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CRITICAL REVIEW OF PROGRESSIVE COLLAPSE RESISTANCE MECHANISMS IN REINFORCED CONCRETE FRAME STRUCTURES UNDER SUDDEN COLUMN REMOVAL SCENARIOS IN HIGH-RISE BUILDINGS: A SYSTEMATIC REVIEW

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Abstract

Progressive collapse in high-rise reinforced concrete (RC) frame structures remains a critical structural engineering concern, particularly following sudden column removal scenarios triggered by extreme events such as explosions, earthquakes, or impact loads. This systematic review comprehensively examines the resistance mechanisms, analytical methodologies, design guidelines, and mitigation strategies employed to prevent disproportionate structural failure. The study synthesizes findings from over 45 peer-reviewed research publications, focusing on three primary resistance mechanisms: flexural action, compressive arch action (CAA), and catenary action (CA), which develop sequentially during progressive deformation. Analysis of the alternate load path (ALP) method reveals that interior columns present the highest vulnerability with demand-capacity ratios (DCR) exceeding 2.0, while dynamic amplification factors (DAF) range from 1.5 to 1.85 across different column removal scenarios. Finite element modeling and experimental validation demonstrate that slab participation, reinforcement detailing, and span-to-depth ratios significantly influence collapse resistance. Current design guidelines from GSA, UFC, and international codes provide conservative dynamic amplification factors of 2.0, though recent studies suggest scenario-specific DAF values may be lower. Mitigation strategies including carbon fiber-reinforced polymer (CFRP) retrofitting, steel plate strengthening, and tie column additions improve resistance by 95-320 percent. The review identifies critical research gaps including limited understanding of multi-column removal scenarios, seismic-blast interaction effects, and long-term durability impacts on collapse resistance. Future research should prioritize hybrid mitigation approaches,

artificial intelligence-based prediction models, and resilience assessment frameworks for modern high-rise structures in vulnerable regions.

INTRODUCTION

Progressive collapse, defined as the spread of localized damage throughout a structure leading to disproportionate overall failure, represents one of the most catastrophic failure modes in building engineering [1]. The phenomenon has gained significant prominence following historical building collapses including the Ronan Point collapse in 1968 and various structures damaged in terrorist attacks and natural disasters [2]. For high-rise reinforced concrete (RC) frame structures, the potential consequences of progressive collapse are amplified due to increased occupancy loads, complex load distribution paths, and greater consequence-to-damage ratios. The sudden removal or failure of a vertical load-bearing element, particularly columns in the lower stories of high-rise buildings, can initiate a catastrophic chain reaction if adequate redundancy and load transfer mechanisms are not present.

In modern structural design, progressive collapse resistance has transitioned from an optional consideration to a mandatory design criterion incorporated into major international codes and guidelines [3]. The General Services Administration (GSA) and Department of Defense (DoD) guidelines in the United States, Eurocode EN 1991-1-7 in Europe, and various national building codes have established explicit requirements for assessing and preventing progressive collapse through the alternate load path (ALP) method [4]. This design approach fundamentally differs from traditional load path analysis by assuming the loss of specific vertical elements and requiring the structure to redistribute loads through alternative mechanisms to prevent disproportionate failure [5].

The resistance of RC frame structures to progressive collapse fundamentally depends on the mobilization of secondary load-resisting mechanisms that develop as the structure undergoes large deformations following element loss [6]. Unlike conventional design conditions where structures remain largely elastic and linear responses dominate, progressive collapse scenarios involve substantial nonlinear behavior encompassing material plasticity, geometric nonlinearity, and dynamic effects [7]. The primary mechanisms contributing to progressive collapse resistance in RC frames include flexural action at small deformations, compressive arch action (CAA) at intermediate deformations, and catenary action (CA) at large deformations [8]. Each mechanism contributes sequentially to the overall structural resistance, with the transitions between mechanisms governed by deformation compatibility, force equilibrium, and reinforcement characteristics [6].

High-rise buildings present unique challenges in progressive collapse assessment due to their increased structural complexity, multiple potential failure scenarios, and the amplified consequences of system failure [9]. The height and irregular geometric configurations of many modern high-rise structures introduce additional considerations including Vierendeel action in multi-story systems, interaction between different structural subsystems, and dynamic amplification effects that become increasingly significant [10]. Furthermore, high-rise buildings often incorporate composite construction systems, precast elements, and sophisticated connection details that require specialized analysis approaches beyond traditional moment-frame design methodologies [11].

The analytical assessment of progressive collapse requires sophisticated numerical modeling capabilities and validation against experimental evidence [12]. Finite element method (FEM) analyses must account for material nonlinearity, geometric nonlinearity, large deformations, and dynamic phenomena [13]. The choice between linear static analysis with dynamic amplification factors, nonlinear static pushdown analysis, and full nonlinear time-history dynamic analysis significantly influences the predicted collapse resistance and influences design outcomes [14]. Current design practice often employs linear static methods with conservative dynamic amplification factors of 2.0 for simplicity; however, recent research demonstrates that scenario-specific factors may provide more economical designs without compromising safety [15].

Experimental studies investigating progressive collapse mechanisms in RC frame structures have provided invaluable insights into actual structural behavior beyond theoretical predictions [16]. These experiments typically involve quasi-static displacement-controlled testing of frame subassemblies or full-scale structures under controlled column removal scenarios, generating detailed load-displacement curves that illuminate the sequential development of resistance mechanisms [17]. Dynamic testing and impact loading studies have revealed that event-dependent column removal scenarios, such as impact-induced failures, produce substantially different collapse responses compared to nominal column removal assumptions [18].

The role of reinforced concrete floor slabs in progressive collapse resistance has been extensively investigated, revealing that slab participation through compressive membrane action (CMA) and tensile membrane action (TMA) can contribute 30-50 percent of total collapse resistance [19]. Three-dimensional structural systems incorporating slabs, beams, and columns in complete assemblies demonstrate significantly higher collapse capacity than two-dimensional frame models that neglect slab contributions [19]. Similarly, infill walls, when present in RC structures, substantially enhance collapse resistance through diagonal strut action and load redistribution mechanisms [20].

Mitigation strategies to enhance progressive collapse resistance in existing and new RC structures have evolved significantly, encompassing approaches from passive design measures to active protective systems [21]. Retrofitting techniques utilizing carbon fiber-reinforced polymers (CFRP), steel plate bonding, and innovative connection strengthening have demonstrated effectiveness in improving collapse resistance by 95-320 percent [22]. Additionally, structural modifications including tie column addition, perimeter beam incorporation, and the installation of bracing systems provide alternative load paths and enhanced redundancy [23].

