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SEISMIC DAMAGE ASSESSMENT OF REINFORCED CONCRETE STRUCTURES: LOW, MEDIUM, AND HIGH RISES USING A PERFORMANCE-BASED SEISMIC DESIGN APPROACH

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ABSTRACT

Purpose: The research addresses the limitations of the traditional force-based approach in earthquake-resistant design, particularly its inability to account for inelastic behaviour fully. It explores the potential of nonlinear static assessment techniques within the performance-based seismic design (PBSD) framework to provide a more accurate measure of earthquake structural performance.

Design/Methodology/Approach: This research develops a set of damage indices to quantify structural damage in moment-resisting frames (MRFs) based on engineering demand parameters obtained through nonlinear analysis. The study examines reinforced concrete (R.C.) structures of various heights, evaluating their seismic load-bearing capacity and resilience using the PBSD approach.

Findings: The proposed damage indices offer a reasonable way to quantify structural damage and enhance the understanding of the plastic collapse process. Performance-based design mainly benefits R.C. structures, improving their seismic resilience and cost-effectiveness.

Research Limitation: Accurately quantifying building damage remains challenging even with nonlinear assessment tools. Further work is required to refine these tools for more precise damage quantification in various building types.

Practical Implication: The findings have practical implications in reducing repair costs and ensuring public safety by providing preliminary damage estimates for tall buildings. The PBSD approach also meets acceptance criteria for immediate occupancy and life safety across various seismic intensities.

Social Implication: By enhancing buildings' resilience to earthquakes, this research contributes to safer urban environments, reducing potential fatalities, economic losses, and downtime associated with earthquake-induced damage.

Originality/Value: This study provides valuable insights into performance-based seismic design and presents a practical method for quantifying structural damage in R.C. structures. The proposed damage indices and PBSD approach significantly advance the safety and cost-effectiveness of earthquake-resistant buildings.

Keywords: *Collapse. earthquake. economic losses. inelastic excursion. seismic design*

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INTRODUCTION

When an earthquake occurs, it can do the most damage. Because earthquake forces are irregular and non-systematic, the engineering instruments used to analyse buildings under the influence of these forces must be at their best. Regarding seismic design, the near-field ground motion (acceleration) is replaced with a performance-based design. Analyse earthquake forces to determine the structure's actual behaviour and recognise that damage is predicted, but it should be coordinated. POA, an iterative approach, will be examined as an alternative to conventional analytic methods in this study (Gönen & Soyöz, 2021).

Low, medium and high-rise RC structures are subjected to lateral pressures with various height-wise allocations until the present performance level [Purposed displacement] is achieved. Performance-based seismic engineering (PBSE) aims to develop structures that can withstand earthquakes (Esteghamati, 2024). These endeavours need a well-coordinated effort by specialists from various disciplines to achieve authenticity. Performance-based design's tremendous return is not generally accessible, which makes PBSD unique and more challenging to implement. Besides massive developments of comparable buildings, each building created using this approach is unique. The expertise gained cannot immediately be transferred to constructing structures of other sorts, sizes, and performance goals. Prescriptive Code Design (PBSD) has not been an economically viable alternative (Monjardin-Quevedo et al., 2022). PBSD is becoming more attractive to building and structural and seismic zone engineers because of recent advancements in seismic hazard assessment, PBSE techniques, experimental facilities, and computer applications (Ghosh, 2013). The PBSD technique will replace all other approaches to designing and delivering earthquake-resistant structures in only a few short years. To make effective and efficient use of PBSD, one must be aware of the uncertainties in the structural performance estimates and the seismic hazard calculations.

Effect of seismic Loads on tall buildings

According to our perspective, increasing the height of high-rise structures has an uncertain influence on their structural stability since it increases the building's mass and height of its centre of gravity. As a result, the shear force generated by seismic stresses is increased. High-rise structures are subject to earthquake loading, a lateral dynamic excitation. Conceptual design, preliminary design, and optimisation for gravity and lateral loads are all part of designing high-rise structures. As the height of a building grows, the amount of structural material needed to withstand lateral stresses, particularly wind and earthquake loads, dramatically increases, making it more vulnerable to these loads (Ciabattoni et al., 2024). Since these loads may cause structural and non-structural earthquake damage, high-rise structures should be safeguarded from them. (Torghabeh et al., 2023).

The seismic load on a building equals the total of the loads on each level. Full dead load plus suitable imposed load constitutes each floor's seismic weight, which is the weight of each floor divided by its dead load multiplied by the appropriate amount of applied load. In this case, it comprises the weight of the permanent walls, the permanent equipment, a portion of the live load, and so on (Rihal et al., 2020). Each level above and below a storey's columns and walls

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must bear an equal share of the storey's weight in seismic calculations. The ground movements (accelerations) that a structure experiences as a consequence of earthquakes are likewise dynamic or variable in character, and in fact, they reverse direction quite chaotically. The magnitude of an earthquake force is determined in part by the magnitude of the earthquake itself, the distance from the epicentre, the local ground conditions that may amplify or dampen ground shaking, the weight (or mass) of the structure, as well as the type of structural system and its capacity to withstand abusive cyclic loading. During an earthquake, the amount of lateral force that a structure is subjected to is directly proportional to both the ground acceleration at the building site as well as the total mass of the building (i.e., a doubling in ground motion acceleration or building mass will double the load) (Palanci, & Senel, 2019).

Developing skyscrapers and other tall structures is a hallmark of contemporary cities since it allows for concentrating residential units, banks, marketplaces, exhibits, and other amenities on tiny plots of land or in a single building without available construction sites. The characteristics of a high-rise building (height, mass, high ground pressures and bottom pressures, oscillations, and difficulties evacuating in an emergency) define the hazards for those who live in the high-rise structure and those who live nearby. Factors such as terrorist attacks, wind and seismic loads, and unrestricted building operations all significantly impact the utilisation of buildings (Sharifi, 2019). The horizontal component of a high-rise structure is vulnerable to earthquakes, which is one of its operational aspects. High-rise structures become more susceptible as they grow in height and decrease their bulk via new materials and the intelligent use of those materials' load-bearing capabilities. Stiffness and frequency oscillations are reduced due to the building's reduced bulk. An increase in structural rigidity makes wind resistance more remarkable, but it also increases the structural susceptibility to a seismic event. Modifying a structure's cross-sectional moment of inertia is one way to alter its rigidity (Heo, 2013).

However, the building's height is the most critical factor in determining its resistance to wind and seismic loads, as well as its interior volume (area), future architectural face, and the validity of its position amid other smaller structures. The higher the building, the more susceptible it is to buckling in the wind. Knoll (2023) argued that increased height has an equivocal influence on the stability of high-rise structures when seismic stresses are involved since it increases the mass of the building and raises the centre of gravity. As a result, the shear force increases by a factor of three when subjected to seismic pressures. When it comes to towering structures, on the other hand, they are more flexible and better able to withstand the acceleration of the bottom. Consequently, it is essential to examine the influence of building height on its seismic resistance. (Mondal, 2013)

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Several publications on high-rise structures and wind loads are accessible. On the other hand, the seismically vulnerable high-rise structures have received little attention. Numerous locations have gone years or decades without any earthquake activity. Are high-rise buildings essential in seismically active areas, given that earthquake damage to supporting structures may be as severe as damage from a fire or a terrorist act?

