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Experimental and Analytical Assessment on RC Interior Beam-Column Subassemblages for Progressive Collapse

Qian Kai¹; and Bing Li²

Abstract: Experimental and analytical studies carried out on a reinforced concrete (RC) moment resisting frame after it is subjected to a loss of its ground storey exterior column is presented within this paper. Four full-scale interior beam-column subassemblages, detailed with varying degrees of non-seismic detailing and improved detailing reinforcements, were subjected to a monotonic loading regime to simulate the effects of re-distributed gravity loads on the subassemblage after the loss of an exterior ground column. The variables in the test specimens include the beam longitudinal reinforcement ratios and the spacing of the transverse reinforcement within the beams, columns and joints. Load-displacement relationships, crack development patterns and failure mechanism obtained from the tests are also discussed. The finite element models are validated by comparing the results with the experimentally obtained data. Parametric studies are then performed to study the influences of various beam transverse reinforcement ratio, and incorporation of an additional exterior beam-column element and slab on the global behavior of the subassemblages.

CE Database subject headings: Reinforced concrete; Column; Residual axial capacity; Finite element analysis; Progressive collapse.

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1 **Introduction**

2 Progressive collapse is characterized by widespread propagation of failure following localized
3 damage to a small portion of a structure. Consideration of how to prevent progressive collapse is
4 not new to the structural engineering community. Several codes and design guidelines
5 (ASCE/SEI 7 2010; GSA 2003; DoD 2009) have formulated their own approach to mitigate
6 progressive collapse of a structure. Although each approach is different, they generally share the
7 same principles: alternate load path, local resistance and integration of continuity requirements.
8 Amongst these methods, the alternate load path method was considered as a major technique in
9 mitigating progressive collapse of the moment resisting frames.

10 A number of researches, design codes, and standards have been reviewed and/or compared in
11 these literatures (Nair 2006; Ellingwood 2006; Mohamed 2006). Generally speaking, the
12 investigated issues are involved with abnormal loading events, assessment of loading, analysis
13 methods and design philosophy. Innumerable numerical studies also have been conducted in the
14 past decade. Marjanishvili (2004) studied the advantage and disadvantage of the above
15 procedures when applying them in the progressive collapse analysis. Powell (2005) compared
16 the linear static (LS), nonlinear static (NS) and nonlinear dynamic (ND) analyses approaches. It
17 was found that the dynamic amplification factor using “2.0” as suggested in the guidelines for
18 static analysis can result in extreme conservative results and insisted in that nonlinear analysis
19 method should be utilized. Ruth *et al.* (2006) found that dynamic amplification factor using 1.5
20 better represents the dynamic effect especially for steel frames. Marjanishvili and Agnew (2006)
21 compared the four analysis methods (LS, NS, LD and ND) by analyzing a nine-story steel
22 moment-resistant frame building. It was found that the four analysis methods had their own

merits. The static and dynamic analyses need to be incorporated properly in order to achieve the best results for progressive collapse analysis.

Despite notable analytical studies (as mentioned in the above), relatively limited experimental data exist as the basis of assessing progressive collapse resistance of reinforced concrete frame structures undergone large deformation. Sasani *et al.* (2007) conducted an in-situ test of a reinforced concrete building with one way floor slabs supported by transverse frames. The dynamic performance of the building after suddenly removal of an exterior ground bearing column was studied. The behavior of a RC moment frame subjected to a loss of an interior column was investigated by Yi *et al.* (2008). The loss of an exterior column in the event of a terrorist attack is more prone to triggering progressive collapse than the loss of an interior column due to lower catenary (beam) or membrane (slab) actions can be developed because of reduced horizontal constraint provided from the surrounding element when the frame loss of an exterior column. It should be pointed out that one of the critical regions of the frame after losing an exterior column was the interior beam-column subassemblages. However, to date there have been limited tests conducted to assess the behavior of RC interior beam-column subassemblages under the loss of an exterior column scenario. Therefore, a series of experimental studies was conducted in Nanyang Technological University, Singapore to assessment the performance of the interior beam-column subassemblages for progressive collapse. Previous researchers (Corley *et al.* 1998) indicated that improved detailing (seismic detailing) might help to enhance the resistance of buildings against progressive collapse. Thus, one of main objectives of this study is to evaluate the effects of different reinforcement detail (non-seismic detailing or improved detailing) on progressive collapse resistance of interior beam-column subassemblages after the loss of a ground exterior column.

