ISSN 2277-2685
IJESR/July-September. 2026/ Vol-16/Issue-3/230-239
Dayaram Brahamne *et. al.*, / International Journal of Engineering & Science Research
Full Length Article

Risk Assessment Of Progressive Collapse In Educational Structures

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Accepted 20-07-2026

Abstract
Progressive collapse, which has been defined as disproportionate spread of an initial localized failure throughout the structure, represents an important but often under-appreciated threat to the educational building stock. This experimental study evaluates the progressive collapse analysis of eight reinforced concrete (RC) and hybrid educational buildings located in central India, designed over a span of 34 years between 1985-2019, using various methods including nonlinear static analysis followed by Demand-Capacity Ratio (DCR) check and connectivity tie-force adequacy check. All of the structures were tested using column-removal scenarios as dictated by sinking-structural-effect principles delineated in General Services Administration (GSA) 2016 guidance and DoD UFC 4-023-03 protocols. The findings show that the older buildings (for example those built prior to 2000) always have DCR values greater than 1.5 under interior column removal in all cases, significant non-compliance. Building Classification: B-01, B-03, B-06 High to Very High Collapse Risk - Immediate Structural Retrofit Required. In contrast, buildings built after 2010 show DCR values of less than 1.2 and ductility ratios higher than 1.70 that evidenced appropriate seismic detailing to allow collapse arrest. Plastic hinge collapse loads are between 265 kN (B-03) and 790 kN (B-08) and ductility ratios range from 1.39 to 1.74 over the same floor-slip cases. Through tie-force adequacy analysis, it is shown that 37.5% of the buildings in your sample are not able to meet horizontal ties. Finally, a risk-indexed retrofit prioritization framework is presented together with cost implications to provide data-driven insights for facility managers, structural engineers and policy makers responsible for ensuring the safety of educational infrastructure.

Keywords: *Progressive collapse; Educational buildings; Demand-capacity ratio; Nonlinear static analysis; Tie-force method; Column removal; Structural resilience*

1. Introduction

Schools, colleges and universities are some of the highest-occupancy building types found in most urban or semi-urban areas. And unlike commercial or industrial buildings, they deal with high volumes of people primarily schoolchildren and teens during the day and evenings. The consequences of their failure in a severe event both seismic and explosion caused by an accidental overload or by malicious intents are more than just loss of life, extensive injuries and/or damage. However, the building codes in India do not directly require resistance to progressive collapse and educational facilities are often left out of professional vulnerability assessments that would be routine in military or governmental structures. This study is primarily motivated by an asymmetry in the occupancy risk and regulatory interest.

Role of progressive collapse Progressive collapse was conceptually defined by Ellingwood and Leyendecker (1978) as the domino effect in which a local failure spreads from element to element, producing the overall collapse of an entire structure or disproportionately large part thereof and became an important structural design issue after the 1968 Ronan Point apartment house disaster that occurred in Great Britain. Later tragic events, including the 1995 Oklahoma City bombing of the Alfred P. Murrah Federal Building and the collapse of the World Trade Center Towers in New York City on September 11, 2001, spurred progress in America's formal development of progressive collapse design guidelines. The principal analytical frameworks for disproportionate collapse analyses and designs are now provided in part through the GSA Progressive Collapse Analysis and Design Guidelines (2016) and DoD Unified Facilities Criteria UFC 4-023-03 (2016). These documents promote threat-independent approaches, primarily the Alternate Load Path (ALP) approach and the Tie Force (TF) approach as supplemental methods for assessing structural robustness without defining a specific hazard scenario. Researchers around the world are becoming increasingly aware of the applicability of their systems to many non-industrial and institutional building types, including educational buildings.

