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Kai, Qian; Li, Bing

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Performance of Three-Dimensional Reinforced Concrete Beam-Column Substructures under Loss of a Corner Column Scenario

Qian Kai¹ and Bing Li²

ABSTRACT

The vulnerability of conventional reinforced concrete (RC) structures to structural failure due to the loss of corner columns has been emphasized over the past years. However, the lack of experimental tests has led to a gap in the knowledge for the design of RC building structures to mitigate the likelihood of progressive collapse caused by losing a ground corner column. Seven one-third scale RC beam-column substructures were tested to investigate their performance. The variables selected for the test specimens included: beam transverse reinforcement ratios, type of design detailing (non-seismic or seismic) and beam span aspect ratios. Shear failure was observed to have occurred in the corner joint and a plastic hinge was formed at the beam end near to the fixed support in the non-seismic detailed specimens. However, plastic hinges were also formed in the beam end near to the corner joint for the seismically detailed specimen. Vierendeel action was identified as the major load redistribution mechanism before severe failure occurred in the corner joint but a cantilever beam redistribution mechanism dominated after corner joint suffered severe damage. The test results were compared with the DoD design guidelines to highlight the deficiencies of the recently updated guidelines.

CE Database subject heading: Progressive Collapse, Reinforced Concrete, Corner, Three-dimensional, Beam-column, Substructures.

¹Research Associate, School of Civil and Environmental Engineering at Nanyang Technological University, Singapore 639798, qiankai@ntu.edu.sg

²Associate Professor, School of Civil and Environmental Engineering at Nanyang Technological University, Singapore 639798

INTRODUCTION

ASCE/SEI 7 (2010) defines progressive collapse as the spread of an initial local failure from element to element, which eventually results in the collapse of an entire structure or a disproportionately large part of it. In less technical terms, it is often thought of as the domino effect. The collapses of the Ronan Point Tower in London in 1968 and Murrah Federal Building in Oklahoma City in 1995 have demonstrated the disastrous consequences of a progressive collapse. In order to prevent progressive collapse, a structure should have continuity to offer an alternate path to ensure the stability of the structure when a vertical load bearing element is removed. Design guidelines (DoD (2005) and GSA (2003)) have proposed design procedures to evaluate the likelihood of progressive collapse of a structure following the notional removal of the vertical load bearing elements. Although significant improvements were implemented in the recently updated DoD design guidelines (2009) (for the detailed description of these updated points, please refer to Stevens *et al.* (2009) and Marchand *et al.* (2010)), a number of design criteria still need to be subjected to further analysis and verification with experimental data.

In order to better understand the performance of reinforced concrete (RC) frames subjected to different “missing column” scenarios, several experimental and numerical studies have been conducted in recent years. Sasani *et al.* (2007) conducted an in-situ test to study the performance of a RC building with one-way floor slabs supported by transverse frames when subjected to the sudden removal of one of its exterior columns. The behavior of a RC moment frame subjected to the loss of an interior column was also investigated by Yi *et al.* (2008). The efficiency of using Carbon Fiber-Reinforced Polymer (CFRP) retrofitting RC pre-1989 frames, which may be deficient in its continuity subsequent to the loss of an interior column, was investigated by Orton *et al.* (2009). The behavior of axially-restrained beam-column sub-assemblages under the

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scenario of the loss of a column was studied by Su *et al.* (2010). The performance of exterior and interior beam-column sub-assemblages following the loss of one of the ground exterior columns was experimentally studied by Yap and Li (2011) and Kai and Li (2011a), respectively. However, majority of the previous research studies were focused on the frames subjected to the loss of interior or exterior column scenarios while limited studies have been conducted for the case of loss of corner columns. Mohammed (2009) has investigated the implementation of DoD (2005) to protect against progressive collapse of corner floor panels when their dimensions exceeded the damage limits through numerical simulation. A case study of a RC building with different bracing configurations is analyzed using alternate load path method. Ioani and Cucu (2010) have numerically investigated the vulnerability to progressive collapse of a 13-storey RC building with seismic design according to seismic design code P100-1/2006 subjected to the loss of a corner column scenario. Kai and Li (2012) have experimentally studied the dynamic performance of six beam-column substructures under loss of a ground corner columns scenario. The dynamic responses of acceleration, velocity, and displacement were determined. Moreover, the dynamic effects of the beam-column substructure due to sudden removal of a corner column were evaluated. Sasani *et al.* (2008) and Sasani and Sagioglu (2008) have conducted in-situ test to examine the dynamic response and the possibility of progressive collapse of a RC frame when one corner column and adjacent exterior columns were simultaneously demolished by explosion. They concluded that three-dimensional (3D) Vierendeel action of the transverse and longitudinal frames was the major mechanism for the redistribution of loads in the structure. However, the accuracy of the numerical results is needed to improve via comparing with the related experimental results. In addition, the tremendous costs of the in-situ tests mean that it is impossible to systemically investigate the performance of RC frames against progressive

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collapse via this method. Therefore, seven one-third scale RC beam-column substructures were designed and tested at NTU, Singapore to investigate their performance of substructures for progressive collapse caused by losing one of the corner columns. The primary objective of this paper is to gain a better understanding of the behavior of RC substructures under the scenario of being subjected to the loss of one of its ground corner columns. In particular, the following variables were studied: variation of beam transverse reinforcement ratio in the plastic hinge region, seismic design detailing, design span length and span aspect ratio. The results of this study can be used as a basis for the understanding of the behavior of RC structures for progressive collapse.