Description of Test Program

Design of Test Setup

An eight-storey RC moment resisting frame as shown in Fig. 1 designed according to the provisions within the British Standards (IBC 2006) was utilized for the investigation. The live load is taken to be 2.4 kPa at each story level. Assuming that the dead load consists of the self weight of the building structure together with 1.8 kPa additional dead load is applied on the floors. Fig. 2(a) and Fig. 2(b) illustrate the change in the bending moment within the frame before and after losing its exterior ground column respectively. The setup and loading procedure were determined through the bending moment diagram of the structural frame after losing its exterior column. The points of contra-flexure were chosen to be the end boundaries of the subassemblages because of the zero moment condition that could be easily attained in the test setup. Fig. 3 shows the free body diagram and the representative simplified boundary conditions of the subassemblages. Fig. 4 depicts the configuration for the loading of the frame. A monotonic vertical load was applied on the free end of the left beam using a 2000 kN hydraulic jack. The bottom of the column was pinned to a strong floor while the top of the column was pinned through two strong frames. The right beam end was connected to the strong floor through a steel link, which allowed for rotation and horizontal movement of the beam while restricting movement in the vertical direction. The column axial load was applied using hydraulic jack placed between column top end and the bottom suffix of the steel plate. Four threaded rods were each fixed at four corners around the test unit to balance the applied axial load.

Loading Method

The axial load was slowly applied on the column prior to the commencement of each test in balanced steps until the designated level of $0.3f'_c A_g$ was achieved. The vertical force was applied statically on the free end of the left beam in a displacement-controlled manner.

Test Specimens

Two series of interior beam-column subassemblies, referred to as 'NS' (non-seismic detailing) and 'LS' (improved detailing) series were constructed and tested. The variables in test specimens include the spacing of transverse reinforcement at the beams and columns near to the joint region, and within the joint region; and the percentage of beam longitudinal reinforcement. Fig. 5 illustrates the schematic dimensions and reinforcement details of all test specimens. In the NS series (Specimens I1 and I2), hoop stirrups with 90° bend were utilized as transverse reinforcements. No transverse reinforcement provided within the joint regions. Lapping of the column longitudinal bars just above the floor level was included in this series. High-yield steel was used for the longitudinal reinforcements while mild-steel was used for the transverse reinforcements. In the LS series (Specimens I3 and I4), closer transverse reinforcement spacing at the beams and columns near to the joint region was used. The percentage of beam bottom longitudinal reinforcement is same as the top and two layers of transverse reinforcement were provided in the joint regions. Column longitudinal bars were continuous throughout the floor level. High-yield steels were used for both the longitudinal and transverse reinforcements.

Material Properties

Longitudinal reinforcement for the beams and columns consisted of deformed bars, designated using letter T and were characterized by a yield strength f_y of 505.6 MPa. The transverse reinforcement of all specimens in NS series comprised of mild steel bars, denoted by R and were

characterized by a yield strength f_y of 461.1 MPa. The average compressive strength of concrete, f'_c , obtained from the concrete cylinder samples, was found to be 29.5 MPa

Instrumentation

To monitor the response of the test specimens, extensive measuring devices were installed or mounted both internally and externally. Almost 100 data channels were active during the test process. Two independent load cells were used to measure the applied vertical force on the free end of the left beam as well as the reaction force on the roller of the right beam. The displacements at the left beam end, where loading was applied, were measured using a 100 mm-LVDT. A series of LVDTs and Linear Potentiometers were also placed at various locations of the specimens to measure the different types of internal deformation. About 45 electrical resistance strain gauges were mounted on reinforcing bars at specific locations.