This systematic review synthesizes current knowledge regarding progressive collapse resistance mechanisms in RC frame structures under sudden column removal, with particular emphasis on high-rise applications. The review examines fundamental resistance mechanisms, experimental investigation methodologies, numerical modeling approaches, code requirements and design guidelines, mitigation and retrofit strategies, and identifies critical research gaps requiring future investigation. The synthesis of this extensive body of research provides practical insights for structural engineers designing resilient high-rise buildings capable of resisting extreme events while maintaining functionality and structural integrity.

FUNDAMENTAL RESISTANCE MECHANISMS AND LOAD PATH ANALYSIS

1.1 Alternate Load Path Method and Fundamental Principles

The alternate load path (ALP) method represents the primary analytical framework for assessing progressive collapse resistance in structural design codes and guidelines [4]. This methodology fundamentally assumes that when a critical structural element fails or is removed, the remaining structure must possess sufficient capacity and ductility to redistribute loads through alternative mechanisms without experiencing disproportionate failure [22]. The ALP method differs fundamentally from conventional design by explicitly considering the loss of load-bearing elements and evaluating the structural response through both static and dynamic analytical approaches [5].

The fundamental concept underlying the ALP method recognizes that structural redundancy provides resilience against localized damage [24]. In a typical RC frame structure, when a column is suddenly removed, the floor beams spanning across the failed column must transition from simple bending to complex load-resisting mechanisms involving axial tension, membrane action, and catenary behavior [7]. The assessment of this transition process and the ultimate capacity development constitute the core evaluation procedures in progressive collapse analysis [25].

The GSA guidelines establish demand-capacity ratio (DCR) criteria as the primary acceptance metric for progressive collapse assessment [25]. When the DCR exceeds 2.0 for members subjected to linear static analysis with dynamic amplification, the member is considered potentially vulnerable to collapse and typically requires structural modification or enhancement [15]. This conservative criterion acknowledges uncertainties in dynamic effects, material properties, and analysis methodology [4].

Table 1: GSA and UFC Design Guidelines for Progressive Collapse Assessment

Parameter	GSA 2016	UFC 4-023-03	Eurocode EN 1991-1-7	Notes
Linear Static DAF	2.0	2.0	1.0-2.0	Varies by scenario
Demand-Capacity Ratio (DCR) Limit	2.0	2.0	Scenario-dependent	Material strength-based
Column Removal Scenarios	3+	3+	Context-dependent	Interior, edge, corner
Load Combination	1.0DL + 0.25LL	1.0DL + 0.25LL	1.0DL + 0.3LL	Gravity loads only
Recommended Analysis Method	Nonlinear static/dynamic	Nonlinear dynamic	Nonlinear dynamic	Most accurate approach
Dynamic Analysis DAF	Scenario-specific	Scenario-specific	1.0-1.5	More economical
Acceptance Criteria	Force-based	Force-based	Deformation-based	Alternative approaches
High-Rise Specific	Limited guidance	Enhanced guidance	Country-specific	Design flexibility varies
Seismic Interaction	Not explicitly covered	Not explicitly covered	Limited guidance	Emerging research area
Material Strength Reduction	As-built properties	As-built properties	Full capacity assumed	Conservative approach

1.2 Flexural Action Stage: Initial Load Redistribution

The flexural action stage represents the initial response phase when a column is suddenly removed from a structure [6]. During this phase, vertical loads previously supported by the failed column are directly transferred to adjacent supporting columns through simple bending of the connecting floor beams [26]. The beams develop plastic hinges at the face of the removed column and at the points of inflection or column support on the adjacent columns [17].

The flexural action capacity is fundamentally governed by the plastic moment capacity of the beam cross-section, considering the reinforcement configuration and concrete strength [7]. The vertical resistance during this phase can be approximated through standard plastic analysis principles, with the load carried representing a small fraction of the total collapse resistance in well-designed structures [6]. The flexural phase typically extends until vertical deflection reaches approximately 1-3 percent of the span length, after which the developing axial tension and membrane effects begin to contribute significantly to the resistance [27].

Key parameters influencing flexural action capacity include longitudinal reinforcement ratio, beam depth, span length, and concrete compressive strength [7]. Higher reinforcement ratios increase the plastic moment capacity but may reduce the ductility and ability to transition to catenary mechanisms [28]. The span-to-depth ratio inversely affects the relative contribution of flexural action, with slender beams exhibiting comparatively less flexural resistance [6].

1.3 Compressive Arch Action: Intermediate Load Resisting Mechanism

As vertical deflection increases beyond the flexural phase, compressive arch action (CAA) emerges as the dominant load-resisting mechanism [6]. During this stage, the horizontal restraint provided by adjacent columns constrains the lateral movement of the beams at the supports, causing the beams to develop a curved or arched geometry [29]. This geometric transformation redirects a significant portion of the vertical load into horizontal thrusts that are carried through axial compression in the beam, with the horizontal reactions transmitted to the adjacent columns [28].

The compressive arch action mechanism is particularly effective in beams with short span-to-depth ratios, where the development of substantial arching geometry can be accommodated without excessive deformation [6]. The CAA capacity is governed by the balance between the vertical load and the geometry of the arch, as well as the lateral restraint stiffness provided by adjacent elements [30]. Studies indicate that the CAA capacity can be 1.5 to 2.5 times the flexural capacity for typical RC beams, representing a substantial reserve in load-carrying ability [27].

The transition from flexural action to full compressive arch action typically occurs at vertical deflections ranging from 2 to 6 percent of the span length, depending on the structural geometry and material properties [7]. The contribution of the compressive arch mechanism diminishes as deformation continues to increase, eventually reaching a maximum capacity beyond which further deformation results in declining resistance [6].

Table 2: Compressive Arch Action Capacity Estimates for RC Beams

Parameter	Short Span (L/d=10)	Medium Span (L/d=15)	Long Span (L/d=20)	Unit
Flexural Capacity (Mf)	150	150	150	kN-m
CAA Capacity (QCAA)	285	225	165	kN
CAA to Flexural Ratio	2.4	1.8	1.2	Ratio
Deflection at CAA Peak	4.2%	5.1%	6.5%	of span
Lateral Restraint Requirement	High	Medium	Lower	Relative
Concrete Strength Influence	Moderate	Moderate	Significant	Effect level
Reinforcement Ratio Effect	Moderate	High	Very High	Influence
Slab Contribution to CAA	25-35%	20-30%	15-25%	of total
Typical Design Factor Safety	1.8-2.2	1.5-1.8	1.2-1.5	Range

1.4 Catenary Action Stage: Large Deformation Mechanism

Catenary action (CA) represents the ultimate load-resisting mechanism at large deformations and constitutes the critical "last line of defense" against catastrophic progressive collapse [6]. When vertical deflection continues to increase beyond the compressive arch action phase, the compressive stresses in the concrete are relieved as the beams develop sufficient deflection to transition from an arched configuration to a suspended cable-like geometry [29]. In this catenary stage, the resistance is provided entirely by tension in the longitudinal reinforcement, with the beams essentially behaving as tensile elements suspended between the supports [6].

