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COMPARATIVE STUDY OF SEISMIC PERFORMANCE OF STEP-BACK RC FRAME BUILDINGS

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ABSTRACT

Architectural requirements many times lead to a structurally irregular building. One such irregularity of the geometry in elevation is called 'step-back'. A step-back building is the one in which a portion of the building above a certain height is pushed-in towards the center or side. This leads to the change in both; mass distribution and stiffness distribution along the height of the building. The center of mass and the center of stiffness of different storeys do not lie along the same vertical line, resulting into predominant torsional modes of vibration during seismic events. The study of past earthquakes worldwide has revealed the poor performance of irregular buildings, which is detrimental to the structural safety and hence requires more comprehensive research. The paper presents study on seismic performance of five and ten storey RC frame buildings with different types of step-backs. The buildings are designed for elastic forces followed by pushover analysis and time history analysis using ETABS software. Further, the effect of step-backs on the dynamic characteristics, yield patterns, and storey drift is studied. The most appropriate step-back configurations from seismic considerations are suggested. This study will be of great help to architects and structural engineers, to select between the different step-back patterns and identify the possible regions within the structure that require special attention in designing and detailing.

KEYWORDS: Irregularity, Step-Back, Centre of Mass, Centre of Stiffness, Pushover Analysis, Nonlinear Analysis, Capacity Curve, Performance Point

INTRODUCTION

The behaviour of structures during many past earthquakes has demonstrated that buildings with simple regular geometry and uniformly distributed mass and stiffness in plan and in elevation, suffer much less damage, than the buildings with irregular configurations. The performance of buildings with irregular configurations; either in plan or in elevation has been extremely poor. Hence all efforts are made to eliminate irregularities by modifying architectural planning and structural configurations. Nonetheless, modern building construction comes with architectural extravaganza. To satisfy the needs of aesthetics, it is frequently required to construct geometrically and structurally irregular buildings. According to IS 1893 (2016) [1], plan irregularities include torsional irregularity, re-entrant corners, floor slabs with excessive openings, out-of-plane offsets in vertical elements, and non-parallel lateral force resisting systems. Stepped building frames, with vertical geometric irregularity, are now increasingly encountered in modern urban construction. The vertical irregularities include stiffness irregularity (soft storey), mass irregularity, vertical geometric irregularity, in-plane discontinuity in vertical elements resisting lateral forces, strength irregularity (weak storey), floating or stub columns, and irregular modes of vibration in two principal plan directions.

Step-back buildings fall in the category of vertical geometric irregularity. They are characterised by staggered abrupt reductions in floor area along the height of the building, with consequent reduction in mass, strength and stiffness. Due to height-wise changes in stiffness and mass, the dynamic characteristics of these buildings are completely different from those of the regular buildings. Figure 1 shows a typical step-back building located in New Delhi, India. Step-back buildings have frames of different height, as seen in Figure 2.

Fig. 1 A typical step-back building, New Delhi [3]

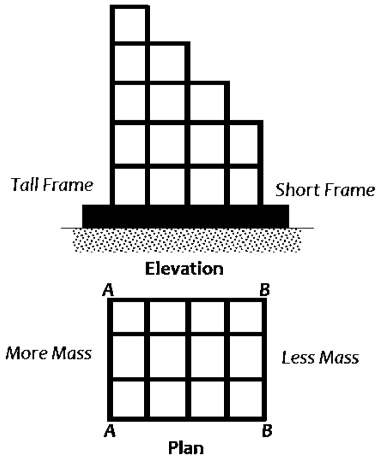


Fig. 2 Tall & short frame of step-back building [2]

LITERATURE REVIEW

Murty et al. [2] compared the deformed shape, axial force, shear force and bending moment of periphery frames (Frame AA and Frame BB in Figure 2), when subjected to seismically induced lateral forces. It was found that the deformation of taller (flexible) frame was larger than that of the shorter (stiff) frame; and force demand imposed on taller frame was higher than that on the shorter frame. Sarkar et al. [3] discussed some of the key-issues regarding analysis and design of stepped buildings. A new method was proposed for quantifying irregularity in such building frames, accounting for dynamic characteristics; mass and stiffness. The proposed 'regularity index' provides a basis for assessing the degree of irregularities in a stepped building frame. A modification in the code-specified empirical formula for estimating fundamental time period for regular frames was proposed, to estimate the fundamental time period of the stepped buildings. Montazeri et al. [4] studied low to mid-rise plane steel moment resisting frames (MRF) with various types of set-backs. The effects of different types of set-backs on the dynamic characteristics of frames were elaborated. Results reveal that dynamic properties of frames, i.e., natural time period, modal participation mass ratio, and mode shapes are strongly affected by different types of set-backs.

El-Esnawy et al. [5] examined the seismic responses of multi-storey RC MRF buildings with single symmetrical set-back by pushover analysis. The study shows that pushover analysis can provide adequate estimation of response of RC MRF buildings with vertical irregularities due the presence of set-backs. Dennis et al. [6] studied the seismic structural performance of the 126 storey Shanghai Tower through nonlinear time history analysis. The procedure for modelling and analysis of code-exceeding buildings is explained. It is recommended to take cognizance of the post-yield structural behaviour while designing. Kiran et al. [7] studied force-based and displacement-based pushover analysis methods. A comparison has been made between the results of the pushover analysis with the dynamic time history analysis. For the analysis purpose, two types of building structures were considered. It was observed that in most of the cases, pushover analysis results are conservative as compared to the time history results. It was also observed that in most of the cases, Equivalent Linearization Method of FEMA 440 gives good result in comparison to time history analysis.

