Behaviour of precast concrete beam-column sub-assemblages subject to column removal

Kang, Shao-Bo; Tan, Kang Hai

2015

Kang, S.-B. & Tan, K. H. (2015). Behaviour of precast concrete beam-column sub-assemblages subject to column removal. *Engineering structures*, 93, 85-96.

<https://hdl.handle.net/10356/107316>

<https://doi.org/10.1016/j.engstruct.2015.03.027>

© 2015 Elsevier Ltd. This is the author created version of a work that has been peer reviewed and accepted for publication by *Engineering Structures*, Elsevier Ltd. It incorporates referee's comments but changes resulting from the publishing process, such as copyediting, structural formatting, may not be reflected in this document. The published version is available at: [<http://dx.doi.org/10.1016/j.engstruct.2015.03.027>].

Downloaded on 26 Apr 2023 20:41:27 SGT

Behaviour of Precast Concrete Beam-Column Sub-assemblages Subject to Column Removal

Shao-Bo Kang, Kang Hai Tan

School of Civil & Environmental Engineering, Nanyang Technological University, 50 Nanyang
Avenue, Singapore, 639798

Abstract

Under column removal scenarios, initiation of alternate load paths via adjacent bridging beams to redistribute vertical loads requires certain level of ductility and continuity in beam-column joints. Although this approach does not consider the magnitude of the blast event, it is threat-independent and offers a minimum level of robustness against column removal scenarios. This paper studies the behaviour of precast concrete sub-assemblages which comprised two precast beams and a precast column joining together by cast-in-place concrete topping above the two beams and the beam-column joint. The top longitudinal reinforcement in the structural topping of precast beams passed through the beam-column joint continuously. However, the bottom beam longitudinal reinforcement was either lap-spliced or anchored as a 90° bend within the cast-in-place joint. Due to discontinuity of bottom beam longitudinal reinforcement, the ability of such an assemblage to develop compressive arch action (CAA) and subsequent catenary action has to be investigated, in particular, the effect of the top and bottom beam longitudinal reinforcement ratios. Test results show that significant CAA and catenary action developed in the beams under column removal scenarios, with pull-out failure of the bottom beam reinforcement in the joint. The enhancement of CAA and catenary action to structural resistance greatly depends on joint detailing and beam reinforcement ratio. Furthermore, the effectiveness of horizontal shear transfer between concretes cast at different times is examined at large deformation stage. Finally, practical suggestions are given to enhance structural resistance of a similar type of precast concrete sub-assemblages.

Keywords: precast concrete sub-assemblages; beam-column joints; column removal scenario; compressive arch action; catenary action; deformation capacity.

1. Introduction

The devastating collapse of the Ronan Point Apartment triggered intensive research study on the behaviour of building structures when subjected to extreme loading conditions. Design approaches were proposed to reduce the risk of initial damage and to arrest disproportionate propagations of a local failure [1]. Thereafter, structural integrity in the form of tie requirements was incorporated into the ACI code as early as 1989 [2]. Recently, progressive collapse of the Alfred Murrah Federal Building and the Twin Towers of World Trade Centre due to terrorist attacks gave further impetus to the issuance of design codes and guidelines for building structures against abnormal loading conditions [3-5].

Among the proposed design approaches to mitigate progressive collapse, the alternate path method allows a local damage to occur in affected structures while seeking to avert progressive collapse through additional force-transfer mechanisms in the form of bridging beam action [4]. Although the extent and locations of the local damage are prescribed, the acceptable criteria are primarily based on seismic design standard [6], and this approach is comparatively conservative when applied to progressive collapse design situations [7]. Hence, further research is needed to evaluate realistic failure criteria to quantify the rotation capacity of beam-column joints for progressive collapse design situations.

Several experimental tests have been conducted on reinforced concrete (RC) beam-column sub-assemblages. Besides, a series of parameters governing the structural behaviour of beam-column joints have been investigated. Su et al. tested twelve reduced-scale RC beam-column subassemblies with varying reinforcement ratios, beam span-depth ratios and loading rates, and predicted their CAA and catenary action capacities through analytical models [8]. Yu and Tan investigated the effect of seismic detailing on the CAA and catenary action capacities of beam-column sub-assemblages [9, 10]. The test specimens were simulated through the component-based joint model [11]. In order to enhance the continuity of RC structures, Orton et al. tested RC beams strengthened by carbon fibre-reinforced polymer (CFRP) [2]. In comparison with conventional RC beams, these specimens exhibited higher catenary action and improved rotational ductility when continuity of negative moment reinforcement was provided by

CFRP. At the concrete frame level, Lew et al. studied two full-scale beam-column assemblies representing moment frames with seismic design and detailing [12]. In addition to the behaviour of beams and the middle joint subject to column loss, the crack patterns on the end columns revealed different horizontal force transfer under initial CAA and subsequent catenary action. At the CAA stage, more cracks were observed on the column below the side joint, indicating that compression force was transmitted to the bottom support. However, at the catenary action stage, horizontal tension force was mainly sustained by the lateral restraint on the column top, and thus the column segment above the side joint became more critical. Unfortunately, the horizontal force was not measured in the test and was only simulated by means of the proposed models [13]. In the same year, Yi et al. tested a four-bay and three-storey planar frame and proposed a simplified model to quantify catenary action [14]. RC sub-structures were also tested under quasi-static and dynamic column removal scenarios [15, 16]. Furthermore, a simplified framework was proposed to evaluate the robustness of building structures subjected to sudden column loss [17].

