Modelling Beam-Column Joints for Progressive Collapse Analysis

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Abstract

When a reinforced concrete frame is subjected to progressive collapse due to the loss of a structural column, the surrounding elements typically experience a significant overload that may lead to their collapse. The rotational capacity of beams and, consequently, the beam-column connections is a critical factor determining the structural resiliency. Numerical models developed to assess the structural response under a progressive collapse situation must incorporate the beam-column joint response. In this study, a review of the beam-column joint modelling approaches, constitutive models, and the ease of their numerical implementation are presented. Some of these models are utilized to simulate the response of a previously-tested reinforced concrete frame. The calculated structural response parameters are compared to the experimental results, and the accuracy of each constitutive model is discussed.

Keywords: beam-column joints, progressive collapse, reinforced concrete, finite element analysis, frame elements, component models, rotational-hinge model.

1 Introduction

Progressive, or disproportionate, collapse refers to a localized structural collapse that forces the adjoining members to fail, initiating a domino effect. Localized fire, natural disasters, vehicle impacts, terrorist attacks, and many other events may trigger the progressive collapse of a structure.

To mitigate the impacts of a progressive collapse, alternative load paths must be present in a structure. In a common progressive collapse scenario where a structural column is lost, three critical load resisting mechanisms form: 1) the compressive arch action (CAA), which is the additional flexural resistance due to the axial restraint of the surrounding structure; 2) the plastic hinge action (PH), where large structural displacements occur on the beams due to the plastic hinge formation; and 3) the catenary action (CA), where tensile resistance develops due to the extreme deflections of the beams (see Figure 1).

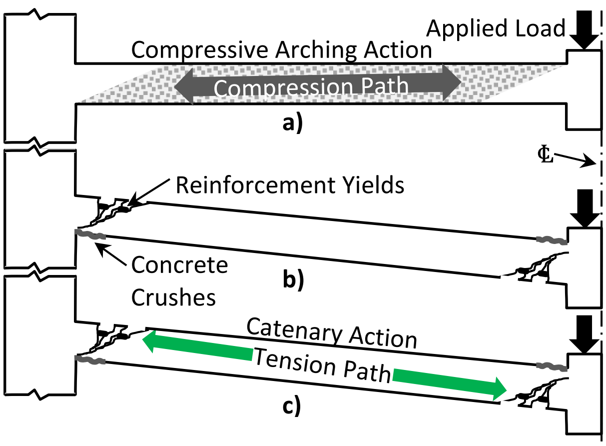

Figure 1. Resisting mechanisms: a) arching action, b) plastic hinge, and c) catenary action

Due to the concentrated deformations of the concrete beams at the beam-column connections, previous studies indicated the rotational capacity of the beams to control the development of catenary actions [1]. Additionally, beam-column joints are critical for resisting and distributing loads [2], as well as determining the rotational capacity of the beams.

In this study, existing state-of-the-art numerical beam-column joint modelling techniques and constitutive behaviours from the literature are assessed using a previously-tested planar reinforced concrete frame subjected to progressive collapse analysis by the removal of a ground-level structural column. The accuracy and easy-of-use of the joint models are evaluated by comparing the calculated response parameters with the experimental results.

2 Modelling of joints

Two main factors affect the beam-column joint behaviour: panel shear and bond-slip actions. The application of extreme loading on members adjacent to a beam-column joint results in substantial shear deformation in the joint panel zone. In addition, the common practice of terminating the longitudinal reinforcing rebar inside the joint diminishes the flexural resistance of the beams. Consequently, the joint damage mechanism due to high shear and bond stresses reduce the strength and stiffness of the frame.

Amongst various beam-column joints modelling techniques, three methods have been widely used: rigid-joint, rotational-hinge, and component models.

In rigid-joint models, joint damage is neglected by modelling a perfectly-rigid connection between the beam and the column elements, where moments are fully transferred from one element to the other (see Figure 2a). The rigid element region encompasses the physical joint core and, due to its stiffer response, the joint damage becomes concentrated at the interface with the beam, or column. Rigid-joints yield reasonably accurate results when beam-column joint damage is not a dominant structural behaviour. When this is not the case (e.g., a progressive collapse loading), such models fail to account for the actual joint panel deformations, which results in a unconservative (i.e., unsafe) strength and deformation calculation.