Wong and Tso [8] studied the validity of design code requirements for irregular buildings with set-backs, that require a dynamic analysis with the base shear calibrated by the static base shear obtained using the equivalent static method. Two major issues are discussed, viz. (i) whether the code static base shear is applicable for buildings with set-backs, and (ii) whether the higher mode period should be used in computing the base shear when the modal weight of a higher mode is larger than that of the fundamental mode. Regarding the first issue, modification factors were derived for adjusting the code period formula so that it can provide a more reasonable estimate for the period of a building with a set-back. Pertaining to the second issue, it was demonstrated that for cases where the modal weight of a higher mode is larger than that of the fundamental mode, using the higher mode period for base shear calculation will result in unnecessarily conservative design. Soni and Mistry [9] reviewed the literature and different building codes, to understand the seismic behaviour of vertically irregular structures. The building codes provide criteria to classify the vertical irregular structures and suggest dynamic analysis to arrive at the design lateral forces.

The authors observed that most of the studies agree on the increase in drift demand in the tower portion of set-back structures, and on the increase in seismic demand for buildings with discontinuous distribution in mass, stiffness and strength. The largest seismic demand is found for the buildings with combined stiffness and strength irregularity.

Haselton et al. [10] presented recommendations for target spectra, selection of ground motions, and scaling of motions to be consistent with different target spectra. The paper provides guidance to design professionals on selection and scaling of ground motions for the purpose of nonlinear response-history analysis. Specific recommendations for ASCE 7-10 are also provided, along with a summary of future research needs. Soumyan and Balakrishnan [11] performed Pushover Analysis of three RC framed structures based on default hinge properties of beams and columns by using the software ETABS as per FEMA-356. The same model was also analysed based on the user defined hinge properties according to the moment-curvature relationship of beams and columns which was developed using the stress-strain curve of RCC sections given in IS 456. The performance points obtained from the analyses by the default and user-defined hinge properties are compared. The performance points of frames obtained by the analysis using default and user-defined hinge properties of concrete sections were different. The difference in the value of lateral displacement obtained from default and user-defined hinge properties was smaller while the base shear corresponding to the performance points was about 2-2.5 times greater for the models with user-defined hinge properties than that observed for the models with default hinge properties. Mandloi et al. [12] analysed four different vertically irregular building models with and without mass irregularity, using four different time histories. The evaluation criteria considered were storey deflection, storey drift, overturning moment and base shear. The necessity of considering time history data of different earthquakes for designing irregular buildings is emphasized.

Shelke et al. [13] carried out Response spectrum analysis (RSA) of vertically irregular RC buildings. Comparison of the results of analysis, and design of irregular structures with regular structure was done, considering three types of irregularities; viz. mass irregularity, stiffness irregularity, and vertical geometry irregularity. The absolute top storey displacements of geometrically irregular structures were found to be greater than those in case of regular structure. Lherminier et al. [14] presented the comparison between seismic time history analysis and pushover analysis, using nonlinear global model for RC mono-layer shell elements. By means of an analytical multi-scale analysis, the model takes into account four different dissipative phenomena; concrete cracking, concrete damage, steel-concrete slip and steel yielding. The paper explores the validity range of this constitutive law through cyclic time history analyses and monotone pushover static analyses on asymmetric structures. Comparisons have shown that forces and displacements estimated by pushover analyses and that time-history analyses give a lot of local results (crack width, steel yielding) that emphasize on these two different nonlinear methods for seismic analysis.

Many researchers have worked on various aspects of the irregular building frames; however, there are limited studies on the comparison of results from the two types of nonlinear analyses, viz. pushover analysis and time history analysis. Hence further research is needed to provide recommendations for selecting a step-back pattern for constructing an irregular building.

In the present study nonlinear static (pushover) and dynamic (time history) analysis procedures for RC frame buildings, and the relevant provisions of FEMA 356 [15], ATC 40 [16] and ASCE 41-17 [17] are employed. The damage pattern, inter-storey drifts, ultimate roof displacement expected during the nonlinear seismic response are evaluated. Performance point of each frame is obtained from analysis on ETABS [18]. The difference in behaviour of regular and step-back buildings, and their susceptibility to seismic damage are detected.

From the literature review on step-back buildings it is concluded that conventional modelling techniques and assumptions are not valid when the structural elements cross their elastic limits. This calls for the necessity of nonlinear analysis for assessing the performance of irregular buildings. The past research also suggests that time history analysis should be used in addition with pushover analysis, to appropriately predict the behaviour of the structure. Thus, the specific objectives of the study are formulated as : (i) to observe the damage pattern, inter-storey drifts, and ultimate roof displacement that can be expected during the nonlinear seismic analysis, (ii) to obtain forces from dynamic analysis and compare them with the static (pushover) analysis results, (iii) to obtain capacity curves and performance points of all frames, and (iv) to identify the difference in performance of regular and step-back buildings and their susceptibility to seismic damage.

METHODOLOGY

A total of eight different RC buildings are considered for the study, out of which four are 5 storey, and four are 10 storey. In each set, one frame is a regular frame and the remaining three are irregular frames i.e., frames with step-backs. The three irregular buildings are assumed to have different step-back configurations only in one direction. Each building has 5 bays, with plan dimensions (25 m × 25 m). For all frames, the floor-to-floor height is 3.5 m and the bay width is 5 m. The buildings are designed according to IS 456:2000 [19] and IS 1893 (Part 1) : 2016 [1]. Nonlinear static (pushover) and nonlinear dynamic (time history) analyses are performed using ETABS 2017 software [18]. Further, the effects of step-backs on the dynamic characteristics of buildings, yield patterns, seismic performance and storey drift are studied.

The analysis is done considering different building performance levels, viz. Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). For nonlinear dynamic analysis, time history data of seven real earthquake ground motions are considered. Critical members and areas of damage concentration are identified based on the hinge formation pattern and the number of hinges formed. Pushover and capacity curves are plotted for each building frame model. The base shear and roof top displacement at the performance point clearly show the difference in performance of regular and step-back buildings.

