSC-11: (a) elevation view; (b) cross

575 sections

576 **Fig. 2.** Test setup and instrumentation layout of the tests: (a) photo; (b) schematic view

577 **Fig. 3.** Vertical load-displacement history: (a) NSC-series; (b) HSC-series

578 **Fig. 4.** Crack pattern: (a) NSC-11; (b) HSC-11

579 **Fig. 5.** Failure mode of Specimen NSC-11

580 **Fig. 6.** Failure mode of Specimen NSC-13

581 **Fig. 7.** Failure mode of Specimen NSC-8

582 **Fig. 8.** Failure mode of Specimen HSC-11

583 **Fig. 9.** Failure mode of Specimen HSC-13

584 **Fig. 10.** Failure mode of Specimen HSC-8

585 **Fig. 11.** Horizontal reaction force-displacement curves: (a) NSC series; (b) HSC series

586 **Fig. 12.** Deformation shape of the beams of Specimen NSC-11 at various stages

587 **Fig. 13.** Strain gauge results of NSC-11: (a) top beam rebar; (b) bottom beam rebar

588   **Fig. 14.** Strain gauge results of HSC-11: (a) top beam rebar; (b) bottom beam rebar

589   **Fig. 15.** Strain gauge results of HSC-8: (a) top beam rebar; (b) bottom beam rebar

590   **Fig. 16.** Dynamic performance of the specimens

591   **Fig. 17.** Relationship of internal forces and the load resistance

592   **Fig. 18.** Collapse Resistance contributions from axial and shear force: (a) NSC-11; (b) HSC-11; (c)

593   NSC-8; (d) HSC-8

594   **Fig. 19.** Variations of bending moments v.s. deflections at different cross-section: (a) NSC-11; (b)

595   HSC-11; (c) NSC-8; (d) HSC-8

596   **Fig. 20.** M-N relationship at the beam end of NSC-11: (a) nearby the middle column; (b) nearby the

597   side column

598   **Fig. 21.** M-N relationship at the beam end of NSC-8: (a) nearby the middle column; (b) nearby the side

599   column

600   **Fig. 22.** M-N relationship at the beam end of HSC-11: (a) nearby the middle column; (b) nearby the

601   side column

602   **Fig. 23.** M-N relationship at the beam end of HSC-8: (a) nearby the middle column; (b) nearby the side

603   column

604   **Fig. 24.** Internal Force diagram for derivation of the analytical model of CAA

605   **Fig. 25.** Comparison of the measured CAA capacity with calculated one: (a) δ=0.5h; (b) measured δ;

606   (c) δ=0.0005L^2/h

607   **Fig. 26.** Comparison of the measured TCA capacity with calculated one: (a) Yi et al. (2008); (b) Su et

608   al. (2009); (c) Yu and Tan (2013b)

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Table 1-Specimen properties

| Test ID | Beam clear span (mm) | Beam longitudinal reinforcement | | | | Concrete |
| | | A-A section | | B-B section | | |
		Top	Bottom	Top	Bottom	
NSC-8	2000	3T12	2T12	2T12	2T12	Normal strength
NSC-11	2750	3T12	2T12	2T12	2T12	Normal strength
NSC-13	3250	3T12	2T12	2T12	2T12	Normal strength
HSC-8	2000	3T12	2T12	2T12	2T12	High strength
HSC-11	2750	3T12	2T12	2T12	2T12	High strength
HSC-13	3250	3T12	2T12	2T12	2T12	High strength

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Table 2-Material properties of reinforcements

Items		Nominal diameter (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Elongation (%)
Transverse reinforcement	R6	6	348	486	25.4
Longitudinal	T12	12	438	577	16.6
Reinforcements	T16	16	442	605	16.0

647 Note: R6 represents plain bar of with diameter of 6 mm; T12 and T16 represent deformed rebar with diameter of 12 mm and 16 mm, respectively.
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Table 3-Test results

