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Effects of Steel Braces on Robustness of Steel Frames against Progressive

Collapse

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ABSTRACT

External installation of steel braces is one of the effective approaches to increase the lateral load resistance of the steel moment-resisting frames. However, the effects of existence of steel braces on the robustness of steel moment-resisting frames to resist progressive collapse is still not clear as little study has been carried out. To fill this gap, in this paper, six multi-story steel moment-resisting sub-frames (three bare frames and three braced frames) were fabricated and tested. Test results indicated that the specimen with reduced beam section in the connection zone performed best among three types of connections, due to the guaranteed formation of plastic hinges at the location of reduced section and avoiding brittle fracture of weld at the connection. Experimental results proved that steel braces could increase the load resisting capacity by 45.1% and 83.9% of the frame with weld connection and end plate connection, respectively. As the gusset plate restricted the rotation of the plastic hinges in the second story of the braced frames with V-shaped bracing, which decreased its deformation capacity and degraded its catenary action capacity. Actually, the ultimate load of the braced frames with V-shaped bracing is only 87.5% of that of the counterpart without any braces. As the compressive braces were severe buckled before the displacement reached 0.4% of the beam span, it has little effects on yield load but increases the initial stiffness of the bare frames. Thus, majority of the benefits of the bracing system were attributed into the tensile braces. Moreover, the analytical results evaluated the differences in load resistance and development of load resisting mechanisms in different stories. Furthermore, the contribution of compressive and tensile braces was de-composed individually by analytical analysis.

CE Database subject heading: progressive collapse; steel moment-resisting frame; braces; experimental, robustness

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35

36 INTRODUCTION

37 Progressive collapse of steel frame structures may be triggered by the failure of one or couple
38 vertical load bearing members in unintentional or intentional events. Majority of building codes only
39 provide general recommendations, rather than detailed design provisions for progressive collapse.
40 Recently, step by step design provisions were addressed by DoD (2010) and GSA (2013). Two major
41 design methods are recommended: direct or indirect design methods. Among them, the alternate load
42 path (ALP) method is commonly recommended as it is independent on the initial damage. ALP method
43 focused on the ability of structures to bridge the lost column. Based on ALP method, extensive tests
44 on steel beam-column connections or sub-assemblages were carried out by using column removal
45 assessment method (Li et al. 2007; Sadek et al. 2011; Alashker et al. 2011; Dinu et al. 2016; Dinu et
46 al. 2017; Liu et al. 2015; Tang et al. 2019; Dimopoulos et al. 2020; Qian et al. 2020; Wang et al. 2020).
47 As the damage was concentrated in the beams or beam-column connections, rather than at the columns,
48 it was recognized that the connections play a critical role in mitigating progressive collapse potential
49 of the steel frames (Lew. et al. 2013, Yang and Tan 2013a, Yang and Tan 2013b). Lee et al. (2010)
50 investigated the moment-axial force interaction of the double-span beams in steel frames under a
51 column-missing scenario. It was found that the flexural action was the dominant mechanism at
52 beginning. Due to buckling of the compressive flange and yielding of the tensile one, flexural action
53 was slowly vanished with further increase of the displacement. When the displacement exceeded 10%
54 of the beam span, the catenary action kicked-in and became the dominant mechanism in large
55 displacement stage. To investigate the key design variables that influence the formation of catenary
56 action, Khandelwal and El-Tawil (2007) carried out the numerical study of two steel joints subjected
57 to the loss of a center column. Similar to the ones investigated by Khandelwal and El-Tawil (2007),
58 Sadek et al. (2011, 2013) conducted the experimental study to provide insight into the behavior and

59 failure modes of steel beam-column connections, including the development of catenary action. Yang
60 and Tan (2013a) tested seven beam-column sub-assemblages with different semi-rigid connections. It
61 was concluded that the rotational capacity of the beam-to-column connections controlled the catenary
62 action capacity. In summary, the connections in steel moment frames can be either rigid or semi-rigid
63 connections. The use of rigid connections is the primary option for frames with medium or high
64 ductility demands in earthquake design. However, semi-rigid connections are also allowed (AISC
65 2005a). Therefore, it is of interest to evaluate whether rigid or semi-rigid connections can provide
66 sufficient strength and ductility to resist progressive collapse.

67 Catenary action, as the second defense line, is possible to increase the load resisting capacity of
68 moment resisting frames significantly (Deng et al. 2020; Qian et al. 2021a). However, for steel
69 moment-resisting frames configured under non-seismic design principles, irregular layout or long
70 span may prone to progressive collapse. Pantidis and Gerasimidis (2017, 2018) pointed out that two
71 possible collapse mechanisms caused by the loss of a column scenario, the yielding type (beam
72 plasticization) or the stability (column buckling) failure mode. Other modes of collapse could be the
73 shear failure of the beam-to-column connections (Khandelwal and El-Tawil 2011). In recent years,
74 progressive collapse events were occurred continuously, such as the collapse of twin towers in World
75 Trade Center in New York, collapse of steel frame at Argyle High School in Texas, or the collapse of
76 steel frame at Jiaxin Hotel in Quanzhou, China. Thus, it was necessary to look for additional load
77 resistance to upgrade the ability of steel frames to mitigate progressive collapse. Galal and El-Sawy
78 (2010) discussed the enhancement of three retrofit strategies on mitigating progressive collapse risks.
79 The post-tensioned (PT) steel frames, are evidenced well robustness under strong earthquakes (Ricles
80 et al. 2001; Christopoulos et al. 2002; Garlock et al. 2005; Pirmoz and Liu 2016; Moradi and Alam
81 2017). Although steel bracing system was able to provide satisfactory seismic resistance of steel frames
82 (AISC 2005a; AISC 2005b), the effects of steel bracing to resist progressive collapse are still unclear.
83 The numerical analyses by Khandelwal et al. (2009) showed that seismically designed steel braced
84 frames could sustain the sudden loss of a column without collapse as steel braces improved the
85 robustness significantly. However, little tests had done in this area. Chen et al. (2012) found that the

86 horizontal braces could upgrade the load resistance of steel frames to mitigate progressive collapse. It
87 was indicated that the horizontal braces formed additional ALP and transferred partial of the vertical
88 loads to adjacent columns directly, which could enhance the vertical load resistance significantly.

89 Additionally, previous tests were mainly based on specimens of single-story beam-column sub-
90 assemblages or connections. It is simply assumed that each story has identical geometric and material
91 properties and thus, assuming each story performed identically. However, as pointed out by Qian et al.
92 (2020), Qian et al. (2021b), and Weng et al. (2020), the load resistance of the beams in the first story
93 is quite different to the second and upper stories. It could be explained that the interaction among
94 different stories, which leads to different mobilization of load resisting mechanisms (flexural and
95 catenary action). Thus, to deep understand the behavior of steel frames subjected to column loss
96 scenarios, six two-story steel sub-frames using either rigid or semi-rigid connections are tested by push-
97 down loading regime in the present study. Based on experimental and analytical results, the effects of
98 steel braces with various configurations are quantified. The contribution of compressive and tensile
99 braces is de-composed individually. Furthermore, as mentioned above, the different behavior of the
100 structural members in different stories is also quantified.

101 **EXPERIMENTAL PROGRAM**

102 **Test specimens**

103 As shown in Fig. 1, a six-story, 6×6 bay prototype steel frame was seismically configured and
104 fabricated in accordance with AISC-341 (2005a). This prototype frame was presumed to be located on
105 a D class site (stiff soil profile), with correspondent acceleration parameters S_{DS} and S_{D1} of the design
106 response spectrums are 0.20 and 0.14, respectively. The story height of the prototype frame is 3.0 m
107 with span length of the frame in longitudinal and transverse direction was 8.4 m by 6.0 m, respectively.
108 The designed dead and live loads are 5.1 kN/m^2 and 3.0 kN/m^2 , respectively. Given the capacity
109 limitation of test facilities, only 1/2-scaled models were fabricated. To well simulate horizontal
110 restraints from the surrounding bays, beams were extended with length of 655 mm beyond the side
111 column, as shown in Fig. 2a. As described below, a horizontal chain-pole would be installed to connect
112 the overhanging beam and A-frame.

113 As listed in Table 1, the six specimens included three steel moment-resisting frames without any
114 braces (WB, EB, and RB) and three counterparts with extra bracing system (WX, EX, and RV).
115 Different connections were employed, the numeric alphabetic W, E, and R represent welded
116 connection, end plate connection, and reduced beam section connection, respectively. Additionally, the
117 numeric alphabetic X and V represent X-shaped bracing and V-shaped bracing, respectively. As shown
118 in Fig. 3, for welded connection, the flange and web of beam were welded to column flange using
119 complete joint penetration welds. For end plate connection, the beam flanges and web were connected
120 to a 10 mm thick end plate by same type of welds, while the end plates were fixed to the column flange
121 by six rows of Grade 8.8 M18 bolts. Bolts were preload of 345 N·m, which was applied by a torque
122 wrench. For reduced beam section connection, similar to welded connection, except, the beam flanges,
123 at a distance of 70 mm from the column flange, was cut in a circular manner, as shown in Fig. 3c.
124 Section HN 200×100×5.5×8 was used for beams whereas HW 150×150×7×10 was for columns.
125 Continuity plates in the column joint was designed with thickness of 10 mm.

126 The braced frames have identical dimension and detail as corresponding bare frames, except extra
127 bracing system with different configurations were installed at the second story. The braces and their
128 connections were also designed according to AISC (2005a). The braces are welded and connected to
129 the beam flanges through a gusset plate. The gusset plate was designed to be stronger than the braces
130 to avoid yielding or fracture occurring at the gusset plate (Khandelwal et al. 2009; AISC (2005a)). The
131 force acting on the weld was determined by the uniform force method (Richard 1986). Taking WX and
132 RV as an example, as shown in Fig. 2a, the brace was an angle steel with dimensions of 36 ×36 × 4
133 mm. X braces were eccentrically installed at WX and EX, the gusset plate was a steel plate with the
134 size of 330×125×12 mm. Moreover, V braces were installed at RV, the gusset plates with the size of
135 160×155×12 mm were fixed at the beams in the second story while the bottom one with size of
136 510×155×12 mm was installed at the middle span of the beams in the first story.

137 **Material properties**

138 Grade Q235 steel is used for column, beam, and angle. The measured yield strength, ultimate
139 strength, and elongation of the angle and structural components are tabulated in Table 2. As no

140 independent coupon test was conducted for bolts (Grade 8.8 M18), the yield strength and ultimate
141 strength of the bolts are provided by the supplier.

142 **Test setup**

143 As illustrated in Fig. 4, the bottom of each side column was pin supported. Each overhanging
144 beam beyond the side column was connected to the A-frame via a horizontal chain-pole (Item 6 in Fig.
145 4). The middle column in ground story was removed prior to applying concentrated load to replicate
146 the scenario of missing a middle column. The load was applied by a hydraulic jack (Item 1 in Fig. 4)
147 at the top of middle column by a displacement-controlled method. To avoid undesired out-of-plane
148 failure, a steel assembly (Item 3 in Fig. 4) was installed beneath the hydraulic jack (Item 1 in Fig. 4).
149 Moreover, hydraulic jack (Item 4 in Fig. 4) together with self-balance loading system was used to
150 simulate axial force of side column with the ratio of axial compressive force of 0.3.

151 **Instrumentations**

152 The layout of instrumentations is shown in Fig. 4. A load cell (Item 2 in Fig. 4) was installed
153 below the hydraulic jack (Item 1 in Fig. 4) to measure the applied concentrated load.
154 Tension/compression load cell (Item 7 in Fig. 4) was installed at each horizontal chain-pole (Item 6 in
155 Fig. 4) to measure its horizontal reaction force. A load pin (Item 8 in Fig. 4) was installed at each pin
156 support to measure the reaction force of the bottom pin support. In addition, a series of linear variable
157 differential transformers (LVDTs) (Item 5 in Fig. 4) were installed along the beam span or column
158 height to measure the beam deformation shape and column lateral drift, as shown in Fig. 4.

159 As shown in Fig. 2, the strain at critical sections was monitored by a series of strain gauges or
160 strain gauge rosettes. Thus, the internal force of the beam and brace could be determined by simplified
161 section analysis.

162 **EXPERIMENTAL RESULTS**

163 Six two-story sub-frames were tested to quantify the effects of steel braces to enhance robustness
164 of steel frames. The key results are listed in Table 3. As the results of WB and RB had been introduced
165 in Qian et al. (2020) in detail, only the main characteristics of these two specimens were addressed

166 herein. The different behavior of bare frames and counterpart braced frames was the main concern in
167 the present study.

168 **Global behavior**

169 **WB:** As shown in Fig. 5a, its yield load of 147.8 kN was measured when the middle column
170 displacement (MCD) reached 45 mm. Therefore, the initial stiffness, which is defined as the ratio of
171 yield load to corresponding yield displacement, was 3.3 kN/mm. At the MCD of 200 mm, WB reached
172 its ultimate load of 197.5 kN, which is defined as the peak load resistance. Afterwards, the load
173 resistance of WB began to drop slowly due to weld fracture occurred at the connections in the first
174 story. However, when MCD reached 281 mm, the load resisting capacity began to re-ascend due to the
175 catenary action developed in the second story. With the increase of the vertical displacement to 377
176 mm, the weld fracture occurred at the beam web near the middle column in the ground story. Further
177 increasing the MCD to 410 mm, the welding at the beam end near middle column in the second story
178 fractured completely. Its failure mode is shown in Fig. 6.

