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1 **Robustness of Steel Braced Frame to Resist Disproportionate Collapse Caused by** 2 **Corner Column Removal**

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7 **Abstract:**

8 Steel braces have been widely used as a major lateral stability system to resist the lateral load. However,
9 the influences of steel braces on enhancing the load resistance in moment-resisting steel frames under
10 column removal have not been studied adequately due to the lack of experimental data, especially
11 under worst case scenarios, corner column removal. Thus, five two-floor steel moment-resisting
12 subframes with or without braces were tested by applying a pushdown force. The purpose of this study
13 is to quantify the effects of steel braces on the robustness of steel moment-resisting frames against
14 disproportionate collapse. The test results indicated that steel braces could enhance the ultimate load
15 bearing capacity up by 102.3 %. Compared to V configuration, X configuration is more efficient in
16 increasing the load resistance since a proportion of the vertical load may be transferred to the side
17 column through diagonal braces straightly. The de-composition of the load bearing capacity indicated
18 that compressive braces only affect the initial stiffness and most of the load resistance is attributed to
19 tensile braces for both X and V configurations. De-composition of the load bearing capacity indicated
20 that the load bearing capacity from the first floor is normally greater than that from the second floor
21 due to greater Vierendeel action mobilized.

22 **Author Keywords:** Disproportionate collapse; Corner column; Steel frame; Braces; Test

1. Introduction

Disproportionate collapse is an event, when the failure of the loss of one or a couple of members, results in the collapse of building disproportionate to the initial local failure. In recent years, the collapses of steel frames occurred frequently, including the landmark building twin towers, in New York in 2001, the steel frame at Xinjia Hotel building in Quanzhou, China in 2020, etc., which have received considerable attention from the public due to catastrophic consequences.

Potential hazards with abnormal load (i.e. vehicular impact, fire, gas explosion, and terrorist attack, etc.) may trigger disproportionate collapse [1-3]. Due to limited alternate load paths of the remaining structure, the removal of columns at corners is more vulnerable than other column missing scenarios. Kim and Kim [4] numerically evaluated the probability of disproportionate collapse of steel moment-resisting frames subjected to various column removal scenarios. They found that the vulnerability of disproportionate collapse was greatest when a corner column was removed suddenly. Gerasidimis [5] investigated the disproportionate collapse vulnerability of steel frames for the case of a corner column loss and developed an analytical method to capture the collapse mechanism of a steel frame under corner column-removal scenarios. Based on numerical analysis, Fu and Tan [6] studied the disproportionate collapse mechanism of composite floor systems after a corner column was removed. Compared with the results obtained in the case of internal column removal, catenary action and tensile membrane action in beams failed to develop. These studies provided insight into the disproportionate collapse of steel frames after a corner column was removed behavior associated with the removal of corner columns from steel frames. However, other studies [7-12] found that the bending moments of the beam end near the corner joints reversed after the corner column was removed, leading to a significantly bending moment (the bottom subject to tension) developed there. Based on tests on single-

46 floor beam-column sub-assemblages, most existing studies [13-16] captured the performance of multi-
47 floor steel frames under the scenario of a corner column removal by simplifying them as cantilever
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348 beams, which unrealistically ignored the interaction of structural members in different floors
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649 (Vierendeel action).

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950 However, structures are not normally designed for the catastrophic consequences provoked by
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1151 abnormal events. On the other hand, it is not economical to rehabilitate the structures just for the
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1452 purpose of increasing disproportionate collapse resistance. Thus, design engineers should be aware of
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1753 the potential vertical load resistance, which had been ignored in conventional design, such as the
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2054 additional load resistance from masonry infilled walls and steel braces. Xavier et al. [17] tested a steel
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2255 substructure incorporating infilled walls, which indicated that infilled walls affect the behavior of steel
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2556 frames significantly. Moreover, Shan et al. [18] investigated the effect of infilled walls on the load
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2857 resistance of the steel moment frames. They indicated that masonry infill walls could enhance the load
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3158 bearing capacity and initial stiffness significantly. However, they will change the failure patterns.
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3359 Seismic investigations had confirmed that the moment resisting frames with braces was an efficient
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3660 seismic resisting system with sufficient lateral load resistance and stiffness [19, 20]. However, the
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3961 ability of steel braces to improve the performance of steel frames to resist disproportionate collapse is
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41
4262 still unclear. Khandelwal et al. [21] revealed that steel braced frames which were designed to meet
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4463 seismic requirements could survive even if a column was removed suddenly, since the steel braces are
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4764 effective in providing additional load resistance. It was found that horizontal braces could be employed
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5065 to retrofit the steel moment-resisting frame [22]. It was indicated that an additional alternate load path
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5366 was formed by horizontal braces, and thus, partial loads were directly transferred to the side columns.

5567 However, experimental investigations of steel bracing systems after the removal of a corner
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5868 column were rare. Moreover, the connections types in previous numerical studies are either pinned or
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6169 fully restrained [23-25], the behavior of steel braced frames using partially restrained connections is

70 still unclear. In this experimental program, five two-floor and two-bay steel moment-resisting frames
71 were designed and tested after the removal of a corner column removal. The influence of different steel
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372 bracing configurations and connection types were quantified experimentally and analytically.
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5 673 **2. Experimental program**

7 8 974 *2.1. Test specimens*

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1175 As listed in Table 1, five test specimens including three braced frames (WX, WV, and EX) and
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1476 two bare frames without any braces (WB and EB), were designed in this experimental program. As the
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1777 main investigated parameters were the connection types and bracing configuration, the specimens were
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2078 labeled as follows: the alphabets “W” and “E” represent welded and end-plate connection, respectively.
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2279 Then, the alphabet of “B”, “X”, and “V” stand for bare frames, braced frames with X-shaped bracing,
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24
2580 braced frames with V-shaped bracings, respectively. Referring to Fig. 1, the prototype frame is designed
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2881 according to ANSI/AISC 360-05 [20]. The prototype frames had 6×6 bays with a transverse and
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3182 longitudinal span of 6.0 m and 8.4 m, respectively. The floor height is 3.0 m. The dead load and live
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3383 load were 5.1 kN/m² and 3.0 kN/m², respectively. For braced frames, the prototype frame is seismic
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35
3684 designed. Site class D was assumed and the critical acceleration parameters S_{DS} and S_{D1} are 0.20 and
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3985 0.14, respectively. For bare frames with non-seismic design, identical frames as the braced frames
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4286 except no braces were installed for comparison. As illustrated in Fig. 1, a two-floor subframe was
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4487 derived from the prototype frame as a specimen for testing. Considering the limitation of lab and facility,
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4788 only half-scale sub-frames were tested.
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5089 In contrast to the corner column without additional horizontal restraints, overhanging beam
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5390 (length of 655 mm) was fabricated beyond the side column to consider the horizontal constraints from
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5691 the interior bays, which will connect with an A-frame by horizontal chain-poles (refer to Fig. 2a). The
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5892 cross-section of beams and columns is HN 200×100×5.5×8 and HW 150×150×7×10, respectively. Fig.
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6193 3 presented the fabrication details of the specimens. For welded connection, complete joint penetration
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94 welds were used to connect the beams and columns. For end-plate connection, the beam was welded
95 to an end plate with thickness of 10 mm. Eight M18 Grade 8.8 frictional bolts are employed for bolt
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396 connection with a pre-loading force of 345 N·m. The braces and the connections were designed based
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697 on ANSI/AISC 341-05 [19]. The braces and beams are connected by the gusset plates welded to the
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998 beam flanges. The uniform force method was employed to determine the force acting on welds [26].
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1199 To avoid the gusset plate premature yield and fracture occurred in the gusset plate before braces failure,
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1400 the gusset plate is designed relatively stronger [19, 21]. Taking WX and EX as an example, the braces
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1701 were made by steel angles with a dimension of 36 × 36 × 4 mm as shown in Fig. 2a. WX and EX have
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2002 X shaped braces and the size of gusset plate is 330×125×12 mm. In addition, WV has V shaped braces.
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2203 The gusset plates installed in the second floor have a size of 160×155×12 mm while the gusset plate
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2504 installed in the first floor has a size of 510×155×12 mm.

205 2.2. Material properties

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3106 All structural members were fabricated by Grade Q235 steel. As displayed in Table 2, the critical
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3407 material properties of each component are measured via coupon tests in accordance with the relevant
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3608 specification [27]. The average value of three coupons was calculated for each set of results in this
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3909 table. The properties of M18bolts were provided by supplier.

40 410 2.3. Test setup

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4511 As illustrated in Fig. 4a, the ground corner column was not assembled to represent the initial
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4812 damage. Beneath each side column, a pin support was applied. The vertical load was imposed at the
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5113 top of corner column through a whisky jack. Displacement-controlled loading method was adopted. At
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5314 the beginning of the test, a loading rate of 5 mm/min was set until reaching the vertical displacement
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5615 of 100 mm. In the subsequent loading process, the loading rate of 10 mm/min was adopted. The applied
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5916 concentrated load was monitored by a load cell, which was placed beneath the whisky jack. To prevent
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6117 undesirable out-of-plane failure, a steel assemblage was specially arranged.

118 To represent the axial loads from upper floors, a whisky jack was applied on the side column to
119 guarantee an axial compressive ratio of 0.3. The overhanging girder, if any, was connected to the
120 reaction frame through a horizontal chain-pole. Tensile/compressive load cell was mounted in the
121 horizontal chain-pole, so that the horizontal reaction force could be monitored. To measure the vertical
122 load redistribution to the side column, each pin support was installed a load pin. Above the corner
123 column, a hinge was set to allow the conceivable rotation of the corner column during the test.
124 Moreover, two transverse beams with rollers were mounted at both sides of the corner column to
125 prevent out-of-plane movement during the test. In addition, as shown in Fig. 4a, three LVDTs were
126 mounted along the beam of first floor in the corner bay. It should be mentioned that the deflection of
127 beam between side columns was negligible during the test of Specimen WB, thus they were not
128 monitored in the following tests. As given in Fig. 2, to determine the variations of the axial forces and
129 bending moments in the beams, a series of strain gauges were attached to the critical sections.

3. Test results

131 In order to assess the robustness of steel bending-moment frames with steel braces, five two-floor
132 steel sub-frames with or without bracings were experimentally tested after a corner column loss. The
133 key results are tabulated in Table 3 and presented below.

3.1 Global behavior

135 **WB:** The load-displacement curves at the corner column of WB, WX, and WV are displayed in
136 Fig. 5. The specimens initially exhibited elastically as the load resistance increased linearly. The yield
137 load was measured to 64.7 kN at the corner column deflection (CCD) increased to 80 mm. Thus, it has
138 initial stiffness of 0.8 kN/mm. The initial stiffness was defined as the ratio of yield load to yield
139 displacement herein. The load resistance started to decrease after the occurrence of local buckling at
140 beam flanges in the first floor, which was attributed to the effects of flexural bending. At an CCD of
141 233 mm, the ultimate load bearing capacity, which was defined as the peak load resistance, of 78.2 kN

142 was reached. On further increasing the displacement, similar local buckling also occurred at beam
143 flanges in the second floor at CCD of 400 mm. Fig. 6 shows the failure pattern of the specimen.
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B44 Although no fracture occurred during the test, the local buckling was severe at the beam ends, which
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B45 resulted in the beam of the first floor failing in torsion in absence of catenary action.
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B46 **WX:** As the second floor has X braces, the failure pattern of WX was changed. In the start of
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11 testing, the compressive braces started to buckle, which indirectly showed that the compressive braces
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14 may have little contribution to the load enhancement. This can be confirmed by the strain gauge results
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17 later. As displayed in Fig. 5, it has a yield load of 116.5 kN at an CCD of 62 mm. Moreover, it has
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20 initial stiffness of 1.9 kN/mm, about 232.3% of that of WB. Similar to WB, local buckling of the bottom
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23 beam flange occurred near the side column in the ground floor at this stage. It has an ultimate load
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25 bearing capacity of 125.4kN. The tensile brace fractured at an CCD of 288 mm, which leads to the load
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28 resistance dramatically dropping from 74.9 kN to 24.3 kN. Fig. 7 gives the failure pattern of the
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31 specimen. Tensile braces were fractured and compressive braces were severely buckled. No yielding
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34 was observed in the gusset plates. Similar to WB, torsional damage occurred in the beam in the first
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37 floor. However, premature weld fractures occurred in Joints S1, S2 and C2 of WX, which were not
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40 observed in WB. This was due to the additional shear forces from the steel braces and the torsion-
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42 induced shear forces in the beams.
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46 **WV:** For WV, it has a yield load of 93.5 kN and initial stiffness of 1.9 kN/mm. Similar to WX,
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49 the compressive braces began to out-of-plane buckling at the very beginning of the load. Different from
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52 WX, the tensile brace fractured at a relatively early stage (corresponding to CCD of 63 mm) and
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55 followed by the drop in load resistance dramatically. Moreover, it has an ultimate load bearing capacity
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58 of 100.9 kN, which is 129.0% and 80.5% of that of WB and WX, respectively. When the fracture
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60
61 occurred at the beam ends, the load resistance dropped significantly. After reaching the CCD of 191
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63
64 mm, the load bearing capacity of WV became even lower than that of WB. Fig. 8 displays the failure
65

166 pattern of WV. Similar to WX, the compressive braces suffered severe buckling and the tensile braces
167 were fractured. However, different from WX, there is no torsional damage occurring in the beams with
168 gusset plate welded to the beam flange were enough to prevent torsional buckling.

169 **EB:** Fig. 9 compares the load-displacement curves of EB and EX. EB achieved a yield load of
170 42.7 kN and initial stiffness of 0.7 kN/mm. When the CCD reached 135 mm, the welds nearby the side
171 column in the second floor fractured. Subsequently, the load bearing capacity could increase until
172 further weld fracture occurred at the end-plate. It has an ultimate load bearing capacity of 55.5 kN.
173 Similar fractures occurred at the beam ends near the corner column at CCD of 406 mm and 487 mm,
174 respectively. Fig. 10 gives the failure pattern of the specimen. The failure of EB was controlled by weld
175 failure at the end-plate.

176 **EX:** It has a yield load of 86.4 kN and initial stiffness of 1.9 kN/mm. The weld fracture was
177 initially developed nearby the corner column. With the displacement kept increasing to 128mm, EX
178 reached its ultimate load bearing capacity of 105.8 kN. When CCD increased to 159 mm and 336 mm,
179 the welds fracture was formed at the beam ends in sequence. The tensile braces fractured at an CCD of
180 602 mm, which was 109.0% later than WX. Fig. 11 gives the failure pattern of EX. Similarly, the
181 compressive brace suffered severe out-of-plane buckling while the tensile brace fractured. Moreover,
182 local buckling occurred at point A.

183 3.2 Deformation measurements

184 The deformation shape of beams at different stages is displayed in Fig. 12. Following DoD [28],
185 the chord rotation was defined as the ratio of CCD to beam span. From the figure, the chord rotation
186 would significantly underestimate the rotation of the beam end nearby the corner column. On the other
187 hand, the chord rotation could assess the rotation of the beam end nearby the side column accurately,
188 especially for WX and WV. The external steel brace would not significantly change the deflection
189 shape of the beams. Similar observation was achieved in EB and EX.

190 3.3 Internal force evaluation

191 To deeply understand the contribution of load resistance from braces, the contribution of braces
192 and frames should be determined individually. Before that, the reliability of calculation formula to
193 determine the internal force of each component based on strain gauge results must be verified. From
194 Fig. 2a, Sections B1-4 installed a series of strain gauges, which could help to determine the internal
195 force. Similar to the calculation method proposed in the previous paper [29, 30], the load-displacement
196 curve based on strain gauge data was determined and compared with the one from load cell results
197 (refer to Fig. 13). As shown in Fig. 13, good agreements are achieved between the one measured by
198 the load cell and analytical results from strain gauge data. For WX and EX, the minor discrepancy of
199 initial stiffness may be caused by unavoidable gaps in the test setup, which did not reflect in the LVDT
200 placed above the corner column. Generally, the analytical results based on strain gauges can well
201 capture the character of the curve until failure.

202 Fig. 14 presents the load bearing capacity of the braces and frame. Relying on the analytical results
203 of internal force, it was revealed that the load bearing capacity from the bare frame was purely provided
204 by flexural action, the contribution of catenary action could be ignored due to the limited tensile forces
205 in the beams. For WX, at the ultimate load resistance stage, the contribution of steel braces was about
206 35.3%. With the increasing vertical displacement, the load resistance of the steel braces began to
207 decrease due to the yield of beam section releasing the constraints for the tensile braces gradually. For
208 WV, at the stage of ultimate load bearing capacity, the contribution of steel bracing was only 27.7% as
209 the tensile braces fractured. Different from the braced frames with welded connection, for EX, initially,
210 the steel braces contributed greater load resistance than the frame. At the stage of ultimate load
211 resistance, the contribution of steel braces was 45.7%, which was comparable to that of the frame.
212 Moreover, after that, the load resistance from braces is always comparable to that from frames until the
213 test final.

214 4. Discussion of experimental results

215 4.1 Contribution of load resistance

216 The de-composition of the frame contribution from each floor is given in Fig. 15. As shown in the
217 figures, the load bearing capacity from the first and second floor has similar trends until failure occurred
218 in the connections, while the first floor has slightly greater load resistance than that of second floor.
219 This was due to the greater rotational constraints and Vierendeel action, for which further explanation
220 would be in section 4.2. However, WX had a greater maximum load resistance from the second floor
221 as the connection in the first floor fractured earlier.

222 Fig. 16 displays the de-composition of load bearing capacity contribution from tensile brace and
223 compressive brace. For WB, WX, and WV, the maximum load resistance from steel braces was 46.4,
224 29.7, and 49.8 kN, respectively. Different from WX and EX, which reached their maximum load
225 resistance until the tensile brace yielded, the maximum load resistance of WV was obtained when the
226 compressive brace buckled. For WV, the compressive brace contributed the maximum load resistance
227 of 14.5 kN, which was 33.7% and 41.2% higher than that of WX and EX, respectively. To better
228 understand the contribution of steel braces, the development of axial force of tensile brace and
229 compressive brace was normalized, as shown in Fig. 17. The tensile brace has an analytical yield load
230 of 85.6 kN. And the compressive brace has buckling loads of 23.0 kN and 57.6 kN for X and V
231 configuration, respectively. As can be seen in the figure, both the tensile and compressive brace in X
232 configuration achieved their yield and buckling loads. However, in V configuration, the tensile brace
233 could not achieve the analytical yield load and the compressive brace could not reach the analytical
234 buckling load as the constraints applied at the braces of V configuration were not translation fixed,
235 which is assumed in analytical analysis.

