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Experimental Study on Progressive Collapse Resistance of Fully Assembled Precast Concrete Frames under Static Loading

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Abstract

To examine how a fully assembled precast concrete frame resists progressive collapse, three half-scale subassemblies were prepared for a middle-column removal test: one monolithic reinforced concrete specimen (RC) as the baseline, and two precast joint specimens distinguished by corbel detail-PC1 with an exposed corbel and PC2 with a hidden corbel. Their load-resisting mechanisms and failure modes were compared under the same quasi-static pushdown protocol. The monolithic RC specimen developed both compressive arch action (CAA) and catenary action (CA); its second, catenary-stage peak reached roughly 22% above the CAA peak. PC2 likewise showed a two-stage response, yet its catenary mechanism remained underdeveloped, with the catenary peak at only 79% of the arching peak. PC1 stopped after the arching stage, failing through shear fracture of its dowel bars. Relative to RC, the CAA peaks of PC1 and PC2 stood at 76% and 81%, while their side-column restraint stiffness was comparable to the monolithic baseline. The test observations are expected to inform the collapse-resistant design of fully assembled precast frames.

Keywords

Fully Assembled Precast Concrete Frame; Progressive Collapse; Beam-column Dry Connection; Quasi-static Pushdown Test.

1. Introduction

Prefabricated frame construction has spread rapidly across China on account of its short erection cycle and reduced on-site pollution, yet its beam-column joints rely on dry assembly, which leaves member continuity and collapse resistance noticeably weaker than in monolithic construction. The Ronan Point apartment collapse triggered by a gas explosion in 1968 first drew engineering attention to disproportionate structural failure, and the topic has remained a persistent concern ever since. A substantial body of experimental and numerical work on precast-frame collapse has accumulated over recent decades [1-3,5-8], advancing the understanding of joint behavior, collapse mechanisms, and design provisions [2,4,7]. Gaps remain, however, for the dowel-bar and steel-angle type of fully assembled beam-column joint—the connection examined in this paper, which is widely adopted in current practice. The present work reports a middle-column removal quasi-static test on one monolithic specimen and two fully assembled specimens; their resistance mechanisms and failure patterns are contrasted to supply experimental evidence that may help refine collapse-resistant design for precast frames.

2. Test Overview

2.1 Specimen Design

Following the relevant national standards, a prototype seven-story office frame with four bays in each orthogonal direction was designed first; from its ground story, a three-column, two-beam

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subassembly was taken out and scaled to half size for laboratory testing. Three specimens were cast or assembled: the monolithic baseline RC, the exposed-corbel PC1, and the hidden-corbel PC2. Each subassembly carries two beams between three columns. Columns measure 300 mm square; beams are 250 mm wide and 400 mm deep; the clear beam span is 2,200 mm, and the unsupported column height is 1,800 mm. In RC the beams and columns were poured as one piece. The precast members, by contrast, were fabricated off site and assembled in the lab through corbel-mounted dowel bars together with steel angle cleats; every gap between precast parts and the dowel holes was grouted with non-shrink mortar. The concrete was proportioned to C30, and its measured cube strength averaged 32.6 MPa. Longitudinal bars were HRB400 deformed steel, with measured yield and ultimate strengths of 435 MPa and 605 MPa; shear reinforcement consisted of HPB300 plain bars, measured yield strength 325 MPa.

What sets PC1 apart from PC2 is the way the corbel is detailed. PC1 leaves the corbel exposed on the column face: the beam end sits directly on this projecting shelf, and its tie to the column is made through dowel bars and steel angles fastened at the beam-end top and bottom. In PC2 the corbel is buried inside the column section, so the column face stays flat and looks no different from a monolithic column; the beam end reaches the joint through embedded steel plates welded to its longitudinal bars. Across both details, dowel bars carry the joint shear while the angle cleats resist moment-two fully assembled joint configurations commonly seen in Chinese precast practice.

Table 1. Main parameters of specimens

Specimen	Joint type	Corbel type	Concrete grade	Rebar grade
RC	Cast-in-place monolithic	-	C30	HRB400
PC1	Dowel-bar + steel angle	Exposed	C30	HRB400
PC2	Dowel-bar + steel angle	Hidden	C30	HRB400

Note: column cross-section 300 mm x 300 mm, beam cross-section 250 mm x 400 mm, beam span 2200 mm, column height 1800 mm.