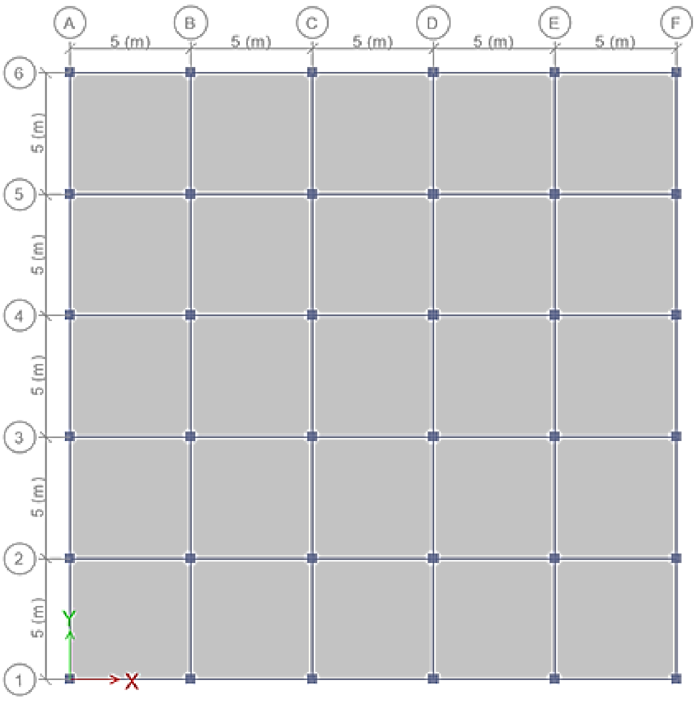


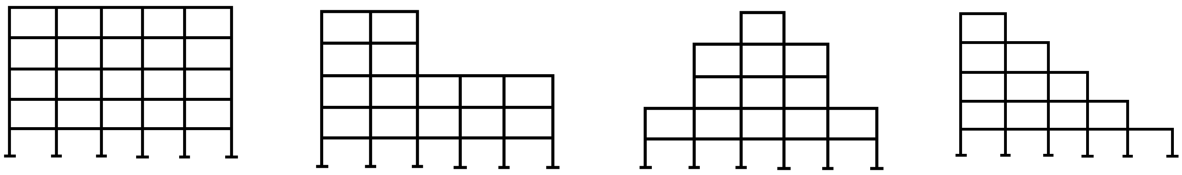
Fig. 3 First floor plan

The first-floor plan (common for all the models) is shown in Figure 3. The terminology used is shown in Table 1, where R stands for regular frame and S stands for building with step-back. The numbers indicate the number of storeys and the type step-back configuration.

Table 1: Frame terminology

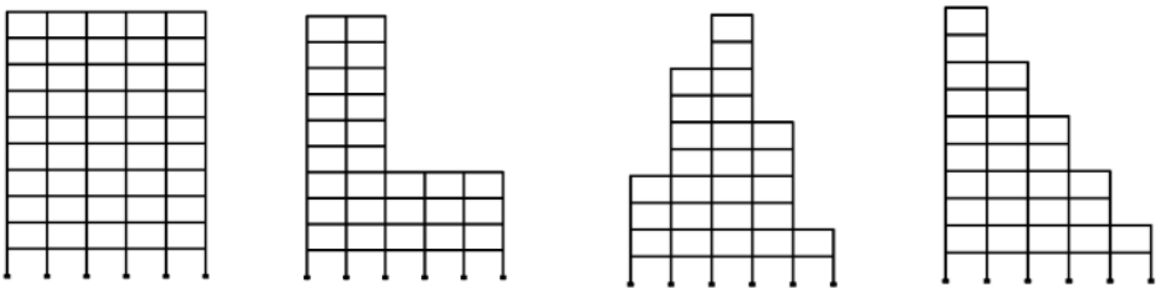
Configuration	Number of storeys	Type of frame
R5	5	Regular
S5-1	5	Irregular
S5-2	5	Irregular
S5-3	5	Irregular
R10	10	Regular
S10-1	10	Irregular
S10-2	10	Irregular
S10-3	10	Irregular

Figure 4 and Figure 5 respectively show the different configurations of frames of 5 storey and 10 storey buildings.



a) R5 (b) S5-1 (c) S5-2 (d) S5-3

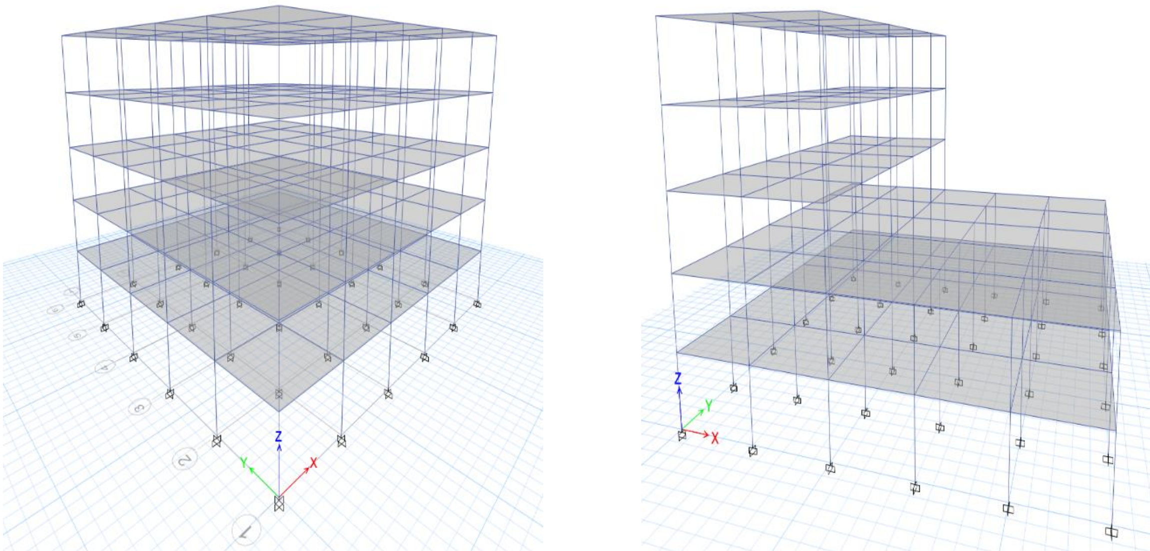
Fig. 4 Different configurations of frames of 5 storey buildings



