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Evaluation of Reduction Factors for Vertically Irregular RC Frames: Conventional vs. Adaptive Pushover Analysis

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Abstract: The significant computational demand of Nonlinear Time-History Analysis (NTHA) often leads seismic design practitioners to opt for the more efficient Nonlinear Static Pushover Analysis. However, conventional pushover methods, typically reliant on the first-mode pattern, fail to capture the significant influence of higher modes. This limitation becomes particularly critical in vertically irregular structures, where geometric discontinuities can amplify higher-mode effects, leading to an inaccurate assessment of seismic performance. This study addresses this gap by employing Displacement-Based Adaptive Pushover Analysis (DAP) to account for these higher-mode effects. We investigated five distinct configurations of vertically irregular reinforced concrete (RC) moment-resisting frames to evaluate and compare the seismic reduction factors derived from both conventional Static pushover analysis first mode pattern CSPA and DAP, benchmarking these results against the prescribed limits in Eurocode 8. The findings are compelling: geometric irregularities located near the base of the structure lead to a substantial discrepancy between the reduction factors calculated by DAP and CSPA. Critically, the adaptive analysis consistently yielded significantly lower and more conservative reduction factors, which frequently fell below the code's minimum values. This result suggests that current code provisions may dangerously overestimate the seismic capacity and ductility of such structures, masking their true vulnerability. The study concludes that for vertically irregular frames, particularly those with significant discontinuities in lower stories, adaptive pushover analysis is a necessity for achieving a safe and realistic seismic performance assessment.

1. Introduction

The seismic design philosophy of reinforced concrete (RC) structures has undergone significant evolution following major seismic events, particularly after the devastating 1992 Cairo earthquake. This pivotal event catalysed rapid advancements in the Egyptian seismic

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code provisions, emphasizing the critical need for ensuring ductile behaviour and reliable seismic performance of RC structures under potential future earthquake scenarios. Nonlinear time-history analysis is the most refined tool that can be used for simulating the seismic response of structures[1]. Clearly, in the case of examining the results of shake table tests, time-history analysis with applied base excitation can be used for direct comparison. In practice, however, the choice of seismic records and the large amount of data generated limit its use to specialist applications. In this context, the use of nonlinear static, or pushover, analysis for seismic assessment and design has increased significantly in recent years. It can be employed to assess overall capacity and stability, and to identify the likely plastic mechanisms and associated dissipative regions. The attractiveness of pushover analysis stems mainly from its relative simplicity, in terms of modelling and computational demands as well as interpretation of results, in comparison with nonlinear dynamic analysis[2], [3], [4]

Nonlinear static or pushover analysis involves a nonlinear static assessment of the structure conducted under constant gravity loads and gradually increasing the horizontal loads of the seismic action, provides insight into internal forces, deformation, failure mechanisms, yield order, plastic hinge formation, and overall capacity[2]. Pushover analysis allows for determining the capacity of a structure to withstand horizontal loads. Becoming a widespread analysis method, it is considered a powerful tool for assessing complex unreinforced masonry structures [5]. It has been integrated into contemporary seismic codes such as Eurocode 8[6] and the American code ASCE 7-16[7]. Pushover analysis is often preferred due to its ability to reduce computational costs and avoid the complexities inherent in dynamic time history analysis. Standard pushover analysis often uses a single load pattern based on the first mode shape, which may not capture the influence of higher modes, especially in structures with vertical irregularities [8].Adaptive and multi-mode pushover methods (e.g., Modal Pushover Analysis) have been proposed to address this limitation by allowing for the influence of higher modes.[9]. In displacement-based adaptive pushover, story forces can reverse sign due to modal forces from higher modes, which must be considered for accurate results [10].Recent studies emphasize the need for adaptive techniques to better account for higher mode contributions in irregular or complex structures[9], [11], [12], [13], [14]

Structures with vertical irregularities (e.g., setbacks, changes in stiffness or mass along height) are more sensitive to higher mode effects. Adaptive pushover methods and multi-mode approaches are particularly beneficial for these structures, as they can more accurately estimate seismic demands by considering the changing dynamic properties and mode contributions [12], [13], [15], [16]. Central to modern seismic design methodology is the response modification factor (R-factor), a fundamental parameter that bridges the gap between linear elastic analysis and the complex nonlinear behaviour exhibited by structures during seismic events. The R-factor, known by various terminologies across different international codes "response modification coefficient" in ASCE 7-16 [7]] , "behaviour factor (q)" in Eurocode 8 [6], "response reduction factor" in the Indian seismic code (IS-1893) [17], and "response modification factor" in ECP-201[18]serves as a crucial design parameter that enables engineers to account for the beneficial effects of structural overstrength and inelastic energy dissipation capacity while maintaining the computational simplicity of linear elastic analysis. Contemporary seismic codes typically establish R-factor values for RC moment resisting frames (MRFs) based primarily on the anticipated ductility level of the structural system. For instance, ECP-201[18] specifies R-factor values of 5 and 7 for RC-MRFs designed with limited and adequate ductility, respectively. Similarly, ASCE 7-16 [7] provides a broader range of R-factor values from 3 to 8, while EC8 [6] suggests values between 1.5 and 6.5, all contingent upon the expected ductility level of the designed frames. The Indian seismic code (IS-1893)[17] adopts R-factor values of 3 and 5 for ordinary and special RC-MRFs, respectively.

The fundamental premise underlying the R-factor approach lies in its ability to reduce design lateral forces by capitalizing on the structure's inherent capacity to dissipate seismic energy through controlled inelastic deformation and its reserve strength beyond the design level. However, current seismic design codes predominantly adopt a simplified approach by prescribing constant R-factor values regardless of the structure's geometric configuration, material properties, or dynamic characteristics. This oversimplification is further compounded by the fact that some codes, including EC8 and ECP-201 [18]. Mwafy and Elnashi, [19] comprehensive research examining seismic force reduction factors in modern building codes revealed that current standards may be overly conservative. Through extensive analysis of twelve medium rise reinforced concrete buildings using over 1,500 simulations, the study demonstrated that Eurocode 8's R factors could be safely increased, particularly for regular frame structures. The research emphasized the critical importance of incorporating both shear failure modes and vertical ground motion when calculating these factors.