Reinforcement was arranged as follows. Each column carried eight 16 mm longitudinal bars, placed symmetrically around the section perimeter. The beams received two 18 mm bars along the top and two along the bottom, four 18 mm bars in total. Shear stirrups were 6 mm plain bars, spaced 50 mm apart in the joint core and beam-end confinement zones and 100 mm apart over the remainder of the span. The precast corbels in PC1 and PC2 were additionally fitted with four 14 mm main bars and 6 mm stirrups at 40 mm spacing; these details are summarized in Table 2.

Table 2. Reinforcement parameters of specimens

Member	Section /mm	Longitudinal bars	Stirrups	Remarks
Column	300x300	8D16	R6@50/100	Symmetric reinforcement
Beam	250x400	4D18	R6@50/100	2 bars top and bottom
Corbel (PC1/PC2)	-	4D14	R6@40	PC specimens only

2.2 Loading Scheme

The middle-column removal protocol, a standard quasi-static method for collapse-resistance assessment, was adopted throughout. Beneath the center column sat a 16 t mechanical jack, which reproduced the support the original column had provided before removal; above the same column, a 30 t manually operated hydraulic jack applied the downward load. Three loading phases were distinguished. First came preloading: small increments seated every contact surface and confirmed that each instrument channel read correctly. The lower jack was then unloaded step by step to emulate sudden middle-column failure. Once it had been removed completely, the upper jack continued

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driving the middle column downward until the specimen reached its ultimate capacity or showed a clear collapse mechanism. Early loading ran under load control, 3 kN per increment; after the peak resistance had passed or softening appeared, control switched to displacement increments of 5 mm.

The apparatus was built around a heavy steel portal frame anchored to the strong floor. A 30 t load cell, paired with a spherical hinge bearing, sat between the actuator and the middle-column top so that the vertical force reached the column concentrically; the second manual hydraulic jack below the specimen served as the middle-column support restraint. The two outer side columns were tied to the reaction floor through pinned bases that permitted rotation while holding back horizontal movement. On either side of the specimen, steel A-frames braced the middle-column region against out-of-plane displacement and rotation. At each side-column inflection point, a rotational hinge with a 50 mm diameter pin allowed the column end to turn freely in plane without sidesway; load cells placed in series with the anchor ties measured the horizontal tension or compression developed at this restraint.

2.3 Measurement Scheme

Instrumentation covered the response quantities considered critical to the test. The 30 t load cell recorded the vertical force on the middle column, and a displacement transducer on an independent reference frame recorded its vertical displacement. At the two side-column inflection points, load cells captured the horizontal reaction and LVDTs captured the horizontal displacement. Along each beam, several gauges traced the vertical deflection profile; on the columns, additional transducers tracked horizontal deformation at successive heights. Resistance strain gauges were bonded to longitudinal bars and to the concrete cover at critical sections-near beam ends and column faces-so that internal force development could be followed through the loading history.

3. Test Results and Analysis

3.1 Load-displacement Curves

For RC, the measured load–mid-column displacement trace separated into two resistance regimes. Resistance climbed steadily in the small-displacement range as compressive arch action developed, peaking at 119.2 kN when the column had moved down 130.9 mm. A short softening branch followed, after which the load climbed again as the structure shifted into catenary action; the second peak reached 145.3 kN at a far larger displacement of 613.9 mm-about 22% above the arching peak. Such a second, tensile-dominated peak is characteristic of a well-detailed monolithic frame, which can mobilize tensile membrane resistance at large drift [1,5].

PC2 also showed two stages, though both occurred at lower displacement levels. Its arching peak of 96.9 kN came at 95.95 mm, and the later catenary peak of 76.2 kN at 474.9 mm; the latter was only 79% of the arching peak, a sign that the tensile membrane mechanism in PC2 fell well short of the RC baseline. PC1 followed a different path altogether: its response stayed within the arching regime, peaking at 90.9 kN at 100.5 mm before the load dropped sharply as dowel bars in the joint sheared one after another. The test was stopped once the connection lost its integrity; no catenary phase could be identified [2].