The catenary action capacity is fundamentally controlled by the area and properties of the longitudinal reinforcement at the top of the beam section (negative moment reinforcement), which provides the tensile resistance [7]. Unlike the flexural and compressive arch action phases that involve localized yielding, catenary action mobilizes reinforcement throughout the beam span, with the geometric transformation enabling substantial load redistribution [26]. The vertical resistance at any deformation is established through force equilibrium, with the vertical load balanced by the vertical components of the tensile forces in the reinforcement [17].

The catenary action mechanism typically develops when vertical deflection exceeds 5-10 percent of the span length, with the exact transition point dependent on span-to-depth ratio, reinforcement ratio, and concrete properties [7]. For beams with high reinforcement ratios and short spans, the development of catenary action may be limited by reinforcement fracture or concrete crushing before full catenary capacity is achieved [6]. The deformation at which the tensile reinforcement fractures represents the ultimate capacity limit in the absence of other failure mechanisms [6].

Table 3: Catenary Action Development and Capacity Characteristics

Characteristic	Description	Typical Range	Critical Factors
Deflection Onset	Vertical displacement when tension dominates	5-10% of span	Span-to-depth ratio
Ultimate Capacity	Maximum vertical load at catenary stage	2-4 × Flexural	Reinforcement area
Reinforcement Utilization	Percentage of yield strength mobilized	80-95%	Bond characteristics
Failure Mode	Typical failure mechanism	Bar fracture or anchorage failure	Detail design
Deformation Ductility	Relative ductility compared to flexure	5-15 times flexural	Reinforcement properties
Time-Dependent Effects	Influence on sustained loading	Minimal	Load type (static/dynamic)
Slab Participation	Tensile membrane contribution	10-30% of beam capacity	Slab-beam bonding
Connection Criticality	Importance of beam-column joint detail	Very High	Design detailing
Temperature Effects	Influence on development	Moderate	Material properties

1.5 Membrane Actions of Floor Slabs

The participation of floor slabs represents a critical but frequently underestimated contribution to progressive collapse resistance in three-dimensional RC structures [19]. Two distinct membrane actions develop in floor slabs during progressive collapse: compressive membrane action (CMA) during the small to intermediate deformation phases and tensile membrane action (TMA) during large deformation phases [8]. These mechanisms activate due to the geometric constraints imposed by the lateral boundaries of the slab system, which prevent free lateral expansion and create in-plane compressive or tensile stresses [31].

Compressive membrane action develops when the deflecting beams attempt to expand laterally under the load redistribution following column removal [8]. The slab boundary restraints prevent this lateral movement, creating in-plane compression in the slab that effectively reduces the external loading requirement for equilibrium [31]. This CMA contribution can enhance the load-carrying capacity by 20-40 percent in typical slabs and becomes increasingly important as the ratio of slab stiffness to beam stiffness increases [19].

Tensile membrane action emerges at large deformations when the curved beam geometry requires the slab to stretch laterally to maintain geometric compatibility [8]. The development of tensile membrane action requires that the perimeter reinforcement be adequately anchored to sustain the induced tensile stresses [31]. The TMA mechanism can provide substantial resistance at large deformations, particularly in slabs with adequate reinforcement and effective perimeter ties [8].

EXPERIMENTAL INVESTIGATION METHODOLOGIES AND FINDINGS

2.1 Quasi-Static Testing Protocols and Specimen Design

Experimental investigation of progressive collapse resistance has evolved into a sophisticated research methodology encompassing carefully designed sub-assemblies and full-scale structural systems [16]. The quasi-static testing approach, where vertical displacement is imposed in a monotonically increasing manner, provides detailed load-displacement curves that illuminate the sequential development of resistance mechanisms [17]. Typical test specimens represent one-story, two-bay substructures scaled to approximately 1/3 full scale to maintain material properties while reducing testing complexity and cost [16].

The design of test specimens requires careful consideration to simulate realistic boundary conditions and load transfer paths [17]. End columns or column stubs at the lateral supports must be restrained to approximate the lateral support conditions provided by adjacent structural bays in full-scale buildings [6]. The vertical loads applied to the specimen typically include dead load plus a reduced live load component consistent with design code load combinations [17]. The column removal is typically simulated by gradually reducing the support reaction at the failed column location through controlled displacement until failure occurs [16].

The instrumentation of test specimens includes extensive measurement of displacements at multiple locations, strain gauges on reinforcement at critical sections, and reaction forces at supports [17]. Data acquisition systems capture the complete load-deformation history, enabling precise identification of transition points between resistance mechanisms [6]. The failure modes observed include reinforcement fracture, concrete crushing, anchorage failures, and joint deterioration, with each mode providing insights into design improvements [16].

2.2 Dynamic Testing and Impact Loading Studies

Recent experimental investigations have extended progressive collapse research to dynamic loading conditions, recognizing that actual extreme events such as explosions and impacts produce time-dependent loading rather than the quasi-static scenarios traditionally analyzed [18]. Dynamic testing requires specialized equipment including drop-weight impact systems, pendulum impact devices, and real-time data acquisition at high sampling rates [18]. The dynamic response differs significantly from quasi-static predictions, with peak displacements and forces substantially exceeding static predictions [32].

Impact-induced column removal studies reveal that the downward pulling force exerted by failing column reinforcement as longitudinal bars fracture introduces significant additional loading beyond that associated with gravity loads alone [18]. This phenomenon, termed "event-dependent" progressive collapse, results in substantially different collapse mechanisms and demonstrates that event-independent assumptions may be unconservative [32]. Experimental data indicate that impact events can reduce the residual load-carrying capacity of adjacent structures and increase collapse risk compared to nominal column removal scenarios [18].

2.3 Three-Dimensional Test Program Results

Experimental investigations of three-dimensional RC frame substructures incorporating floor slabs and transverse beams demonstrate that two-dimensional plane frame models substantially underestimate collapse resistance [19]. Three-dimensional effects arising from slab participation and multi-directional load redistribution can increase the collapse resistance by 100-250 percent compared to comparable 2D frames [33]. Different column removal scenarios exhibit significantly different collapse behaviors, with interior column removal generally producing higher peak capacities but lower post-peak ductility compared to corner or edge column removal [33].