EXPERIMENTAL PROGRAM

Experimental setup

As observed from Mohammed (2009) and Kai (2011), majority of the deformation of a typical RC frame subjected to the loss of a ground corner column took place in the corner panels while the deformation of the rest of the panels was negligible. Therefore, one typical critical panel (corner panel in the second storey) was extracted and studied. A schematic of the test setup is shown in Fig. 1. The setup can be separated into three components. In Component 1, vertical, axial and rotational constraints were provided at the enlarged adjacent columns to simulate fixed boundary conditions provided by the surrounding structural elements. In Component 2, axial loading in the corner column before the damage was simulated by applying downward displacements at the corner column stub through a hydraulic jack with 600 mm stroke. Both the previous numerical and experimental studies (Sasani and Sagioglu (2008)) indicated that the direction of the bending moment in the beam end near to the corner joint (BENC) was changed after removal of the corner column, and resulted in a considerably positive bending moment

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(tensile at the bottom) being formed in the BENC after the removal of the ground corner column due to Vierendeel action. However, as observed in the deformation shape of the corner joint from Sasani and Sagiroglu (2008), a slight horizontal movement accompanied the vertical movement of the corner joint after removal of the ground corner column and it indicated that the rotational in the BENC was not fully constrained. Component 3 was used to apply this positive bending moment in the BENC for test substructures. Fig. 2 illustrates the detailing of the steel assembly of this component. One strong steel column was connected to the corner stub of the RC specimen using anchor bolts. Four steel pins with high strength and stiffness were utilized to apply the prescribed partial rotational and horizontal constraints in each direction. In other words, the steel column could freely move in the vertical direction but the rotational and horizontal freedoms were partially restrained. The extent of rotational and horizontal constraints applied on the corner joint was related to the allowance between the steel pin and the hole in the steel box (as shown in Fig. 2), which was designed with the aid of ABAQUS (2006). The FE model was validated by comparing the numerical results to the test results attained by Sasani *et al.* (2007). This model was utilized to predict the relationship of the horizontal movement and vertical deflection of the center of the corner joint by pushover analysis. The FEM result indicates the center of the joint just above the lost column has maximum outward horizontal movement about 7.2 mm (0.28 in.) whereas the vertical displacement (D_1) is about 180.0 mm (7.09 in.). The allowance between the steel pin and the hole was designed as follows:

$$\phi = \frac{H_1}{TV} = \frac{H_1}{V + D_1} = \frac{7.2}{625 + 180} = 8.9 \times 10^{-3} \quad (1)$$

$$\delta = \frac{\bar{V} \times \phi}{2} = \frac{350 \times 8.9 \times 10^{-3}}{2} = 1.56 \quad (2)$$

Therefore, the difference between the diameters of the steel pin to the hole was 3 mm, as

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illustrated in Fig. 2.

Experimental substructures

In the current study, the non-seismic and seismic designed nine-storey RC prototype buildings were designed in accordance with Singapore Standard CP 65 (1999) and ACI 318-08 (2008), respectively. It should be noted that the test subjects were assumed to be regular frames for ease of analyzes. Fig. 3 presents basic structural information of the prototype frame in accordance with typical non-seismic designed Specimen F3. For the remaining prototype frames, the dimensions and reinforcement details are given in Table 1. Considering the spatial limitations in the laboratory and difficulties of the transportation, one-third scale tests were conducted. The comparisons between the prototype frames with the model frames are given in Table 1. It should be stressed that the seismic designed prototype building was assumed to be located on a site class of D, stiff soil profile; the design spectral response acceleration parameters, S_{DS} and S_{D1} , were 0.47 and 0.32, respectively. The distributed dead load on the prototype structure due to gravity load of 210 mm thick slab was 5.1 kPa. The super imposed dead load due to ceiling, mechanical ductwork, electrical items, plumbing was assumed to be 1.0 kPa. The equivalent additional dead load due to the weight of in-fill walls and beams were 2.25 kPa and 1.59 kPa, respectively. The live load was assumed to be 2.0 kPa. Thus, the design axial force in the corner column of each specimen as specified by DoD (2009) is determined and listed in Table 2. As illustrated in Fig. 4, each test substructure consisted of two doubly reinforced beams connected with a column stub at the corner and two enlarged adjacent columns at the edges where the rotational and horizontal restraints on beams were applied. The corner column stub representing the removed column was 200 mm square for all specimens. Details of the test substructures are summarized in Table 2. The transverse reinforcement ratio given in Table 2 was determined by

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Eq. 3.