Table 3. Summary of test results

Specimen	CAA peak /kN	CA peak /kN	Max. outward side-column disp. /mm	Max. inward side-column disp. /mm	Failure mode
RC	119.2	145.3	7.44	13.56	Beam rebar fracture
PC1	90.9	-	8.11	-	Dowel bar shear fracture
PC2	96.9	76.2	8.09	3.12	Concrete spalling

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Table 3 makes the capacity gap explicit: the arching peaks of PC1 and PC2 reached 76% and 81% of RC, respectively, a loss of vertical capacity attributable to the precast joint configuration. The side-column outward displacement histories help trace how catenary action evolved. In both RC and PC2, the outward side-column displacement after the arching peak eased back toward zero and then reversed inward, evidence that the transition into catenary tension had taken place. PC1 tells the opposite story: its outward displacement reached just 1.49 mm before the dowels failed in shear, and the side columns never moved inward-consistent with the absence of a developed catenary stage.

3.2 Side-column Restraint Reaction

The horizontal force at the side-column inflection point gives a direct read on the boundary restraint acting on the beam ends during collapse. All three specimens followed the same evolutionary pattern. Early in the arching phase the beam splayed outward under the thrust, so the reaction read as compression; once the beams entered the catenary regime and began to contract longitudinally, that reaction reversed into inward tension [7].

Table 4. Restraint properties at side-column inflection point

Specimen	Max. outward compression /kN	Max. inward tension /kN	Middle-column disp. at return to zero /mm	Restraint stiffness /(kN/mm)
RC	40.8	112.27	117.73	9.06
PC1	73.4	-	-	11.47
PC2	74.2	44.37	63.2	11.11

Table 4 lists the restraint forces. PC1 and PC2 developed maximum outward compressive reactions of 73.4 and 74.2 kN, both above the 40.8 kN recorded for RC—an indication that the corbel and dowel detailing in the precast specimens actually supplies stiffer horizontal restraint during arching than the monolithic joint; their restraint stiffness values, 11.47 and 11.11 kN/mm, sit somewhat above the 9.06 kN/mm of RC. The picture changes once tensile capacity is examined. PC2's maximum inward tension was only 44.37 kN, roughly 40% of the 112.27 kN developed by RC; PC1 never generated a tensile reaction at all, having never entered the catenary stage. The comparison exposes the central weakness of these dry joints: adequate compressive restraint during arching, but limited tensile capacity under large deformation that ultimately controls whether catenary action can develop [4].

3.3 Failure Modes

RC's failure unfolded gradually. Flexural tensile cracks appeared first at the beam ends as arching developed; with continued loading they spread toward mid-span, while the compressive beam toe began to crush. In the catenary stage the beam-end tensile bars yielded in succession-the bottom bars nearest the middle column broke first, other bars following as displacement grew. By the end, beam-end concrete had spalled extensively and several bars had ruptured, yet the specimen still carried large overall deformation, the ductile response expected of a well-detailed monolithic joint.

Cracking in PC1 started within the corbel concrete during arching. Past the peak, the dowel bars tying the beam end to the column through the steel angles began failing in shear; each dowel that sheared produced a distinct drop in the recorded load. The remaining dowels carried the joint until they had all failed in sequence, after which the beam end pulled free of the column. Little warning deformation preceded this sudden loss of capacity-the failure was plainly brittle.

PC2's hidden corbel stayed intact through the arching phase, with no visible joint distress. As the middle-column displacement mounted, the beam-end concrete started crushing and spalling. Upon entering the catenary regime the dowels were placed in direct tension; final failure came when that beam-end concrete had completely disintegrated and the dowels themselves fractured in tension.

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Between PC1's brittle shear failure and RC's ductile bar rupture, PC2 sat at an intermediate level of ductility.

3.4 Rebar Strain Analysis

Strain readings from RC's beam-end bars show the tensile reinforcement beginning to yield at a mid-column displacement near 50 mm. Through the catenary stage the bars kept elongating beyond yield, strains rising monotonically until ultimate tensile strain was reached and fracture occurred. The strain history confirms a full, sustained yielding process, matching the high energy dissipation and ductility observed in the specimen.