Test Observations and Results

Cracking Patterns and Failure Mechanism

The behavior of all test specimens was controlled by the formation of a plastic hinge in the left beam. As shown in Fig. 6, severe cracking and spalling of the concrete at the left beam near to the joint region together with local buckling of the longitudinal reinforcement were observed in all test specimens. The diagonal shear cracks occurred in the joint region of Specimens I1 and I2 at a load of 80.0 kN and 90.0 kN respectively; whereas the first sign of diagonal cracking was observed in Specimens I3 and I4 at a load of 120.0 kN and 150.0 kN respectively. The columns of all test specimens were almost intact except for several hairline flexural cracks that were observed.

Load-Displacement Response

Fig. 7 shows the vertical applied force versus displacement at the free end of the left beam of all test specimens. In general, similar trends of the curves were observed in all test specimens. As shown in Fig. 7(a), a linear relationship between the vertical displacement in beam and the vertical applied force was observed up to a load of 30.6 kN, where the first crack was developed in Specimen I1. After which, the slope of the curve decreased slightly. After loading to a load of 142.0 kN, the stiffness of the specimen reduced significantly due to the yielding of the beam longitudinal reinforcing bars. The resistance of Specimen I1 dropped suddenly after reaching a maximum strength of 195.5 kN. After this stage, flexural tension cracks began to progress and penetrate into the compression zone. Significant concrete spalling was observed in the compression zone near the column surface of left beam as shown in Fig 6(a). A similar behavior was recorded in other specimens. A comparison between the key parameters of force-displacement responses of all test specimens is illustrated in Table 1. As compared to Specimen I1 (typical NS specimen), Specimen I4 (typical LS specimen) had a yield strength (YS), ultimate strength (US) and ultimate displacement (UD) larger than that of Specimen I1 by about 13 %, 17 % and 38 % respectively. The major reason is Specimen I4 has higher percentage of the beam transverse reinforcement within its beam located near the joint region.

Reaction Force-Displacement Response

The reaction force in the roller of the right beam was measured by the load cell as shown in Fig. 4. Fig. 8 illustrates the comparison of the reaction force versus the vertical displacement at the free end of the right beam response in each specimen. The curves were almost linear up to a reaction force of about 0.55 times the vertical yielding force of each specimen. After that, a mild slope was observed in these curves till failure of the specimens. As compared to Specimen I2, Specimen I3 has a slightly higher maximum reaction force. Because the vertical loading apply

on the left beam will create a moment at the left beam-column interface. This moment should be balanced by the top, bottom column component together with right beam. The contribution of the moment for these three components is dependent on their relative stiffness. For Specimen I3, which has higher beam longitudinal reinforcement ratio, has higher relative stiffness compared with Specimen I2. This result in the right beam of Specimen I3 contributes larger resistant moment and has higher reaction force.

Strains in Reinforcing Bars and Concrete

The strain profiles of longitudinal beam and column bars corresponded to characteristic load stages (first cracking, yield strength and ultimate strength) are plotted for specimens of typical NS and LS series in Figs. 9-12. Fig. 9 shows strains in the top and bottom longitudinal reinforcing beam bars of Specimen I1 at different loading stages. As shown in Fig. 9(a), the recorded strain in the top longitudinal reinforcement of the left beam at 175 mm from the column center-line exceeded a yield strain of 2516μ at a load of 142.0 kN. Upon loading to a maximum force of 195.5 kN, yielding in the top longitudinal reinforcing bar extended to a distance of 775 mm from the column center-line. No yielding was observed in the right beam throughout the test. As shown in Fig. 9 (b), no compressive yielding was recorded in the bottom reinforcing bar till the end of the test. The maximum compressive strain in the bottom reinforcing bar was observed at 175 mm from the column center-line.

Fig. 10 illustrates the strain profiles of the longitudinal reinforcement bars within the column of Specimen I1. It can be seen that the strains in longitudinal reinforcing column bars of Specimen I1 were significantly smaller than its yield strain, indicating that the column of the specimen was in its elastic region throughout the test.