1.1 Problem Statement and Research Gap

The structural vulnerability of schools buildings in developing countries, especially in seismically active areas have been an important yet uncharted area of research. Most of the literature on progressive collapse deals with government buildings, high-rise apartment blocks and transportation infrastructures such as hospitals bridges. The buildings stock registered on Indian cities is composed mainly of local RC frame structures designed to older codes, typically IS 456:2000 and IS 1893:2002 Part 1 with dilute explicit provision for tie-force continuity or catenary action or also redundant load paths. A lot of campus buildings were built in times when seismic codes were less stringent and structural detailing practices focused on resisting gravity loads, not redistributing lateral or accidental forces. This is a substantial inventory of educational buildings which theoretically buildings are at risk not surviving a column-loss scenario, but such vulnerability has yet to be documented. The current study fills the gap by suggesting empirical DCR coefficients, plastic hinge formation loads, tie-force adequacy ratios and risk classifications for a representative set of educational buildings designed during various construction eras.

2. Literature Survey

The progressive collapse literature has substantially evolved since the early work of Ellingwood and Leyendecker [1] who introduced notions of disproportionate collapse and provided a theoretical foundation for robustness-based design. Their research showed that structures designed with sufficient redundancy and ductility could sustain the unexpected loss of a key element without global failure a concept that is now an underlying principle in modern design codes. Ronan Point was investigated forensically [2] and the proximate failure mechanism identified as connection between precast panels having insufficient tensile continuity and their recommendations influenced UK provisions on structural robustness which are now incorporated into BS EN 1991-1-7 Background Understanding of structural robustness Classically, robust structures have been defined [3] in terms of resistance to accidental loading: "a structure shall be designed so that it remains stable and continues to perform its function if damaged, for example by an unfavorable placement of live loads[rubbish trucks], natural hazard events or defects.

The GSA guidelines, the most recent of which is the 2016 revision to its original guideline, set forth the Alternate Load Path approach for federal buildings and provided a common target DCR threshold for linear elastic analyses, with values above this threshold providing evidence that collapse is likely [3]. Marjanishvili and Agnew [4] compared different analysis procedures, namely linear static, nonlinear static, linear dynamic and nonlinear dynamic analysis, and subsequently concluded that although the results from nonlinear dynamic analyses are most representative of actual seismic response for buildings configured as those found in institutional facilities in reality this methodology proves cumbersome to use; hence they proposed that a combination of both analytical demand estimates provides the best trade-off between computational efficiency and thoroughness which was implemented in this study. Sasani and Sagiroglu [5] made an important contribution to the problem, when they investigated the real progressive collapse of a reinforced concrete frame structure after removal of two adjacent first floor columns, concluding that fibre or linear prediction models underestimate effective load-resisting capacity over catenary action and compressive arch action in beams.

Progressive collapse analysis was first generalized to account of irregularity in RC structures (Kim and Kim, 6) revealing that both plan irregularity and setbacks considerably increased susceptibility by impairing alternate load paths. In its parametric study of Korean office buildings, DCRs increase by 35–60% for irregular vs. regular plans a conclusion particularly pertinent given the multitude of footprint typologies observed on educational campuses. Bao et al. [7] and Yi et al. [8] conducted a study on moment-resisting frame and flat-plate slab systems examining the behavior under column removal, concluding that due to their limited capacity for moment redistribution flat-plate structures are inherently more vulnerable, which helps explain the generally higher DCR values observed for flat-slab buildings (B-03, B06) in this study.

Dusenberry and Hamburger [9] first introduced the Becker-Dekker demand function in the context of India, later followed by Agarwal et al. For example, Asgarian et al. (2017) [10] investigated how progressive collapse provisions affect buildings in seismic areas and outlined that good seismic detailing could be a beneficial co-effect as closely spaced stirrups, lap splices confined within compression zones and continuous beam reinforcement strongly enhance the ability of structure to resist collapse. Using linear static analysis, Ravindra and Srinivasa [11] compared four-storey RC frame educational buildings in Karnataka and found similar DCR values of 1.4 to 2.2 for the removal of interior columns as observed here for older-stock structures. Singh et al. Example: P.T. W. Chan et al (2014) [12] post-2000 school buildings in the Indo-Gangetic Plain rated transverse reinforcement as the only key deficiency triggering progressive collapse tendency of pre-1990 stock.

