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A New Method For Estimation For Estimation Of Seismic Behaviour Of Setback Buildings

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Abstract
A Setbacks are widely used in modern architecture because of the aesthetic and practical benefits they give. However, the existence of setbacks diminishes the structure's floor area and affects its mass and stiffness, resulting in variances in the building's dynamic behaviour. Seismic design codes sometimes ignore this component, resulting in structure collapses during earthquakes. This research offers a new index for setback structures to replace and improve on code-based indexes. The suggested index's ability to concurrently quantify mass, stiffness, and strength irregularity proves its effectiveness in assessing setback irregularity.
Keywords: *Setback Irregularities, Irregular building Frames, Seismic Analysis, Setback buildings.*

INTRODUCTION

Setbacks are fairly common in modern buildings, with aesthetic and practical considerations driving their popularity. Setback buildings are highly beneficial in urban settings because they give ample sunshine and air for the lower levels while still conforming to the Building Standards of India's 'floor area ratio' requirements [1]. However, the existence of setbacks diminishes the floor area of the structure and changes its mass and stiffness, resulting in variations in its dynamic behaviour. Seismic design codes throughout the world have disregarded this element [2-4,69,70]. Failure of code-designed setback buildings after prior earthquakes [2-4] demonstrates this. Furthermore, the recommended rules in the code are based on elastic analysis, which is inefficient for constructing these structures. A careful examination of setback structures reveals that the seismic behaviour of setback structures is perplexing, with some studies reporting appropriate seismic performance [5-31] and others presenting contradicting views [31-53]. As a result, it is critical to investigate the seismic behaviour of setback structures, and it is desirable to evaluate the suitability of suggested concrete on setback structures

Modeling and analysis

In the current study, RC plane frames with bay widths of 3m and 3m were used. This study applies analytical analysis in (Figure 1) story frames with storey heights of 12m, 18m, and 24m. The 66 geometrical categories employed are as follows: (a) simple regular frames, (b) building frames with minor setbacks, (c) building frames with huge setbacks, (d) building frames with centre setback tower, and (e) buildings with setback at various levels. The time period of the building frames is retained in line with Goel and Chopra [54], as shown in Figure 2, to guarantee that the building frames reflect the general moment resistant frames. The multifiber FE model given by Jaafari et al. 2022 was used to model the setback building models utilising the attributes of the proposed concrete, as illustrated in Figure 3. The frames were modelled using a multifiber beam technique utilising the Jaafari et al. 2022 model [55]. The beams were modelled as two-node three-dimensional Timoshenko beam components (Figure 2 a). Each node represents three rotations (ix, iy, and iz) and three displacements (Uix, Uiy, and Uiz) using concrete fibres positioned at Gauss points. Jaffari et al. 2022 suggested form functions for determining displacement along the beam. Jaffari et al. 2022 employed the Timoshenko suggested equations to compute strain values given known displacement values. Furthermore, stress is calculated at various fibres using nonlinear stress-to-strain values. Furthermore, force at corresponding gauss locations was calculated by integrating the cross section. As shown in Figure 2a, K represents the stiffness matrix, B represents the gradient metric, L represents the beam element length, U represents the nodal displacement vector (including nodal displacements Ui and nodal rotations I), S represents the stress matrix, P represents the load vector, and H represents the damaged Young modulus matrix. Jaffari et al. 2022 criteria were used to simulate the concrete. The dead load is considered to be the frame's self-weight, and the soil type is assumed to be hard soil. The EC8 design spectrum with a PGA of 0.5 g [56] defines the projected earthquake ground motion. The design produced optimal beam and column cross sections ranging between 275 mm x 275 mm and 400 mm x 400 mm (for beams), 350 mm x 350 mm (for columns), and 550 mm x 550 mm (for columns) for all of the building designs considered in the current research (Figure 1). (This is for columns). Uniform cross sections of beams and columns of 350 mm x 350 mm (for beams) and 550 mm x 550 mm (for columns) are utilised to ease the study. The structural elements cross section for all of the frames shown in Figure 1 were chosen to meet the strength and stiffness criteria (as defined in EC 8: 2004). Furthermore, structural member cross sections were altered to provide three distinct beam to column stiffness ratio (r) values. Chopra [54] recommends r as a parameter.

