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<td>Author(s)</td>
<td>Yu, Jun; Tan, Kang Hai</td>
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Progressive Collapse Resistance of RC Beam-Column Sub-assemblages

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Abstract

To investigate the structural resistance of reinforced concrete frames to mitigate progressive collapse, testing was conducted on two simplified RC beam-column sub-assemblies which were designed in accordance with ACI 318-05, with seismic or non-seismic detailing, under middle-column-removal scenarios. The sub-assemblies consisted of a two-span beam, a middle beam-column joint and two column stubs at both ends of the beam. Test results show that under increasing load at the middle beam-column joint, flexural action and compressive arch action are mobilised sequentially, followed by catenary action when the middle joint has undergone large deformations. Plus, the resistance of compressive arch action (with large axial compression in the beam) and catenary action (with axial tension along the beam) is greater than the flexural capacity based on conventional sectional plastic hinge analysis. Detailed information from the embedded strain gauges on longitudinal steel reinforcement along the whole beam was used to shed light on the development of structural mechanisms. Moreover, interaction diagrams of axial forces and bending moments at critical sections, such as joint interfaces and beam ends, were analysed to further illustrate different mechanisms. Finally, the analytical methods to calculate the respective capacities of flexural action and compressive arch action are presented in this paper.

Keywords: progressive collapse; flexural action; compressive arch action; catenary action; reinforced concrete

1. Introduction

With the threat of terrorist attack looming large, the ability of a building to mitigate progressive collapse is of key interest to government agencies. The indirect method and the alternate load path (ALP) method are quite popular in current building codes [1~3]. The indirect method is a descriptive approach of providing minimum level of connectivity and integrity among various structural components. However, the effect of the indirect method according to modern building codes [1~3] is rarely verified by test results. The ALP method is the first proposed quantifiable model for designing robust buildings [4]. Therefore, it can be used to check the effect of the indirect method.

In these codes, the ALP method is a quasi-static approach and is conducted for a building by introducing a column or a bearing wall removal scenario. However, the assessment of the ultimate capacity of a building is limited to the ultimate state of flexural mechanism, i.e. the occurrence of plastic hinges at critical sections. Moreover, the plastic moments are computed without considering the presence of axial forces along the beams with the supporting column removed. In fact, the existence of beam axial forces can enhance the resistance of a local damaged building [5~6]. Therefore, this approach recommended by the codes can be too conservative.
To investigate the resistance of reinforced concrete frames to mitigate progressive collapse, quasi-static tests were conducted on two RC beam-column sub-assemblage specimens which consisted of a two-span beam, a middle beam-column joint and column stubs at both ends of the beam. Test results indicated that three mechanisms of the test specimens can be mobilised sequentially to mitigate progressive collapse, i.e. flexural action, compressive arch action and catenary action. Particularly, the last two mechanisms are still not incorporated into current design and not well-known for designers. This paper illustrates the development of different mechanisms through the detailed information of strain gauge readings and sectional analyses. Finally, the analytical methods to compute the respective capacities of flexural action and compressive arch action are presented in this paper.

2. Test Description

The two specimens were designed in accordance with ACI 318-05 [7] with seismic and non-seismic detailing to study the effect of the detailing of specimens on the development of different structural mechanisms. The dimensions of the specimens and boundary conditions are shown in Fig. 1. Each end was restrained by two rods and a pin on rollers. The detailing of the specimens is shown in Fig. 2, and more information about specimen design is presented in reference 8, but the basic information of the specimens is listed in Table 1.

![Fig.1 Test specimens and boundary conditions](image)

![Fig.2 The detailing of simplified beam-column sub-assemblages (unit: mm)](image)
### Table 1 Specimen properties

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Detailing</th>
<th>Beam size (mm)</th>
<th>Reinforcement ratio at the middle joint*</th>
<th>$f_{cu}$  (MPa)</th>
<th>$f_y$  (MPa)</th>
<th>$f_u$  (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Seismic</td>
<td></td>
<td>Top</td>
<td>0.90% (1T13+2T10)</td>
<td>0.49% (2T10)</td>
<td>31.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bottom</td>
<td>0.73% (3T10)</td>
<td>0.49% (2T10)</td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>Non-Seismic</td>
<td></td>
<td></td>
<td>511 (for T10); 527 (for T13); 731 (for T10); 640 (for T13)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Concrete cover thickness is 20 mm; “T” means high tensile strength reinforcement.