Abou-Elfath and Elhout [20] conducted a comprehensive evaluation of response modification factors (R-factors) for reinforced concrete moment-resisting frames designed according to Egyptian code provisions, examining nine different geometric configurations through both static pushover and earthquake time-history analyses. Their investigation revealed that R-factors are significantly influenced by structural height, with values decreasing from 9.85 to 6.98 for 3-story to 12-story frames respectively, while bay configuration parameters showed minimal impact on R-factor values. The study found that calculated R-factors ranged from 6.18 to 9.85 for static analysis and 5.43 to 8.43 for dynamic analysis, with the minimum values falling below the Egyptian code-specified R-factor of 7 for adequately ductile RC-MRFs. Importantly, the research demonstrated that earthquake analysis consistently yielded lower R-factors compared to static pushover analysis due to the concentration of story drifts under dynamic loading, [21]highlighting potential non-conservative aspects of current code provisions and the need for geometry-dependent R-factor specifications in seismic design codes.

Many buildings feature architectural designs that don't adhere to uniform configurations, which can lead to vulnerabilities. These irregular configurations have been linked to early structural failures during past earthquakes. Therefore, thorough analysis is essential to ensure strong performance, even when structural components are not ideally arranged for seismic activity. Vertical irregularities involve sudden changes in a building's geometry, stiffness, and mass as you move up to its height, posing significant challenges to structural integrity.[22], [23], [24], [25]The significant effects of the plan configuration irregularity on the seismic demands that necessitate an integrated cooperation between the architect and structural engineer from the earliest planning phase of building to guarantee structural safety and reduce vulnerability. The study compares conventional pushover and Adaptive Force-Based Multimode Pushover Analysis (AFMP). AFMP accounts for higher mode effects and lateral load changes, yielding higher seismic responses on upper floors compared to conventional methods, which resulted in Life Safety performance level., the study compares conventional pushover and AFMP. AFMP accounts for higher mode effects and lateral load changes, yielding higher seismic responses on upper floors compared to conventional methods, which resulted in Life Safety performance level [8]. Displacement-Based Adaptive Pushover Analysis (DAP) is specifically designed to account for the effects of higher modes, including sign reversals in story forces—a phenomenon that occurs due to the influence of higher vibration modes. This is achieved by combining modal story displacements to obtain a single lateral displacement distribution, and the resulting story forces (derived from equilibrium) can exhibit sign reversals, reflecting the true dynamic behaviour under seismic loading [12]. Conventional pushover analyses often fail to capture the effects of higher and torsional modes, which are particularly important in structures with vertical irregularities. Adaptive pushover methods, including DAP, have been developed to address these shortcomings by more accurately representing the dynamic response of irregular [26].

This paper presents a displacement-based adaptive pushover method that improves response predictions compared to conventional force-based methods. It demonstrates numerical stability in highly inelastic regions, making it a promising tool for structural assessment in earthquake engineering. While the R-factor approach significantly simplifies the seismic design process by avoiding computationally intensive nonlinear dynamic analyses, it inherently introduces uncertainties in the design methodology. The approximation involved in substituting detailed inelastic earthquake analysis with a single modification factor necessitates comprehensive evaluation and validation of these factors through both experimental investigations and advanced numerical simulations. This validation process requires comparing calculated R-factors with code-specified values to assess their adequacy and reliability. The study investigates the variability in response factors for vertically irregular structures across distinct geometric configurations, including N-shaped (NCIM), O-shaped (OCIM), U-shaped (UCIM), and H-shaped (HCIM) building layouts, benchmarked against a conventional rectangular regular model (RRM). The comparative analysis employs two nonlinear static pushover methodologies: the traditional first-mode approach and an advanced adaptive force-based technique.

1.1 Determining of Response Modification Factors

Response modification factors (R-factors) can be evaluated through two primary analytical methods: static pushover analysis and dynamic time-history analysis. The static pushover method involves applying incremental lateral forces following the load distribution patterns specified in seismic design codes. This analysis employs a displacement-controlled procedure that progressively increases lateral deformation until the structure reaches a predetermined maximum displacement limit. The target displacement level (δ_u) in this investigation was established using the life safety performance criteria outlined in FEMA-273[27] for reinforced concrete frame structures subjected to seismic hazards with a 10% probability of exceedance over 50 years. This hazard level aligns with the design earthquake intensity specified in Egyptian seismic provisions. The life safety performance threshold is quantified through story drift and residual drift limitations, with maximum story drift ratios of 2.0% and residual drift ratios of 1.0% for RC frame systems. Various researchers have proposed computational approaches for R-factor determination [19], [28], [29], [30], [31]. The most comprehensive methodology decomposes the R-factor into four constituent components:

$$R = R_\mu \, R_s \, R_\rho \, R_\xi \tag{1}$$

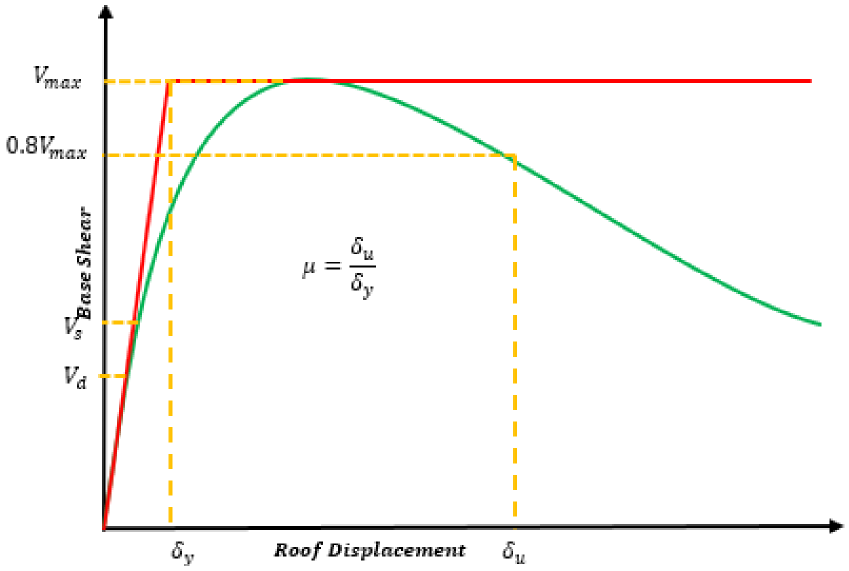

Figure 1. Idealized Nonlinear Static Pushover Curve

The ultimate displacement, a critical parameter in seismic analysis, is determined by a specific performance criterion. Initially, the maximum base shear (Vmax) on the pushover curve is identified. Subsequently, the ultimate displacement (δu) is defined as the roof displacement corresponding to the point on the descending branch where the base shear has degraded to 0.8Vmax. Should the analysis conclude, or the curve terminate at a base shear value exceeding 0.8Vmax, the ultimate displacement is then defined by the final displacement achieved in that analysis. This methodology offers a precise, quantitative approach to assessing a structure's maximum displacement capacity prior to substantial strength degradation. These essential behavioural components are derived from the nonlinear pushover response curve, which is idealized into a bilinear elastic-plastic relationship for simplified analysis.