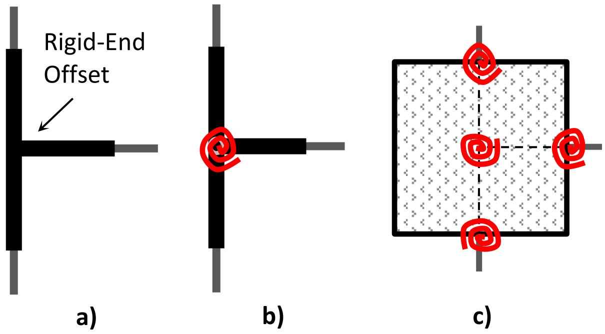

Figure 2. Beam-column joint modelling methods

In the rotational-hinge joint models, a single rotational-spring that accounts solely for the shear panel stress-strain deformation is incorporated at the centre of the beam-column connection, which is modelled with rigid-end offsets (see Figure 2b). The rigid links are used to neglect the damage in the elements at the joint panel, while joint deformations are simulated by the moment-rotation constitutive behaviour of the centre spring. This model was widely used in the literature (e.g. [3,4]) and, despite its simplified methodology, provided reasonably accurate results. This model should not be used, however, when the bond-slip action is a critical behaviour.

Component models incorporate a more realistic constitutive model, where joint panel shear-deformation and bond-slip are explicitly modelled. The shear deformation is usually incorporated either by springs or continuous panel elements, whereas the bond-slip relationships of adjacent elements are accounted using 1-D springs (see Figure 2c). Many component models have been proposed in the literature (e.g. [5-7]); however, these models require many constitutive models for each considered behaviour (i.e. spring), which in most cases are not readily available or difficult to obtain, thus hindering their effective and practical applications.

In this study, the rotational-hinge joint model is incorporated in the numerical analyses due to its relative simplicity, reasonable accuracy, and given that the bond-slip effects are not a critical mechanism for the frame structure examined. For comparison and quantification of the effects of beam-column joint modelling, a rigid-joint model is also examined.

2.1 Shear panel constitutive model

The available models in the literature typically use calibrated joint-panel shear stress-strain response derived from experimental testing of a set of specimens with certain geometry and reinforcement configuration. When analysing an existing or planned structure using these models, the calculation accuracy will be significantly affected by similarities between the structure being modelled and the experimental dataset used in the model calibration. Consequently, the existing joint models should be used with caution.

The backbone of the joint panel shear stress-strain response is generally controlled by four damage states: concrete cracking, yielding of stirrups, shear strength, and residual joint shear capacity, as shown in Figure 3.

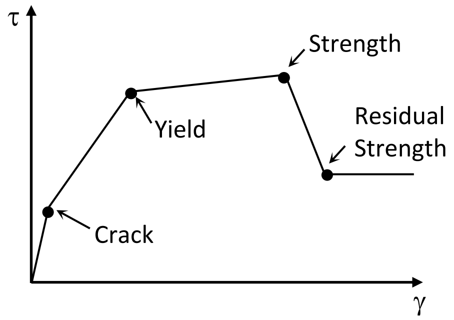

Figure 3. Joint shear panel damage states

In this study, five constitutive models from the literature were considered: Teraoka and Fujii [8], Theiss [9], Anderson et al. [10], Birely et al. [11], and Kim and LaFave [12].

The Teraoka and Fujii [8] constitutive model defines each damage state with a fixed strain pattern obtained from an experimental database though curve fitting. The relationships were derived based solely on concrete properties and joint type (i.e., exterior or interior joint, and transverse beams or not). Consequently, four joint backbone points can be quickly defined using this model. However, this simplicity may result in deteriorated reliability and accuracy.

Theiss [9] proposed a constitutive model that employs fixed strain values and percentages of the maximum shear stress for the joint backbone response. The Theiss [9] model uses the Modified Compression Field Theory (MCFT) [13] to determine the nominal shear capacity of the joint. The MCFT, however, employs an iterative, 17-step, calculation procedure to estimate the shear stress capacity, which hinders the practical application of this model.