179 **WX:** The compressive braces of the X bracing system began to buckle at the beginning of the test,
180 which means the compressive braces may not be so effective to provide additional alternate load path
181 of the specimen, which will be further confirmed in the section of analytical analysis. However, when
182 the MCD reached 11.2 mm (0.4% of the beam span), the buckling of compressive braces become
183 severe. Thus, they have little contribution for yield load enhancement as the yield load of 233.9 kN
184 was measured at a MCD of 31 mm. This will be further confirmed by following analytical results. The
185 initial stiffness was 7.5 kN/mm, which is 227.3% of that of WB. At a MCD of 137 mm, the tensile
186 brace in the left bay fractured. When the displacement increased to 221 mm, weld fracture both
187 occurred at the connections near the side column in the ground story and the middle column in the
188 second story. Actually, the weld fracture accompanied by considerable decrease of the load resistance.
189 The load-displacement curve kept increasing until weld fracture occurred at the connection near side
190 column in the ground story at a MCD of 311 mm. WX reached its ultimate load of 286.5 kN, which
191 was 145.1% of WB, at the MCD of 382 mm as complete weld fracture occurred at the connection near
192 left side column in the ground story. However, the load resistance did not lose completely owing to the

193 catenary action developed in the second story, regardless the loss of load resistance of the entire first
194 story. Further increasing the MCD, the complete fracture occurred at the connection near the middle
195 column in the second story. The failure mode of WX is shown in Fig. 7. Fracture occurred in the tensile
196 brace. Severe out-of-plane buckling was observed in compressive braces. No yielding was observed in
197 the gusset plates. However, although the failure mode of the connections in WX is generally similar to
198 that of WB, the fracture of the connections near the side column was more severe than that near the
199 middle column.

200 **EB:** As shown in Fig. 5b, the yield load of 94.4 kN was measured at a MCD of 42 mm. Thus, the
201 initial stiffness was 2.2 kN/mm, which was only 66.7% of that of WB. However, EB exhibited more
202 flexibly than that of WB. Thus, the load resistance kept increasing until weld fracture occurred at the
203 connection in the second story at a MCD of 452 mm. EB reached its ultimate load of 255.4 kN at a
204 MCD of 515 mm. The failure mode of EB is shown in Fig. 8. Similar to WB, the weld fracture occurred
205 at Joint A.

206 **EX:** The yield load of 148.5 kN was observed at a MCD of 30 mm, which is about 157.3% of that
207 of EB. Thus, the initial stiffness was 5.0 kN/mm. The tensile braces in the right bay fractured at a MCD
208 of 320 mm. Further increasing MCD to 407 mm, the bolts in the connection near the side column
209 fractured. However, the load resistance rapidly recovered after shortly decrease. The weld fracture
210 occurred near the middle connection in the second story at a MCD of 450 mm. Subsequently, bolts in
211 the top row of this connection fractured at the MCD of 467 mm. However, the load resistance was able
212 to increase sustainably until the test was stopped due to reaching stroke capacity of the jack. The
213 ultimate load of 469.8 kN was measured at a MCD of 526 mm. The failure mode of EX is shown in
214 Fig. 9. Similar to WX, fracture occurred in the tensile brace and severe buckling was observed in the
215 compressive braces. The connections failed either in bolt fracture or weld fracture. The 45 degrees full-
216 slant fracture surface of the bolts indicated that the bolts were suffered both tension and shear. Different
217 to EB, although weld fracture was also observed in EX, it did not propagate into the web. Thus, the
218 braces increased the deformation capacity of EX significantly.

219 **RB:** As shown in Fig. 5c, for RB, the yield load of 106.8 kN was measured at a MCD of 37 mm.
220 Thus, the initial stiffness was 2.9 kN/mm, which was only 87.9% of that of WB. For RB, the plastic
221 hinges were formed at the center of reduced beam section, which prevented the early fractures at the
222 welded connection. RB reached its ultimate load of 407.0 kN, which was 206.1% of WB, at the MCD
223 of 468 mm. Its failure mode is shown in Fig. 10.

224 **RV:** The yield load of 165.2 kN, which is about 154.7% of that of RB, was measured at a MCD
225 of 30 mm. Thus, the initial stiffness was 5.5 kN/mm. Before the tensile braces fractured, the weld
226 fracture already occurred at the connection near the middle column in the second story. Different to
227 RB, the fracture occurred near the weld connection rather than the reduced beam section due to the
228 gusset plates strengthened the reduced section of the beam in the second story. At the MCD of 253 mm,
229 the tensile brace in the left bay was fractured. At the MCD of 421 mm, RV reached its ultimate load of
230 356.3 kN before the weld fractured completely. The deformation capacity was only 90.0% of RB, as
231 the V braces strengthened the zone of plastic hinges in the beam of second story, which restricted the
232 rotation of the plastic hinge. The failure mode of RV is shown in Fig. 11. Similarly, fracture occurred
233 in the tensile brace while severe buckling occurred in the compressive braces. The weld fracture
234 occurred in Joint A. The V braces are welded on one side of the gusset plate and the brace forces were
235 not along the center axis of the beam, which resulted in torsional failure of the beams. The buckling
236 occurred at point D. The torsion mainly concentrated between beam segment CD leading to the tear
237 failure occurred at Joint B.

238 **Horizontal reaction force**

239 Fig. 12 decomposes the contribution of horizontal reaction force at the right side. Negative values
240 represent compressive force whereas positive values mean tensile force. As shown in Fig. 12, at small
241 deformation stage, the compressive reaction force of total reaction force was mainly provided by the
242 bottom pin support. The maximum compressive reaction force in WB, WX, EB, EX, RB, and RV was
243 -24.7, -7.8, -35.8, -23.5, -31.9, and -22.1 kN, respectively. With further increasing the displacement,
244 the compressive force began to decline and transferred into tension. However, the tensile force from
245 the bottom pin support kept marginal in catenary action stage and majority of tensile reaction force was

246 provided by the horizontal chain-poles connected to the overhanging beams. As shown in Figs. 12a, c,
247 and e, for bare frames, the tensile force was equally from the ground story and second story, although
248 the pre-mature failure occurred at the ground story of WB. Conversely, as shown in Figs. 12b, d, and
249 f, for braced frames, the tensile force from the second story was larger than that from the ground one.
250 The maximum tensile reaction force of WB, WX, EB, and EX was 363.5 kN, 406.8 kN, 551.5 kN, and
251 835.8 kN, respectively. Thus, contrary to compressive forces, with the help of braces, greater tensile
252 reaction force and catenary action was developed in braced frames. However, the maximum tensile
253 reaction force of RB and RV was 875.9 kN and 462.8 kN, respectively. This could be explained at the
254 gusset plate restricted the rotation of plastic hinges of the beams in the second story of RV, which
255 prevented the development of catenary action.

256 **Deformation measurements**

257 The deformation shape of edge column was shown in Fig. 13. It should be noted that the positive
258 value and negative value represents inward and outward movement, respectively. As shown in Fig. 13a,
259 for WB, initially, the horizontal inward movement was observed at the beam-column joints. However,
260 the horizontal inward movement at the joint in the second story was much larger than that in the ground
261 story. The maximum horizontal inward movement of the joint in the first and second story was only
262 3.2 mm and 9.1mm, respectively. As shown in Fig. 13b, for WX, outward movement was initially
263 measured in the ground story. The outward movement began to decrease when the vertical
264 displacement exceeded 60 mm. This is because the compressive brace generated additional outward
265 movement in the ground floor. However, the tensile braces are prone to pull the joint in the second
266 floor inward. The maximum inward movement of joint in the first and second stories was 4.3 mm and
267 15.2 mm, respectively. Similar results were observed for other specimens.

268 **Internal force measurements**

269 To further understand the load resisting mechanism of frames and de-composite the load
270 contribution of braces, the internal forces in beams and braces were quantified individually. The
271 reliability of strain gauge results to determine the load resistance of the specimens was evaluated first.
272 The vertical force and horizontal reaction force of braced specimens were determined by Eqs.1 and 2:

273
$$F_V = F_{V-Frame} + F_{V-Brace} \quad (1)$$

274
$$F_H = F_{H-Frame} + F_{H-Brace} \quad (2)$$

275 where F_V and F_H are the vertical force and horizontal reaction force of braced frame; $F_{V-Frame}$ and
 276 $F_{H-Frame}$ are the vertical force and horizontal reaction force of the bare frame, which has been
 277 described in detail in authors' previous work (Qian et al. 2020); $F_{V-Brace}$ and $F_{H-Brace}$ are the vertical
 278 force and horizontal reaction force provided by braces.

279 For X bracing configuration, the vertical and horizontal component force from braces could be
 280 calculated by Eqs. 3 and 4, respectively. However, for V bracing configuration, the force of braces
 281 could not be equivalent into the load resistance at the middle column directly. A schematic diagram
 282 was presented to illustrate the relationship, as shown in Fig. 14 and Eq. 5.

283
$$F_{V-Brace} = 2(N_T \sin \alpha - N_C \sin \beta) \quad (3)$$

284
$$F_{H-Brace} = 2(N_T \cos \alpha + N_C \cos \beta) \quad (4)$$

285
$$N'_T = N_T \cos \theta \quad (5)$$

286 where N_T and N_C are the axial force of the tensile and compressive brace, respectively; α is the
 287 angle between tensile brace and horizontal axis; β is the angle between compressive brace and
 288 horizontal axis; N'_T is the adopted axial force of tensile brace for RV; θ is the angle between tensile
 289 brace and diagonal line.

290 The axial force of braces can be calculated according to Eq. 5.

291
$$N = EA \left(\frac{\sum_{i=1}^n \varepsilon_i}{n} \right) \quad (6)$$

292 where E is the elastic modulus; A is the section area of the brace; $\left(\frac{\sum_{i=1}^n \varepsilon_i}{n} \right)$ is the average axial strain
 293 of the cross-section.

294 Fig. 15 compares the vertical load-displacement curve and horizontal reaction force-displacement
 295 curve based on load cell results and strain gauge results. From the figures, it can be seen that the
 296 analytical results based on strain gauge generally agree with the load cell results well. Thus, the

297 analytical results based on strain gauge results were used in following analysis and discussion. Fig. 16
298 illustrates the de-composition of the load resistance from the frame and steel braces. As shown in the
299 figures, the load resistance from steel braces was higher than that from frame initially. However, the
300 load resistance of the frame kept increasing due to the development of flexural action. Conversely, the
301 load resistance from braces began to reduce soon due to compressive braces buckled. When partial of
302 the tensile brace fractured, the contributions of steel braces are reduced to 37.7%, 38.5%, and 29.5 %
303 for WX, EX, and RV, respectively.

304 **DISCUSSION OF TEST RESULTS**

305 **Effects of connection types**

306 Fig. 17 compares the behavior of different connections. As shown in Fig. 17a, for bare frames,
307 the greatest load resisting capacity was measured in RB due to the reduced section prevents the pre-
308 mature fracture of the welds. Thus, it can be concluded that, when properly designed and fabricated,
309 reduced beam section connections can be a cost-effective solution to enhance the robustness of steel
310 frames with rigid connections. The second largest capacity was observed in EB. Among these three
311 tests, EB achieved the greatest deformation capacity. Although the failure modes of EB are also
312 controlled by weld fracture, it has a more ductile process. Before weld failure, the end plate yielded
313 and experienced considerable local plastic deformation, which brought in greater catenary action
314 capacity. The load resistance from flexural action and catenary action are compared in Figs. 17b and c,
315 respectively. As shown in Fig. 17b, WB has the greatest flexural action capacity. At flexural action
316 stage, WB reached the largest initial stiffness and load resistance. As shown in Fig. 17c, when catenary
317 action is included in the load resistance, the load resisting capacity could increase significantly
318 especially for RB and EB.

319 **Effects of braces**

320 As shown in Fig. 5 and Table 3, the measured yield load of WB, WX, EB, EX, RB, and RV were
321 147.8, 233.9, 94.4, 148.5, 106.8, and 165.2 kN, respectively. Thus, the X bracing system increased the
322 yield load of WB and EB by 58.3% and 57.3%, respectively. Similarly, the V bracing system increased
323 the yield load of RB by 54.7%. Regarding the initial stiffness, WB, WX, EB, EX, RB and RV were

324 3.3, 7.5, 2.2, 5.0, 2.9, and 5.5 kN/mm, respectively. Thus, the X bracing system increased the initial
325 stiffness of WB and EB by 127.3% and 120.2%, as the X bracing system in WB and EB could transfer
326 partial of the vertical load to the side column directly. Similarly, the V bracing system increased the
327 initial stiffness of RB by 89.7%. The ultimate load capacities of WB, WX, EB, EX, RB, and RV in the
328 large deformation stage were 197.5, 286.5, 255.4, 469.8, 407.0, and 356.3 kN, respectively. WX and
329 EX could increase the ultimate load capacity of the counterpart bare frames by 45.1% and 83.9%. This
330 could be explained as one of tensile braces in WX and EX were not fractured until test stopped, which
331 increased the load resistance in catenary action stage. Conversely, RV only achieve 87.5% of the
332 ultimate load capacity of RB. This is because the gusset plate of RV bracing system in the second story
333 restricted the rotation of plastic hinges, which was designated to form at the reduced section. Thus, the
334 catenary action capacity and deformation capacity of RV was even less than RB. Moreover, it should
335 be noted that for a steel frame, only partial of peripheral structural bays have bracings. Thus, the effects
336 of bracings mentioned above are only suit for these braced bays.