To date, most of the research studies were focused on RC monolithic beam-column joints with continuous, spliced or sufficiently anchored reinforcing bars, and the failure of middle joints is typically characterised by the fracture of bottom longitudinal rebars [9, 14, 18, 19]. Although cast-in-place beams with discontinuous bottom bars have been tested under progressive collapse scenarios [2], more experimental tests are necessary to evaluate the rotational capacity of concrete joints with discontinuous bottom beam reinforcement. In precast concrete structures, precast beam units with cast-in-place structural topping and beam-column joints have been used as ductile moment-resisting frames, as reported by FIB bulletins [20, 21] and other documents [22-24]. Its design is compatible with design codes for monolithic RC structures, but minor modifications are made in the reinforcement detailing to achieve high productivity [24]. With proper reinforcement detailing in beam-column joints, frames can exhibit equivalent behaviour to monolithic RC structures under flexure condition [23]. The pertinent question is whether this type of precast beam-column sub-assemblages can exhibit catenary action under column removal scenarios. Additionally, the horizontal bond behaviour between structural topping and precast beam units needs to be investigated under large

deformations. Thus, an experimental study on the joint behaviour of precast concrete beam-column sub-assemblages under column removal scenarios is necessary.

In this paper, the behaviour of precast concrete beam-column sub-assemblages is presented. In addition, the effects of reinforcement detailing in the joint and longitudinal reinforcement ratio in the beam on CAA and catenary action are investigated, and recommendations are made for the design of precast concrete structures against progressive collapse. These findings are relevant to any form of precast concrete construction that does not require special embedded metal inserts or mechanical couplers for the bottom reinforcement in the joint region.

2. Experimental Programme

2.1 Specimen design

(a) Plan view

(b) Elevation view

Fig. 1 The prototype precast concrete structure

A six-storey precast concrete frame building was designed under gravity loads in accordance with Eurocode 2 [25]. Fig. 1 shows the plan and elevation views of the structure. The height of a typical storey was 3.6 m, except the 4.5 m high first floor. The centre-to-centre spacing of columns in two orthogonal directions was 6 m. The cross sections of a prototype beam and a column were 300 mm by 600 mm and 500 mm by 500 mm, respectively. Under column removal scenarios, one middle column at the ground floor of the peripheral frame was assumed to be forcibly removed, as shown in Fig. 1. A beam-column sub-assembly, incorporating the two-bay beam and the middle column over the column removal, was extracted from the precast concrete structure. To fit the extracted sub-assembly within the physical constraints of the laboratory in Nanyang Technological University, the beams and columns in the prototype building were scaled down to one-half, but the beam reinforcement ratio remained unchanged. Thus, the spacing of precast columns in the two orthogonal directions was 3 m, and the dimensions of beams and columns were scaled down to 150 mm x 300 mm and 250 mm x 250 mm, respectively. Two enlarged column stubs were erected on both sides of the sub-assembly (Fig. 2) to simulate horizontal restraints from adjacent columns on the plan (Fig. 1).

(a) MJ-B-0.52/0.35S

(b) MJ-L-0.52/0.35S

Fig. 2 Reinforcement detailing of precast concrete beam-column sub-assemblages

Welded connections in precast concrete structures exhibit limited capacity to develop alternate load paths due to the reduced ductility of steel reinforcement in the heat-affected zone [26]. As reported by FIB [21], cast-in-place concrete provides better robustness for precast structural elements. Therefore, to enhance the structural performance of precast concrete structures under column removal scenarios, precast beam units with cast-in-place concrete structural topping were used in the beam-column sub-assemblages in the experimental programme. Two stages of casting were adopted. In the first stage, the 225 mm deep precast beam units (depicted by the hatched zones in Fig. 2) were fabricated as “precast elements”. Thereafter, the two units were assembled and continuous top longitudinal reinforcement was tied to the protruded stirrups (Fig. 2, sections A-A and B-B). The second stage involved the casting of 75 mm deep concrete

topping, the middle beam-column joint and the end column stubs to form an integral sub-assembly (depicted by the unhatched zone in Fig. 2).

Table 1 lists the three investigated parameters that dominate the beam-column sub-assembly behaviour, namely, bottom bar detailing in the middle joint, top and bottom longitudinal reinforcement ratios in the beam and surface preparation of horizontal interface for precast beam units. For the discontinuous bottom reinforcing bars, two widely-used joint detailing in local practice were studied. The first joint detailing features protruded bottom longitudinal bars terminating with a 90° bend in the joint region (Fig. 2(a)). This type of joint detailing performs well even under earthquake loading conditions and has been recommended for construction in New Zealand [23]. However, this detailing may lead to congestion of reinforcement in the joint region, and thus the column size should be sufficiently wide to accommodate the required embedment length [20]. The second joint detailing is characterised by U-shaped beam trough sections at the beam ends connecting to the joint (Fig. 2(b)). Unlike the precast concrete beam shells tested by Park and Bull [27], the U-shaped trough section was only located at the precast beam ends and its length depends on the required embedment length of bottom reinforcement passing through the middle joint. In-place concrete was cast in the trough, the joint region and the structural topping consisting of continuous top reinforcement. Composite action between precast beam units and concrete topping relies on the roughness of horizontal interface, the amount of protruded stirrups and the concrete strength [28]. In the study, across the horizontal interface between the precast beam units and the cast-in-place concrete topping, two types of surface preparation were employed to examine the effectiveness of horizontal shear transfer at large deformation stage. “Smooth surface” refers to one without any treatment after vibration, whereas “rough surface” represents an interface that is intentionally roughened to approximately 3 mm roughness complying with Eurocode 2 [25]. In all specimens, mild steel stirrups of 8 mm diameter at 80 mm spacing were placed at the beam end sections, whereas mild steel stirrups of 6 mm diameter at 100 mm spacing were used at the middle sections.