$$R = I_{bo}/4\, I_{co} \qquad (1)$$

where *Ibo* and *Ico* are the moment of inertia of the beams and the columns in a storey respectively. In addition to the standard cross - sections of structural components used to meet strength and stiffness standards, two additional column and beam sizes are used to change r. Many analytical methodologies may be used to analyse building frames. Analytic methods are classified as linear or nonlinear, static or dynamic. Table 1 shows the results of an analysis of the building frames in Figure 1 using E-Tabs v 9.0 software [57] and 13 ground movements. The crucial time period is established using the Newmark and Riddle technique [59], and the ground motion data is collected from the PEER database [58].The analytical models were exposed to an ensemble of 13 ground movements, as shown in Table 1, in order to examine the seismic behaviour of setback frames at four performance levels [60].The critical time period (Tc) for these ground motions is estimated using the Newmark and Riddle technique [59], and the specifics of the ground motions are collected from the PEER database [34]. Figure 4 [60] depicts the stiffness degradation model provided by Dutta and Das for modelling inelastic behaviour. P Delta effects were also taken into account, with a damping proportion of 5%.
The scale factors for the accelograms for the specified performance levels were acquired from the SEAOC 1999 Manual [61]. The analytical analysis takes into account the following performance levels: a) Immediate occupancy level. The effects of P-Delta forces have also been considered in this analytical study as per SEAOC Manual [62] for performance level P1L1 corresponding to immediate occupancy, B) Corresponding to occurrence of first plastic hinge 0.5% (PL2), C) Safety of life corresponding to 0.9% occurrence of first plastic hinge, and d) Prevention of collapse occurrence corresponding to 2.5%. The behaviour factor is calculated by dividing the acceleogram scale factor at the chosen performance level by the acceleogram scale factor at the first plastic hinge [63]. The ground motion data was adopted in line with the PEER Database using the Newmark and Riddle Algorithm. The next sections explain the detailed reaction of the building model.

a) The irregularity has no bearing on the P2L2 performance level.
b) Structures with no irregularities have identical IDR characteristics

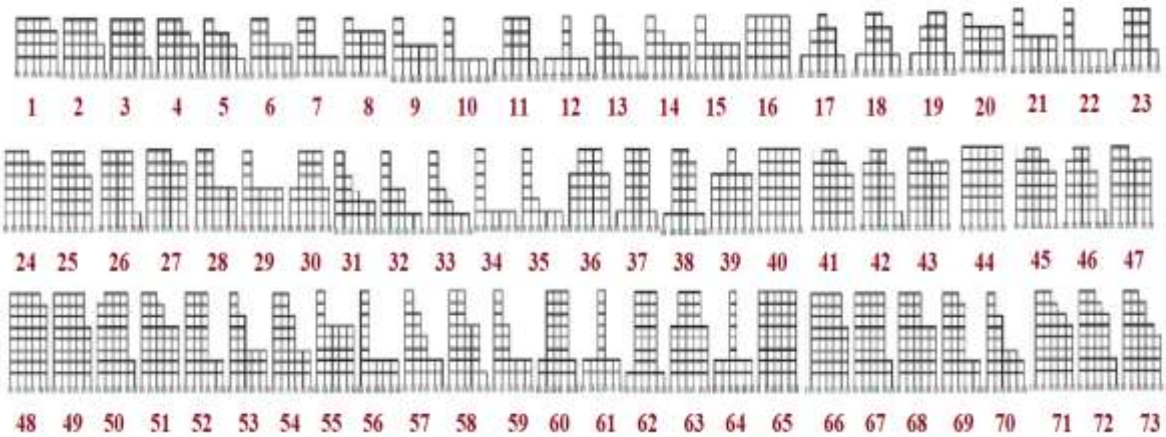


Figure 1 : Setback Building frames adopted in the present study

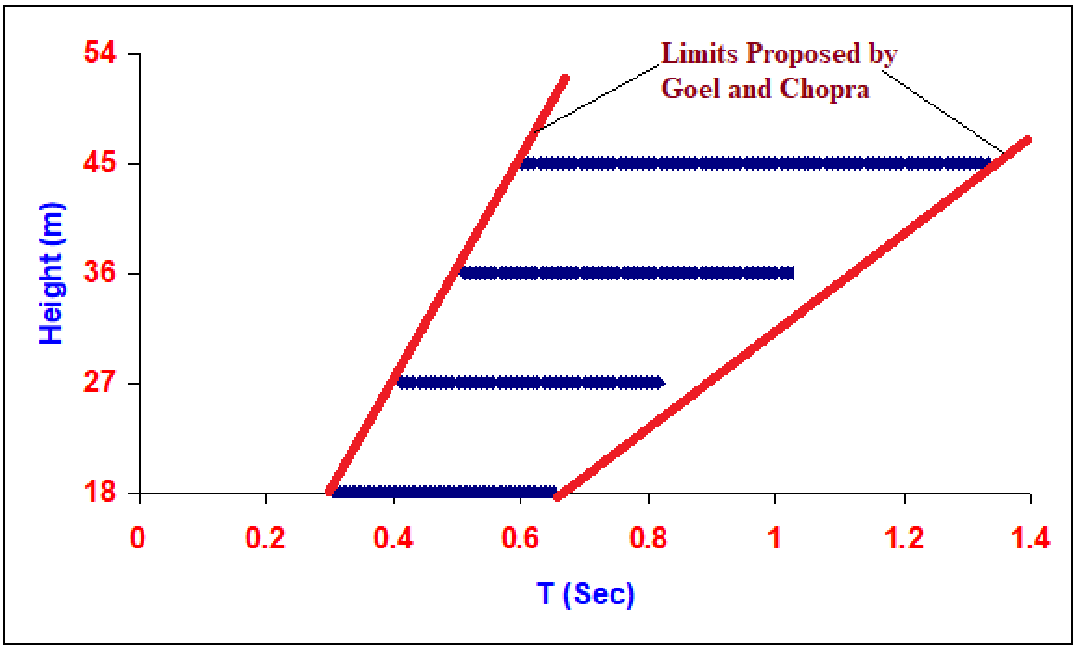


Figure 2 : Limits Proposed by Goel and Chopra (1976)