337 **Contribution of load resisting mechanisms**

338 Fig. 18 gives the de-composition of load resistance of frame from catenary action and flexural
339 action. As shown in Fig. 18a, before MCD reached 128 mm, significant bending moment was
340 developed and thus, flexural action dominated the load resistance of WB at the beginning of the test.
341 In large deflection stage, axial tensile force and catenary action was mobilized. Flexural action and
342 catenary action worked together to redistribute the applied load. The maximum flexural action and
343 catenary action were 175.5 kN and 78.7 kN, respectively. For WX, flexural action was higher than that
344 of WB before weld fracture occurred. Different to WB, the vertical load of frame could keep increasing
345 until reached its maximum load resistance due to the significant catenary action. Due to X braces, the
346 flexural action and catenary action were increased by 17.8% and 61.3%, respectively. As shown in Fig.
347 18c, although the maximum flexural action was only 74.1% of that of WB, the catenary action was
348 207.1% of that of WB. Thus, the ultimate load of EB is 129.3% of that of WB. For EX, it is worth
349 noting that significant tensile force was developed in the beams. Due to X braces, the flexural action
350 and catenary action was increased by 25.7% and 137.9%, respectively. As shown in Fig. 18e, although

351 both WB and RB were welded connection, greater catenary action was developed in RB. This is
352 because the failure of WB was controlled by weld fracture, which was brittle and pre-mature. The
353 reduced beam section would avoid this brittle failure and increase the ductility and deformation
354 capacity. Although the maximum flexural action was only 82.1% of that of WB, the catenary action of
355 RB was 380.8% of that of WB and thus, the ultimate load of RB is 203.9% of that of WB. As shown
356 in Fig. 18f, RV sustained substantial bending moment before the connection fractured. However, with
357 further increasing the displacement, the increase of catenary action was limited, which was unlike RB.
358 This is mainly due to gusset plates at the second story prevents the development of catenary action.
359 Comparing with RB, the flexural action and catenary action was 115.6% and 55.8% of its counterpart.

360 Fig. 19 decomposes the load resistance from the first and second stories. It can be seen that the
361 trend of load resistance from the first and second story is similar before failure first occurred at the
362 connection. However, the load resistance from the ground story was slightly larger than that from the
363 second story due to different horizontal constraints. To further study on the difference, as labeled in
364 Fig. 20, the flexural and catenary action of each story were extracted. Due to higher rotational restraints
365 from the side column, the flexural action from the first story may be larger than that of second story.
366 In the first story, the catenary action developing in the beams is always in tension. However, the
367 catenary action of the second story is in compression (negative) firstly, and then transfers into tension
368 at the large deformation stage. Therefore, both flexural and catenary action could develop in the first
369 story effectively, leading to larger resistance.

370 **De-composition of the load resistance from braces**

371 Fig. 21 gives the de-composition of contribution of braces from tensile and compressive braces.
372 For WX, EX, and RV, the maximum load resistance of steel bracing was 111.9, 120.3, and 87.1 kN,
373 respectively. As shown in the figures, WX and EX could keep increasing after buckling of compressive
374 brace. These two specimens reached their maximum value when the one of the tensile braces fractured.
375 Before buckling of the compressive brace, the maximum load resistance from compressive brace of
376 WX and EX were 19.2 kN and 19.1 kN, respectively. Different to WX and EX, after buckling of the
377 compressive braces, RV reached its maximum value as large part of the load resistance provided by

378 the compressive braces. For RV, the maximum load resistance from compressive brace was 51.7 kN,
379 which was 269.2% and 270.7% of that of WX and EX, respectively. However, as shown in in Fig. 21,
380 due to the buckling of the compressive braces at very early stage, they only affect the initial stiffness
381 significant. To further understand the contribution of steel braces, the axial force of tensile and
382 compressive braces was determined and presented in Fig. 22. From basic analysis, the yield load and
383 ultimate load of the tensile braces were determined as 85.6 kN and 115.9 kN, respectively. Tensile
384 braces in X configuration could achieve their yield load and even achieved their ultimate load before
385 fracture. However, the tensile braces in V configuration only achieved their yield load. From basic
386 analysis, the buckling load of compressive brace were 23.0 kN and 57.6 kN for X and V configuration,
387 respectively. As shown in the figures, the compressive braces in X configurations could achieve their
388 buckling load. Different to X configuration, the compressive braces in V configuration could not
389 achieve their buckling load.

390 **CONCLUSIONS**

391 To evaluate the effects of steel braces on the behavior of steel frames to mitigate progressive
392 collapse, six two-story steel sub-frames (three bare frames and three braced frames) with different
393 connections were tested under a middle column missing scenario. Based on experimental results,
394 following conclusions are drawn:

- 395 1. The results from bare frames revealed that various of failure mode, ductility, and load resisting
396 mechanisms were developed in the steel frames with different types of connections. Different to
397 weld fractures in WB and EB, RB failed by fracture at the reduced beam section. Thus, the greatest
398 deformation capacity and catenary action capacity were achieved in RB. The ultimate load
399 capacity of RB was 206.1% and 159.4% of that of WB and EB, respectively.
- 400 2. The test results demonstrated that for WB, the main load resistance was from the flexural action,
401 rather than catenary action, which is mainly due to pre-mature weld fracture at the connections.
402 Conversely, for EB and RB, relatively less flexural action was developed, catenary action
403 increased the load resisting capacity of EB and RB by 81.6% and 125.8%, respectively. Thus, the
404 load resistance of these two connections were dominated by catenary action, rather than flexural

405 action.

406 3. The test results indicated that steel braces could provide considerable additional load resistance
407 for steel frames to resist progressive collapse effectively. The steel braces in WX, EX, and RV
408 increased their initial stiffness by 127.3%, 120.2%, and 89.7%, respectively. For RV, the
409 effectiveness of the steel braces is relatively milder because their braces could not transfer the
410 vertical load to side columns directly. The X brace in WX and EX increased the ultimate load
411 capacity of the bare frames by 45.1% and 83.9%. However, the V braces in RV even decreased
412 the ultimate load capacity of RB by 12.5% as the gusset plates restricted the formation of plastic
413 hinges in reduced beam section. Thus, the gusset plate should not be installed in the reduced beam
414 section to avoid negative effects on braced specimens, as the gusset plate may restrict the rotation
415 capacity of the reduced beam section.

416 4. The out-of-plane buckling of compressive braces occurred from the beginning of the test, which
417 could not provide sufficient load resistance after buckling, especially for X bracing system.
418 However, due to the difficulties in predicting the location of column missing accurately, a
419 symmetric bracing configuration is still recommended. As the additional load resistance mainly
420 attributed into the tensile braces, the fracture of tensile braces normally accompanied by the drop
421 of load resistance significantly. From the failure modes, it was found that the braces may change
422 the failure mode of the beams as the beam ends may suffer greater shear force, which may lead to
423 pre-mature weld fracture or bolt fracture. It should be noted that above conclusions regarding steel
424 braces are only suit for the structural bays with steel bracings. For the structural bays without any
425 bracings, the benefits of bracings could not be incorporated in design or analysis.

426 **DATA AVAILABILITY**

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

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515 **FIGURE CAPTIONS**

516 **Fig. 1.** Location of the extracted frame in the prototype building (unit in mm): (a) plan view; (b)
517 elevation view

518 **Fig. 2.** Dimensions of the specimen and locations of strain gauge and displacement transducers (unit
519 in mm): (a) arrangements of strain gauges/rosettes and displacement transducers; (b) strain gauges
520 positions on sections

521 **Fig. 3.** Geometric details of connections (unit in mm): (a) welded connection; (b) end plate connection;
522 (c) reduced beam section connection

523 **Fig. 4.** Test setups of EX: (a) schematic view; (b) photograph

524 **Fig. 5.** Vertical force-middle column displacement curves: (a) comparison of WX and WB; (b)
525 comparison of EX and EB; (c) comparison of RV and RB

526 **Fig. 6.** Failure mode of WB

527 **Fig. 7.** Failure mode of WX

528 **Fig. 8.** Failure mode of EB

529 **Fig. 9.** Failure mode of EX

530 **Fig. 10.** Failure mode of RB

531 **Fig. 11.** Failure mode of RV

532 **Fig. 12.** Horizontal reaction force-middle column displacement curves: (a) WB; (b) WX; (c) EB; (d)
533 EX; (e) RB; (f) RV

534 **Fig. 13.** Horizontal movement of exterior joints: (a) WB; (b) WX

535 **Fig. 14.** Schematic diagram of internal force calculation of RV: (a) idealized model; (b) internal forces
536 transformation

537 **Fig. 15.** Comparisons of vertical load resistance and horizontal reaction force from strain gauge and
538 load cells: (a) WB; (b) WX

539 **Fig. 16.** De-composition of the load resistance from bareframe and steel brace: (a) WX; (b) EX; (c)
540 RV

541 **Fig. 17.** Comparison of the behaviour of different connections: (a) vertical force; (b) flexural action;
542 (c) catenary action

543 **Fig. 18.** De-composition of the load resistance of bareframe from different actions (Note: FA and CA
544 represent flexural action and catenary action, respectively) : (a) WB; (b) WX; (c) EB; (d) EX; (e) RB;
545 (f) RV

546 **Fig. 19.** De-composition of load resistance from 1st story and 2nd story: (a) WB; (b) WX; (c) EB; (d)
547 EX; (e) RB; (f) RV

548 **Fig. 20.** Comparisons of the flexural and catenary action variation in 1st story and 2nd stories (Note: FA
549 and CA represent flexural action and catenary action, respectively) : (a) WB; (b) WX; (c) EB; (d) EX;
550 (e) RB; (f) RV

551 **Fig. 21.** De-composition of the load resistance of brace from different braces (Note: TB and CB
552 represent tensile brace and compressive brace, respectively) : (a) WX; (b) EX; (c) RV

553 **Fig. 22.** Axial force of braces (Note: TB and CB represent tensile brace and compressive brace,
554 respectively) : (a) WX; (b) EX; (c) RV

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Table 1-Specimen properties

Test ID	Connection type	Steel brace
WB	Welded connection	Bare frame without braces
WX	Welded connection	Braced frame with X braces
RB	Reduced beam section connection	Bare frame without braces
RV	Reduced beam section connection	Braced frame with V braces
EB	End plate connection	Bare frame without braces
EX	End plate connection	Braced frame with X braces

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Table 2-Material properties

Items	Plate thickness (mm)	Yield strength (MPa)	Yield strain	Ultimate strength (MPa)	Ultimate strain	Elongation (%)
Beam flange	8.0	310	0.0019	420	0.0240	12.0
Beam web	5.5	320	0.0021	430	0.0249	13.5
Column flange	10.0	300	0.0019	410	0.0267	14.0
Column web	7.0	295	0.0023	375	0.0242	13.0
Steel brace	4.0	310	0.0021	420	0.0256	12.5

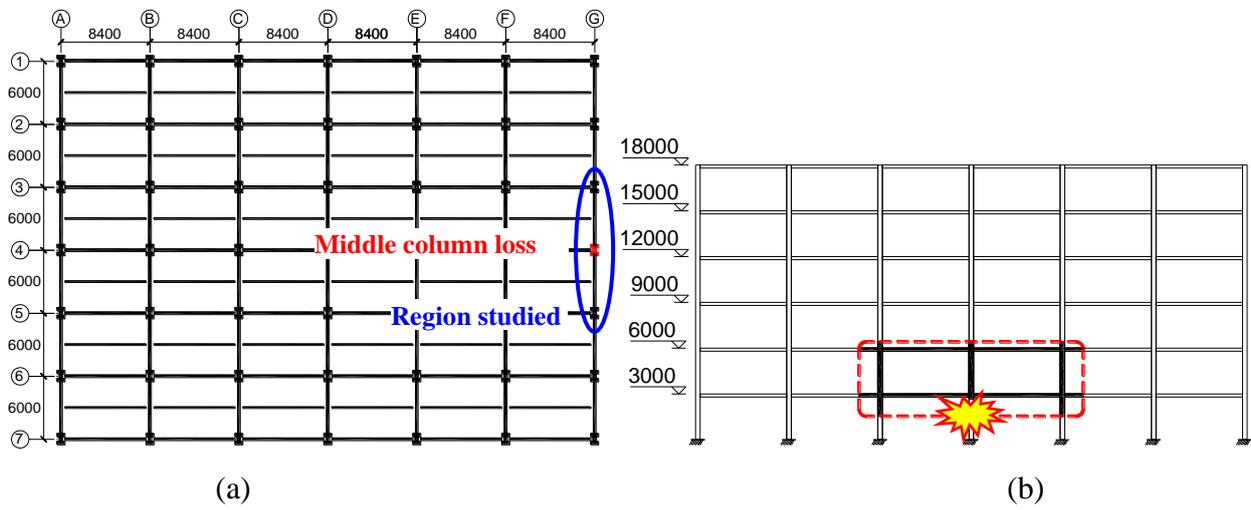
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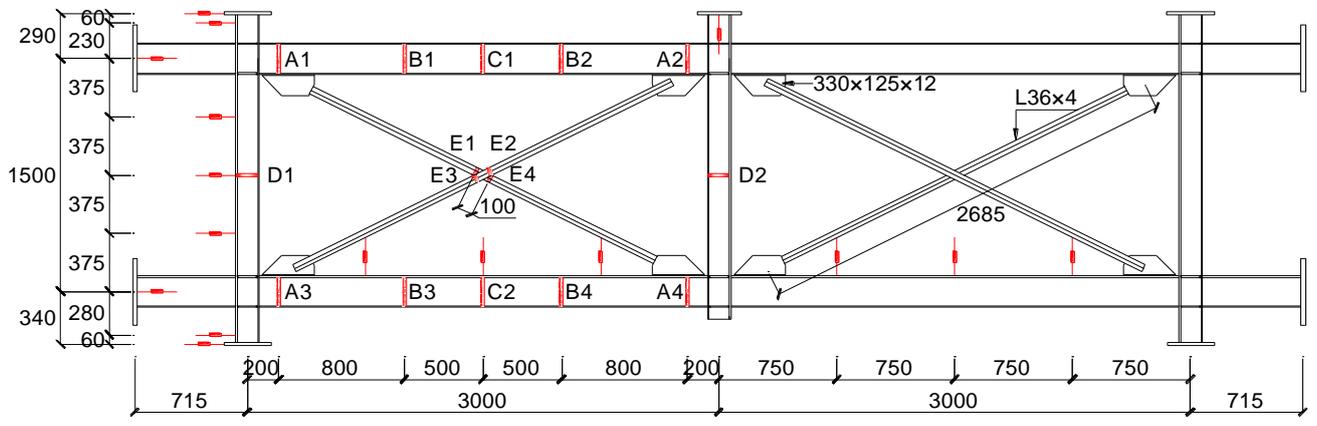
Table 3-Test results