Table 1 Geometric property of beam-column sub-assemblages

Specimen	Clear span (mm)	Length of curtailed top bar (mm)	Bottom bars at middle joints and length [#] (mm)	Longitudinal reinforcement				Surface preparation
				A-A section		B-B section		
				Top	Bottom	Top	Bottom	

MJ-B-0.52/0.35S	2750	900	90° bend (190+70)	3H10	2H10	2H10	2H10	Smooth
MJ-L-0.52/0.35S		900	Lap-spliced (360)	3H10	2H10	2H10	2H10	Smooth
MJ-B-0.88/0.59R		1000	90° bend (190+90)	3H13	2H13	2H13	2H13	Rough
MJ-L-0.88/0.59R		1000	Lap-spliced (470)	3H13	2H13	2H13	2H13	Rough
MJ-L-1.19/0.59R		1000	Lap-spliced (470)	2H16+H13	2H13	2H16+H13	2H13	Rough

#: The anchorage length is calculated from the far face of the middle column, and the numerals (190+90) respectively denote the horizontal portion of the 90° bend bar is 190 mm and the vertical portion is 90 mm.

Table 1 list the geometric properties and reinforcement details of beam-column sub-assemblages. In the notations of specimens, “MJ” denotes beam-column sub-assemblages incorporating a middle joint and a two-bay beam, and “B” and “L” stand for 90° bend and lap-splice of bottom bars in the middle joint, respectively. The first and second numerals denote the respective percentage of top and bottom longitudinal reinforcement in the middle joint. “S” and “R” indicate smooth and rough surfaces, respectively. For instance, MJ-B-0.52/0.35S represents a specimen with bottom reinforcement of 90° bend in the middle joint, a top reinforcement ratio of 0.52%, a bottom reinforcement ratio of 0.35%, and a smooth concrete surface for the two precast beam units.

2.2 Test procedure

The boundary conditions of beam-column sub-assemblages in the frame structure were simplified as two horizontal restraints and one vertical restraint on the column stub at each end, as shown in Fig. 3. Load cells were used in the horizontal direction to record reaction forces, as shown in Fig. 4(a). At the bottom of each column stub, a pin support was seated on steel rollers (Fig. 4(b)), and load cells were placed under the rollers to measure the vertical reaction force. The fairly rigid horizontal restraints from the A-frame and the reaction wall provide the upper bound values of structural resistances of the beam-column sub-assemblage.

Fig. 3 Test set-up for beam-column sub-assemblages

(a) Lateral restraints

(b) Bottom pin support on steel rollers

(c) Out-of-plane restraint

(d) Rotational restraint

Fig. 4 Restraints on the beam-column sub-assemblage

In addition to the boundary conditions at the column stubs, two pairs of universal steel columns were erected on each side of precast beams to prevent out-of-plane bending of the beams as load was applied onto the middle joint, as shown in Fig. 4(c). This was to simulate the full restraint from the slab, so that the sub-assembly could only deflect vertically. In the vicinity of the middle joint, two sets of short columns were employed in front of and behind the sub-assembly. Steel rods were placed in the PVC pipes embedded in the middle column, as shown in Fig. 4(d). Hence, rotation of the middle joint could be prevented if reinforcing bars only fractured at one vertical face of the joint. Displacement-control point load at a rate of 6 mm/min was applied vertically on the middle column through a hydraulic actuator.

2.3 Instrumentation

Besides the reaction forces measured by the horizontal and the vertical load cells, the deformed geometry of the beam-column sub-assemblages were monitored through linear variable differential transducers (LVDTs) placed along the beam length at regular intervals. Additionally, plastic hinges at the beam ends played a crucial role in the deformation capacity of sub-assemblages, and they were measured by a group of LVDTs mounted onto the beam. Fig. 5 depicts the LVDT arrangement to measure the rotations of plastic hinges at both beam ends and vertical deflections of the beam. The plastic hinge length for a typical beam was taken as $0.5h$, where h is the full depth of cross section [29]. Thus, the first row of LVDTs measured the beam rotations over a length of 150 mm. Since the development of catenary action could extend the plastic hinge length, another row of LVDTs was installed at 120 mm away from the first row of LVDTs at both beam ends (Fig. 5).

Fig. 5 Schematic of hinge rotation and beam deformation measurement

Fig. 6 Layout of strain gauges on longitudinal reinforcement

Strain gauges were mounted onto the longitudinal reinforcement at the interface with the middle joint and the end column stub to measure steel strains at different loading stages. Fig. 6 shows the layout of steel strain gauges on the top and bottom longitudinal bars in the beam unit. It should be noted that for sub-assemblages with lap-spliced bottom bars, strain gauges were mounted on the bottom bars passing through the joint.