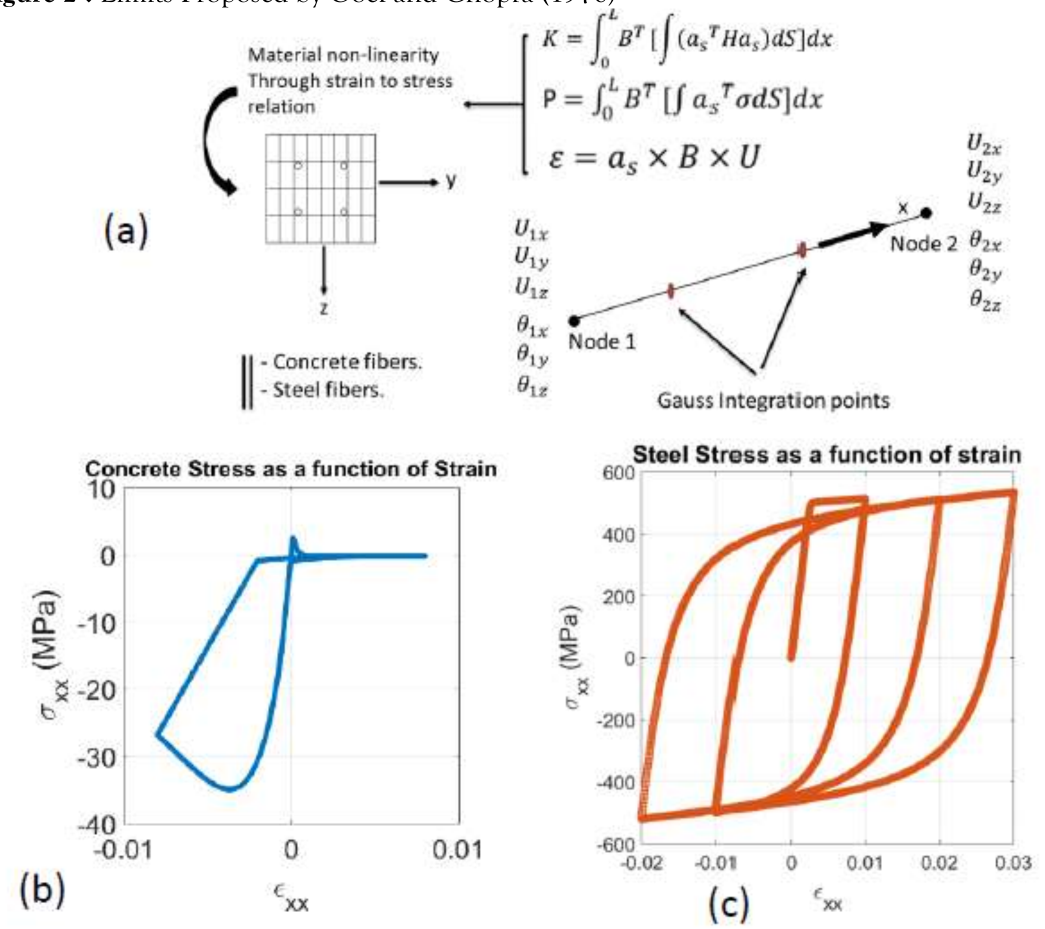


Figure 3 : Multifiber beam model proposed by Jafaari et al. (2022) a) Principle of Multifiber beam model with material constitutive laws, b) Cyclic model for concrete c) Cyclic degradation model for steel [Jafaari et al. (2022)]

K denotes the stiffness matrix; B represents the gradient function; L represents length of beam element; U denotes the nodal displacement vector

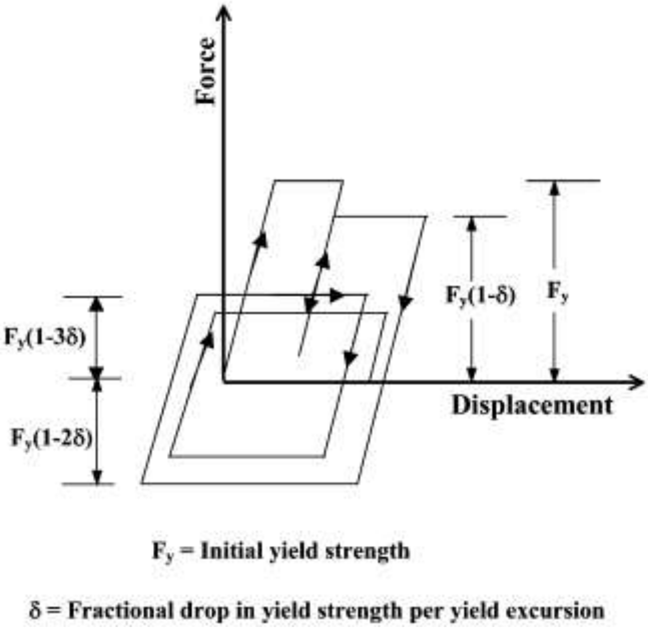


Figure 4 : Dutta and Das model for stiffness deterioration [30]