Figs. 11 and 12 illustrate the strain profiles of the longitudinal reinforcement in beam and column of Specimen I4, respectively. General trends of the graphs were similar to that observed in Specimen I1. The strains in the top longitudinal reinforcing bar of the right beam were relatively small. No tensile yielding was observed in these locations. Similar to Specimen I1, the strains in the bottom longitudinal reinforcing bars of the beam and longitudinal reinforcing bars of the column of Specimen I4 did not exceed the yield strain till the end of the test.

Fig. 13 shows the relationships of the strain in the joint transverse reinforcement versus the applied vertical force of Specimen I4. It can be seen that the strain was less than $100\ \mu\epsilon$ before Specimen I4 reaches yield load. Although the strain increase rapidly when the specimen close to the ultimate capacity, the maximum strain recorded in the joint transverse rebar is less than $500\ \mu\epsilon$. This is consistent with the crack development patterns of Specimen I4 (Noted: only hairline cracks observed in the joint). Fig. 14 presents the relationship of the concrete strain at the bottom of left beam (175 mm from the column center-line) versus the applied vertical force of Specimen I4. It is to be noted that at near failure stages of the test, crushing and spalling of the concrete located at the bottom of the beam near the column interface had damaged instruments and gauges at that location. Therefore, the strain in concrete was only shown up to a loading of 179.3 kN.

Force Transferring Mechanism

In this part of the paper, statically indeterminate truss models were developed to predict the behavior of the test specimens. Material and geometrical properties for each member in the truss model are necessary for the analysis of a statically indeterminate truss model. The model proposed by Kent and Part (1971) was adopted to represent the concrete. A bilinear stress-strain relation, with the tangent modulus in the strain-hardening regime taken to be 0.01 of the elastic

modulus was used for the reinforcing bars. The geometrical properties for each member were defined following the suggestions made by Khoo and Li. (2007). Fig. 15 presents the possible truss model for Specimen I1. Similar models were applied to other specimens except for some modifications in the joint region as shown in Fig. 16.

Fig. 16 shows the internal stress distribution in each specimen at its last stage of loading. The stresses are expressed in terms of f'_c, f_y for concrete and steel members, respectively. For the diagonal compression struts with cracks parallel to the struts as expected, Schlaich and Schafer. (1991) recommended that the strength of the concrete is $0.8f'_c$. Based on the above mentioned models, the maximum compressive stress in the joint diagonal strut of Specimen I1, I2, I3 and I4 were $0.50f'_c, 0.50f'_c, 0.41f'_c$ and $0.42f'_c$ respectively, which were much less than the allowable concrete stress. Therefore, there was no severe damage observed in the joint panels during the tests. The compressive stress in the major joint diagonal strut of Specimen I4 was slower by 18 % as compared to that of Specimen I1, which was attributed to the joint transverse reinforcement within the joint panel of Specimen I4.

The longitudinal tensile chords justifiably yielded when the induced tensile stress exceeded the nominal yield strength f_y . As shown in Fig. 16, yielding was observed in the tensile chord members near the column interface in all test specimens, which was consistent with the experimental results.

Fig. 17 presents the distribution of stresses in the concrete and reinforcement along the beam compression chords of Specimen I3 at its first yield and ultimate load. It is to be noted that the compression chords in the model were a combination of compression reinforcement and concrete components; thus, it is interesting to note that the compressive forces could be contributed between these two components. The concrete stress ratio of the compressive chord

C3 reached $-0.74f'_c$ and concrete block carried about 65 % of the total induced force at the first yield load of 160.0 kN. Compression force carried by the compression reinforcement increased significantly when loading to the ultimate load. At the failure stage, only about 17 % of the total compressive force was carried by the concrete. This implied that the concrete has been stressed beyond its ultimate strain, causing the loss of its compressive strength. In fact, concrete crushing was observed at the left beam-column interface at the failure stage.