The Anderson et al. [10] model calculates the stress and strain backbone points using fixed stiffness values for each segment that are based on the joint maximum shear stress. It was calibrated for internal beam-column joint assemblies with insufficient amounts of transverse reinforcement, which might result in a reduced accuracy for adequately-designed joints.

The Birely et al. [11] model defines the joint-shear backbone with only two points: flexural yield of the adjacent beam, and the maximum shear capacity, with a brittle failure once the capacity is reached. This model is placed at the beam-joint interface, not at the centre of the beam-column connection. The model limits the beam moment capacity as per the joint response. The bilinear constitutive behaviour makes the implementation relatively simple. However, this model was developed solely for interior joints, and uses fixed maximum strain and stiffness values.

The Kim and LaFave [12] defines the crack, yield, and residual strength damage states as proportional to the maximum shear and strain values. Its main advantage is its "unified" constitutive model that does not employ fixed values of stress or strains. It incorporates the effects of the compressive strength of the concrete, in-plane and out-of-plane geometry, joint eccentricity, beam reinforcement, and joint transverse reinforcement for a comprehensive maximum shear and strain calculation.

2.1.1 Cyclic hinge response

When subjected to cyclic loading conditions, beam-column joints typically experience a highly-pinched hysteresis response. Even though this study performs only nonlinear static analyses, the beam-column joint still experiences unloading due to the compression-tension alternation between the CAA and CA mechanisms. Consequently, it is important to consider the hysteretic response of the joint for progressive collapse analyses. Figure 4 shows the joint cyclic behaviour proposed by Lowes et al. [14].

Similar to the backbone response of the joint, the majority of existing studies derive the cyclic pinching parameters based on an experimental curve fitting approach; very few studies propose generally applicable pinching. This study incorporates the hysteretic parameters obtained by Jeon et al. [4], due to its comprehensible set of 124 beam-column joint specimens analyzed.

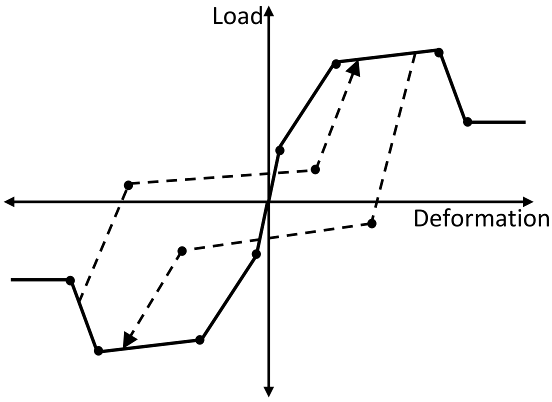

Figure 4. Beam-column joint hysteretic behavior (adopted from Lowes et al. [14])

3 Experimental verification

Amongst the experimental studies available in the literature for the progressive collapse conditions, the work of Lew et al. [1] was selected for this numerical study due to its planar configuration and elements designed per modern building codes. The planar configuration of the frame permits a simpler and computationally efficient numerical modelling. In addition, the proper design of frame elements allows for the exclusion of structural behaviours such as bond-slip and column shear failure, thereby isolating the behaviours of beams and beam-column joints studied herein.

The frame is part of a 10-story structure with the design carried out by a consulting engineering firm as per the requirements of ACI 318-02 [15] for the seismic design category C. The frame is comprised of a symmetric beam and column assembly, with the middle column representing a column loss scenario. Each external column was embedded into spread footings, which was clamped down to the strong floor. The top of the column was restrained by a two-roller fixture and the load was applied in the form of a displacement on the middle column (see Figure 5).