Fig. 3 shows the test set-up. Each end column stub was supported by two horizontal restraints and one vertical restraint. Please note that the effect of the vertical support on the horizontal reaction forces was eliminated by placing steel rollers beneath the vertical support. All reaction forces at the right side in Fig. 3 were measured by load cells. The measured forces can be used to evaluate the internal forces of the beam along the deformed configuration. The final collapse mode of the specimen is shown in Fig. 4. The collapse was caused by the fracture of top reinforcing bars at the right beam end which was connected to the end column stub.

3. **Test results**

3.1 **Test results at structure level**

All test results have excluded the effect of beam self-weight. At the structure level, the test results can be characterized as the relation of middle joint displacements and applied forces or beam axial forces, as shown in Figs. 5 and 6, respectively. Please note that the sudden drop of the applied forces in Fig. 5 was caused by reinforcing bar fracture. The applied vertical forces and the beam axial forces corresponding to the full mobilisation of flexural action and compressive arch action are also marked in Figs. 5 and 6. Since the bottom reinforcement (2T10) at the middle joints are less than the top reinforcement (1T13+2T10, or 3T10) and the middle joint region is subjected to sagging bending moments, the bottom reinforcement is expected to yield first. Similar to design purpose, the beam reinforcement near the end supports resists hogging bending moments. The yield of the top reinforcement at two beam ends indicated that the plastic hinges form at all critical sections. Fig. 6
indicates that at the ultimate state of flexural action, the beam axial forces are still less than $0.1f'_cA_g (=117 \text{ kN})$; this value is used to classify members as beams or columns in ACI 318-05. Beams are designed without considering axial forces but designing columns needs to consider them. Due to the effect of beam axial forces, the resistances of two specimens have been enhanced. However, due to geometrical and material nonlinearity, the resistance after the first peak value decreases with increasing middle joint displacements until catenary action is mobilised. Catenary action fully utilises the strength of reinforcement to sustain vertically applied forces and can attain much higher resistance.

Based on the relationship of the applied force and the middle joint displacement of the specimen S1, the classification of three different mechanisms, viz. flexural action, compressive arch action and catenary action, is shown in Fig.5. Flexural action develops until all plastic hinges occur at the critical sections. Catenary action kicks in at the moment of the applied force reversing and increasing again. Similar classification can be applied to specimen S2 as well. However, more accurately, the onset of catenary action should be defined at the moment when the beam axial force changes from compression to tension. It can be seen in Fig.6 that between the middle joint displacement of around 250 mm and 300 mm, the beam axial force of specimen S1 was still compression, but the applied force has already increased slightly, probably due to cantilever mechanism of the less severely damaged beam [8].

![Fig. 5 The relationship of the applied force to the middle joint displacement](image)
![Fig. 6 The relationship of the horizontal reaction force and the middle joint displacement](image)

### 3.2 Test results at fibre level

The readings of strain gauges mounted on the surfaces of reinforcement provide information on the sequential development of different mechanisms at the fibre level. The strain development of different reinforcing bars at specified sections of specimen S1 and S2 are shown in Figs.7 and 8, respectively. Also the reinforcement yield strain $2800\mu$ is marked in Figs.7 and 8.

Positive strain and negative strain are corresponding to tensile and compressive strain, respectively. Figs. 7 and 8 show that the reinforcement initially under tension yielded very quickly and reinforcement initially under compression experienced the increasing and decreasing of compressive strain. At some sections, the compressive reinforcement finally changed to tension, such as bar b2’ at section LF shown in Fig. 7(b) and bar t1 at section LA shown in Fig.8 (a), indicating that the whole beam section was in tension during catenary action. After reinforcement yielding, some strain gauges might debond and stop working. Thus further strain readings on those reinforcing bars were lost. The yielding of tensile reinforcement indicates the occurrence of plastic hinges. Due to reversal of bending moments at the middle joints and less reinforcement at the bottom of the joints, the bottom reinforcement yielded at very small joint displacements. At the peak capacity of compressive arch action, the compressive reinforcement of specimen S1 at the interfaces of the beam-middle joint and the beam-end column yielded as well. However, the counterpart of specimen S2 was still within the
elastic range. Thereafter, the strain of compressive reinforcement of all sections decreased and tended to change into tension, as shown in Fig. 8 (a)
3.3 Test results at section level