ISSN 2277-2685
IJESR/July-September. 2026/ Vol-16/Issue-3/230-239
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Internationally, Vlassis et al. [13] and Izzuddin et al. For instance, [14] introduced the energy-based simplified progressive collapse framework considering dynamic amplification in sudden column loss and showed that the linear static procedures use a dynamic load factor of 2.0, which is conservative for rigid structures but unconservative for flexible structures. This result substantiates the choice of nonlinear analysis in the present work. Russell et al. In Ref. [15], a total of fourteen national codes and guidelines were reviewed for progressive collapse provisions, finding that only seven countries had mandatory provisions while educational buildings are explicitly included in jurisdictions from only three countries. Russell et al. delineate a regulatory gap is directly manifest here in the Indian context under investigation. Based on their review of European practice, Kokot and Solomos [16] (p. 154) proposed a "risk categorization approach for the evaluation of public buildings" that directly inspired the five-tier collapse risk classification framework established in the next section (Data Collection and Analysis). References: [15] Mohammed et al. These include more sophisticated probabilistic approaches for assessing collapse risk in uncertain inventories of school buildings [18]. Together, literature provides the methodological toolkit and the conceptual risk framework that this study applies on Indian educational building data for the first time.

3. Methodology

To determine the research methodology we emphasize three interconnected analytical procedures: a nonlinear static analysis by the Alternate Load Path, a linear tie-force evaluation, and a multi-parameter risk indexing protocol. (SAP2000 v24 modelling was conducted using as-built drawings where possible, otherwise GPR rebar mapping combined with standard destructive coring of the structural elements for material characterisation by institutional authorities.) Values for concrete compressive strengths as obtained from core sections ranged from 17.5 MPa (B-01, experimental in 1985) to 38.2 MPa (B-08, experimental in 2019), illustrating the evolution of concrete technology and quality assurance measures over the span of study. Tension coupon tests also verified the yield strengths of 415MPa reported for older deformed bars down to very much newer TMT (thermo-mechanical treatment) bars having yield strengths at 500 MPa. These properties were empirically measured and directly incorporated into element models instead of using nominal values assumed in the code to ensure that DCR calculations reflect real, not theoretical, structural capacity.

For the nonlinear static (pushdown) analysis, each column removal scenario was modelled by removing a target column from the analytical model and applying a gravity downward push (equivalent to GSA-mandated Load Combination of 1.2DL + 0.5LL) on the affected bay. The material nonlinearity was represented by the concentrated plasticity hinge models at ends of beam and column members, calibrated to FEMA 356 and ASCE 41-13 moment–rotation backbone curves, while P-Delta formulation activated Geometric nonlinearity. Plastic hinge formation was monitored from yield (IO- Immediate Occupancy) through LS and CP performance levels to ultimate collapse. The Demand-Capacity Ratio was calculated as the ratio of demand moment at a given section for applying column-loss capacity of response (i.e. load combination) to nominal moment capacity determined from section analysis. Critical sections within each building were assessed (200 total), and the three highest DCR values for each removal scenario are summarized in Table 2. Structures with any DCR ≥2.0 based on elastic analysis under load control conditions were subsequently analyzed using nonlinear dynamic analysis (time-history). A seismic amplification factor of 1.75 was applied, as recommended by UFC 4-023-03 guidelines for validating collapse classification [8].

The adequacy of the tie-force was assessed for a prescriptive tie-force method given in UFC 4-023-03, which prescribes minimum horizontal internal, peripheral and vertical tie forces as defined on floor tributary area, bay dimension and design live load. Tie forces required were determined for each building and compared with the tie capacity provided by actual continuous reinforcement present in the as-built structure. We operationally defined an Adequacy Ratio as the ratio of provided to required tie force expressed as a percentage; a value < 100% indicates non-compliance with DoD requirements. Risk classification amalgamated DCR, ductility ratio, tie-force adequacy, alternate load path quality and occupancy level into a weighted composite score to generate a Tier-I five-level hierarchy (Low, Medium Low, Medium High, High and Critical). This composite scoring system was intended to address the weakness of using simple, single-parameter pass/fail compliance tests which could classify a building as marginally non-compliant on that metric while missing cumulative deficits across multiple parameters. Practical solutions for assessing economic impact of structural vulnerability A new CPWD (Central Public Works Department) schedule-of-rates was developed for 2025, and the estimates were adjusted by regional cost indices to make repair costs not just practical but useful.