236 4.2 Effects of connection types

237 Unlike braced frames where the maximum deformation would be introduced by the failure of the

238 steel bracing, the maximum deformation of bare frames difficult to identify from the load-displacement
 239 curve. The deformation capacities of bare frame were defined as vertical displacement at ultimate load
 1 bearing capacity. For WB and EB, the maximum deformation was 233 and 228 mm, respectively. Thus,
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 240 WB and EB had a similar maximum deformation, which was quite different from the case of missing
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 241 a middle column [29, 30]. WB was able to sustain large deformation caused by the torsion developed
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 242 in beams, and avoided the brittle weld fracture. Fig. 18 shows the development of bending moments at
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 243 the beam ends. From the figure, not only the beam end near the side column, the beam end near the
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 244 corner column also experienced large positive bending moments, which was different from the
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 245 behavior of unsupported cantilevers. This indicated that Vierendeel action played an important role in
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 246 the load resisting mechanism after corner column removal. Moreover, DoD [28] defined the acceptable
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 247 plastic rotation angles for different type of steel connections. Taking Section 1B1 as an example, the
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 248 parameters ‘a’ and ‘b’ were defined as shown in Fig. 18. Table 4 compared the measured parameters
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 249 with the requirements in specification, which showed that the recommended ductility acceptance
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 250 criteria were conservative for both fully and partially restrained moment connections in current
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 251 specimens.

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 3253 Although this experimental test was focused on the static performance of welded and endplate
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 254 connections, disproportionate collapse following sudden column removal exhibits a typical dynamic
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 44 response. Based on the available static load-displacement curve, a dynamic capacity evaluation was
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 46
 256 applied using an energy-based method proposed by Izzuddin et al. [31]. This approach has been applied
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 257 and verified in previous studies related to disproportionate collapse [32-34]. As described in Eq. (1),
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 258 the equivalence between external work (dynamic response) and internal energy (static response) was
 53
 54
 259 used to obtain the dynamic response shown in Fig. 19.

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

261 As labeled in Fig. 19, the maximum dynamic load capacities of WB, EB, WX, and EX were 65.5,
262 45.8, 106.7, and 93.2 kN, respectively. Prior to this point being reached, it was unlikely for the
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263 specimens to undergo disproportionate collapse even if a sudden column failure occurred. The
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264 maximum dynamic deformations of WB, WX and WV were 380, 380, 200 and 320 mm, which were
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265 163%, 194%, 167% and 250% of that of maximum static deformations, respectively. This implied that
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266 the deformation capacity of the frames under sudden column removal were greater than those under
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267 quasi-static loading for the same applied load.

268 4.3 Effects of braces types

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269 As seen from Fig. 5, the yield load of WB, WX, and WV were 64.7, 116.5 and 93.5 kN,
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270 respectively. Therefore, the X and V braces enhanced the yield load of WB by 80.1% and 44.5%,
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271 respectively as the X bracing configuration in WB can directly transfer a portion of the load to the side
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272 column. Regarding the ultimate load bearing capacity, WB, WX and WV were 78.2, 125.4 and 100.9
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30
273 kN, respectively. Therefore, the X and V bracing configuration enhanced the ultimate load bearing
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274 capacity of WB by 60.4% and 29.0%, respectively. This could be attributed into the tensile braces in V
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275 configuration fractured much earlier, even earlier than the yielding of the beams. Moreover, the
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276 maximum deformation of WB, WX and WV were 233, 288 and 63mm, respectively. Thus, the steel
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277 bracing with X configuration increased the maximum deformation of WB by 23.6%, while the steel
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278 bracing with V configuration decreased the maximum deformation of WB by 73.0%. The maximum
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279 deformation of WX was 457.1% of that of WV. Regarding to failure patterns, as shown in Fig.12,
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280 torsional damage occurred at interfaces between the beam and corner column joint of WB and WX [35,
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281 36]. However, due to the gusset plate of V bracing configuration, the critical beam section shifted close
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282 to the beam midspan, which prevented the torsional damage of WV [37].
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283 5. Conclusions

284 A series of five two-bay and two-floor steel sub-frames were tested subjected to a corner column

285 missing scenario to investigate the robustness of steel braced frames to resist disproportionate collapse.

286 Following conclusions are obtained:

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287 1. Steel braces could increase the initial stiffness and load resistance of steel frames whatever X
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288 configuration or V configuration was adopted. However, torsion may control the failure of bare
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289 frame, while the braces also amplified the shear force demand at the beam ends. Therefore, it is
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1290 necessary to consider the torsional shear forces of adjacent structures where corner column is
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1291 removed in the alternate load path method. The connection design can be controlled by the shear
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1292 forces generated by torsion, especially for braced frame.
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293 2. Experimental results indicated that the tensile braces in steel frames with end-plate connections
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294 did not fracture until the vertical deformation reached 20% of the beam span. X configuration
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295 performed better than V configuration regarding ultimate load bearing capacity and initial stiffness,
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296 as X configurations could transfer partial of the load to the side column directly while V
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297 configuration are only subjected to axial forces within the elastic range. Whatever X or V
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298 configurations, the compressive braces only affect the initial stiffness as they were severely
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299 buckled from the very early beginning of the test.
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300 3. Different from the scenario of loss of an interior column, the bare frame with weld connection
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301 achieved a similar maximum deformation than that of steel frame with end-plate connections when
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302 the scenario of corner column missing was concerned. Moreover, the value of plastic hinge
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303 properties was too conservative for the bare frame with weld connection and end-plate connection
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304 under corner column removal.
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305 4. Analytical analysis found that the load resistance of the frame in the ground floor is generally
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306 larger than that in the second floor because the structural components in the second floor could
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307 provide horizontal constraints to the joints in the ground floor, in other words, greater Vierendeel
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308 action was mobilized in the frames in the ground floor.
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313 do not necessary reflect the view of National Natural Science Foundation of China and Natural Science
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10
11
314 Foundation of Guangxi.
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415
416
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418 **Table 1.** Summary of specimen

Specimen ID	Connection	Bracing configurations
WB	Welded	N/A
WX	Welded	X-shaped braces
WV	Welded	V-shaped braces
EB	End-plate	N/A
EX	End-plate	X-shaped braces

419
420 **Table 2.** Material properties

Items	Plate thickness (mm)	Yield strength (MPa)	Yield strain	Ultimate strength (MPa)	Ultimate strain	Elongation (%)
Beam flange	8	310	0.0019	420	0.024	12
Beam web	5.5	320	0.0021	430	0.0249	13.5
Column flange	10	300	0.0019	410	0.0267	14
Column web	7	295	0.0023	375	0.0242	13
Steel brace	4	310	0.0021	420	0.0256	12.5

421
422 **Table 3.** Test results

Test ID	U_{YL} (mm)	F_{YL} (kN)	K_{YL} (kN/mm)	U_{PL} (mm)	F_{PL} (kN)
WB	80	64.7	0.8	233	78.2
WX	62	116.5	1.9	103	125.4

WV	49	93.5	1.9	63	100.9
EB	60	42.7	0.7	228	55.5
EX	46	86.4	1.9	128	105.8

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

Table 4. Comparison of the measured and recommended plastic hinge parameters in DoD [28]

Test ID	Section	'a' at the beam end (rad)	'a' in DoD [28] (rad)	'b' at the beam end (rad)	'b' in DoD [28] (rad)
WB	1B1	0.070	0.025	N/A	0.038
	1B5	0.073	0.025	N/A	0.038
	2B1	0.118	0.025	N/A	0.038
	2B5	0.093	0.025	N/A	0.038
	EB	1B1	0.050	0.012	0.109
EB	1B5	0.063	0.012	0.084	0.018
	2B1	0.094	0.012	0.136	0.018
	2B5	0.026	0.012	0.061	0.018

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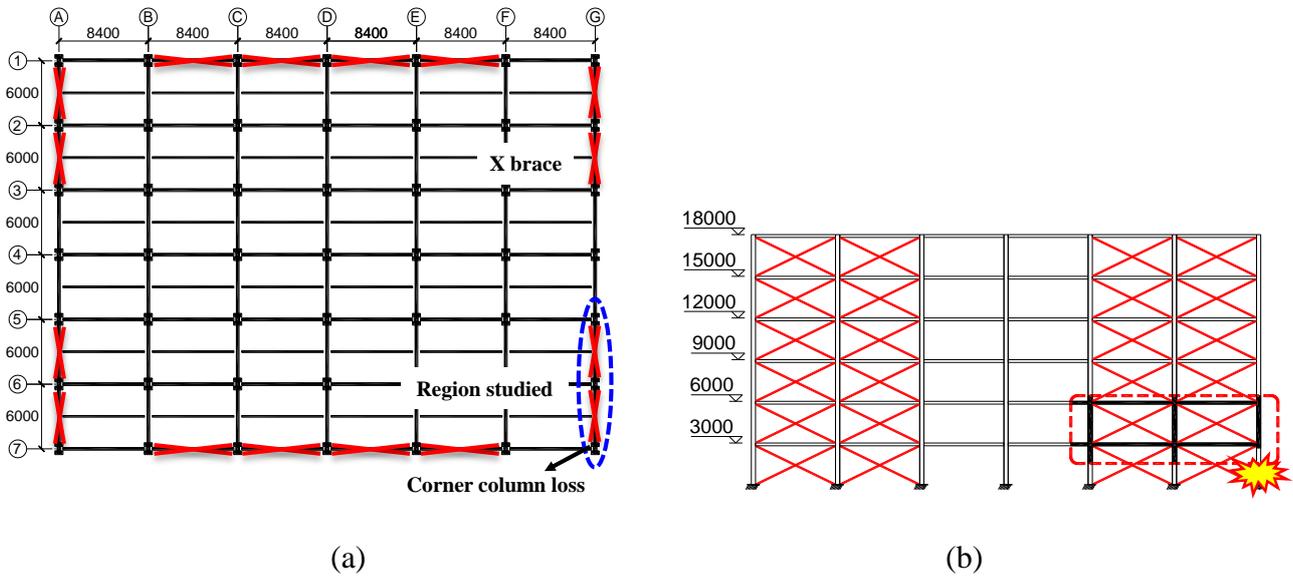
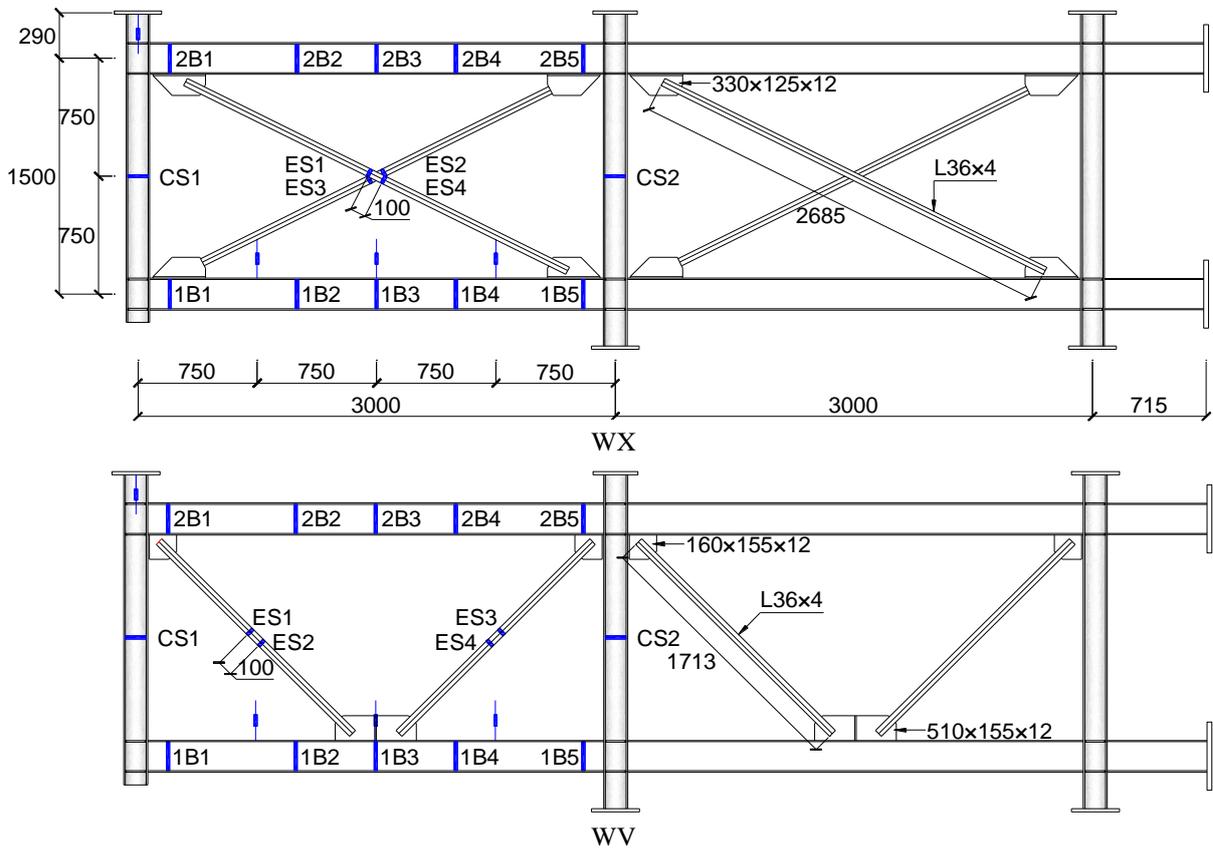


Fig. 1. Prototype building and extracted frame (unit in mm): (a) front view; (b) side view

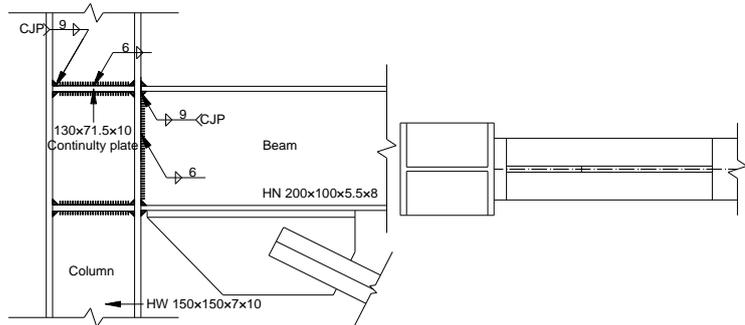


(a)

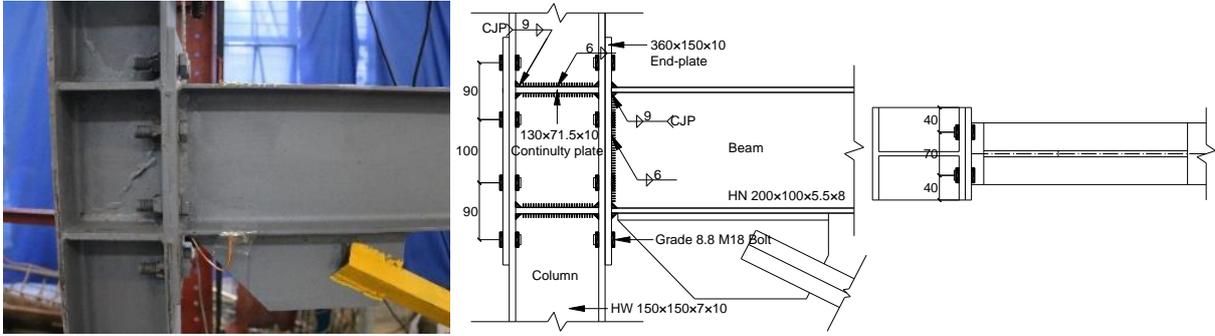


(b)

Fig. 2. Dimensions of the specimen and locations of instrumentations: (a) layout of strain gauge and displacement transducer; (b) position of strain gauges on sections

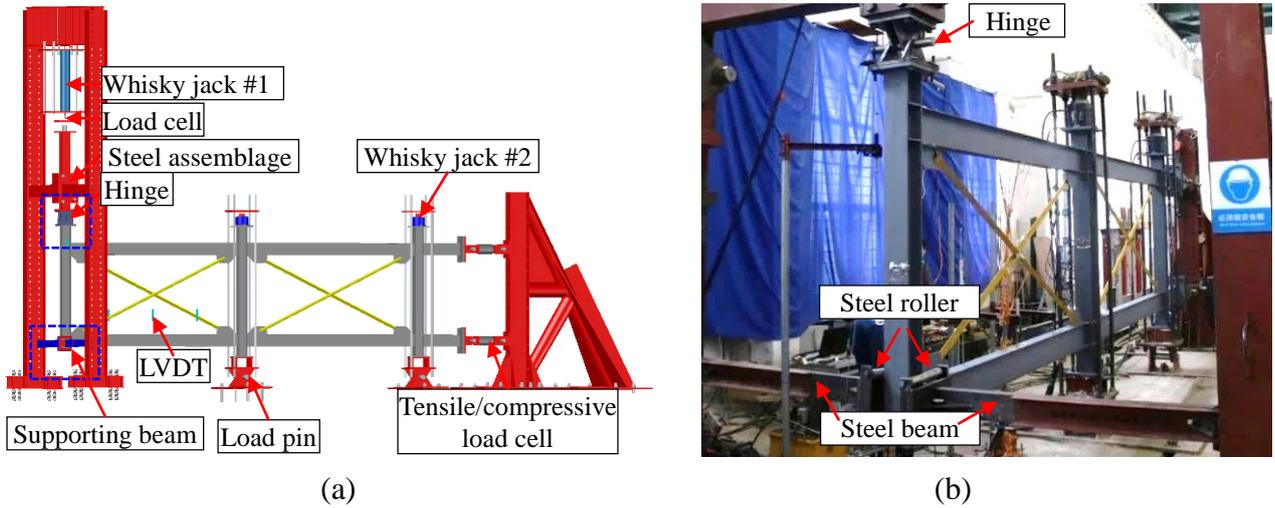


(a)



(b)

Fig. 3. Details of the connections: (a) welded connection; (b) end-plate connection



(a)

(b)