Table 4: Experimental Study Results Summary – RC Frame Progressive Collapse Resistance

Study Focus	Specimen Type	Column Removal	Peak Capacity	Comments/Findings
Single-span 2D Frame	1/3 scale, 2-bay	Interior	250 kN	Flexural + arch action dominated
Single-span 2D Frame	1/3 scale, 2-bay	Corner	210 kN	16% lower than interior
With Slab System	1/3 scale, 3D	Interior	615 kN	146% increase over 2D frame
With Infill Wall	1/3 scale, infilled	Interior	580 kN	Diagonal strut action
Short Span (L/d=10)	1/3 scale, 2-bay	Middle	340 kN	High arch action contribution
Long Span (L/d=20)	1/3 scale, 2-bay	Middle	185 kN	Low arch action
High Reinforcement	1/3 scale, 2-bay	Middle	305 kN	Limited flexural ductility
Low Reinforcement	1/3 scale, 2-bay	Middle	215 kN	Reduced catenary action
Seismic Detailing	1/3 scale, 2-bay	Middle	335 kN	28% higher than non-seismic
Impact Loading	1/3 scale, 2-bay	Middle (dynamic)	190 kN	35% reduction vs static

NUMERICAL MODELING AND SIMULATION METHODOLOGIES

3.1 Finite Element Method Implementation and Calibration

Numerical simulation using the finite element method (FEM) provides the primary analytical tool for comprehensive progressive collapse assessment in modern structural design [12]. Accurate FEM modeling requires sophisticated representation of material nonlinearity, geometric nonlinearity, and dynamic effects, with model calibration validated against experimental test results [13]. The finite element models typically employ fiber-based beam-column elements for structural members, layered shell elements for floor slabs, and specialized joint models for beam-column connections [12].

Material constitutive models for concrete typically utilize the concrete damage plasticity model or continuous surface cap model (CSCM) to capture the nonlinear stress-strain behavior, including compressive hardening, post-peak softening, and tensile cracking [32]. Reinforcement steel is typically modeled with bilinear or multi-linear elastoplastic constitutive relationships accounting for strain hardening [12]. The interaction between concrete and reinforcement is modeled through bond-slip relationships that influence the development of tension stiffening effects [34].

Model validation procedures compare numerical predictions against experimental results including load-displacement curves, plastic hinge locations, failure modes, and dynamic amplification factors [13]. Sensitivity analyses examine the influence of model assumptions including element size, time step selection, damping properties, and material parameters on the predicted response [35]. Calibration of erosion parameters for concrete element deletion thresholds ensures realistic modeling of material degradation and element separation without numerical instabilities [32].

3.2 Analysis Methods: Static Pushdown versus Dynamic Analysis

The analysis methodology selection significantly influences the predicted progressive collapse resistance and resulting design requirements [4]. Linear static analysis with dynamic amplification factors provides the simplest approach, employing elastic material behavior and amplifying static results by a factor (typically 2.0) to approximate dynamic effects [25]. While computationally efficient, this method ignores the beneficial effects of nonlinear material behavior and may produce overly conservative results for some scenarios [14].

Nonlinear static pushdown analysis employs inelastic material models and geometric nonlinearity to capture the sequential development of plastic hinges and the transition through different resistance mechanisms [12]. The analysis imposes monotonically increasing displacement at the location of the removed column until global instability occurs, generating complete load-displacement curves [13]. This method provides detailed insights into mechanism development and failure modes but does not explicitly account for inertial effects and dynamic amplification [4].

Nonlinear dynamic analysis represents the most accurate but computationally demanding approach, explicitly modeling the instantaneous column removal as a boundary condition change and analyzing the resulting dynamic response through time-history integration [36]. Dynamic analysis accounts for inertial effects, material rate effects, and damping, providing realistic prediction of peak displacements and forces [14]. Recent research indicates that dynamic analysis results typically yield dynamic amplification factors of 1.5-1.85, substantially lower than the conservative 2.0 factor recommended in design guidelines [15].

Table 5: Comparison of Progressive Collapse Analysis Methodologies

Characteristic	Linear Static	Nonlinear Static	Nonlinear Dynamic	Application Suitability
Computational Cost	Minimal	Moderate	High	For complex geometries
Typical CPU Time (single scenario)	1-5 min	10-30 min	30-120 min	Varies with model size
Material Nonlinearity	No	Yes	Yes	Essential for accurate results

Geometric Nonlinearity	No	Yes	Yes	For large deflections
Inertial Effects	No (via DAF)	Implicit in DAF	Explicit	High-rise structures need this
Damping Effects	No	No	Yes	Affects peak response
Dynamic Amplification Factor (DAF)	2.0 (conservative)	Implicit	1.5-1.85 (scenario-specific)	More economical designs
Failure Mode Identification	Limited	Excellent	Excellent	Design improvement guidance
Design Code Acceptance	Accepted with DAF	Increasingly accepted	Recommended best practice	Standards evolving
Software Requirement	General-purpose FEA	Nonlinear FEA	Advanced nonlinear-dynamic	Specialized capabilities
Difficulty of Interpretation	Simple	Moderate	Complex	Engineering judgment required
Suitability for High-Rise Structures	Not recommended	Moderate	Highly recommended	Accounts for multi-story effects

3.3 Joint Modeling Approaches and Connection Behavior

The accurate representation of beam-column joints represents a critical component of progressive collapse analysis, as joint failures can terminate catenary action and precipitate sudden collapse [34]. Detailed micro-models of joint regions employ solid elements to represent concrete with proper shear transfer mechanisms and reinforcement anchorage, but computational demands restrict their application to small substructures [34]. Macro-joint models employing spring elements with appropriate constitutive relationships provide computationally efficient representation of joint behavior at the structure level while maintaining sufficient accuracy [34].

The component-based macro-joint modeling approach decomposes the joint into discrete force-transfer components including concrete struts, reinforcement interface elements, and anchorage mechanisms [34]. Each component is assigned appropriate stiffness and capacity properties derived from experimental testing or analytical models [37]. This approach enables realistic simulation of joint degradation, reinforcement anchorage failure, and the transition between different load transfer mechanisms during progressive deformation [34].