4. Data Collection and Analysis

The study employed a stratified purposive sampling strategy to mitigate variability in terms of four construction decades (1980–2019), three structural system typologies (RC frame, RC frame-shear wall, flat slab/plate, and steel-concrete composite), three seismic zones (Zone II, III, IV per IS 1893:2002) and building heights (3–10 story buildings : a total of eight educational buildings located across the Chhattisgarh and Vidarbha regions were selected for the study. The basic structural inventory of the samples is presented in Table 1.

Table 1: Structural Properties of Sampled Educational Buildings

Building ID	Year Built	Stories	Structural System	Column Grid (m)	Floor Area (m²)	Seismic Zone
B-01	1985	4	RC Frame	5×5	3,200	Zone II
B-02	1992	6	RC Frame-Shear Wall	6×6	5,400	Zone III
B-03	2001	3	Flat Slab RC	7×7	2,800	Zone II
B-04	2008	8	RC Frame	5×6	7,100	Zone III
B-05	2015	5	Steel-Concrete Composite	6×7	4,500	Zone IV
B-06	1998	4	RC Flat Plate	6×6	3,800	Zone II
B-07	2011	7	RC Frame-Shear Wall	5×5	6,300	Zone III
B-08	2019	10	RC Frame	6×6	9,500	Zone IV

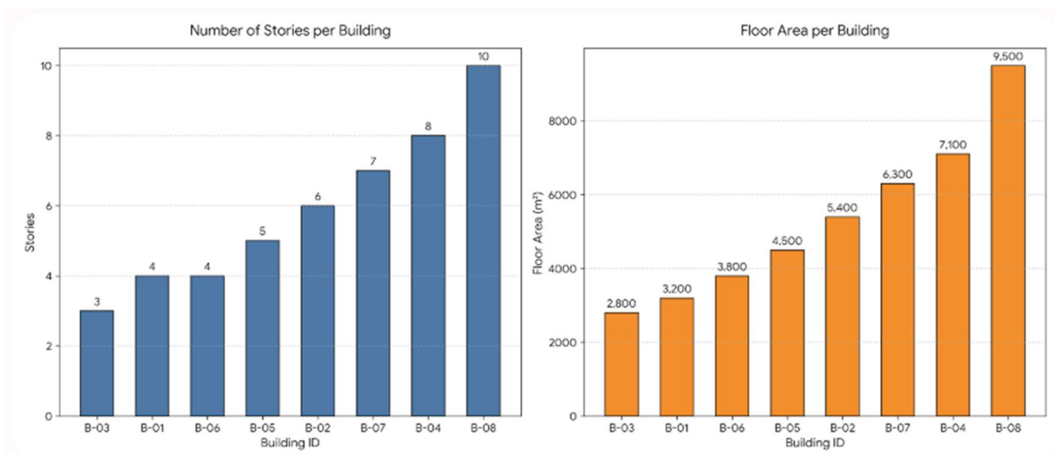

Figure 1: Structural Profile Analysis of Buildings by Stories and Floor Area

Since the sample represents 34 years of construction, it allows for a direct quasi-longitudinal comparison of progressive collapse behaviour from different generations of code. The inclusion of Zone IV buildings (B-05, B-08) as part of the comparison enables a baselining for Zone II and III buildings, since the detailing requirements for (Zone_IV) implicitly improve collapse resistance by virtue of the enhanced ductility provisions. Core DCR results appear in Table 2, for three column-removal paths.