1.1.1 Quantification of R-Factor Components

The elastic base shear is determined through linear elastic seismic analysis of the structural system. Following Uang's [14] methodology, the ductility reduction factor is expressed as:

$$R_\mu = \frac{V_e}{V_y} \tag{2}$$

Alternative estimation of the ductility reduction factor can be achieved using the structural ductility capacity (μ), fundamental vibration period (T), and seismic characteristics. This investigation employs the empirical relationships developed by Newmark and Hall [18] :

$$\begin{aligned} R_\mu &= 1 && for\, T < 0.2\, s \\ R_\mu &= \sqrt{(2\mu - 1)} && for\, 0.2\, s < T < 0.5\, s \\ R_\mu &= \mu && for\, T > 0.5\, s \end{aligned} \tag{3}$$

The structural ductility capacity (μ) is defined as the ratio of the ultimate displacement δ_u to the yield displacement δ_y, where $\delta_u$is taken at the point where the pushover curve shows a 20% drop in base shear strength methodology discussed in FEMA P695 document Here a variant of FEMA P695 method is used because it is found to be a simple and practical one [16], [17]:

$$\mu = \frac{\delta_u}{\delta_y} \tag{4}$$

The overstrength factor (Rs) represents the ratio between the actual yield capacity and the design-level base shear:

$$R_s = \frac{V_y}{V_d} \tag{5}$$

The redundancy factor (R_ρ) It has been observed that normally local yielding does not result in the failure of the structure. This is because the excess load in a particular element gets distributed among redundant elements which provide reserve strength. This is termed as redundancy:

$$R_\rho = \frac{V_y}{V_s} \tag{6}$$

The damping modification factor (R_ξ) incorporates the influence of supplemental damping systems within the structure and is primarily relevant for buildings equipped with energy dissipation devices. For conventional structures without such systems, R_ξ is assigned a value of 1.0. where δ_u represents corresponding to ultimate displacement response, and δ_y yield displacement. The overstrength factor and the ductility factor determined through static pushover analysis (Equation 4, 5,6). Consequently, the complete response modification factor is expressed as follows:

$$R = \frac{V_y}{V_d} \times \frac{\delta_u}{\delta_y} \times \frac{V_y}{V_s} \tag{7}$$

2. Methodology

The research as showed in figure (2) findings underscore the critical importance of implementing adaptive force-based pushover analysis over conventional loading patterns including triangular, uniform, and first-mode distributions to achieve more accurate quantification of response factors for buildings exhibiting vertical geometric irregularities. The study validates these recommendations through rigorous evaluation using established E8 and ECP201 design codes, thereby providing robust evidence for the superiority of adaptive methodologies in structural assessment.