Another problem with simulating the behaviour of a high-rise structure during an earthquake is that it is unpredictable in terms of length, force, direction, and other factors. It's also worth noting that high-rise building seismic safety systems have gathered data on wind speed and direction from meteorological measurements in this region. The topics of seismic isolation and seismic dumping are covered. When protecting buildings against earthquakes, it has been shown that bottom-mounted and other height-mountable seismic isolation systems are more effective than seism-stable structures (columns, walls, frames) (Ghoohestani et al., 2022). Only a few studies have been done on the stiffness of high-rise buildings, and they should be noted.

For high-rise buildings and passive earthquake compensation needs, one's design offers a kinematic earthquake dumper system, as is the study of passive and active earthquake safeguard systems. Roller friction and dumpers with specific stiffness and damping parameters are included in the latter. Both a model of a sliding zone with nonlinear damping and a study of basement compliance to reinforced concrete buildings under seismic impact are used to isolate the high-rise building in the event of an earthquake (Meral, 2021).

THE PRESENT STATE OF SEISMIC ANALYSIS

Understanding structural analysis in depth is required to design and evaluate seismic systems. More than a century ago, seismic regulations mandated static analysis with lateral stresses equal to about 10 percent of the structure's total weight. The majority of seismic codes around the world have, for quite some time, incorporated this size in the definition of seismic loads. Over the course of history, advancements in structural dynamics and nonlinear response have been implemented, making it possible for increasingly complicated analytical procedures to be carried out. In the not-too-distant future, methods including explicit probabilistic considerations may be used. This work presents a study of seismic provisions as they apply to analysis, together with a discussion of the current situation and potential future revisions. Also included in this work is a discussion of the current posture. (Zameeruddin, 2017).

Probabilistic approaches to seismic performance evaluation are suitable because of the high level of uncertainty associated with ground motion and structural modelling. However, most engineers are unfamiliar with probabilistic approaches and are reluctant to employ them. Many in the scientific community are also sceptical about explicit probabilistic methodologies other than those used in seismic hazard assessments (Rezaeian et al., 2024).

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A quick poll of researchers worldwide shows that almost all doubt that their countries' building codes will adopt an explicit probabilistic approach any time soon. The only country that has done so explicitly is the United States, where it has already been done in ASCE 7 but has been applied sparingly in actual construction.

Codes for critical infrastructure should include an explicit quantitative evaluation of risk.

But in the long run, it is impossible to forgo quantitative risk analysis completely. For the four most developed nations with significant seismicity, the profession will be obliged to adopt some risk-based design and assessment sooner or later, at the very least to better calibrate various safety factors and force reduction factors used in codes. Designers, building owners, and other stakeholders might benefit from information on the likelihood of earthquakes. Applying explicit probabilistic techniques is still a long way off when it comes to seismic construction standards.

There must be trustworthy input data and substantially simplified techniques known to engineers, and need just a little extra work and ability before quantitative risk assessment can be included in the codes. The educational value of including reliability-based content in seismic codes cannot be overstated. An informative annexe to EC8 was recently drafted in Europe and is described in more detail in the following section as the first step. It was in 2012 that the Applied Technology Council [ATC] created a technique for assessing a building's seismic performance [FEMA P-58, 2012]. Between 1997 and 2010, PEER researchers established a framework for performance-based seismic engineering that was used as the foundation for the technique used in this study. In the dedication of the FEMA P-58 report, A. Cornell and H. Krawinkler are recognised as the driving factors behind this framework's creation. Performance-based seismic design is the intended usage for these methods.

It may be used for the evaluation of both new and existing buildings. A probabilistic approach is used, and uncertainties are explicitly taken into account. Performance is expressed in terms of human losses [deaths and serious injuries], direct economic losses (building repair or replacement costs), and indirect losses [repair time and unsafe placarding] resulting from earthquake shaking. A PACT - a computerised performance assessment calculator - was made available as an added convenience. However, the complete method is still missing from the codes. At the very least, more straightforward practice-oriented methodologies for assessing seismic risk are required to help gradually include probabilistic concerns into practice and standards. Simplified reliability-based verification format [CEN 2017] has just been published as an Annex to EC8, Part 1, by Dolek & Co. (Zameeruddin, 2017).

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Next Generation Procedures

Seismic risk decision-making may be improved by using new design and assessment methodologies based on probabilities and attempting to account for the inherent uncertainty in seismic risk. Approaches that allow for the explicit assessment of collapse risk and distinct damage states can meet the objectives of performance-based earthquake engineering. Protocols are already in place for this kind of thing.

For example, the process presented in Appendix F of the FEMA P-695 paper uses current research findings to offer a rigorous explicit probabilistic approach for structural analysis. As a result, a structural model that can simulate collapse is required, as are several [perhaps hundreds] nonlinear response history studies, as well as explicit consideration of a wide range of potential uncertainties. It is expected that designers adopting performance-based design show via testing and analysis that the design can meet these reliability requirements. "Subject to the permission of the authorities having jurisdiction for specific projects," study and design are required.

A failure probability calculation will very definitely never be done in reality since the standard allows five implied demonstrations that the desired dependability can be reached. The standard pushover-based analysis and the pushover-based N2 method used to determine a structure's capacity can easily estimate the annual probability of "failure" for a structure, provided that predetermined default values for dispersions are used. The Pushover-based Risk Assessment Method is straightforward in line with the Annex and requires only a minor effort in addition to a standard pushover-based analysis (Munoz et al., 2024). Compared to Cornell's initial approach, the PRA technique substitutes nonlinear response history analyses for a few pushover tests (Fajfar, 2018; Cornell, 1996). This change was made to simplify the process (in most cases, it was just a single one). The pushover-based risk assessment approach, Cornell's closed-form solution, and the essential N2 technique fall within the category of other more straightforward approaches subject to the same limitations. (Habibi, 2013)

The research introduces advanced nonlinear static analysis techniques, which have demonstrated practical superiority. However, it is essential to note that these methodologies do not completely address all the deficiencies associated with Nonlinear Static Procedures (NSP). Recent enhancements in inelastic seismic analysis have unveiled crucial insights into areas requiring further development and enhancement. This necessitates additional research in the following key aspects:

1. Development of a nonlinear model to describe the degradation of strength and stiffness under cyclic and in-cycle loading conditions.

2. Investigation of the nonlinear interaction between soil and foundation structures in multi-degree-of-freedom systems with significant simplifications for design purposes.