Evaluate the Dynamic Effect

A key issue in progressive collapse is to understand that it is a dynamic and nonlinear event. The relationship between the performance of the frames under quasi-static and dynamic load scenario is dynamic amplification factors. GAS (2003) suggested a constant factor 2.0 to account for the dynamic effect for both LS and NS analysis. The previous researchers (Powell 2005; Ruth *et al.* 2006) have found that the dynamic amplification factor 2.0 to relate the NS and ND analysis is extremely conservative. Recently updated guideline DoD (2009) have re-defined the dynamic amplification factor by decoupling of the load increase factor (LIF) and dynamic increase factors (DIF) to be considered individually as the LS procedure and NS procedure, respectively. DIF was used to relate the NL analysis to the ND analysis.

The equation used to determining the DIF introduced in DoD (2009) for RC frame is:

$$DIF = 1.04 + 0.45 / (\theta_{aP} / \theta_y + 0.48) \quad (1)$$

The chord rotation of each specimen is compared with the acceptance criteria provided in DoD (2009) in Table 2. As illustrated in the Table, the acceptance criteria provided in the DoD (2009) is extremely conservative possible due to the acceptance criteria given in DoD (2009) is adapted or adopted the acceptance criteria in ASCE 41-06 (2006). However, it should be emphasized that the acceptance criteria provided in ASCE 41-06 (2006) is obtained from seismic

tests. The DIF for Specimens I1, I2, I3 and I4 are 1.08, 1.10, 1.10, and 1.09 respectively based on Eq. 1. The value of DIF for test specimens is significantly less than 2 due to the behavior of the test specimens expressed considerable ductile. The dynamic ultimate strength of each specimen is also given in Table 2.

Finite Element Analysis

In this part of the paper, finite element (FE) analysis was carried out to study the response of the test specimens. Parametric studies were then performed to investigate the effects of beam transverse reinforcement ratio, an additional exterior beam-column element and slab on the global behavior of the subassemblages. The present study uses the ABAQUS (2006) package for the analysis.

Material Model

In this study, the plasticity-based model is used to represent concrete as proposed by Lublinear *et al.* (1989). According to CEB-FIP Model code (1993), the tensile strength of concrete f_t was:

$$f_t = 0.30(f'_c)^{2/3} \quad (2)$$

The compression hardening behavior of the concrete was defined based on Saenz (1964)'s suggestions as:

$$\sigma_c = \frac{E_0 \varepsilon_c}{1 + \left(\frac{E_0 \varepsilon_0}{f'_c} - 2\right) \left(\frac{\varepsilon_c}{\varepsilon_0}\right) + \left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2} \quad (3)$$

The tension softening relationship proposed by Gopalaratnam and Shah (1985) was used to simulate the tension softening behavior of the concrete as follows.

$$\sigma_t = f_t e^{-k w^\lambda} \quad (4)$$

This non-linear constitutive model has the following advantages: (1) assumes the plain concrete to be an equivalent isotropic continuum and assumes two main failure mechanisms,

1 tensile cracking and compressive crushing of the concrete material; (2) tension stiffening option
2 allows for the definition of strain softening for the cracked concrete and also allows the effects
3 of the reinforcement interaction with concrete to be modeled (Gil and Bayo 2008); and (3) a
4 nonlinear stress-strain relationship enabling the weakening of the material under increasing
5 compressive stresses. The steel reinforcement is modeled as an elasto-plastic material with strain
6 hardening beyond its elastic phase. It is assigned a bilinear stress-strain relationship, with the
7 tangent modulus in the strain-hardening regime taken to be 0.01 of the elastic modulus.

