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

Kai Qian, A.M.ASCE1; and Bing Li2

Abstract: Experimental and analytical studies carried out on a RC moment-resisting frame after it is subjected to the loss of its ground-story exterior column is presented in this paper. Four full-scale interior beam-column subassemblages with varying degrees of nonseismic detailing were subjected to a monotonic loading regime to simulate the effects of redistributed 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. The 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 influence factors, such as the beam transverse reinforcement ratio and the incorporation of an additional exterior beam-column element and slab on the global behavior of the subassemblies. DOI: 10.1061/(ASCE)CF.1943-5509.0000284, © 2012 American Society of Civil Engineers.

CE Database subject headings: Reinforced concrete; Beam columns; Finite element method; Progressive collapse; Experimentation.

Author keywords: Reinforced concrete; Column; Residual axial capacity; Finite-element analysis; Progressive collapse.

Introduction

Progressive collapse is characterized by widespread propagation of failure following localized damage to a small portion of a structure. Consideration of how to prevent progressive collapse is new to the structural engineering community. Several codes and design guidelines [ASCE 2010; General Services Administration (GSA) 2003; Department of Defense (DoD) 2009] have formulated their own approach to mitigate progressive collapse. Although each approach is different, they share the same principles: alternate load path, local resistance, and integration of continuity requirements. Among these methods, the alternate load path method has been considered to be a major technique in mitigating progressive collapse of moment resisting frames.

A number of studies, design codes, and standards have been reviewed and/or compared in Nair (2006), Ellingwood (2006), Mohamed (2006), and Li et al. (2011). Generally, the investigated issues are involved with abnormal loading events, assessment of loading, analysis methods, and design philosophy. Numerous numerical studies also have been conducted in the past decade. Marjanishvili (2004) studied the advantages and disadvantages of the aforementioned procedures when applying them in progressive collapse analysis. Powell (2005) compared the linear static, nonlinear static, and nonlinear dynamic analyses approaches. It was found that adopting a dynamic amplification factor of 2.0 as suggested in the guidelines for static analysis can result in extremely conservative results. Ruth et al. (2006) found that using a dynamic amplification factor of 1.5 better represents the dynamic effect, especially for steel frames. Marjanishvili and Agnew (2006) compared four analysis methods, linear static, nonlinear static, linear dynamic, and nonlinear dynamic, by analyzing a nine-story steel 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 to achieve the best results for progressive collapse analysis.

Despite the notable analytical studies mentioned previously, limited experimental data exist on assessing the progressive collapse resistance of RC frame structures undergoing large deformation. Sasani et al. (2007) conducted an in situ test of a RC building with one-way floor slabs supported by transverse frames. The dynamic performance of the building after the sudden removal of an exterior ground bearing column was studied. The behavior of a RC moment frame subjected to the 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 as a result of the lower catenary (beam) or membrane (slab) actions that develop because of reduced horizontal constraint provided from the surrounding element when the frame loses an exterior column. It should be pointed out that one of the critical regions in the frame after losing an exterior column is the interior beam-column subassemblage. However, 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 at Nanyang Technological University, Singapore, to assess the performance of interior beam-column subassemblages in progressive collapse. Previous research (Corley et al. 1998; Yap and Li 2011) indicated that improved detailing (seismic detailing) may help to enhance the resistance of buildings against progressive collapse. Thus, one of the main objectives of this study is to evaluate the effects of reinforcement detail (nonseismic detailing or improved detailing) on the progressive collapse resistance of interior beam-column subassemblages after the loss of a ground exterior column.
Description of the Test Program