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METHODOLOGY

The research design employs nonlinear static analysis (pushover analysis) to assess damage indices (DIs) for reinforced concrete Moment-Resisting Frames under various seismic loading conditions. Damage levels are quantified by comparing the structural stiffness at different performance levels, utilizing linearization techniques for more accurate damage estimations.

Damage indices based on drift

$\frac{dj-dop}{du-dop}$Drift (conditions) restrictions are used to establish distinct degrees of performance (criteria) in PBSD (Table 6). Identifying performance levels and collapse mechanisms using this limit is possible, but no damage value is provided. To get around this problem, a drift-based damage index was put out. In the equation, the available ductility for a given performance level is used to calculate the damage value in the DI_d. [1].

$DI_d =$ *(1)*

$\mathrm{DI_d} = \frac{\mathrm{d_j\text{-}d_{OP}}}{\mathrm{d_U\text{-}d_{OP}}}$$d_j$, d_{op} and d_u are the storey displacement values at the considered performance level, operational level, and permissible displacement at collapse (2.5 % H).

The damage indices dependent on strength.

Base shear is used in strength-based damage interactions (DIs) as a damage variable to signify a loss of strength during a POA. It is possible to utilise strength-based DI to accurately determine the behaviour of a structure during an inelastic displacement excursion. This may then lead to an objective assessment of the amount of damage that a structure has sustained. The following formula may be used to compute DIs that are based on the strength of an opponent:

For example, in POA, the base shear values are defined as V (performance level), Vop (operation level), and Vmax (maximum base shear). The formula for the strength-based DIs is as follows:

$\frac{Vj\ \ -\ Vop}{Vmax-Vop}$$DI =$ (2)

$\mathrm{DI_s} = \frac{\mathrm{V_j\text{-}V_{OP}}}{\mathrm{V_{max}\text{-}V_{OP}}}$

Where; VV, $\mathrm{V_{OP}}$, V_{op} *and Vmax* $\mathrm{V_{max}}$are the base shear values at the considered performance level, operational level, and maximum base shear observed in POA.

Damage index based on stiffness

$1 - \frac{Kj}{Kop}$The damage value is computed by matching the structure's stiffness at any defined performance level to its operational stiffness, according to the suggested damage indicator. Eq. describes the DI depending on the stiffness

$DI_k =$ (3)

K_j and K_{op} denote stiffness values at the system's evaluated performance level and operational level, respectively.

Many more EDPs may be used to estimate damage worth, but only a few are mentioned here, and the rest are left open for future research.

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FINDINGS AND DISCUSSION

The response and damage states of MRFs representing low, medium, and high-rise buildings were examined. For example, the MRF contains stories of Low, Medium and High-Rise, as indicated in Figure 3. Nonlinear static analysis was performed on the sample MRF (POA). First, second, and third-generation procedures were utilised to assess example MRF's response. Analytical modelling of the sample structures was performed using SAP 2000 V 21.0. One bay, three bays, and five bays, each measuring three meters in width, are shown in the MRF example in three different levels (figure 3): low rise, medium rise, and high rise. The example MRF was designed using IS 456:2000, IS 1893:2002 [Part 1], and IS 13920:1996. There were lateral stresses on the frame. The MRF may be found in seismic zone V, which has a z-value of 0.36 and a relevance factor of 1. (Soil type).

The material characteristics of RC sections, as shown in Table 1 and Figure 1, align with (Kaveh et al., 2020), who emphasised material properties' role in optimising RC frame performance. Their study highlights how material selection impacts strength and sustainability, complementing your findings by demonstrating the importance of balancing efficiency and performance in structural design.

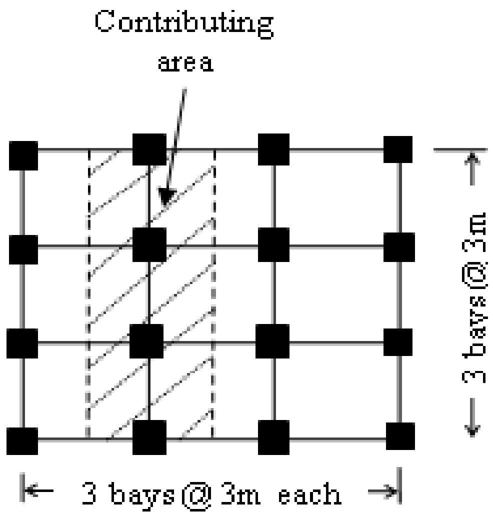

Figure 1: Typical Plan and Elevation of example S3B3 MRF

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Table 1: Material attributes taken into consideration throughout the design process of the sample MRF [IS 456, IS 1786]

Material Characteristic	M 25 Grade of Concrete	Steel Fe 415 grade
Weight per volume unit [kN/m^3]	25	76.97
Volume/mass [kN/m^3]	2.548	7.849
Elastic modulus of the material [kN/m^2]	25E+06	2E+08
Characteristic strength [kN/m^2]	25000 [for 28 days]	415000 [yield]
Tensile strength [Minimum] [kN/m^2]	-	485800
Yield Strength [Expected] [kN/m^2]	-	456500
Tensile Strength [Expected] [kN/m^2]	-	533500

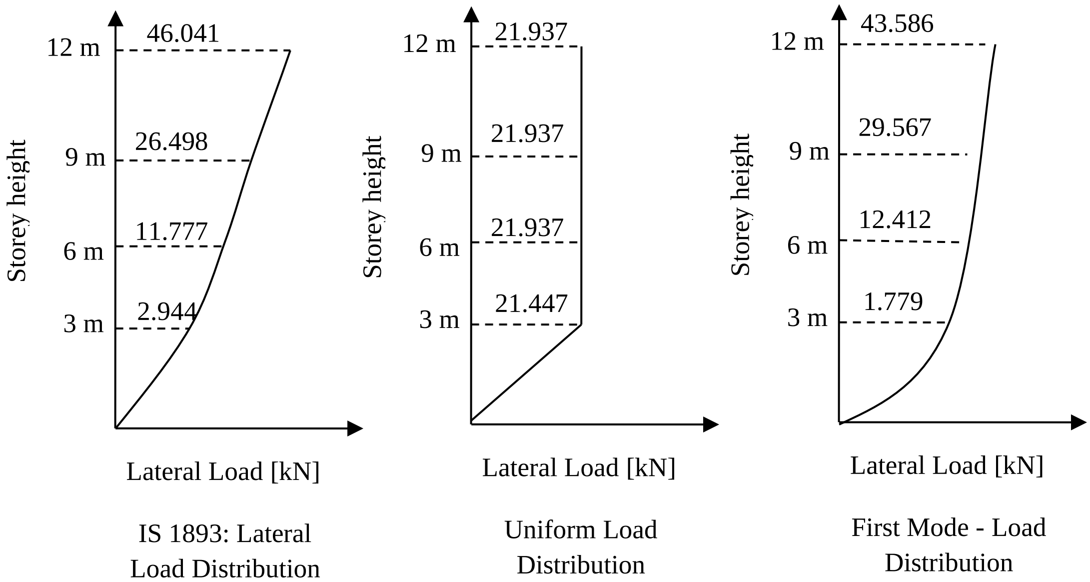

Figure 2: Depicts a variety of lateral load patterns applied in POA to the example MRFs.