Test ID	U _{YL} (mm)	F _{YL} (kN)	K _{YL} (kN/mm)	U _{UL} (mm)	F _{UL} (kN)
WB	45	147.8	3.3	200	197.5
WX	31	233.9	7.5	382	286.5
EB	42	94.4	2.2	515	255.4
EX	30	148.5	5.0	526	469.8
RB	37	106.8	2.9	468	407.0
RV	30	165.2	5.5	421	356.3

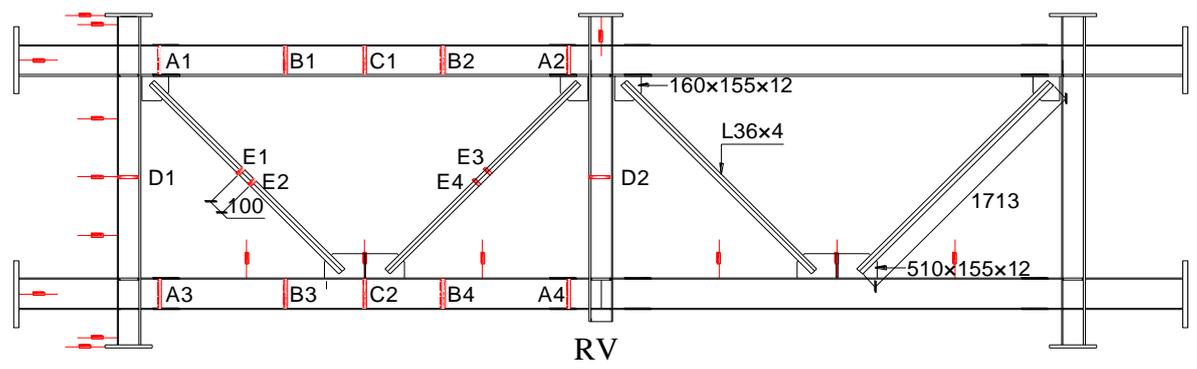
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Note: F_{YL} and F_{UL} represent yield load and ultimate load, respectively; U_{YL} and U_{UL} represent displacements corresponding the yield load and ultimate load, respectively; K_{YL} represents initial stiffness corresponding the yield load.



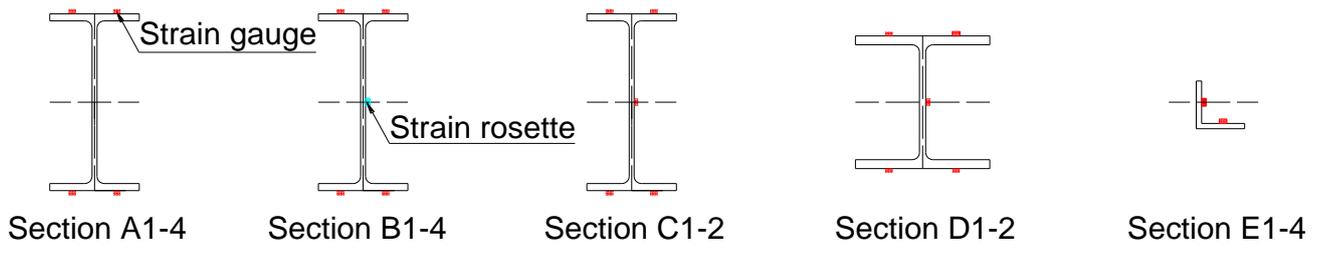


WX

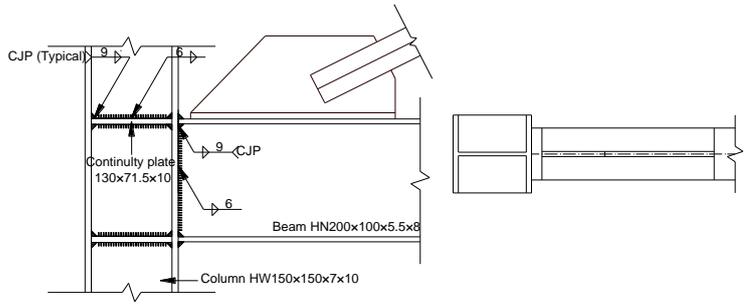


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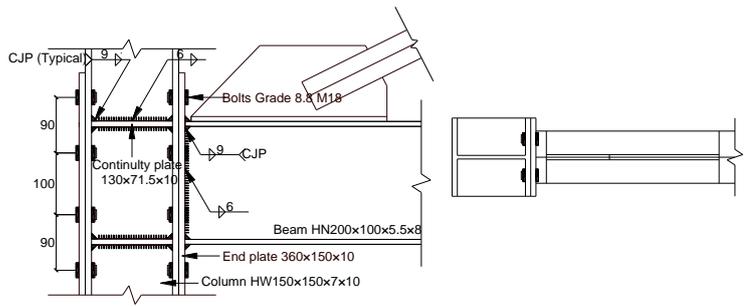
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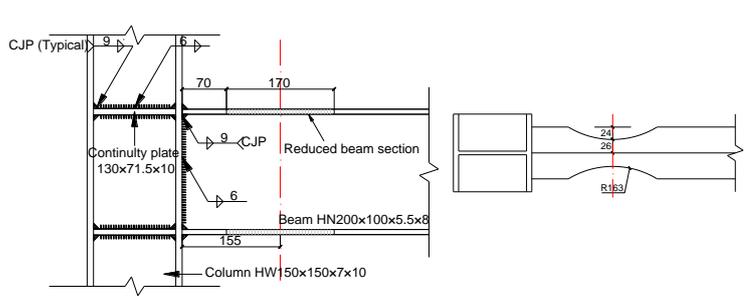
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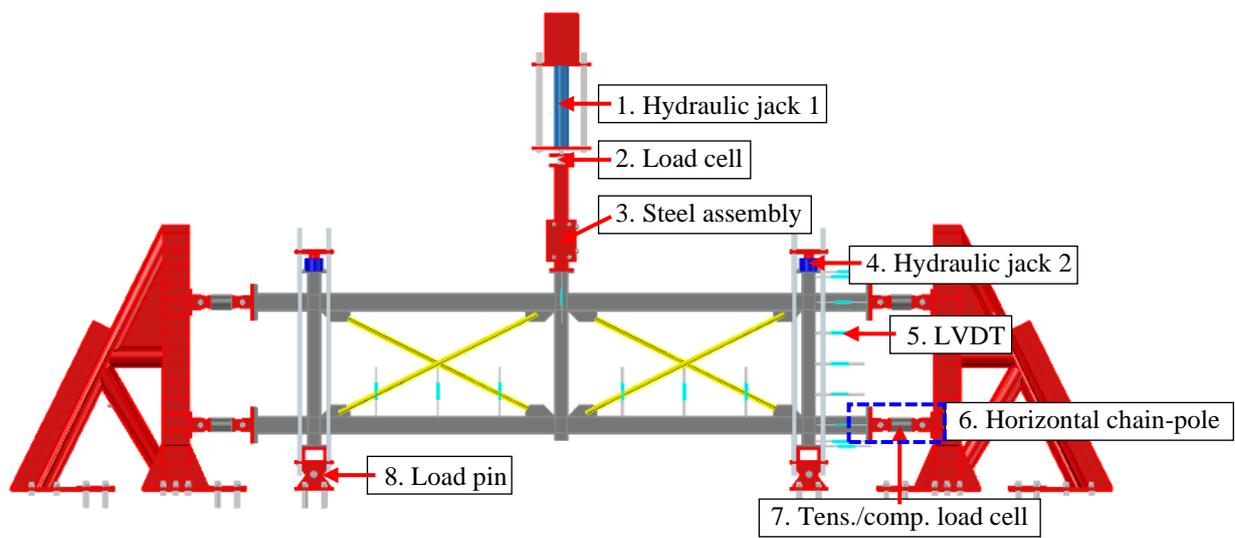
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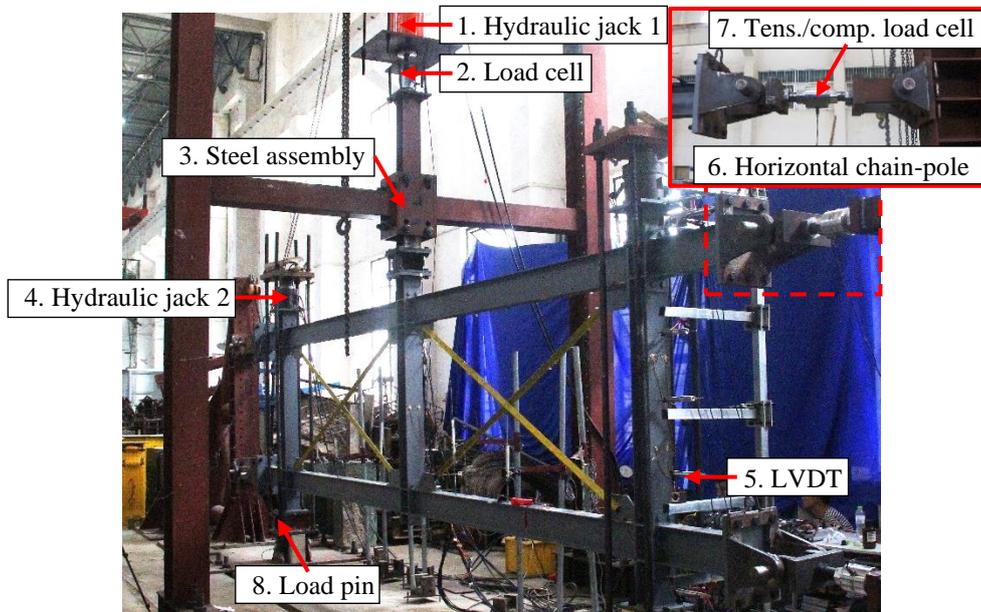
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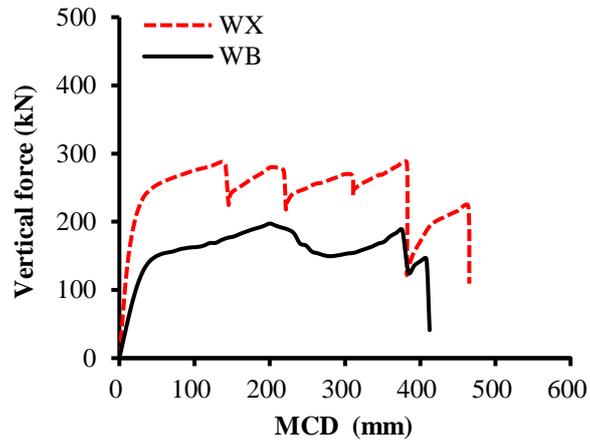
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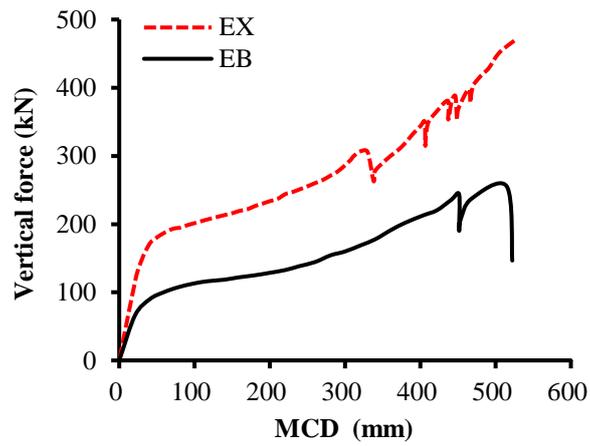
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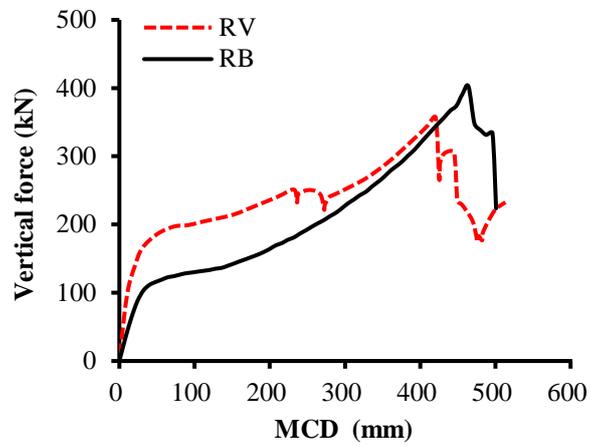
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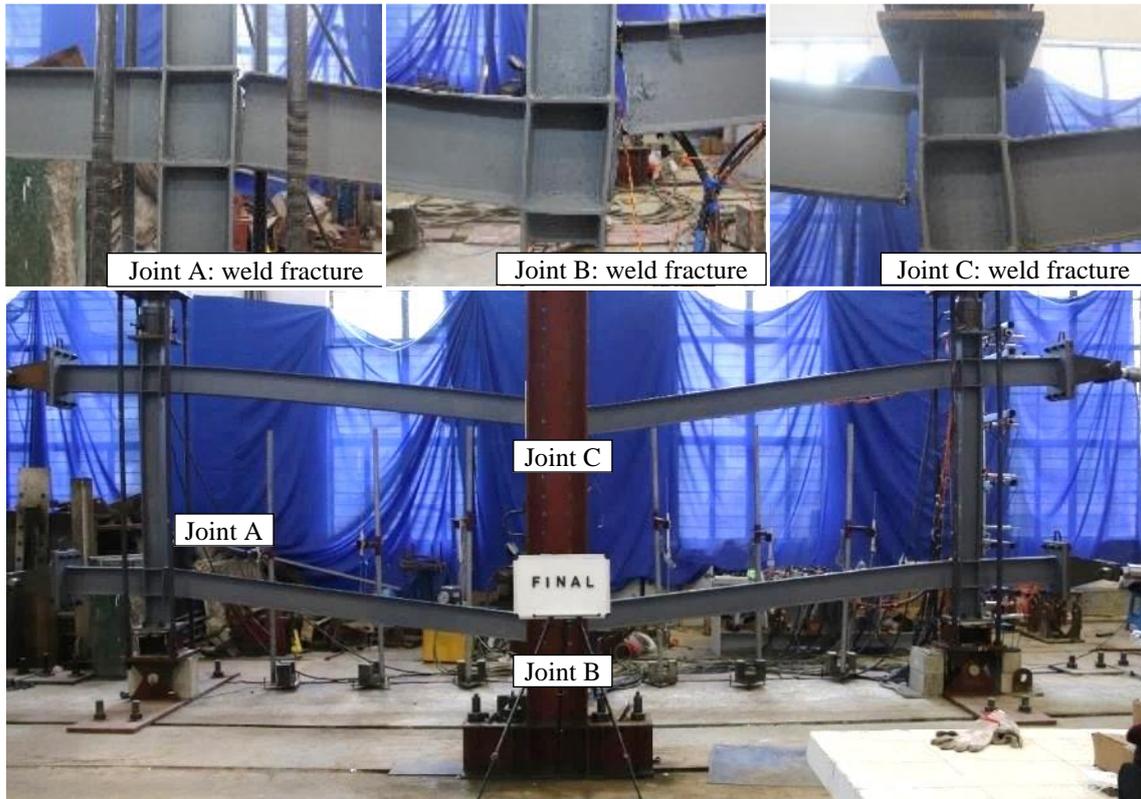
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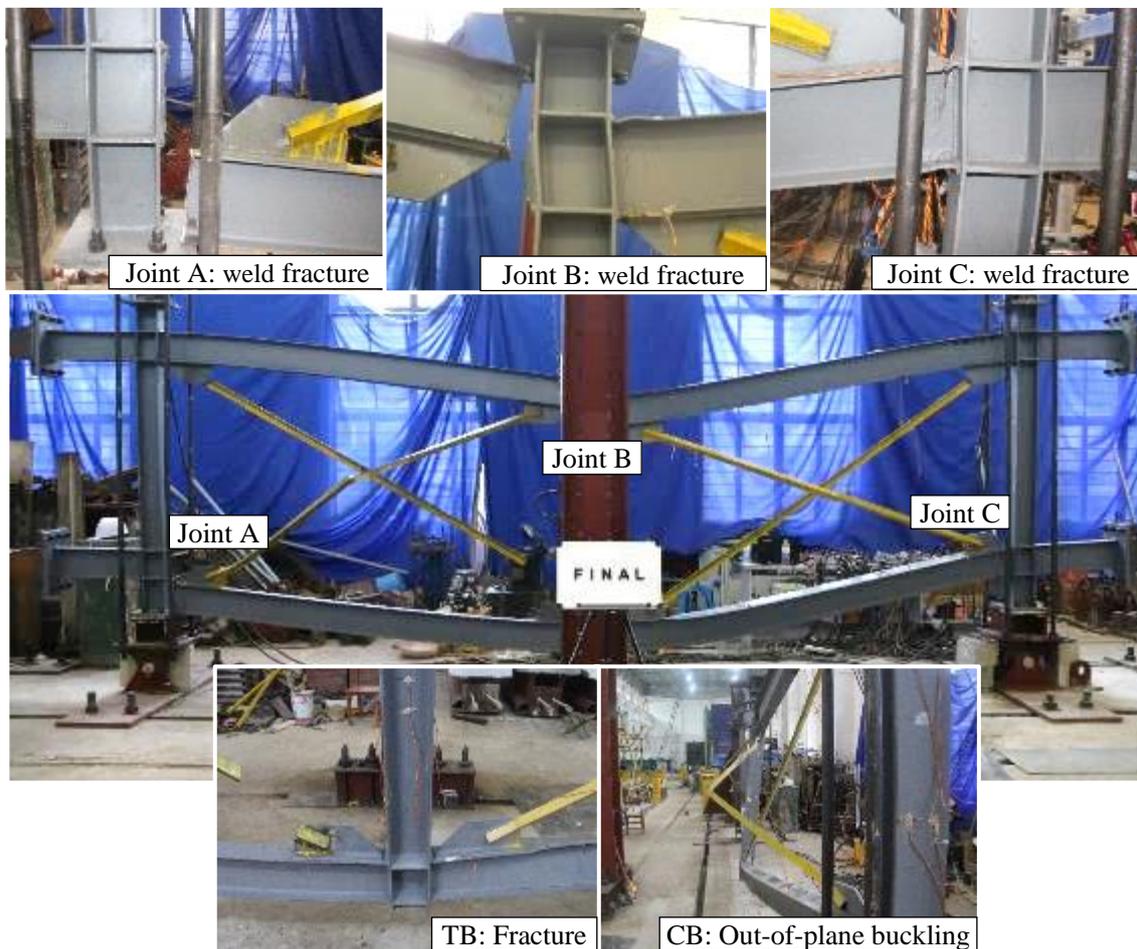