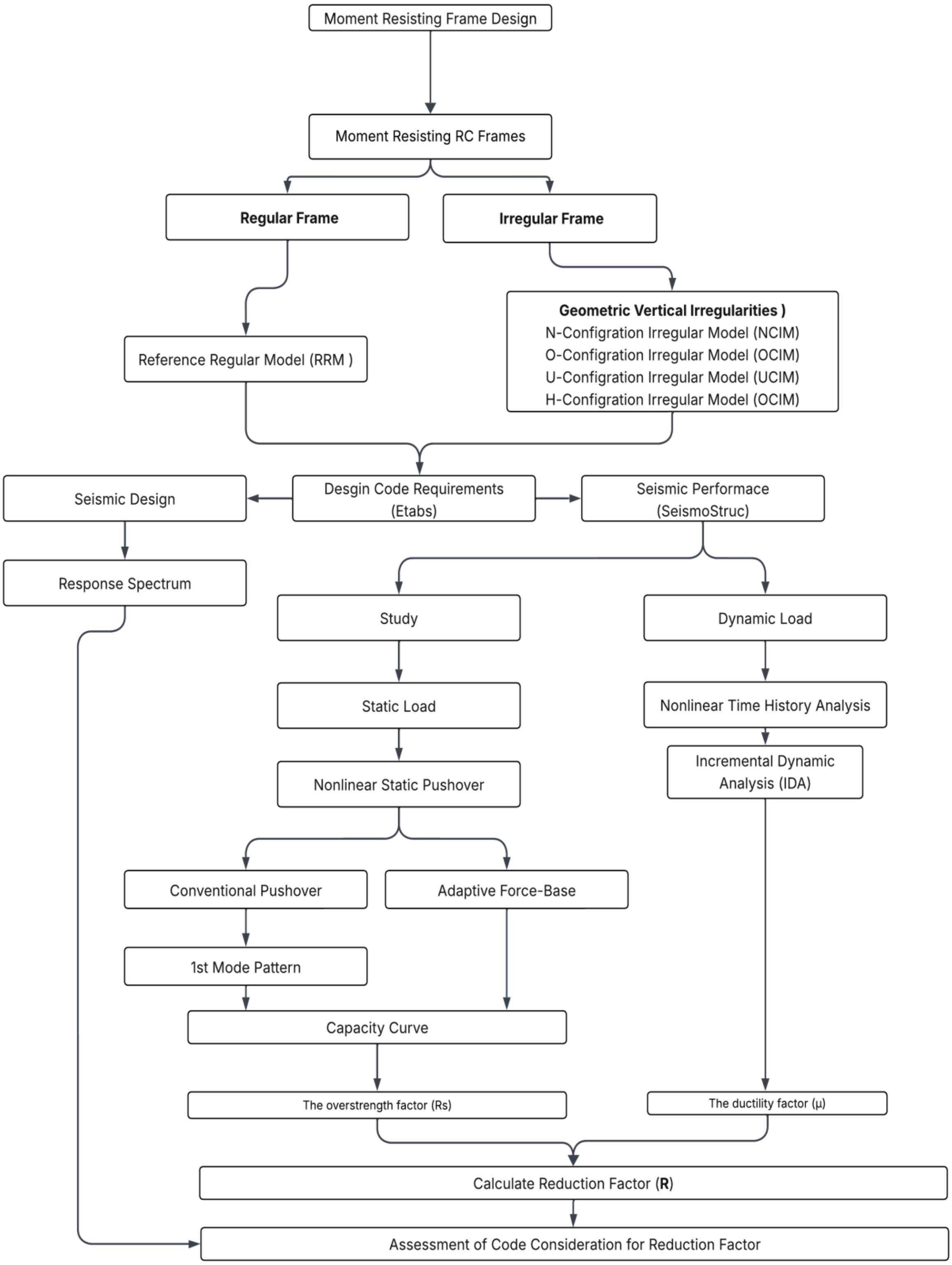

Figure 2. flowchart of the Study methodology

2.1 Example Structure

Many buildings feature architectural designs that don't adhere to uniform configurations, which can lead to vulnerabilities as illustrated in Figure (3). In this study, five distinct building models are considered one regular and four geometrically irregular. Each structure has five bays in the horizontal direction, with each bay measuring 5 m in span, and a total of twelve stories in height. The regular building, referred to here as the Rectangular Regular Model (RRM), serves as a baseline for comparison. The four irregular configurations labelled N-Configuration Model (NCIM), O-Configuration Model (OCIM), U-Configuration Model (UCIM), and H-Configuration Model (HCIM) feature different vertical geometrical irregularities, such as recesses or protrusions in various parts of the exterior frame. These variations in geometry produce notable differences in mass and stiffness distribution, thereby affecting the buildings' seismic behaviour. Figure 4 illustrates the five configurations, highlighting their characteristic shapes and structural layouts

Figure 3. Four Seasons Jeddah Hotel.

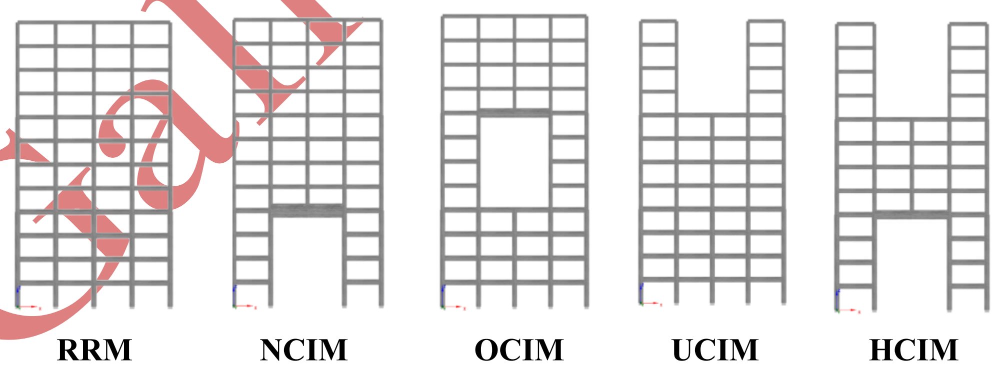

Figure 4. An elevation view of the RC moment-resisting frames considered

The structural design takes into account both mass and load calculations for a reinforced concrete slab system. The concrete has a density of 24 kN/m³. The slab covers an area of 25 m², measuring 5 m by 5 m, and has a thickness of 0.15 m. The dead load calculations include several components: finishes contribute 1.2 kN/m², the self-weight of the slab adds 3.6 kN/m², and services account for 0.5 kN/m². This results in a total dead load of 5.3 kN/m². For the live load, an imposed load of 1.5 kN/m² is applied to the structure. Table 1 shows the dimensions and reinforcement details of the designed MRFs for columns and beams.

Table 1| dimensions and reinforcement details of the designed MRFs for Column and Beams According to EN 1992 (Eurocode 2)

MRFs	Stories	Transfer Beam		Beams		Exterior Column		Interior Column	
		Size (mm)	RFT	Size (mm)	RFT	Size (mm)	RFT	Size (mm)	RFT
RRM	1 - 4	-	-	700X250	5Ø16	600X600	16Ø22	700X700	20Ø22
	5 - 8			600X250	4Ø16	500X500	12Ø20	600X600	16Ø22
	9 - 12			500X250	3Ø16	400X400	12Ø16	500X500	12Ø20
NCIM	1 - 4	1800X700	12Ø25	700X250	5Ø16	600X600	16Ø22	700X700	20Ø22
	5 - 8			600X250	4Ø16	500X500	12Ø20	600X600	16Ø22
	9 - 12			500X250	3Ø16	400X400	12Ø16	500X500	12Ø20
OCIM	1 - 4	1200X600	7Ø25	700X250	5Ø16	600X600	16Ø22	700X700	20Ø22
	5 - 8			600X250	4Ø16	500X500	12Ø20	600X600	16Ø22
	9 - 12			500X250	3Ø16	400X400	12Ø16	500X500	12Ø20
UCIM	1 - 4	-	-	700X250	5Ø16	600X600	16Ø22	700X700	20Ø22
	5 - 8			600X250	4Ø16	500X500	12Ø20	600X600	16Ø22
	9 - 12			500X250	3Ø16	400X400	12Ø16	500X500	12Ø20
HCIM	1 - 4	1200X600	7Ø25	700X250	5Ø16	600X600	16Ø22	700X700	20Ø22
	5 - 8			600X250	4Ø16	500X500	12Ø20	600X600	16Ø22
	9 - 12			500X250	3Ø16	400X400	12Ø16	500X500	12Ø20

2.2 Description of nonlinear analyses

2.2.1 Materials Nonlinearity

The concrete compressive strength members are assumed equal to 38 MPa Pausing Mander et al. nonlinear concrete model However. The minimum yield strength of the reinforcement is also assumed to be equal to 400 MPa Menegotto-Pinto steel model [32] updated by Prota A., Cicco F., Cosenza E [33],Using FEA software SeismoStruc V.25 [34]

