PROGRESSIVE COLLAPSE BEHAVIOR OF THREE-DIMENSIONAL FRAME STRUCTURES SUBJECTED TO COLUMN LOSS DURING SEISMIC EXCITATION

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Abstract - This study presents a nonlinear dynamic OpenSees framework for investigating the seismic progressive collapse response of three-dimensional reinforced concrete frames subjected to first-storey corner-column loss. The model incorporates material and geometric nonlinearities, distributed plasticity, rigid diaphragm constraints, and spatial force redistribution while maintaining computational efficiency for nonlinear time-history analysis. Two configurations, namely an intact frame and an initially damaged frame, were analyzed under earthquake ground motion. The results show that corner-column loss significantly increases compressive axial-force demand in the adjacent column, whereas the corresponding shear and bending moment responses increase only moderately. In the beam above the removed column, the response changes from slight compression to tensile axial force, while the bending moment decreases by approximately 50%, indicating a transition toward an axial tension-dominated catenary-like mechanism. The damaged frame also develops substantially larger lateral displacement demand, with peak roof displacement increasing by approximately 275% compared with the intact configuration.

Key words - Progressive collapse; column loss; Seismic excitation; Catenary action; Nonlinear dynamic analysis; 3D frame; OpenSees

1. Introduction

Progressive collapse (PC) of building structures has received sustained attention following several disproportionate collapse events, most notably the Ronan Point apartment failure in London in 1968, where the loss of a single load-bearing element led to extensive structural damage. Consequently, preventing PC has become an important objective in structural design, particularly for buildings subjected to accidental or extreme actions such as explosions, impacts, and earthquakes. Robustness-based design philosophies and alternate load path concepts have therefore been incorporated into analytical frameworks and design recommendations [1], [2].

Among possible initiating scenarios, column loss is widely recognized as a critical trigger of PC in framed structures. The removal of a vertical load-carrying member disrupts force equilibrium and deformation compatibility, causing rapid redistribution of gravity loads and amplification of internal forces and displacements. When structural redundancy, ductility, or deformation capacity is insufficient, local damage may propagate through the structural system and lead to partial or global collapse [3]-[5].

Early analytical studies of PC commonly relied on linear static or nonlinear static pushdown procedures because of their simplicity and low computational cost. However, static approaches cannot fully capture the dynamic nature of column loss, including inertia effects, transient force redistribution, and time-dependent deformation evolution. Even when dynamic amplification factors are introduced, static procedures may provide inaccurate estimates of peak response and may not adequately represent changes in load-resisting mechanisms at large deformations [6], [7].

Nonlinear dynamic analysis has therefore been increasingly used for progressive-collapse assessment. Previous experimental and numerical studies have shown that, after column loss, framed structures may experience a shift from flexural-dominated behavior toward axial force-dominated load-transfer mechanisms. These mechanisms include tensile axial action in beams and increased compressive demand in adjacent columns. The ability of beams, joints, and connections to sustain large deformation and combined force demands is essential for maintaining post-damage stability.

Despite these advances, many existing studies remain based on two-dimensional frame models. Such models are useful for understanding local mechanisms but cannot fully represent three-dimensional load redistribution, diaphragm compatibility, participation of orthogonal frames, or torsional tendency induced by asymmetric damage. Full three-dimensional nonlinear simulations can provide a more realistic representation, but they are often computationally demanding and may suffer from convergence difficulties, particularly when repeated nonlinear time-history analyses are required. This creates a need for simplified but mechanics-consistent three-dimensional modeling approaches that can capture the essential features of seismic progressive-collapse response with acceptable computational efficiency.

Another important issue is the interaction between local column loss and seismic excitation. Many PC studies focus mainly on gravity-dominated loading, whereas the seismic response of a structure with a missing vertical load-carrying member remains less well understood at the system level. In such cases, the missing column alters the stiffness distribution, force-transfer paths, and deformation compatibility of the frame, which may significantly affect the subsequent dynamic response under earthquake excitation.