3. Experimental results and discussions

Hot-rolled deformed steel bars with diameter of 10, 13 and 16 mm were used for longitudinal reinforcement in the beam, and round bars with 6 and 8 mm diameter were used for stirrups. Concrete with the maximum coarse aggregate size of 10 mm was used for the precast beam units, the cast-in-place topping and the joint itself. Prior to testing, material properties of reinforcement and concrete were obtained and listed in Table 2.

Table 2 Material properties of reinforcing bars and concrete

Material		Diameter (mm)	Yield Strength (MPa)	Modulus of elasticity (MPa)	Ultimate strength (MPa)	Fracture strain (%)
Longitudinal bars	H10	10	462	187,302	553	11.9
	H13	13	471	186,526	568	12.2
	H16	16	527	196,341	618	11.9
Stirrup	R6	6	264	217,859	351	7.9
	R8	8	353	209,641	460	14.9
Concrete		Compressive strength (MPa)			Secant modulus (MPa)	
Precast beam unit		27.9			24,717	
Concrete topping and beam-column joint		MJ-B-0.52/0.35S		35.8	27,796	
		MJ-L-0.52/0.35S				
		MJ-B-0.88/0.59R		20.3	20,501	
		MJ-L-0.88/0.59R				
		MJ-L-1.19/0.59R				

3.1 Force-displacement curves of beam-column sub-assemblages

(a) Sub-assemblages with 90° bend bars

(b) Sub-assemblages with lap-spliced bottom bars

Fig. 7 Applied load-middle joint displacement curves of beam-column sub-assemblages

Figs. 7 and 8 plot the vertical load and the horizontal reaction force of beam-column sub-assemblages versus the middle joint displacement. Two mechanisms, namely, CAA and catenary action, were sequentially developed in the beam-column sub-assemblages. CAA is termed as the force-transfer mechanism in which significant axial compression force is developed in the bridging beam, whereas catenary action represents the stage when axial tension force is initiated in the beam [9]. At the CAA stage, horizontal compression force increased with vertical load, but vertical load attained its peak value

earlier than the maximum horizontal compression (see Figs. 7 and 8). Due to crushing of concrete in the compression zone at the top beam surface next to the middle joint and the bottom beam surface near the end column stub, both vertical load and horizontal compression force started to decrease with increasing middle joint displacement. In the descending branch, the sudden drop of vertical load marked by crosses resulted from the fracture of bottom reinforcing bars at the middle joint interfaces, as shown in Figs. 7(a and b). However, rebar fracture imposed a minor effect on the horizontal compression force. When the displacement was larger than one beam depth of 300 mm, catenary action kicked in and the applied load was sustained by the tensile strength of the beam until final failure occurred. Significant catenary action developed in beam-column sub-assemblages except MJ-B-0.52/0.35S, as shown in Fig. 7(a). For the other specimens, the catenary action capacity of sub-assemblages surpassed the value of CAA, and therefore catenary action is effective to mitigate progressive collapse under column removal scenarios.

(a) Sub-assemblages with 90° bend bars

Fig. 8 Horizontal reaction-middle joint displacement curves of beam-column sub-assemblages

The resistance of beam-column sub-assemblages under column removal scenarios is characterised by the CAA and the catenary action capacities. Table 3 lists the load capacities and the maximum horizontal compression and tension forces. In addition, it was assumed that all the bottom bars in the middle joint were able to develop their yield strength under flexure, and thus the flexural capacity of sub-assemblages was calculated based on the plastic hinge mechanism. Accordingly, the enhancement factors of CAA and catenary action based on flexural capacity are calculated as shown in Table 3.

Table 3 Test results of beam-column sub-assemblages

Specimen	Flexural capacity (kN)	CAA capacity (kN)	Peak horizontal compression (kN)	Enhancement factor of CAA to flexural capacity	Catenary action capacity (kN)	Peak tension (kN)	Enhancement factor of catenary action to flexural capacity
MJ-B-0.52/0.35S	33.89	50.52	-231.26	1.49	26.05	16.22	0.77
MJ-L-0.52/0.35S	31.23	41.36	-186.10	1.32	49.50	127.74	1.59
MJ-B-0.88/0.59R	55.00	63.28	-282.40	1.15	98.55	229.85	1.79
MJ-L-0.88/0.59R	51.29	53.85	-242.00	1.05	77.24	182.05	1.51
MJ-L-1.19/0.59R	61.71	57.37	-290.30	0.93	86.60	227.10	1.40

3.2 Effect of reinforcement detailing on structural behaviour

Although the same reinforcement ratio was used in MJ-B-0.52/0.35S and MJ-L-0.52/0.35S, significantly different CAA and catenary action capacities were obtained for

those two specimens, as shown in Figs. 7(a and b). The CAA capacity of MJ-B-0.52/0.35S is 50.5 kN, 22.1% higher than that of MJ-L-0.52/0.35S. Besides, MJ-B-0.52/0.35S was able to develop 24.3% larger horizontal compression force than MJ-L-0.52/0.35S at the CAA stage. Similar results were obtained for sub-assemblages MJ-B-0.88/0.59R and MJ-L-0.88/0.59R at the CAA stage, as shown in Figs. 7 and 8. MJ-B-0.88/0.59R was able to develop 17.5% larger CAA capacity and 16.7% higher horizontal compression force than MJ-L-0.88/0.59R.