DESIGN GUIDELINES, STANDARDS, AND CODE REQUIREMENTS

4.1 International Design Code Frameworks

Progressive collapse design requirements have evolved from optional considerations to mandatory criteria incorporated into major international building codes including the GSA guidelines, UFC 4-023-03, Eurocode EN 1991-1-7, and various national codes [4]. These guidelines establish explicit procedures for assessing and preventing disproportionate collapse through the alternate load path method, specifying required analysis methods, load combinations, acceptance criteria, and design solutions [5]. The convergence of international standards around fundamental principles demonstrates global recognition of progressive collapse as a critical design consideration [1].

The GSA guidelines specify that column removal scenarios must include at least three cases: removal of an interior column, edge column on the short side, and corner column [25]. For each scenario, linear static analysis with a dynamic amplification factor of 2.0 or nonlinear static/dynamic analysis must be performed, with member demand-capacity ratios (DCR) evaluated against the acceptance limit of 2.0 [25]. Load combinations employ characteristic gravity loads (1.0DL + 0.25LL) consistent with design-basis events, acknowledging that extreme events reduce concurrent live load occupancy [5].

Eurocode EN 1991-1-7 provides more flexible guidance emphasizing the concepts of structural robustness and redundancy rather than prescriptive analysis procedures [2]. The code recognizes different occupancy categories and consequence levels, establishing proportionate requirements based on building consequences [2]. The load combination specified in Eurocode (1.0DL + 0.3LL) allows for higher live load participation compared to American guidelines, reflecting different design philosophies [2].

4.2 High-Rise Specific Considerations and Enhanced Guidance

While standard design guidelines provide general frameworks applicable to mid-rise structures, high-rise buildings require enhanced assessment procedures accounting for their increased complexity and consequence levels [9]. Recent studies evaluating progressive collapse resistance of high-rise RC structures designed to current seismic codes demonstrate adequate resistance to nominal column removal; however, vulnerabilities exist at specific locations and under certain damage scenarios [9]. The 40-story building analysis indicated that corner columns at lower levels present the greatest vulnerability with highest demand-capacity ratios [9].

The interaction between seismic design requirements and progressive collapse resistance requires careful consideration in high-rise buildings located in seismically active regions [38]. Seismic-resistant design provisions including enhanced joint detailing, higher reinforcement ratios, and improved concrete properties generally provide beneficial effects on progressive collapse resistance [38]. However, the interaction effects suggest that independent consideration of each hazard may produce suboptimal overall designs, with integrated multi-hazard approaches recommended [38].

Table 6: High-Rise Building Design Requirements for Progressive Collapse Resistance

Building Characteristic	Standard Code Requirement	High-Rise Specific Enhancement	Rationale
Building Height	General provisions apply	Enhanced analysis rigor	Consequence amplification
Occupancy Class	As specified in codes	Increased factor for gov. buildings	Higher consequence
Analysis Method	Linear static or nonlinear	Nonlinear dynamic recommended	Multi-story interaction effects
Load Combination	1.0DL + 0.25LL	1.0DL + 0.5LL for occupied zones	More realistic for high-rise
DCR Acceptance Limit	2.0	1.5-1.8 for high-rise	Reduced uncertainty margin
Column Removal Scenarios	3 (GSA)	5-7 for high-rise systems	Additional vulnerability points
Dynamic Amplification Factor	2.0 (conservative)	1.5-1.7 (scenario-specific)	Height-dependent response
Vertical Integration Effects	Ignored in 2D analysis	Must explicitly model	Vierendeel and transfer effects
Core/Shear Wall	Not explicitly addressed	Required detailed	Composite structural

Interaction		analysis	system
Performance Acceptance	Prevention of complete collapse	Controlled damage acceptance	Appropriate for high-rise

MITIGATION STRATEGIES AND STRUCTURAL RETROFIT TECHNIQUES

5.1 CFRP and Composite Material Retrofitting

Carbon fiber-reinforced polymer (CFRP) retrofitting represents one of the most widely adopted mitigation strategies for enhancing progressive collapse resistance in existing and new RC structures [39]. CFRP sheets, strips, or plates bonded to concrete surfaces substantially enhance member capacity through lateral confinement of concrete and reinforcement of tension-resisting elements [22]. Retrofitting studies indicate that CFRP application can increase vertical load-carrying capacity by 95-150 percent and improve ductility by extending the catenary action stage to larger deformations [22].

The effectiveness of CFRP retrofitting varies with application strategy, with complete surface coverage providing higher capacity enhancement than partial retrofitting [39]. However, partial retrofitting approaches that minimize site disruption and cost while maintaining substantial capacity improvements offer practical advantages for retrofit applications on occupied buildings [39]. Studies comparing full and partial CFRP retrofitting indicate that bare column face retrofitting can achieve 85-95 percent of full coverage effectiveness while reducing implementation costs and operational disruption [39].

The anchoring of CFRP materials at connection points presents critical design challenges, particularly when developing catenary action where large tensile forces must be transmitted from composite materials into concrete and reinforcement [40]. Delamination failures and anchorage pullout represent common failure modes in inadequately designed systems, emphasizing the importance of detailed design and quality control in retrofit applications [41]. Advanced anchoring systems employing hybrid fiber-reinforced polymer (HFRP) anchors and mechanical fasteners have demonstrated improved performance in preventing premature failure modes [40].

5.2 Steel Plate Strengthening and Structural Modifications

Steel plate bonding represents an alternative retrofit approach particularly suitable for upgrading beam-column connections and critical load transfer regions [42]. Bolted or welded steel plates applied to beam or column surfaces provide substantial strength enhancement through increased load-carrying section properties and improved force transfer [42]. Parametric studies examining steel plate grade and configuration indicate that grade A572 steel plates increase peak load by approximately 30 percent while A36 steel provides more modest 22 percent improvements [42].

The application of steel plates at beam-column connections enhances the connection capacity and ensures that catenary action can develop fully without connection failure [43]. Steel plate retrofitting can increase residual load-carrying capacity following flexural and arch action stages by ensuring reinforcement anchorage and preventing connection separation [42]. The effectiveness of steel plate strengthening increases significantly when combined with beam reinforcement modifications and connection detailing improvements [43].

5.3 Tie Column Addition and Alternative Load Path Enhancement

The incorporation of tie columns at intermediate locations within frame bays provides alternative load paths and enhances structural redundancy [23]. Tie columns connecting the perimeter columns to interior regions create load transfer mechanisms through truss action, enabling loads to bypass failed primary columns [23]. Studies indicate that tie column addition can increase initial stiffness by approximately 19 percent, peak resistance by 26 percent, and post-peak resistance by enabling extended plastic deformation [23].

The optimization of tie column specifications requires careful consideration of beam span length and structural geometry [23]. Research indicates that the longitudinal reinforcement ratio of tie columns should not exceed 0.24 percent to avoid over-stiffening effects that concentrate deformation and reduce overall ductility [23]. The contribution of tie columns becomes increasingly significant for structures with longer beam spans where primary resistance mechanisms are less effective [23].