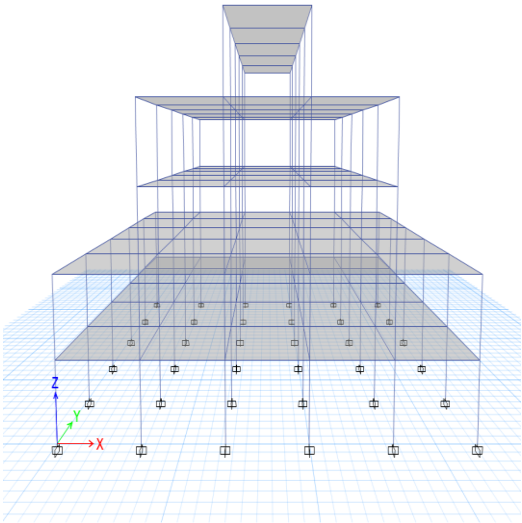
(a) R10 (b) S10-1 (c) S10-2 (d) S10-3

Fig. 5 Different configurations of frames of 10 storey buildings

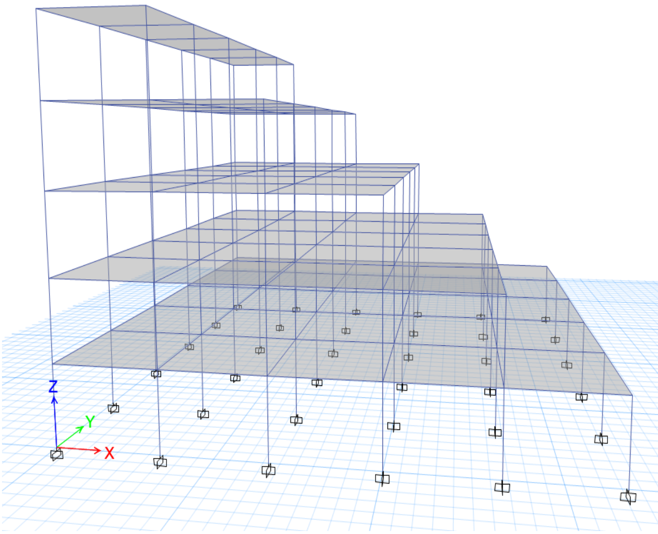
In Figure 4 and Figure 5, (a) shows regular frames, (b) shows frames with single step-back on one side, (c) shows frames with step-backs on both sides, and (d) shows frames with five step-backs on only one side; at each storey for 5 storey frame, and at alternate storeys for 10 storey frames.