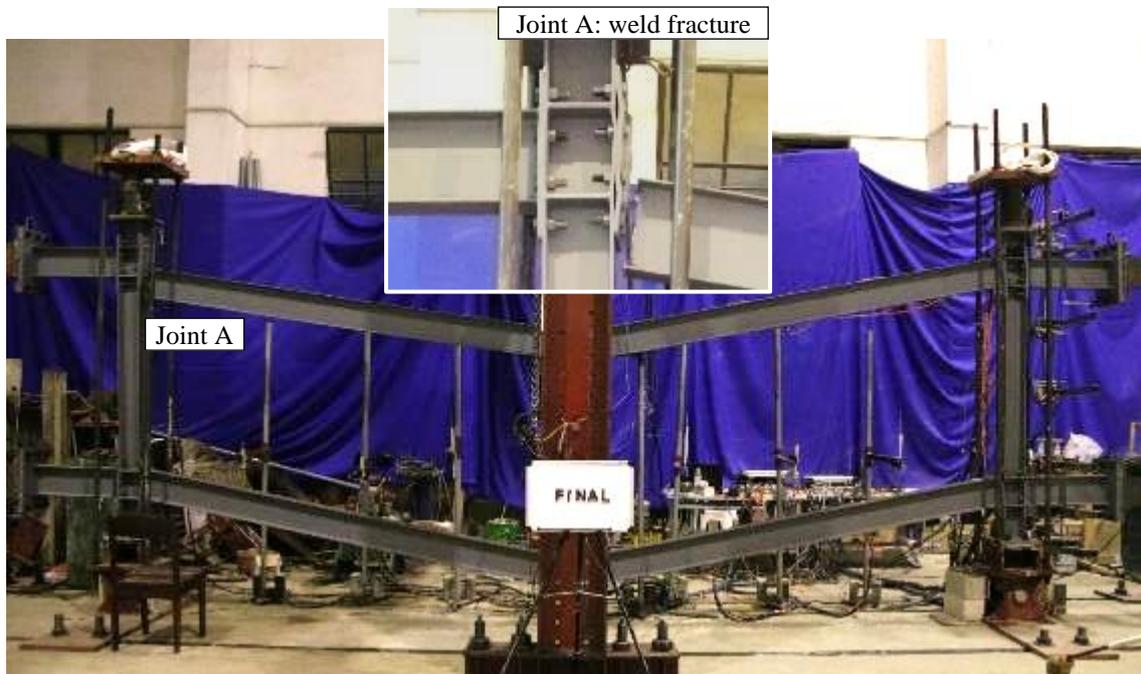
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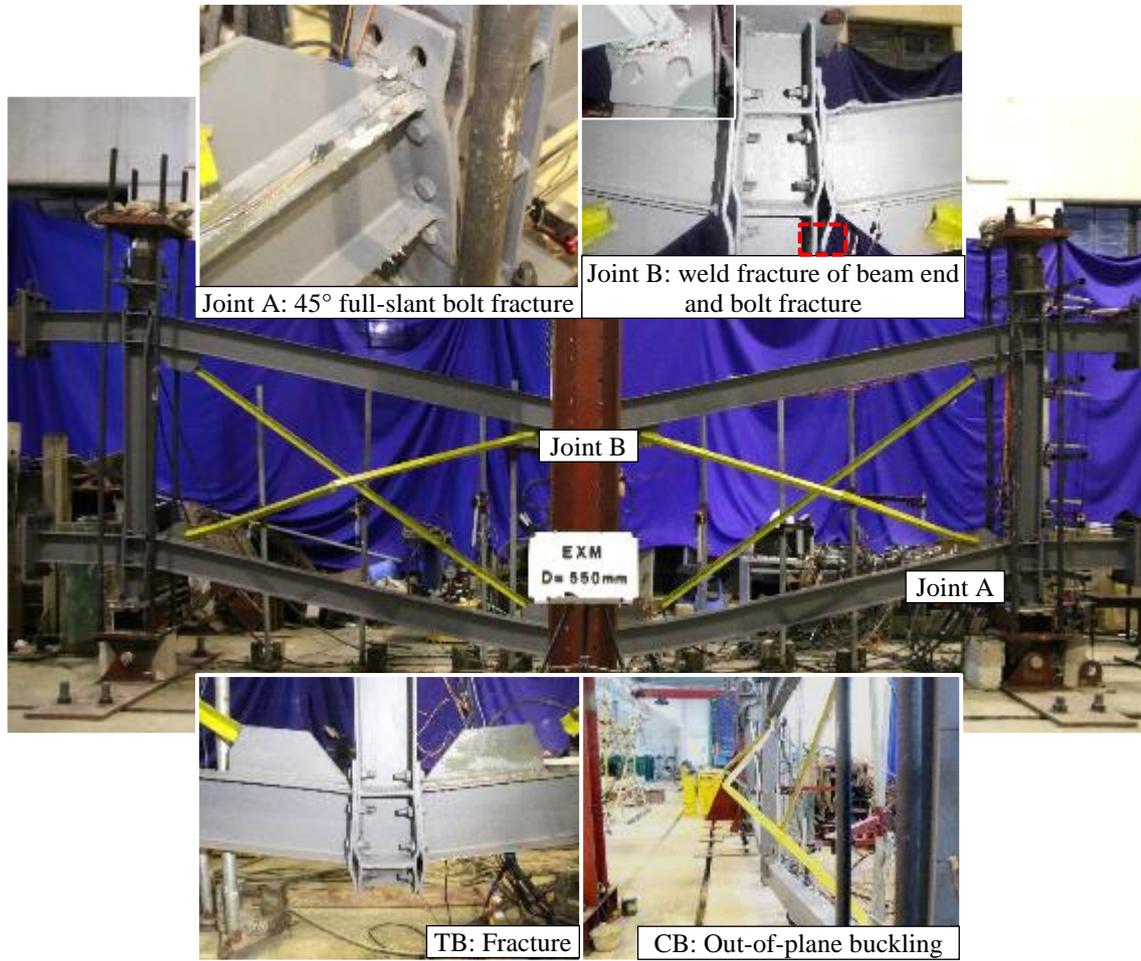