Table 1 : Ground motions used for the analytical study

S.No	Name of earthquake and station	Date	Magnitude Mu	Distance of epicenter (D)	Peak Ground acceleration (PGA)	Critical time period (Tc)
1	Imperial valley 1979 (EI Centro Array # 4)	15/10/1979 6.5 15.5 1.31 0.60	6.5	15.5	1.31	0.60
2	Imperial Valley (Delta)	15/10/1979	6.5	44	3.44	0.61
3	San Fernado (Pacoima Dam)	9/02/1971	6.6	2.8	12.01	0./60
4	San Fernado (Castaic)	9/02/1971	6.6	29	2.63	0.60
5	Northridge,LA—116th StSchool	17/01/1994	6.7	41.9	2.04	0.60
6	Northridge,LA—Cypress Ave.	17/01/1994	6.7	32.8	2.06	0.60
7	Northridge,LA—Fletcher dr	17/01/1994	6.7	29.5	2.35	0.60
8	Northridge(Hollywood—WiloughbyAve.)	17/01/1994	6.7	26	2.41	0.91
9	Northridge(LA—N FaringRd)	17/01/1994	6.7	24	2.68	0.61
10	Northridge(LA—S GrandAve.)	17/01/1994	6.7	37	2.84	0.65
11	Northridge(LA—ObregonPark	17/01/1994	6.7	37	3.10	0.5
12	Northridge(LA—WonderlandAve.	17/01/1994	6.7	31	1.65	0.5

13	Imperial Valley-06, Country center FF	EC	15/10/1979	6.5	7.60	2.41	0.6

3. New index based on material characteristics for setback buildings with proposed concrete

A review of seismic design codes reveals a lack of understanding of the irregularity element, as well as a lack of difference in criteria for estimating seismic response for regular and irregular structures. However, as illustrated in Figure 5, the seismic behaviour of both of these buildings is completely different (Varadharajan et al. 2013). Despite the fact that EC 8 mandates a 20% reduction in behaviour factor for irregular structures, EC 8 rules have inherent limitations because they are suggested for single degree of freedom systems and elastic analysis. Maximum displacement ductility may be computed using EC 8 as

$$\mu = q \qquad (2)$$

and the corresponding displacements and interstorey drifts as per EC 8 can be calculated as

$$D = Dd' \ x \ q \qquad (3)$$

$$d = dd' \ x \ q \qquad (4)$$

where Dd' is the maximum displacement, D′ is the yield value of maximum displacement when design lateral forces are decreased, D is the maximum interstory drift, and dd′ is the yield value of maximum interstory drift when design lateral forces are reduced. The fundamental flaw in the aforementioned calculations is the unrealistic assumption of uniform drift profiles along the building's height [64-66,68]. These criteria are also inapplicable to irregular constructions. Dynamic response characteristics, namely a) Mass Participation factor and b) Natural frequency, may effectively reflect the irregularity of the structures. According to the sensitivity study, the sensitivity index [67] product of these two values has higher Sensitivity (Figure 6). As a result, the irregularity coefficient (ϒu) is presented here.

$$\Upsilon u = K\alpha/\beta \qquad (5)$$

Where $\alpha = (M_k/\ N_N), and\ \beta = (\omega_K/\ \omega_N)$ (6)

K is the constant with values varying from 0.46 to 0.85 for building models used in the present study

MK and MN are mass participation factors of irregular and irregular buildings, ωK/ ωN are corresponding resonant frequencies of irregular and regular buildings.

$$M_k = \frac{\left[\sum_{1}^{n} W_k \phi_{ik}\right]^2}{g\left[\sum_{1}^{n} W_k (\phi_{ik})^2\right]} \qquad [7]$$

For the building models examined in the analytical analysis, the irregularity coefficient varies from 0.295 to 1. It is useful to compare the proposed index's efficiency against earlier indexes. The Setback building models depicted in Figure 7 are used for analytical purposes, and the irregularity indices suggested by various authors are presented in Table 2, demonstrating the efficacy of the proposed technique.