During the tests, critical sections, such as joint interfaces (LA) and beam sections near the end supports (LF for specimen S1 and LD for specimen S2), were subjected to coupled bending moments and beam axial forces. The relationships of bending moments and beam axial forces at different critical sections are shown in Figs. 9 and 10. The sign conventions of bending moments and axial forces are also shown in Figs. 9 and 10. The geometrical centre of the sections is selected as the reference point to calculate bending moments, because all measured internal forces are referred to this point. Theoretical M-N interaction diagrams [9] are computed by assuming a series of section strain distributions based on the assumptions that: 1) the plane section remains plane; 2) the ultimate compressive strain of concrete is 0.003; and 3) tensile strain of extreme tension reinforcement layer ranges from the ultimate concrete compressive strain to 10 times the tensile yield strain. In addition, material properties of reinforcing steel are assumed to be perfect elastic-plastic. For sections near the beam end supports, the tensile hardening of steel reinforcement is also taken into account in the sectional analyses. Since at the middle joint interface, two bottom reinforcing bars fractured sequentially during testing on specimen S1, M-N interaction diagram of the section with only one bottom bar (i.e. after fracture of one bottom bar) and no bottom bar (i.e. after fracture of two bottom bars) are analyzed as well, represented respectively in the dash line and the dash dotted line in Fig. 9(a). Particularly for the case with only one top rebar layer, the bending moment is equal to the product of the axial force of a rebar layer and the distance from the centroid of the top rebar layer to the geometrical centre, which is constant for a given section. As a result, the M-N interaction diagram is a straight line. It also applies to the middle joint interface (section LA) of specimen S2. However, there is no sequential fracture of the second reinforcing bar at section LA. Therefore, the M-N interaction diagram of section LA with one bottom reinforcing bar is not shown in Fig. 9(b).

![Diagram](image-url)

(a) Section LA of specimen S1

(b) Section LA of specimen S2

Fig. 9 Interaction diagram of axial force and bending moment at joint interface

The PM curves in Figs. 9 (a) and (b) follow the path O-A-B-C-D-E-F-G, where O is the origin of the coordinates. At point A, the section has reached the theoretical M-N interaction curve, suggesting that plastic hinges have formed, since point A is located at the tension-controlled region (i.e. crushing of concrete after yielding of the extreme tensile layer of reinforcement). Due to the presence of axial compression forces, the ultimate bending moment increased from A to B. This is the reason why compressive arch action can enhance structural resistances of two specimens. The bending moment and the axial force at a section are produced by three components: 1) the compressive force in concrete; 2) the axial force in the top reinforcement layer; and 3) the axial force in the bottom reinforcement layer. From B to C, the beam compressive force decreased slightly and the bending moment decreased more considerably attributed to respective variations of different components. The compressive force in concrete decreased due to crushing of concrete cover. The compressive force in the top reinforcement layer increased due to increasing of reinforcement compressive strain and the...
tension force in the bottom reinforcement layer remained unchanged due to increased reinforcement tensile strain still at yielding plateau. Moreover, the reduced compressive force in concrete was less than the increased compressive force in top reinforcement, resulting in the axial compression increased slightly from B to C. However, the bending moment decreased more considerably because of the larger lever arms of the concrete compressive force and the smaller lever arms of the reinforcement compressive force. From C to D, the combination of these three components gave a slightly reduced beam axial compression and an increased bending moment. The tension force in the bottom reinforcement increased due to tension hardening. The compressive force in concrete increased due to confinement effect and the compressive force in the top layer reinforcement decreased slightly due to a slight reduction of reinforcement compressive strain when the middle joint displacement was between 100 and 200 mm as shown in Figs.7(a) and 8(a). Please note that due to tension hardening of reinforcement and confinement effect on concrete, the coupled bending moment and beam axial force can even exceed the computed M-N interaction diagram of specimen S1. Since the confinement effect of specimen S1 (seismic-detailing) was greater than that of specimen S2 (non-seismic detailing), the middle joint interface of S1 could sustain a much greater bending moment (around 40 kNm at point D) than that of S2 (around 23 kNm at point D). From the path D to E for S1, one bottom reinforcing bar fractured and the PM curve regressed towards the theoretical M-N interaction diagram of the section with one bottom reinforcing bar. However, before reaching that theoretical curve, another bottom bar fractured, as shown from E to F in Fig.9 (a). Eventually, the section contained only the top layer reinforcement. For specimen S2, only one bottom reinforcing bar fractured as shown from point E to F in Fig.9 (b), but another bottom bar failed without fracture. Within the path FG, the bending moment was merely caused by the tension centroid not coinciding with the reference point.