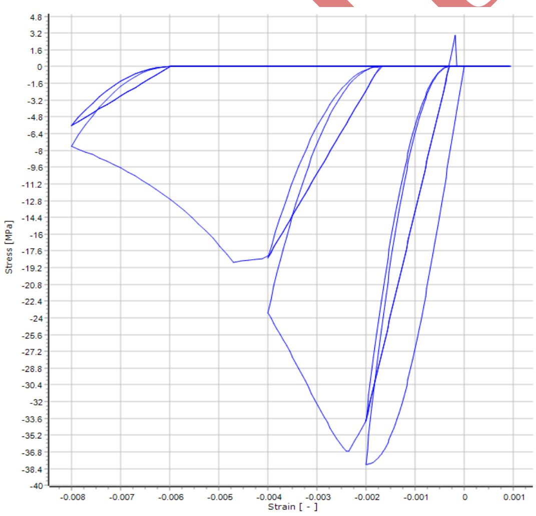

Figure 5. Mander et al. nonlinear concrete

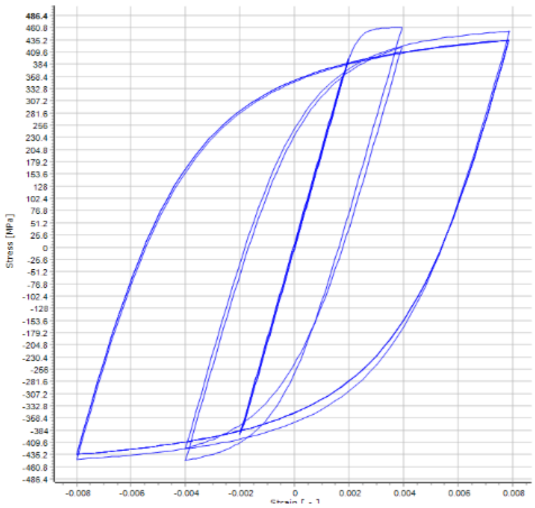

Figure 6. Pinto nonlinear steel model

2.2.2 Elements Connectivity

The nonlinear analysis utilized inelastic force based plastic hinge frame elements for structural connectivity[35]. The fiber discretization was set to 200 fibers per cross-section to ensure accurate representation of the stress-strain distribution across complex sections under high inelasticity levels. Plastic hinge lengths were defined as 16.67% of the respective member lengths, providing controlled localization of inelastic behavior for both beam and column element.

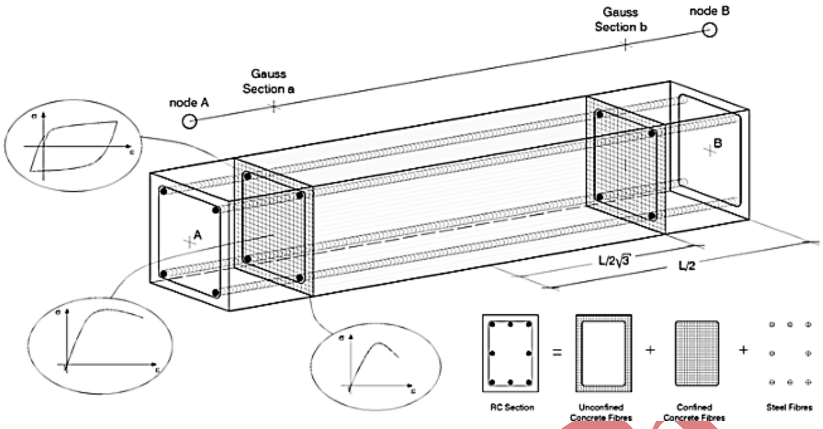

Figure 7. Plastic hinge lengths

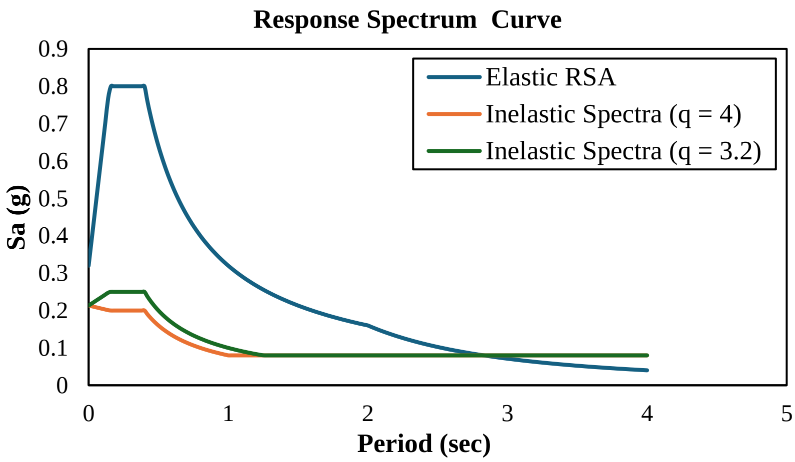

Figure 8. The elastic and inelastic response spectra

2.3 Description of Linear Analyses

In accordance with EN 1998-1 specifications, the behavior factor (q, R) was assigned a value of 4.0 for regular structural systems. For structures exhibiting irregularities, the standard mandates the application of a reduction factor K = 0.8, thereby yielding a reduced behavior factor of 3.2. The constituent components and their respective values were established in strict compliance with the EN 1998-1[36] regulatory framework and EN 1998-1-2 for structures that are regular in both plan and elevation as shown in figure 8. The structural analysis incorporated the DCM (Ductility Class Medium) as recommended in EN 1998-1. The seismic action definition adheres to EN 1998-1 provisions, which recommend seismic intervention for local collapse prevention with a 10% exceedance probability over 50 years for structures of ordinary importance. For this investigation, ground type A and importance class I were utilized with a corresponding importance factor γ = 1.0 to develop Type 1 elastic and design response spectra for elastic analysis in accordance with EN 1998-1. The target peak ground acceleration was established at a_g= 0.4g for linear response spectrum analysis validation purposes. The elastic response spectra parameters were characterized using a damping ratio of ξ = 5% for the analyzed structural models.

3. Results

3.1 Dynamic Characteristics

The natural periods and the elastic mode-shapes for the first three modes of vibration are also shown in Table 2. Similar seismic masses including the dead load plus 20% of the live load are assumed at the floor levels for each building.