Table 2 : Indices proposed by different researchers to capture setback irregularity

Name of author and year	Building type	Proposed Index	Values of irregularity Index			
			A	**B**	**C**	**D**
Karavasilis (2007)	Steel Frame	$\phi_S = \frac{1}{n_s - 1}\sum_{1}^{n_s-1}\frac{L_i}{L_{I+1}}$	1.22,2.66	1.45 ,2.66	1.21 ,2.66	1.14 ,2.66

		$\phi_b = \frac{1}{n_b - 1}\sum_{1}^{n_b-1}\frac{H_i}{H_{I+1}}$				
Sarkar et al. (2013)	RC stepped frame	$ŋ = \frac{\Gamma_1}{\Gamma_{ref}}$	1	0.486	0.4232	0.4342
Varadharajan	Rc stepped frame	$\lambda_r = \frac{1}{n_b - 1}\sum_{k=1}^{n}\frac{\omega_{ir}}{\omega_r}$	1	0.4421	0.4265	0.3843
Proposed irregularity index	RC stepped frame	$\Upsilon u = K\alpha/\beta$	1	0.56542	0.3712	0.2954
IS 1893:2002	Stepped Frame	-	1	0.6	0.6	0.6
ASCE 7.05	Stepped Frame	-	1	2	2	2
EC 8 :2004	Stepped Frame	-	1	0.5	0.5	0.5

Where Hi and Li are height from base and length of I th floor, ns and nb are number of storey and number of bays at first storey of the frame, , λr is irregularity index proposed by Varadharajan et al. (2013), N represents number of storeys, ωr and ωir represents frequency of regular and irregular building, ŋ is the irregularity coefficient proposed by Sarkar et al. (2010),Γ1 is the first mode participation factor and Γref is participation factor for corresponding regular building

4. New rules to estimate deformation demands of setback buildings

As explained in the preceding section, the proposed EC 8: 2004 [56] regulation has inherent drawbacks since it is based on incorrect assumptions. As a result, there is an urgent need to establish new criteria for estimating deformation needs of setback structures constructed with suggested concrete. The equations to determine maximum roof displacement (μrk), maximum interstorey drift ratio (Ik) and maximum rotational ductility (μkθ) are based on the non-linear regression analysis findings and can be proposed as

$$\mu rk^{0.936} = (q - 1.12/1.62\,\alpha^{-0.38}\,\Upsilon^{0.35}) + 1.23 \quad (8)$$

$$I_k = \frac{\mu_{rk}}{1.44H}(1 - 0.121r^{0.723}\alpha^{-0.122}\Upsilon^{2.33} \quad (9)$$

$$\mu_{i\theta} = \frac{(q-1.345)}{20.34\Upsilon^{0.91}A}0.932r \quad (10)$$

where μrk, Ik and μiθ represents maximum roof displacement, maximum interstorey drift ratio and maximum rotational ductility.

The comparison of the suggested equations with dynamic analysis reveals that the proposed equations have a close association with dynamic analysis when compared to the proposed equations of EC 8 (Figure 8 - Figure 10).