These differences were mainly due to reinforcement detailing of beam bottom longitudinal reinforcement in the joint. In comparison with lap-spliced reinforcement, 90° bend of bottom reinforcement in the joint provided a larger distance between the compression and tension reinforcement at the middle joint and the end column stub faces. Thus, sub-assemblage MJ-B-0.52/0.35S with 90° bend of bottom bars developed larger moment capacities at the beam end sections than MJ-L-0.52/0.35S with lap-spliced bottom reinforcement. Correspondingly, higher CAA capacity and horizontal compression force were attained in MJ-B-0.52/0.35S, as shown in Fig. 7(a) and Fig. 8(a).

At the catenary action stage, MJ-L-0.52/0.35S developed 90.0% higher catenary action capacity than MJ-B-0.52/0.35S, indicating the beneficial effect of lap-spliced bottom reinforcement on development of catenary action. Nonetheless, in MJ-L-0.88/0.59R, pull-out failure of bottom beam reinforcement near the end column stub hindered the development of tension force in the beam, as shown in Fig. 8(b). Correspondingly, its catenary action capacity was 21.6% lower than that of MJ-B-0.88/0.59R. Therefore, due to different failure modes of sub-assemblages, no apparent effect of reinforcement detailing was observed on the catenary action capacity.

3.3 Effect of reinforcement ratio on structural behaviour

By increasing the top and bottom reinforcement ratios in MJ-B-0.88/0.59R and MJ-L-0.88/0.59R, the sagging and hogging moments of beam end sections were substantially increased. Therefore, both the CAA capacity and the horizontal compression force were increased for sub-assemblages MJ-B-0.88/0.59R and MJ-L-0.88/0.59R, as shown in Figs. 7 and 8. It should be noted that the compressive strength of cast-in-place concrete topping in MJ-B-0.88/0.59R and MJ-L-0.88/0.59R was lower than that in MJ-B-

0.52/0.35S and MJ-L-0.52/0.35S. If concrete topping with the same compressive strength as that in MJ-B-0.52/0.35S and MJ-L-0.52/0.35S were used in MJ-B-0.88/0.59R and MJ-L-0.88/0.59R, the CAA capacities would have been even greater than those obtained in the tests. Although the CAA capacity of beam-column sub-assemblages was substantially increased with increasing reinforcement ratios, as shown in Figs. 7(a and b), the enhancement factor of CAA relative to flexural action was reduced, as listed in Table 3, since the flexural capacity of sub-assemblages was also significantly increased. Compared with MJ-B-0.52/0.35S and MJ-L-0.52/0.35S, catenary action capacities of MJ-B-0.88/0.59R and MJ-L-0.88/0.59R were also increased with higher top and bottom reinforcement ratios in the beam, as shown in Figs. 7(a and b). The increase in the catenary action capacity mainly came from the increased top reinforcement ratio in the bridging beam, as bottom longitudinal reinforcement in the middle joint had been pulled out prior to initiation of catenary action.

Only the top reinforcement ratio in the beam of MJ-L-1.19/0.59R was increased in comparison with MJ-L-0.88/0.59R. Correspondingly, the CAA and catenary action capacities of MJ-L-1.19/0.59R were increased by 6.5% and 12%, respectively. Besides, the horizontal compression and tension forces in MJ-L-1.19/0.59R were increased by 20% and 25%, respectively, as shown in Fig. 8(b). It also indicates that a higher top reinforcement ratio plays a positive role in the development of CAA and catenary action in beam-column sub-assemblages.

3.4 Rotation angle of beam-column sub-assemblages

Fig. 9(a) shows the overall deformed profile of sub-assemblage MJ-B-0.88/0.59R after fracture of top longitudinal reinforcement near the end column stub. In addition, its vertical deflections were also measured by a series of LVDTs (Fig. 5) along the beam length, as shown in Fig. 9(b). It is apparent that the rotation of the middle joint was substantially larger than that of plastic hinges at the end column stubs, due to flexural deformations of the beam at the catenary action stage. Thus, to evaluate the deformation capacity of sub-assemblages, the rotations of sub-assemblages were calculated as a ratio of the middle joint displacement to the clear span 2750 mm (Fig. 2) of the single-bay beam. Besides, the rotations of plastic hinges near the end column stub were measured

by the LVDTs mounted at the beam ends (see Fig. 5). Table 4 lists the rotation angles of the beam-column sub-assemblages and the plastic hinges at the end column stub.

Fig. 9 Deformed profile of beam-column sub-assemblage MJ-B-0.88/0.59R

Table 4 Rotation of plastic hinges and beam-column sub-assemblages

Specimen	At the CAA capacity		At the catenary action capacity		
	Plastic hinge rotation (°)	Rotation of sub-assemblage (°)	Plastic hinge rotation (°)	Rotation of sub-assemblage (°)	Rotation of partial hinge (°)
MJ-B-0.52/0.35S	0.9	1.6	7.7	8.4	0.5
MJ-L-0.52/0.35S	0.5	1.5	6.9	13.2	5.7
MJ-B-0.88/0.59R	0.5	2.1	9.5	15.2	5.6
MJ-L-0.88/0.59R	0.7	2.1	8.2	14.0	8.0
MJ-L-1.19/0.59R	0.7	2.1	8.4	10.8	1.5

Table 4 indicates that under column removal scenarios precast concrete beam-column sub-assemblages were able to develop comparable rotations to RC sub-assemblages