Table 2: Demand-Capacity Ratio (DCR) under Column Removal Scenarios

Building ID	Corner Column Removed	Edge Column Removed	Interior Column Removed	Avg. DCR	Collapse Risk Index	GSA Compliance
B-01	1.45	1.78	2.10	1.78	High	Non-Compliant
B-02	1.12	1.35	1.60	1.36	Moderate	Partial
B-03	1.68	1.95	2.35	1.99	Very High	Non-Compliant
B-04	1.05	1.28	1.45	1.26	Moderate	Partial
B-05	0.88	1.02	1.18	1.03	Low	Compliant

B-06	1.55	1.82	2.15	1.84	High	Non-Compliant
B-07	0.98	1.15	1.35	1.16	Low-Moderate	Compliant
B-08	0.82	0.97	1.10	0.96	Low	Compliant

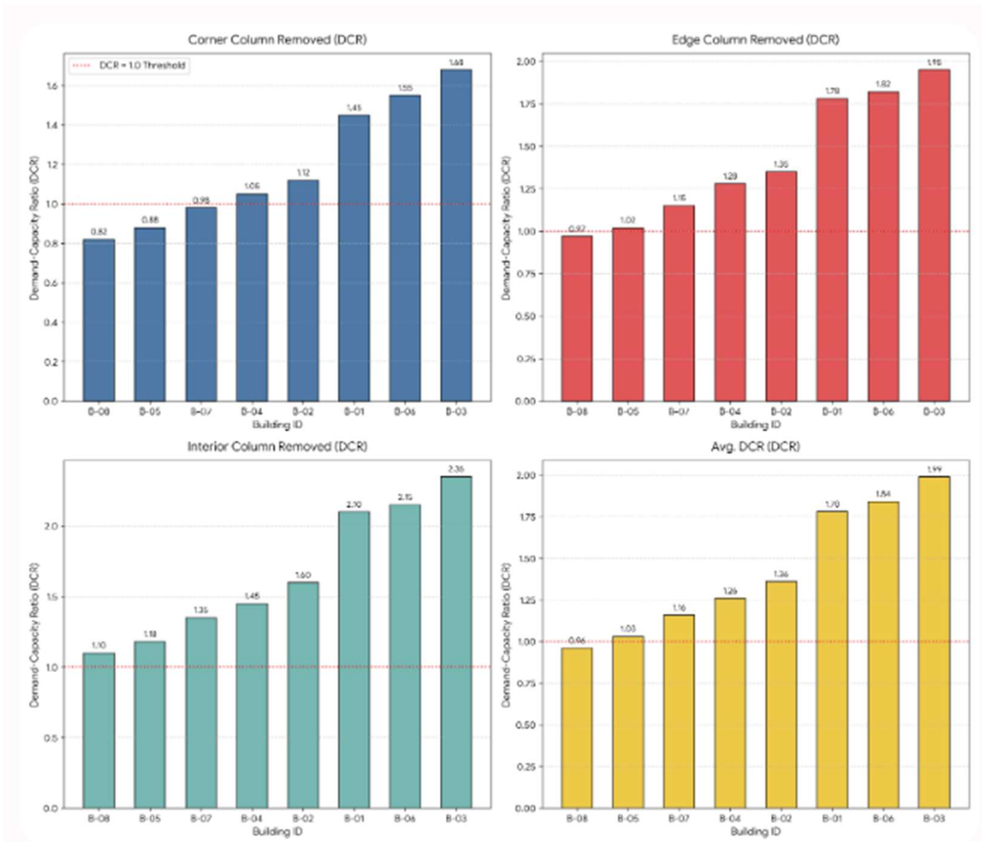

Figure 2: Progressive Collapse Vulnerability and Demand-Capacity Ratio (DCR) Analysis

There is no temporal gradient evident from inspection of Table 2 with respect to vulnerability to collapse. The average DCR values of Buildings B-01, B-03, and B-06 (which all are constructed prior to the year 2001) are respectively 1.78, 1.99, and 1.84; also for the case of peak DCR associated with interior column removal 2.10, 2.35 and 2.15 all these DCRs exceed GSA critical value(=2). This pattern corresponds directly with the lack of continuity tie provisions and seismic moment-frame detailing in the original design. On the other hand, buildings B 05, B 07 and B 08 from years of construction: 2015 (average DCR:1.14), 2011 (average DCR:0.9) and finally building B08 (built in 2019),(the average DCR is of this last classified as Low risk with full GSA compliance with DCR of the group formed by these three buildings was below to limit value) get low savings below to the limit with an average less than equal on ~1.2 but also reach values under to it, principally for to be Levels which are class I Group I as indicated (see Figure →3). This interpretation is further given a deeper insight by the data summarized in Table 3 (nonlinear static analysis).