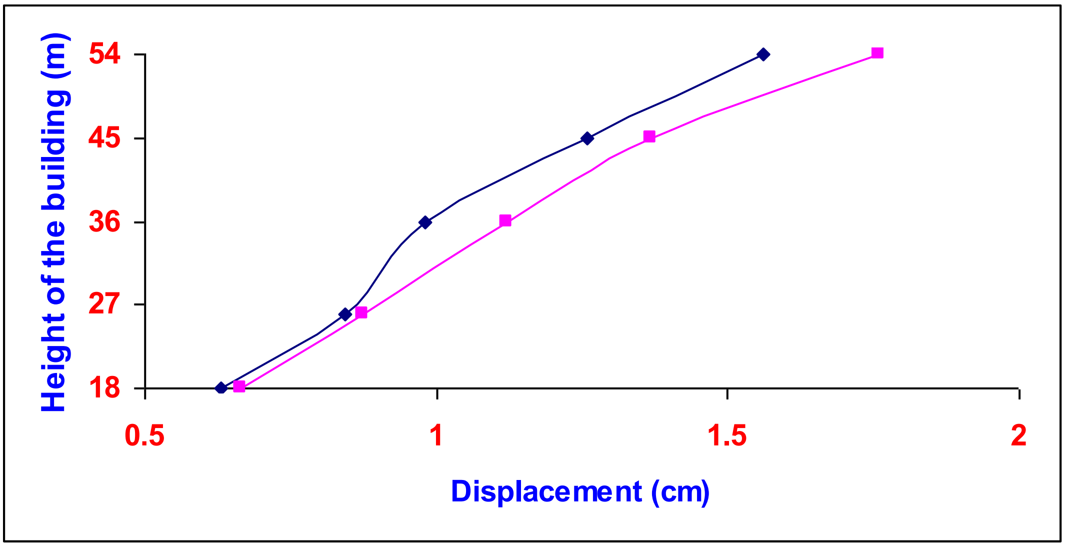


Figure 5 : Drift profiles of building with and without ISF slag

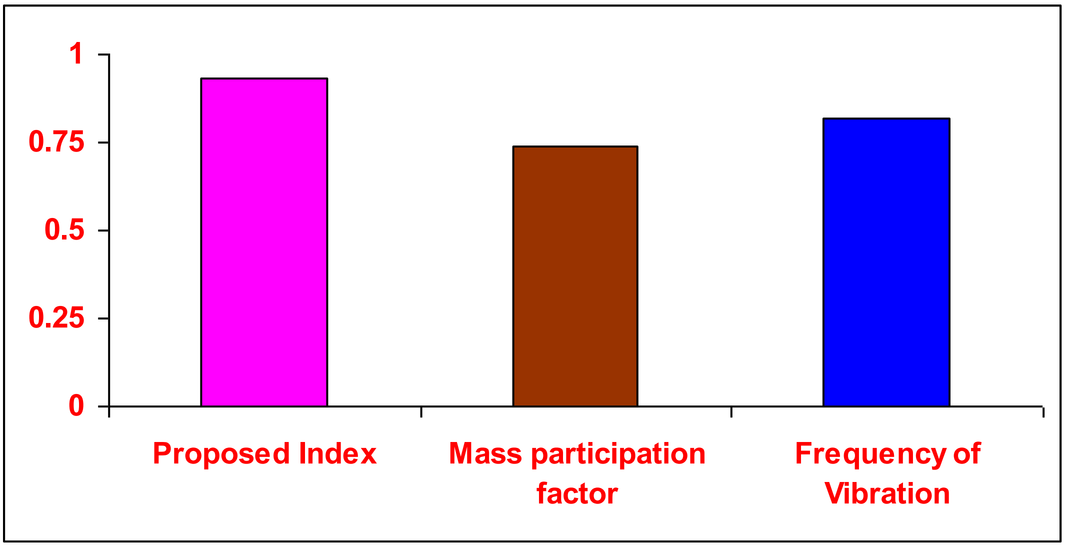


Figure 6 : Results of sensitivity analysis

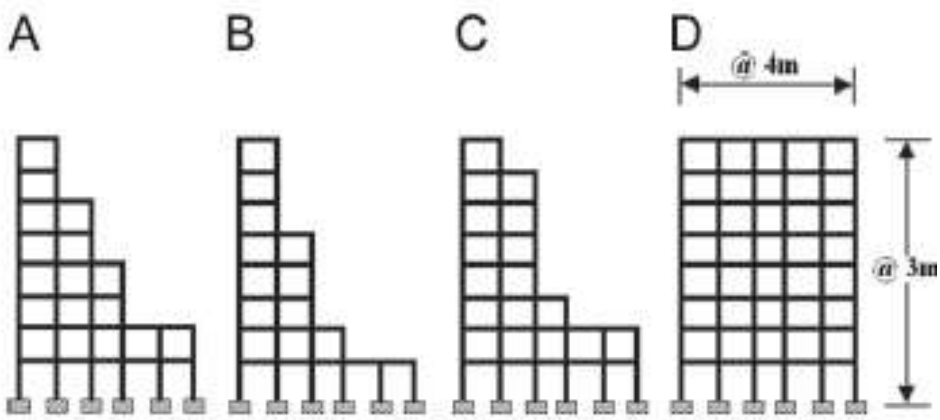
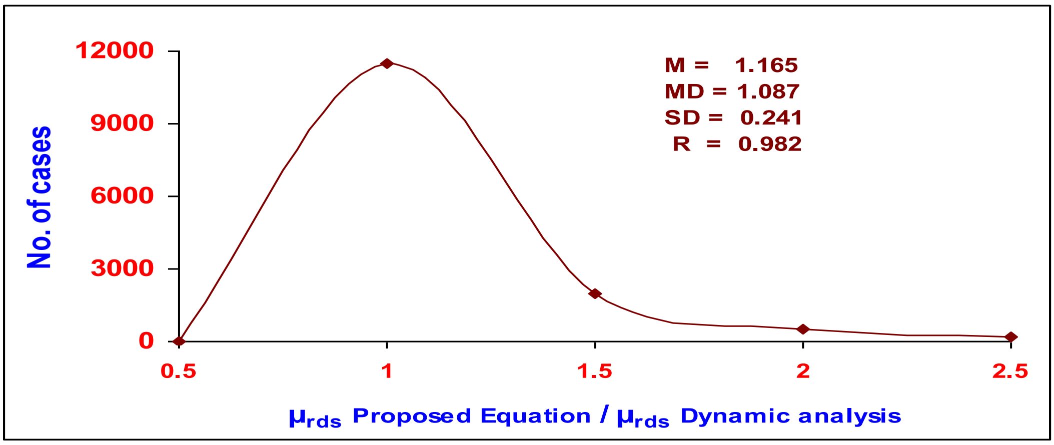
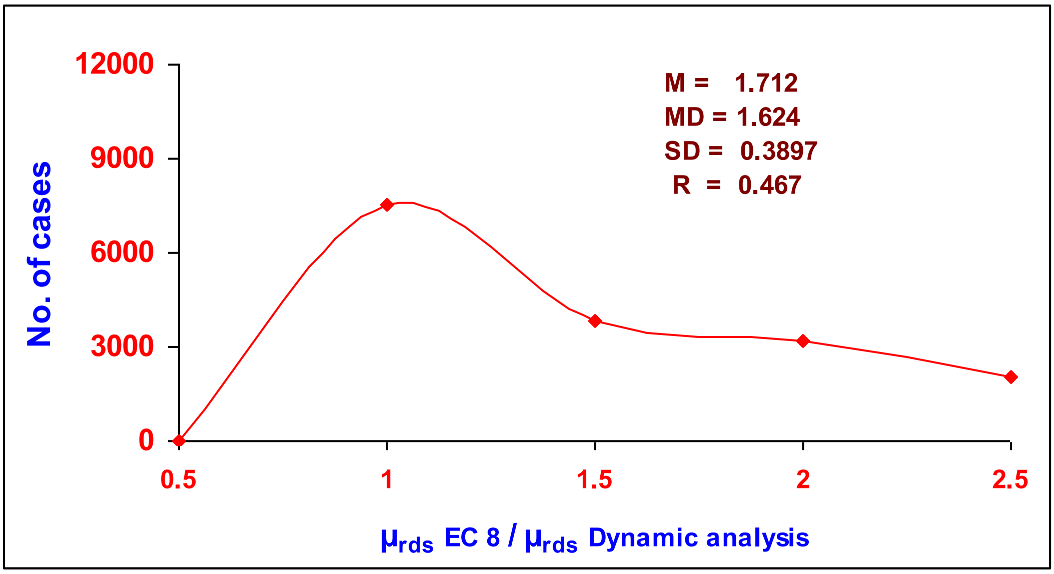