In POA, the first mode, uniform load distribution, and the lateral load pattern established following IS1893 are all applied. Figure 2 shows a range of lateral load patterns used.

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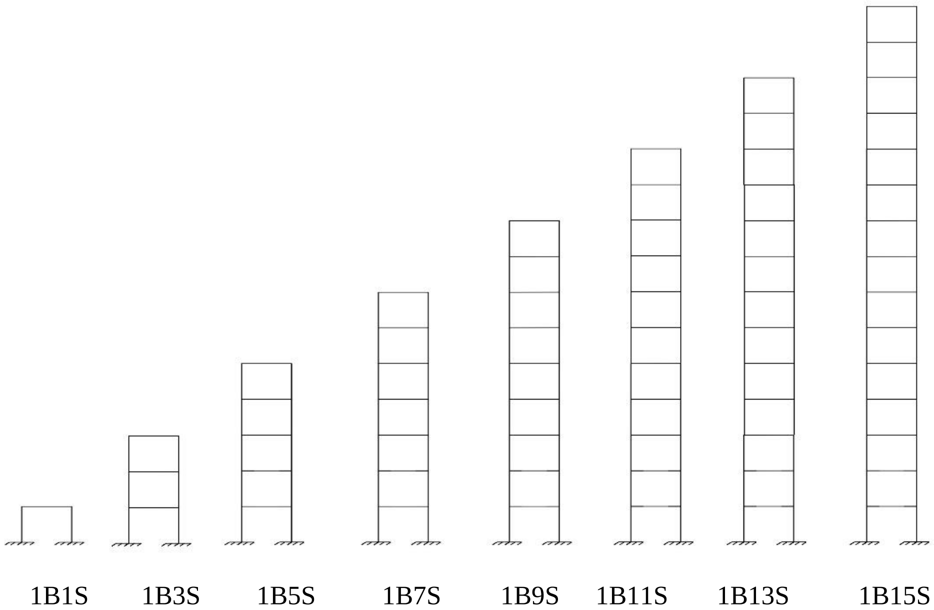

Figure 3a: Series-I: One bay MRFs, MRFs

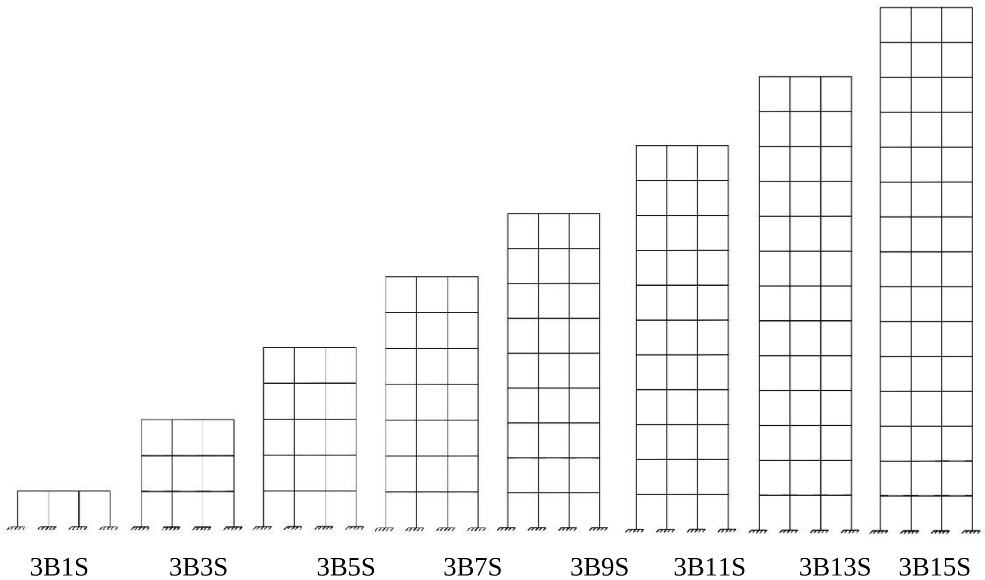

Figure 3b: Series-II: Three bay MRFs

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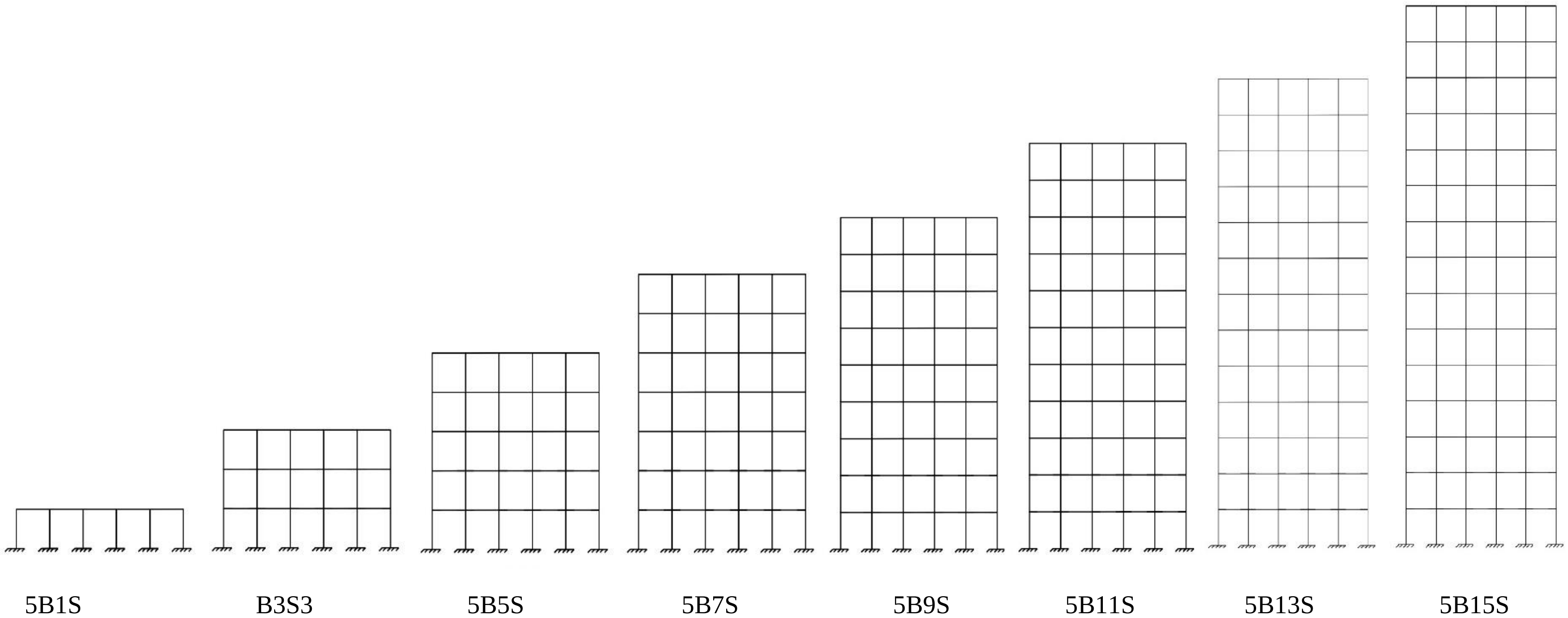