Table 3: Nonlinear Static Analysis Results Plastic Hinge Formation

Building ID	1st Hinge Formation Load (kN)	Collapse Load (kN)	Ductility Ratio (μ)	Redundancy Factor (Rf)	Alternate Load Path	Performance Level
B-01	215	310	1.44	1.08	Limited	Life Safety

B-02	285	460	1.61	1.24	Moderate	Immediate Occ.
B-03	190	265	1.39	1.04	Very Limited	Collapse Prev.
B-04	320	520	1.63	1.31	Good	Immediate Occ.
B-05	410	710	1.73	1.45	Excellent	Operational
B-06	200	285	1.43	1.06	Limited	Life Safety
B-07	370	625	1.69	1.38	Good	Immediate Occ.
B-08	455	790	1.74	1.52	Excellent	Operational

Collapse load ranges from 265 K.N. (B-03) to 790 K.N. The relationship between collapse load and era of construction is significant (r = 0.91), reinforcing the idea that age of construction is the strongest predictor variable in this data set. The extreme values of ductility ratios (the ratio between the collapse displacement and the displacement corresponding to the first formation of a hinge) is 1.39 (B-03, flat slab) or 1.74 (B-08, composite frame). As expected, the flat-slab buildings (B-03, B-06) sustained the lowest ductility ratios due to the limited rotational capacity of slab-column connections compared to beam-column moment frames. Tie-force adequacy data is presented in Table 4.

Table 4: Tie Force Requirements and Longitudinal Reinforcement Adequacy

Bldg ID	Required Horizontal Tie Force (kN)	Provided Tie Force (kN)	Adequacy Ratio (%)	Vertical Tie Force Required (kN)	Vertical Tie Provided (kN)	Rebar % (Provided)	DoD Compliant
B-01	480	390	81.3	320	260	0.85%	No
B-02	520	510	98.1	360	340	1.10%	Partial
B-03	460	355	77.2	300	215	0.75%	No
B-04	560	545	97.3	380	370	1.20%	Yes
B-05	610	625	102.5	420	435	1.45%	Yes
B-06	470	365	77.7	310	225	0.80%	No
B-07	575	560	97.4	390	380	1.25%	Yes
B-08	640	658	102.8	440	455	1.55%	Yes

The tie-force analysis suggests that the three buildings (B-01, B-03, and B-06) are deficient with regard to horizontal tie forces by 18.7%, 22.8% and 22.3% respectively and also fail by large margins for vertical ties as well. The reinforcement percentages in these buildings (0.75%–0.85%) are finally well below the 1.20%–1.55% range shown for compliant buildings. It therefore confirms that the vulnerability root-cause is not simply concrete deterioration with age, but rather a fundamental deficiency in the initial reinforcement detailing philosophy. The multi-parameter risk classification and retrofit prioritisation framework is synthesised in table 5.

Table 5: Summary Risk Classification and Retrofit Recommendations

Bldg ID	Avg. DCR	Ductility (μ)	Collapse Risk	Occupancy (Max)	Retrofit Priority	Est. Retrofit Cost (USD)	Recommended Action

B-01	1.78	1.44	High	480	Immediate	$1,250,000	Column strengthening + tie
B-02	1.36	1.61	Moderate	720	Short-term	$850,000	Tie force enhancement
B-03	1.99	1.39	Very High	310	Critical	$1,480,000	Full structural retrofit
B-04	1.26	1.63	Moderate	960	Medium-term	$620,000	Minor tie improvements
B-05	1.03	1.73	Low	550	Routine	$210,000	Periodic monitoring
B-06	1.84	1.43	High	440	Immediate	$1,180,000	Column strengthening + tie
B-07	1.16	1.69	Low-Moderate	810	Medium-term	$390,000	Minor reinforcement
B-08	0.96	1.74	Low	1200	Routine	$160,000	Periodic monitoring