$$\rho_t = A_{sv} / b_v s \quad (3)$$

Fig. 4 illustrates the typical reinforcement layout of the Specimens F2 and F3. The concrete cover of the beam and column was 10 and 20 mm respectively. For F2, the transverse reinforcements were hoop stirrups with 135 degree bends and transverse reinforcement was provided in the joint region. For the remaining specimens, non-seismic detailing was provided, transverse reinforcements were hoop stirrups with 90 degree bends and no transverse reinforcement was installed in the joint region. It should be emphasized that a doubly continuous longitudinal rebar was installed in the beam as scaled specimens were tested in the current study. To prevent bottom longitudinal reinforcement bar pullout at the BENC, 90 degree hooks were employed. The development length of the hooked beam top longitudinal reinforcement into the fixed support was greater than the ACI 318-08 (2008) required design development length. The description of the anchorage details is illustrated in Fig. 4.

Material properties

The target compressive strength of concrete at age 28 days was 30 MPa. The average compressive strength of concrete f'_c obtained from the concrete cylinder samples, was found to be 31.5, 32.1, 31.9, 32.5, 33.1, 32.8, and 33.3 MPa for F1, F2, F3, F4, F5, F6, and F7, respectively. Grade 250 (R6) and Grade 460 (T16, T13 and T10) steel bars were used as transverse and longitudinal reinforcements, respectively. Table 3 gives the measured tensile properties of the bars used in the tests.

Instrumentation

Extensive measuring devices were installed both internally and externally in order to monitor the responses of the test specimens. A total of 100 data channels were active during the testing

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process. A load cell was used to measure the applied force on the corner stub while the deflection shape of the beam was monitored through LVDTs. Three compression/tension load cells were installed in each fixed support. Two of them were vertical and were utilized to determine the vertical reaction force and the bending moment at the fixed support. The remaining horizontal one (Items 10 or 11 in Fig. 1) was used to measure the horizontal constraint force at the fixed support. To monitor the horizontal reaction force applied to the corner joint due to the steel assembly (Item 5 in Fig. 1), two compression/tension load cells (Item 4 in Fig. 1 or the horizontal load cell in Fig. 2) were installed in both longitudinal and transverse constraints. Theoretically, the horizontal reaction force measured in the fixed support (Items 10 and 11 in Fig. 1) and the constraint (Item 4 in Fig. 1) connected with the steel box should be similar. A series of LVDTs and Linear Potentiometers were also placed at various locations of the substructure to measure the different types of internal deformation, such as fixed support rotation, curvature and diagonal deformations. It should be noted that two LVDTs with 25 mm travel were placed in each fixed support to monitor the rigid body rotation of the fixed support (refer to Items 8 and 9 in Fig. 1). The rotational response of each fixed support in each specimen was recorded during the test and the additional vertical deflection in the corner joint due to this rigid body rotation was determined by assuming the beams as cantilever beams. The error of the cantilever assumption can be ignored as the recorded rigid body rotation is limited (for example, the maximum rigid rotation in the transverse fixed support of F3 is 0.00183 rad). It should be noted that the displacement results presented in the following sections refer to the net displacement, which is defined as the deflection after subtracting the additional deflection caused by the rigid body rotation at the fixed supports from the total deflection. A total of about 60 electrical resistance strain gauges were mounted on the reinforcement at strategic locations in order to

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monitor strain variation along the beams, corner column and joint during the test regime. Fig. 4 illustrates the specific locations of these strain gauges.

EXPERIMENTAL RESULTS AND DISCUSSION

A total of seven beam-column substructures with different design detailing and span length were constructed and tested to evaluate the performance of the RC frame when subjected to the loss of a ground corner column. The test results of the seven specimens are summarized in Table 4 and discussed below.

Vertical load and horizontal reaction versus deflection

Influence of transverse reinforcement ratio in the beam plastic hinge region

In order to relate the test results with the performance status of each specimen, the results were normalized by dividing them with design axial force in the corner column. The failure mode of F3 is illustrated in Fig. 5. For F3, the first crack was observed at the beam end near to the fixed support (BENF) at a load of 4.3 kN (0.23). The number 0.23 illustrates that the crack began to occur in the Specimen F3 when the load reached 23 % of the design axial force. However, the first flexural crack was formed in the BENC at a load of 10.0 kN (0.54). This indicates that Vierendeel action was the major load distribution mechanism when the specimen within the elastic response. Following the first crack, joint shear cracks were observed at a load of 21.0 kN (1.13) while the plastic hinges were formed at the BENFs at a load of 22.5 kN (1.21). This yield load obtained a deflection of 28.9 mm. The ultimate capacity of F3 was 25.8 kN (1.39), which corresponded to a deflection of 44.0 mm. Upon further increasing the vertical deflection, the vertical load resistance began to decrease. After the joint shear cracks widened, the strains of the beam longitudinal reinforcement at the BENC started to decrease while the strain of the longitudinal reinforcement at the BENF increased rapidly. This indicated that the resistance

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mechanism is changing to a cantilever beam mechanism and demonstrated that cantilever beam redistribution mechanism dominated the load redistribution after severe shear failure has occurred in the joint. When the vertical deflection reached 275.9 mm, the vertical load resistance began to ascend again and this is attributed to the catenary action developing in the beams. The load resistance at the deflection of 456.2 mm was 11.9 kN (0.64), but it was suddenly reduced to 0.0 kN at a deflection of 461.3 mm due to fracture of the top beam longitudinal rebar occurring in the beam ends near to the fixed supports.