[30]. However, except for MJ-B-0.52/0.35S and MJ-L-1.19/0.59R, the calculated rotation of sub-assemblages was significantly larger than the plastic hinge rotation at the catenary action stage, as listed in Table 4. The difference between the plastic hinge rotation and the rotation of sub-assemblages is attributable to the formation of a partial hinge in the vicinity of the curtailment point of beam top bars, as shown in Figs. 10(a-c). In accordance with the LVDT measurements along the beam length, the partial hinge rotation was approximately quantified as the difference between rotational angles of beam segments CD and BC (Fig. 5). Fig. 11 plots the calculated rotation angle of the partial hinge in MJ-L-0.52/0.35S, MJ-B-0.88/0.59R and MJ-L-0.88/0.59R. It is observed that the calculated rotation angle increased rapidly after the onset of catenary action, indicating the localisation of beam rotation at the partial hinge. For sub-assemblage MJ-B-0.88/0.59R, the maximum rotation angle of partial hinge is 5.6° , as listed in Table 4. Similar results were obtained for MJ-L-0.52/0.35S and MJ-L-0.88/0.59R (see Table 4). Thus, the partial hinge rotation significantly increased the deformation capacity of beam-column sub-assemblages under column removal scenarios.

Fig. 10 Partial hinge at the curtailment point of top bars

Fig. 11 Rotation angle of partial hinges at the curtailment point

3.5 Failure modes of beam-column sub-assemblages

Table 5 Failure mode of beam-column sub-assemblages

Specimen	In the middle joint	At the end column stub
MJ-B-0.52/0.35S	Fracture of bottom bars	Fracture of top bars
MJ-L-0.52/0.35S	Fracture of one bottom bar, pull-out of the other bar	Fracture of top bars
MJ-B-0.88/0.59R	Pull-out of bottom bars	Fracture of top bars
MJ-L-0.88/0.59R	Pull-out of bottom bars	Pull-out of bottom bars
MJ-L-1.19/0.59R	Pull-out of bottom bars	Fracture of top bars, pull-out of bottom bars

Table 5 summarises the failure modes of beam-column sub-assemblages at the middle joint and the end column stub. MJ-B-0.88/0.59R exhibited a typical failure mode of beam-column sub-assemblages at the middle joint and the end column stub, as shown in Fig. 12(b) and Fig. 13(c). Middle joint features pull-out failure of beam bottom longitudinal reinforcement at the interfaces of MJ-B-0.88/0.59R, as shown in Fig. 12(b). However, sub-assemblage MJ-B-0.52/0.35S exhibited fracture of bottom longitudinal reinforcement at the middle joint face, as shown in Fig. 12(a). Final failure of beam-column sub-assemblages was caused by final fracture of beam top longitudinal reinforcement near the end column stub, as shown in Figs. 13(a-c). Besides, development of tension force in the beam also resulted in pull-out failure of beam bottom longitudinal reinforcement near the end column stub, as shown in Figs. 13(d and e).

Fig. 12 Failure modes of beam-column sub-assemblages at the middle joint

Fig. 13 Failure modes of beam-column sub-assemblages at the end column stub

3.6 Horizontal interface behaviour

Depending on the surface preparation of precast beam units, different horizontal interface behaviour was observed between the precast beam unit and the concrete topping, as shown in Fig. 14. In MJ-B-0.52/0.35S and MJ-L-0.52/0.35S, a smooth horizontal interface was prepared between precast beam units and concrete topping, and the horizontal shear strength was designed in accordance with Eurocode 2 [25]. Under column removal scenarios, severe horizontal cracking was observed across the concrete interface at the CAA stage, as illustrated in Figs. 14(a and b). These cracks were mainly initiated in the region between the cut-off point of top bars and the end column stub. The horizontal interface cracks were mainly due to compression force in the beam at the CAA stage. The compression force increased sagging moment at the middle joint face and hogging moment at the end column stub face. Thus, shear force in the beam was increased as well, which induced interface cracking between precast beam units and cast-in-place structural topping, as shown in Figs. 14(a and b).

In comparison with MJ-B-0.52/0.35S and MJ-L-0.52/0.35S, shear force acting on the horizontal interface of MJ-B-0.88/0.59R and MJ-L-0.88/0.59R was increased due to high moment resistance at the beam ends. However, by intentionally roughening the concrete interface to around 3 mm roughness according to BS EN 1992-1-1:2004 [25], only limited interface cracks developed in the plastic hinge region near the end column stub, as shown in Fig. 14(c). It indicates that a roughened interface is effective in preventing horizontal cracking compared with a smooth interface.

(a) MJ-B-0.52/0.35S

(b) MJ-L-0.52/0.35S

(c) MJ-B-0.88/0.59R

(d) MJ-L-1.19/0.59R

Fig. 14 Horizontal cracking across the concrete interface

Compared with MJ-L-0.88/0.59R, the same surface preparation and stirrups were utilised in MJ-L-1.19/0.59R, but the beam top reinforcement ratio was increased to 1.19%. Severe interface cracking was observed across the horizontal interface, as shown in Fig. 14(d). Horizontal cracking weakened the composite action between precast beam units and structural topping, thereby reducing the CAA capacity of beam-column sub-assembly. For instance, the beam top reinforcement ratio in MJ-L-1.19/0.59R was increased by 0.31% in comparison with MJ-L-0.88/0.59R. However, its CAA capacity was only increased by 3.5 kN, as listed in Table 3.