Figure 3c: Three bay MRFs and (c) Series-III: Five bay MRFs

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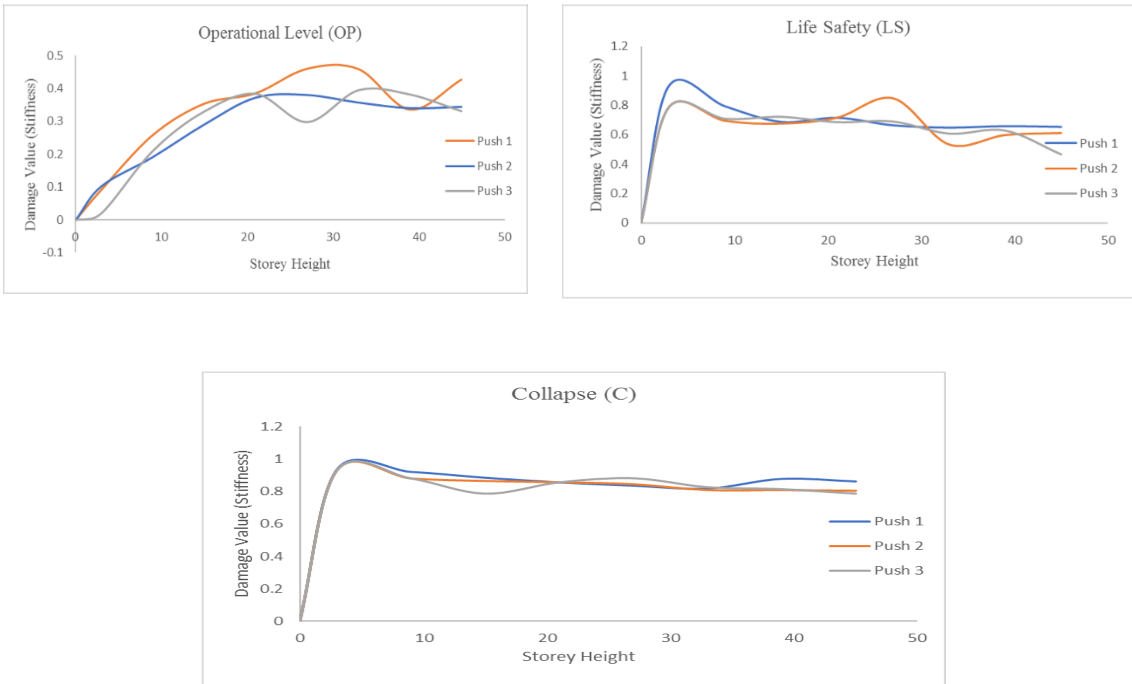

Figure. 4: Damage variations in Series-II MRFs with three storeys subjected to different Push Load cases.

The findings from the Push 3, Push 1, and Push 2 loading scenarios, which reveal distinct lower, upper, and median bounds for Moment-Resisting Frames (MRFs), align with observations by (Habibi et al., 2013). Their study on inelastic damage analysis of RC-MRFs using the pushover method emphasised the importance of simulating multiple loading patterns to capture the full range of inelastic responses. The ductile failure mechanisms observed during plastic hinge formation, as shown in Figure 5, resonate with their findings on the critical role of hinge mechanisms in determining structural performance.

The approximately 10% drift reduction in Push 2 compared to Push 3 can be attributed to the nonlinear yielding of higher stories, which (Habibi et al., 2013) also identified as a key factor in lateral load responses of RC structures. As this study suggests, they advocate for using varied loading scenarios to ensure a comprehensive evaluation of the structure's inelastic response and collapse mechanisms. This agreement underscores the necessity of multiple pushover analyses for reliable damage assessment.

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Table 2 shows examples of MRFs exposed to POA for various Push load scenarios. The damage values were calculated using examples of MRFs with varying performance levels.

Table 2: Collapse Mechanism of Example MRFs subjected to Push 1 Load case

Operational Level (OP						
Series I (Low Rise)			Series II (Medium Rise)		Series III (High Rise)	
Storey Height	Damage Value (Strength)	Damage Value (Stiffness)	Damage Value (Strength)	Damage Value (Stiffness)	Damage Value (Strength)	Damage Value (Stiffness)
0	0	0	0	0	0	0
3.000	0.343	0.092	0.789	0.103	0.567	0.020
9.000	0.591	0.258	0.766	0.192	0.736	0.210
15.000	0.553	0.353	0.747	0.291	0.747	0.330
21.000	0.681	0.385	0.724	0.373	0.727	0.384
27.000	0.774	0.459	0.663	0.381	0.577	0.298
33.000	0.713	0.458	0.603	0.358	0.613	0.395
39.000	0.790	0.335	0.611	0.341	0.610	0.382
45.000	0.795	0.426	0.620	0.345	0.526	0.331

Table 3: Collapse Mechanism of Example MRF subjected to Push 2 Load case

Life Safety (LS)						
Series I (Low Rise)			Series II (Medium Rise)		Series III (High Rise)	
Storey Height	Damage Value (Strength)	Damage Value (Stiffness)	Damage Value (Strength)	Damage Value (Stiffness)	Damage Value (Strength)	Damage Value (Stiffness)
0	0	0	0	0	0	0
3.000	0.991	0.934	0.926	0.789	0.926	0.788
9.000	0.916	0.793	0.870	0.693	0.859	0.708
15.000	0.910	0.687	0.840	0.674	0.871	0.721
21.000	0.865	0.715	0.854	0.714	0.865	0.685
27.000	0.864	0.664	0.900	0.846	0.787	0.689
33.000	0.856	0.648	0.928	0.532	0.812	0.607
39.000	0.950	0.658	0.808	0.596	0.819	0.628
45.000	0.992	0.653	0.811	0.610	0.876	0.466

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Table 4: Collapse Mechanism of Example MRF subjected to Push 3 Load case

Collapse (C)						
	Series I (Low Rise)		Series II (Medium Rise)		Series III (High Rise)	
Storey Height	Damage Value (Strength)	Damage Value (Stiffness)	Damage Value (Strength)	Damage Value (Stiffness)	Damage Value (Strength)	Damage Value (Stiffness)
0	0	0	0	0	0	0
3.000	0.991	0.934	0.994	0.931	0.995	0.930
9.000	0.901	0.921	1.000	0.882	1.000	0.881
15.000	1.000	0.885	1.000	0.866	1.000	0.787
21.000	0.996	0.856	1.000	0.858	1.000	0.857
27.000	0.998	0.836	1.000	0.846	0.994	0.882
33.000	1.000	0.818	1.075	0.810	0.995	0.827
39.000	0.909	0.878	0.995	0.810	0.994	0.815
45.000	0.927	0.862	0.993	0.806	1.000	0.787