Design of the Test Setup

An eight-story RC moment resisting frame (shown in Fig. 1), designed according to the provisions within the British Standards [International Code Council (ICC 2006)], was utilized for the investigation. The live load was taken to be 2.4 kPa at each story level. It was assumed that the dead load consisted of the self-weight of the building structure and an additional dead load of 1.8 kPa was applied to the floors. Figs. 2(a and 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 contraflexure were chosen to be the end boundaries of the subassemblies 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 subassemblies. 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 2,000 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 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 the 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 to the column prior to the commencement of each test in balanced steps until the designated level of 0.3$f_yA_e$ 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 (nonseismic detailing) and LS (improved detailing) were constructed and tested. The variables in the test specimens include the spacing of the transverse reinforcement at the beams and columns near to the joint region and within the joint region and the percentage of the 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 a 90° bend were utilized as the transverse reinforcements. No transverse reinforcement was 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 the joint region was used. The percentage of beam bottom longitudinal reinforcement was the same as the top, and two layers of transverse reinforcement were provided in the joint regions. The column longitudinal bars were continuous throughout the floor level.

![Fig. 1. Sketch of the investigated RC frame (in mm)](image)

![Fig. 2. Schematic bending moment diagrams of the investigated RC frame under gravity loads: (a) before loss of the exterior column; (b) after loss of the exterior column](image)

![Fig. 3. Illustration of the free-body diagram and representative simplified boundary condition of the interior beam-column subassemblies](image)
Fig. 4. Overview of a specimen in position ready for testing

Fig. 5. Detailing of tested specimens: (a) Specimens I1 and I2; (b) Specimens I3 and I4
High-yield steels were used for both the longitudinal and transverse reinforcements.

Material Properties

The longitudinal reinforcement for the beams and columns consisted of deformed bars, designated by T, and were characterized by a yield strength $f_y$ of 505.6 MPa. The transverse reinforcement of all specimens in the NS series was 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 various types of internal deformations. About 45 electrical resistance strain gauges were mounted on the 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 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 loads of 80.0 and 90.0 kN, respectively; whereas the first sign of diagonal cracking was observed in Specimens I3 and I4 at loads of 120.0 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 the beam and the vertical applied force was observed up to a load of 30.6 kN, where the first crack developed in Specimen I1. After which, the slope of the curve decreased slightly. After loading to 142.0 kN, the stiffness of the specimen was reduced significantly because of the yielding of the beam longitudinal reinforcing bars. The resistance of Specimen I1 dropped suddenly after reaching a maximum strength.
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 the left beam as shown in Fig. 6(a). Similar behavior was recorded in other specimens. A comparison among the key parameters of the force-displacement responses of all test specimens is given in Table 1. Compared with 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 that Specimen I4 had a higher percentage of beam transverse reinforcement within its beam located near the joint region.

Fig. 6. Cracking patterns of the test specimens at failure

Fig. 7. Comparison between the experimental and analytical load displacement responses

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 the left beam as shown in Fig. 6(a). Similar behavior was recorded in other specimens. A comparison among the key parameters of the force-displacement responses of all test specimens is given in Table 1. Compared with 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 that Specimen I4 had a higher percentage of beam transverse reinforcement within its beam located near the joint region.
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 are almost linear up to a reaction force of about 0.55 times the vertical yielding force of each specimen. After that, a mild slope is observed in these curves until failure of the specimens. Compared with Specimen I2, Specimen I3 has a slightly higher maximum reaction force because the vertical loading applied 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 the right beam. The contribution of the moment to these three components is dependent on their relative stiffness. Specimen I3 had a higher beam longitudinal reinforcement ratio and higher relative stiffness compared with Specimen I2. This result for the right beam of Specimen I3 contributed a larger resistant moment and higher reaction force.

### Reaction Force-Displacement Response

Table 1. Comparison of Special Parameters for All Specimens

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>First cracking (kN)</th>
<th>Yielding strength (kN)</th>
<th>First joint cracking (kN)</th>
<th>Ultimate strength (kN)</th>
<th>Static ultimate displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I1</td>
<td>30.6</td>
<td>142.0</td>
<td>80</td>
<td>195.5</td>
<td>237.4</td>
</tr>
<tr>
<td>I2</td>
<td>30.0</td>
<td>160.8</td>
<td>90</td>
<td>204.9</td>
<td>251.9</td>
</tr>
<tr>
<td>I3</td>
<td>29.3</td>
<td>160.0</td>
<td>120</td>
<td>206.5</td>
<td>299.7</td>
</tr>
<tr>
<td>I4</td>
<td>30.0</td>
<td>161.1</td>
<td>150</td>
<td>228.9</td>
<td>327.0</td>
</tr>
</tbody>
</table>