Fig. 4. Test setups of WX: (a) drawing; (b) photo

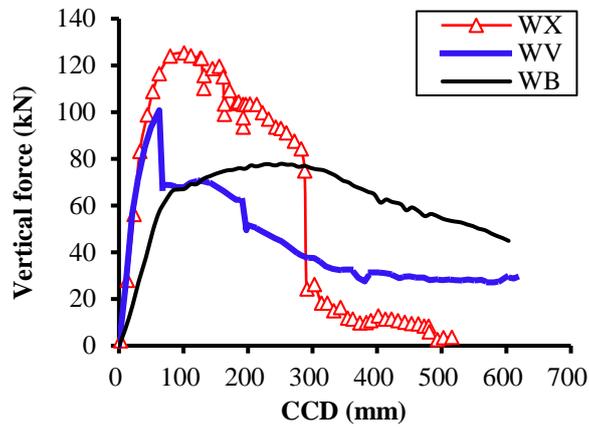


Fig. 5. Load-displacement curves of specimens: WB, WX and WV

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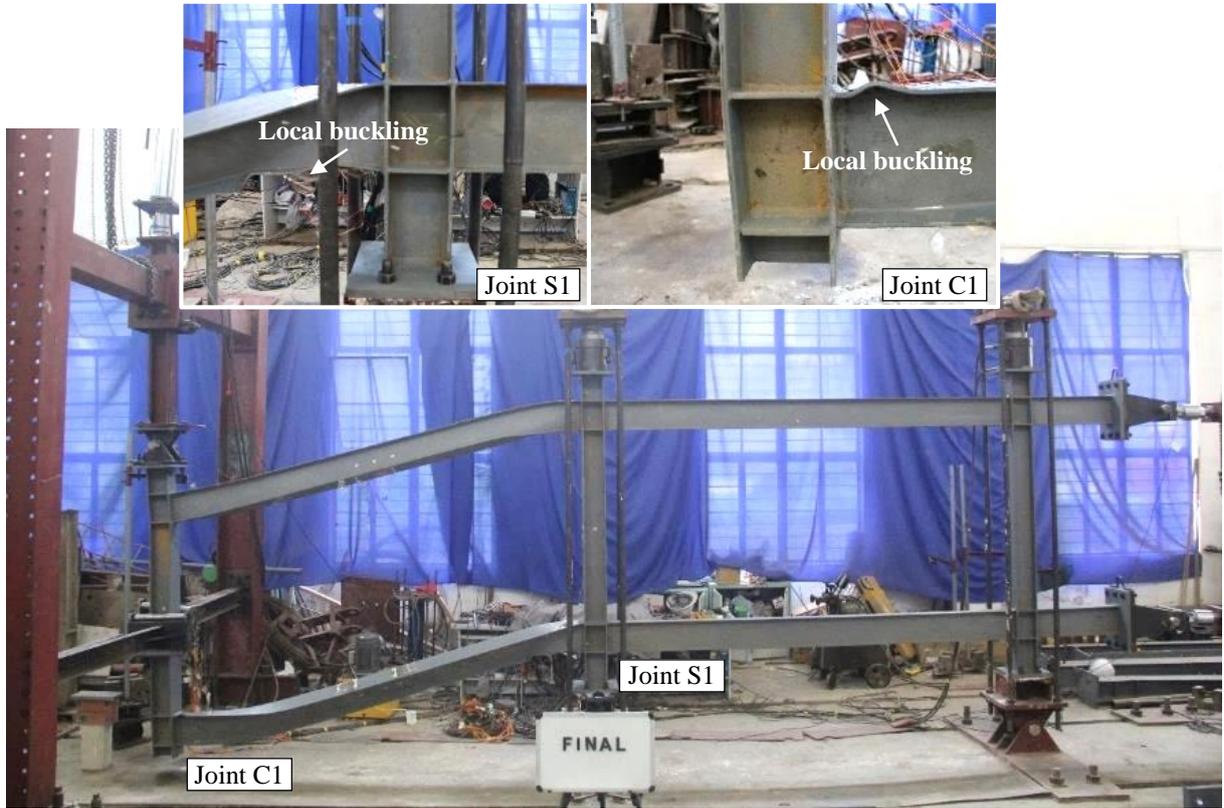


Fig. 6. Failure pattern of Specimen WB

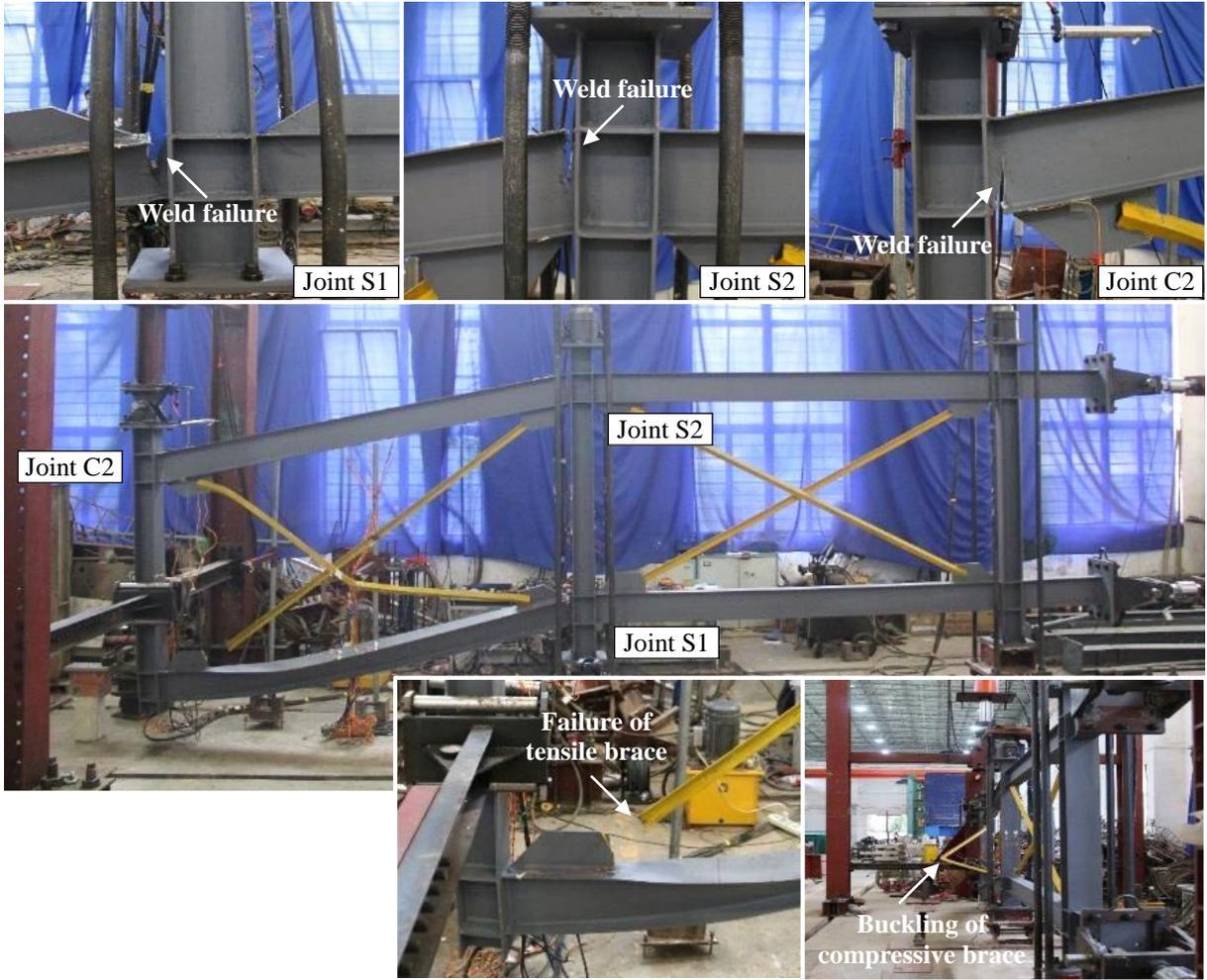


Fig. 7. Failure pattern of Specimen WX

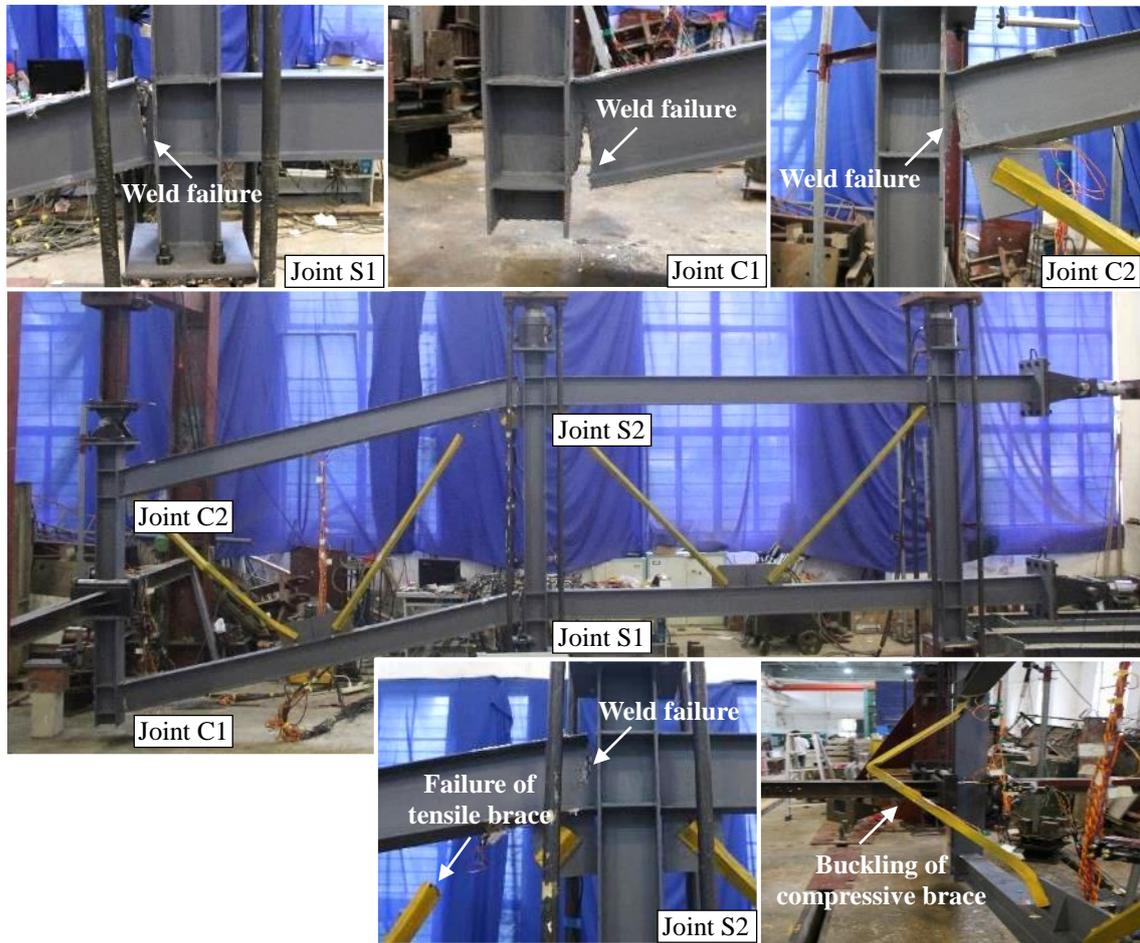


Fig. 8. Failure pattern of Specimen WV

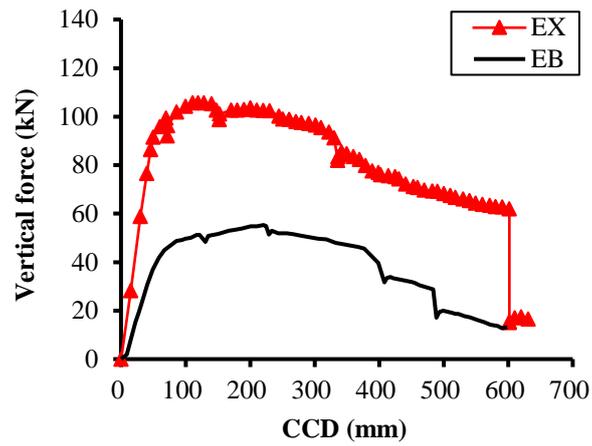


Fig. 9. Load-displacement curves of specimens: EB and EX

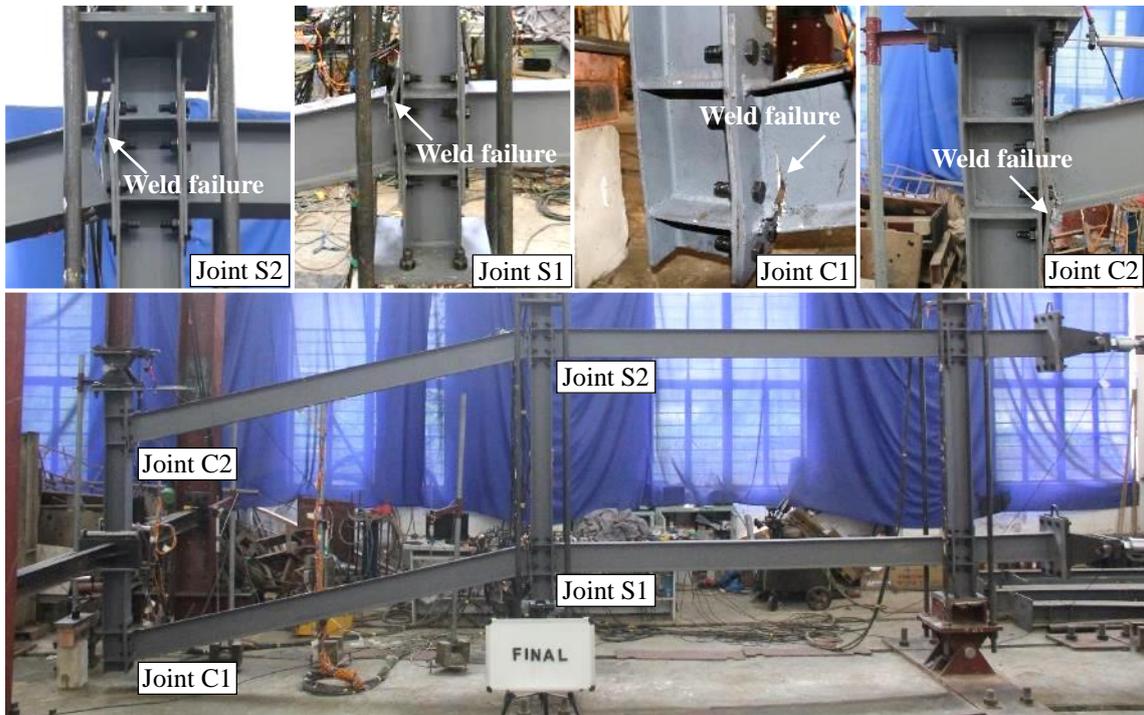


Fig. 10. Failure pattern of Specimen EB

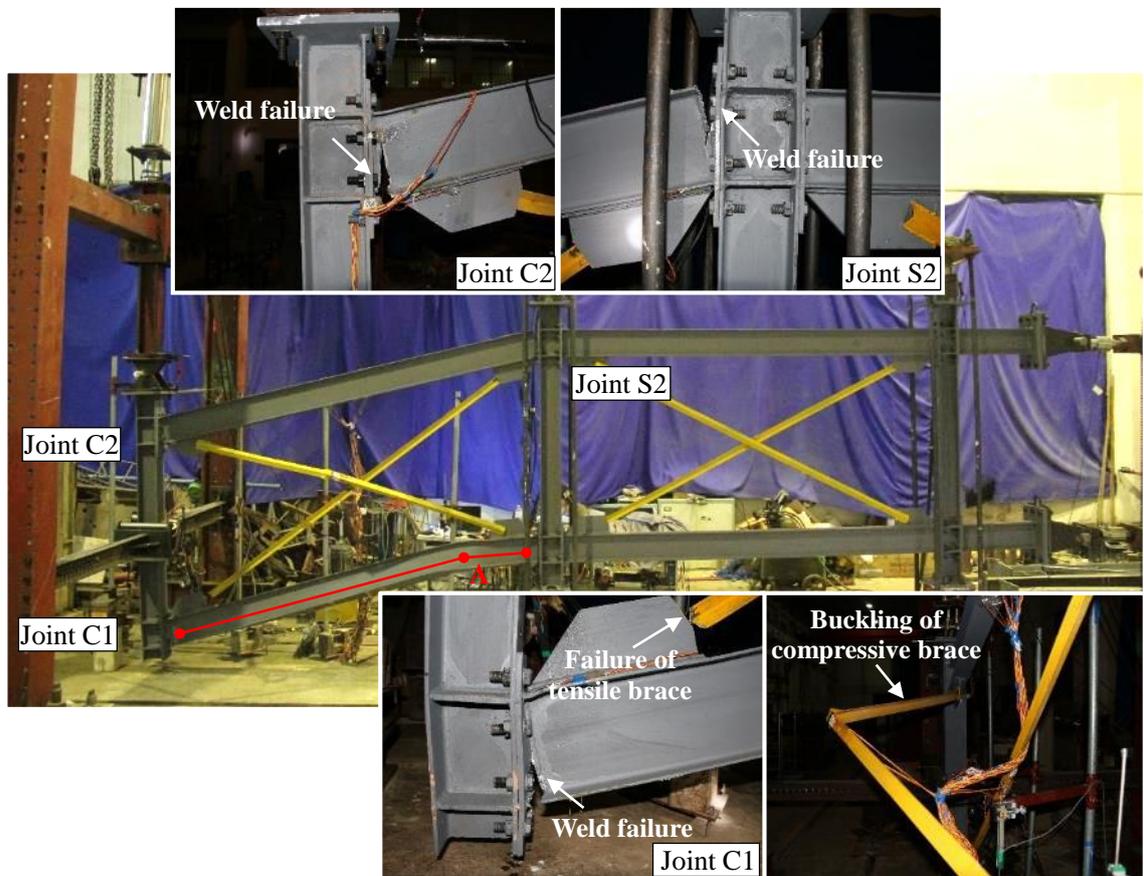
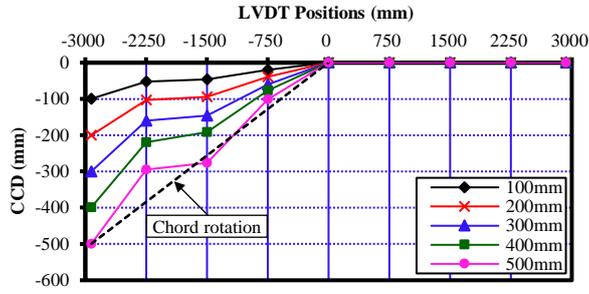
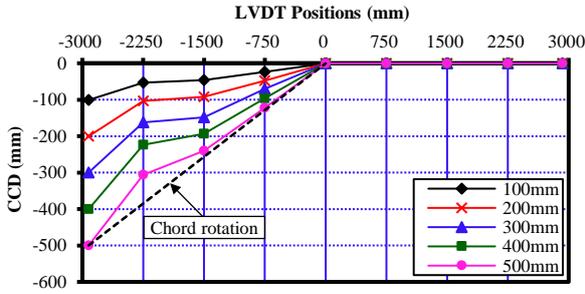


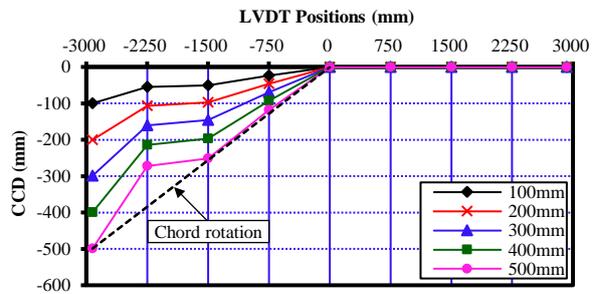
Fig. 11. Failure pattern of Specimen EX



(a)

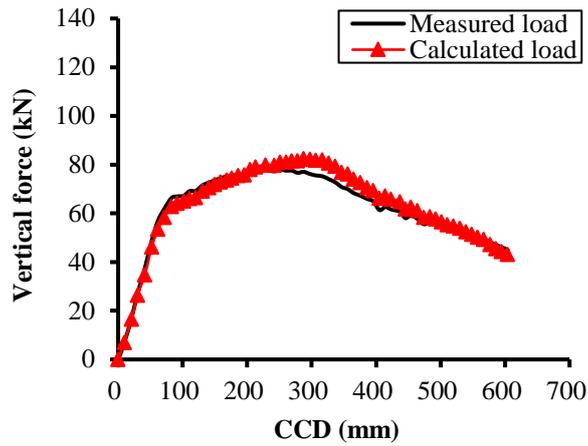


(b)