8 ***Verification of Finite Element Model***

9 The analytical results were compared with those obtained from the experiment to verify the
10 accuracy of the FE models. The FE models have the same geometry configuration and
11 dimensions as the test specimens. The material properties of concrete and reinforcing steels were
12 modeled based on the measured values. Concrete was modeled using a solid eight nodes with
13 reduced integration element (C3D8R) and the reinforcement steel bars were modeled as two
14 nodes linear 3D truss elements (T3D2) whose nodes were embedded within concrete elements.
15 In order to prevent the stress concentration at specific point, several elastic plates were placed at
16 the ends of beams and columns, which are the locations of boundary conditions and loading.
17 These elastic plates were also modeled using solid eight nodes with reduced integration element
18 (C3D8R). Similar boundary conditions as experimental setup were applied on this FE studies.
19 The constant axial load on the top of the column was applied as a distributed loading while the
20 vertical load at the end of left beam was applied through a displacement control mode.

21 ***Computed Responses***

22 The comparison between the analytically and experimentally observed load-displacement
23 responses of the test specimens is shown in Fig. 7. The analytical response seems to be

consistent with the experimental observations from all test specimens. However, the initial stiffness of the analytical result was slightly higher. This is due to micro-cracks formed due to drying shrinkage and during handling when transporting the concrete specimens. These would cause a reduction in the stiffness of the specimens as compared to the finite element models on which these micro-cracks were not present. The small gap present in the pin boundary of the top and bottom columns and roller boundary of the right beam end could be another reason for the difference in stiffness. In comparison the boundary conditions in the finite element models would not have such gaps present. The ultimate displacement from the FEM was slightly lower as compared with that of the experimental result. This is probably because of the interaction between the rebar and concrete that is indirectly considered through tension stiffening option. When the beam-column subassemblages attain large deformations, the relatively larger slip between the rebar and concrete may not be well reflected by this tension stiffening option and thus resulting in the finite element model attaining a slightly lower ultimate displacement. The general behavior in terms of strength, ductility and yield loading between the experimental and analytical results were in a good agreement. The minimum principal stress distribution in the concrete of Specimen I4 together with its deformed shape at the first yield is shown in Fig. 18. Extensive deformations were observed in the left beam, whereas only a small amount of deformations was contributed by the column and joint.

Comparisons of the analytical and experimental results of all specimens showed that the vertical load versus vertical displacement responses obtained from the FE analyses were similar to the experimental observations. From the aforementioned observations and predictions of the global behaviour using the FE analysis, the use of FE modelling techniques can, therefore, be further extended to study the behaviour of the subassemblages by varying different parameters.

Parametric Studies

To further improve the understanding of structural response of interior beam-column subassemblages under the loss of a column scenario, the following section presents the application of the FE modeling technique to investigate the most critical parameters such as the beam transverse reinforcement ratio, and incorporating additional exterior beam-column element and slab.

Influence of the Percentage of Transverse Reinforcement in the Plastic hinge zone

The experimental results of both NS and LS series showed that an increase in the percentage of transverse reinforcement in the plastic hinge zone improved the performance of the test specimens. This section of the paper further investigates the effect of the percentage of transverse reinforcement in the plastic hinge zone. The percentage of the transverse reinforcement in the plastic hinge zone of Specimen I4 varied from 0.251 % to 1.256 %. As shown in Fig. 19, the strength and maximum displacement are only increased by 3 % and 9 % respectively, with an increase in the percentage of the transverse reinforcement from 0.25 % to 0.314 %. However, with an increase from 0.628 % to 0.837 % in the percentage of the transverse reinforcement, the strength and maximum displacement are enhanced by about 10.5 % and 109 % respectively. Further increase in the percentage of the transverse reinforcement, only provided an enhancement in the maximum displacement.

Influence of the Length of Strengthening Zone

Severe flexural cracks were observed in the region out of the supposed plastic hinge zone and the tension chord out of the supposed hinge region in the modified truss model had near yield stress. Both indicated that yielding of tensile rebar exceeded the potential plastic hinge zone when the frame had lost one of its exterior columns. As required in seismic detailing, higher