Motivated by these gaps, this study develops a computationally efficient nonlinear dynamic modeling framework in OpenSees for evaluating the seismic progressive-collapse response of a three-dimensional RC frame with an initially removed first-storey corner column. The proposed model is intentionally simplified to improve numerical robustness while retaining the key physical mechanisms governing post-damage behavior, including geometric nonlinearity, axial force redistribution, spatial load transfer, and axial tension-dominated catenary-like response. The specific objectives are to: (i) quantify internal force redistribution in the column adjacent to the removed corner column; (ii) examine the axial-force reversal and change in load-transfer mechanism in the beam above the removed column; and (iii) evaluate the global displacement amplification of the damaged frame under seismic excitation. By linking local member-force response with global deformation behavior, this study aims to provide mechanical insight and a practical modeling tool for performance-based seismic progressive-collapse assessment of three-dimensional frame structures.

2. Literature review

Research on PC has expanded considerably over the past two decades because of the need to understand structural response under extreme and accidental actions. Early studies by Izzuddin et al. [8] and Vlassis et al. [9] established analytical frameworks for assessing collapse following column removal and highlighted the importance of dynamic effects and alternate load paths. Later investigations showed that conventional static procedures may significantly underestimate the response after column loss because they cannot adequately capture inertia effects, time-dependent redistribution, and large-deformation behavior [10] - [12].

Nonlinear dynamic analyses further demonstrated that framed structures may gradually transition from flexural-dominated behavior to axial-force-dominated mechanisms, particularly beam catenary action [13]. Adjacent columns also experience increased axial demand after column removal, which may govern failure if axial–flexural interaction is not properly considered [14] - [16]. Although detailed finite element models can reproduce these mechanisms with high accuracy, they are computationally expensive and often encounter convergence difficulties. Consequently, simplified yet mechanically consistent approaches have been proposed to improve computational efficiency while retaining essential collapse mechanisms [17] - [19].

Most previous studies rely mainly on two-dimensional frame models and gravity-dominated scenarios, with limited consideration of three-dimensional seismic column-loss interaction. Therefore, a clear research need exists for efficient 3D nonlinear dynamic frameworks capable of capturing axial-force redistribution, catenary action, and global instability under seismic column-loss scenarios.

3. Proposed OpenSees modeling framework

This study develops a simplified three-dimensional nonlinear dynamic modeling framework in OpenSees to evaluate the seismic progressive-collapse response of frame structures with an initially removed corner column (Figure 1). The framework is designed to capture the dominant post-damage mechanisms while maintaining numerical stability and computational efficiency for repeated nonlinear time-history analyses.

The structural system is represented by a full three-dimensional frame model to account for spatial force redistribution and interaction among orthogonal framing members. Beams and columns are modeled using distributed-plasticity beam–column elements, allowing axial–flexural interaction and large-deformation response to be represented within a computationally efficient formulation. Rigid floor diaphragms are assigned at each storey to enforce in-plane compatibility among floor nodes and to facilitate three-dimensional load transfer after local damage.

Material nonlinearity is simulated using established uniaxial constitutive models for concrete and reinforcing steel, whereas geometric nonlinearity is included to account for second-order effects and large-displacement behavior. These modeling features are necessary to represent post-damage force redistribution, deformation compatibility, and the possible development of axial tension-dominated load-transfer mechanisms in beams adjacent to the removed-column region.

In the proposed framework, the column-loss condition is introduced after the gravity-load equilibrium state has been established and before the application of seismic excitation. The subsequent nonlinear time-history analysis therefore represents the seismic response of an initially damaged structure. This modeling strategy enables a direct comparison between the intact and column-loss configurations under the same ground-motion input, thereby isolating the influence of the missing vertical load-carrying member on the dynamic response.

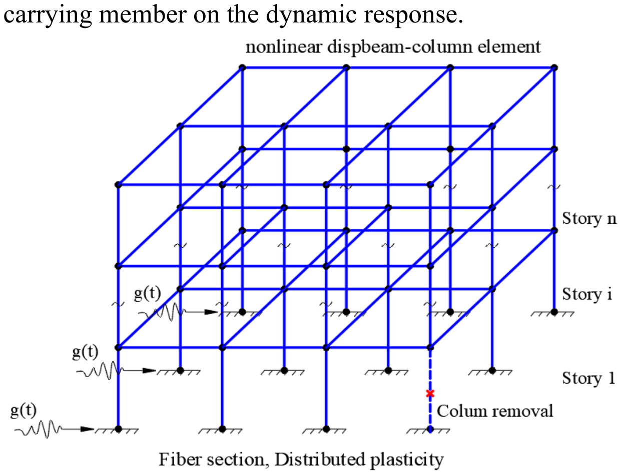

***Figure 1.** Proposed model in Opensees*

Compared with high-fidelity finite element models, the proposed approach avoids detailed local modeling of fracture, bond–slip, joint failure, and slab damage. This simplification reduces computational demand and improves convergence robustness, while retaining the key mechanics relevant to system-level progressive-collapse assessment, including axial-force redistribution, dynamic amplification, geometric nonlinearity, and spatial load transfer.