(a) Configuration R5 (b) Configuration S5-1



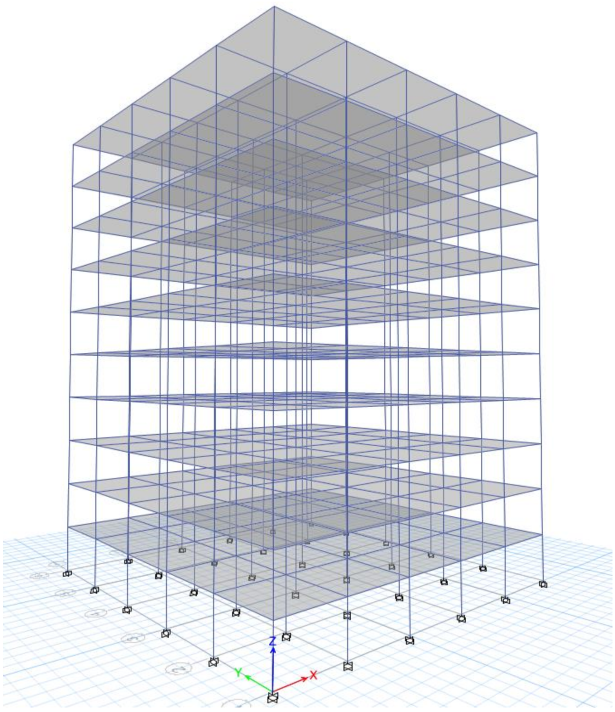
(c) Configuration S5-2



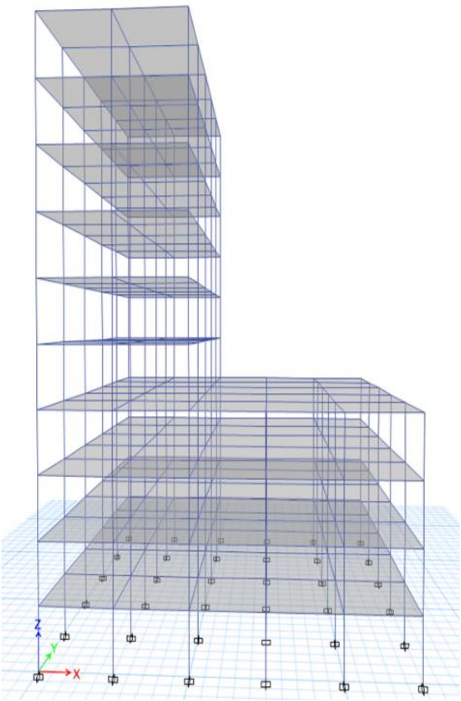
(d) Configuration S5-3

Fig. 6 Elevations of different configurations of frames of 5 storey buildings

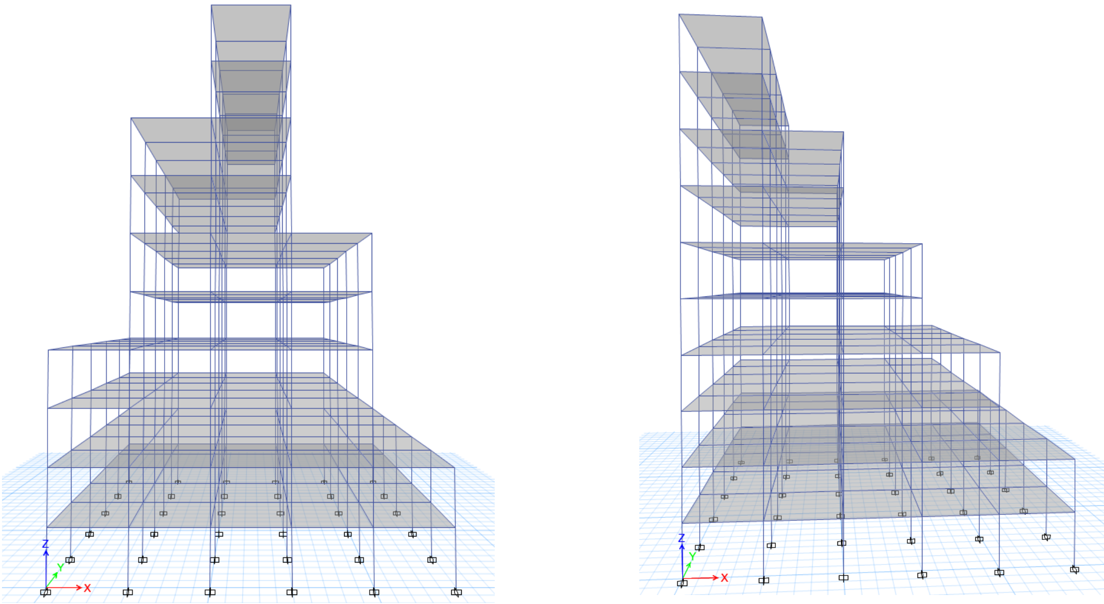
Figure 6 and Figure 7 respectively show the elevations of different configurations of 5 storey and 10 storey buildings.



(a) Configuration R10



(b) Configuration S10-1



(c) Configuration S10-2 (d) Configuration S10-3

Fig. 7 Elevations of different configurations of frames of 10 storey buildings

For obtaining precise results in nonlinear analysis, the particulars of reinforcement provided are required in order to calculate exact hinge properties using concrete models, such as the confined Mander model in ETABS. The structure has to be designed in order to obtain the reinforcement details. POA and THA are second-stage analyses procedures. Consequently, the methodology followed herein to an accurate seismic design is: (i) a linear seismic analysis, based on which primary structural design is done, (ii) insertion of hinges, determined based on the design, and (iii) pushover analysis and time history analysis. The specific purpose of nonlinear analysis is to identify the members or regions, where modification in the design or structural strengthening is required. Linear seismic analysis is done using equivalent static method (ESM) and response spectrum method (RSM), considering M30 concrete and Fe415 steel. The seismic parameters considered are: seismic zone IV (Z = 0.24), importance factor (I) = 1.2, response reduction factor (R) = 5 (SMRF), and soil type = II (medium stiff soil site).

TIME HISTORY ANALYSIS

The fundamental time period of the buildings is shown in Table 2 and Table 3.

Table 2: Time period of 5 storey buildings

Configuration	T (sec)
R5	1.429
S5-1	1.141
S5-2	1.031
S5-3	1.105

Table 3: Time period of 10 storey buildings

Configuration	T (sec)
R10	1.680
S10-1	1.562
S10-2	1.273
S10-3	1.457

From the tables it is observed that the step-back buildings have smaller time period as there is reduction in mass and stiffness, compared to the regular ones. Seven pairs (for two orthogonal directions) of ground motion data are used considering three Indian and four global earthquakes which occurred in the span of 76 years. Care has been taken to select the data of various time durations (short and long) ranging from 40 sec to 140 sec, and peak ground acceleration (PGA) ranging from 0.10g to 0.83g. Table 4 shows the particulars of the earthquake data, arranged as per the year of occurrence. The earthquake ground acceleration is applied simultaneously in both directions.

Table 4: Earthquake Ground Motion Data [20, 21]

Sr. No.	Earthquake	Magnitude	Duration (sec)	PGA (g)
1	Imperial Valley, USA (1940)	6.9	53	0.32
2	Uttarkashi, India (1991)	7.0	40	0.31
3	Northridge, USA (1994)	6.7	60	0.59
4	Kobe, Japan (1995)	6.9	140	0.83
5	Bhuj, India (2001)	7.6	133	0.10
6	Sikkim, India (2011)	6.9	92	0.27
7	Christchurch, New Zealand (2016)	5.7	132	0.36

1 Comparison of Twisting Moments

Maximum twisting moment at the base of the 5 storey and 10 storey models is obtained. The results are presented in Table 5 and Table 6, respectively. The percentage increase in the twisting moment as compared to the regular frame is depicted in parenthesis.

Table 5: Maximum twisting moment at the base of 5 storey structure (× 10^3 kNm)

Earthquake Pair	R5	S5-1	S5-2	S5-3
Imperial Valley	110.26	149.73 **(35.8%)**	137.38 **(24.6%)**	134.18 **(21.7%)**
Uttarkashi	54.28	76.57 **(41.2%)**	68.81 **(23.1%)**	66.49 **(22.5%)**
Northridge	39.73	44.65 **(12.4%)**	41.75 **(5.1%)**	42.07 **(5.9%)**
Kobe	50.11	61.84 **(23.4%)**	57.97 **(15.7%)**	56.37 **(12.5%)**
Bhuj	74.96	101.72 **(35.7%)**	82.83 **(10.5%)**	85.38 **(13.9%)**
Sikkim	86.53	103.48 **(19.6%)**	91.11 **(5.3%)**	94.66 **(9.4%)**
Christchurch	39.12	49.92 **(27.6%)**	42.28 **(8.1%)**	43.03 **(10%)**

From Table 5 and Table 6 it can be seen that the twisting moment for each irregular building is higher than that of the corresponding regular building. The % increase in the twisting moment is the highest for frame S5-1 and S10-1 for 5 storey and 10 storey models, respectively, for all earthquakes. The performance of frames S5-2, S5-3, S10-2 and S10-3 is superior to S5-1 and S10-1, but the twisting moments are still significantly higher than those of the regular frames R5 and R10.