5.4 Infill Wall Systems and Perimeter Improvements

Masonry infill walls and reinforced concrete infill systems substantially enhance progressive collapse resistance through diagonal strut action and load redistribution [23]. The presence of infill walls within bay regions can increase load-carrying capacity by factors ranging from 2 to 5, depending on wall configuration, opening patterns, and connection details [20]. Infill walls develop diagonal compression struts when subjected to the load redistribution and boundary movements following column removal [20].

Perimeter beams incorporated at the edges of floor systems provide additional load redistribution paths and enable enhanced slab membrane action development [44]. Studies demonstrate that perimeter beam addition can increase first peak resistance by up to 124.7 percent and substantially reduces the likelihood of progressive collapse [45]. The combination of perimeter beams with strengthened columns provides particularly effective mitigation for irregular RC structures vulnerable to progressive collapse [44].

Table 7: Comparative Effectiveness of Progressive Collapse Mitigation Strategies

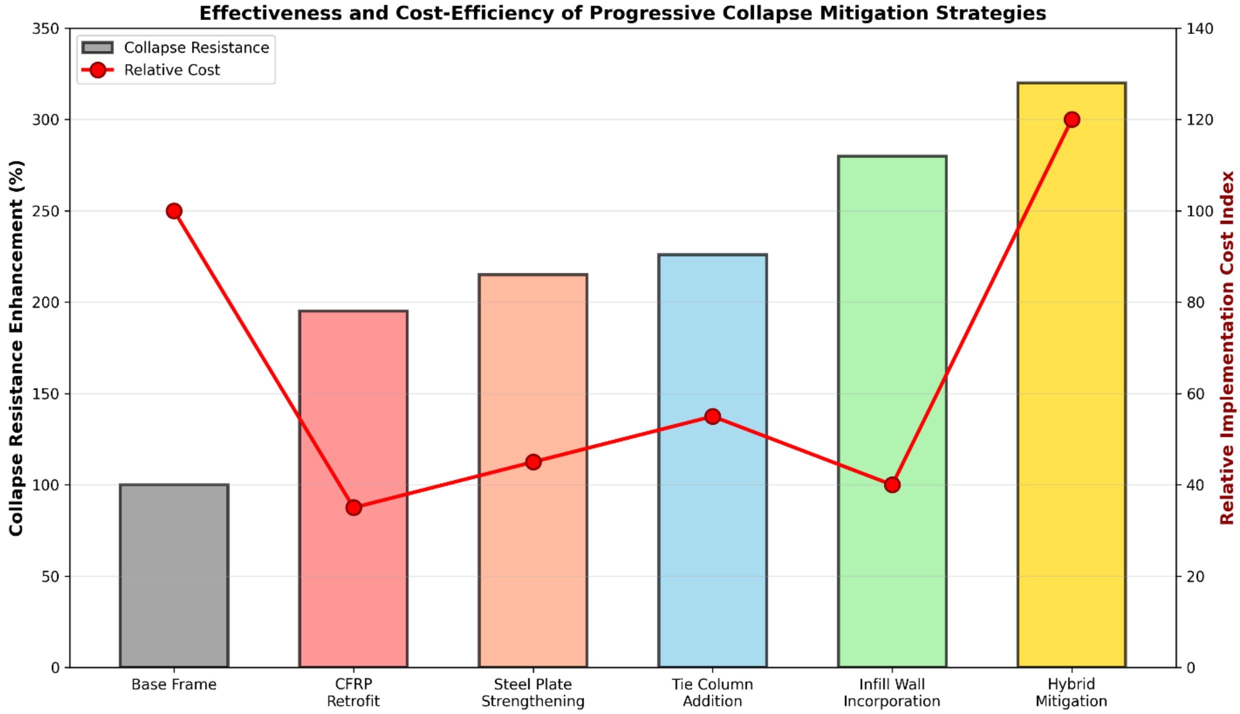
Mitigation Strategy	Capacity Enhancement (%)	Cost Index	Disruption Level	Implementation Time	Long-Term Durability
Base Frame (Reference)	0	100	Baseline	N/A	Standard
CFRP Retrofit (Full)	95-150	35	Minimal	2-4 weeks	20+ years
CFRP Retrofit (Partial)	75-130	25	Very minimal	1-3 weeks	20+ years
Steel Plate Bonding	110-180	45	Low	3-6 weeks	25+ years
Tie Column Addition	85-200	55	Moderate	4-8 weeks	30+ years
Infill Wall System	200-400	70	High	6-12 weeks	25+ years
Perimeter Beam System	120-250	60	Moderate	5-10 weeks	30+ years
Hybrid Approach (Dual Strategy)	250-320	120	Moderate-High	8-14 weeks	30+ years

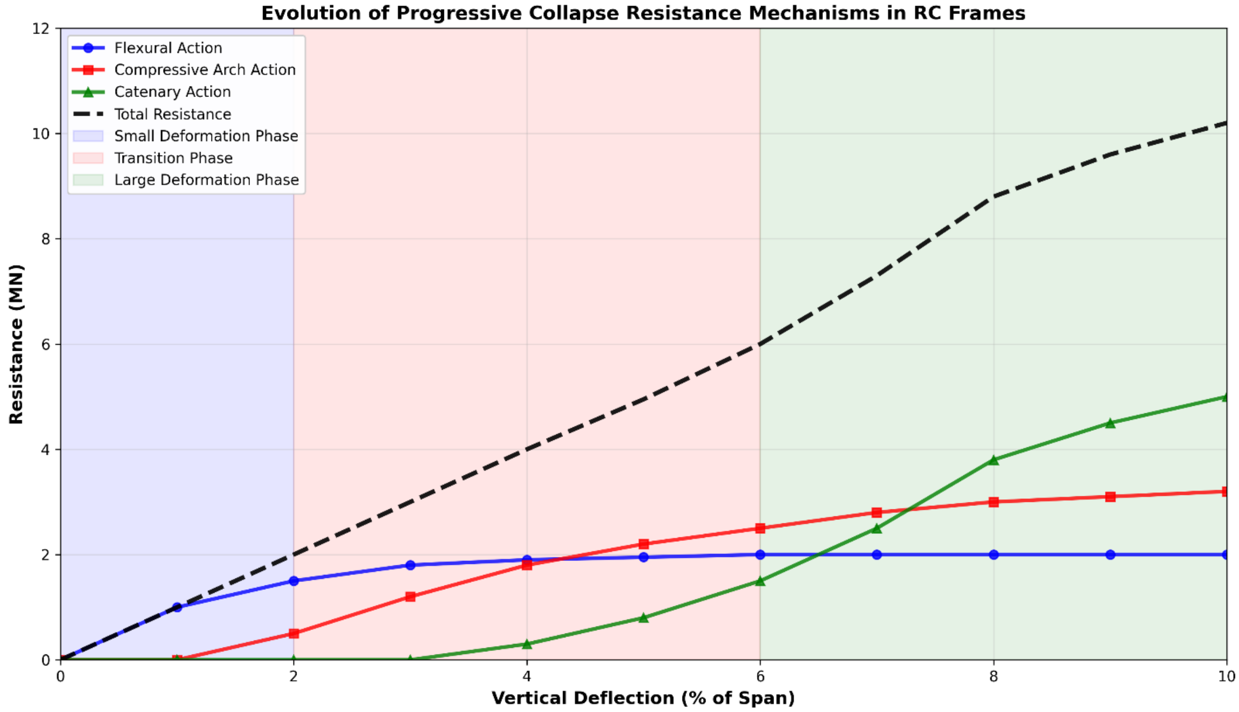
RESEARCH GAPS, CHALLENGES, AND FUTURE DIRECTIONS