The use of a three-dimensional modeling framework is essential for the present problem because corner-column loss produces an inherently asymmetric and spatial load-redistribution mechanism. A two-dimensional frame model can only represent force transfer within a selected planar frame and cannot capture the participation of transverse beams, orthogonal frames, diaphragm compatibility, or torsional tendency induced by eccentric damage. In the adopted 3D model, rigid diaphragms allow gravity and seismic demands to be redistributed through beams and columns in both principal directions, providing a more realistic representation of the interaction between local column-loss effects and global seismic response.

Model validation:

The adopted OpenSees modeling strategy was validated against the experimental results of reinforced concrete interior beam–column subassemblages I1 and I2 reported by Qian and Li [20]. These specimens have been widely used as benchmark tests in progressive-collapse studies because they reproduce the local response of RC beam–column subassemblages under a column-removal-type loading condition. To quantify the agreement between the numerical and experimental responses shown in Figure 2. The comparison indicates that the OpenSees model predicts the peak resistance with errors of approximately −3.34% for specimen I1 and −4.57% for specimen I2. The displacement corresponding to the peak or plateau resistance is also reproduced with negligible deviation, while the initial secant stiffness differs by approximately 3–4%. These results indicate that the adopted fiber-section modeling strategy can reasonably capture the local nonlinear load–displacement response of RC beam–column subassemblages under column-removal-type loading, particularly in terms of initial stiffness, peak resistance, displacement capacity, and the overall response trend.

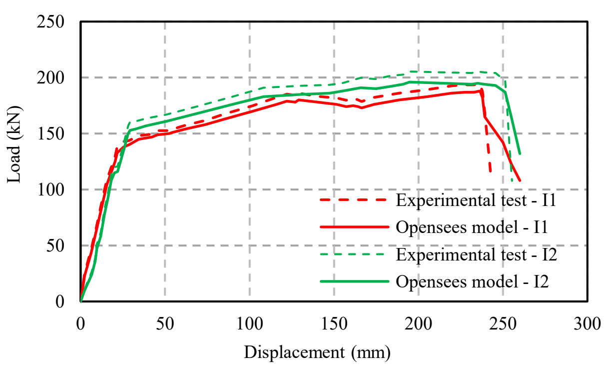

Figure 2. *Validation of the OpenSees model against the experimental of Qian and Li [20]*

It should be emphasized, however, that this validation is limited to the component/subassemblage level. The comparison with Qian and Li's tests supports the adopted local beam–column modeling strategy, but it does not constitute a full validation of the complete three-dimensional seismic progressive-collapse model. System-level effects, including rigid diaphragm action, torsional response, spatial load redistribution through orthogonal frames, multi-storey interaction, and the coupling between seismic excitation and column-loss-induced damage, are not directly validated by the subassemblage tests. Full system-level validation would require additional evidence from large-scale shake-table tests, system-level column-removal experiments, or high-fidelity finite element benchmark simulations.

4. Numerical model and analysis methodology

A six-storey three-dimensional RC moment-resisting frame was modeled in OpenSees to investigate the seismic progressive-collapse response associated with first-storey corner-column loss. The frame has a regular 4 × 4 bay plan, 6.0 m spans in both directions, and a uniform storey height of 3.3 m. Rigid floor diaphragms were assigned at each level, and all base nodes were fully restrained. Beams and columns were modeled using distributed-plasticity beam–column elements with fiber-section discretization, while geometric nonlinearity was considered through the P–Δ transformation. Gravity loads were first applied to establish the initial equilibrium state, and the seismic mass was derived consistently from the applied nodal gravity load using $m = W/g$. Thus, a 200 kN nodal load corresponds to an equivalent mass of approximately 20,390 kg.

Two configurations were analyzed under the same El Centro ground motion: an intact frame and an initially damaged frame. In the damaged model, the selected first-storey corner column was removed after gravity-load equilibrium and before seismic excitation, corresponding to $t_r = 0$ s with respect to the seismic record. The recorded responses include internal forces in the column adjacent to the removed column, internal forces in the beam above the removed column, and lateral roof displacement. The main modeling assumptions, material properties, loading conditions, mass definition, seismic input, and recorded response quantities are summarized in Table 1.