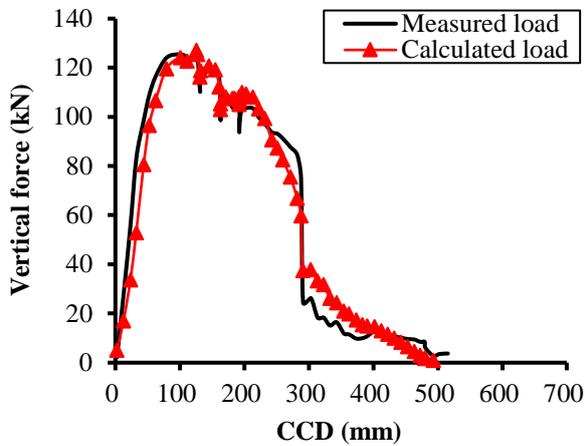


(c)

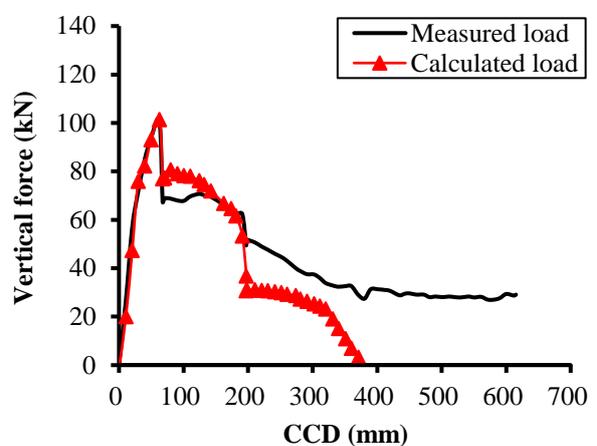
Fig. 12. Deflection profile of the beams in different stages: (a) WB; (b) WX; (c) WV



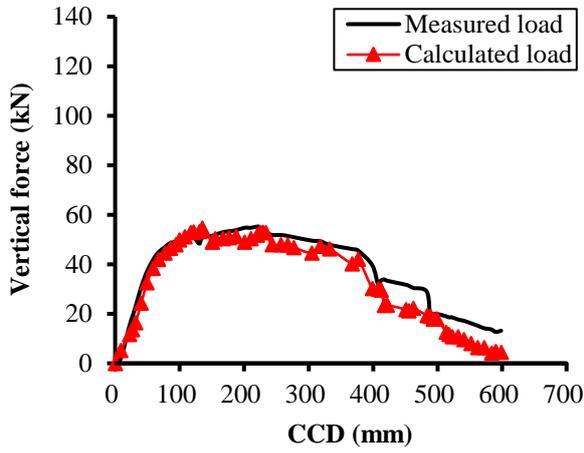
(a)



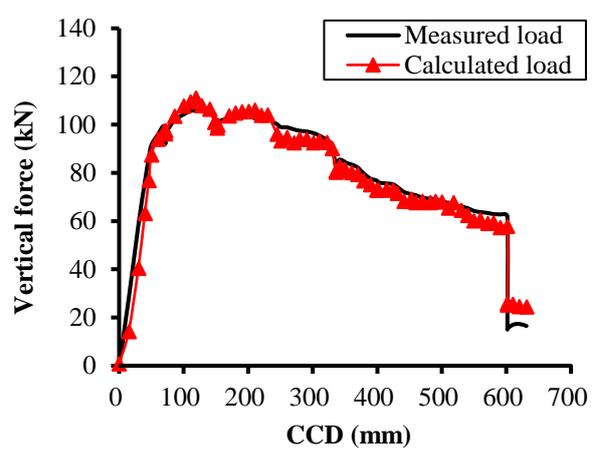
(b)



(c)

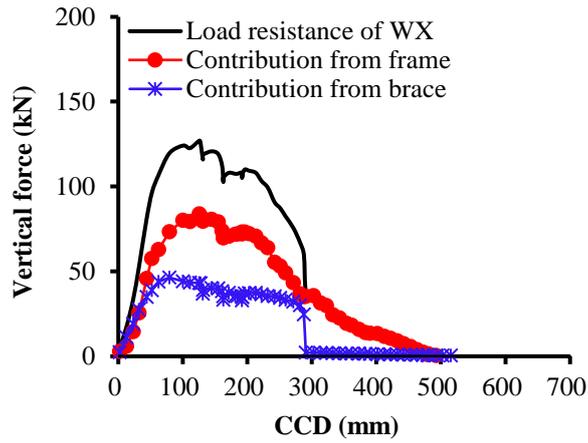


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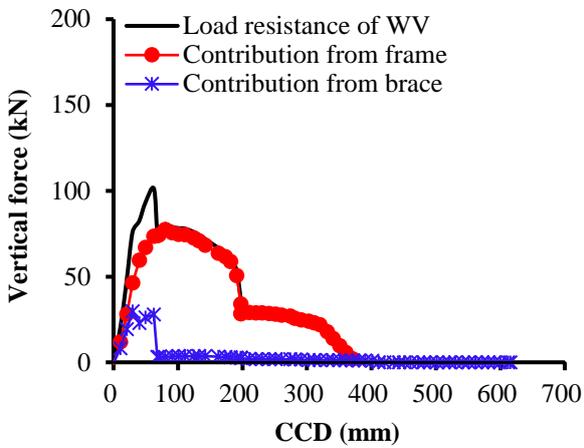


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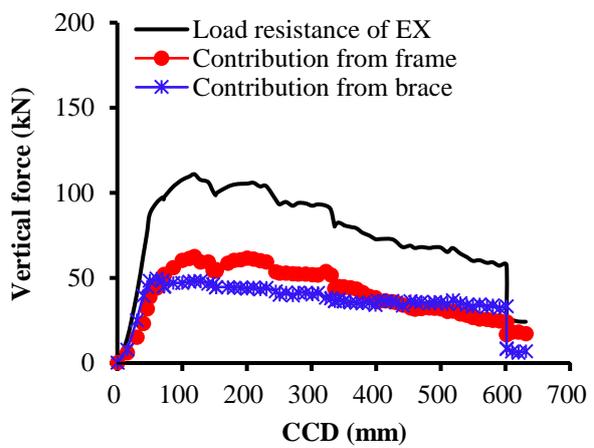
Fig. 13. Load-displacement curves from strain gauge and load cells: (a) WB; (b) WX; (c) WV; (d) EB; (e) EX



(a)

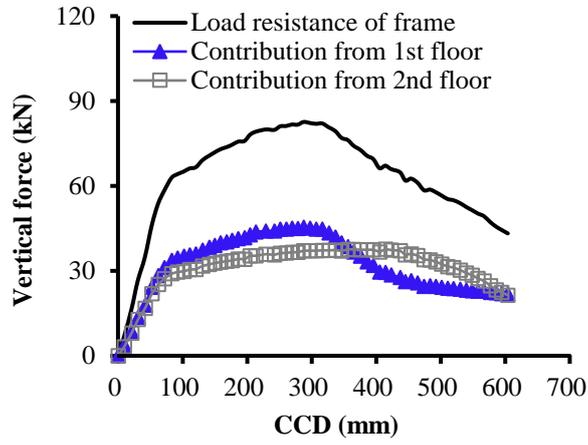


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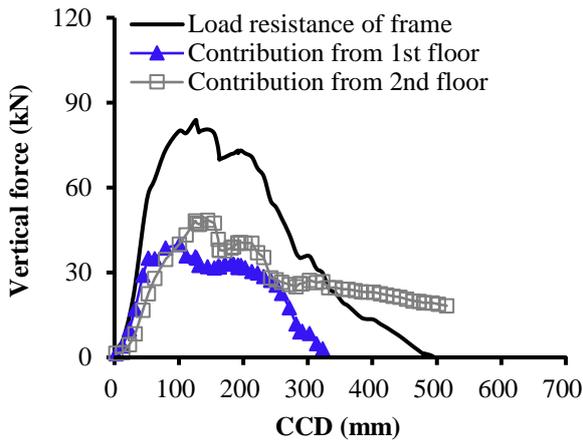


(c)

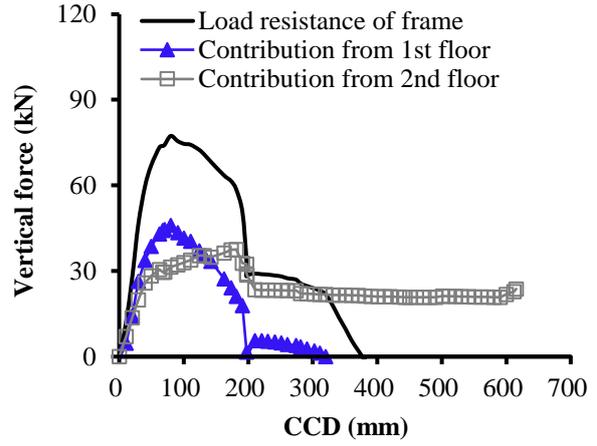
Fig. 14. De-composition of the load bearing capacity from frame and braces



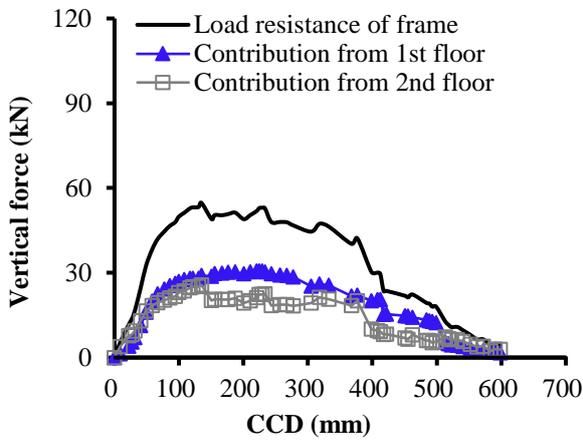
(a)



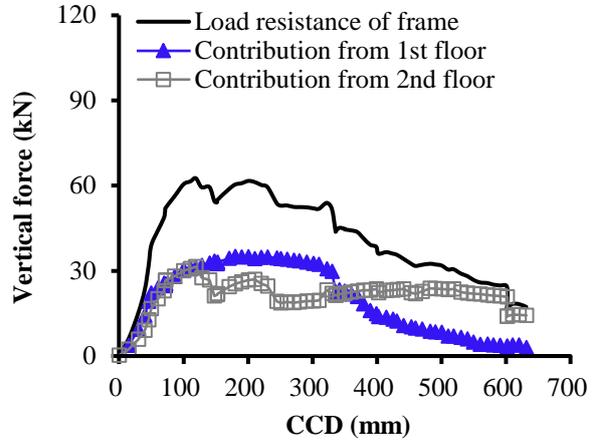
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(c)

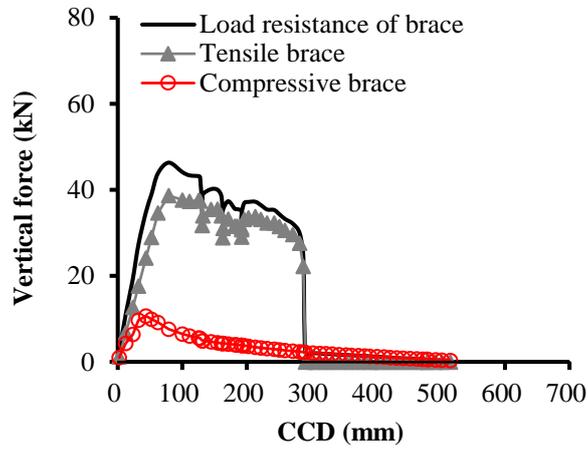


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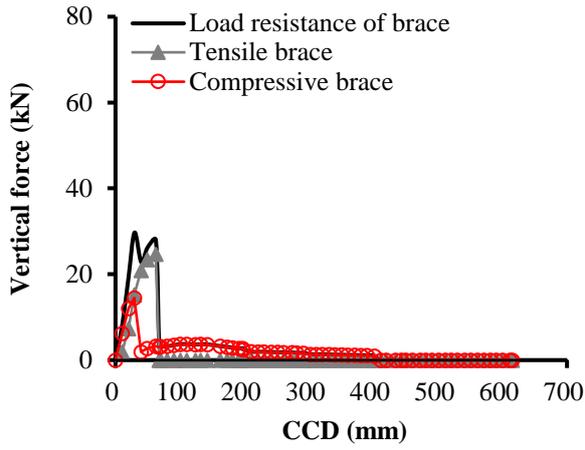


(e)

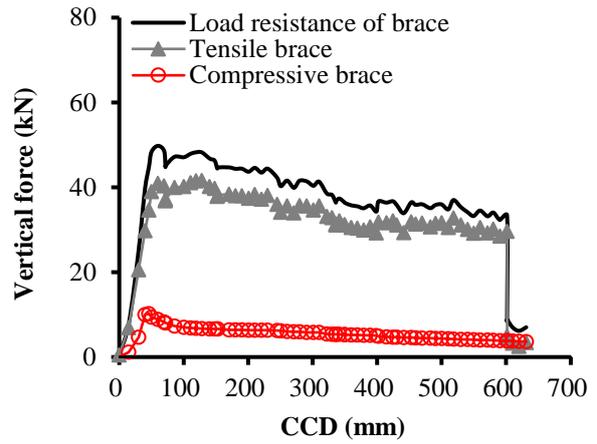
Fig. 15. De-composition of load bearing capacity from the 1st floor and 2nd floor



(a)

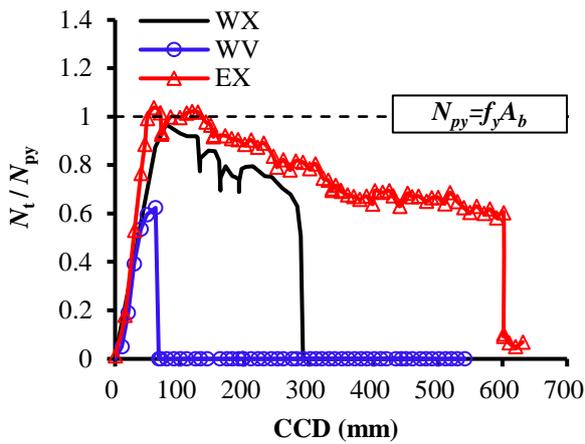


(b)

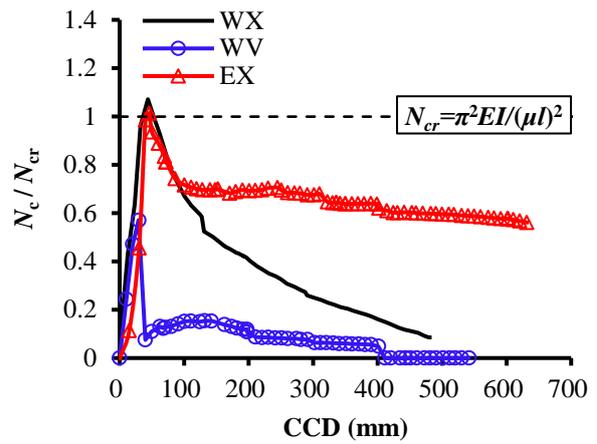


(c)

Fig. 16. De-composition of the load bearing capacity from tensile brace and compressive brace



(a)



(b)

Fig. 17. Axial force of braces: (a) tensile brace; (b) compressive brace

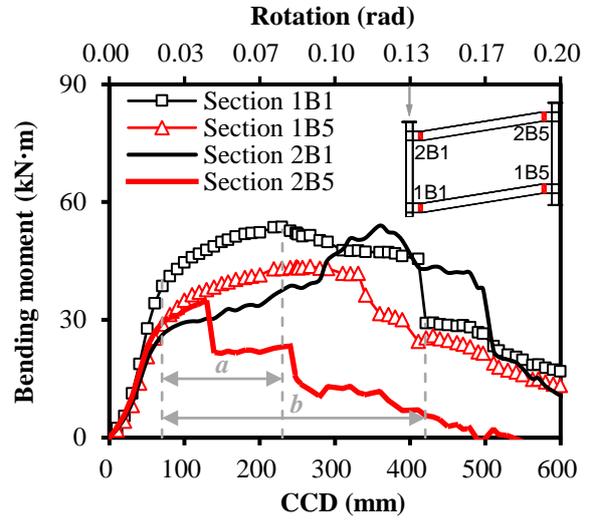
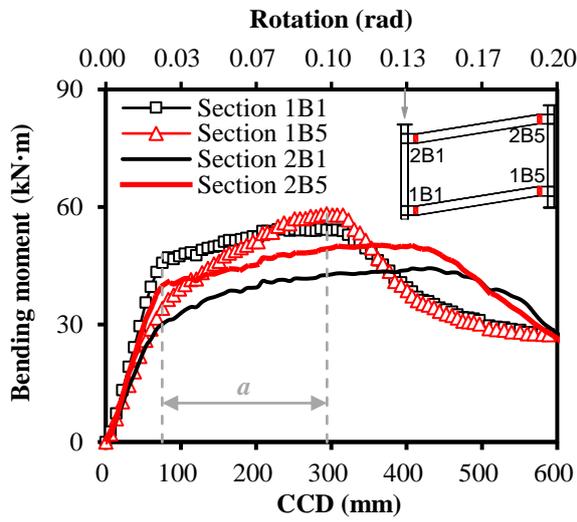


Fig. 18. Bending moment development at the beam ends: (a) WB; (b) EB

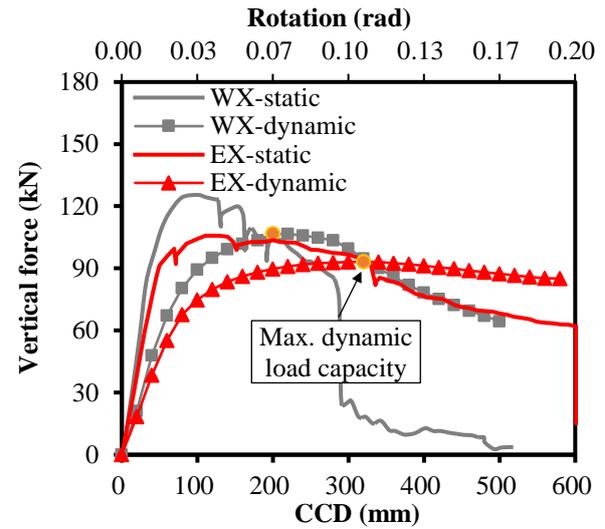
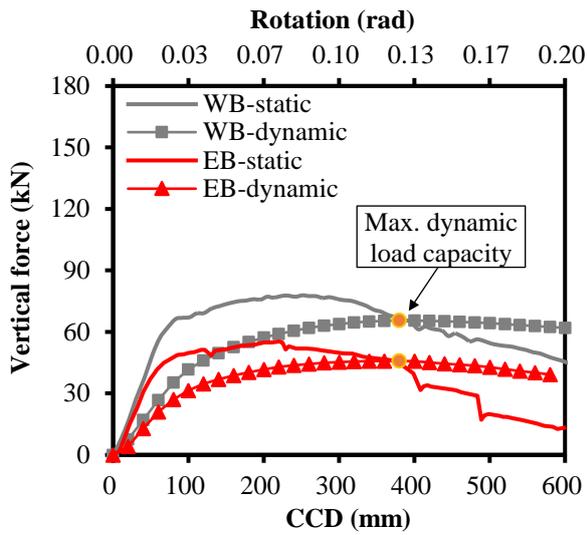


Fig. 19. Comparison of the static and dynamic load resistance: (a) WB and EB; (b) WX and EX

Conflict of interest

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

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24 **1. Introduction**

25 Disproportionate collapse is an event, when the failure of the loss of one or a couple of members,
26 results in the collapse of building disproportionate to the initial local failure. In recent years, the
27 collapses of steel frames occurred frequently, including the landmark building twin towers, in New
28 York in 2001, the steel frame at Xinjia Hotel building in Quanzhou, China in 2020, etc., which have
29 received considerable attention from the public due to catastrophic consequences.