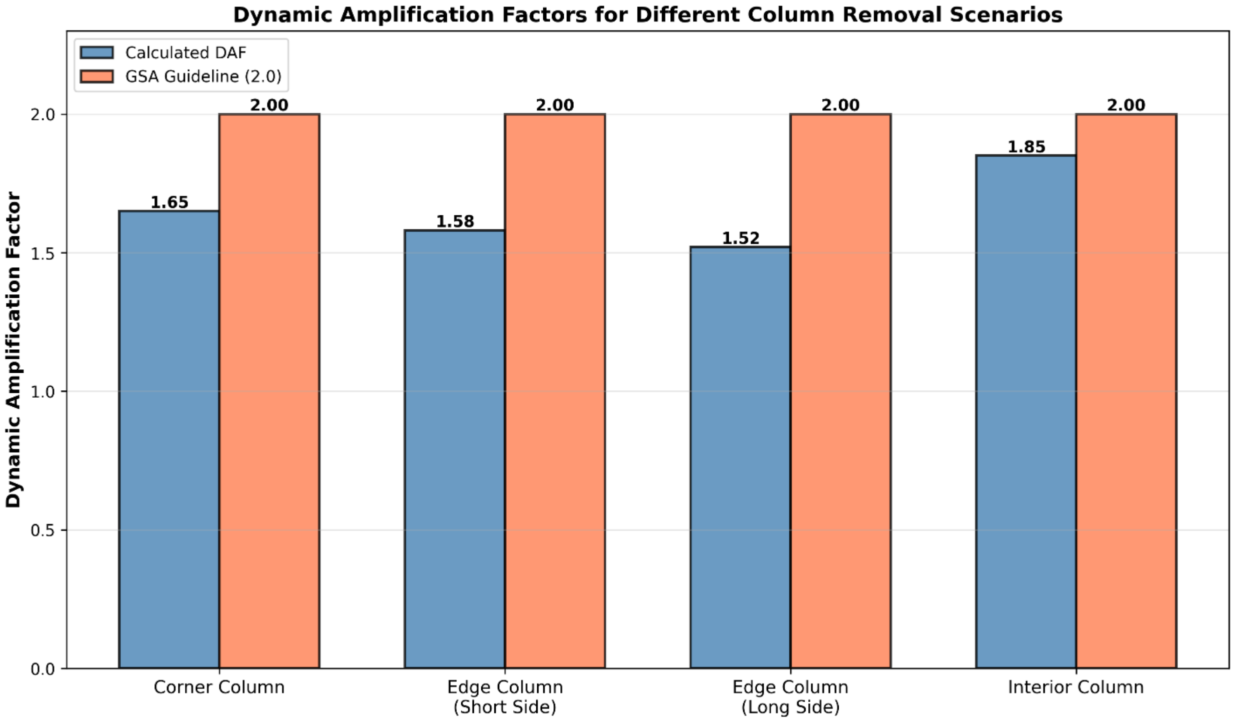
6.1 Multi-Column Removal Scenarios and Complex Failure Modes

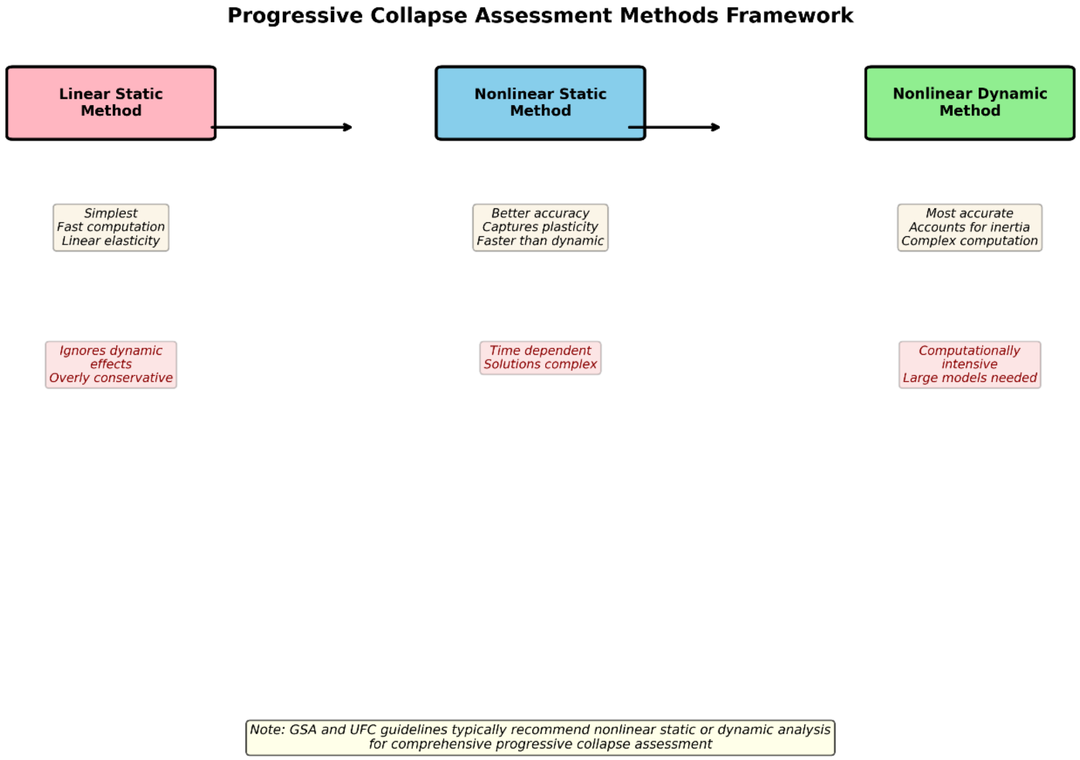
While extensive research has examined single-column removal scenarios, the behavior of RC structures under multi-column removal events remains inadequately studied [46]. The cascade effects and complex interaction mechanisms that develop when multiple columns fail in sequence or simultaneously represent critical gaps in current understanding [46]. Studies examining multi-column demolition scenarios indicate that the number of removed columns, their spatial configuration, and removal sequence substantially influence structural response and failure modes [46].