Table 1. *Summary of numerical model, seismic input, and column-removal parameters*

Parameter	Value / Assumption
Number of storeys	6
Storey height	3.3 m
Total height	19.8 m
Plan layout	4 × 4 bays
Bay length	6.0 m in both X and Y directions
Plan dimension	24.0 m × 24.0 m
Removed column	First-storey corner column
Column-removal time	$t_r = 0\ s$
Column-removal procedure	The selected first-storey corner column is removed after gravity-load equilibrium and before the seismic excitation
Static re-equilibration after removal	Not performed
Main post-removal mechanism	Dynamic amplification, spatial load redistribution, axial force redistribution, and catenary action
Analysis scenario	Comparison between intact and initially damaged column-loss models under the

	same seismic record
Column section	600 mm × 600 mm
Beam section	300 mm × 600 mm
Concrete model	Concrete02
Concrete compressive strength	30 MPa
Peak concrete strain	0.002
Ultimate concrete strain	0.02
Steel model	Steel02
Steel yield strength	400 MPa
Steel elastic modulus	200 GPa
Steel hardening ratio	0.01
Beam reinforcement	$4\phi20$ top + $4\phi20$ bottom
Column reinforcement	$12\phi25$
Concrete cover	40 mm
Section model	Fiber section
Element type	Nonlinear beam-column element
Plasticity model	Distributed plasticity
Geometric nonlinearity	P–Delta transformation
Base condition	Fully fixed
Floor diaphragm	Rigid diaphragm at each floor
Gravity load	200 kN vertical nodal load at each floor node
Gravity-load application	Static load-control analysis before transient analysis
Initial condition	Gravity-equilibrium state
Mass source	Equivalent nodal mass derived from gravity load, $m = W/g$
Equivalent nodal mass	20390 kg per floor node corresponding to a 200 kN nodal gravity load
Seismic record	El Centro
Excitation type, direction	Uniform base excitation, X-direction
Scaling method	Unscaled benchmark record
Record duration	31.18 s (time step 0.02s)
Damping model, ratio	Rayleigh damping, 5%
Integration scheme	Newmark, average-acceleration method, ($\beta = 0.25, \gamma = 0.5$)
Recorded responses	Member internal forces and roof displacement

5. Result analysis

5.1. Internal force redistribution in the column adjacent to the removed column

5.1.1. Axial force (N_c) response

The axial-force response of the column adjacent to the removed corner column increases significantly in the column-loss model. As shown in Figure 3, the compressive axial force in the damaged configuration reaches a peak value of approximately 2250 kN near the end of the seismic record, whereas the intact frame develops a substantially lower peak axial force of approximately 1000 kN. The values summarized in Table 2 confirm that axial-force amplification is the most pronounced response increase observed in the adjacent column after corner-column loss.

This increase reflects the role of the neighboring column in the alternative load-transfer path formed after the loss of the corner column. Once the original vertical load-carrying mechanism is interrupted, part of the gravity and seismic demand is redistributed through the surrounding beams and adjacent columns, leading to a substantial increase in compressive axial-force demand in the neighboring column. The gradual growth of axial force throughout the seismic excitation indicates that the redistribution process is governed mainly by global deformation compatibility, spatial frame interaction, and progressive stiffness degradation, rather than by an abrupt local instability mechanism.

Compared with the corresponding shear and bending responses, the axial-force amplification is substantially more significant, indicating that post-damage behavior of the adjacent column is governed primarily by additional axial compression associated with three-dimensional load redistribution. This observation highlights the importance of checking adjacent columns for increased axial compression and axial–flexural interaction in seismic progressive-collapse assessment.