The precast specimens followed a markedly different strain path. In PC1 the beam bars stayed at low strain levels the whole test and never yielded; because dowel shear governed failure, the longitudinal reinforcement was never called on to develop tensile membrane action, and the catenary mechanism never switched on. PC2's beam-end bars did strain somewhat during the catenary stage, but crushing and spalling at the beam end degraded the bond and anchorage, capping how far the strains could grow. Its reinforcement therefore never reached the yielded elongation seen in RC. This reinforces the earlier point: discontinuous bars and degraded anchorage in fully assembled joints hold back the development of catenary tension.

4. Load-Resisting Mechanism Analysis

Pulling the three specimens together, the behavior under a lost column can be explained through two mechanisms that come into play one after another as deformation grows.

(1) Compressive arch action. At small mid-column displacements the beam behaves like a horizontal deep arch: its ends are held from moving horizontally, and downward deflection builds axial compression inside. The monolithic RC specimen lets this arching mechanism develop to the full, because beam ends and column are one cast piece, so concrete and reinforcement share the combined axial and flexural stresses effectively. The corbel shelf and dowel bars in the precast specimens proved able to transmit both vertical shear and horizontal thrust at this stage; their restraint stiffness values-PC1 11.47 kN/mm, PC2 11.11 kN/mm-even edge above RC's 9.06 kN/mm, confirming the corbel detail supplies adequate, if somewhat stiffer, horizontal restraint during arching. Restraint aside, the precast arching peaks reached only 76%–81% of the monolithic value, a shortfall traced to weaker overall joint integrity and bar discontinuity at the precast interface [1,5,7].

(2) Catenary action. Once mid-span deflection passes a certain threshold, beam-end rotation grows large enough to overcome the arching compression, and the beam turns into a tensile tie-catenary action. At this stage resistance comes mainly from the tension developed in the longitudinal bars along the inclined cable profile. RC's continuous reinforcement through the joint lets this tensile mechanism develop fully, pushing the second peak about 22% above the arching peak [8]. The dry connection in the precast specimens lacks the ductility to survive such large rotations undamaged: the angle cleats buckle or fracture, and the dowels either shear off (PC1) or break in tension after limited deformation (PC2). The catenary mechanism is therefore absent in PC1 and under-developed in PC2.

A side-by-side look at the two precast configurations clarifies why they differ. PC1's exposed corbel makes the joint behave more nearly as a pinned support; after the arching peak the dowels face increasingly severe cyclic shear and fail in succession, leaving the joint with almost no residual capacity and no route to catenary action. PC2's buried corbel, set inside the column, keeps somewhat better joint continuity, which allowed partial catenary development. Even there, however, large-deformation crushing and spalling of beam-end concrete still capped how far the catenary mechanism could proceed. The overall picture suggests that what governs collapse resistance in these fully assembled joints is primarily connection ductility-not the small-deformation arching capacity.

The catenary-stage capacity can be approximately estimated by the following formula [4,8]:

$$P=2\cdot As\cdot fy\cdot \sin\theta \quad (1)$$

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P denotes the vertical load carried by the middle column in the catenary stage; A_s is the total area of tensile longitudinal beam reinforcement; f_y is that reinforcement's yield strength; θ is the average angle between the inclined beam and the horizontal. Read mechanically, the expression ties catenary capacity to three quantities: the amount of tensile steel, its yield strength, and the rotation the joint can reach before failure. In fully assembled precast frames the limited ductility of the dry connection caps how far θ can grow-which is why, even with adequate beam reinforcement, the catenary mechanism cannot be fully mobilized.

5. Conclusion

Middle-column removal quasi-static tests on one monolithic reference and two fully assembled joint specimens were used to examine the progressive collapse resistance of fully assembled PC frames. The findings are summarized below.

(1) The monolithic RC specimen passed through both arching and catenary stages, its catenary peak about 22% above the arching peak, with good ductility throughout. PC1 stayed in the arching stage only and failed by brittle dowel shear fracture. PC2 showed a two-stage response, but its catenary action remained underdeveloped, reaching just 79% of the arching peak.

(2) The arching peaks of the two fully assembled specimens lay between 76% and 81% of the monolithic RC value, and their side-column restraint stiffness was actually slightly higher than RC's; the corbel-type precast joints thus transfer internal forces effectively during the arching phase with good local stiffness. The principal bottleneck for catenary action, nonetheless, is the limited ductility of the dry beam-column connection. Improving joint detailing and deformation capacity offers the most direct route to better collapse resistance in fully assembled precast frames; the present results can serve as an engineering reference in that effort.

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