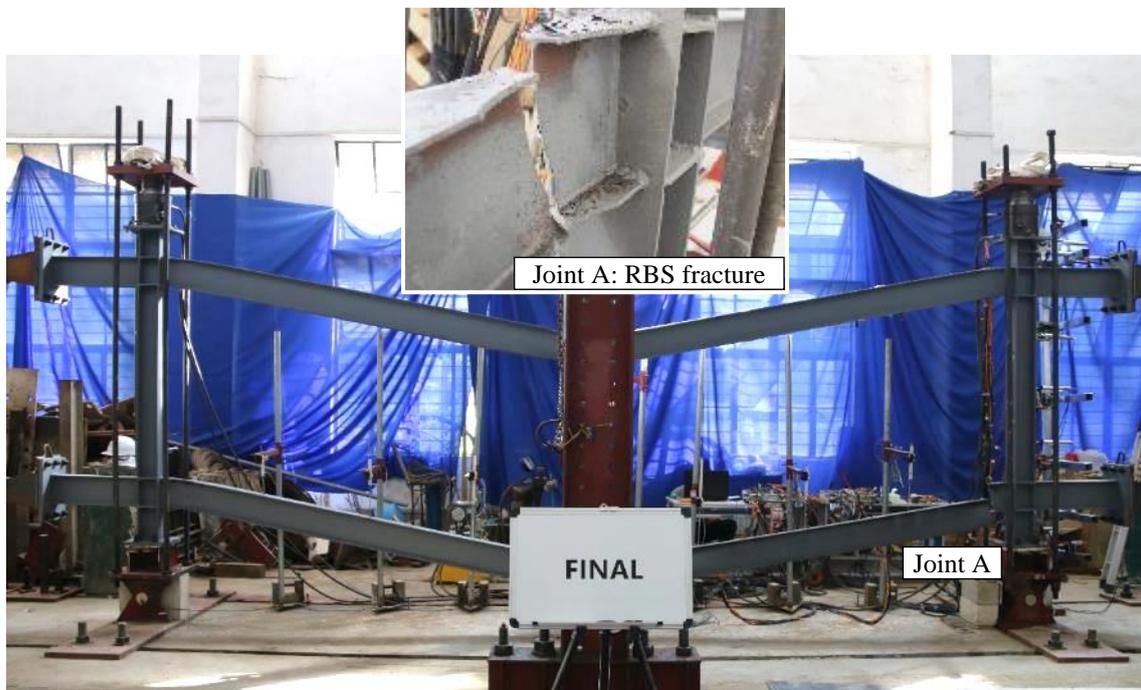
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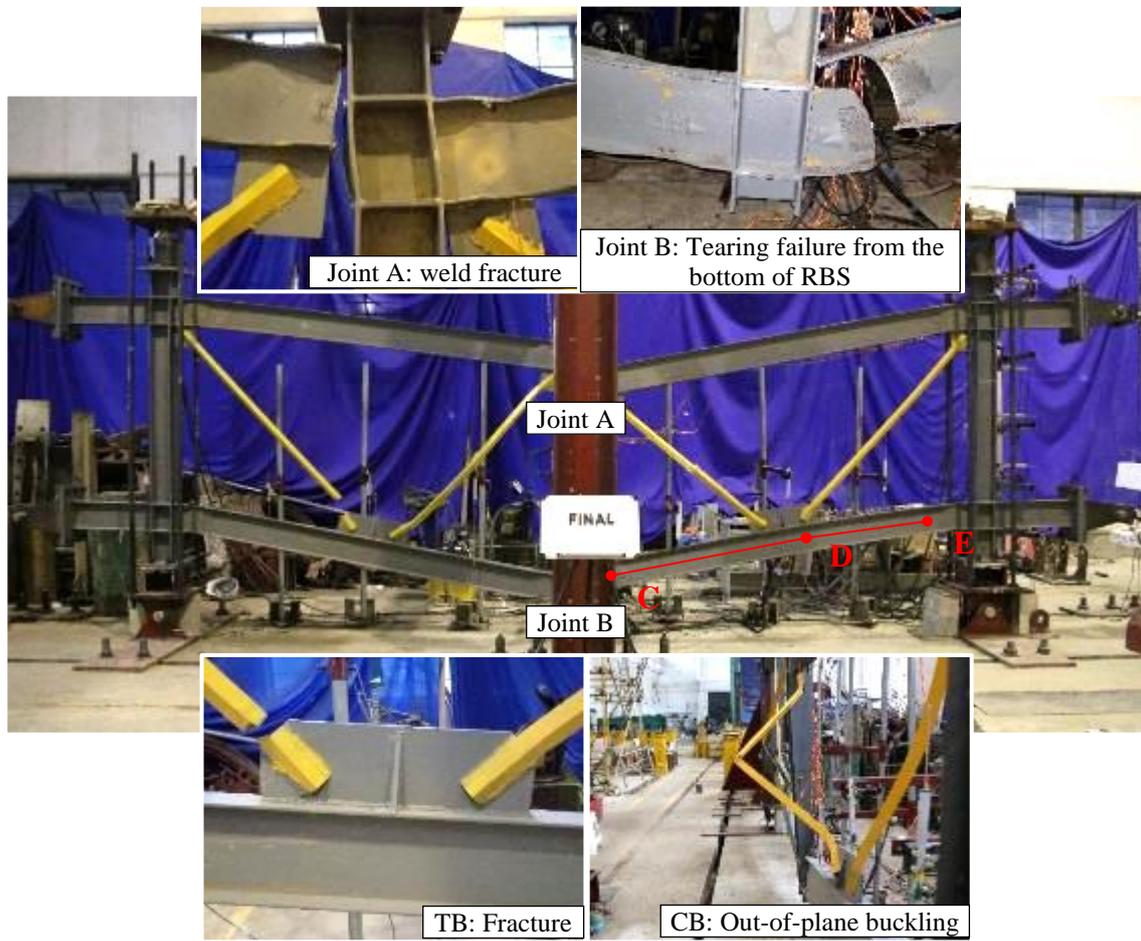


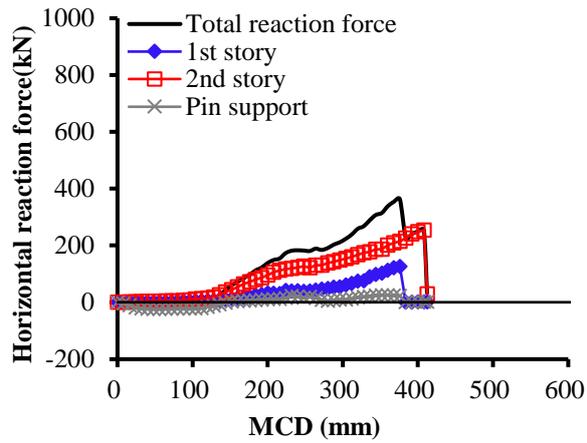




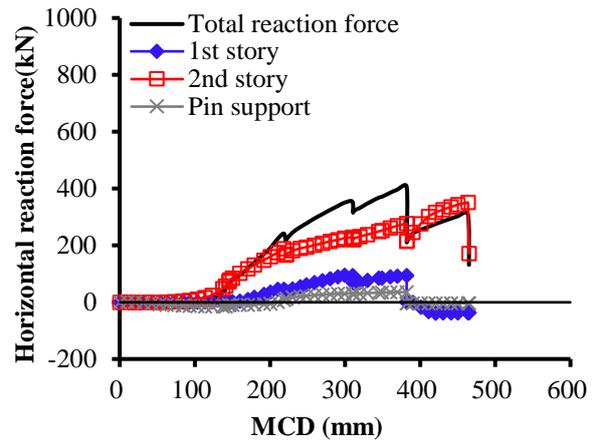




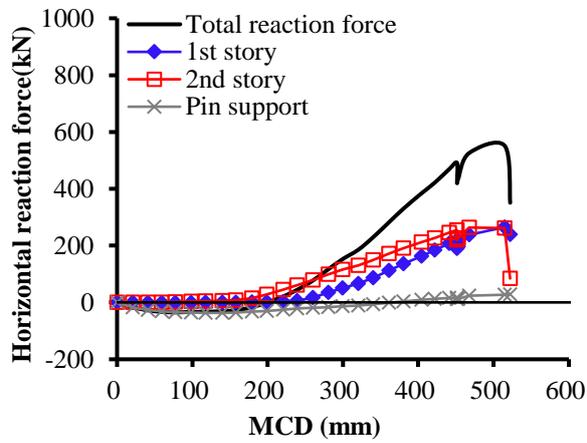




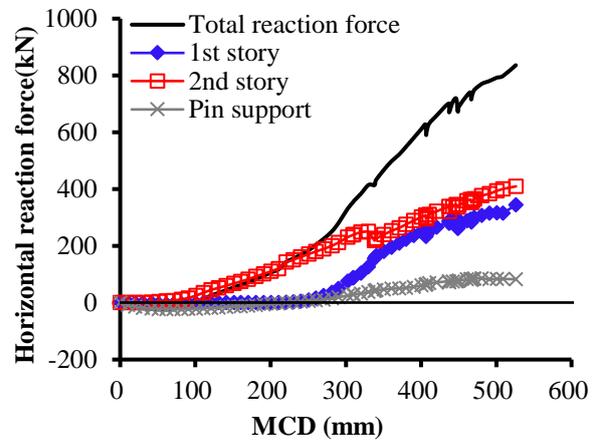
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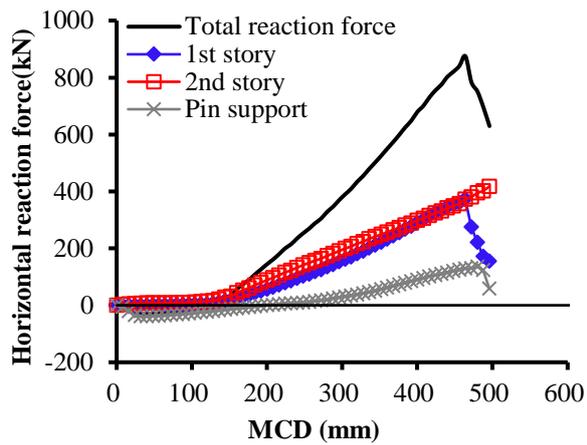
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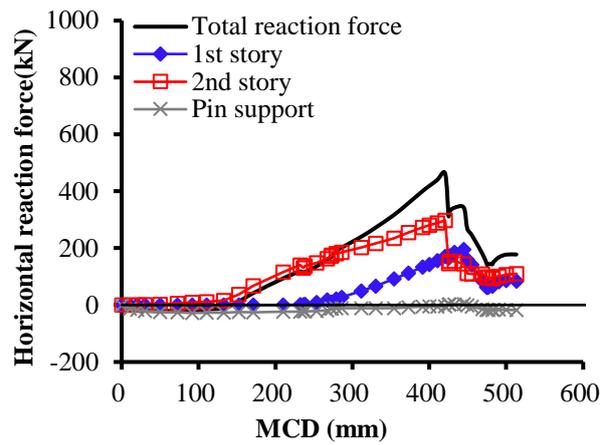
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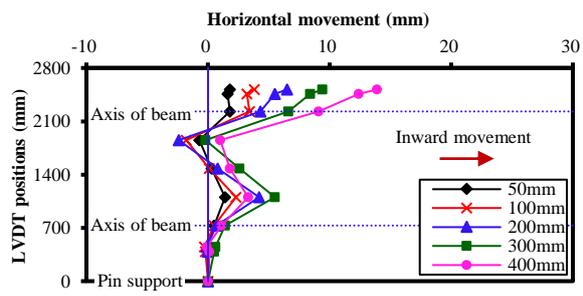
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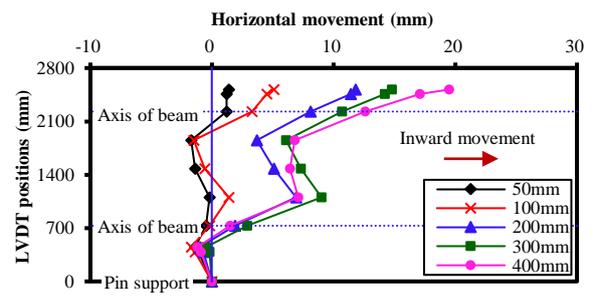
(e)



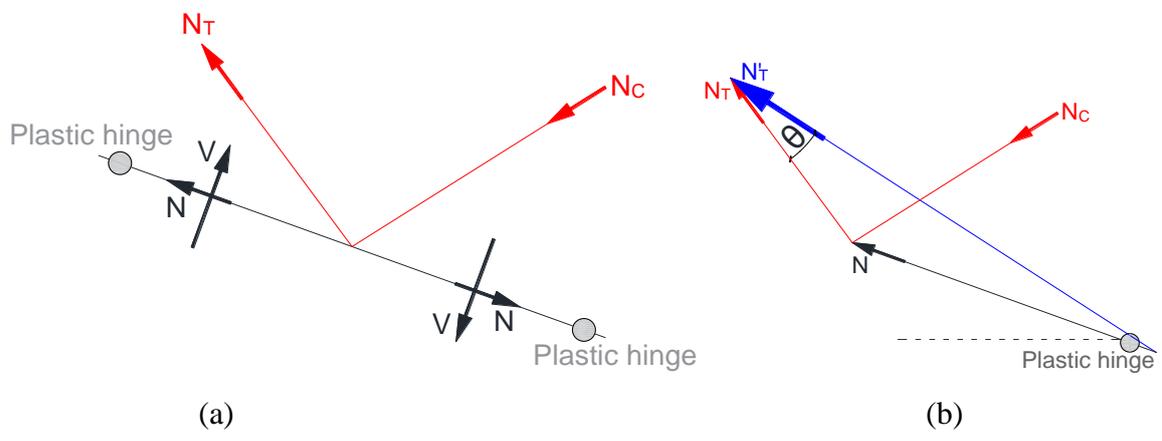
(f)

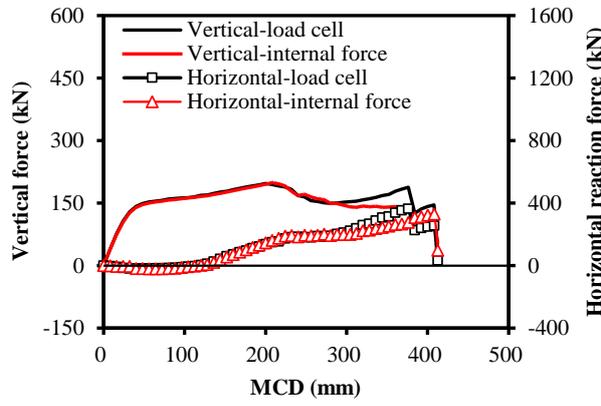


(a)