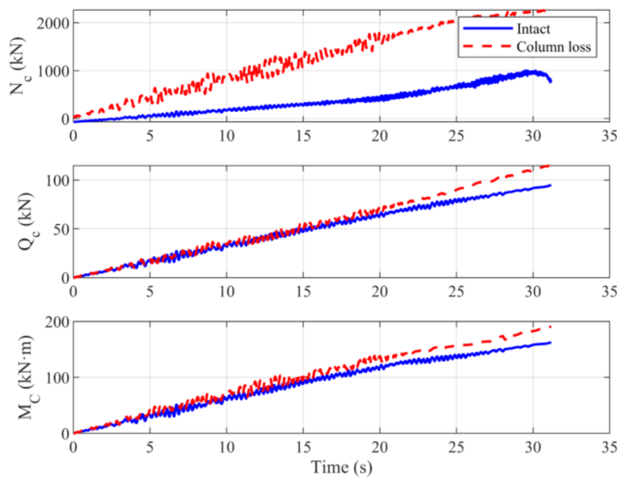

Figure 3. *Internal forces in adjacent column*

Note: N_c^+ denotes compression; N_c^- denotes tension; Q_c, M_c follow the local element force convention adopted in OpenSees.

5.1.2. Shear force (Q_c) response

The shear-force histories of the adjacent column exhibit comparatively limited sensitivity to corner-column loss. As illustrated in Figure 3 and Table 2, the intact frame develops a maximum shear force of approximately 95 kN, while the corresponding value in the damaged configuration increases to about 115 kN. Although a noticeable increase is observed after column removal, the magnitude of change remains modest relative to the axial-force response.

The evolution of the shear histories indicates that the redistribution caused by the missing corner column modifies the local force transfer within the surrounding frame region, resulting in slightly higher shear demand in the adjacent column. Nevertheless, the overall trend suggests that shear effects play a secondary role in the post-damage response. The structural behavior is influenced more strongly by the redistribution of axial compression through the three-dimensional framing system than by a substantial increase in shear transfer.

5.1.3. Bending moment (M_c) response

A moderate increase in bending moment is also observed in the column adjacent to the removed member. According to Figure 3 and Table 2, the peak bending moment rises from approximately 160 kNm in the intact frame to approximately 195 kNm after corner-column loss. Compared with the significant increase in axial-force demand, the change in flexural response remains relatively limited.

This trend indicates that the adjacent column continues to participate in resisting lateral and gravity-induced actions through flexural behavior; however, bending effects do not dominate the redistribution mechanism after local damage. Instead, the response of the damaged frame is governed primarily by the increase in axial compressive demand associated with the development of alternative load-transfer paths and spatial interaction among neighboring structural members.

5.1.4. Implications for progressive collapse resistance

The response indicates that the column adjacent to the removed column serves as the primary load-redistributing element. After column loss, axial force increases dramatically, while shear and bending demands exhibit moderate increases due to internal force redistribution. This suggests that PC resistance is governed mainly by axial force redistribution, with shear and bending providing supplementary resistance within the global frame mechanism. The observed development of catenary action further highlights the importance of beam continuity and ductile detailing to maintain stable alternative load paths after local damage.

5.2. Internal force evolution in the beam above the removed column

5.2.1. Axial force (N_b) response

The axial-force histories of the beam above the removed corner column differ substantially between the intact and damaged configurations (Figure 4 and Table 2). In the intact frame, the beam develops only a small compressive axial force during the seismic excitation, with the peak value remaining close to 10 kN. This behavior indicates that the beam response is governed primarily by conventional frame interaction and deformation compatibility under earthquake loading.

After corner-column removal, the axial-force response changes significantly. The beam gradually develops tensile axial force, and the peak tensile demand reaches approximately 80 kN near the end of the ground-motion record. The development of tensile force indicates that the beam begins to participate directly in the redistribution of gravity and seismic loads after the loss of vertical support.

The reduction in flexural demand together with the increase in tensile axial force suggests that the load-transfer mechanism in the damaged bay changes progressively during the analysis. Instead of resisting the redistributed load mainly through bending action, the beam increasingly behaves as an axial tie member connecting the surrounding columns. This behavior is consistent with the development of a catenary-like response in the damaged region and reflects the importance of beam continuity and axial deformation capacity in maintaining post-damage stability.

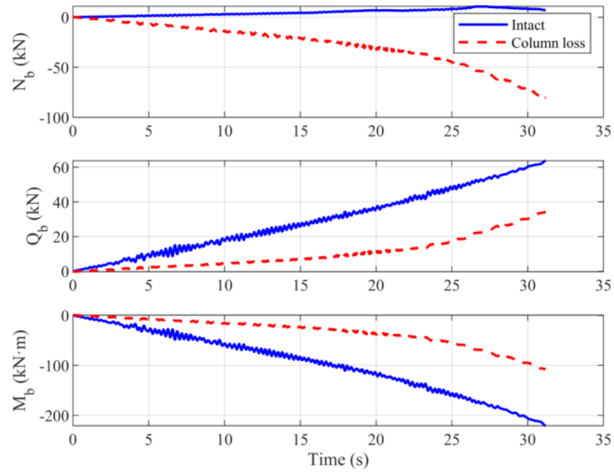

Figure 4. *Internal forces in above beam*

Note: N_b^+ denotes compression; N_b^- denotes tension; Q_b, M_b follow the local element force convention adopted in OpenSees.