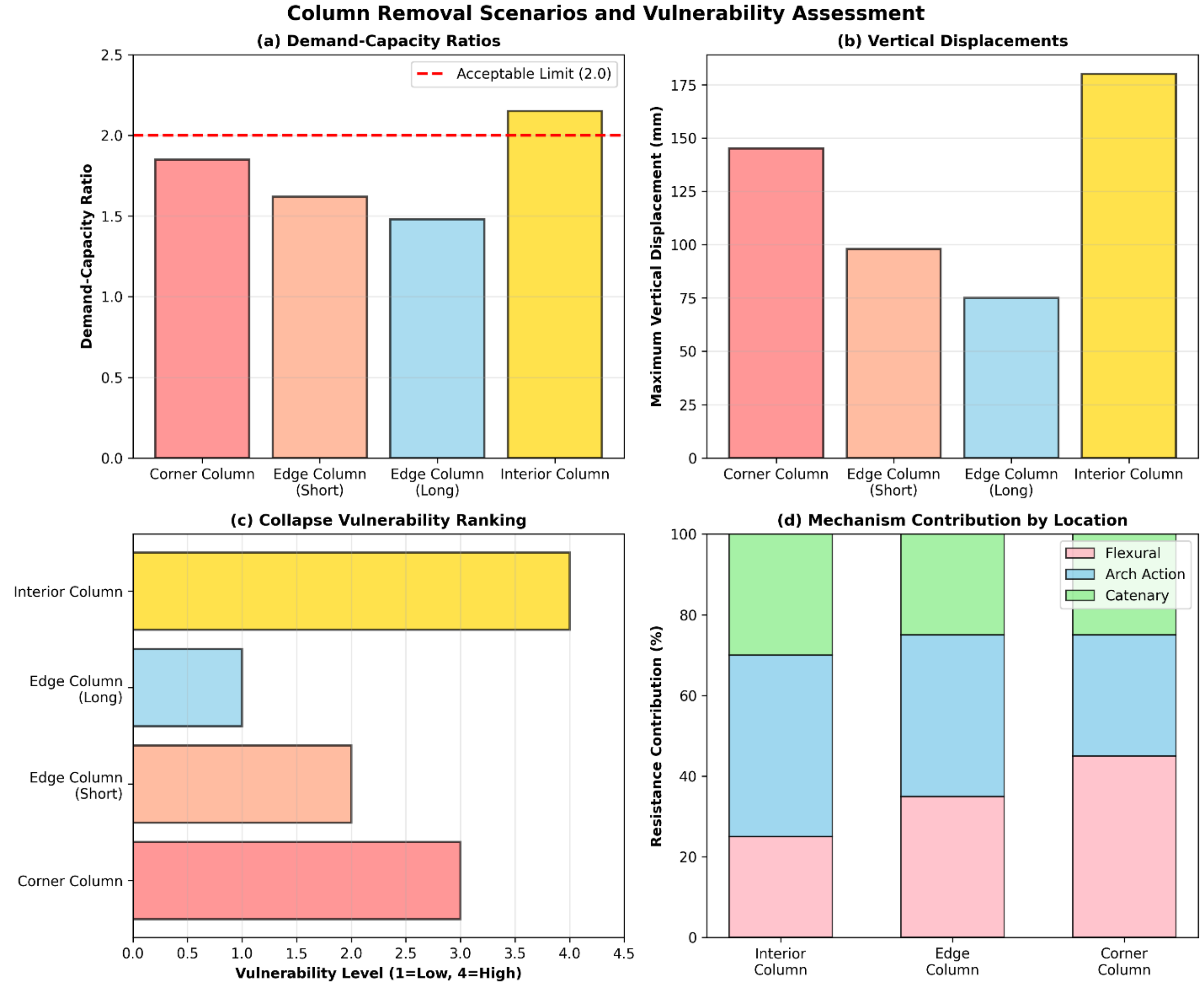
The research demonstrates that as the number of adjacent removed columns increases, the collapse speed accelerates and the dynamic response of the remaining structure is significantly amplified [46]. The force transmission paths become increasingly complex in multi-column scenarios, with load distribution dependent on the precise spatial arrangement and timing of failures


Effectiveness and Cost-Efficiency of Progressive Collapse Mitigation Strategies
Collapse Resistance
Relative Cost
Collapse Resistance Enhancement (%)
Relative Implementation Cost Index
0
50
100
150
200
250
300
350
0
20
40
60
80
100
120
140
Base Frame
CFRP Retrofit
Steel Plate Strengthening
Tie Column Addition
Infill Wall Incorporation
Hybrid Mitigation


Evolution of Progressive Collapse Resistance Mechanisms in RC Frames
Flexural Action
Compressive Arch Action
Catenary Action
Total Resistance
Small Deformation Phase
Transition Phase
Large Deformation Phase
Resistance (MN)
Vertical Deflection (% of Span)
0
2
4
6
8
10
12
0
2
4
6
8
10


Dynamic Amplification Factors for Different Column Removal Scenarios
Calculated DAF
GSA Guideline (2.0)
Dynamic Amplification Factor
0.0
0.5
1.0
1.5
2.0
1.65
2.00
1.58
2.00
1.52
2.00
1.85
2.00
Corner Column
Edge Column (Short Side)
Edge Column (Long Side)
Interior Column


Progressive Collapse Assessment Methods Framework
Linear Static Method
Nonlinear Static Method
Nonlinear Dynamic Method
Simplest
Fast computation
Linear elasticity
Better accuracy
Captures plasticity
Faster than dynamic
Most accurate
Accounts for inertia
Complex computation
Ignores dynamic effects
Overly conservative
Time dependent
Solutions complex
Computationally intensive
Large models needed
Note: GSA and UFC guidelines typically recommend nonlinear static or dynamic analysis for comprehensive progressive collapse assessment



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