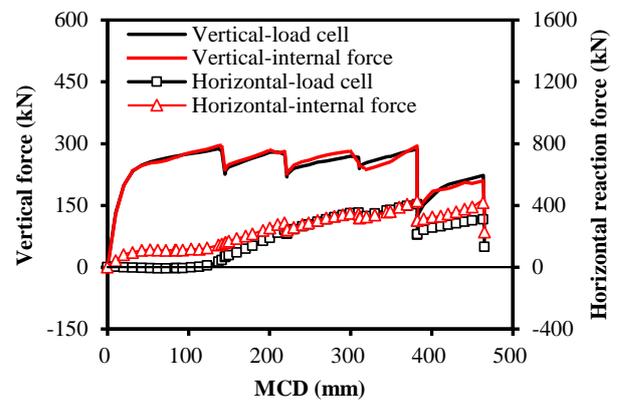


(b)

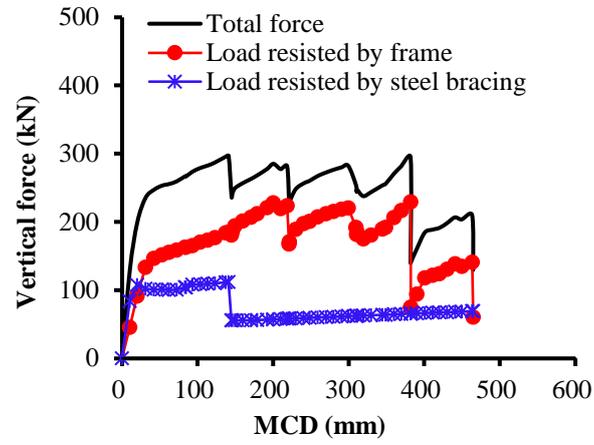




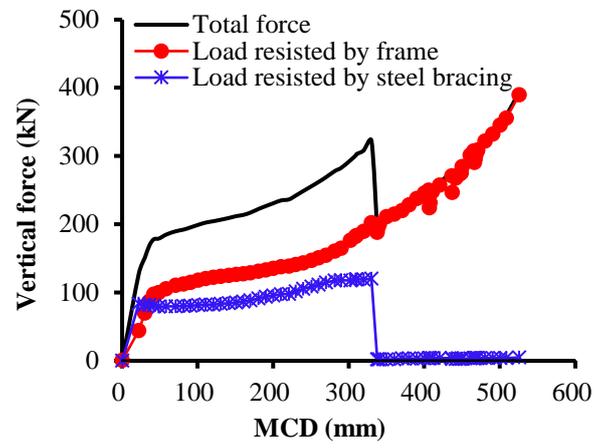
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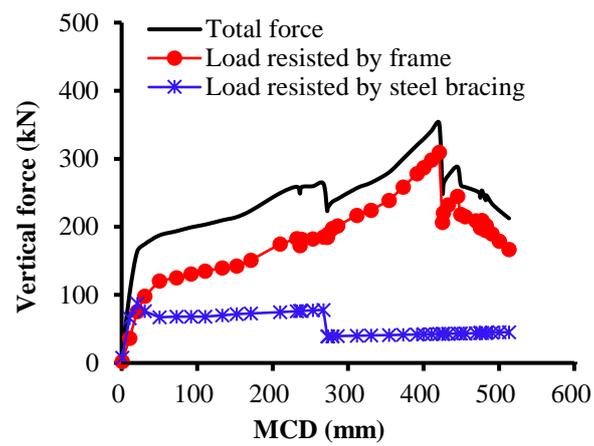
(b)



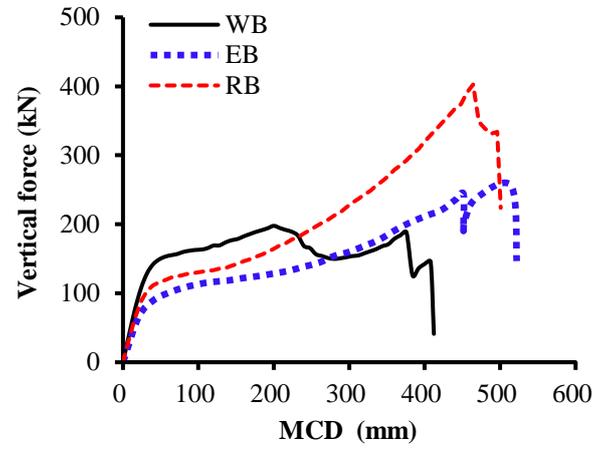
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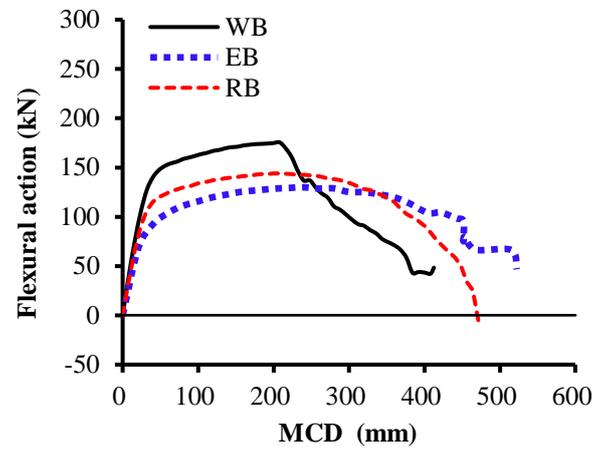
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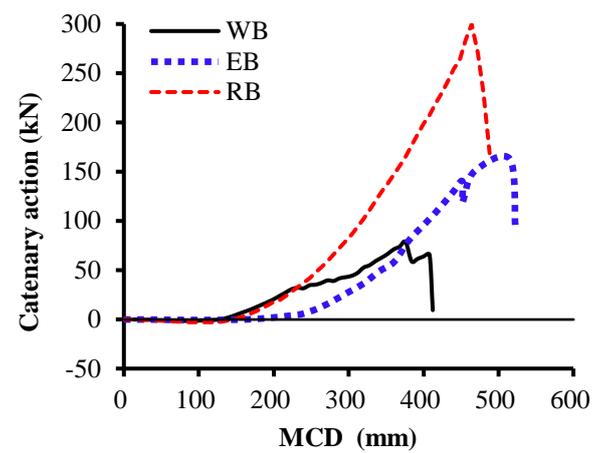
(c)



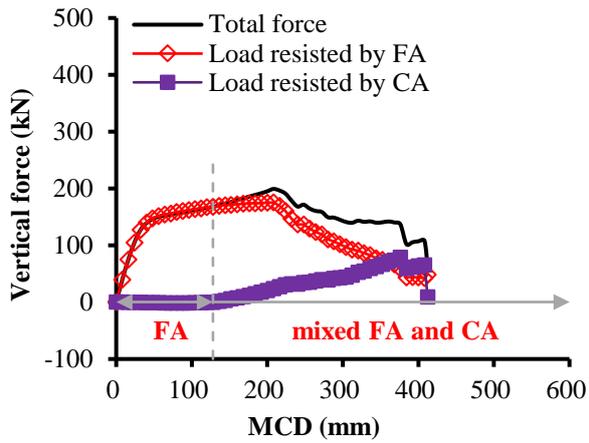
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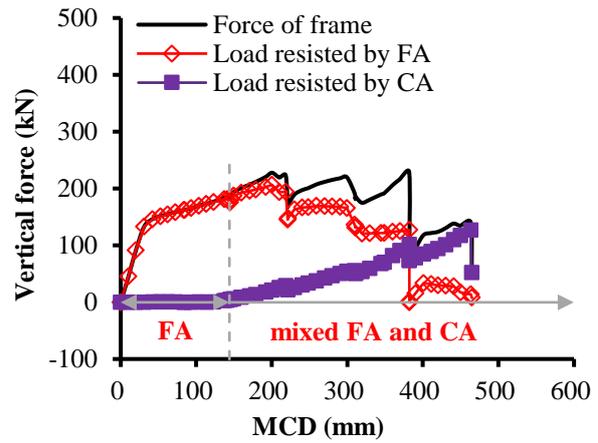
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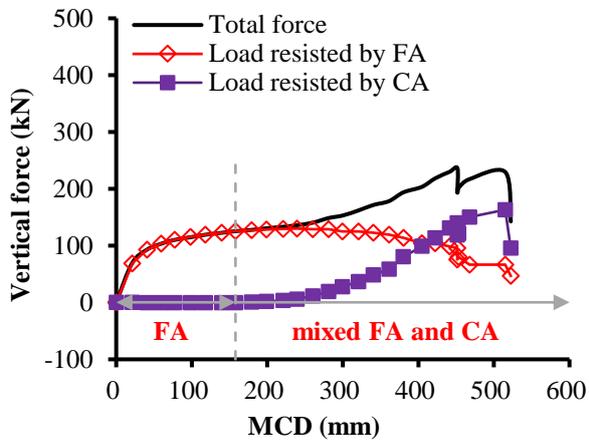
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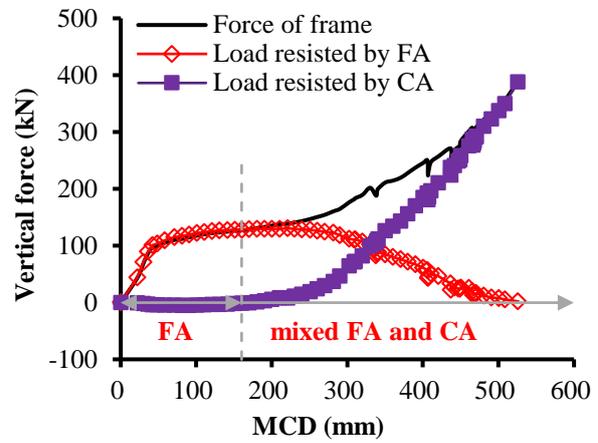
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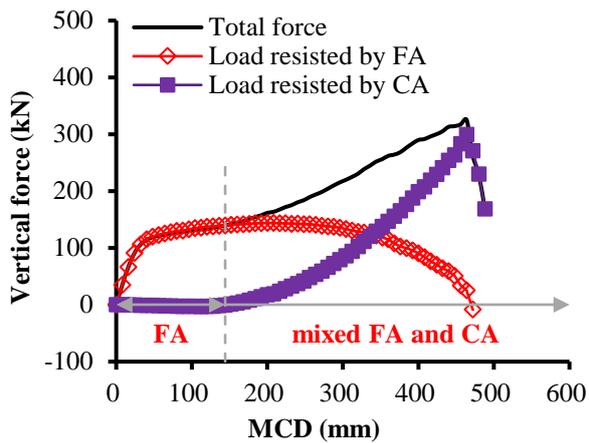
(b)



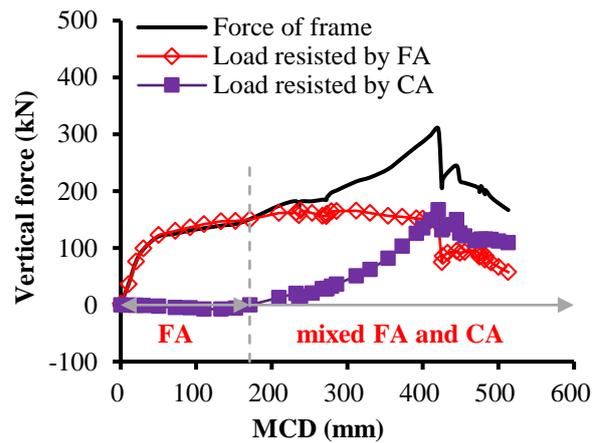
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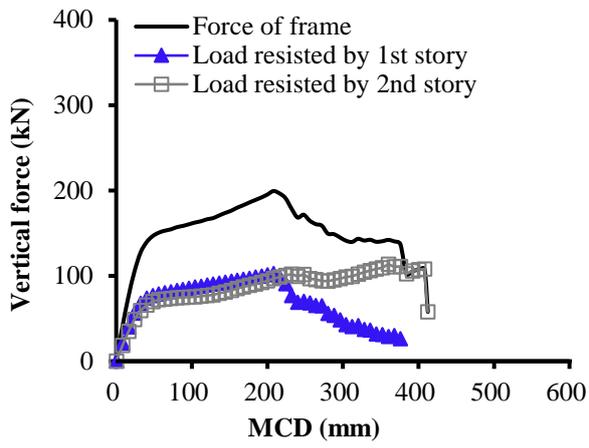
(d)



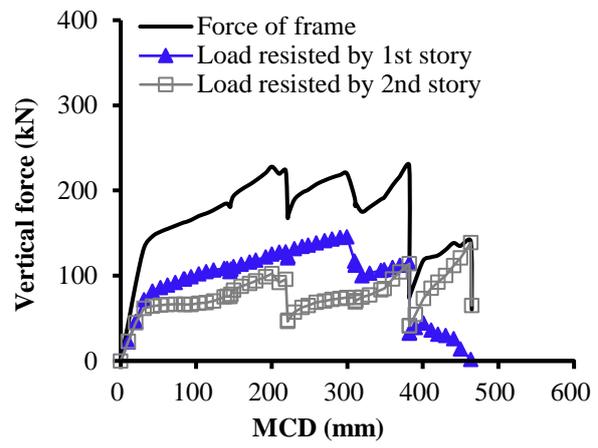
(e)