5.2.2. Shear force (Q_b) response

The shear response of the beam above the removed corner column shows a different trend from the corresponding axial-force behavior. As presented in Figure 4 and Table 2, the intact frame develops a peak shear force of approximately 60 kN, whereas the maximum value in the column-loss model remains lower, at approximately 35 kN. This reduction suggests that the contribution of shear transfer becomes less significant after the loss of vertical support.

The lower shear demand observed in the damaged configuration is associated with the change in the load-transfer mechanism within the affected bay. As the beam begins to develop tensile axial force and larger deformation compatibility requirements, part of the redistributed load is carried through axial action rather than through conventional flexural–shear resistance. Consequently, the beam response in the column-loss model becomes less dependent on shear transfer and more influenced by the development of axial tension within the damaged structural region.

5.2.3. Bending moment (M_b) response

The bending-moment response of the beam above the removed corner column changes significantly after column loss. As shown in Figure 4 and Table 2, the intact frame develops a continuously increasing negative bending moment during the seismic excitation, reaching a peak value of approximately 210 kNm near the end of the record. In contrast, the column-loss configuration exhibits a substantially lower bending-moment demand, with the peak magnitude remaining close to 110 kNm.

The reduction in bending moment after corner-column loss indicates that the beam no longer resists the redistributed load primarily through conventional flexural action. Instead, part of the load is progressively transferred through axial tensile force as the damaged bay undergoes larger deformation and compatibility demands. This trend is consistent with the simultaneous development of tensile axial force observed in the beam response and suggests a gradual transition from flexural-dominated behavior toward an axial tension-dominated catenary-like mechanism.

Compared with the intact configuration, the beam in the column-loss model therefore experiences a significant reduction in flexural demand, while axial action becomes increasingly important for maintaining load transfer and post-damage stability within the damaged structural region.

5.2.4. Structural implications for progressive collapse resistance

The combined axial–shear–moment response of the beam above the removed column suggests a transition from flexural-dominated behavior toward an axial tension-dominated catenary-like load-transfer mechanism. The substantial tensile axial force demand (up to about 80 kN) underscores the critical importance of axial ductility, connection continuity, and reinforcement anchorage in beams located above potential column-loss scenarios. While shear and bending demands are significantly reduced after column loss, the pronounced increase in axial tension highlights a potential vulnerability if axial capacity or deformation compatibility is insufficient. Therefore, design strategies aimed at improving PC resistance should prioritize robust axial force transfer paths, adequate tensile reinforcement, and detailing capable of sustaining large rotations and elongations without brittle failure.

5.3. Lateral roof displacement response

5.3.1. Lateral roof displacement (Δ)

As shown in Figure 5 and Table 2, the displacement responses of the intact and column-loss frames remain relatively similar during the early stage of the seismic excitation. A pronounced divergence begins at approximately $t \approx 24$–25 s, when the damaged frame starts to develop rapidly increasing lateral drift. The intact structure reaches a peak roof displacement of approximately 280 mm near the end of the record, whereas the column-loss configuration experiences a much larger displacement response, reaching approximately 1050 mm at $t \approx 31$ s.

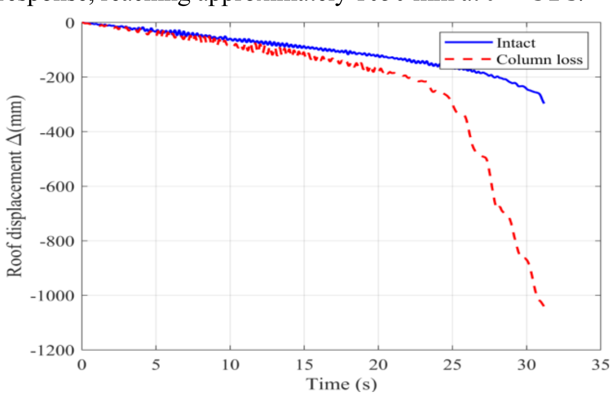

Figure 5. *Lateral roof displacement*

This rapid increase in displacement does not correspond to the instant of column removal, since the corner column has already been removed before seismic excitation begins. Instead, it reflects the progressive deterioration of the effective lateral stiffness of the damaged frame under sustained earthquake loading. The altered load-transfer path, combined with axial-force redistribution, stiffness degradation, and large-deformation compatibility, produces a significantly softer structural response and reduced global stability. These observations are consistent with the previously identified development of tensile axial action in the beam above the removed column and increased compressive demand in adjacent columns.