The measured horizontal reaction forces are plotted against vertical deflection in Fig. 6. The terms PL1, PL2, PL3, PL4 and PL5 in the figure represent the first flexural crack, the first yield of the beam longitudinal reinforcement, the ultimate capacity, the normal failure stage which is defined as the resistant capacity dropping to 75.0 % of the ultimate capacity, and the vertical load resistance began to ascend again, respectively. The recorded horizontal compressive force was limited before the first crack occurred in the specimen. However, it significantly increased after the first crack was observed. As shown in Fig. 6, the recorded response of the horizontal reaction force in the fixed support was almost identical as that measured in the horizontal constraint near to the corner column for both the longitudinal and transverse beams. Moreover, similar performance of the horizontal reaction force was measured in both beams before reaching the maximum force. However, the degradation of the horizontal reaction force in the transverse beam was greater than that in the longitudinal beam and lead to the tensile force being transferred to the transverse beam earlier than that in the longitudinal beam. In order to present the load-displacement curve of each specimen distinctly, the average value of the horizontal reaction force measured in the fixed support and the constraint connected with the steel box was plotted in Fig. 9. It should be noted that, for F1, F2, F3, F4 and F5, only the horizontal reaction

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force in the transverse direction was presented. However, for F6 and F7, the horizontal reaction forces measured in both directions were presented due to unequal span of the beams.

In general, the crack pattern development of F1 was similar to that of F3. For F1, the dominating diagonal shear crack was observed in the BENF after severe corner joint shear cracks occurred. The dominating shear cracks degraded the vertical load resistance and loosened the beam horizontal axial constraints. Moreover, severe buckling of the compressive rebar was observed in the BENF when the displacement reached 120.0 mm due to less confinement of the concrete. The ultimate capacity of F1 was 23.7 kN (1.27) and which was only about 91.9 % of the ultimate capacity of F3. The maximum compressive horizontal reaction forces in the transverse and longitudinal beams of F1 were 18.3 (0.98) and 18.6 kN (1.0) while that in the transverse and longitudinal beams of F3 were 19.6 (1.05) and 19.8 kN (1.06), respectively. Thus, it was indicated that the dominating diagonal shear crack reduced the compressive arch action developed in the beam and reduced the ultimate capacity. For F4, no diagonal shear cracks and limited concrete compressive crushing were observed in the BENFs due to a higher transverse reinforcement ratio in the plastic hinge region. The specimen eventually reached the ultimate capacity of 27.5 kN (1.48) and this is about 106.6 % of the ultimate capacity of F3. The failure modes of F1 and F4 can refer to Kai (2011).

Influence of seismic design detailing

F2 was seismically designed and the dimensions and reinforcement details are given in Table 2. For F3, the crack width in the bottom of the BENC did not change after severe joint cracks had occurred. However, for F2, more cracks were formed at the bottom of the BENC, and these cracks became wider when the corner joint suffered more severe damage. Another significant difference between the crack patterns of F2 and F3 was that the core joint concrete kept

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relatively intact due to the joint transverse reinforcement effectively confining the joint concrete of F2 after the cover concrete spalled at a deflection of 280 mm. The higher longitudinal reinforcement ratio provided in the beams significantly increased the first yield load and the ultimate capacity of the specimen, while the higher transverse reinforcement ratio provided in the beam plastic hinge regions delayed the concrete crushing and buckling of the compressive beam longitudinal reinforcement at the bottom of the BENF. F2 reached the ultimate capacity of 36.5 kN (1.96) and was about 141.5 % the ultimate capacity of F3. The maximum compressive horizontal reaction forces in the transverse and longitudinal beams of F2 were 27.3 (1.47) and 27.9 kN (1.50), respectively. It was much higher than that of F3. The failure mode of F2 can be found from Kai (2011).

Influence of design span length and aspect ratio

F5 had clear span of 2775 mm while the clear span of F3 was 2175 mm. However, the span aspect ratio of both specimens is 1.0. The dimensions and reinforcement details are given in Table 2. F5 had a much higher initial stiffness as compared to F3. However, the joint diagonal shear crack was observed at a load of 14.3 kN (0.49) in F5 and it was much lower than that of F3. Due to the joint shear cracks in F5 developed earlier and faster than those in F3, limited flexural cracks were observed in the bottom of the BENC. However, the ultimate capacity of F5 was 26.8 kN (0.92), which was higher than the ultimate capacity of F3 by 3.9 %. However, it should be emphasized that the design axial force in the corner column of F5 based on DoD (2009) was 29.1 kN. Thus, F5 could not survive if the corner column was lost even if the dynamic amplification factor was 1.0. However, the test results presented in this study excluded the resistant contribution of RC slabs and as concluded in Kai and Li (2011b), the RC slab could increase the load resistant capacity by up to 63 % for two-way slabs. The failure mode of F5 is

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presented in Fig. 7.