30 Potential hazards with abnormal load (i.e. vehicular impact, fire, gas explosion, and terrorist attack,
31 etc.) may trigger disproportionate collapse [1-3]. Due to limited alternate load paths of the remaining
32 structure, the removal of columns at corners is more vulnerable than other column missing scenarios.
33 Kim and Kim [4] numerically evaluated the probability of disproportionate collapse of steel moment-
34 resisting frames subjected to various column removal scenarios. They found that the vulnerability of
35 disproportionate collapse was greatest when a corner column was removed suddenly. Gerasidimis [5]
36 investigated the disproportionate collapse vulnerability of steel frames for the case of a corner column
37 loss and developed an analytical method to capture the collapse mechanism of a steel frame under
38 corner column-removal scenarios. Based on numerical analysis, Fu and Tan [6] studied the
39 disproportionate collapse mechanism of composite floor systems after a corner column was removed.
40 Compared with the results obtained in the case of internal column removal, catenary action and tensile
41 membrane action in beams failed to develop. These studies provided insight into the disproportionate
42 collapse of steel frames after a corner column was removed behavior associated with the removal of
43 corner columns from steel frames. However, other studies [7-12] found that the bending moments of
44 the beam end near the corner joints reversed after the corner column was removed, leading to a
45 significantly bending moment (the bottom subject to tension) developed there. Based on tests on single-

46 floor beam-column sub-assemblages, most existing studies [13-16] captured the performance of multi-
47 floor steel frames under the scenario of a corner column removal by simplifying them as cantilever
48 beams, which unrealistically ignored the interaction of structural members in different floors
49 (Vierendeel action).

50 However, structures are not normally designed for the catastrophic consequences provoked by
51 abnormal events. On the other hand, it is not economical to rehabilitate the structures just for the
52 purpose of increasing disproportionate collapse resistance. Thus, design engineers should be aware of
53 the potential vertical load resistance, which had been ignored in conventional design, such as the
54 additional load resistance from masonry infilled walls and steel braces. Xavier et al. [17] tested a steel
55 substructure incorporating infilled walls, which indicated that infilled walls affect the behavior of steel
56 frames significantly. Moreover, Shan et al. [18] investigated the effect of infilled walls on the load
57 resistance of the steel moment frames. They indicated that masonry infill walls could enhance the load
58 bearing capacity and initial stiffness significantly. However, they will change the failure patterns.
59 Seismic investigations had confirmed that the moment resisting frames with braces was an efficient
60 seismic resisting system with sufficient lateral load resistance and stiffness [19, 20]. However, the
61 ability of steel braces to improve the performance of steel frames to resist disproportionate collapse is
62 still unclear. Khandelwal et al. [21] revealed that steel braced frames which were designed to meet
63 seismic requirements could survive even if a column was removed suddenly, since the steel braces are
64 effective in providing additional load resistance. It was found that horizontal braces could be employed
65 to retrofit the steel moment-resisting frame [22]. **It was indicated that an additional alternate load path
66 was formed by horizontal braces, and thus, partial loads were directly transferred to the side columns.**

67 However, experimental investigations of steel bracing systems after the removal of a corner
68 column were rare. Moreover, the connections types in previous numerical studies are either pinned or
69 fully restrained [23-25], the behavior of steel braced frames using partially restrained connections is

70 still unclear. In this experimental program, five two-floor and two-bay steel moment-resisting frames
71 were designed and tested after the removal of a corner column removal. The influence of different steel
72 bracing configurations and connection types were quantified experimentally and analytically.

73 **2. Experimental program**

74 *2.1. Test specimens*

75 As listed in Table 1, five test specimens including three braced frames (WX, WV, and EX) and
76 two bare frames without any braces (WB and EB), were designed in this experimental program. As the
77 main investigated parameters were the connection types and bracing configuration, the specimens were
78 labeled as follows: the alphabets “W” and “E” represent welded and end-plate connection, respectively.
79 Then, the alphabet of “B”, “X”, and “V” stand for bare frames, braced frames with X-shaped bracing,
80 braced frames with V-shaped bracings, respectively. Referring to Fig. 1, the prototype frame is designed
81 according to ANSI/AISC 360-05 [20]. The prototype frames had 6×6 bays with a transverse and
82 longitudinal span of 6.0 m and 8.4 m, respectively. The floor height is 3.0 m. The dead load and live
83 load were 5.1 kN/m² and 3.0 kN/m², respectively. For braced frames, the prototype frame is seismic
84 designed. Site class D was assumed and the critical acceleration parameters S_{DS} and S_{D1} are 0.20 and
85 0.14, respectively. For bare frames with non-seismic design, identical frames as the braced frames
86 except no braces were installed for comparison. As illustrated in Fig. 1, a two-floor subframe was
87 derived from the prototype frame as a specimen for testing. Considering the limitation of lab and facility,
88 only half-scale sub-frames were tested.

89 In contrast to the corner column without additional horizontal restraints, overhanging beam
90 (length of 655 mm) was fabricated beyond the side column to consider the horizontal constraints from
91 the interior bays, which will connect with an A-frame by horizontal chain-poles (refer to Fig. 2a). The
92 cross-section of beams and columns is HN 200×100×5.5×8 and HW 150×150×7×10, respectively. Fig.
93 3 presented the fabrication details of the specimens. For welded connection, complete joint penetration

94 welds were used to connect the beams and columns. For end-plate connection, the beam was welded
95 to an end plate with thickness of 10 mm. Eight M18 Grade 8.8 frictional bolts are employed for bolt
96 connection with a pre-loading force of 345 N·m. The braces and the connections were designed based
97 on ANSI/AISC 341-05 [19]. The braces and beams are connected by the gusset plates welded to the
98 beam flanges. The uniform force method was employed to determine the force acting on welds [26].
99 To avoid the gusset plate premature yield and fracture occurred in the gusset plate before braces failure,
100 the gusset plate is designed relatively stronger [19, 21]. Taking WX and EX as an example, the braces
101 were made by steel angles with a dimension of $36 \times 36 \times 4$ mm as shown in Fig. 2a. WX and EX have
102 X shaped braces and the size of gusset plate is $330 \times 125 \times 12$ mm. In addition, WV has V shaped braces.
103 The gusset plates installed in the second floor have a size of $160 \times 155 \times 12$ mm while the gusset plate
104 installed in the first floor has a size of $510 \times 155 \times 12$ mm.

105 *2.2. Material properties*

106 All structural members were fabricated by Grade Q235 steel. As displayed in Table 2, the critical
107 material properties of each component are measured via coupon tests in accordance with the relevant
108 specification [27]. The average value of three coupons was calculated for each set of results in this
109 table. The properties of M18bolts were provided by supplier.

110 *2.3. Test setup*

111 As illustrated in Fig. 4a, the ground corner column was not assembled to represent the initial
112 damage. Beneath each side column, a pin support was applied. The vertical load was imposed at the
113 top of corner column through a whisky jack. Displacement-controlled loading method was adopted. At
114 the beginning of the test, a loading rate of 5 mm/min was set until reaching the vertical displacement
115 of 100 mm. In the subsequent loading process, the loading rate of 10 mm/min was adopted. The applied
116 concentrated load was monitored by a load cell, which was placed beneath the whisky jack. To prevent
117 undesirable out-of-plane failure, a steel assemblage was specially arranged.

118 To represent the axial loads from upper floors, a whisky jack was applied on the side column to
119 guarantee an axial compressive ratio of 0.3. The overhanging girder, if any, was connected to the
120 reaction frame through a horizontal chain-pole. Tensile/compressive load cell was mounted in the
121 horizontal chain-pole, so that the horizontal reaction force could be monitored. To measure the vertical
122 load redistribution to the side column, each pin support was installed a load pin. Above the corner
123 column, a hinge was set to allow the conceivable rotation of the corner column during the test.
124 Moreover, two transverse beams with rollers were mounted at both sides of the corner column to
125 prevent out-of-plane movement during the test. In addition, as shown in Fig. 4a, three LVDTs were
126 mounted along the beam of first floor in the corner bay. It should be mentioned that the deflection of
127 beam between side columns was negligible during the test of Specimen WB, thus they were not
128 monitored in the following tests. As given in Fig. 2, to determine the variations of the axial forces and
129 bending moments in the beams, a series of strain gauges were attached to the critical sections.

130 **3. Test results**

131 In order to assess the robustness of steel bending-moment frames with steel braces, five two-floor
132 steel sub-frames with or without bracings were experimentally tested after a corner column loss. The
133 key results are tabulated in Table 3 and presented below.

134 *3.1 Global behavior*

135 **WB:** The load-displacement curves at the corner column of WB, WX, and WV are displayed in
136 Fig. 5. The specimens initially exhibited elastically as the load resistance increased linearly. The yield
137 load was measured to 64.7 kN at the corner column deflection (CCD) increased to 80 mm. Thus, it has
138 initial stiffness of 0.8 kN/mm. The initial stiffness was defined as the ratio of yield load to yield
139 displacement herein. The load resistance started to decrease after the occurrence of local buckling at
140 beam flanges in the first floor, which was attributed to the effects of flexural bending. At an CCD of
141 233 mm, the ultimate load bearing capacity, which was defined as the peak load resistance, of 78.2 kN

142 **was reached.** On further increasing the displacement, similar local buckling also occurred at beam
143 flanges in the second floor at CCD of 400 mm. Fig. 6 shows the failure pattern of the specimen.
144 Although no fracture occurred during the test, the local buckling was severe at the beam ends, which
145 resulted in the beam of the first floor failing in torsion in absence of catenary action.

146 **WX:** As the second floor has X braces, the failure pattern of WX was changed. In the start of
147 testing, the compressive braces started to buckle, which indirectly showed that the compressive braces
148 may have little contribution to the load enhancement. This can be confirmed by the strain gauge results
149 later. As displayed in Fig. 5, it has a yield load of 116.5 kN at an CCD of 62 mm. Moreover, it has
150 initial stiffness of 1.9 kN/mm, about 232.3% of that of WB. Similar to WB, local buckling of the bottom
151 beam flange occurred near the side column in the ground floor at this stage. It has an ultimate load
152 bearing capacity of 125.4kN. The tensile brace fractured at an CCD of 288 mm, which leads to the load
153 resistance dramatically dropping from 74.9 kN to 24.3 kN. Fig. 7 gives the failure pattern of the
154 specimen. Tensile braces were fractured and compressive braces were severely buckled. No yielding
155 was observed in the gusset plates. Similar to WB, torsional damage occurred in the beam in the first
156 floor. **However, premature weld fractures occurred in Joints S1, S2 and C2 of WX, which were not**
157 **observed in WB. This was due to the additional shear forces from the steel braces and the torsion-**
158 **induced shear forces in the beams.**

159 **WV:** For WV, it has a yield load of 93.5 kN and initial stiffness of 1.9 kN/mm. Similar to WX,
160 the compressive braces began to out-of-plane buckling at the very beginning of the load. Different from
161 WX, the tensile brace fractured at a relatively early stage (corresponding to CCD of 63 mm) and
162 followed by the drop in load resistance dramatically. Moreover, it has an ultimate load bearing capacity
163 of 100.9 kN, which is 129.0% and 80.5% of that of WB and WX, respectively. When the fracture
164 occurred at the beam ends, the load resistance dropped significantly. After reaching the CCD of 191
165 mm, the load bearing capacity of WV became even lower than that of WB. Fig. 8 displays the failure

166 pattern of WV. Similar to WX, the compressive braces suffered severe buckling and the tensile braces
167 were fractured. However, different from WX, there is no torsional damage occurring in the beams with
168 gusset plate welded to the beam flange were enough to prevent torsional buckling.

169 **EB:** Fig. 9 compares the load-displacement curves of EB and EX. EB achieved a yield load of
170 42.7 kN and initial stiffness of 0.7 kN/mm. When the CCD reached 135 mm, the welds nearby the side
171 column in the second floor fractured. Subsequently, the load bearing capacity could increase until
172 further weld fracture occurred at the end-plate. It has an ultimate load bearing capacity of 55.5 kN.
173 Similar fractures occurred at the beam ends near the corner column at CCD of 406 mm and 487 mm,
174 respectively. Fig. 10 gives the failure pattern of the specimen. The failure of EB was controlled by weld
175 failure at the end-plate.

176 **EX:** It has a yield load of 86.4 kN and initial stiffness of 1.9 kN/mm. The weld fracture was
177 initially developed nearby the corner column. With the displacement kept increasing to 128mm, EX
178 reached its ultimate load bearing capacity of 105.8 kN. When CCD increased to 159 mm and 336 mm,
179 the welds fracture was formed at the beam ends in sequence. The tensile braces fractured at an CCD of
180 602 mm, which was 109.0% later than WX. Fig. 11 gives the failure pattern of EX. Similarly, the
181 compressive brace suffered severe out-of-plane buckling while the tensile brace fractured. Moreover,
182 local buckling occurred at point A.

183 *3.2 Deformation measurements*

184 The deformation shape of beams at different stages is displayed in Fig. 12. Following DoD [28],
185 the chord rotation was defined as the ratio of CCD to beam span. From the figure, the chord rotation
186 would significantly underestimate the rotation of the beam end nearby the corner column. On the other
187 hand, the chord rotation could assess the rotation of the beam end nearby the side column accurately,
188 especially for WX and WV. The external steel brace would not significantly change the deflection shape
189 of the beams. Similar observation was achieved in EB and EX.

190 3.3 Internal force evaluation

191 To deeply understand the contribution of load resistance from braces, the contribution of braces
192 and frames should be determined individually. Before that, the reliability of calculation formula to
193 determine the internal force of each component based on strain gauge results must be verified. From
194 Fig. 2a, Sections B1-4 installed a series of strain gauges, which could help to determine the internal
195 force. Similar to the calculation method proposed in the previous paper [29, 30], the load-displacement
196 curve based on strain gauge data was determined and compared with the one from load cell results
197 (refer to Fig. 13). As shown in Fig. 13, good agreements are achieved between the one measured by
198 the load cell and analytical results from strain gauge data. For WX and EX, the minor discrepancy of
199 initial stiffness may be caused by unavoidable gaps in the test setup, which did not reflect in the LVDT
200 placed above the corner column. Generally, the analytical results based on strain gauges can well
201 capture the character of the curve until failure.

202 Fig. 14 presents the load bearing capacity of the braces and frame. **Relying on the analytical results**
203 **of internal force, it was revealed that the load bearing capacity from the bare frame was purely provided**
204 **by flexural action, the contribution of catenary action could be ignored due to the limited tensile forces**
205 **in the beams.** For WX, at the ultimate load resistance stage, the contribution of steel braces was about
206 35.3%. With the increasing vertical displacement, the load resistance of the steel braces began to
207 decrease due to the yield of beam section releasing the constraints for the tensile braces gradually. For
208 WV, at the stage of ultimate load bearing capacity, the contribution of steel bracing was only 27.7% as
209 the tensile braces fractured. Different from the braced frames with welded connection, for EX, initially,
210 the steel braces contributed greater load resistance than the frame. At the stage of ultimate load
211 resistance, the contribution of steel braces was 45.7%, which was comparable to that of the frame.
212 Moreover, after that, the load resistance from braces is always comparable to that from frames until the
213 test final.

214 **4. Discussion of experimental results**

215 *4.1 Contribution of load resistance*

216 The de-composition of the frame contribution from each floor is given in Fig. 15. As shown in the
217 figures, the load bearing capacity from the first and second floor has similar trends until failure occurred
218 in the connections, while the first floor has slightly greater load resistance than that of second floor.
219 This was due to the greater rotational constraints and Vierendeel action, for which further explanation
220 would be in section 4.2. However, WX had a greater maximum load resistance from the second floor
221 as the connection in the first floor fractured earlier.

222 Fig. 16 displays the de-composition of load bearing capacity contribution from tensile brace and
223 compressive brace. For WB, WX, and WV, the maximum load resistance from steel braces was 46.4,
224 29.7, and 49.8 kN, respectively. Different from WX and EX, which reached their maximum load
225 resistance until the tensile brace yielded, the maximum load resistance of WV was obtained when the
226 compressive brace buckled. For WV, the compressive brace contributed the maximum load resistance
227 of 14.5 kN, which was 33.7% and 41.2% higher than that of WX and EX, respectively. To better
228 understand the contribution of steel braces, the development of axial force of tensile brace and
229 compressive brace was normalized, as shown in Fig. 17. The tensile brace has an analytical yield load
230 of 85.6 kN. And the compressive brace has buckling loads of 23.0 kN and 57.6 kN for X and V
231 configuration, respectively. As can be seen in the figure, both the tensile and compressive brace in X
232 configuration achieved their yield and buckling loads. However, in V configuration, the tensile brace
233 could not achieve the analytical yield load and the compressive brace could not reach the analytical
234 buckling load as the constraints applied at the braces of V configuration were not translation fixed,
235 which is assumed in analytical analysis.

236 *4.2 Effects of connection types*

237 Unlike braced frames where the maximum deformation would be introduced by the failure of the

238 steel bracing, the maximum deformation of bare frames difficult to identify from the load-displacement
239 curve. The deformation capacities of bare frame were defined as vertical displacement at ultimate load
240 bearing capacity. For WB and EB, the maximum deformation was 233 and 228 mm, respectively. Thus,
241 WB and EB had a similar maximum deformation, which was quite different from the case of missing
242 a middle column [29, 30]. WB was able to sustain large deformation caused by the torsion developed
243 in beams, and avoided the brittle weld fracture. Fig. 18 shows the development of bending moments at
244 the beam ends. From the figure, not only the beam end near the side column, the beam end near the
245 corner column also experienced large positive bending moments, which was different from the
246 behavior of unsupported cantilevers. This indicated that Vierendeel action played an important role in
247 the load resisting mechanism after corner column removal. Moreover, DoD [28] defined the acceptable
248 plastic rotation angles for different type of steel connections. Taking Section 1B1 as an example, the
249 parameters ‘a’ and ‘b’ were defined as shown in Fig. 18. Table 4 compared the measured parameters
250 with the requirements in specification, which showed that the recommended ductility acceptance
251 criteria were conservative for both fully and partially restrained moment connections in current
252 specimens.

253 Although this experimental test was focused on the static performance of welded and endplate
254 connections, disproportionate collapse following sudden column removal exhibits a typical dynamic
255 response. Based on the available static load-displacement curve, a dynamic capacity evaluation was
256 applied using an energy-based method proposed by Izzuddin et al. [31]. This approach has been applied
257 and verified in previous studies related to disproportionate collapse [32-34]. As described in Eq. (1),
258 the equivalence between external work (dynamic response) and internal energy (static response) was
259 used to obtain the dynamic response shown in Fig. 19.

260
$$P_d(u_d) = \frac{1}{u_d} \int_0^{u_d} P_{NS}(u) du \quad (1)$$

261 As labeled in Fig. 19, the maximum dynamic load capacities of WB, EB, WX, and EX were 65.5,
262 45.8, 106.7, and 93.2 kN, respectively. Prior to this point being reached, it was unlikely for the
263 specimens to undergo disproportionate collapse even if a sudden column failure occurred. The
264 maximum dynamic deformations of WB, WX and WV were 380, 380, 200 and 320 mm, which were
265 163%, 194%, 167% and 250% of that of maximum static deformations, respectively. This implied that
266 the deformation capacity of the frames under sudden column removal were greater than those under
267 quasi-static loading for the same applied load.