### Strains in the Reinforcing Bars and Concrete

The strain profiles of the longitudinal beam and column bars corresponding to the characteristic load stages (first cracking, yield strength, and ultimate strength) are plotted for the specimens of the 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 various 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 centerline exceeded a yield strain of 2516με 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 centerline. 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 until the end of the test. The maximum compressive strain in the bottom reinforcing bar was observed at 175 mm from the column centerline.

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 the 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 the beam and column of Specimen I4. The general trends of the graphs were similar to those 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 until 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 100με before Specimen I4 reached yield load. Although the strain increased rapidly when the specimen was close to the ultimate capacity, the maximum strain recorded in the joint transverse rebar was less than 500με. This is consistent with the crack development patterns of Specimen I4 (where only hairline cracks were observed in the joint). Fig. 14 presents the relationship of the concrete strain at the bottom of the left beam (175 mm from the column centerline) versus the applied vertical force of Specimen I4. It is noted that at the 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 is only shown up to a loading of 179.3 kN.

### Force Transferring Mechanism

Sparingly indeterminate truss models were developed to predict the behavior of the test specimens. The material and geometrical properties for each member in the truss model are necessary for analysis of a statically indeterminate truss model. The model proposed by Kent and Park (1971) was adopted to represent the concrete. A bilinear stress-strain relationship, 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 $f_c'$ and $f_y$ for the 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 be $0.8f_c'$. Based on the aforementioned models, the maximum

---

compressive stresses in the joint diagonal struts of Specimens I1, I2, I3, and I4 were 0.50, 0.50, 0.41, and 0.42 f_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% compared with 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 f_y strength. 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 the stresses in the concrete and reinforcement along the beam compression chords of Specimen I3 at its first yield and ultimate load. It is 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 to both of these components.

The concrete stress ratio of Compressive Chord C3 reached $\frac{20}{74}f_c$ and the concrete block carried about 65% of the total induced force at the first yield load of 160 kN. The 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 had been stressed beyond its ultimate strain, causing 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 the quasi-static and dynamic load scenario involves the dynamic amplification factors. The GSA (2003) suggested a constant factor of 2.0 to account for the dynamic effect for both linear static and nonlinear static analysis. Powell (2005) and Ruth et al. (2006) found that a dynamic amplification factor of 2.0 to relate the nonlinear static and nonlinear dynamic analysis is extremely conservative. The DoD (2009) updated guideline redefined the dynamic amplification factor by decoupling of the load increase factor and dynamic increase.
Fig. 16. Internal stress distribution of the test specimens at failure: (a) Specimen I1; (b) Specimen I2; (c) Specimen I3; (d) Specimen I4
of the test specimens expressed considerable ductility. The dynamic acceptance criteria provided in ASCE 41-06 (ASCE 2006) is obtained
\[ \text{DIF} = 1.04 + 0.45/(\theta_{up}/\theta_{y} + 0.48) \]  
(1)

The chord rotation of each specimen is compared with the acceptance criteria provided in DoD (2009) in Table 2. As illustrated in Table 2, the acceptance criteria provided in DoD (2009) are extremely conservative, which is possibly because the acceptance criteria given in DoD (2009) adapted or adopted the acceptance criteria in ASCE 41-06 (ASCE 2006). However, it should be emphasized that the acceptance criteria provided in ASCE 41-06 (ASCE 2006) is obtained from seismic tests. The DIFs for Specimens I1, I2, I3, and I4 were 1.08, 1.10, 1.10, and 1.09, respectively, based on Eq. (1). The DIF values for the test specimens were significantly less than 2 because the behavior of the test specimens expressed considerable ductility. The dynamic ultimate strength of each specimen is also given in Table 2.