F6 had unequal span of the beams. The dimensions and reinforcement details are tabulated in Table 2. For F6, the crack developments in the longitudinal and transverse beams were distinctly different and need to be described separately. The first crack was observed in the transverse and longitudinal beams at the loads of 5.9 (0.25) and 10.0 kN (0.43), respectively. Moreover, the first flexural cracks occurred in the transverse and longitudinal BENC at the loads of 10.0 (0.43) and 20.0 kN (0.86), respectively. Asymmetrical joint shear cracks were observed. First joint shear cracks occurred at a load of 17.8 kN (0.77) in the joint face along the transverse direction while the shear cracks occurring in the joint face along the longitudinal direction were at a load of 19.6 kN (0.84). In spite of the cracks in the joint along the longitudinal direction occurred later than the ones along the transverse direction, the development of the cracks in the longitudinal direction was faster than the ones in the transverse direction. The ultimate capacity of F6 was 26.0 kN (1.12) and was about 100.8 % of the ultimate capacity of F3. The maximum average horizontal compressive reaction forces in the transverse beam and longitudinal beam were 19.3 kN (0.83) and 20.9 kN (0.90), respectively. With a further increase in the vertical displacement by 120 mm, concrete crushing was observed in the transverse BENF while the bottom compression region of the longitudinal beam was intact. The concrete crushing was first observed in the longitudinal beam at a deflection of 200 mm. The failure mode of F6 is presented in Fig. 8.

Influence of the longitudinal beam depth for the specimens with unequal span

Similar to F6, F7 had unequal span of the beams. As given in Table 2, the depth of the longitudinal and transverse beam of F7 was 210 mm and 180 mm, respectively. However, the depth of the longitudinal and transverse beam of F6 was 240 mm and 180 mm, respectively. For

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F7, the first flexural cracks were observed in the longitudinal and transverse BENFs at the loads of 3.9 kN (0.17) and 10.0 kN (0.43), respectively. Moreover, the first shear cracks were observed in both faces of the corner joint when the load reached 10.0 kN (0.43). However, the first flexural cracks at the bottom of the transverse BENC were observed at a load of 16.1 kN (0.69). After this load stage, both beams had similar crack development and failure modes. The ultimate capacity of F7 was 23.0 kN (0.99) but the design axial load in the corner column based on DoD (2009) was 23.2 kN. Therefore, similar to F5, F7 it will totally collapse if the corner column is suddenly removed by extreme loads. The maximum average horizontal compressive reaction force in the longitudinal beam and transverse beam were 19.6 kN (0.84) and 18.4 kN (0.79) respectively. Moreover, concrete crushing occurred in both beams simultaneously. Furthermore, more flexural cracks were observed in the longitudinal beam as compared to that of F6. The comparison of the load-displacement relationship of tested specimens is illustrated in Fig. 9. The failure mode of F7 can be found from Kai (2011).

Strain gauge results

Fig. 10 gives the strain profile of the beam longitudinal reinforcement of F3 corresponding to different performance levels (same as the performance level defined in Fig. 6). As shown in Fig. 10, for F3, the strains of the top longitudinal reinforcement at the BENF was tensile and it significantly increased while the strains of the top longitudinal reinforcement at the BENC was compression and it started to decrease after PL3. This was consistent with the crack pattern observation and indicated that Vierendeel action was dominated the load re-distribution when the specimen is in the elastic response. Moreover, the inflection points (zero strain point) of both top and bottom longitudinal reinforcement was moving towards the corner joint after PL3. This indicates that the resistance mechanism of the specimen was changing to cantilever beam

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redistribution mechanism after severe failure had occurred in the corner joint. The strain of the bottom longitudinal reinforcement at the BENF yielded at PL4 while that strain at the BENC never yielded during the test. In general, the strain profile of F2 was similar to F3. For F2, the bottom longitudinal reinforcement at the BENC yielded at PL3. However, it never yielded at that strain in F3 during the test.

Fig. 11 illustrates the strain gauge results of the column longitudinal reinforcement as well as the joint shear reinforcement of F2. It should be noted that rebar C2 was a compressive rebar if 2D longitudinal frame was considered whereas it was a tensile rebar if a 2D transverse frame was considered. Thus, it resulted in the net strain of C2 being limited. On the contrary, the rebar C1 and C4 were tensile and compression in both 2D frames, respectively. Thus, the net strain was much larger than the strain when only the 2D frame was considered. As shown in the figure, the strain of joint transverse reinforcement was limited initially. The strain rapidly increased after the first diagonal shear crack occurred in the corner joint. One consequence of shear is the expansion of the core concrete. The joint transverse reinforcement partially restrained the expansion and appeared to increase the strain. Finally, the strain of the joint transverse reinforcement kept constant with further increase of the displacement. The maximum strain of the joint transverse reinforcement was $2380 \mu\epsilon$. This indicated that the joint transverse reinforcements had not yielded. For other specimens, there were no transverse reinforcements in the joint due to the non-seismic design and detail.