Test ID	Critical displacements (mm)			Critical loads (kN)			MHCF (kN)	MHTF (kN)
	YL	CAA	TCA	YL	CAA	TCA		
NSC-8	25	79	581	53	77	88	-202	147
NSC-11	36	90	712	37	52	94	-178	154
NSC-13	45	108	731	33	43	81	-153	148
HSC-8	16	80	547	56	91	82	-321	145
HSC-11	28	74	663	42	60	80	-259	142
HSC-13	35	104	701	36	48	72	-233	150

651 Note: YL means yielding load capacity; CAA represents CAA capacity; TCA represents TCA capacity; MHCF means maximum horizontal compressive
652 force; and MHTF means maximum horizontal tensile force.
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Table 4-Summary of key parameters and analytical results of CAA

Test	ID	Lateral stiffness (kN/m)	Total length (mm)	Span to depth ratio	Beam section h×b (mm×mm)	Beam rebar Top (mm)	Beam rebar Bottom (mm)	Material properties f_c' (MPa)	Material properties f_y (MPa)	Test results Δ (mm)	Test results P (kN)	Δ_L (mm)	$P_{\delta M}$ [*] (kN)	$P_{\delta L}$ [#] (kN)	$P_{\delta p}$ [$] (kN)
This Test	NSC-8	1×10^5	4250	8.0	250×150	3Φ12	2Φ12	31.7	438	78.6	69.2	36.1	69.3	79.2	57.1
	NSC-11	1×10^5	5750	11.0	250×150	3Φ12	2Φ12	31.1	438	89.8	46.3	66.1	48.4	52.5	42.2
	NSC-13	1×10^5	6750	13.0	250×150	3Φ12	2Φ12	30.5	438	108.1	36.3	91.1	38.1	40.5	35.7
	HSC-8	1×10^5	4250	8.0	250×150	3Φ12	2Φ12	60.5	438	80.4	87.6	36.1	84.5	98.2	70.8
	HSC-11	1×10^5	5750	11.0	250×150	3Φ12	2Φ12	61.2	438	90.3	56.8	66.1	59.2	65.0	52.3
	HSC-13	1×10^5	6750	13.0	250×150	3Φ12	2Φ12	59.3	438	103.8	43.6	91.1	46.6	49.6	43.7
Su et al. (2009)	A1	1×10^6	2700	4.1	300×150	2Φ12	2Φ12	25.8	350	48.9	168.0	12.2	130.3	145.3	82.3
	A2	1×10^6	2700	4.1	300×150	3Φ12	3Φ12	28.2	350	56.4	221.0	12.2	159.5	180.3	109.9
	A3	1×10^6	2700	4.1	300×150	3Φ14	3Φ14	31.2	340	76.4	246.0	12.2	180.3	215.1	138.3
	A4	1×10^6	2700	4.1	300×150	2Φ12	1Φ14	23.0	350	65.0	147.0	12.2	104.6	126.7	68.0
	A5	1×10^6	2700	4.1	300×150	3Φ12	2Φ12	26.5	350	70.7	198.0	12.2	132.7	160.3	93.9
	A6	1×10^6	2700	4.1	300×150	3Φ14	2Φ14	28.6	340	69.2	226.0	12.2	159.5	188.0	116.8
	B1	1×10^6	4200	6.6	300×150	3Φ14	3Φ14	18.6	340	100.0	125.0	29.4	91.3	107.6	80.8
	B2	1×10^6	5700	9.1	300×150	3Φ14	3Φ14	19.3	340	102.0	82.9	54.2	64.6	75.7	60.0
	B3	1×10^6	5700	9.1	300×150	3Φ14	2Φ14	21.1	340	85.5	74.7	54.2	63.0	68.6	51.8
	C1	1×10^6	2700	6.1	200×100	2Φ12	2Φ14	15.9	350	33.7	60.9	18.2	44.5	46.7	35.7
	C2	1×10^6	2700	6.1	200×100	2Φ12	2Φ12	16.8	350	33.5	64.9	18.2	45.3	47.5	36.0
	C3	1×10^6	2700	6.1	200×100	2Φ12	2Φ12	16.3	350	28.7	68.6	18.2	45.6	47.1	35.8
Choi et al. (2011)	5S	N/A	3315	6.7	225×150	5Φ10	2Φ10	17.0	493	103.0	39.9	24.4	57.6	71.7	57.2
	5G	N/A	3325	8.2	185×150	2Φ10	2Φ10	17.0	493	84.5	22.8	29.9	30.6	48.2	30.3
	8S	N/A	3315	7.7	195×140	5Φ10	3Φ10	30.0	493	59.3	54.1	28.2	70.0	76.6	61.6
	8G	N/A	3325	9.4	160×125	2Φ10	2Φ10	30.0	493	59.0	23.7	34.5	29.4	33.4	26.3
Sasani et al. (2011)	P1	N/A	4170	10.4	190×190	5Φ9.5	2Φ9.5	41.0	516	41.0	71.8	45.8	44.9	60.7	44.1
Yu and Tan (2013b)	S1	1.06×10^5	5750	11.0	250×150	2Φ10 1Φ13	2Φ10	31.2	511	78.0	41.6	66.1	47.9	50.0	39.8
	S2	1.06×10^5	5750	11.0	250×150	3Φ10	2Φ10	31.2	511	73.0	38.4	66.1	45.7	46.9	36.7
	S3	4.29×10^5	5750	11.0	250×150	3Φ13	2Φ10	38.2	511	74.4	54.5	66.1	59.8	61.7	48.1
	S4	4.29×10^5	5750	11.0	250×150	3Φ13	2Φ13	38.2	494	81.0	63.2	66.1	64.9	68.3	54.8
	S5	4.29×10^5	5750	11.0	250×150	3Φ13	3Φ13	38.2	494	74.5	70.3	66.1	75.4	77.4	63.8
	S6	4.29×10^5	5750	11.0	250×150	3Φ16	2Φ13	38.2	494	114.4	70.3	66.1	66.8	78.0	64.4
	S7	4.29×10^5	4550	8.6	250×150	3Φ13	2Φ13	38.2	494	74.4	82.8	41.4	84.5	94.4	69.1