Table 2: The natural periods Cumulative modal mass percentages for each mode shape

Mode	(RRM)		(UCIM)		(HCIM)		(OCIM)		(NCIM)	
	T (Sec)	$\sum_{i=1}^{n} m_i$	T (Sec)	$\sum_{i=1}^{n} m_i$	T (Sec)	$\sum_{i=1}^{n} m_i$	T (Sec)	$\sum_{i=1}^{n} m_i$	T (Sec)	$\sum_{i=1}^{n} m_i$
1	1.026	71%	0.883	66.36%	1.009	81.52%	0.950	63%	1.087	81%
2	0.386	14%	0.492	0.00%	0.496	0.00%	0.318	19%	0.411	11%
3	0.221	6%	0.4065	15.52%	0.441	9.11%	0.221	8%	0.224	2%
4	0.148	2%	0.219	8.32%	0.232	3.12%	0.125	1%	0.143	1%

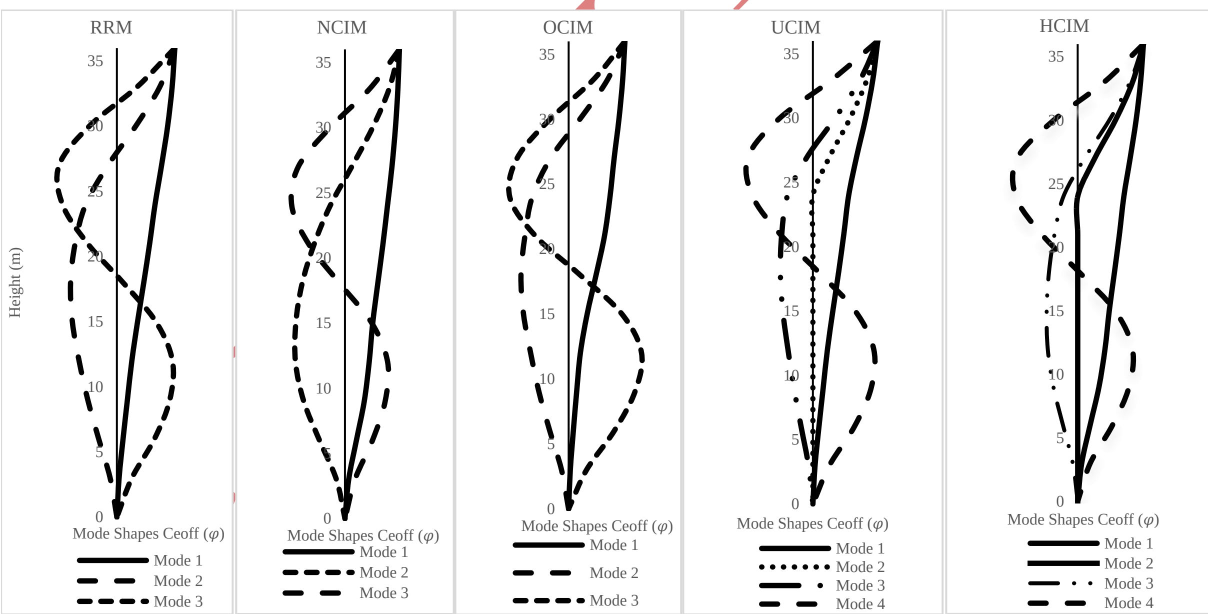

Figure 9 The most influential modes of vibration for each configuration are presented in Table (2), with their normalized elastic mode-shapes and corresponding effective modal mass percentages.

3.1.1 Neglecting Localized Modes

Based on table (2) and the diagram in figures (9,10), the reason for neglecting the second mode in UCIM and HCIM (Hybrid Coupled Irregular Configuration Model) is due to their low modal mass percentages.

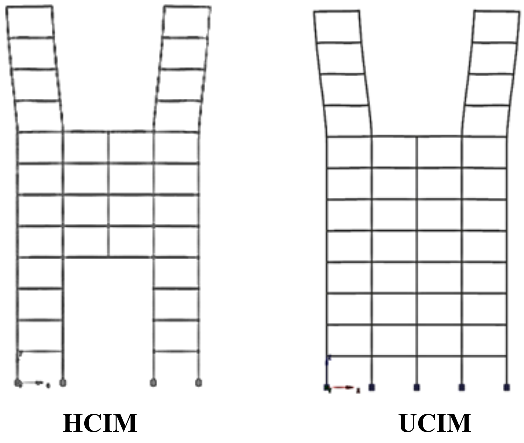

Figure 10. The localized 2nd mode shape

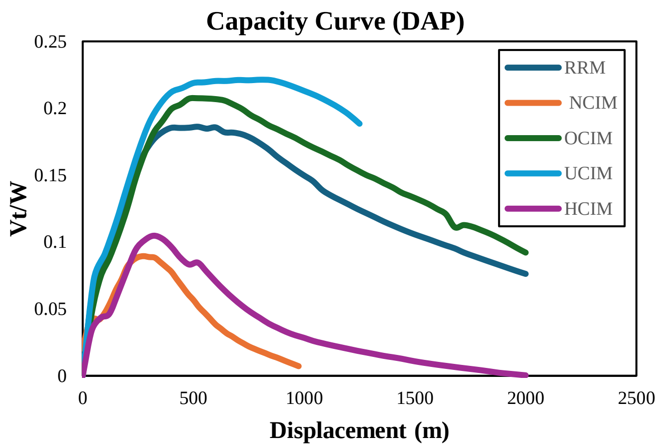

Figure 12. capacity curve for displacement pushover -based analysis | 1st mode pattern

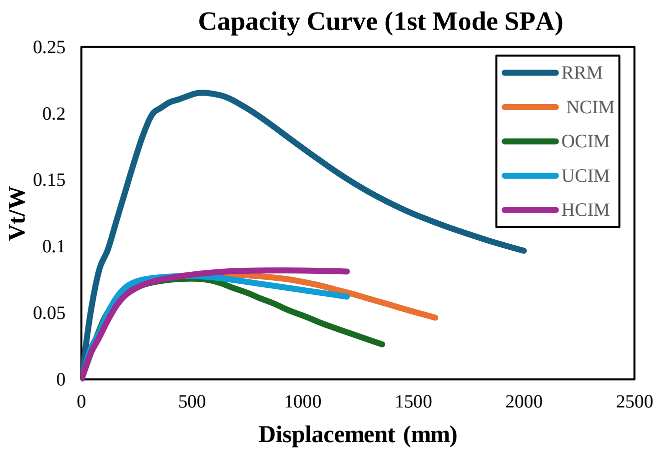

Figure 11. conventional static adaptive pushover analysis (DPA)