transverse reinforcement ratio was provided in the potential plastic hinge zone to strength the beam. In order to further understanding the effect of the length of strengthening zone exceeded the length of potential plastic hinge zone on the performance of interior beam-column subassemblages under loss of an exterior column scenario, the length of strengthening zone were varied from $1.0 d$ to $1.8 d$ and the other characteristics are same as Specimen I4. Fig. 20 presents the load-displacement responses corresponding to different length of strengthening zone. There was no significant change in US by increasing the length of strengthening zone. However, there was a consistent increase in UD as the length of the strengthening zone was increased. It can be seen that a change in length from $1.4 d$ to $1.6 d$ increased the UD of about 32 %. However, increasing its length from $1.6 d$ to $1.8 d$, only provided an increase in the UD about 13 %. Therefore, due to giving practicality and economy, the authors recommend extending of the length of strengthening zone from d to $1.6 d$ to achieve the optimum ductility behavior of a RC structural frame subjected to a loss of its exterior column scenario.

Influence of Additional Exterior Beam-Column Element

As shown in Fig. 2b, the exterior beam-column element just above the removed column also provides resistance and has a distinct deformation. This indicates that the exterior beam-column element can provide additional strength and stiffness to re-distribute the loading, which is originally carried by column that is removed. In order to study this effect, one sub-frame having the same detailing of the beam and column components as that of Specimen I4 was modeled through FE. Fig. 21 illustrates the FE model of this sub-frame including a view of the boundary conditions and loading configuration. A span length of 5400 mm was selected for sub-frame I4 to enable the distance from the center of the column to the inflection point on the beam to coincide with that of Specimen I4. Comparing the load-displacement responses of Specimen I4

with that of I4-subframe in the Fig. 22, it can be seen that the exterior beam-column element can increase the YS and US about 31 % and 18 % respectively. However, the UD decreases by about 29 % if failure is defined by reduction of the resistance capacity by more than 30 %. Moreover, it is interesting to note that I4-subframe, unlike Specimen I4 had its resistance decreasing upon attaining its UD but almost kept constant when it reached a stage near point B illustrated in Fig. 22. This only can be explained by the catenary effect. In order to further understand this catenary effect, another series of tests were conducted by the authors.

Influence of Slab

In monolithic reinforced concrete structures, a portion of the floor slabs acts as flanges to the beams, thereby increasing the strength and stiffness of the beams. Consequently, floor slabs can have a significant contribution to the resistance of a structure during progressive collapse, which should not be ignored in the design stage. Two FE models with an added RC slab flange on the Specimen I4 and I4-subframe were created respectively. The effective length of the additional slab in the FE models is 900 mm. The top and bottom layer of reinforcement along the beam length is $T12@350mm$ while the slab reinforcement perpendicular to the beam length is $T10@225mm$. The analytical global response of these FE models is shown in the Fig. 22. For Specimen I4, including the slab flange can increase the YS and US by 20 %, 18 % respectively. However, the UD decreased by about 21 %. For I4-subframe, including the slab flange in the FE model increased the YS and US by 33 %, 25 % respectively while the UD decreased by about 40 %.

This indicated that the slab worked as beam flange and significantly increased the stiffness and strength of the structural frame when subjected to a loss of its exterior ground column scenario. However, it should be noted that for a real structural frame, the slab not only worked as

the flange of the beam but also provided resistance to structural frame through a membrane effect. It is unfortunate that the membrane effect cannot be incorporated into this study due to the 2D frame models utilized. However, this would be an interesting topic to study in future.

Conclusions

Based on the experimental and finite element numerical study results, the following conclusions can be drawn:

- For modern designed structural frame satisfying the strong column and weak beam design philosophy, the failure mechanism of the interior beam-column subassemblages is normally due to the formation of plastic hinge in the beam instead of failure of the joint panel or columns under the loss of an exterior ground column scenario.
- The dynamic effect on the test subassemblages was limited due to the test specimens performed with considerable ductile. Moreover, the acceptance criteria provided in the DoD (2009) is extremely conservative for beam element against progressive collapse. Further dynamic tests are needed to well capture the dynamic performance of the frame under the loss of an exterior column scenario.
- Improved detailing (seismic detailing) can significantly improve the global behavior of the RC frames in resisting progressive collapse caused by the loss of an exterior column scenario.
- With regards to the rescue and survival, the ductility of the RC structural frame is extremely important. In order to increase the ductility of RC structural frame for loss of exterior column scenario, the authors recommend extending the length of strengthening zone from d to $1.6d$. Moreover, increasing the transverse reinforcement ratio in the beam plastic hinge zone is another effective method.