Table 5: Nonlinear responses of example MRF in reference to various PBSE methods

Sr. No.	PBSE Method	Push 1 Base Shear (kN)	Push 1 Displ. (m)	Push 2 Base Shear (kN)	Push 2 Displ. (m)	Push 2 Base Shear (kN)	Push 2 Displ. (m)
1	ATC 40 (CSM)	234.48	0.064	340.28	0.050	269.72	0.059
2	FEMA 440 (CSM)	241.61	0.075	344.86	0.058	276.96	0.072
3	FEMA 356 (DCM)	251.26	0.095	346.39	0.073	283.18	0.088
4	FEMA 440 (DCM)	256.52	0.125	347.56	0.084	286.02	0.109

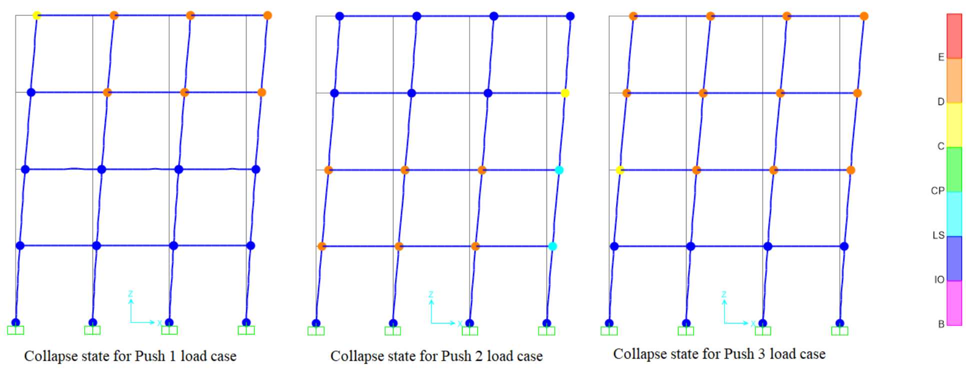

Figure 5: The plastic hinge mechanism of the S3B3 MRF in its collapsed form. It was designed to accommodate a variety of load scenarios.

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This study's use of nonlinear responses from Performance-Based Seismic Evaluation (PBSE) approaches, incorporating displacements and base shear to estimate Damage Indices (DIs), aligns with findings by (Zameeruddin & Sangle, 2017). Their work highlights the importance of nonlinear analysis for assessing seismic performance, mainly through the behaviour of plastic hinges as performance indicators. The two performance levels, PL1 and PL2, used in this study to evaluate the Moment Resisting Frame (MRF) under different lateral load patterns agree with their emphasis on identifying critical thresholds through hinge behaviour. Both studies demonstrate the value of nonlinear responses for accurate damage assessment in structural evaluations.

Table 6: The performance levels, as well as the damage and drift restrictions that correspond to them [Ahmed Ghobarah 2001]

Performance	Damage state	Drift	Performance Level
Fully operational, immediate occupancy	No damage	<0.32%	**Performance Level-1 [PL-1]**
Operational, damage control, Moderate	Repairable	<0.5%	
Life in a non-harmful condition.	Irreparable	<1.5%	**Performance Level-2 [PL-2]**
Near collapse, limited safety, hazard reduce	Severe	<2.5%	
Collapse		>2.5%	

PL1 refers to performance levels for which the limits of drifts are listed in Table 6. However, PL2 refers to performance levels for which the limits of drifts are not included. A-B level plastic hinges are believed to be in OP, and subsequent falls between B-C level plastic hinges are utilised to designate IO, LS, and CP levels. In Tables 2-4, such identifications are described. The DI's represent a loss of drift, strength, and stiffness. It is possible to identify damage using many more engineering criteria. However, this study has been confined to a specific DI and has left room for future research into additional DI's.

The damage thresholds and drift values presented in Figure 07 and Table 6, along with DI values of 0.59 and 1.0 for PL1 and PL2, align with findings by (Zameeruddin & Sangle, 2017), who conducted nonlinear static analyses of reinforced concrete (RC) structures. They emphasised the importance of drift limits and damage indices in evaluating structural performance, noting that lower thresholds at initial performance levels indicate early vulnerability, consistent with your findings for PL1. Additionally, the observed balance between compressive and tensile forces in RC sections, influenced by steel reinforcement

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quality, resonates with their conclusions about the role of material properties in improving ductility under seismic loading.

While Zameeruddin and Sangle (2017) performed case-specific analyses to derive detailed seismic responses, your study focuses on identifying areas for improvement without detailed case studies. This difference in scope highlights the need for further research to comprehensively address factors such as optimising steel quality and section design to enhance ductility. By linking your results to their findings, the study reinforces the significance of stiffness degradation and drift control in structural resilience while also pointing toward the necessity of further investigation into RC section behaviour to achieve a comprehensive understanding of damage mechanisms.

Table 7: Calculation of Drift-based DI value

	Push 1				Push 2				Push 3			
Performance Level	d_{op}	d_j	d_u	DI_d	d_{op}	d_j	d_u	DI_d	d_{op}	d_j	d_u	DI_d
OP	0.014	0.014	0.30	0.000	0.010	0.010	0.30	0.000	0.010	0.010	0.30	0.00
B-IO	0.014	0.015	0.30	0.002	0.010	0.013	0.30	0.010	0.010	0.014	0.30	0.015
IO-LS	0.014	0.101	0.30	0.302	0.010	0.105	0.30	0.328	0.010	0.105	0.30	0.330
C-D	0.014	0.278	0.30	0.924	0.010	0.286	0.30	0.951	0.010	0.286	0.30	0.951
D-E	0.014	0.278	0.30	0.924	0.010	0.286	0.30	0.951	0.010	0.286	0.30	0.951