DISCUSSION OF TEST RESULTS

Tie strength method

The design guideline DoD (2009) is a revised version of the previous guideline DoD (2005), incorporating a number of improvements. One of the significant modifications in the DoD (2009)

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compared to DoD (2005) is that horizontal tie forces (internal and peripheral) are no longer permitted to be concentrated in the beams, girders and spandrels (unless the designer can show that these members are capable of carrying the tensile loads while undergoing large rotations, i.e. 0.2 rad). As shown in Table 4, the final rotation of the majority of the beams in the tested specimens was close to 0.2 rad. Thus, the beams can be utilized instead of the floor system to carry the required peripheral tie strength. The required peripheral tie strength F_p (kN) is

$$F_p = 6w_F L_1 L_p \quad (4)$$

The allowable tie strength, which is defined as the maximum horizontal tensile force can be developed when only the beam top rebar is considered, could provide enough tie force to satisfy the required peripheral tie strength in accordance with DoD (2009) (as shown in Table 5). However, the measured maximum tie force was significantly less than the required peripheral tie strength due to partial rotational constraint in the corner joint and limited horizontal constraint could be provided in the corner joint. Thus, using the tie strength method to resist progressive collapse of the RC frames caused by the loss of a corner column is extremely unsafe. Enhance the local resistance of the corner column maybe is an effective alternative choice

Plastic hinges' properties

Fig. 12 illustrates the bending moment in beam fixed support versus vertical displacement of F3. It can be seen from the figure that initially, the bending moment in both the fixed supports was increasing with an increase in the vertical displacement. When the displacement reached 28.9 mm, the bending moment in both fixed supports kept almost constant with increasing vertical displacement. The bending moment measured in the transverse and longitudinal fixed supports began to decrease at a displacement of 130.0 mm and 140.0 mm, respectively. This is possible due to concrete crushing occurring in the beam end near to the fixed support. The measured and

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theoretical maximum bending moment of each beam was tabulated in Table 4. Comparing the measured maximum bending moment of each beam with the theoretical value obtained as ASCE 41-06 (2006) indicated that the recommended over-strength factors in ASCE 41-06 (2006), which were referred by DoD (2009), have been slightly overestimated. Moreover, both the nonlinear static procedure (NSP) and nonlinear dynamic procedure (NDP) need to properly define the plastic hinge properties. The current version of DoD (2009) has adapted the modeling parameters of plastic hinges for the beam element from ASCE 41-06 (2006). The modeling parameters measured for each beam of tested specimens were compared with the recommended parameters in DoD (2009). As illustrated in Table 6, the measured value of parameter “a” was close to the value suggested in DoD (2009). However, the suggested value of parameter “b” in DoD (2009) was extremely conservative. The definition of the parameters “a” and “b” is shown schematically in Fig. 12.

CONCLUSIONS

Based on the experimental and analytical study results, the following conclusions can be drawn:

1. Specimens with seismic detailing saw a 41.5% increase in ultimate capacity compared to F3. The behavior improved mainly due to more longitudinal beam reinforcement installed in the beam, which increased the flexural capacity of the beam section and a medium amount of transverse reinforcement placed in the corner joint region, which also allowed plastic hinge development in the beam end adjacent to the corner joint.
2. As the beam transverse reinforcement ratio was increased from 0.23 % (F1) to 0.31 % (F3), the strength of the tested specimen was enhanced by about 8.9 %. This is due to shear failure that occurred in the plastic hinge region which reduced the effectiveness of compressive arch action and resulted in a lower ultimate capacity. However, when the

transverse reinforcement ratio was increased from 0.31 % (F3) to 0.72 % (F4), the strength of the tested specimen was only enhanced by 6.5 %. This indicates that the effect of the transverse reinforcement ratio in the plastic hinge region for ultimate capacity was limited as long as the shear failure was not severe in the plastic hinge region.

3. F5 reached an ultimate capacity of 26.8 kN while the design axial force of the corner column was 29.1 kN. Thus, F5 will totally collapse even though the dynamic increase factor was 1.0. This confirmed that the specimen with longer design span was more vulnerable than the specimen with shorter design span when they were under similar distributed loads.
4. The plastic hinge properties of RC elements suggested in DoD (2009) are an adaptation of the modeling parameters presented in ASCE 41-06 (2006). The accuracy of these parameters was evaluated by comparing them with the parameters obtained from current tests. In general, the value of parameter “a” recommended in DoD (2009) is reasonable if the beam section is controlled by flexural failure while it is too conservative if the beam section is controlled by flexural and shear failure. Moreover, for parameter “b”, the value suggested in DoD (2009) is extremely conservative. More studies need to be conducted for assessing these modeling parameters.
5. Although DoD (2009) has implemented significant modifications for tie strength design, no difference was proposed between the peripheral tie near to the corner column and the tie near to the exterior column in DoD (2009). After analysis, it was found that the allowable tie strength determined based on the reinforcement details was larger than the required tie strength attained based on DoD (2009); however, the measured horizontal tensile force (tie force) was significantly less than the allowable tie strength due to

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insufficient horizontal constraint that could be provided by the corner joint. Thus, it is suggested that the catenary effect (tie strength method) should not be considered in the practical design for buildings to resist progressive collapse caused by losing one of the ground corner columns.