At sections near the beam end supports, once the section has reached the theoretical M-N interaction curves, the test results agree well with theoretical results, as shown in Figs.10(a) and 10(b). Due to confinement effect on concrete and tension hardening of reinforcement, the coupled bending moments and beam axial forces at beam ends can even exceed the theoretical M-N interaction curve. Compared with the section 250 mm from the beam end (LE for specimen S1 and the section at near LC for specimen S2), the tensile reinforcement at the beam ends (LF or LD) experienced larger tensile stress, even to the tension-hardening stage, since the PM curves are closer to the theoretical results with consideration of tension-hardening.

4. Capacity of flexural action

Although the detailing of reinforcement at the beam-column joint regions is designed for sustaining negative bending moments, due to the requirements of integrity specified by ACI 318-05, the sections within these regions still have certain capacities to resist positive bending moments. As a result, after a middle column is removed and the bending moment above the removed column changes
its direction, flexural action can sustain certain external loads. The capacity of flexural action is determined by the yielding moments at critical sections without considering the existence of beam axial forces. In this case, the plastic hinges occurred at the middle beam-column joint interface and the beam-end column stub interface. The positive bending moment of the middle beam-column joint interface is denoted as $M_{nn}$, and the negative bending moment of the beam-end column stub interface is denoted as $M_{ne}$. Then the capacity of flexural action can be computed as

$$ P_{fn} = 2(M_{nn} + M_{ne}) / L $$

(1)

where $L$ is shown in Fig.1; $M_{nn}$ and $M_{ne}$ are nominal bending moments of specified sections without considering the strength reduction factors specified by building codes.

Finally, the analytical results of flexural capacities of two specimens are listed in Table 2. The applied forces at the moment when the top reinforcement at beam ends yielded are listed in Table 2 as well. It can be seen that the analytical results are less than the test results since the yielding of the bottom reinforcement at the middle joint interfaces and the top reinforcement at the beam ends was not simultaneous. When the latter occurred, the bottom reinforcement at the joint interfaces might have reached hardening stage and thus $M_{nn}$ is greater than the calculated value which is based on yield strength of reinforcement.

Table 2: The calculation of flexural capacity

<table>
<thead>
<tr>
<th>Specimen*</th>
<th>$M_{nn}$ (kN*m)</th>
<th>$M_{ne}$ (kN*m)</th>
<th>$P_{fn}$ (kN)</th>
<th>The applied force at beam end yielding (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>16.40</td>
<td>29.08</td>
<td>33.08</td>
<td>37.01</td>
</tr>
<tr>
<td>S2</td>
<td>16.30</td>
<td>23.60</td>
<td>29.02</td>
<td>34.02</td>
</tr>
</tbody>
</table>

*: S1 and S2 were designed with seismic and non-seismic detailing, respectively.

5. Capacity of compressive arch action

Park’s model [10] proposed for one-way slabs longitudinally restrained at the slab boundaries can be modified for axially restrained beams [6]. The capacity of compressive arch action of RC beam-column sub-assemblages subjected to a concentrated load at the middle joint is determined as follows:

$$ P = \frac{2}{L_n} (0.85 f_u \beta b h \left[ \frac{h}{2} \left( 1 - \frac{\beta_1}{2} \right) + \frac{\delta}{4} (\beta_1 - 3) + \frac{L_n^2}{2 \delta} (\beta_1 - 1) \varepsilon_t + \frac{L_n^2}{8h} \left( 2 - \frac{\beta_1}{2} \right) \varepsilon_t^2 \right] - \left( \frac{T - T' - C_s + C_i}{3.4 f_u b} \right) + \left( C_s + C_i \right) \left( \frac{h}{2} - d - \frac{\delta}{2} \right) + (T + T') \left[ d - \frac{h}{2} + \frac{\delta}{2} \right]) $$