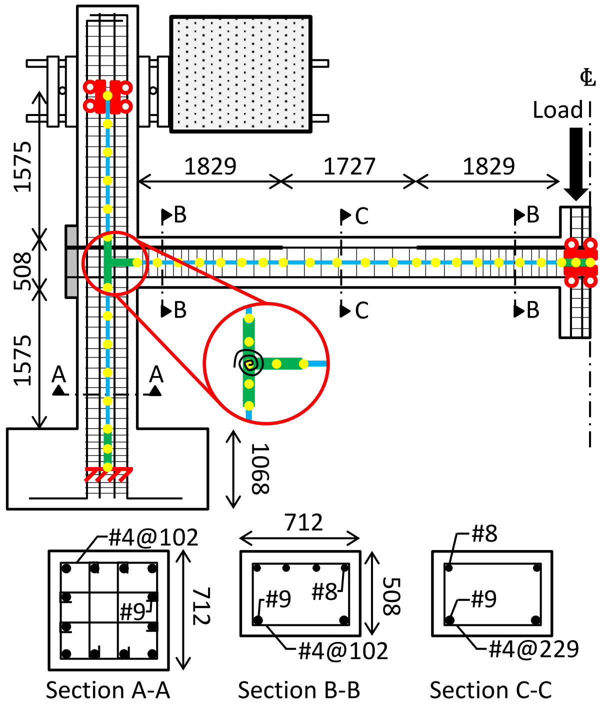

Figure 5. Lew et al. [1] frame's specimen and numerical model

The beam's reinforcing steel was anchored by an external plate attached to the exterior of the beam-column joint face, as shown in Figure 5, to represent the continuity of the longitudinal bars. The average compressive strength of the concrete was 32 MPa, and the average reinforcing steel properties were as listed in Table 1.

Table 1. Average reinforcing properties

Bar Size	Yield Stress [MPa]	Ultimate Stress [MPa]	Hardening Strain [me]	Rupture Strain [me]
#4	524	710	9	140
#8	476	648	8	210
#9-B	462	641	7	180
#9-C	483	690	7	170

The reported failure mode was the rupture of the longitudinal bottom reinforcement of the beam at the beam-middle column interface. In Figure 6, the cracking pattern at the failure load is shown.

Although the beam-column joint was not the main failure mechanism, Figure 6 shows extensive shear cracking at the joint, which indicates a high joint stress demand.

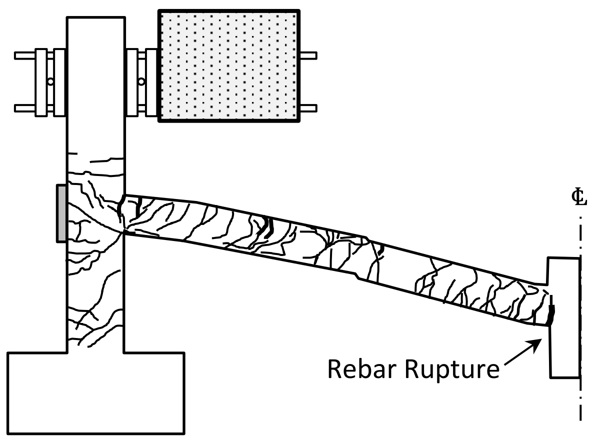

Figure 6. Cracking pattern at failure condition (adopted from Lew et al. [1])

4 Numerical model

The numerical model was developed using the OpenSees [16] software with displacement-based frame elements idealized at the centreline of the structural components (see Figure 5). One half of the structure was modelled due to symmetry. The height of the column spans from the mid-height of the concrete foundation to the mid-height of the two-roller fixture as shown in Figure 5.

Semi rigid-end offsets were incorporated at the beam-column joints and the column footing, where the area of longitudinal reinforcement was doubled and the transverse reinforcement space reduced in half. These semi rigid-end elements cover the beam-column intersection and half the column's foundation height. As mentioned for rigid joints, these elements act to shift the damage concentration from the joint panel to the beam or column interfaces. The semi-rigid modelling approach permits more flexibility at the joint panel zones as compared to the rigid-end offsets. At the axis of symmetry, semi rigid-ends were also included to account for the increased stiffness of the intersection.

A two-dimensional fibre based cross-section with concrete and longitudinal steel fibres were incorporated in each frame element, following the design shown in Figure 5. Confinement of each element concrete core was calculated using the Mander [17] model.

The column foundation was idealized as a rigid support. The two-roller horizontal fixture allows the column to displace vertically; consequently, it was modelled with a rigid vertical roller. Finally, at the beam-middle column intersection at the plane of symmetry, a rigid horizontal roller was defined (see Figure 5).

Load was statically applied in the form of vertical displacements on the three nodes that comprise the beam-middle column connection. A 1-mm displacement increment was imposed downwards on each of these nodes over the middle column joint up to the structural failure, characterized as the structural collapse or non-convergence of the load stage results.