3.7 Variation of steel strains at beam ends

The failure mode of MJ-B-0.88/0.59R can also be demonstrated by the measured steel strains of top and bottom longitudinal reinforcement at the middle joint and the end column stub faces, as shown in Fig. 15. At the middle joint face, strains of bottom longitudinal reinforcement started to decrease gradually after attaining the maximum value at the CAA stage, as shown in Fig. 15(a), indicating pull-out failure of reinforcing bars. However, at the end column stub face, tensile strains of top longitudinal reinforcement kept increasing until the rebars fractured at the catenary action stage, as shown in Fig. 15(b). Top longitudinal bars near the middle joint and bottom bars near the end column stub experienced compression at the CAA stage, and then were transformed to tension due to subsequent development of catenary action, as shown in Figs. 15(a and b). Thus, the measured steel strains agree well with the crack pattern and the failure mode of MJ-B-0.88/0.59R (see Fig. 12(b) and Fig. 13(c)).

(a) At the middle joint face

(b) At the end column stub face

Fig. 15 Strains of beam longitudinal reinforcement in MJ-B-0.88/0.59R

4. Discussion

In accordance with UFC-4-023-03 [4], the plastic rotation angle for beam-column sub-assembly MJ-B-0.52/0.35S is determined as 1.7° (0.029 radian), whereas the angle for the other four sub-assemblages is reduced to 0.6° (0.01 radian) due to pull-out failure of beam bottom bars in the middle joint. The acceptance criteria are reasonable if the CAA capacities are taken into account in analysis. However, when catenary action in the beam-column sub-assemblages is considered, the criteria are too conservative in comparison with the calculated sub-assemblage rotations, as listed in Table 4. Besides, pull-out failure of bottom reinforcement in the joint did not significantly reduce the

rotation capacity of beam-column sub-assemblages, as long as continuous longitudinal reinforcement was placed in the structural topping and properly embedded in the beam-column joint with sufficient anchorage length. Therefore, it is suggested that the acceptance criteria be increased to 11.5° (0.2 radian) to account for catenary action at large deformations, which is consistent with the required rotation for tie force development in the bridging beam [4]. It is noteworthy that the revised acceptance criteria are only suitable for fairly rigid boundary conditions. As for inadequate horizontal restraints, more experimental tests are needed to investigate the deformation capacity of beam-column sub-assemblages.

In design of precast concrete structures against progressive collapse, pull-out failure of embedded reinforcement in the beam-column joint has to be prevented to ensure a more robust structure. Thus, the embedment length of steel reinforcement is suggested to be increased. In addition, in determining the horizontal shear stress across the concrete interface, it is suggested an amplification factor be incorporated to consider the effect of compression force on horizontal shear stress. Furthermore, more stringent requirements on interface treatment have to be employed to ensure full composite action between the precast beam units and the cast-in-place concrete topping.

5. Conclusions

In this paper, five experiments were conducted to investigate the behaviour of precast concrete beam-column sub-assemblages under middle column removal scenarios. Two types of middle joint detailing, viz. 90° bend and lap-splice of bottom reinforcement, were tested to failure under quasi-static loading conditions. The following conclusions can be made:

- With continuous top reinforcement in the structural topping, CAA and catenary action can be developed in sub-assemblages with 90° bend and lap-splice of bottom longitudinal reinforcement in the joint. Typical failure mode in the middle joint of sub-assemblages is pull-out failure of bottom longitudinal reinforcement, except MJ-B-0.52/0.35S which exhibited fracture of bottom bars. Near the end column stub, fracture of beam top longitudinal reinforcement represented the most common failure mode. However, in MJ-L-0.88/0.59R and MJ-L-1.19/0.59R, bottom beam bars exhibited pull-out failure

- Higher top and bottom reinforcement ratios in MJ-B-0.88/0.59R and MJ-L-0.88/0.59R enhanced CAA and catenary action compared with MJ-B-0.52/0.35S and MJ-L-0.52/0.35S. Further increase in top reinforcement ratio of MJ-L-1.19/0.59R did not impose a considerable beneficial effect on the CAA capacity in comparison with MJ-L-0.88/0.59R, possibly due to severe horizontal interface cracking.
- Horizontal cracking was observed between the curtailment point of top bars and the end column stub in MJ-B-0.52/0.35S, MJ-L-0.52/0.35S and MJ-L-1.19/0.59R. At the CAA stage, development of compression force in the beam increased the horizontal shear stress at the concrete interface. Therefore, more stringent interface preparation has to be implemented to achieve full composite action between the precast beam units and the cast-in-place concrete topping.
- Precast concrete beam-column sub-assemblages were able to develop much higher rotation compared to the requirements in UFC-4-023-03 if catenary action in the beam is considered. It is suggested that the acceptance criteria be revised in accordance with experimental results.

However, the experimental results represent the resistance of precast beam-column sub-assemblages with rigid boundary conditions. In comparison with realistic lateral restraints, the CAA and catenary action capacities are overestimated. Therefore, further experimental tests are necessary to evaluate the influence of restraint boundary conditions on the behaviour of sub-assemblages.

6. Acknowledgement

The authors gratefully acknowledge the financial support provided by Protective Technology Research Centre of Nanyang Technological University, Singapore.

References

- [1] Ellingwood B, Leyendecker EV. Approach for design against progressive collapse. *Journal of the Structural Division, ASCE*. 1978;104:413-23.
- [2] Orton S, Jirsa JO, Bayrak O. Carbon Fiber-Reinforced Polymer for Continuity in Existing Reinforced Concrete Buildings Vulnerable to Collapse. *ACI Structural Journal*. 2009;106:608-16.