Finite-Element Analysis

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 the beam transverse reinforcement ratio and an additional exterior beam-column element and slab on the global behavior of the subassemblies. The current study used ABAQUUS (ABAQUUS/CAE 2006) in the analysis.

Material Model

In this study, the plasticity-based model was used to represent concrete as proposed by Lubliner et al. (1989). According to the Comité Euro-International du Béton–Fédération International de la Précontrainte (CEB-FIP) model code (CEB 1993), the tensile strength of concrete, \( f_t \), was

\[ f_t = 0.30 (f'_c)^{2/3} \]  
(2)

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

\[ \sigma_c = \frac{E_0 e_c}{1 + \left( \frac{E_0 e_0}{f'_c} - 2 \right) \left( \frac{e_c}{e_0} + \frac{e_c}{e_0} \right)^2} \]  
(3)

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

\[ \sigma_t = f_t e^{-k w^+} \]  
(4)

This nonlinear constitutive model has the following advantages: (1) it assumes the plain concrete to be an equivalent isotropic continuum and assumes two main failure mechanisms, tensile cracking and compressive crushing of the concrete material; (2) tension stiffening option allows for the definition of strain softening for the cracked concrete and also allows the effects of the reinforcement interaction with concrete to be modeled (Gil and Bayo 2008); and (3) a nonlinear stress-strain relationship enables the weakening of the material under increasing compressive stresses. The steel reinforcement was modeled as an elastoplastic material with strain hardening beyond its elastic phase. It was assigned a bilinear stress-strain relationship, with the tangent modulus in the strain-hardening regime taken to be 0.01 of the elastic modulus.

Verification of the Finite-Element Model

The analytical results were compared with those obtained from the experiment to verify the accuracy of the FE models (FEMs). The FEMs had the same geometry configuration and dimensions as the test specimens. The material properties of the concrete and reinforcing steels were modeled based on the measured values. The concrete was modeled using a solid 8-node reduced integration element (C3D8R) and the reinforcement steel bars were modeled as 2-node linear three-dimensional (3D) truss elements (T3D2), in which the nodes were embedded within the concrete elements. To prevent the stress concentration at specific points, several elastic plates were placed at the ends of the beams and columns, which were the locations of the boundary conditions and loading. These elastic plates were also modeled using C3D8R. Similar boundary conditions as in the experimental setup were applied to these FE studies. The constant axial load on the top of the column was applied as distributed loading while the vertical load at the end of the left beam was applied through a displacement control mode.

Computed Responses

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

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Measured ( \theta_y ) (rad)</th>
<th>Measured ( \theta_{up} ) (rad)</th>
<th>Acceptance criteria in DoD (2009)</th>
<th>Dynamic increase factor</th>
<th>Dynamic ultimate strength (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I1</td>
<td>0.014</td>
<td>0.132</td>
<td>0.050</td>
<td>1.09</td>
<td>179.4</td>
</tr>
<tr>
<td>I2</td>
<td>0.018</td>
<td>0.137</td>
<td>0.063</td>
<td>1.10</td>
<td>186.3</td>
</tr>
<tr>
<td>I3</td>
<td>0.019</td>
<td>0.142</td>
<td>0.063</td>
<td>1.10</td>
<td>187.7</td>
</tr>
<tr>
<td>I4</td>
<td>0.019</td>
<td>0.178</td>
<td>0.063</td>
<td>1.09</td>
<td>210.0</td>
</tr>
</tbody>
</table>
experimental observations from all test specimens. However, the initial stiffness of the analytical result was slightly higher. This was caused by microcracks that formed as a result of shrinkage during the drying and handling when transporting the concrete specimens. These caused a reduction in the stiffness of the specimens compared with the FEMs in which these microcracks were not present. The small gap present in the pin boundary of the top and bottom columns and the roller boundary of the right beam end could be another reason for the difference in stiffness. In comparison, the boundary conditions in the FEMs would not have such gaps present. The ultimate displacement from the FEM was slightly lower compared with that of the experimental result. This is probably because of the interaction between the rebar and concrete that was indirectly considered through the tension stiffening option. When beam-column subassemblies attain large deformations, the relatively larger slip between the rebar and concrete may not be well reflected by this tension stiffening option and, thus, will result in the FEM attaining a slightly lower ultimate displacement. The general behavior in terms of strength, ductility, and yield loading between the experimental and analytical results was in 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 behavior using the FE analysis, the use of FE modeling techniques can, therefore, be further extended to study the behavior of the subassemblies by varying different parameters.