Figure 7: Building models consideed for the evaluation of Uncertainity Index.

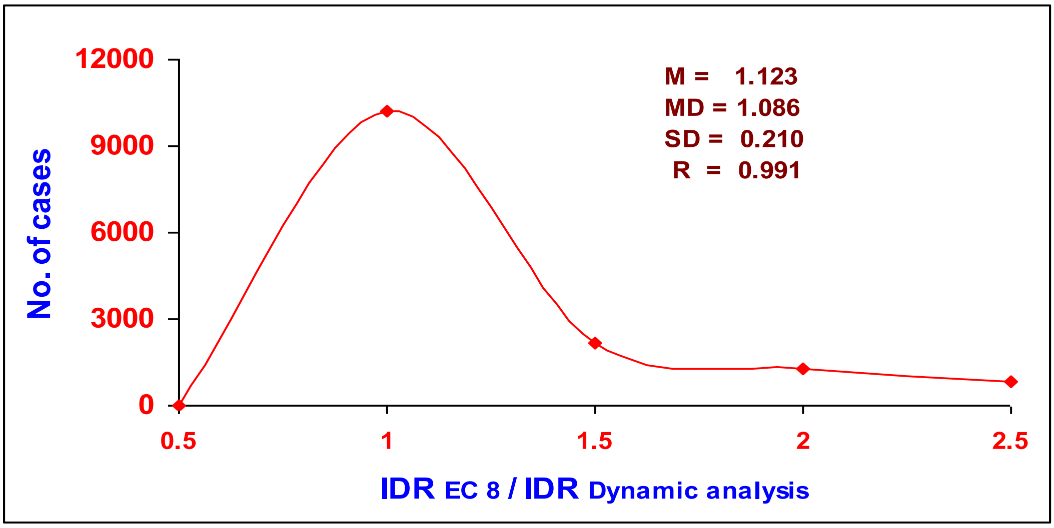


a)



b)

Figure 8. Results of comparison of proposed equations with dynamic analysis for maximum roof displacement



a)