Table 6: Maximum twisting moment at the base of 10 storey structure (×10^3 kNm)

Earthquake Pair	R10	S10-1	S10-2	S10-3
Imperial Valley	134.93	229.77 **(70.2%)**	202.92 **(50.3%)**	164.65 **(22.0%)**
Uttarkashi	74.18	130.22 **(75.5%)**	126.77 **(70.8%)**	110.93 **(49.5%)**
Northridge	58.25	79.80 **(36.9%)**	69.46 **(19.2%)**	65.64 **(12.7%)**
Kobe	89.45	**144.19** (61.2%)	**140.17** (56.7%)	**107.33** (19.9%)
Bhuj	101.79	**155.94** (53.2%)	**132.44** (30.1%)	**136.29** (33.9%)
Sikkim	127.76	200.02 **(56.5%)**	131.23 **(2.7%)**	146.07 **(14.3%)**
Christchurch	50.12	**73.32** (46.3%)	**61.44** (22.6%)	**54.05** (7.85%)

PUSHOVER ANALYSIS

Pushover analysis is carried out on buildings which are previously in a stressed state due to gravity loads. The model of a structure considered for the analysis is gently ‘pushed over’ by a monotonically increasing lateral load, applied in steps up to a predetermined value or state. In the Capacity Spectrum Method (CSM) [16], the load is incremented and checked at each stage, until the 'Performance Point' condition is reached. In this study, the CSM is dealt with, as it is found to be more suitable for RC structures [16]. The location of the performance point relative to the performance levels defined by the capacity curve indicates whether or not the performance objective is met. The nonlinearity in POA is incorporated in the form of nonlinear hinges inserted into an otherwise linear elastic model generated in ETABS. The maximum roof top displacement is set to 2% of the total height of building, which is adequate to achieve the performance point [3]. For 5 storey buildings, the total height is 17.5 m and for 10 storey buildings it is 35 m. Thus, the roof top displacements are set to 0.35 m and 0.7 m respectively. This displacement is applied in the direction of the load, at the centre-most joint of each frame.

1 Capacity-Demand Curves and Performance Point

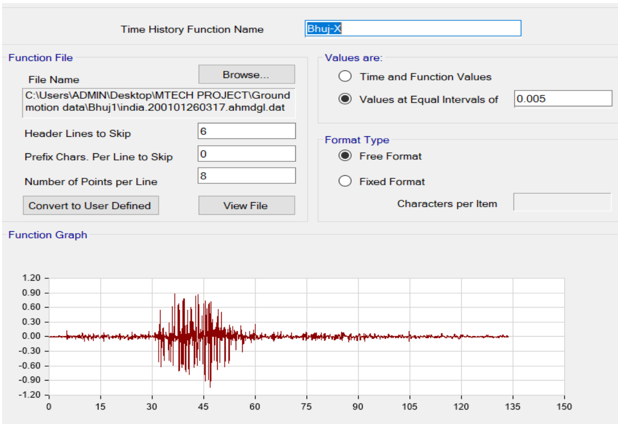


Fig. 8 Time history of Bhuj earthquake

The earthquake data is scaled according to the seismic parameters of response spectra selected for the study. ETABS provides option for spectral matching in time domain. ASCE 7-10 [22] suggests that each pair of ground motions shall be scaled over a period range from 0.2T to 1.5T, where T is the fundamental time period of the building under consideration. This means that the response spectrum of a particular time-history data should lie above that of the target spectra at least for the period range of 0.2T to 1.5T. Hence all the time-history pairs were matched using this procedure and the scaling factor, defined as the ratio of the spectral acceleration corresponding to the targeted time period to the spectral acceleration corresponding to the fundamental time period of the structure. The time history function defined for Bhuj earthquake in ETABS is shown in Figure 8. The unscaled and unmatched time history data is then matched to the target response spectrum. Figure 9 shows the time history of Bhuj earthquake in x-direction matched to the response spectrum.

The behaviour of each building is studied in pushover analysis. Capacity curve (base shear vs roof displacement), and the demand curve (from the target response spectrum of each earthquake) are plotted in ETABS. The paper considers data of Bhuj (2001) earthquake. Further, these curves are converted into Acceleration Displacement Response Spectra (ADRS) format. Since both the curves are in the same format, they are superimposed. Their point of intersection is the 'Performance point'. The significance of the performance point is that it informs the user about the shear and maximum displacement that can be expected in the building during the event of an earthquake. The performance point is identified for each frame under study.

Hinges are points on a structure where cracking and yielding is expected to occur in relatively higher intensity so that they show high flexural (or shear) displacement, as it approaches its ultimate strength under cyclic loading. Cross-diagonal cracks are seen in an actual building structure after a seismic mayhem, at the either ends of beams and columns. The cross of the cracks is at a small distance from the joint, where the hinges are inserted in the beams and columns of the model. The flexural and shear hinges are inserted into the ends of beams and columns. Since the presence of masonry infills has significant influence on the seismic behaviour of the structure, they are modelled using equivalent diagonal struts in POA. The axial hinges are inserted at either ends of the diagonal struts thus modelled, to simulate cracking of infills during analysis.