The analysis was repeated using different constitutive models incorporated in the rotational-hinge beam-column joint model studied.

4.1 Beam-column joint

OpenSees [16] provides a number of beam-column joint elements in its online user's manual. However, these elements are component model elements, for which the analyst needs to specify all constitutive model parameters for each behaviour considered (e.g., shear panel stress-strain and bond-slip responses). The reference manual, however, does not offer guidance on how to obtain or calculate such constitutive models, leaving it to the discretion of the analyst. This limits the software usage to analysts who have expert knowledge on the beam-column joint models (e.g., researchers), which hinders the practical use of OpenSees [16] by the engineering community and practicing engineers.

In this study, the previously-discussed damage state parameters were calculated and incorporated into the joint element using a *zeroLength* rotational-hinge. The *pinching4* uniaxial material model that employs the hysteretic material behaviour of the joint (shown in Figure 4), developed by Lowes et al. [14], was incorporated in the hinge. The shear stress-strain backbone response calculated at each joint model was converted to a moment-rotation response using the equations derived by Celik and Ellingwood [3]. Figure 7 shows the calculated backbone moment-rotation responses for each model.

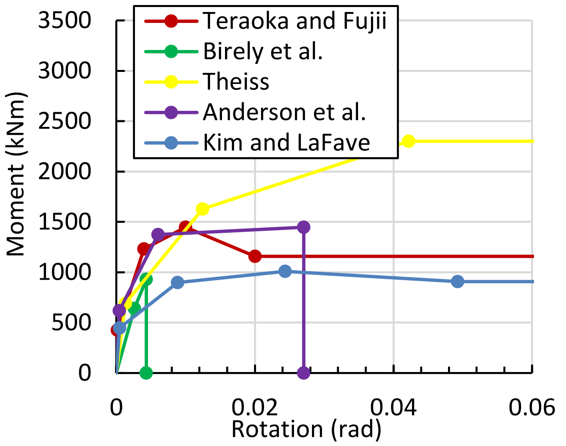

Figure 7. Calculated moment-rotation joint response

As shown in Figure 7, the Kim and LaFave [12] and the Theiss [9] models calculated the lowest and highest moment-rotation capacity, respectively, for the frame analyzed in this study. Anderson et al. [10] and Teraoka and Fujii [8] models calculated a similar joint response up to the joint moment capacity, with different post-peak behaviours. Birely et al. [11] calculated the lowest moment capacity. However, this model's moment capacity refers to the beam moment capacity and not the joint moment capacity, since it is incorporated as a spring in the beam-joint interface, and not at the joint centre like the other models. In this study, the joint was properly designed and was not expected to reach its shear strength capacity before the flexural capacity of the beam was reached. Thus, only the initial damage states of the moment-rotation curves should govern the joint response.

5 Results and discussion

The total load-displacement response of the middle column is shown in Figure 8 for three of the joint models examined. These three models represent the effects of the five joint models analyzed in this study. The response obtained from the Theiss [9] and the Anderson et al. [10] models were virtually the same as the Teraoka and Fujii [8] model, with a response deviation of less than 3%.

The Kim and LaFave [12] and the rigid joint model calculated the softest and stiffest numerical responses, respectively, whereas the Birely et al. [11] model calculated a response essentially identical to the rigid joint model (see Figures 8 and 9). The Teraoka and Fujii [8] model calculated a response in between the Birely et al [11] and the Kim and LaFave [12] models.