3.2 Discussion of Capacity Curves Results

In both analyses, the regular reference model (RRM) exhibits the highest base shear and energy dissipation capacity, as demonstrated by its pronounced peak and sustained post-peak response. This response reflects the inherent advantages of geometric regularity, yielding maximum lateral load resistance and global ductility. In contrast, frames exhibiting various degrees of vertical irregularity (NCIM, OCIM, UCIM, HCIM) consistently show reductions in both peak base shear and overall displacement capacity, with the hybrid column irregular model (HCIM) performing the weakest in both analyses. It is noteworthy that the adaptive pushover analysis (Figure 12) results in lower maximum base shear and steeper post-peak degradation for irregular frames compared to the conventional pushover approach (Figure 11). This trend underscores the conservative and more realistic nature of the adaptive pushover method in capturing the detrimental effects of vertical irregularities. The adaptive approach dynamically updates the lateral force distribution to better reflect the true progressive demand on the structure during seismic action, as opposed to the static first-mode pattern assumed in the conventional pushover. Consequently, the adaptive method identifies earlier onset of strength degradation and reduced displacement capacity in irregular frames behaviors that are insufficiently captured by the conventional method.

Table 3: Derived Seismic Performance Parameters and Reduction Factor (q) from DAP

Variables \Configuration	RRM	NCIM	OCIM	UCIM	HCIM
$V_y\ (kN)$	1349.01	579.31	1359.61	1359.13	567.15
$V_u\ (kN)$	1927.15	827.59	1942.30	1941.61	810.21
$V_s\ (kN)$	1587.71	595.02	1005.09	1279.92	609.88
$V_d\ (kN)$	675.00	676.00	607.50	565.80	537.90
$\delta_u\ (mm)$	1009.32	449.56	1090.51	1250.00	496.66
$\delta_y\ \ (mm)$	252.30	212.70	323.50	307.00	243.50
μ	4.00	2.11	3.37	4.07	2.04
R_R	0.85	0.97	1.35	1.06	0.93
Ω	2.35	0.88	1.65	2.26	1.13
q	7.9950	1.81130	7.54438	9.7807	2.1506

Table 4: Derived Seismic Performance Parameters and Reduction Factor (R) from CSPA

Variables\Configuration	RRM	NCIM	OCIM	UCIM	HCIM
$Vy\ (kN)$	1560.96	511.62	496.22	476.74	443.83
$Vu\ (kN)$	2229.94	730.89	708.88	681.06	634.04
$Vs\ (kN)$	1869.29	545.68	474.38	541.85	431.76
$Vd\ (kN)$	765.00	676.00	607.50	565.80	537.90
$\delta u\ (mm)$	1030.20	1246.11	813.08	1200.00	1200.00
$\delta y\ \ (mm)$	296.1	235.50	203.90	201.20	283.30
μ	3.48	5.29	3.99	5.96	4.24
RR	0.84	0.94	1.05	0.88	1.03
Ω	2.44	0.81	0.78	0.96	0.80
q	7.0993	4.00469	3.25719	5.0254	3.4950

The comparative evaluation of seismic performance parameters obtained from DPA is presented in Table 3 and CSPA is presented in Table 4. The results reveal substantial differences between the two analytical approaches, with significant implications for the seismic assessment of vertically irregular RC moment-resisting frames.

3.2.1 Behaviour Factor Assessment q:

the most significant finding pertains to the behaviour factors (q) derived from both methods. The DPA yields considerably lower behaviour factors for irregular configurations, with NCIM and HCIM exhibiting values of 1.81 and 2.15, respectively. These values represent substantial deviations from the Eurocode 8 prescribed behaviour factors of q = 3.2 for irregular structures and q = 4.0 for regular structures. Conversely, CSPA produces behaviour factors that more closely align with or exceed code provisions (NCIM: q = 4.00, HCIM: q = 3.49), potentially masking the true seismic vulnerability of these configurations as shown in figure13. The regular reference model (RRM) demonstrates behaviour factors of q = 7.99 (DPA) and q = 7.09 (CSPA), both exceeding the code value for regular structures, indicating adequate seismic performance. However, the OCIM and UCIM models show markedly different responses between methods, with DPA yielding q = 7.54 and q = 9.78, respectively, while 1st Mode SPA produces more conservative values of q = 3.25 and q = 5.02.

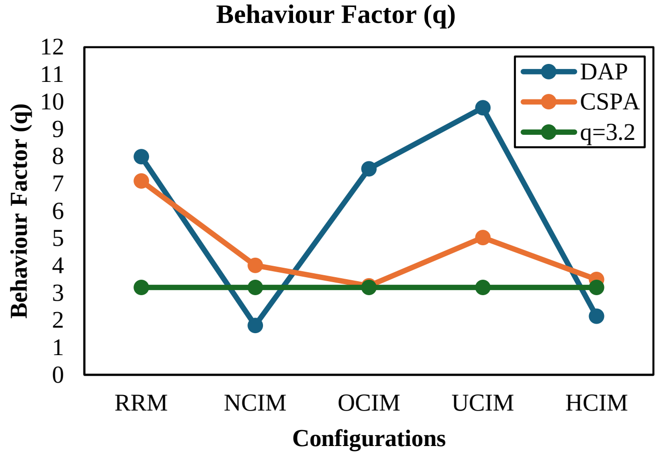

Figure 13. Behavior factor (q) for each procures DPA and 1st mode SPA compared with code limitations (q = 3.2)

3.2.2 Ductility and Capacity Parameters

In figure 14, the ductility factors (μ) reveal contrasting trends between the two methods. DPA indicates significantly reduced ductility for irregular frames, particularly NCIM (μ = 2.11) and HCIM (μ = 2.04), compared to the RRM (μ = 4.00). The 1st Mode SPA, however, suggests higher ductility values for these same configurations (NCIM: μ = 5.29, HCIM: μ = 4.24), potentially overestimating their deformation capacity.