268 *4.3 Effects of braces types*

269 As seen from Fig. 5, the yield load of WB, WX, and WV were 64.7, 116.5 and 93.5 kN,
270 respectively. Therefore, the X and V braces enhanced the yield load of WB by 80.1% and 44.5%,
271 respectively as the X bracing configuration in WB can directly transfer a portion of the load to the side
272 column. Regarding the ultimate load bearing capacity, WB, WX and WV were 78.2, 125.4 and 100.9
273 kN, respectively. Therefore, the X and V bracing configuration enhanced the ultimate load bearing
274 capacity of WB by 60.4% and 29.0%, respectively. This could be attributed into the tensile braces in V
275 configuration fractured much earlier, even earlier than the yielding of the beams. Moreover, the
276 maximum deformation of WB, WX and WV were 233, 288 and 63mm, respectively. Thus, the steel
277 bracing with X configuration increased the maximum deformation of WB by 23.6%, while the steel
278 bracing with V configuration decreased the maximum deformation of WB by 73.0%. The maximum
279 deformation of WX was 457.1% of that of WV. Regarding to failure patterns, as shown in Fig.12,
280 torsional damage occurred at interfaces between the beam and corner column joint of WB and WX [35,
281 36]. However, due to the gusset plate of V bracing configuration, the critical beam section shifted close
282 to the beam midspan, which prevented the torsional damage of WV [37].

283 **5. Conclusions**

284 A series of five two-bay and two-floor steel sub-frames were tested subjected to a corner column

285 missing scenario to investigate the robustness of steel braced frames to resist disproportionate collapse.

286 Following conclusions are obtained:

- 287 1. Steel braces could increase the initial stiffness and load resistance of steel frames whatever X
288 configuration or V configuration was adopted. However, torsion may control the failure of bare
289 frame, while the braces also amplified the shear force demand at the beam ends. Therefore, it is
290 necessary to consider the torsional shear forces of adjacent structures where corner column is
291 removed in the alternate load path method. The connection design can be controlled by the shear
292 forces generated by torsion, especially for braced frame.
- 293 2. Experimental results indicated that the tensile braces in steel frames with end-plate connections
294 did not fracture until the vertical deformation reached 20% of the beam span. X configuration
295 performed better than V configuration regarding ultimate load bearing capacity and initial stiffness,
296 as X configurations could transfer partial of the load to the side column directly while V
297 configuration are only subjected to axial forces within the elastic range. Whatever X or V
298 configurations, the compressive braces only affect the initial stiffness as they were severely
299 buckled from the very early beginning of the test.
- 300 3. Different from the scenario of loss of an interior column, the bare frame with weld connection
301 achieved a similar maximum deformation than that of steel frame with end-plate connections when
302 the scenario of corner column missing was concerned. Moreover, the value of plastic hinge
303 properties was too conservative for the bare frame with weld connection and end-plate connection
304 under corner column removal.
- 305 4. Analytical analysis found that the load resistance of the frame in the ground floor is generally
306 larger than that in the second floor because the structural components in the second floor could
307 provide horizontal constraints to the joints in the ground floor, in other words, greater Vierendeel
308 action was mobilized in the frames in the ground floor.

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315

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416
417
418 **Table 1.** Summary of specimen

Specimen ID	Connection	Bracing configurations
WB	Welded	N/A
WX	Welded	X-shaped braces
WV	Welded	V-shaped braces
EB	End-plate	N/A
EX	End-plate	X-shaped braces

419
420 **Table 2.** Material properties

Items	Plate thickness (mm)	Yield strength (MPa)	Yield strain	Ultimate strength (MPa)	Ultimate strain	Elongation (%)
Beam flange	8	310	0.0019	420	0.024	12
Beam web	5.5	320	0.0021	430	0.0249	13.5
Column flange	10	300	0.0019	410	0.0267	14
Column web	7	295	0.0023	375	0.0242	13
Steel brace	4	310	0.0021	420	0.0256	12.5

421
422 **Table 3.** Test results

Test ID	U_{YL} (mm)	F_{YL} (kN)	K_{YL} (kN/mm)	U_{PL} (mm)	F_{PL} (kN)
WB	80	64.7	0.8	233	78.2
WX	62	116.5	1.9	103	125.4

WV	49	93.5	1.9	63	100.9
EB	60	42.7	0.7	228	55.5
EX	46	86.4	1.9	128	105.8

423 Note: F_{YL} and F_{PL} represent yield load and ultimate load bearing capacity, respectively; U_{YL} and U_{PL} represent displacements corresponding the yield
424 load and ultimate load bearing capacity, respectively; K_{YL} represents initial stiffness corresponding the yield load.

425

426 **Table 4.** Comparison of the measured and recommended plastic hinge parameters in DoD [28]

Test ID	Section	'a' at the beam end (rad)	'a' in DoD [28] (rad)	'b' at the beam end (rad)	'b' in DoD [28] (rad)
WB	1B1	0.070	0.025	N/A	0.038
	1B5	0.073	0.025	N/A	0.038
	2B1	0.118	0.025	N/A	0.038
	2B5	0.093	0.025	N/A	0.038
EB	1B1	0.050	0.012	0.109	0.018
	1B5	0.063	0.012	0.084	0.018
	2B1	0.094	0.012	0.136	0.018
	2B5	0.026	0.012	0.061	0.018

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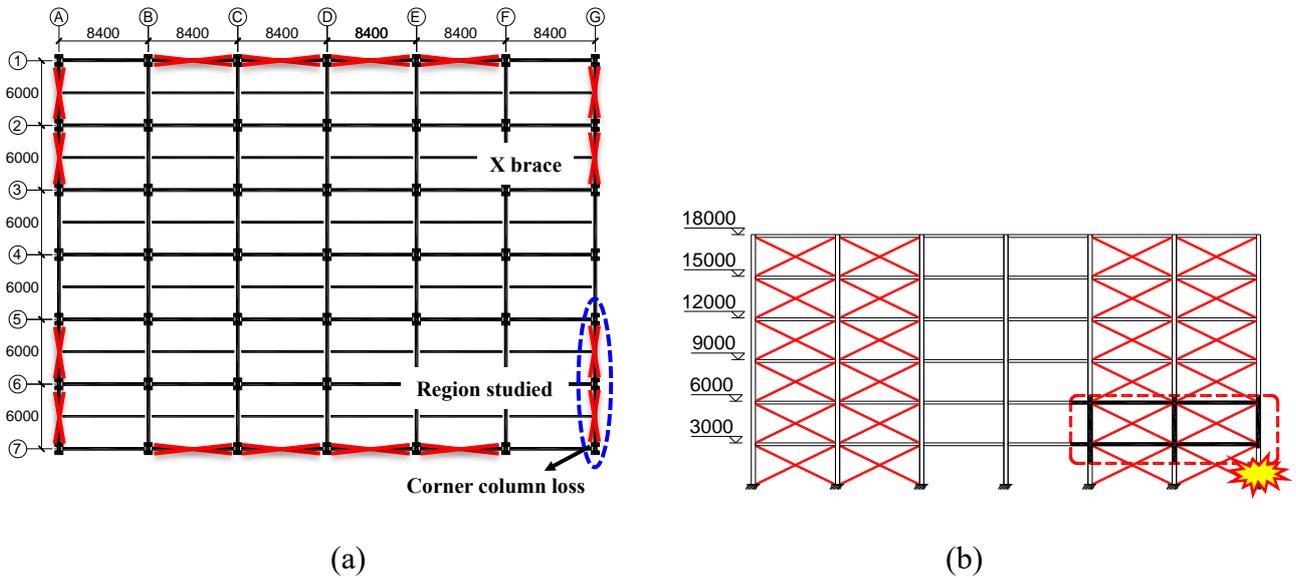
446 **Fig. 16.** De-composition of the load bearing capacity from tensile brace and compressive brace

447 **Fig. 17.** Axial force of braces: (a) tensile brace; (b) compressive brace

448 **Fig. 18.** Bending moment development at the beam ends: (a) WB; (b) EB

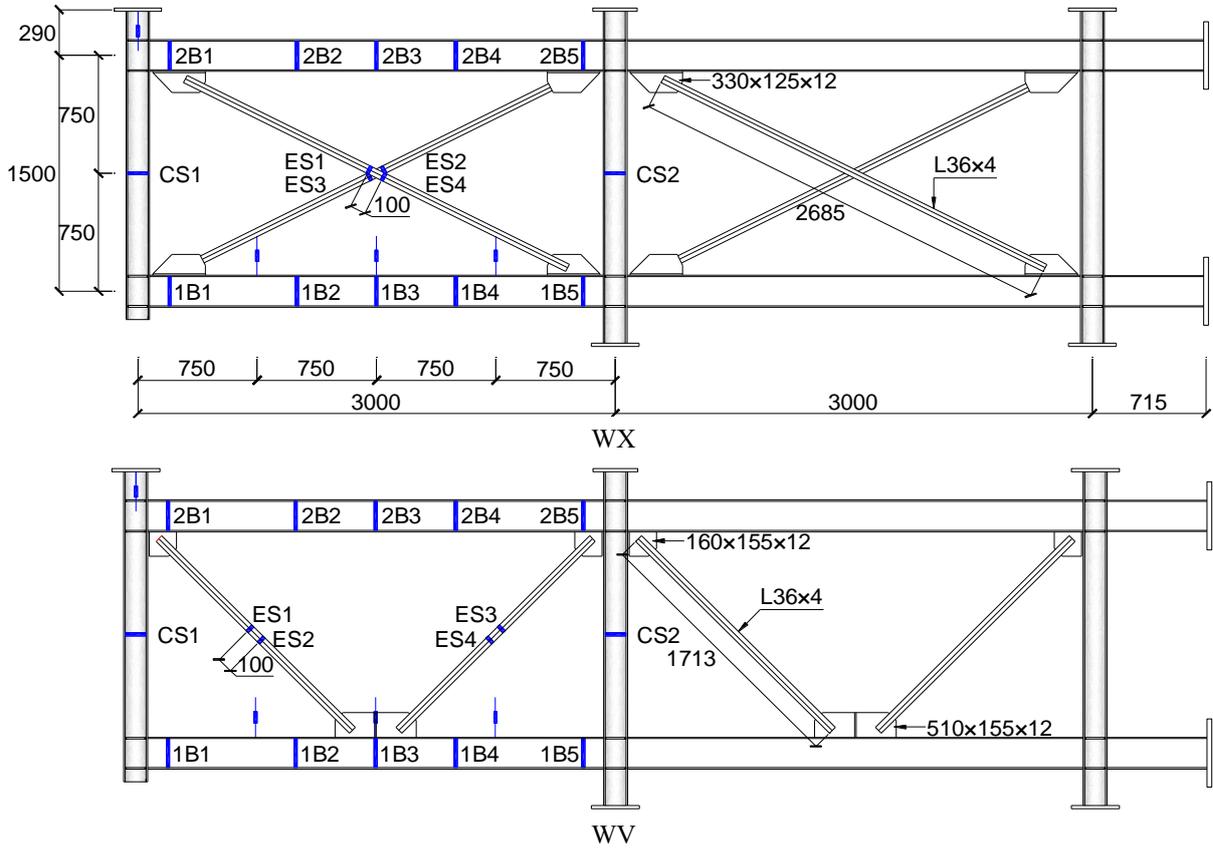
449 **Fig. 19.** Comparison of the static and dynamic load resistance: (a) WB and EB; (b) WX and EX

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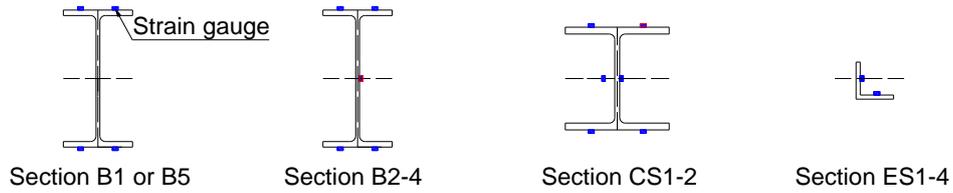


453 **Fig. 1.** Prototype building and extracted frame (unit in mm): (a) front view; (b) side view

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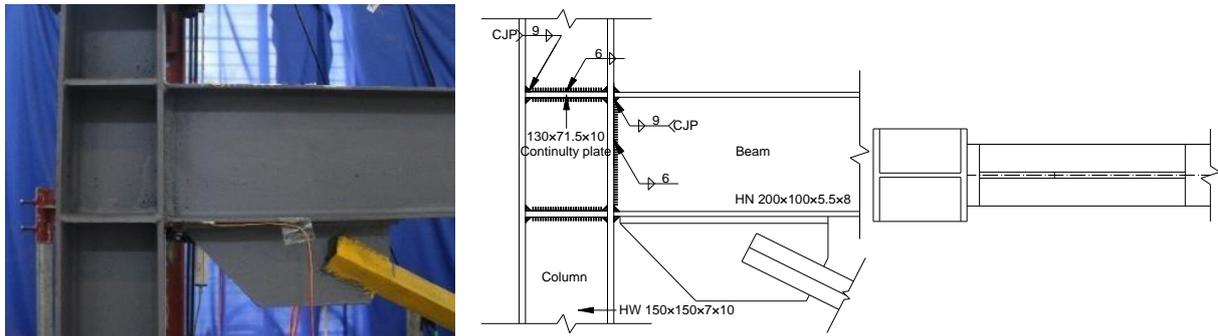


(a)

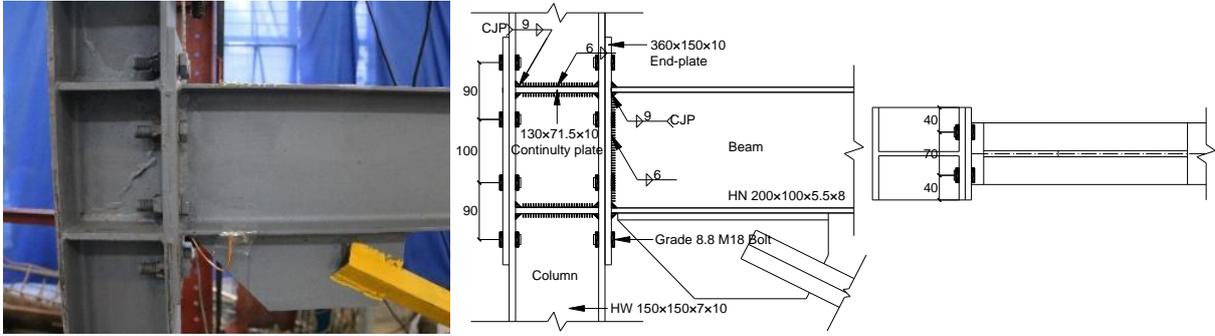


(b)

Fig. 2. Dimensions of the specimen and locations of instrumentations: (a) layout of strain gauge and displacement transducer; (b) position of strain gauges on sections

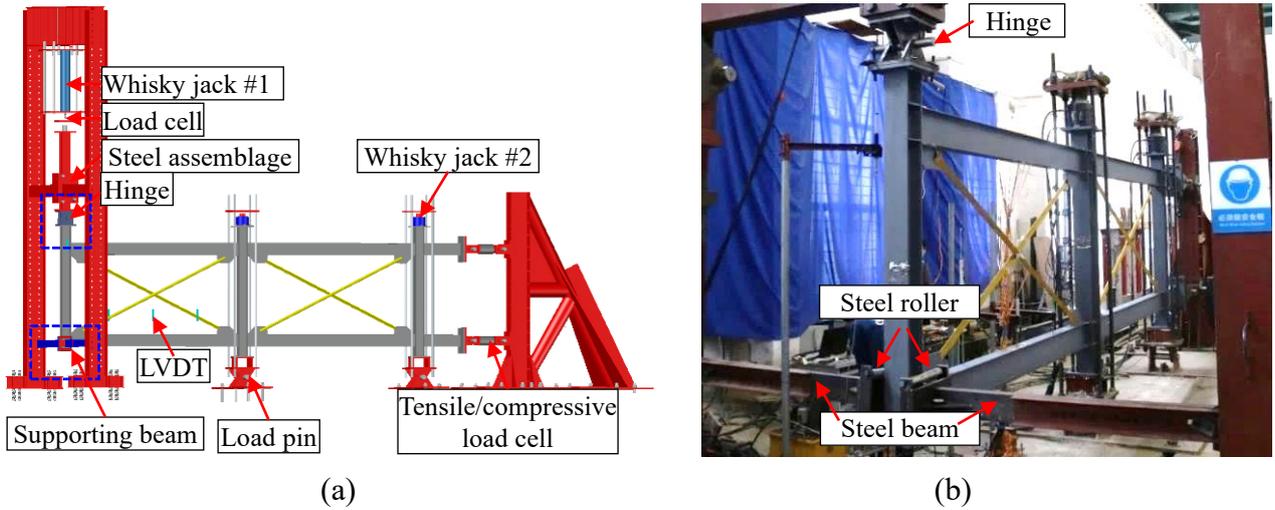


(a)



(b)

Fig. 3. Details of the connections: (a) welded connection; (b) end-plate connection



(a)

(b)