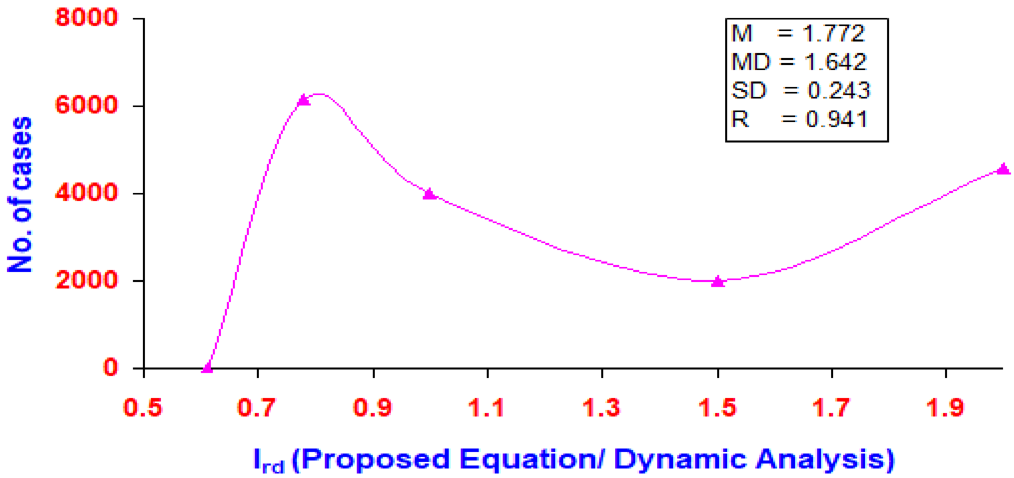
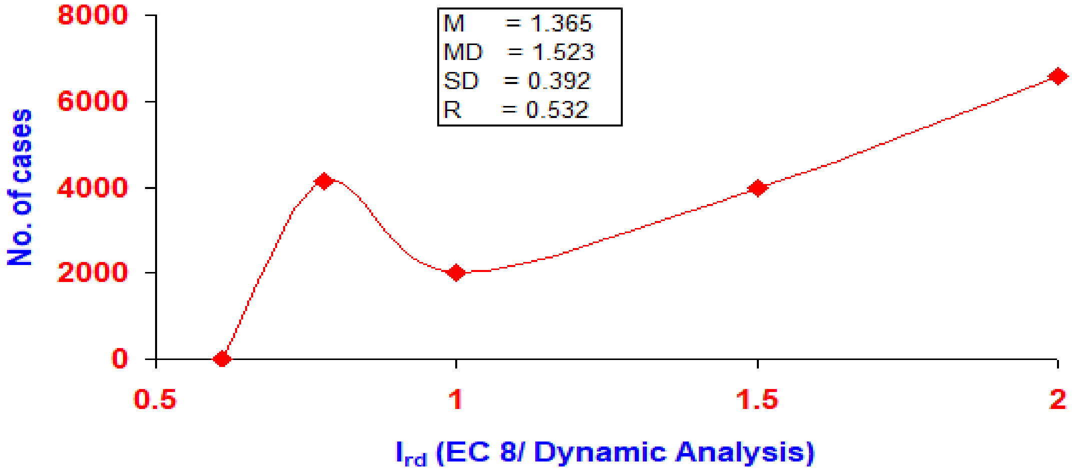
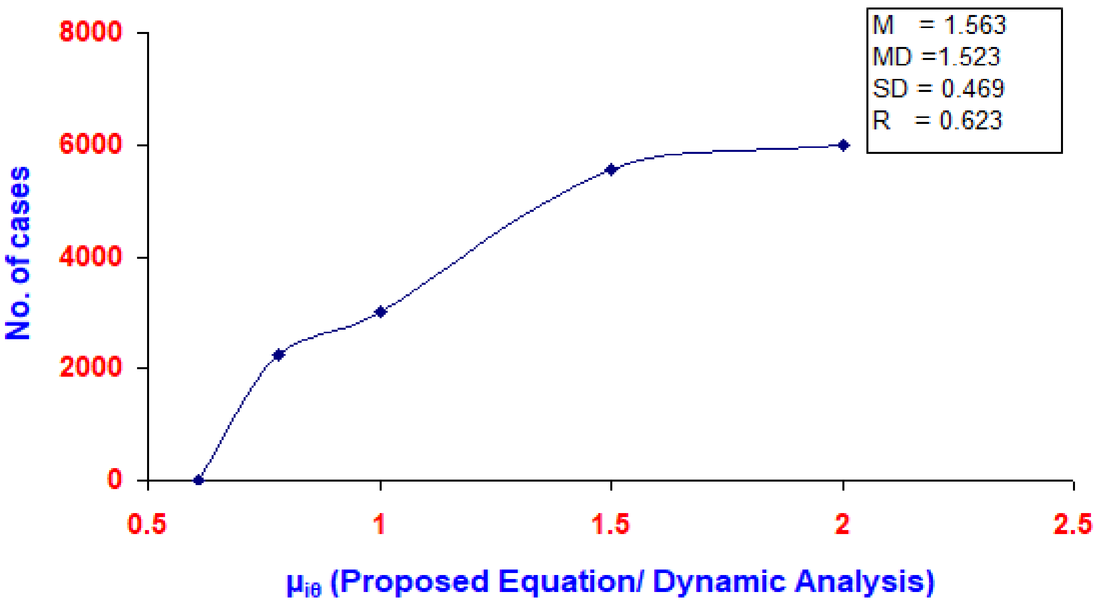


Figure 9. Results of comparison of proposed equations with dynamic analysis for maximum roof displacement



a)



b)

Figure 10. Results of comparison of proposed equations with dynamic analysis for interstorey drift ratio

5. CONCLUSIONS

This study proposes a new index for setback constructions to replace the indices supplied by code and prior studies. The indices used to assess setback irregularity have the intrinsic disadvantage of being based on Rayleigh's analysis and assuming homogeneous mass, stiffness, and strength distribution along the building height, which is impractical. A new index to assess both setback irregularity, which reflects a mix of mass, stiffness, and strength irregularity, has been developed in this study effort, resolving earlier limitations. The suggested index's effectiveness in quantifying setback irregularity is demonstrated by comparison with previously published indices.

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Author Contributions

A.J.: Conceptualized the research idea, developed the proposed irregularity index, led the analytical modeling, performed simulations, and coordinated the manuscript writing and editing process.

P.S.: Supervised and assisted in drafting and revising the manuscript.

S.V.: Provided expert guidance on seismic performance evaluation, supervised the validation of modeling approaches, reviewed and edited the manuscript critically for important intellectual content, and ensured alignment with current code provisions and engineering standards.

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