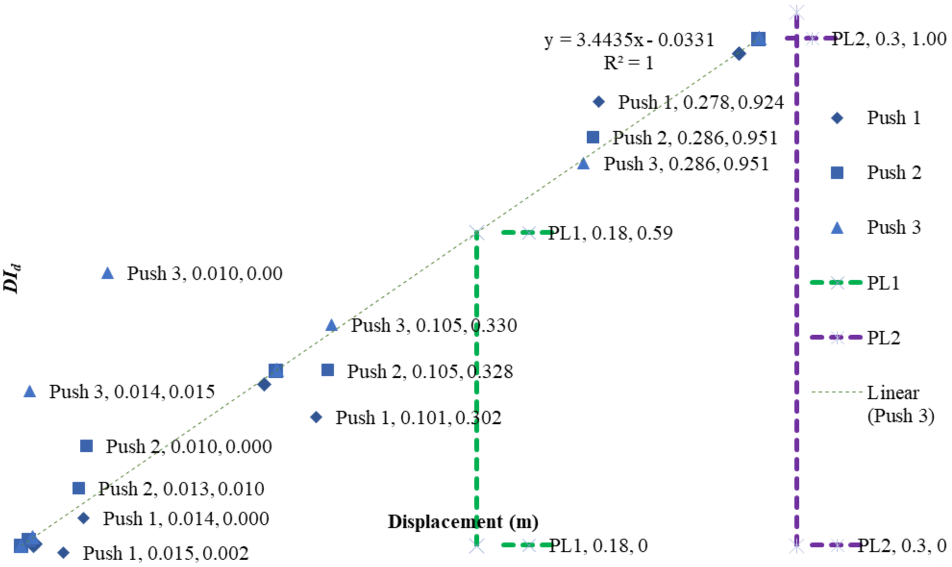

Figure 6: Drift-based DI of example MRF for PL1 and PL2

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The damage indices (DIs) presented in Table 08 and Figure 07, with values of 0.69 and 0.98 for PL1 and PL2 under the Push 3 load case, align with findings by (Zameeruddin and Sangle, 2017). Their nonlinear static analysis of RC structures similarly identified variations in damage indices across different load cases, showing how specific loading scenarios can act as upper and lower bounds for structural performance. The observed trends in your study, where Push 1 forms the upper bound and Push 3 represents the lower bound, reflect their findings that seismic load scenarios influence the extent of nonlinear responses, dependent on material properties and force distributions.

While (Zameeruddin and Sangle, 2017) provided detailed case-specific evaluations, your study complements this by focusing on broader trends using linearisation techniques. Both studies emphasise the need to evaluate multiple loading scenarios to capture a structure's full range of performance and damage potential.

Table 8: Calculation of Strength-based DI value

	Push 1				**Push 2**				**Push 3**			
Performance Level	V_{op}	V_j	V_{max}	DI_s	V_{op}	V_j	V_{max}	DI_s	V_{op}	V_j	V_{max}	DI_s
OP	159.6	159.6	268.4	0.000	151.5	151.5	368.9	0.000	120	120.06	300.2	0.00
B-IO	159.6	167.1	268.4	0.069	151.5	198.8	368.9	0.218	120	173.99	300.2	0.299
IO-LS	159.6	253.3	268.4	0.861	151.5	349.8	368.9	0.912	120	284.70	300.2	0.914
C-D	159.6	268.4	268.4	1.000	151.5	368.9	368.9	1.000	120	300.22	300.2	1.000
D-E	159.6	210.2	268.4	0.465	151.5	304.6	368.9	0.704	120	256.58	300.2	0.758

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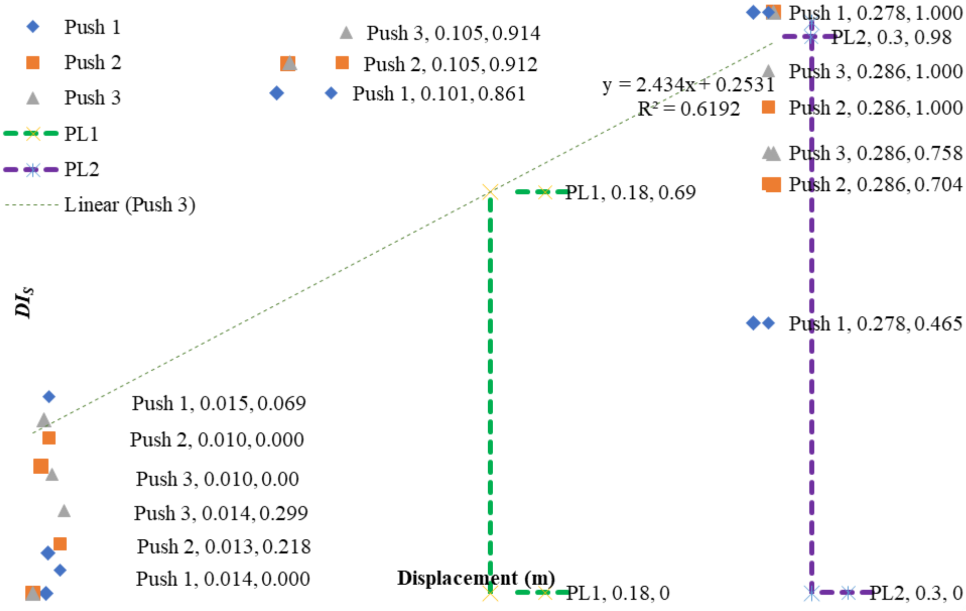

Figure. 7: Strength-based DI of example MRF for different push load cases

The reduction in stiffness under incremental loading, as observed in study and presented in Figure 09 and Table 09, aligns with structural mechanics principles where stiffness degradation indicates progressive damage. The damage indices (DIK) at PL1 and PL2, identified as 0.65 and 1.0 for the Push 3 load case, highlight critical thresholds in structural performance. This approach resonates with (Atakok et al., 2022), who explored how mechanical properties of recycled 3D-printed filaments degrade under environmental and loading conditions, emphasising stiffness as a key factor in assessing material performance. Similarly, (Kam et al., 2023) used linear correlations to study mechanical property variations in 3D-printed materials, showcasing the utility of linearised models, like DIK, for predictive damage assessments. These parallels validate stiffness-based damage evaluation as an effective tool for structural integrity analysis.

However, a key contrast lies in methodologies. While your study employs DIK linearisation to predict damage, the referenced works, such as (Çevik & Kam, 2023), focus on experimental evaluations, particularly for FDM-printed products. Their findings on the mechanical variability of composite filaments under different conditions highlight the need for tailored assessments based on material and loading specifics. By linking stiffness degradation to damage, as shown in Figure 09 and Table 09, your findings extend these insights into structural applications, emphasising its practicality for predictive performance assessment. Future exploration could bridge these approaches by applying DIK-based methods to emerging

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materials, such as 3D-printed composites, to enhance the accuracy of damage prediction models.