Fig. 4. Test setups of WX: (a) drawing; (b) photo

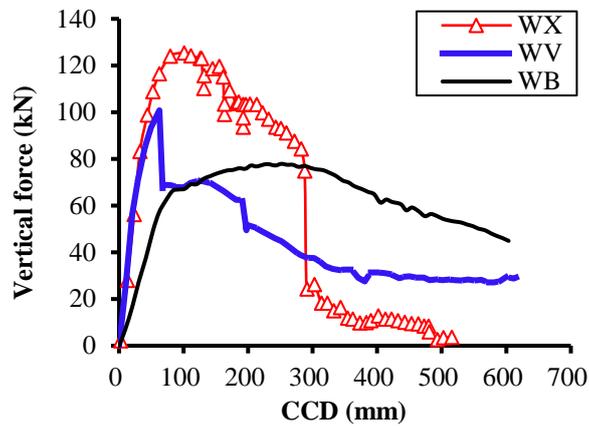


Fig. 5. Load-displacement curves of specimens: WB, WX and WV

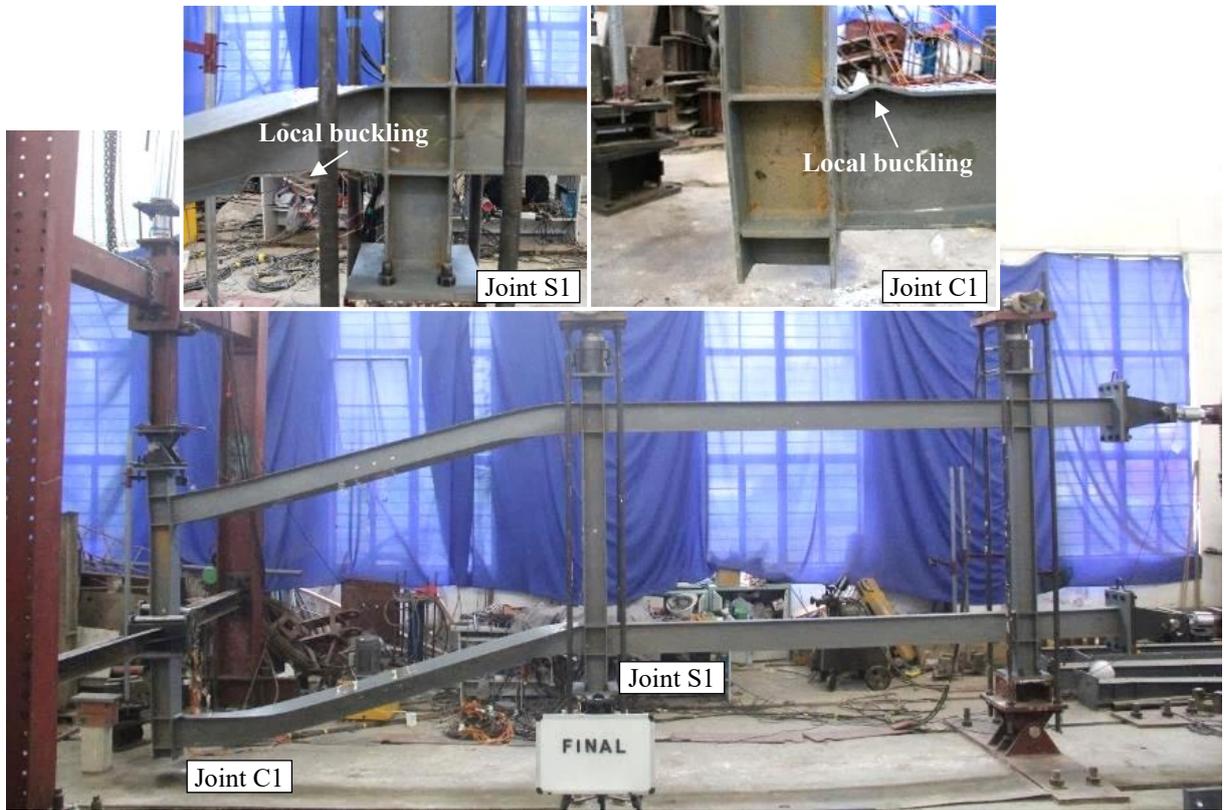


Fig. 6. Failure pattern of Specimen WB

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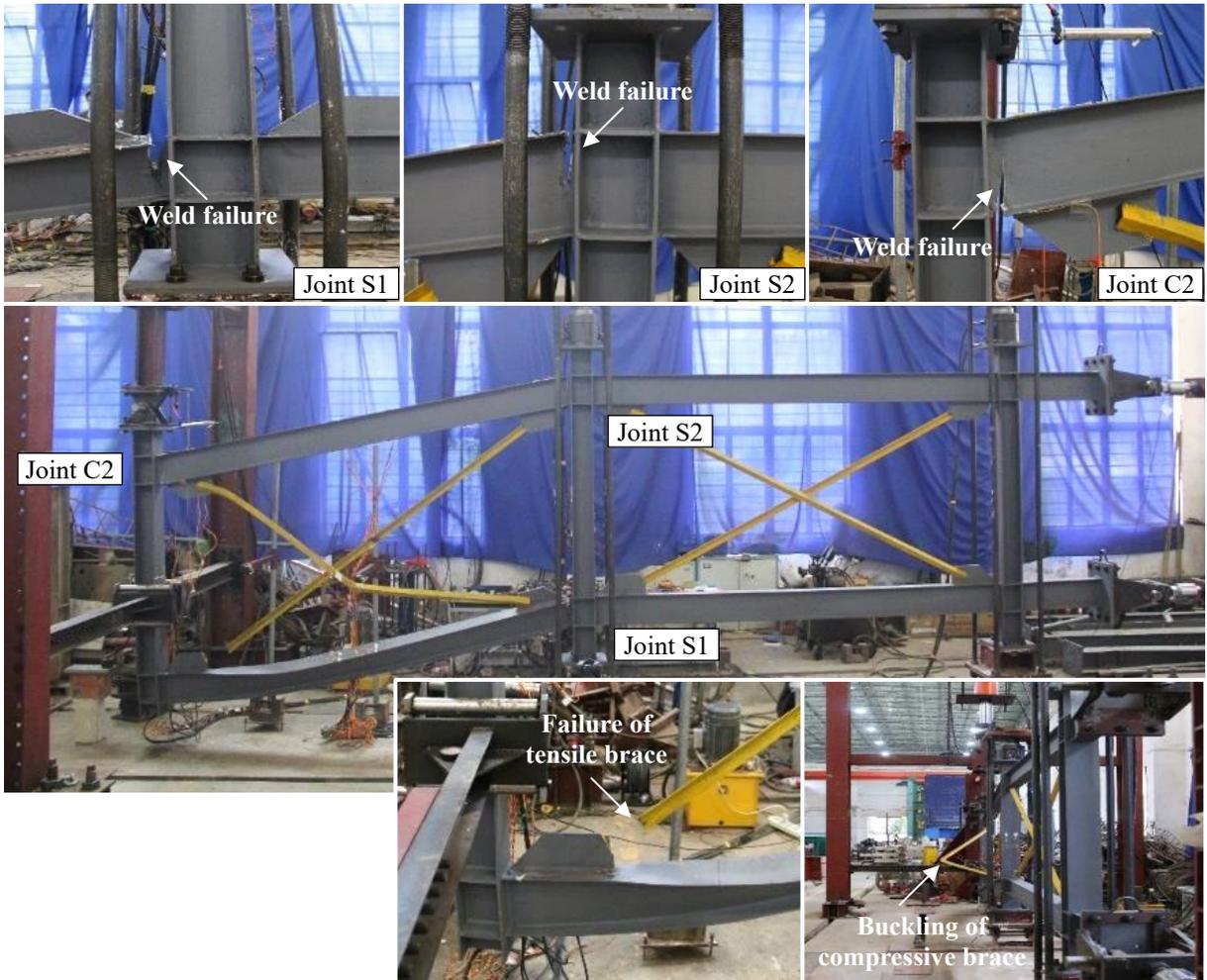


Fig. 7. Failure pattern of Specimen WX

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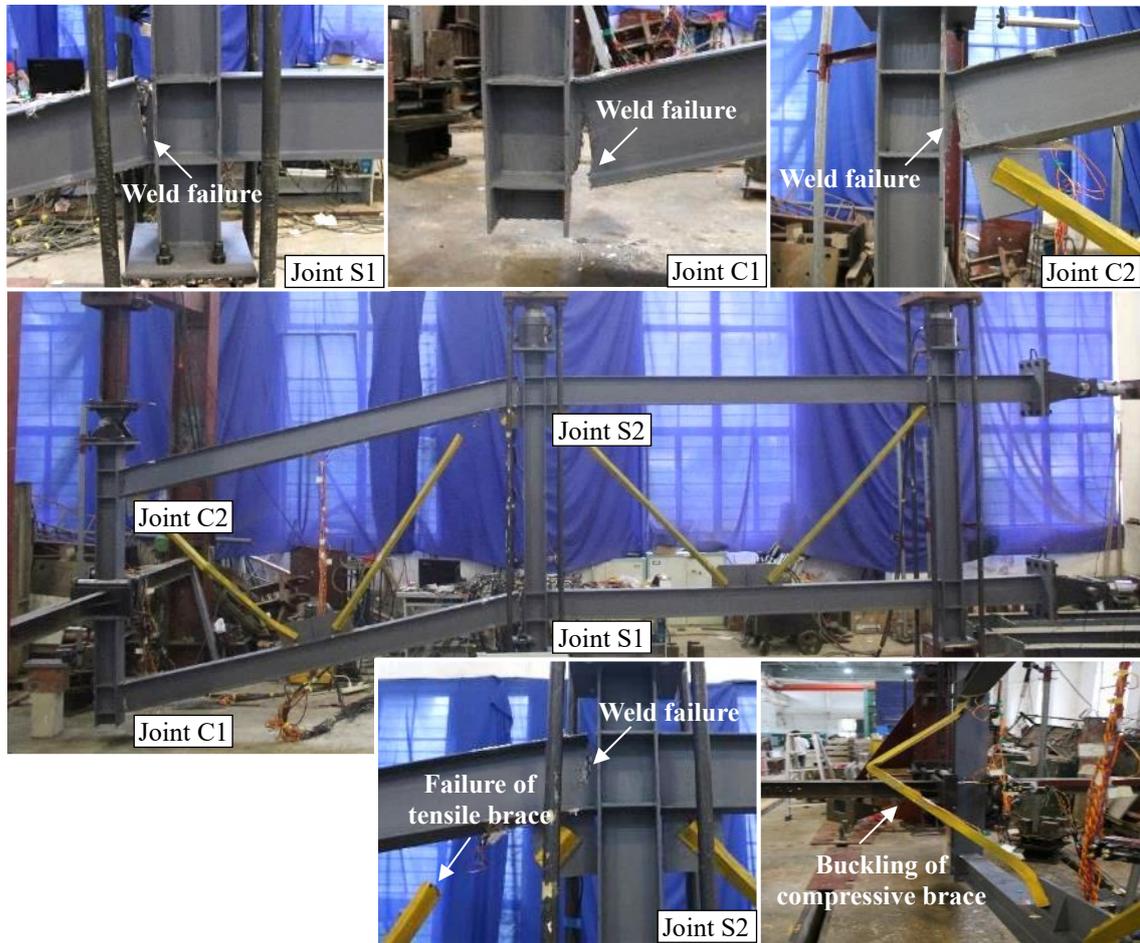


Fig. 8. Failure pattern of Specimen WV

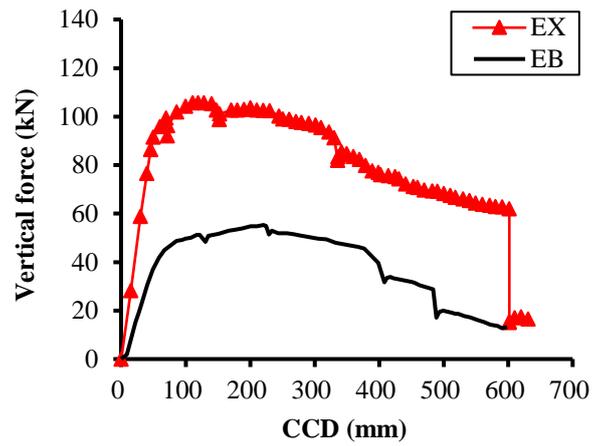
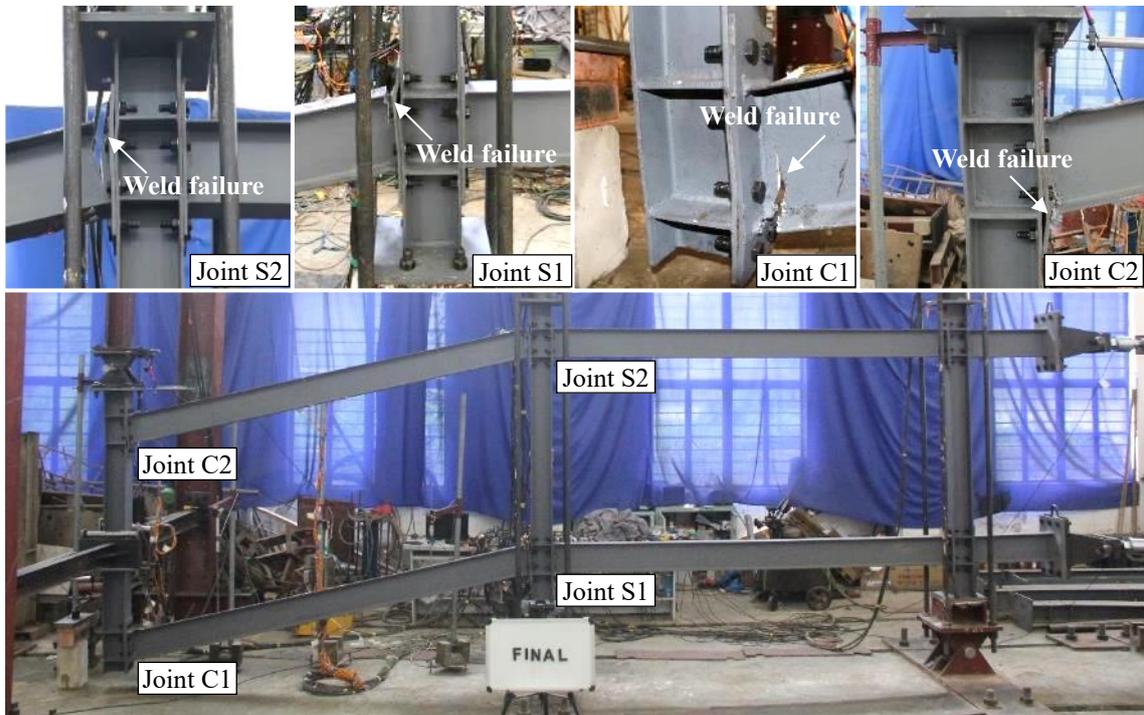
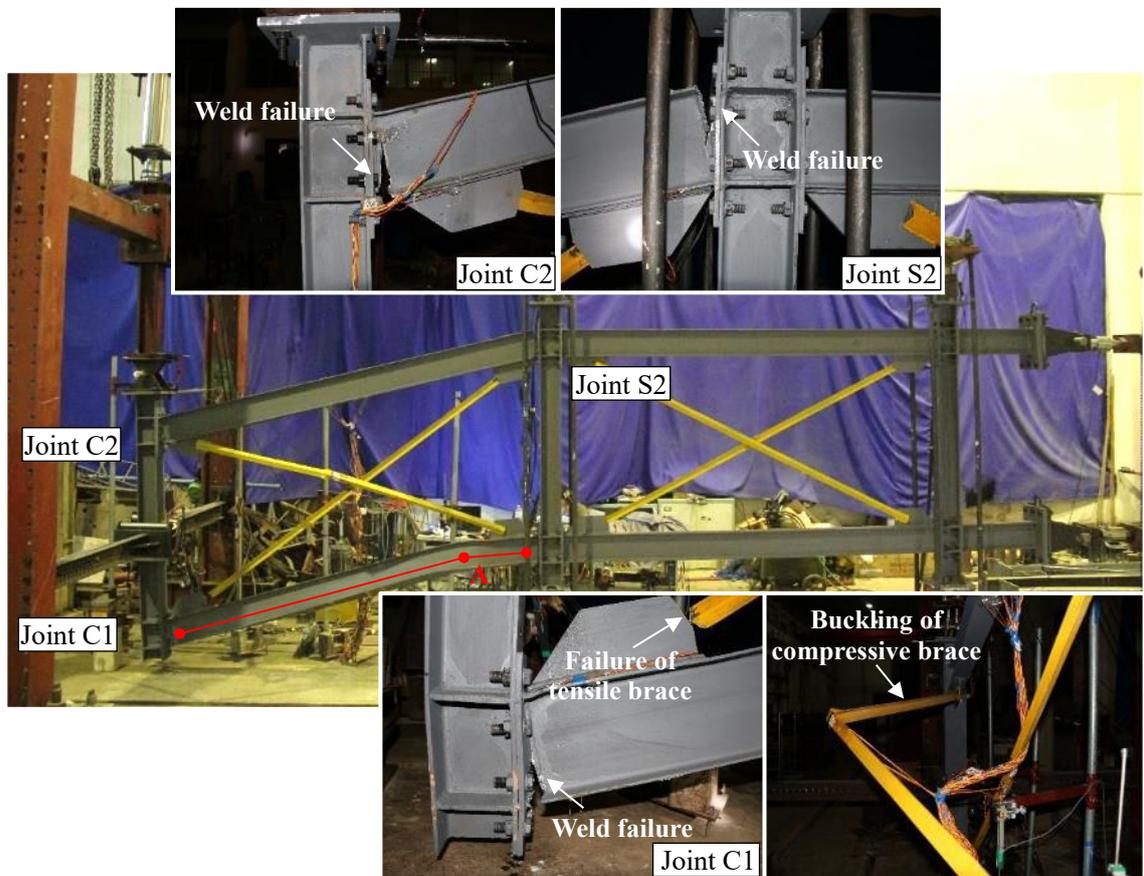


Fig. 9. Load-displacement curves of specimens: EB and EX



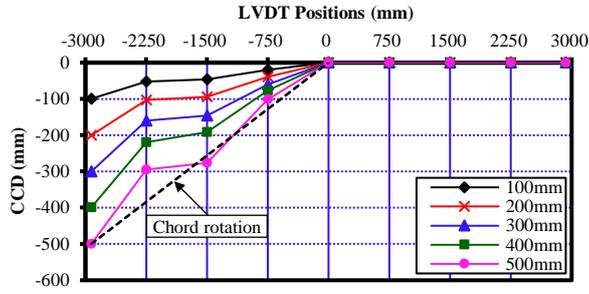
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Fig. 10. Failure pattern of Specimen EB



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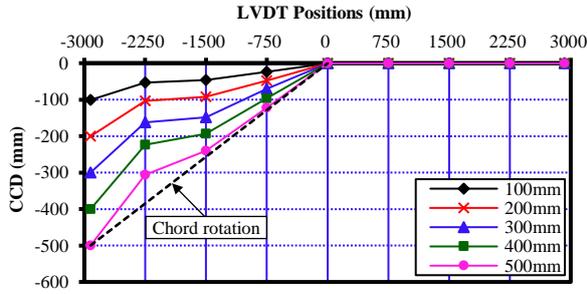
Fig. 11. Failure pattern of Specimen EX



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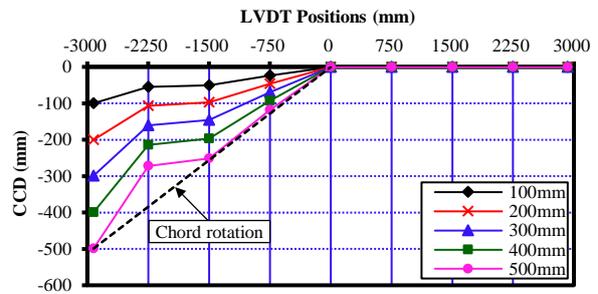
(a)



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(b)



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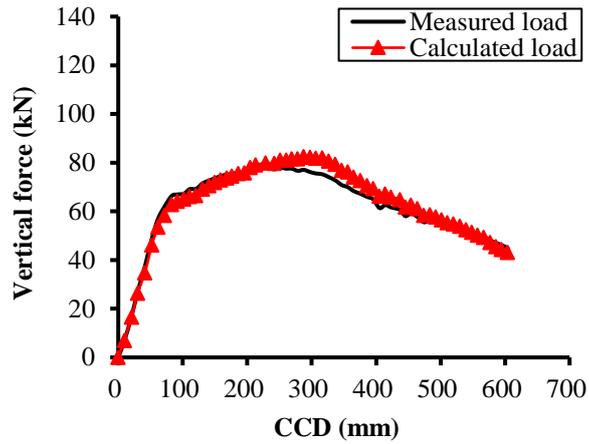
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(c)

Fig. 12. Deflection profile of the beams in different stages: (a) WB; (b) WX; (c) WV