Reference	Specimen														
	S8	4.29×10⁵	3350	6.2	250×150	3Φ13	2Φ13	38.2	494	45.9	121.3	22.4	128.3	136.6	93.4
Yu and Tan (2014)	F1-CD	4.29×10⁵	5750	11.0	250×150	3Φ13	2Φ13	27.5	488	87.0	51.1	66.1	56.8	60.5	50.3
	F2-MR	4.29×10⁵	5750	11.0	250×150	3Φ13	2Φ13	27.5	488	51.0	62.8	66.1	63.1	60.5	50.3
FarhangVesali et al. (2013)	1	N/A	4400	11.7	180×180	2Φ10	2Φ10	30.5	620	49.0	40.5	53.8	36.9	36.0	29.4
	2	N/A	4400	11.7	180×180	2Φ10	2Φ10	27.0	620	44.0	35.7	53.8	35.7	34.4	28.5
	3	N/A	4400	11.7	180×180	2Φ10	2Φ10	30.0	620	50.0	41.4	53.8	36.5	35.8	29.3
	4	N/A	4400	11.7	180×180	3Φ10	3Φ10	26.0	620	54.0	40.1	53.8	38.6	38.7	32.9
	5	N/A	4400	11.7	180×180	3Φ10	3Φ10	29.5	620	54.0	41.6	53.8	40.3	40.3	33.9
	6	N/A	4400	11.7	180×180	3Φ10	3Φ10	30.0	620	52.0	39.4	53.8	40.9	34.1	34.0
Qian et al. (2015)	P1	N/A	4000	10.5	180×100	2Φ10	2Φ10	19.9	437	35.8	31.6	44.4	24.6	23.8	19.5
	P2	N/A	2800	9.3	140×80	2Φ10	2Φ10	20.8	437	32.9	35.5	28.0	21.0	21.4	17.7
Valipour et al. (2015a)	No. 1	N/A	4400	11.7	180×180	3Φ10	2Φ10	67.0	480	59.0	51.3	53.8	48.8	47.2	38.0
	No. 2	N/A	4400	11.7	180×180	2Φ10	2Φ10	67.0	480	54.8	42.5	53.8	46.4	43.3	34.1
	No. 3	N/A	4400	11.7	180×180	3Φ10	2Φ10	48.0	480	55.4	47.4	53.8	43.2	41.3	33.4
	No. 4	N/A	4400	11.7	180×180	2Φ10	2Φ10	48.0	480	56.3	38.5	53.8	39.1	37.4	29.5
Ren et al. (2016)	B2	N/A	4000	9.5	175×85	2Φ8 1Φ6	2Φ8	35.2	450	33.0	34	40.0	23.6	22.0	16.6
	B3	N/A	4000	10.9	200×85	2Φ8 1Φ6	2Φ8	35.2	450	33.3	41.0	45.7	30.7	29.7	20.6
Alogla et al. (2016)	SS1	N/A	5750	11.1	250×150	3Φ10	2Φ10	19.4	510	101.0	34.0	66.1	35.1	39.1	32.4
	SS2	N/A	5750	11.1	250×150	3Φ10	2Φ10	19.4	510	96.8	37.9	66.1	35.6	39.1	32.4
	SS3	N/A	5750	11.1	250×150	3Φ10	2Φ10	19.9	510	86.8	37.2	66.1	37.1	39.5	32.6
	SS4	N/A	5750	11.1	250×150	3Φ10	2Φ10	19.9	510	91.4	36.7	66.1	36.5	39.5	32.6