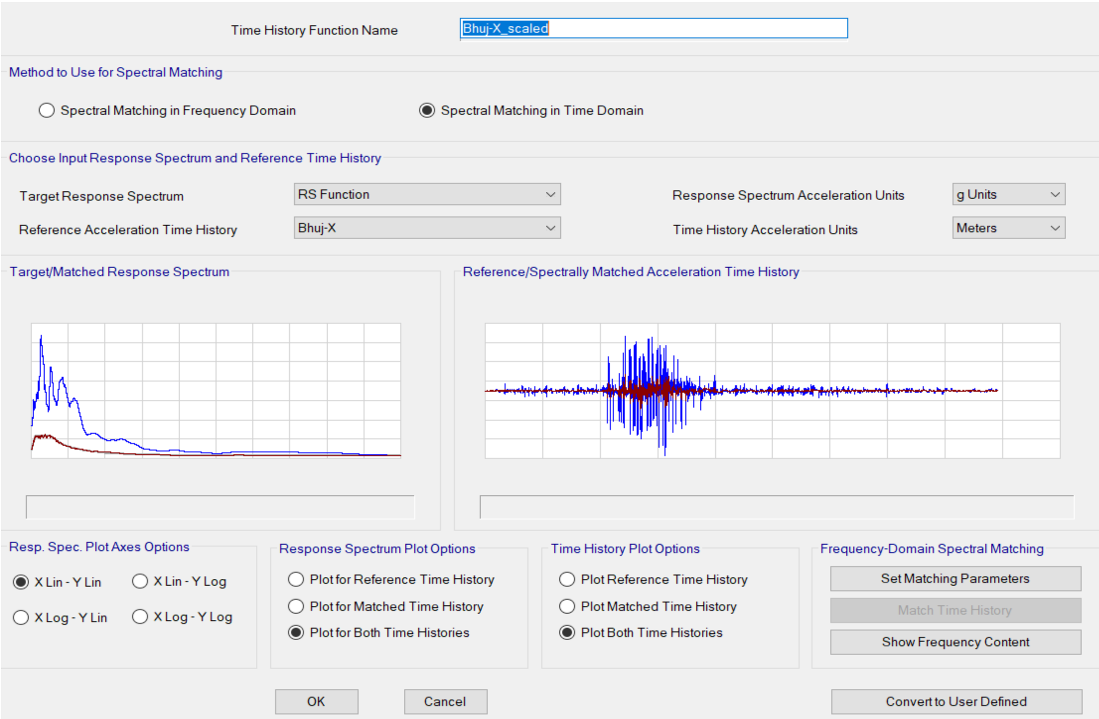


Fig. 9 Time history of Bhuj earthquake matched to response spectrum

A hinge represents localised force-displacement relation of a member through its elastic and inelastic phases under seismic loads; a flexural hinge represents the moment-rotation relation of a beam. Hinges have nonlinear states defined as ‘Immediate Occupancy’ (IO), ‘Life Safety’ (LS) and ‘Collapse Prevention’ (CP). The P-M2-M3 hinge type is considered for the nonlinear static pushover analysis.

Table 7 presents the performance point and the number of hinges formed at different performance levels for 5 storey models. It is observed that out of the three step-back buildings, maximum number of hinges are formed in the configuration S5-1, and also beyond the CP level. Table 8 presents the performance point and the number of hinges formed at different performance levels for 10 storey models. It is observed that out of the three step-back buildings, maximum number of hinges are formed in the configuration S10-1.

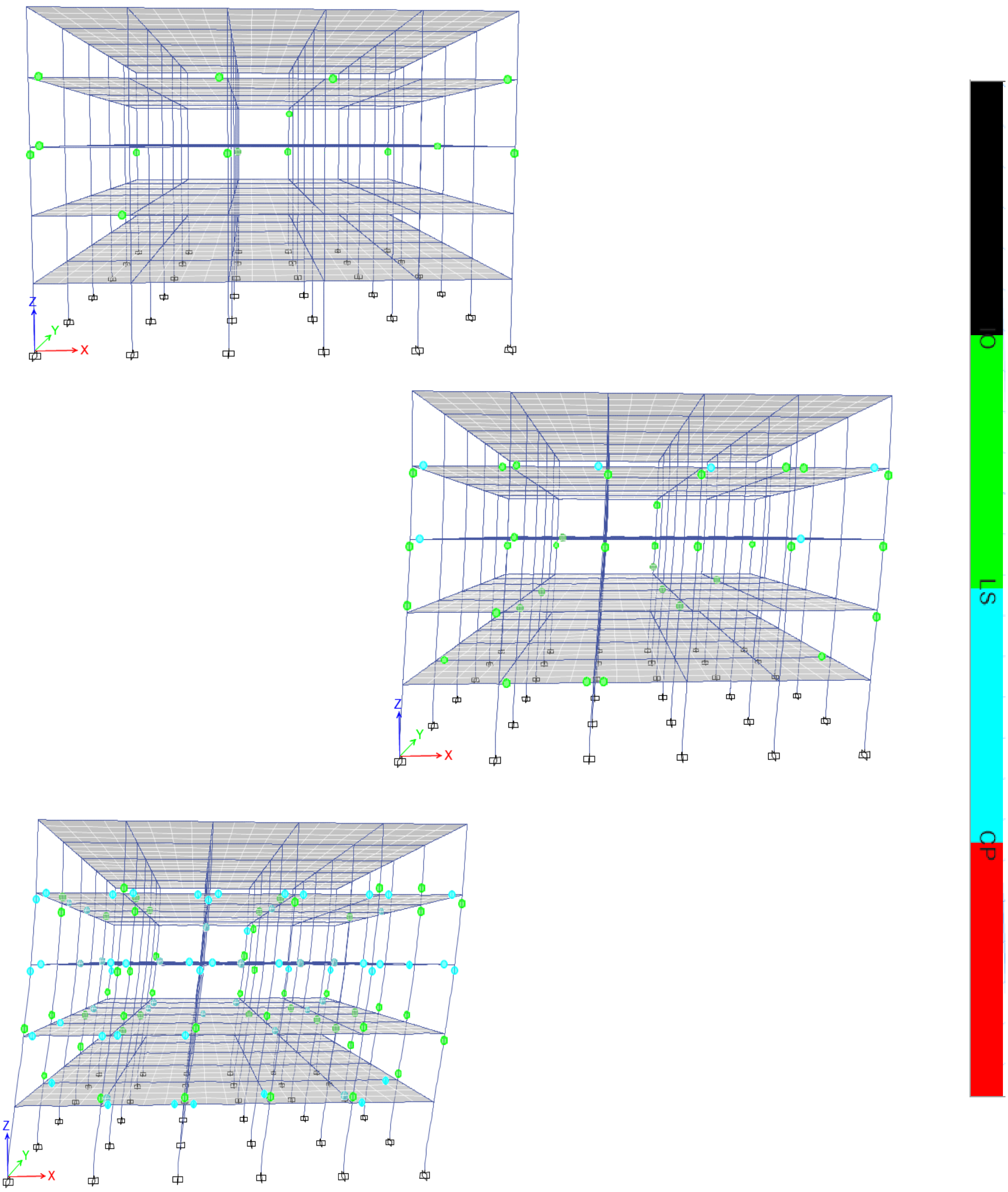
Table 7: Performance point and hinge status for 5 storey models