Table 09: Comparison of drift limits for various Push load cases

PUSH 1 Load Case

Performance Level	Drift Limit in % (Permissible)	**Series 1 (Low Rise)**							
		S1B1	S3B1	S5B1	S7B1	S9B1	S11B1	S13B3	S15B1
OP	0.2	0.00	0.01	0.01	0.01	0.05	0.67	0.04	0.05
LS	1.5	0.07	0.22	0.34	0.32	0.46	1.36	0.65	0.73
C	>2.5	0.23	0.69	1.15	1.46	1.49	3.91	2.53	2.39
Performance Level	Drift Limit in % (Permissible)	**Series 2 (Medium Rise)**							
		S1B3	S3B3	S5B3	S7B3	S9B3	S11B3	S13B3	S15B3
OP	0.2	0.00	0.01	0.02	0.02	0.02	0.00	0.04	0.07
LS	1.5	0.06	0.19	0.30	0.36	0.49	0.05	0.77	0.94
C	>2.5	0.23	0.68	1.18	1.63	2.08	1.36	2.93	3.53
Performance Level	Drift Limit in % (Permissible)	**Series 3 (High Rise)**							
		S1B5	S3B5	S5B5	S7B5	S9B5	S11B5	S13B3	S15B5
OP	0.2	0.00	0.01	0.01	0.02	0.02	0.04	0.06	0.01
LS	1.5	0.06	0.19	0.28	0.39	0.43	0.53	0.57	0.64
C	>2.5	0.23	0.69	1.13	1.64	1.62	2.35	2.23	2.68

PUSH 2 Load Case

Performance Level	Drift Limit in % (Permissible)	**Series 1 (Low Rise)**							
		S1B1	S3B1	S5B1	S7B1	S9B1	S11B1	S13B3	S15B1
OP	0.2	0.00	0.01	0.01	0.01	0.04	0.57	0.03	0.05
LS	1.5	0.07	0.22	0.34	0.35	0.55	1.54	0.75	0.83
C	>2.5	0.35	0.79	1.25	1.56	1.69	3.01	2.65	2.59
Performance Level	Drift Limit in % (Permissible)	**Series 2 (Medium Rise)**							
		S1B3	S3B3	S5B3	S7B3	S9B3	S11B3	S13B3	S15B3
OP	0.2	0.00	0.01	0.02	0.02	0.02	0.00	0.03	0.06
LS	1.5	0.07	0.20	0.35	0.40	0.49	0.05	0.77	0.95
C	>2.5	0.25	0.70	1.28	1.69	2.58	1.46	2.93	3.60
Performance Level	Drift Limit in % (Permissible)	**Series 3 (High Rise)**							
		S1B5	S3B5	S5B5	S7B5	S9B5	S11B5	S13B3	S15B5
OP	0.2	0.00	0.12	0.15	0.02	0.04	0.04	0.06	0.01
LS	1.5	0.02	0.29	0.30	0.39	0.45	0.54	0.60	0.65
C	>2.5	0.25	0.70	1.17	1.69	1.64	2.39	2.29	2.60

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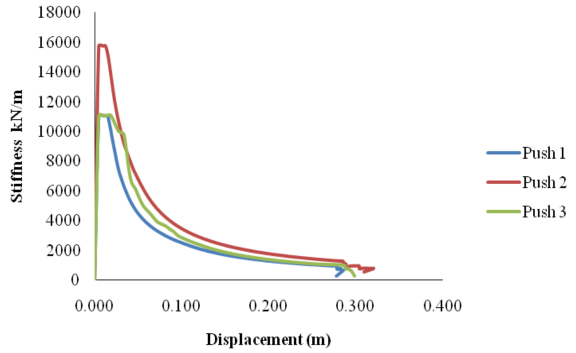
PUSH 3 Load Case

Performance Level	Drift Limit in % (Permissible)	**Series 1 (Low Rise)**							
		S1B1	S3B1	S5B1	S7B1	S9B1	S11B1	S13B3	S15B1
OP	0.2	0.00	0.01	0.01	0.01	0.05	0.60	0.04	0.05
LS	1.5	0.08	0.32	0.44	0.45	0.65	1.84	0.85	0.93
C	>2.5	0.45	0.89	1.35	1.66	1.79	3.11	2.75	2.69
Performance Level	Drift Limit in % (Permissible)	**Series 2 (Medium Rise)**							
		S1B3	S3B3	S5B3	S7B3	S9B3	S11B3	S13B3	S15B3
OP	0.2	0.01	0.07	0.01	0.03	0.03	0.00	0.04	0.07
LS	1.5	0.05	0.09	0.55	0.50	0.59	0.05	0.87	0.95
C	>2.5	0.35	0.80	1.38	1.79	2.78	1.66	2.93	3.80
Performance Level	Drift Limit in % (Permissible)	**Series 3 (High Rise)**							
		S1B5	S3B5	S5B5	S7B5	S9B5	S11B5	S13B3	S15B5
OP	0.2	0.00	0.22	0.25	0.03	0.05	0.05	0.07	0.05
LS	1.5	0.02	0.29	0.30	0.59	0.55	0.56	0.70	0.85
C	>2.5	0.27	0.70	1.17	1.69	1.84	2.49	2.39	2.76



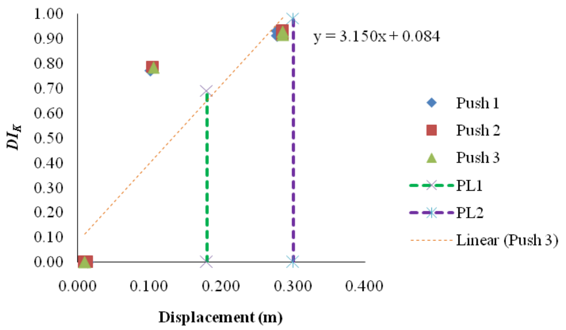


Figure 8: Loss of stiffness during POA for different load case

Figure 9: Stiffness-based DI of example MRF push load cases

CONCLUSION

The findings from this research have significant practical implications in earthquake-resistant design by providing preliminary damage estimates for tall buildings, which help reduce repair costs and ensure public safety. The Performance-Based Seismic Design (PBSD) approach meets acceptance criteria for immediate occupancy and life safety across various seismic intensities, offering an innovative solution to the limitations of traditional nonlinear dynamic

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analysis and making it easier and less labour-intensive for structural engineers to adopt in practical applications.

This study contributes to safer urban environments by enhancing the resilience of buildings against earthquakes, reducing potential fatalities, economic losses, and downtime due to earthquake-induced damage, thus addressing a critical public safety concern.

The originality of this study lies in its novel approach that integrates both performance and damage parameters for rapid damage assessment using parameters derived from Pushover Analysis (POA). Unlike traditional methods that rely on nonlinear dynamic analysis, often too labour-intensive for widespread practical use, this approach leverages Performance-Based Seismic Evaluation (PBSE) techniques and Nonlinear Static Procedures (NLSP) to simplify damage assessment while maintaining accuracy. The development of performance level indicators, PL1 and PL2, provides a practical tool for structural design optimization, allowing engineers to better correlate building performance levels with the corresponding degree of damage. This advancement is particularly beneficial for reinforced concrete structures, enhancing the precision, cost-effectiveness, and accessibility of earthquake-resistant design.

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Appendix I: Abbreviations

S	Storeys
B	Bays
ATC	Applied Technological Council
CSM	Capacity Spectrum Method
DCM	Displacement CoefficientMethod
EDPs	Engineering Demand Parameters
PBSE	Performance-based Seismic Evaluation
PBSA	Performance-based Seismic Assessment
POA	Pushover Analysis
MRF	Moment resisting frame
NLSP	nonlinear static pushover analysis

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