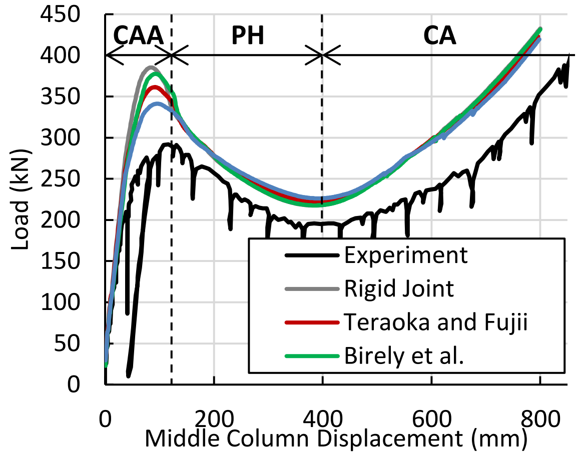

Figure 8. Numerical models' load-displacement response

The numerical models captured the three load resisting mechanisms (see Figure 1) with varying degrees of success. The first region of the curve, (i.e., the compressive arch action region) goes up to the beam flexural capacity, where a peak form in the load-displacement response. In the plastic hinge region, the capacity of the beam, and the frame, starts to degrade due to the concrete crushing and steel yielding. The catenary action region progresses based on the additional strength provided by the development of tension forces in the beam-column assembly (see Figure 8).

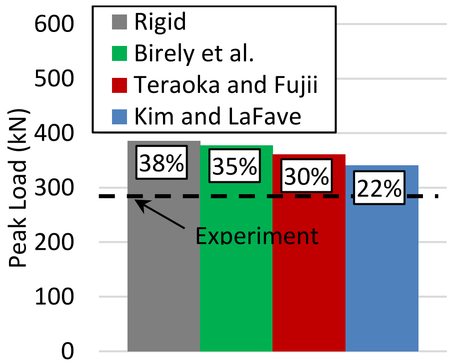

Figure 9. Peak compressive arch response and percent deviation to experimental result

All models calculated the rupture of the bottom beam reinforcement at the middle column interface at approximately the same middle column displacement, which indicates that the strains at the beam-column interface reached the ultimate value for the reinforcement regardless of the joint model analyzed. In Figure 8, the response is shown up to a displacement close to where the models failed. Experimental response is terminated at this point due to the extreme damage state that the structure sustained; the experimental response continues up to 536 kN at a displacement of 1076 mm.

As seen in Figures 8 and 9, the difference in each calculated response is not significant. The Kim and LaFave [12] model provided a better correlation to the experimental response, still with an overestimation of more than 20%. The "unified" stress-strain formulations utilized in this model was the most comprehensive of all models considered and are thought to be responsible for the "closer-to-experimental" calculated structural response.

All joint models considered in this study exhibited negligible discrepancies in the calculated responses at the PH and CAA regions. This indicates that the beam-column joint does not play a critical role after the force peak strength at the CA region (see Figure 8). This might be due to the large rotation of the beam once the plastic hinge started to form at the PH and the CAA regions.

The lack of a visual post-processor interface in OpenSees [16] makes it practically impossible to determine the cracking and damage conditions of the frame. The failure mode can only be estimated manually based on stress-strain response plots of each material, where it can be checked if the ultimate strength has been reached.

6 Conclusions

This study presented the numerical simulation results for a planar reinforced concrete frame subjected to a progressive collapse scenario by the removal of a column. The findings of this study support the following conclusions:

- Beam-column joint response is of critical importance in the compressive arch region of the structural response under progressive collapse conditions. In this study, the incorporation of beam-column joint models resulted in a decrease in the peak strength in this region by up to 16% as compared to not modelling the joint damage through the use of a rigid-joint model.
- All models that considered the joint response calculated similar structural responses regardless of the beam-column joint model utilized, with a maximum strength difference of 13% between the models.
- The beam-column joint models did not have a significant impact on the plastic hinge and catenary action regions. A possible reason for this is that the structural response is mainly dominated by the large plastic-hinge deflections of the beams in these regions.
- Most of the beam-column joint constitutive models available in the literature are highly dependent on the experimental dataset used for their calibration. This requires an expert knowledge from the engineer as to which model is more appropriate for the structure being analyzed, and hinders their practical application in industry.
- OpenSees presents a number of beam-column joint elements for numerical analysis. However, very limited user documentation is available as to what constitutive model is recommended to use, where to find them, and how to calculate the input parameters. This results in significant challenges for the correct use of these models. In addition, the lack of a visual post-processor interface makes it practically impossible to assess the structural deflections, crack patterns, damage states, and failure modes.
- The results of this study indicate that even properly designed joints have a significant effect on the structural performance under progressive collapse conditions. It is expected that poorly-designed joints will exhibit a much severe performance loss.

7 References

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