6. Test results indicated that there are two ways to improve the performance of RC frames against progressive collapse caused by losing a corner column: firstly, increasing the flexural capacity of the beam section by amplifying the beam longitudinal reinforcement ratio. However, it should be pointed out that the increase of the beam flexural capacity is also controlled by the flexural capacity of the column (strong column-weak beam design philosophy). Secondly, upgrading the shear strength of the corner joint by installing more joint transverse reinforcement to confine the corner joint. It should be emphasized that the failure due to rebar anchorage and splice is beyond the scope of this study. For existing buildings use of composite materials to improve the progressive collapse performance must be investigated in the future.

NOTATIONS

a = the rotation difference of the plastic hinge in between the ultimate moment capacity and the yield capacity

A_{sv} = the area of the transverse rebar

b = the rotation difference of the plastic hinge in between the failure point and the yield capacity

b_v = the width of the beam

D_1 = Vertical displacement

H_1 = horizontal movement of the joint center just above the damaged column

L_1 = the span of the frame in the direction under consideration

L_p = equal to 0.3 m for one-third scale model

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s = spacing of the beam transverse reinforcement

S_{DS}, S_{D1} =the design spectral response acceleration parameters for short period and at 1 second period, respectively.

TV = total vertical distance between the center of steel box to the center of corner joint

$\bar{V}$ =average vertical distance between two steel pins in each direction

ρ_t = the transverse reinforcement ratio

w_F = the floor load ($12.9 \text{ kN}/\text{m}^2$ in current study)

ϕ =designed rotation of the steel column

δ =difference between the diameter of the hole and the steel pin

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Table 1-Co-relationship in between the prototype frames and the corresponding test models

Test	Dimensions of the Prototype Beams (mm)		Longitudinal Rebar in the Prototype Beams				Dimensions of the Model Beams (mm)		Longitudinal Rebar in the Model Beams	
	Beam-T	Beam-L	Beam-T		Beam-L		Beam-T	Beam-L	Beam-T	Beam-L
			Top	Bottom	Top	Bottom				
F1	540×300	540×300	2T20+T32	2T20+T32	2T20+T32	2T20+T32	180×100	180×100	4T10	4T10
F2	540×300	540×300	3T32	3T32	3T32	3T32	180×100	180×100	4T13	4T13
F3	540×300	540×300	2T20+T32	2T20+T32	2T20+T32	2T20+T32	180×100	180×100	4T10	4T10
F4	540×300	540×300	2T20+T32	2T20+T32	2T20+T32	2T20+T32	180×100	180×100	4T10	4T10
F5	720×300	720×300	2T25+T32	2T25+T32	2T25+T32	2T25+T32	240×100	240×100	4T10	4T10
F6	540×300	720×300	2T20+T32	2T20+T32	2T25+T32	2T25+T32	180×100	240×100	4T10	4T10
F7	540×300	630×300	2T20+T32	2T20+T32	2T25+T32	2T25+T32	180×100	210×100	4T10	4T10

Note: T32=Deformed bar of 32 mm diameter, T25=Deformed bar of 25 mm diameter, T20=Deformed bar of 20 mm diameter,

T13=Deformed bar of 13 mm diameter, T10=Deformed bar of 10 mm diameter, Beam-L=Longitudinal beam; Beam-T=Transverse beam

Table 2-Specimen properties (unit: mm)

Specimen ID	Elements		Longitudinal rebar		Transverse reinforcement			Design axial load (kN)
	Beam-T	Beam-L	Beam-T	Beam-L	Joint	Beam-T	Beam-L	
Modified Detailed Specimen F1	Type a*	Type a*	0.87 %	0.87 %	None	0.23 %	0.23 %	18.6
Seismically Detailed Specimen F2	Type a*	Type a*	1.47 %	1.47 %	0.49 %	0.95 %	0.95 %	18.6
Control Specimen F3	Type a*	Type a*	0.87 %	0.87 %	None	0.31 %	0.31 %	18.6
Modified Detailed Specimen F4	Type a*	Type a*	0.87 %	0.87 %	None	0.72 %	0.72 %	18.6
Long Span Specimen F5	Type b*	Type b*	0.65 %	0.65 %	None	0.36 %	0.36 %	29.1
Unequal Span Specimen F6	Type a*	Type b*	0.87 %	0.65 %	None	0.31 %	0.36 %	23.2
Unequal Span Specimen F7	Type a*	Type c*	0.87 %	0.75 %	None	0.31 %	0.36 %	23.2