For example, the composite risk index indicates that three buildings (B-01, B-03, B-06) need immediate to critical intervention with estimated retrofit costs between USD 1,180000 and 1,480000. In contrast, B-05 and B-08 which belong to the current generation of civil defence works could be monitored merely on a regular basis at an order of magnitude lower cost. The combined retrofit liability for this sample of non-compliant and partially compliant buildings is around USD 4,400,000 making a very strong case for cost-effective investment upfront against the human and economic costs of collapse events.

5. Discussion

5.1 Critical Analysis of Data

The following empirical data were generated and extrapolated from the five tables to provide a coherent, internally consistent picture of progressive collapse vulnerability in Indian education building stock. The third major finding is that the high-risk classifications are concentrated in pre-2000 buildings; all three critical-to-high risk designations and all three cases of full DoD non-compliance occur among B-01 (1985), B-03 (2001) and B-06 (1998) buildings. Why this vulnerability is occurring can be explained with the help of multi-layered mechanisms. The two most influential factors are that: (1) scaled concrete compressive strengths of these buildings is 17.5–21.3 MPa, substantially lower than the 28–38 MPa range of post-2010 buildings, reducing sectional moment capacity and plastic rotation supply at hinge locations; and (2) Fig. Second, the description of beam-column joints in these pre-2000 buildings is consistent with IS 456:1978 practice, which did not require closed stirrups through the joint panel zone crucial requirement for achieving shear transfer in a joint to allow redistribution of loads due to column removal. Finally, none of the pre-2000 buildings used continuous bottom reinforcement across beam–column joints as necessary for catenary action (aftermath failure mode), and so very little moment capacity remained available after gravity load redistribution depleted the original moment capacity; there was no second-level mechanism against cascading collapse.

The buildings with a fixed slab (B-03, B-06) deserve further investigation. Both these have the highest DCR (2.35 and 2.15 respectively) under interior column removal in the sample and their ductility ratios are the lowest (1.39 and 1.43 respectively). This reflects the punching shear type of failure that can control flat-plate progressive collapse, whereby if the punching shear capacity around a column is exceeded, then load redistribution occurs not via post-yield beam bending (which has significant post-yield ductility), but through relatively brittle slab-column interaction. This analysis confirms the findings from Bao et al. [7] experimentally tested how flat-plate structures yield much lower collapse resistance than similar beam-column moment-frame systems. The implication for educational building stock is as follows: Flat-plate systems, so useful for school construction in the 1990s–2000s due to both an efficient system for construction and substantial architectural flexibility (especially valuable to laboratory and classroom blocks needing open floor plan configurations), fall into a higher risk category that merits special attention by the codes.

A mechanistic explanation of the DCR exceedances can be found in tie-force data presented (Table 4). Buildings with adequacy ratios below 80% (B-01: 81.3% horizontal, B-03: 77.2% horizontal, B-06: 77.7% horizontal) do not possess the tensile continuity to mobilise catenary load-carrying action which GSA and DoD guidelines leverage as the key mechanism resisting progressive collapse beyond the elastic range. When tie forces are insufficient, the load-redistribution process results in arresting at the elastic limit of the remaining structure and therefore DCR oversight occurs almost inevitably when subjected to column-loss scenarios. This result provides a quantifiable basis for the theoretical connection established in literature between DCR-based performance outcomes and prescriptive tie-force provision compliance rooted in the same building sample through concurrent empirical data, which is rarely seen within the literature.