Table 2. *Summary of peak response amplification caused by column loss*

Structural response	Intact model	Column-loss model	Amplification ratio	Error (%)	Main observation
$N_c\ (kN)$	1000	2250	2.25	125.0	Very large increase in axial demand after column loss
$Q_c\ (kN)$	95	115	1.21	21.1	Moderate increase in shear demand
$M_c\ (kNm)$	160	195	1.22	21.9	Slight-to-moderate increase in flexural demand
$N_b\ (kN)$	10	80	8.00	700.0	Significant redistribution, indicating axial tie/catenary action
$Q_b\ (kN)$	60	35	0.58	-41.7	Shear demand decreases in the damaged configuration
$M_b\ (kNm)$	210	110	0.48	-52.4	Flexural demand decreases as axial mechanism becomes more dominant
Δ (mm)	280	1050	3.75	275.0	Large increase in global displacement due to stiffness degradation and dynamic amplification

5.3.2. Implications for global stability and collapse resistance

The displacement response indicates that corner-column loss significantly reduces the post-damage stability of the structural system under seismic excitation. Compared with the intact frame, the damaged configuration develops substantially larger lateral drift, reflecting severe stiffness degradation and dynamic amplification caused by altered load-transfer paths.

The observed response trends also highlight the importance of considering local damage scenarios in seismic progressive-collapse assessment, since simplified static procedures may not adequately capture spatial force redistribution and large-deformation effects after column loss. The development of tensile axial force in the beam above the removed column further suggests that progressive-collapse resistance depends not only on flexural strength, but also on axial continuity, deformation capacity, and robust alternative load paths. In addition, adjacent columns and beam-column joints should be designed to sustain combined axial, shear, and flexural demands without brittle deterioration. Overall, the results emphasize the need for nonlinear dynamic analysis in evaluating the robustness of three-dimensional frame structures subjected to potential column-loss scenarios.

6. Conclusion

This study investigated the seismic progressive-collapse response of a three-dimensional RC frame with an initially removed first-storey corner column using nonlinear dynamic analysis in OpenSees. The comparison between the intact and initially damaged configurations leads to the following conclusions:

• The column adjacent to the removed corner column experienced a substantial increase in axial-force demand, whereas the corresponding shear and bending moment responses increased only moderately. This behavior indicates that post-damage force redistribution is governed primarily by additional axial compression in the neighboring column.

• The beam above the removed column exhibited a clear axial-force reversal from compression in the intact frame to tension in the damaged frame. The tensile axial-force magnitude became approximately eight times larger than that of the intact configuration, while the bending moment demand decreased by approximately 50%. These trends indicate a transition from flexural-dominated behavior toward an axial tension-dominated catenary-like load-transfer mechanism.

• The initially damaged frame developed significantly larger lateral roof displacement than the intact structure, with the peak displacement increasing by approximately 275%. This amplification reflects the combined influence of stiffness degradation, spatial load redistribution, dynamic amplification, and large-displacement compatibility under seismic excitation.

• From a design perspective, the results highlight the importance of continuous beam reinforcement, adequate anchorage capacity, ductile beam-column joints, adjacent-column checks for increased axial compression and axial–flexural interaction, and diaphragm continuity to facilitate three-dimensional force redistribution. Nonlinear dynamic analysis or equivalent dynamic procedures are recommended for robustness assessment of structures subjected to potential column-loss scenarios.

The study is limited to one ground-motion record and one corner-column-loss scenario. Bond–slip, reinforcement fracture, joint shear failure, slab contribution, and cyclic deterioration were not explicitly modeled. Future work should consider multiple seismic records, additional column-loss locations, detailed deterioration models, slab–frame interaction, and system-level experimental or high-fidelity numerical validation.

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