Note: Δ_L represents peak displacement proposed by Lu et al. (2018); $P_{\delta M}$ [*], $P_{\delta L}$ [#], and $P_{\delta p}$ [$] represent the calculated CAA capacity in accordance with the measured peak displacement, peak displacement proposed by Lu et al. (2018), and peak displacement proposed by Park and Gamble (2000), respectively.

(a)

(b)

Figure 2

Click here to access/download;Figure;Figure-2.pdf ⬇

(a)

(b)

Figure 3

(a)

(b)

Figure 4　　　　　　　　　　　　　　　　　　Click here to access/download;Figure;Figure-4.pdf ⬇

MJD=36mm (Yield Strength)

MJD=90mm (CAA Strength)

MJD=288mm (Transition Stage)

MJD=712mm (TCA Strength)

(a)

MJD=28mm (Yield Strength)

MJD=74mm (CAA Strength)

MJD=280mm (Transition Stage)

MJD=663mm (TCA Strength)

(b)

Figure 5
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Crushing

Rebar Fracture

Figure 6

Figure 7

Figure 8

Figure 9

Figure 10

Figure 11   Click here to access/download;Figure;Figure-11.pdf ⬇

(a)   (b)

Figure 12   Click here to access/download;Figure;Figure-12.pdf ⬇

Figure 13   Click here to access/download;Figure;Figure-13.pdf ⬇

(a)   (b)

Figure 14   Click here to access/download;Figure;Figure-14.pdf ±

(a)   (b)

Figure 15

(a)   (b)

Figure 16

Figure 17　　　　　　　　　　　　　　　Click here to access/download;Figure;Figure-17.pdf ⬇

Figure 18

(a)

(b)

(c)

(d)

Figure 19

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(a)

(b)

(c)

(d)

Figure 20

(a)   (b)

Figure 21

(a)

(b)

Figure 22

(a)　　　　　　　　　　　　　　　(b)

Figure 23

(a)   (b)

Figure 24

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Figure 25

(a)

(b)

(c)

(a)

(b)

(c)