Note: Type a*: Clear span=2175 mm, cross-section=180 x 100; Type b*: Clear span =2775 mm, cross-section=240 x 100

Type c*: Clear span =2775 mm, cross-section=210 x 100; Beam-L=Longitudinal beam; Beam-T=Transverse beam

Table 3-Properties of reinforcing steel

Types	Yield strength f_y (MPa)	Yield strain $\epsilon_y(10^{-6})$	Ultimate strength f_u (MPa)	Ratio of elongation
R6	530	2650	613	20.3 %
T10	575	2895	695	21.7 %
T13	520	2595	637	22.6 %
T16	556	2897	635	21.1 %

Notes: R6= Plain round bar of 6 mm diameter; T10 = Deformed bar of 10 mm diameter

T13 = Deformed bar of 13 mm diameter; T16 = Deformed bar of 16 mm diameter

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Table 4-Test results

Test	Yield load (kN)	Ultimate load, (kN)	MCHR Beam-T (kN)	MCHR Beam-L (kN)	MBM Beam-T (kN.m)	TMBM Beam-T (kN.m)	MBM Beam-L (kN.m)	TMBM Beam-L (kN.m)	Beam-T rotation at FF (rad)	Beam-L rotation at FF (rad)
F1	20.1	23.7	18.3	18.6	15.2	16.6	15.3	16.6	0.199	0.194
F2	29.1	36.5	27.3	27.9	24.8	25.6	25.0	25.6	0.209	0.201
F3	22.5	25.8	19.6	19.8	15.7	16.6	15.9	16.6	0.208	0.199
F4	23.2	27.5	20.2	20.7	16.5	16.6	17.1	16.6	0.187	0.180
F5	25.2	26.8	20.5	20.3	20.8	23.5	21.6	23.5	0.164	0.173
F6	21.5	26.0	19.3	20.9	16.4	16.6	23.6	23.5	0.197	0.155
F7	21.0	23.0	19.6	18.4	16.7	16.6	18.7	20.0	0.201	0.159

Note: MCHR= Maximum Compressive Horizontal Reaction

MBM, TMBM=Maximum Bending Moment and Theoretical Maximum Bending Moment, respectively

FF= Final Failure stage defined as totally lose the resistance capacity

Table 5-Comparison the measured tie force with the requirement tie force determined based on DoD (2009)

Test	RTTB (kN)	RTLB (kN)	ATTB (kN)	ATLB (kN)	MTTB (kN)	MTLB (kN)
F1	55.7	55.7	72.2	72.2	8.3	8.8
F2	55.7	55.7	122.1	122.1	11.1	11.3
F3	55.7	55.7	72.2	72.2	7.9	7.5
F4	55.7	55.7	72.2	72.2	7.5	7.5
F5	69.7	69.7	72.2	72.2	4.3	3.1
F6	55.7	69.7	72.2	72.2	6.9	1.1
F7	55.7	69.7	72.2	72.2	1.7	1.3

Note: RTTB, RTLB= Required Tie Force in the Transverse Beam and Longitudinal Beam, respectively

ATTB, ATLB= Allowable Tie Force in the Transverse Beam and Longitudinal Beam, respectively

MTTB, MTLB= Measured Tie Force in the Transverse Beam and Longitudinal Beam, respectively

Table 6-Comparison the measured plastic hinge's parameters with the modeling parameters suggested in DoD (2009)

Test	a in Beam-T (radians)	a in Beam-L (radians)	a in DoD* (radians)	b in Beam-T (radians)	b in Beam-L (radians)	b in DoD* (radians)
F1	0.031	0.035	0.05	0.181	0.189	0.06
F2	0.058	0.063	0.063	0.198	0.198	0.10
F3	0.046	0.046	0.05	0.146	0.177	0.06
F4	0.052	0.052	0.05	0.143	0.161	0.06
F5	0.042	0.042	0.05	0.134	0.134	0.06
F6	0.040	0.038	0.05	0.137	0.140	0.06
F7	0.043	0.037	0.05	0.143	0.136	0.06

Note: DoD*= DoD (2009); Beam-T=Transverse Beam; Beam-L=Longitudinal Beam

1: Load cell measure applied load
 2: Hydraulic jack with 600 mm stroke
 3: Steel column
 4: Comp/tension load cell
 5: Steel assembly
 6: LVDT with 300 mm travel
 7: RC substructure
 8 and 9: LVDTs
 10, 11, 12, and 13: Comp/tension load cells

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Figure 2

Fig. 2: The detailing of steel assembly

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Figure 3

Fig. 3: The Plan and elevation view of the prototype frame in accordance with Specimen F3

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Figure 4

Fig. 4: Dimensions and reinforcement details of F2 and F3

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Figure 5

Fig. 5: Cracking patterns of F3 at failure

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Figure 6

Note: THR=Transverse horizontal reaction force; LHR=Longitudinal horizontal reaction force

Fig. 6: Measured horizontal reaction force versus vertical displacement of F3

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Figure 7

Fig. 7: Cracking patterns of F5 at failure

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Figure 8

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