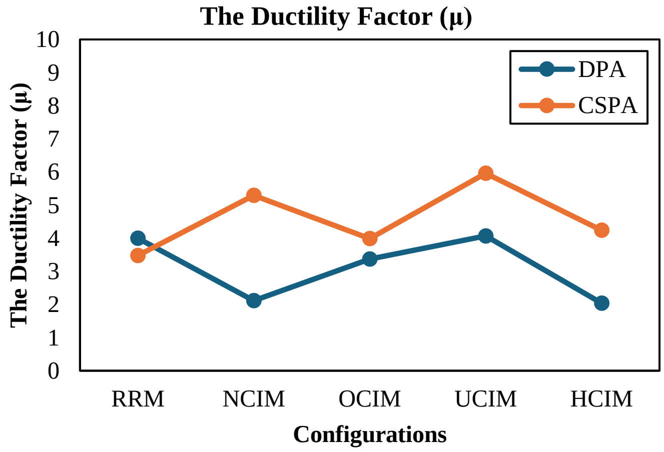

Figure 14. The ductility factor (μ) for each procures DAP and 1st mode SPA for each configuration.

3.2.3 Overstrength and Redundancy Factors

The overstrength factors (Ω) from DPA reveal significant variations among irregular configurations as illuminated in figure 15. While OCIM and UCIM maintain reasonable overstrength (Ω = 1.65 and 2.26, respectively), NCIM and HCIM show critically low values (Ω = 0.88 and 1.13, respectively), indicating limited reserve strength capacity. The redundancy factors (R_r) similarly demonstrate reduced structural robustness for irregular configurations, with values generally below unity for most cases.

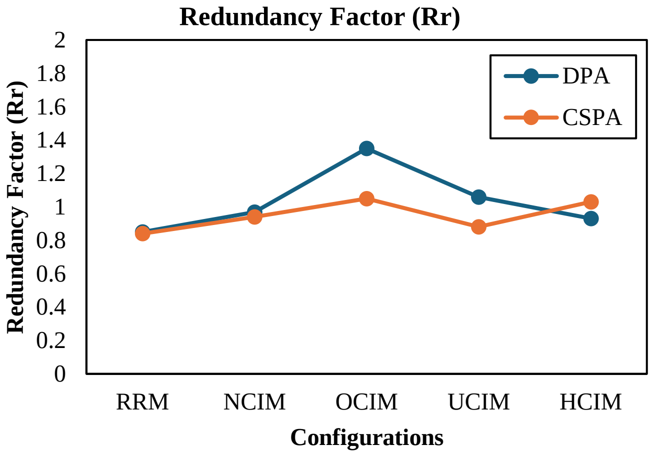

Figure 15. Redundancy factor (R_r) for each procures DAP and 1st mode SPA for each configuration.

4. Conclusions

This study provides a comprehensive evaluation of reduction factors for vertically irregular reinforced concrete moment-resisting frames by comparison between Adaptive Displacement -based Pushover Analysis (DPA) and conventional first-mode Static Pushover Analysis (SPA), which shows that vertical geometric irregularities have a significant effect on the seismic performance of RC frames, and the degree of influence depends on the analytical method used. The DAP consistently shows smaller and more conservative behaviour factors compared with conventional pushover analysis, especially for irregular configurations (NCIM, q = 1.8113; HCIM, q = 2.1506), which is substantially lower than Eurocode 8 prescribed behaviour factors (q = 3.2 for irregular structures). The conventional first-mode SPA produces behaviour factors that more closely align with code values, potentially masking the true vulnerability of irregular structures. This discrepancy highlights the critical limitation of simplified analytical approaches in capturing the complex nonlinear response characteristics of vertically irregular frames, including higher mode effects, progressive stiffness degradation, and realistic force redistribution patterns.

The large differences between DPA and typical pushover results demonstrate the inadequacy of simplified analytical approaches for irregular structures, and the incorporation of higher mode effects, progressive stiffness degradation, and realistic force redistribution patterns in the DPA result in more accurate evaluation of structural behaviour. The lower behaviour factors obtained from DPA suggest that current code provisions may overestimate the seismic capacity of irregular RC frames, especially with large geometric discontinuities, indicating that vertically irregular RC frames, especially NCIM and HCIM configurations, may not achieve the seismic performance levels assumed by conventional design codes. The findings support the use of advanced nonlinear analysis methods for evaluating irregular structures and suggest that more conservative behaviour factors for certain irregular configurations may be required to achieve the seismic performance levels assumed by conventional design codes.

Furthermore, the calculated reduction factors based on the adaptive pushover curves are consistently lower than those derived from the conventional analysis across all irregular models. This discrepancy highlights the limitations of traditional methods adequately representing the complex seismic response of vertically irregular RC frames. The findings emphasize the necessity of employing advanced adaptive analysis techniques, such as DPA, particularly in the seismic assessment and design of irregular structures compliant with modern codes (e.g., Eurocode 8) [2]. In summary, the results confirm that the presence and degree of vertical irregularity have a pronounced adverse effect on both strength and ductility capacities of RC MRFs. Adaptive pushover analysis provides a more accurate and conservative estimation of the structure's true behavior, advocating for its adoption in engineering practice for irregular frame configurations

Abbreviation List:

R: Response Modification Factor

q: Behaviour Factor

R_μ: The Ductility Reduction Factor

Rs: The Overstrength Factor

R_ρ: The Redundancy Factor

R_ξ: The Damping Modification Factor

MRFs: Moment Resistance Frames

RC: Reinforcement Concrete

V_y :Base Shear Corresponding to Yield Load Capacity

V_{max}: Base Shear Corresponding to Maximum Load Capacity

V_s: Base Shear Corresponding to The First Hinge Formation in The Structure

V_d: Design Base Shear

δ_u : Ultimate Displacement

δ_y: The Yield Displacement.

RRM: Rectangular Regular Model

NCIM: N-Configuration Model

OCIM: O-Configuration Model

UCIM: U-Configuration Model

HCIM: H-Configuration Model

DAP: Displacement-Based Adaptive Pushover Analysis

CSPA: conventional Static Pushover Analysis

m_i: Modal mass participation

n : Number of modes considered

Vt: Total Base Shear

W: Structure Weight

FEA: Finite Element Analysis

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