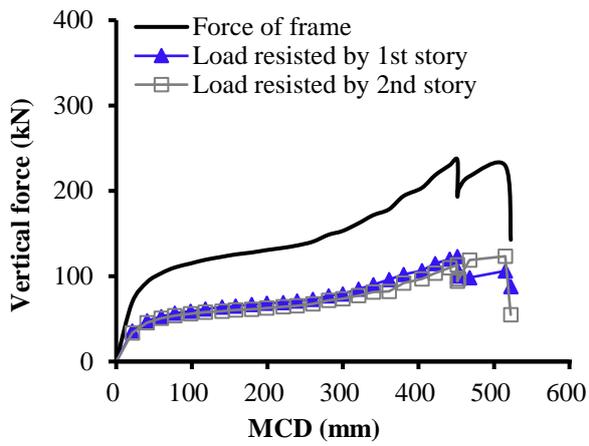
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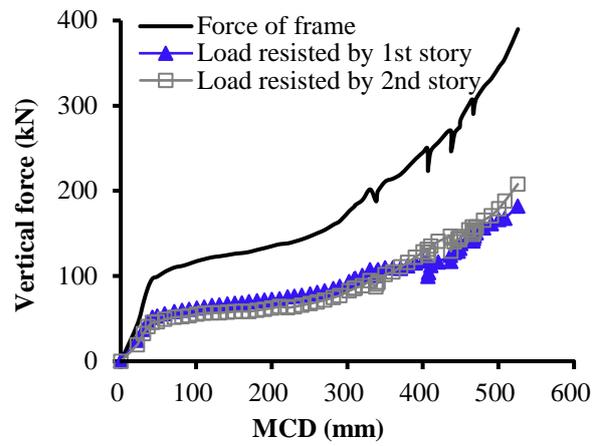
(a)



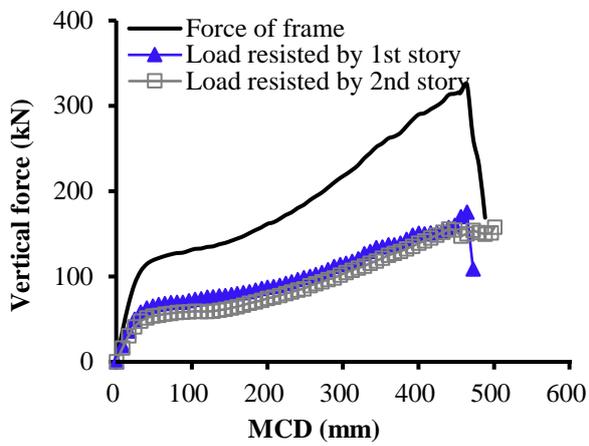
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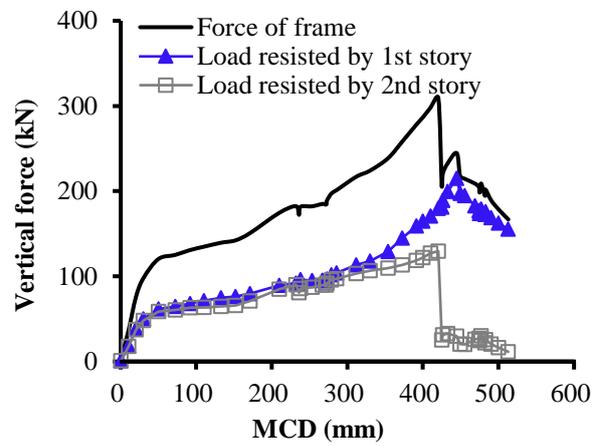
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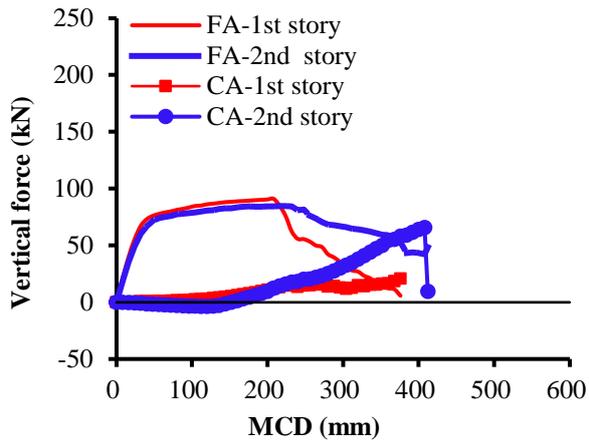
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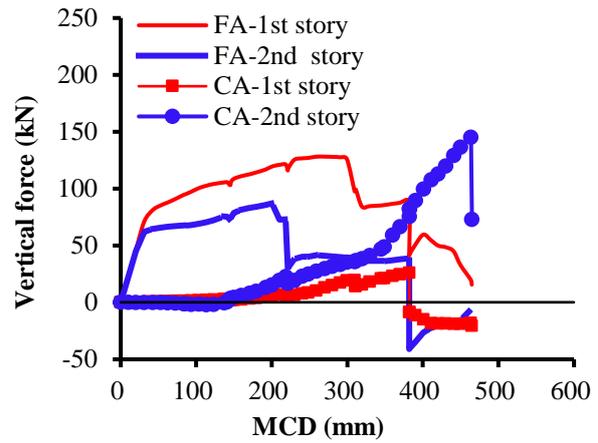
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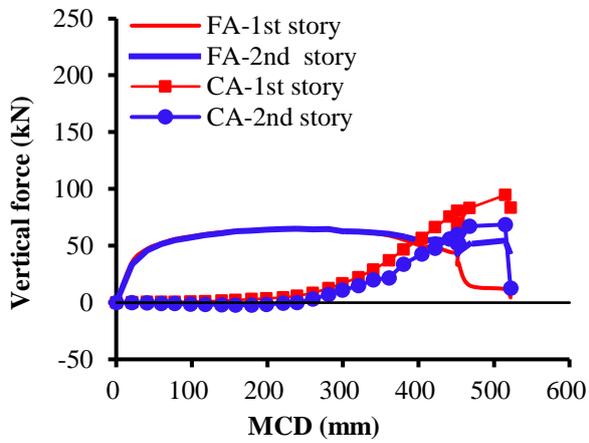
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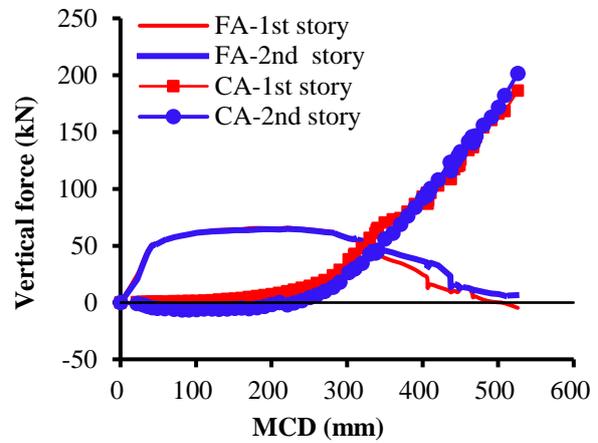
(a)



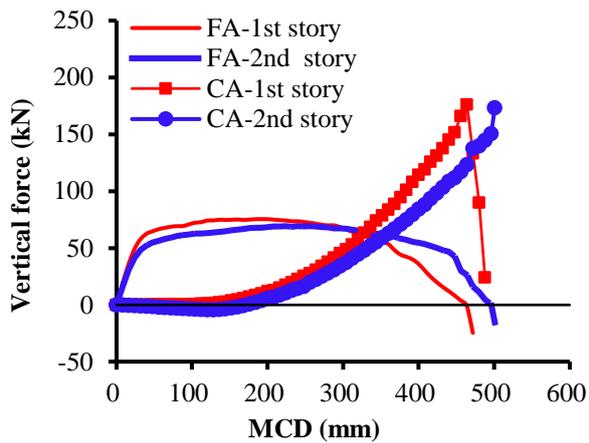
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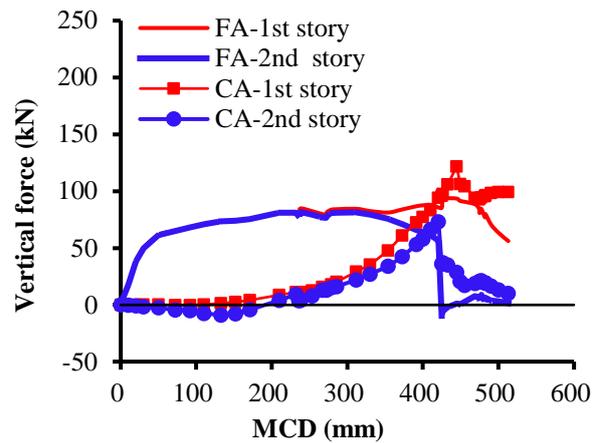
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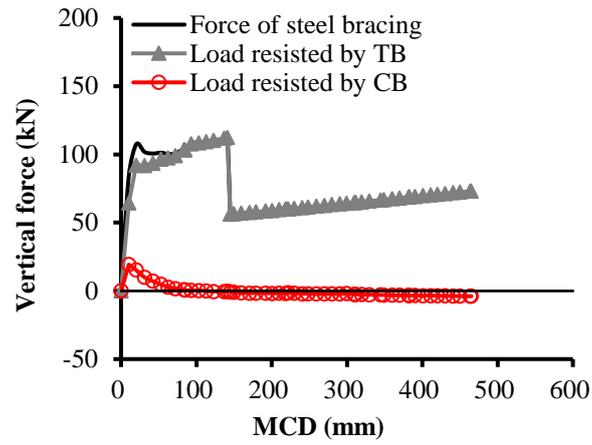
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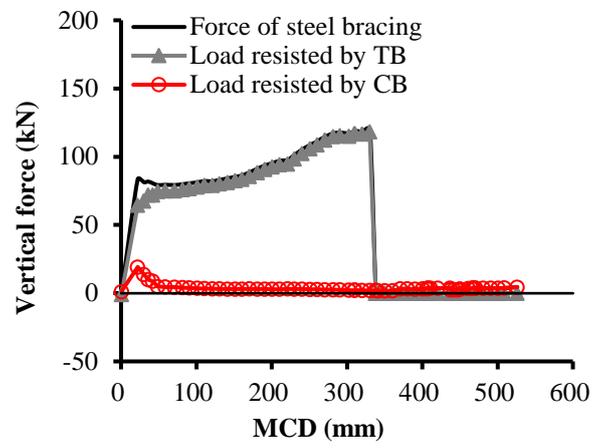
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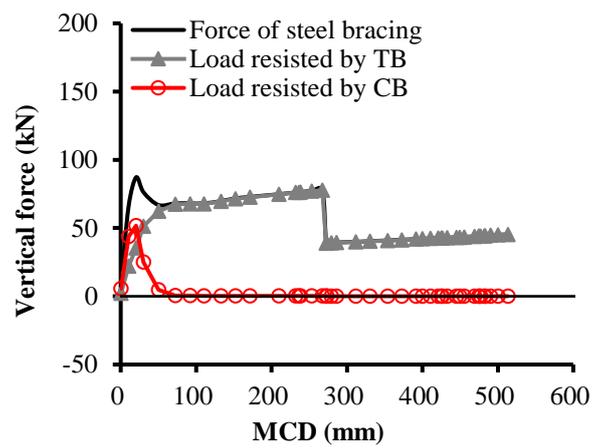
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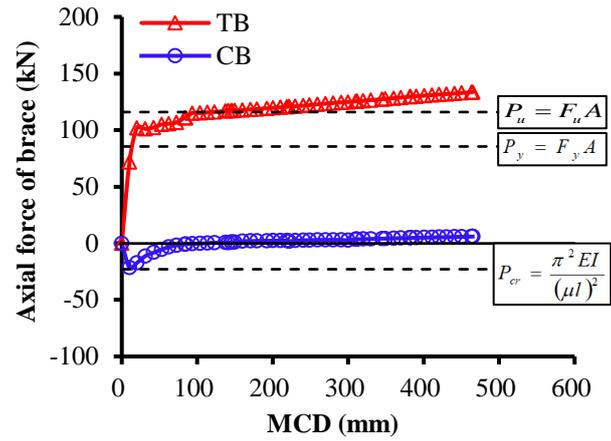
(a)



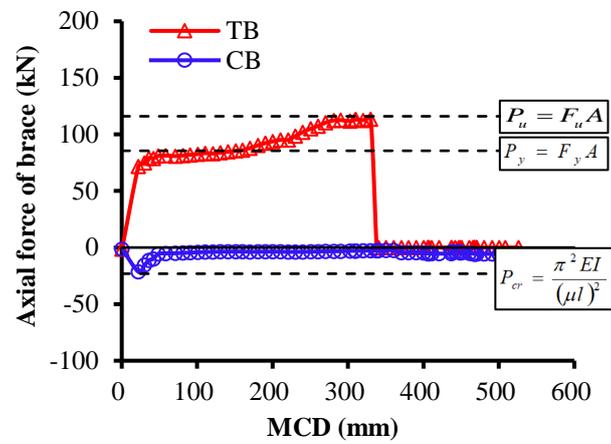
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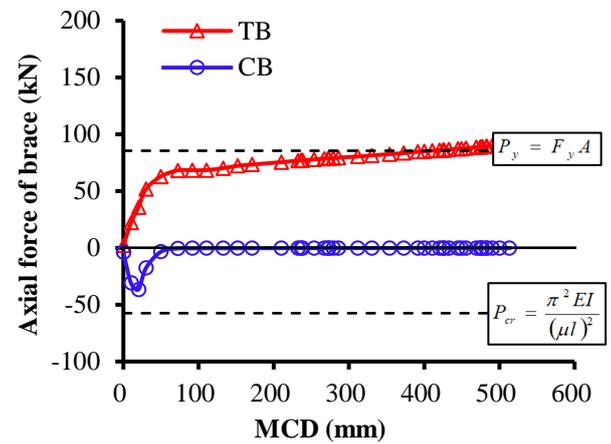
(c)



(a)



(b)



(c)