(2)

where $L_n$ is the net span length of beams; $b$ and $h$ are the beam width and depth, respectively; $f_{u'}$ is the concrete compressive strength determined from concrete cylinder tests; $\beta_1$ is the ratio of the depth of concrete equivalent stress block to the depth of section neutral axis (according to ACI 318-05); $T$ and $T'$ are tensile resultant forces of reinforcing steel of sections at the interface of middle joints and beam-end column stubs, respectively; $C_s$ and $C_{i'}$ are compressive resultant forces of reinforcing steel of sections at the interface of middle joints and beam-end column stubs, respectively; $d'$ is the distance from the centroid of compressive reinforcement to the extreme compressive concrete fibre; $d$ is the beam effective depth; $\delta$ is middle joint displacement; and $\varepsilon_t$ is total strain due to beam axial deformation and movement of beam end supports. It can be computed as

$$ \varepsilon_t = \left( \frac{1}{bhE_t} + \frac{1}{L_n S} \right) \left[ 0.85 f_u \beta b \left( \frac{h}{2} - \frac{\delta}{4} \frac{T - T' - C_s + C_{i'}}{1.7 f_{u'} \beta b} \right) + C_s - T \right] + \left( \frac{1}{bhE_t} + \frac{1}{L_n S} \right) \left[ 0.425 f_u \beta b L_n^2 \left( \frac{1}{bhE_t} + \frac{1}{L_n S} \right) \right] $$

(3)
where $S$ is the stiffness of horizontal restraints, and $E_c$ is elastic modulus of concrete.

Based on reinforcement strains obtained in the tests, as shown in Figs. 6 and 7, $T$ and $T'$ can be determined according to yield strength of steel reinforcement. However, there is no convincing proof to determine $C_s$ and $C_s'$ based on yield strength. For convenience, $C_s$ and $C_s'$ are tentatively calculated based on yield strength.

In the tests, the equivalent stiffness of the axial restraints is $1 \times 10^5$ kN/m. Due to the gap of horizontal connections, before horizontal reaction forces were mobilised, the middle joints have displaced vertically. Therefore, the analytical results of the middle joint displacement corresponding to compressive arch action are modified by adding the joint displacement due to gap to the calculated values. The comparison of test results and analytical results is shown in Table 3. It can be seen that the analytical model always overestimates the capacity of compressive arch action of two specimens at relatively smaller joint displacements, probably because the analytical model does not consider concrete softening stage and assumes overestimated values of $C_s$ and $C_s'$. Once the compressive reinforcement has yielded with a strain around 0.28%, the strain of concrete above the compressive reinforcement has exceeded the strain around 0.2% corresponding to the peak strength.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test results</th>
<th>Analytical results</th>
<th>$P_{cu,analytical}/P_{cu,tested}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P_{cu}$ (kN)</td>
<td>Corresponding displacement (mm)</td>
<td>$P_{cu}$ (kN)</td>
</tr>
<tr>
<td>S1</td>
<td>41.64</td>
<td>77.5</td>
<td>49.13</td>
</tr>
<tr>
<td>S2</td>
<td>38.38</td>
<td>72.5</td>
<td>45.92</td>
</tr>
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</table>

6. Summaries and discussions

The test results are demonstrated at structure, fibre and section levels to illustrate the mobilisation of different mechanisms of two RC beam-column sub-assemble specimens to mitigate progressive collapse under a middle column removal scenario. Both compressive arch action and catenary action can provide a much higher resistance than flexural action. Test results indicate that specimens designed according to ACI 318-05 have sufficient integrity of reinforcement so that catenary action can be mobilised successfully. However, there is no obvious difference in structural performance due to different detailing rules. It can be seen that after plastic hinges have formed, the M-N resistance interaction curve can be predicted well through theoretical M-N interaction diagrams of different sections. The conventional plastic hinge mechanism is used to compute the capacity of flexural action and Park’s model can be modified into calculating the capacity of compressive arch action with acceptable accuracy.

Similar to research work done on steel and composite structures [11], the M-N interaction curves at critical sections can be extended to calculate the capacity of catenary action of RC beam-column sub-asessemblages in our future work.

Acknowledgement

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Speaker’s biography

Yu Jun is a Ph.D. Student in School of Civil & Environmental Engineering, Nanyang Technological University, Singapore. He received his BS (Eng) and MS (Eng) from Zhejiang University, China. His research interests are testing and numerical modeling on progressive collapse analysis of reinforced concrete structures subjected to extreme loadings.