**Parametric Studies**

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

![Fig. 18. Deformed shape in Specimen I4](image-url)
Influence of the Percentage of Transverse Reinforcement in the Plastic Hinge Zone

The experimental results of both the 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. The effect of the percentage of transverse reinforcement in the plastic hinge zone was further investigated. 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 were 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 were 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 the Strengthening Zone

Severe flexural cracks were observed in the region outside the supposed plastic hinge zone, and the tension chord outside the supposed hinge region in the modified truss model had near yield stress. Both indicated that yielding of the tensile rebar exceeded the potential plastic hinge zone when the frame had lost one of its exterior columns. As required in seismic detailing, a higher transverse reinforcement ratio was provided in the potential plastic hinge zone to strengthen the beam. To further understand the effect of the length of the strengthening zone exceeding the length of the potential plastic hinge zone on the performance of interior beam-column subassemblies under loss of an exterior column scenario, the length of the strengthening zone were varied from 1.0 to 1.80d and the other characteristics were same as for Specimen I4. Fig. 20 presents the load-displacement responses corresponding to various lengths of the strengthening zone. There was no significant change in the US by increasing the length of the strengthening zone. However, there was a consistent increase in the UD as the length of the strengthening zone was increased. It can be seen that a change in length from 1.4 to 1.6d increased the UD by about 32%. However, increasing its length from 1.6 to 1.8d only provided an increase in the UD by about 13%. Therefore, for practical and economical purposes, the writers recommend extending the length of strengthening zone from 1.0 to 1.6d to achieve the optimum ductility behavior of a RC structural frame subjected to loss of its exterior column.

Influence of an Additional Exterior Beam-Column Element

As shown in Fig. 21(b), 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 redistribute the loading, which is originally carried by column that is removed. To study this effect, one subframe having the same detailing of the beam and column components as that of Specimen I4 was modeled through FE modeling. Fig. 21 illustrates the FEM of this subframe including a view of the boundary conditions and loading configuration. A span length of 5.400 mm was selected for Subframe 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 Subframe I4 in Fig. 22, it can be seen that the exterior beam-column element can increase the YS and US by about 31 and 18%.

Fig. 19. Influence of the beam transverse reinforcement percentage

Fig. 20. Influence of the length of the strengthening zone

Fig. 21. Finite-element model of the complete subframe

Fig. 22. Influence of the slab and additional exterior beam-column element
The dynamic effect on the test subassemblages was limited. For structural frames, designed as per modern design guidelines, conclusions can be drawn:

Based on the experimental and FE numerical studies, the following conclusions can be drawn:

- The FE results indicated that ignoring the slab contribution to resist the resistance capacity by more than 30%. Moreover, Subframe 14, unlike Specimen 14, had its resistance decrease upon attaining its UD but was almost kept constant when it reached a stage near Point B, as illustrated in Fig. 22. This only can be explained by the catenary effect. To further understand this catenary effect, another series of tests were conducted by the writers.

**Influence of the Slab**

In monolithic RC 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 FEMs with an added RC slab flange for Specimen 14 and Subframe 14 were created, respectively. The effective length of the additional slab in the FEMs was 900 mm. The reinforcement of the top and bottom layers along the beam length was T12 at 350 mm, while the slab reinforcement perpendicular to the beam length was T10 at 225 mm. The analytical global response of these FEMs is shown in the Fig. 22. For Specimen 14, including the slab flange can increase the YS and US by 20 and 18%, respectively. However, the UD decreased by about 21%. For Subframe 14, including the slab flange in the FEM increased the YS and US by 33 and 25%, respectively, while the UD decreased by about 40%.