- 1 ● The finite element model incorporating the exterior beam-column element indicated that
2 catenary action could develop in the beam to resist progressive collapse caused by the
3 loss of an exterior column. However, the extent of the resistance contribution from
4 catenary action is limited due to no obviously re-ascending branch was observed in the
5 load-displacement relationship. The whole bay tests should be conducted in the future to
6 further understand the catenary action developing in the frame against the progressive
7 collapse caused by the loss of an exterior column scenario.
- 8 ● The FE results indicated that ignoring the slab contribution to resist the progressive
9 collapse is extremely conservative. However, the slab membrane effect could not be
10 investigated due to the limitations of the 2D frame models in this paper. This is an
11 interesting phenomenon and its effects should be studied in future.
- 12 ● The experimental study conducted also provides evidence that the boundary conditions
13 used for future experimental studies can be simplified. Instead of utilizing complicated
14 boundary conditions to simulate the constraint of the surrounding element on the studied
15 beam or slab, which was in the bay of the loss of column, surrounding elements can be
16 simulated by an equivalent fixed end that has the combined stiffness of top, bottom
17 column and right beam element summed up together. This simplified set-up is illustrated
18 in Fig. 23.

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22 Technological University, Singapore, Any opinions, findings and conclusions expressed in this
23 paper are those of the writers and do not necessarily reflect the view of DSTA, Singapore.

1 **Notations**

2 A_s = area of tension reinforcement layer

3 A_g = gross area of section

4 d = effective depth of beam

5 E_0 = initial stiffness

6 f'_c = concrete compressive strength

7 f_t = concrete tensile strength

8 k = constant

9 λ = constant

10 w = crack width

11 ε_0 = concrete ultimate strain

12 ε_c = concrete compressive strain

13 σ_c = concrete compressive stress at any compressive strain ε_c

14 σ_t = concrete tensile stress

15 θ_{ap} = allow plastic rotation angle defined as chord rotation in here.

16 θ_y = yield rotation angle defined as chord rotation in here

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Captions to tables and figures

Table 1-Comparison of special parameters for all specimens

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Table 1. Comparison of Special Parameters for All Specimens

Specimen ID	First cracking (kN)	Yielding Strength (kN)	First Joint Cracking (kN)	Ultimate Strength (kN)	Static Ultimate Displacement (mm)
I1	30.6	142.0	80	195.5	237.4
I2	30.0	160.8	90	204.9	251.9
I3	29.3	160.0	120	206.5	299.7
I4	30.0	161.1	150	228.9	327.0

Table 2. Comparison of DIF and Dynamic Ultimate Strength for All Specimens

Specimen ID	Measured θ_y (rad)	Measured θ_{ap} (rad)	Acceptance Criteria in DoD (2009)	DIF	Dynamic Ultimate Strength (kN)
I1	0.014	0.132	0.050	1.09	179.4
I2	0.018	0.137	0.063	1.10	186.3
I3	0.019	0.142	0.063	1.10	187.7
I4	0.019	0.178	0.063	1.09	210.0

Fig.1

Fig. 2a

Fig. 2b

Fig. 3

Fig 4.

Fig. 5a

Fig.5b

Fig. 6

Fig. 7

Fig. 8

Fig.9

(a) Top longitudinal beam rebar-I1

(b) Bottom longitudinal beam rebar-I1

Fig. 10

Fig. 11

(a) Top longitudinal beam rebar-I4

(b) Bottom longitudinal beam rebar-I4

Fig. 12

Fig. 13

Fig. 14

Fig.15

Fig. 16a

Fig. 16b

Fig. 16c

Fig. 17

Fig. 18

Fig. 19

Fig. 20

Fig. 21

Fig. 22

Fig. 23