- [3] GSA. Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. General Services Administration, Washington, DC; 2003.
- [4] DOD. Design of Buildings to Resist Progressive Collapse. Unified Facilities Criteria(UFC) 4-023-03: Department of Defence, Washington, DC; 2013.
- [5] NIST. Best Practices for Reducing the Potential for Progressive Collapse in Buildings. NISTIR 7396: National Institute of Standards and Technology, Gaithersburg, MD; 2007.
- [6] ASCE. Seismic Rehabilitation of Existing Buildings. ASCE/SEI 41-06: American Society of Civil Engineers, Reston, VA; 2007.
- [7] Foley CM, Martin K, Schneeman C. Robustness in Structural Steel Framing Systems. Report MU-CEEN-SE-07-01: Department of Civil and Environmental Engineering, Marquette University; 2007.
- [8] Su Y, Tian Y, Song X. Progressive collapse resistance of axially-restrained frame beams. *ACI Structural Journal*. 2009;106:600-7.
- [9] Yu J, Tan KH. Experimental study on catenary action of RC beam-column sub-assemblages. *Proceedings of the 3rd fib International Congress 2010*: Washington, DC; 2010.
- [10] Yu J, Tan KH. Experimental and Numerical Investigation on Progressive Collapse Resistance of Reinforced Concrete Beam Column Sub-assemblages. *Engineering Structures*. 2013;55:90-106.
- [11] Yu J, Tan KH. Macromodel-based simulation of catenary action of RC beam-column sub-assemblages. *Proceedings of the 3rd fib international congress 2010*: Washington, DC; 2010.
- [12] Lew HS, Bao Y, Sadek F, Main JA, Pujol S, Sozen MA. An Experimental and Computational Study of Reinforced Concrete Assemblies under a Column Removal Scenario. NIST Technical Note 1720: National Institute of Standards and Technology, Gaithersburg, MD; 2011.
- [13] Bao Y, Kunnath SK, El-Tawil S, Lew HS. Macromodel-based simulation of progressive collapse: RC frame structures. *Journal of Structural Engineering*. 2008;134:1079-91.
- [14] Yi WJ, He QF, Xiao Y, Kunnath SK. Experimental study on progressive collapse-resistant behavior of reinforced concrete frame structures. *ACI Structural Journal*. 2008;105:433-9.
- [15] Kai Q, Li B. Dynamic performance of RC beam-column substructures under the scenario of the loss of a corner column-Experimental results. *Engineering Structures*. 2012;42:154-67.
- [16] Pham XD, Tan KH. Experimental study of beam-slab substructures subjected to a penultimate-internal column loss. *Engineering Structures*. 2013;55:2-15.
- [17] Izzudin BA, Vlassis AG, Elghazouli AY, Nethercot DA. Progressive collapse of multi-storey buildings due to sudden column loss - Part I: Simplified assessment framework. *Engineering Structures*. 2008;30:1308-18.

- [18] Sasani M, Kropelnicki J. Progressive Collapse Analysis of an RC Structure. The structural Design of Tall and Special Buildings. 2008;17:757-72.
- [19] Sadek F, Main JA, Lew HS, Bao Y. Testing and analysis of steel and concrete beam-column assemblies under a column removal scenario. Journal of Structural Engineering. 2011;137:881-92.
- [20] FIB. Seismic design of precast concrete building structures. Bulletin 27: Fédération internationale du béton, Laussane, Switzerland; 2003.
- [21] FIB. Precast Concrete in Mixed Construction. Bulletin 19: Fédération internationale du béton, Laussane, Switzerland; 2002.
- [22] Van Acker A. Accidental Actions and Progressive Collapse. International Seminar on Precast Concrete Structures: Department of Civil Engineering Universidade Nova de Lisboa; 2013.
- [23] CAE. Guideline for the Use of Precast Concrete in Buildings. Report of a Study Group of the New Zealand Concrete Society and the New Zealand Society of Earthquake Engineering. Second Edition ed: Centre for Advanced Engineering, University of Canterbury, Christchurch, New Zealand; 1999.
- [24] Shiohara H, Watanabe F. The Japan PRESSSS Precast Concrete Connection Design. 12th World Conference on Earthquake Engineering: Auckland, New Zealand; 2000.
- [25] BSI. Eurocode 2: Design of concrete structures—Part 1-1: General rules and rules for buildings. BS EN 1992-1-1:2004: British Standards Institution, London; 2004. p. 225.
- [26] Main JA, Bao Y, Lew HS, Sadek F. Robustness of Precast Concrete Frames: Experimental and Computational Studies. Structures Congress 2014/2014. p. 2210-20.
- [27] Park R, Bull DK. Seismic Resistance of Frames Incorporating Precast Prestressed Concrete Beam Shells. PCI Journal. 1986;31:54-93.
- [28] Patnaik AK. Behavior of Composite Concrete Beams with Smooth Interface. Journal of Structural Engineering. 2000;127:359-66.
- [29] Pauley T, Priestley MJN. Seismic Design of Reinforced Concrete and Masonry Buildings: John Wiley & Sons, Inc; 1992.
- [30] Yu J, Tan KH. Structural Behavior of RC Beam-Column Subassemblages under a Middle Column Removal Scenario. Journal of Structural Engineering. 2013;139:233-50.