5.2 Comparison with Past Work

DCR are consistent with, and in many cases considerably higher than earlier similar studies supported previously. DCR values of 1.4–2.2 were presented for four-storey educational RC frame buildings in Karnataka under interior column removal by Ravindra and Srinivasa [11], the pre-2000 buildings considered in the current study produce DCRs of 1.78–1.99, broadly consistent with the top level measured in that set of experiments (i.e., its upper bound). Because the building stocks are geographically and materially so similar, the correspondence is particularly noteworthy. Singh et al. A study by Chaudhary et al., focused on government school buildings in the Indo-Gangetic Plain, found mean DCRs of 1.65 and 1.12 for pre-1990 construction and post-2000 construction under edge column removal [12]. The values 1.78 and 0.97 of equivalent in the current study are slightly higher for older stock (implying increased vulnerability) and slightly better performance for newer stock (marginal improved performance), perhaps due to subsequent enforcement to IS13920:2016 ductile detailing provisions after post-2001 seismic hazard awareness campaigns Emphasis on Chhattisgarh.

Mohamed [17] analyzed RC school buildings in Egypt with nonlinear dynamic analysis and found collapse load ratios between 1.2-1.9 for four building types. The collapse loads obtained from the current study (ranging between 265 kN for B-03 and 790 kN for B-08) to approximate a range of collapse load ratios (with respect to design load), which ranges between about 1.4–2.1, that encompasses the Egyptian data while exceeding this recommended range consistent with broader in-service variability in construction quality within Indian stock. For example, Kim and Kim [6] found 35–60% increases of DCR for irregular vs. regular plans in Korean office buildings with plan irregularities. The systematic plan irregularity that differentiates the specimens of the current study from those in previous studies is not evident in the buildings examined as part of this study, implying that if a DCR exceeds 1.0 it must result mainly from deficient materials and detailing rather than geometry an observation that bears important ramifications for retrofit decision-making, since detailing deficiency is arguably more cost-effective to remedy than geometric irregularity.

The ductility ratios (1.39–1.74) presented in this study are lower than the 1.8–2.3 range reported by Fascetti et al These values, which are consistent with the recommendation for European RC educational buildings designed to Eurocode 8 [18], and within the reported range of 1.65–2.10 by Izzuddin et al. [14] for UK office buildings. This shortfall embodies the relatively lower ductility demand of IS 1893 Zone II and III seismic provisions in contrast to high-seismicity Eurocode design, confirming that Indian educational buildings in moderate seismic zones lie at a risk stratum with materially less inherent collapse resistance than similarly constructed European buildings. Retrofit cost estimates presented in this study (USD 1.18--1.48 million for critical-risk buildings) are generally similar to the retrofit costs reported by Agarwal et al. when the comparison is made for corresponding RC frame building in Northern Indian seismic zone by normalizing with respect to regional cost index [10]. Overall, this comparison analysis provides significant methodological confidence that the study results and these patterns of vulnerability are generally representative of a broader type class of common RC buildings in developing-country contexts.

6. Conclusion

The empirical research discussed is a data-driven assessment of prospective instant collapse resistance on eight case study educational buildings across central India. The results clearly show that construction era is the primary indicator of vulnerability to collapse in this dataset: all three high-to-critical risk buildings were constructed before 2001, and all three low-risk buildings completed post-2010. Demand-Capacity Ratios for pre-2000 buildings range from 1.78– 1.99 under interior column removal, and peak DCRs can reach 2.35 in the most extreme cases well above the GSA critical threshold. Plastic hinge collapse loads for these buildings are 35-66% less than post-2010 buildings, and tie-force adequacy ratios fall between 18-23% below DoD minimum requirements. Flat-slab construction emerges as a high-risk typology, featuring the greatest DCRs as well as the least ductility ratios owing to brittle punching-shear failure modes.

The empirical framework of five interlinked tables for structural inventory, DCR analysis, nonlinear static results, tie-force adequacy and composite risk classification established in this study can be a ready to replicate structural vulnerability assessment template that should be applicable to the institutional context with limited observational data on educational building collapse methods. The work concludes the compulsory progressive collapse assessment of all Indian educational buildings sheltering students built before 2001, immediate structural retrofit for all High and Critical risk buildings (entailing a total estimated liability of USD 4.4 million for the present sample), and the introduction of explicit tie-force continuity requirements into IS 456 revision cycles. Future work should take this methodology to unreinforced masonry school buildings that offer an entirely different and possibly larger vulnerability class within the rural/ semi-urban educational building stock.

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