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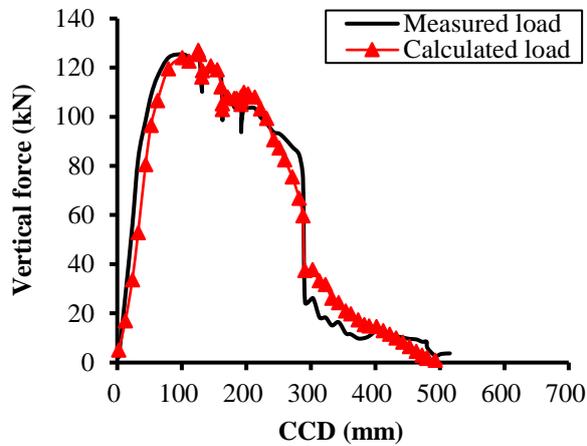
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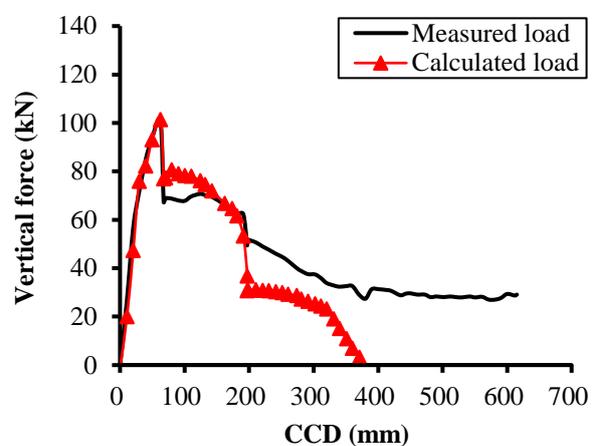
(a)



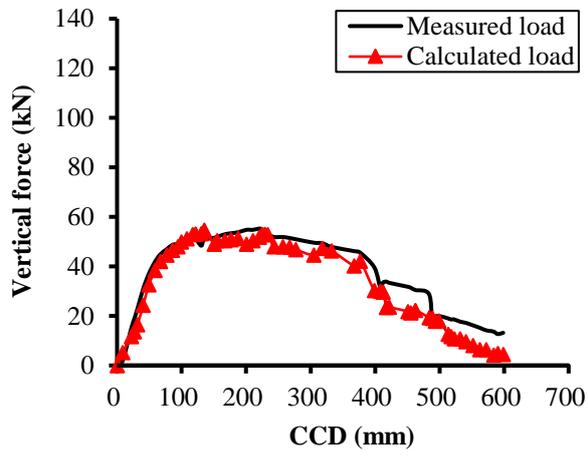
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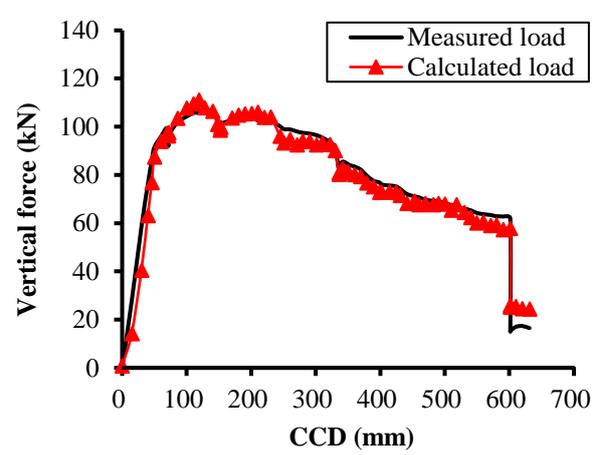
(b)



(c)

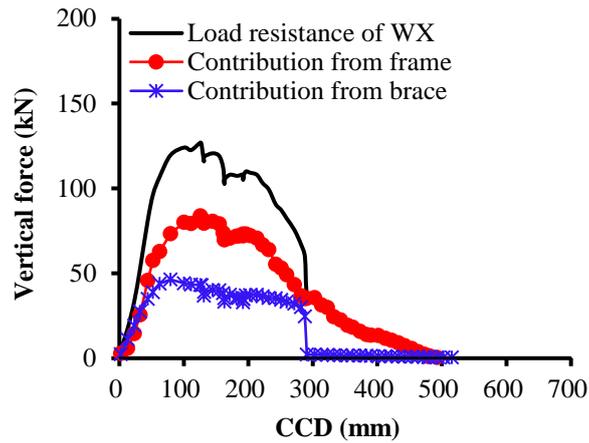


(d)

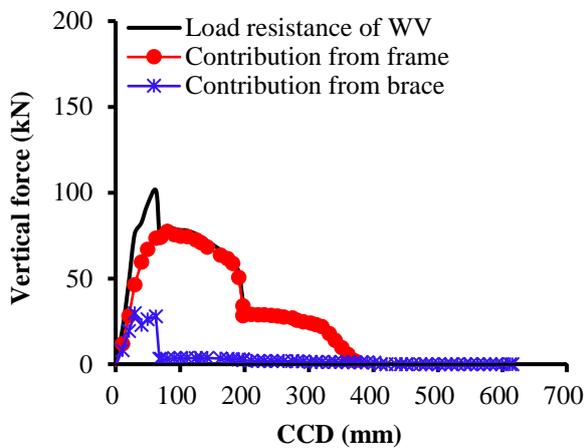


(e)

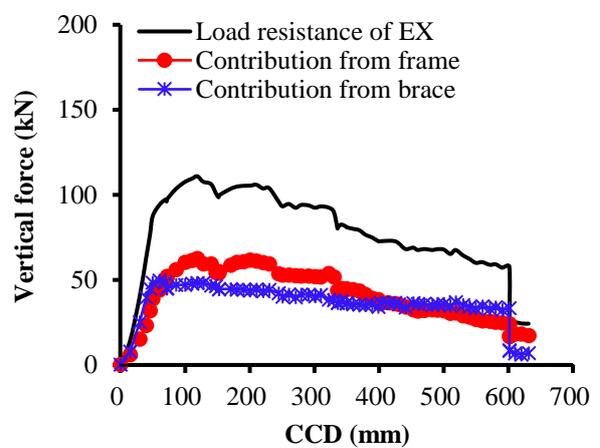
Fig. 13. Load-displacement curves from strain gauge and load cells: (a) WB; (b) WX; (c) WV; (d) EB; (e) EX



(a)

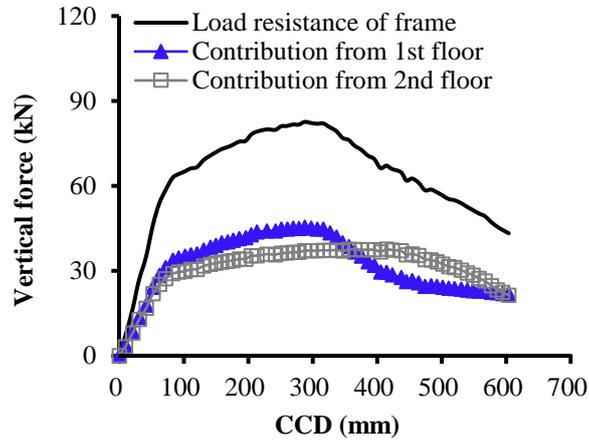


(b)

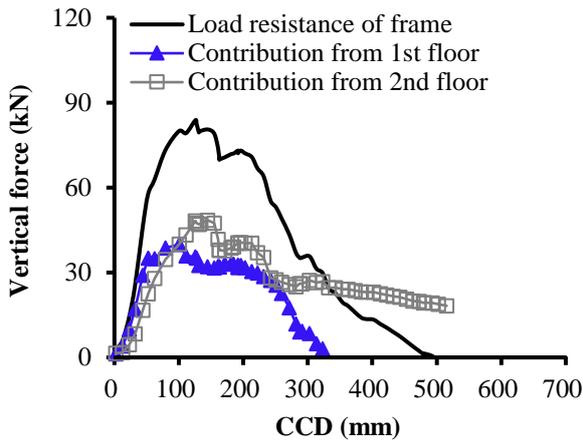


(c)

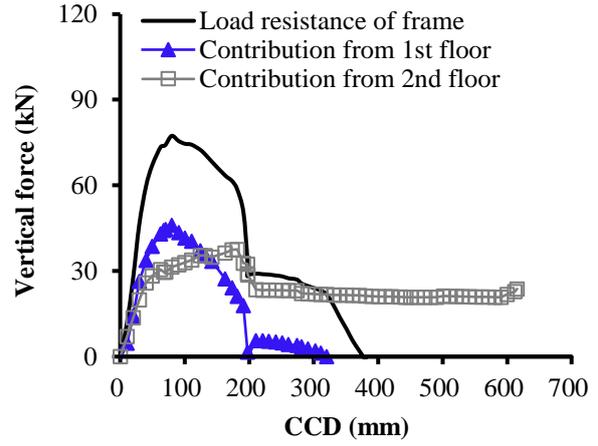
Fig. 14. De-composition of the load bearing capacity from frame and braces



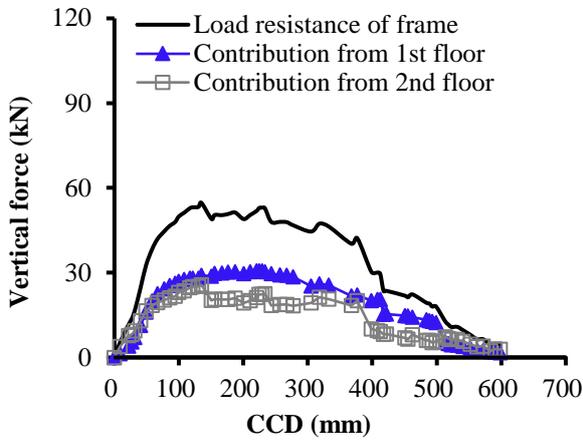
(a)



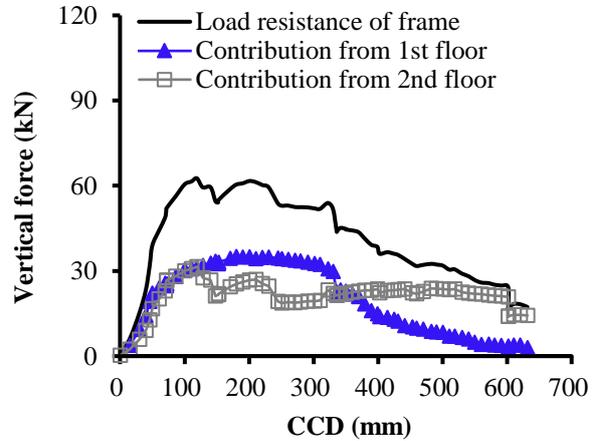
(b)



(c)

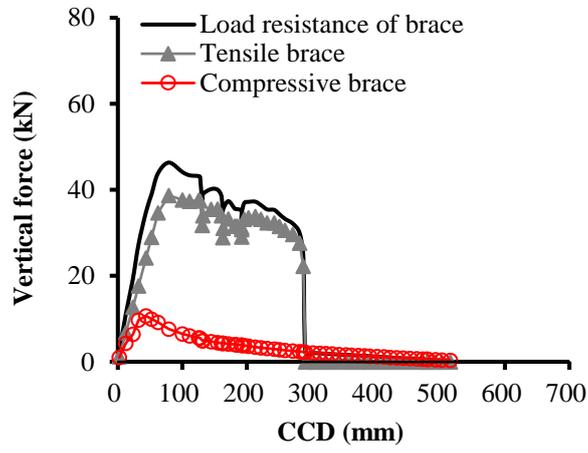


(d)

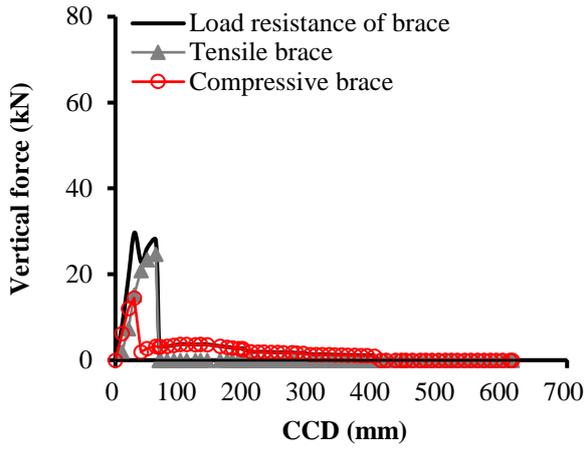


(e)

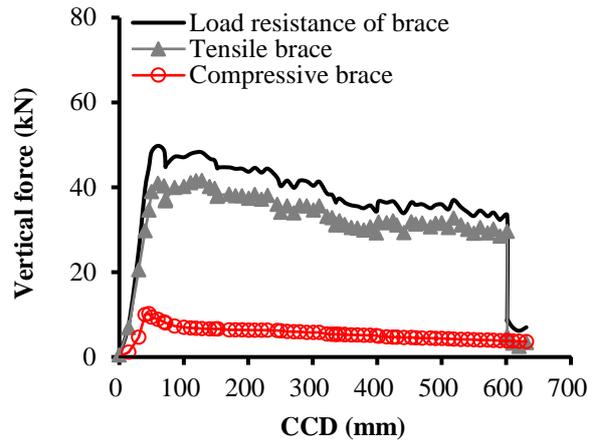
Fig. 15. De-composition of load bearing capacity from the 1st floor and 2nd floor



(a)

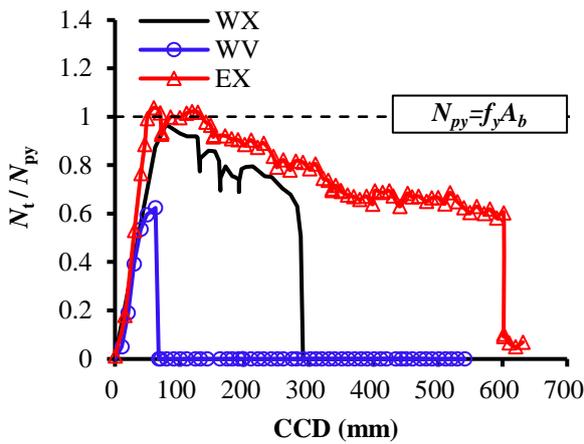


(b)

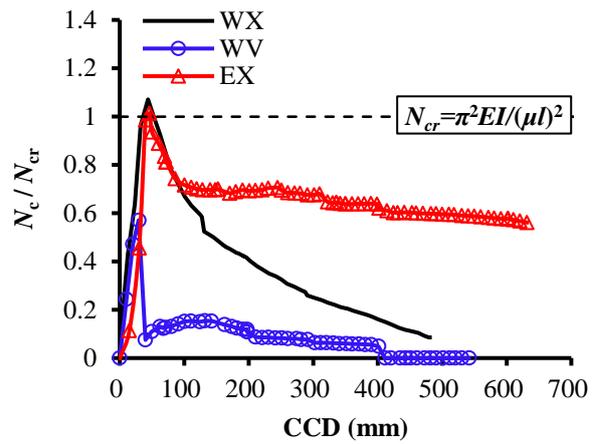


(c)

Fig. 16. De-composition of the load bearing capacity from tensile brace and compressive brace

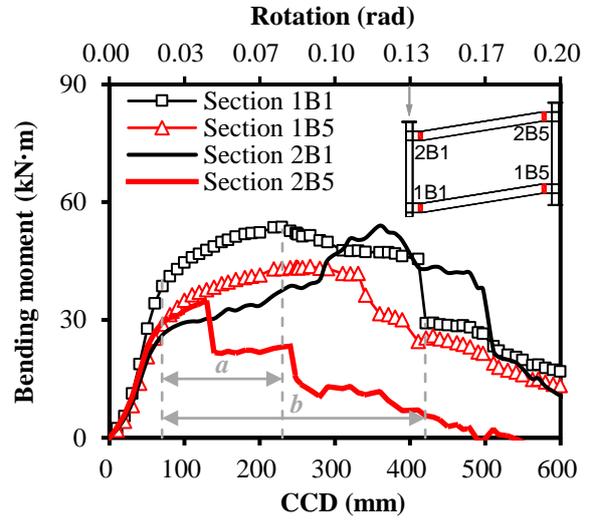
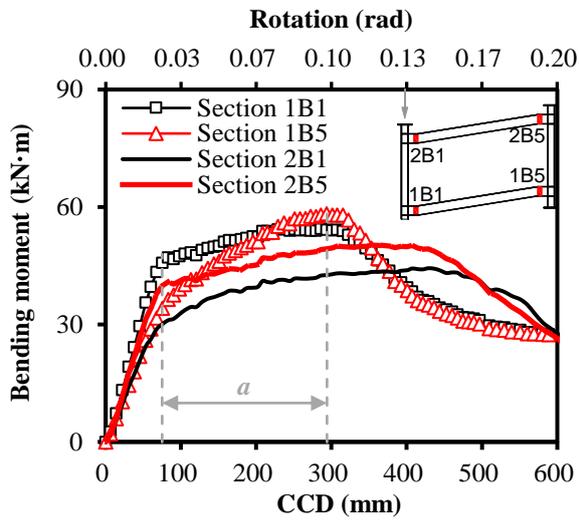


(a)



(b)

Fig. 17. Axial force of braces: (a) tensile brace; (b) compressive brace



(a)

(b)

Fig. 18. Bending moment development at the beam ends: (a) WB; (b) EB

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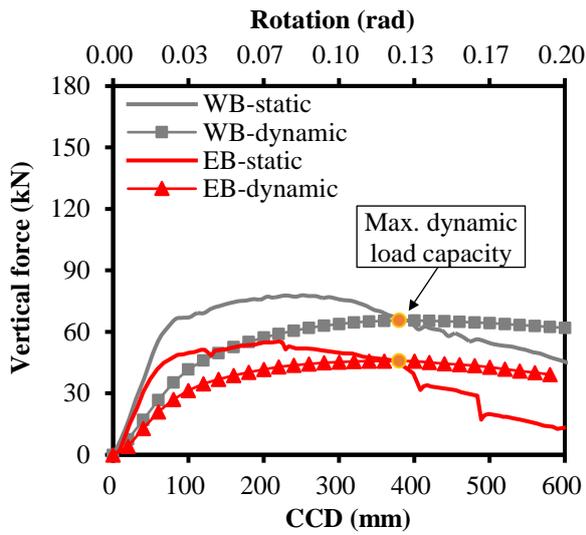
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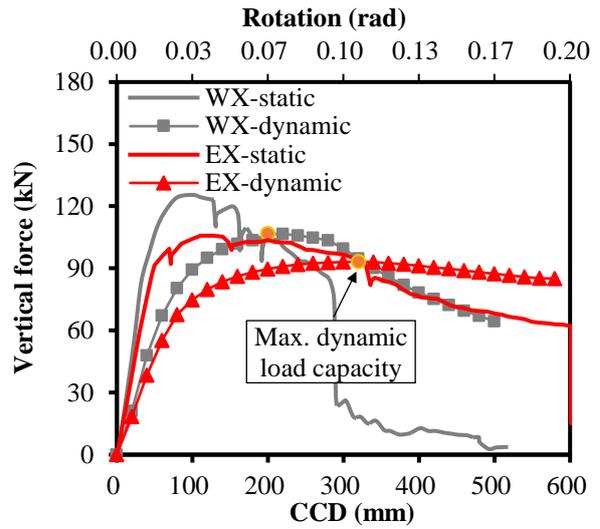
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(a)



(b)

Fig. 19. Comparison of the static and dynamic load resistance: (a) WB and EB; (b) WX and EX

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