This indicated that the slab worked as a beam flange and significantly increased the stiffness and strength of the structural frame when subjected to loss of its exterior ground column. However, for a real structural frame, the slab not only worked as the flange of the beam but also provided resistance to the structural frame through a membrane effect. It was unfortunate that the membrane effect could not be incorporated into this study because of the two-dimensional (2D) frame models that were utilized. However, this would be an interesting topic to study in the future.

**Conclusions**

Based on the experimental and FE numerical studies, the following conclusions can be drawn:

- For structural frames, designed as per modern design guidelines satisfying the strong column and weak beam design philosophy, the failure mechanism of the interior beam-column subassemblages is caused by the formation of the plastic hinge in the beam instead of the failure of the joint panel or columns.
- The dynamic effect on the test subassemblages was limited because the test specimens were considerably ductile. Moreover, the criteria provided in DoD (2009) for beam elements are extremely conservative. Further dynamic tests are needed to capture accurately the dynamic performance of the frame following the loss of an exterior column.
- Improved detailing (seismic detailing) can significantly improve the global behavior of RC frames in resisting progressive collapse caused by the loss of an exterior column.
- Regarding rescue and survival, the ductility of the RC structural frame is extremely important. To increase the ductility of the RC structural frame after the loss of an exterior column, the writers recommend extending the length of the strengthening zone from 1.0 to 1.6d. Increasing the transverse reinforcement ratio in the beam plastic hinge zone could be another effective method that can be considered.
- The FEM incorporating the exterior beam-column element indicated that catenary action could develop in the beam to resist progressive collapse caused by the loss of an exterior column. However, the extent of the resistance contribution from catenary action is limited because no obvious reassuming branch was observed in the load-displacement relationship. Whole bay tests should be conducted in the future to further understand the catenary action that develops in the frame under progressive collapse caused by the loss of an exterior column.
- The FE results indicated that ignoring the slab contribution to resist the progressive collapse is conservative. However, the slab membrane effect could not be investigated because of the limitations of the 2D frame models that were utilized. This is an interesting phenomenon and its effects should be studied in the future.
- The experimental study conducted also provides evidence that the boundary conditions used for future experimental studies can be simplified. Instead of utilizing complicated boundary conditions to simulate the constraints of the surrounding element on the studied beam or slab, which was in the bay of the lost column, the surrounding elements can be simulated by an equivalent fixed end that has the combined stiffness of both the top and bottom columns and the right beam element summed up together. This simplified setup is illustrated in Fig. 23.

**Acknowledgments**

This research was supported by a research grant provided by the Defense Science & Technology Agency (DSTA), Singapore, under the Protective Technology Research Center, Nanyang Technological University, Singapore. Any opinions, findings, and conclusions expressed in this paper are those of the writers and do not necessarily reflect the view of DSTA, Singapore.

**Notation**

The following symbols are used in this paper:

- \( A_g \) = gross area of a section;
- \( A_t \) = area of tension reinforcement layer;
- \( d \) = effective depth of a beam;
- \( E_0 \) = initial stiffness;
\( f'_c \) = concrete compressive strength;  
\( f_t \) = concrete tensile strength;  
\( k \) = constant;  
\( w \) = crack width;  
\( e_c \) = concrete compressive strain;  
\( \varepsilon_0 \) = concrete ultimate strain;  
\( \theta_{ap} \) = allowed plastic rotation angle defined as the chord rotation;  
\( \theta_y \) = yield rotation angle defined as the chord rotation;  
\( \lambda \) = constant;  
\( \sigma_e \) = concrete compressive stress at any compressive strain \( e_c \); and  
\( \sigma_t \) = concrete tensile stress.

References

ASCE. (2010). Minimum design loads for buildings and other structures (7-10), ASCE, Reston, VA.  