Frame Type	Performance point		Number of hinges formed at different performance levels				Total no. of Hinges
	Δ_{roof} (mm)	V_b (kN)	< IO	IO to LS	LS to CP	> CP	
R5	196.11	6408.15	833	112	15	0	960
S5-1	166.90	3183.24	655	74	24	**3**	**756**
S5-2	115.08	3696.60	578	87	22	1	688
S5-3	175.68	4157.63	546	61	12	1	620

Table 8: Performance point and hinge status for 10 storey models

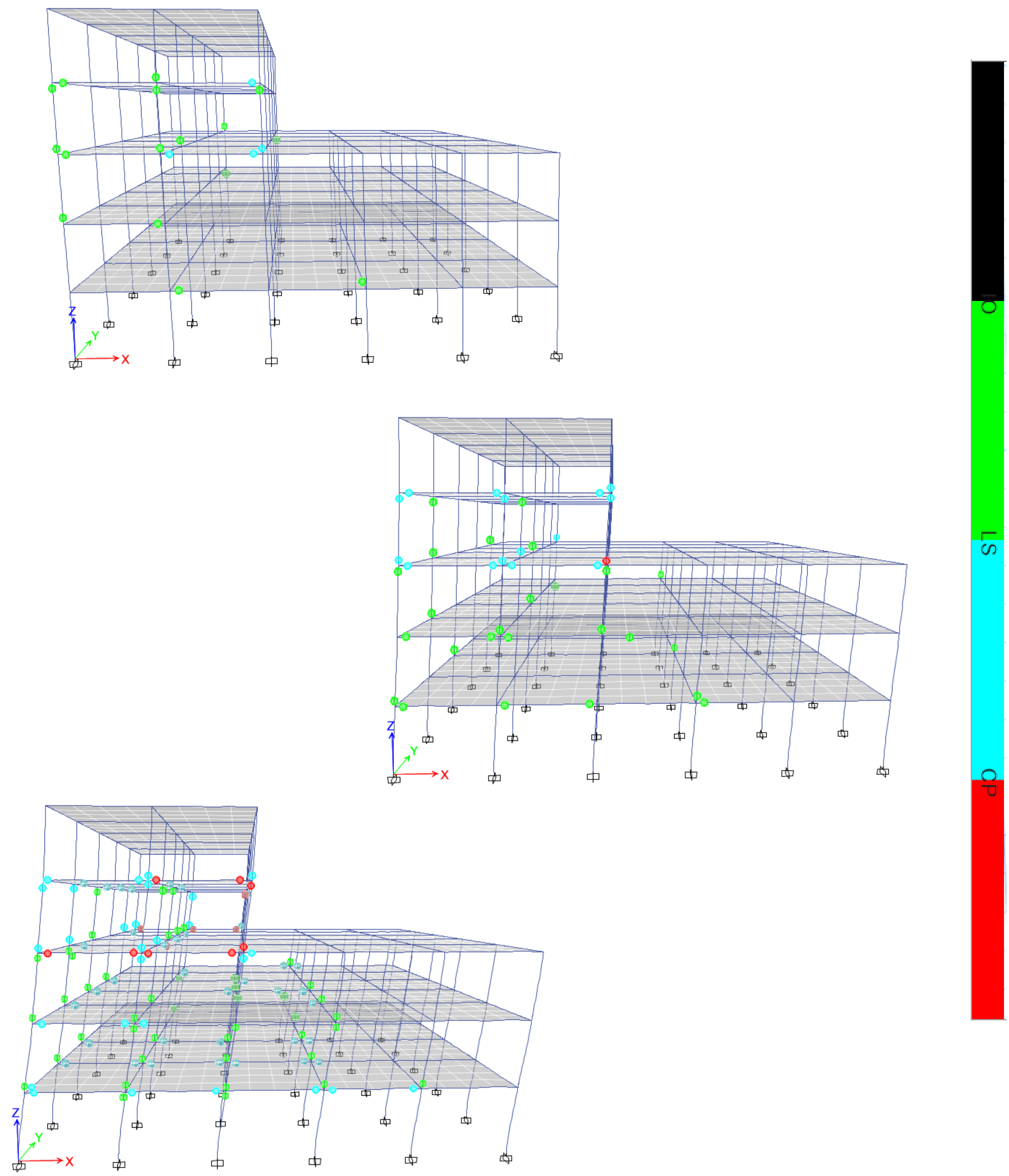
Frame	Performance point		<IO	IO to LS	LS to CP	>CP	Total no. of Hinges
	Δ_{roof} (mm)	V_b (kN)					
R10	344.84	8860.32	1667	219	32	2	1920
S10-1	252.11	4349.23	1134	126	41	7	**1308**
S10-2	207.42	4780.87	1042	148	39	11	1240
S10-3	295.34	5101.66	1111	99	25	5	1240

The hinge formation and hence the damage concentration are shown in Figures 10 and 11 for the 5 storey and 10 storey buildings, respectively. The three stages are - initial (when hinges start to form), intermediate and final (at performance point). It is observed that the damage in the step-back frame is higher as compared to the regular frame. This happens due to the way the base shear is distributed throughout the structure in case of nonlinear analysis. Due to lower stiffness at the level of step-back, the damage and number of critical hinges is higher.



(a) Initial step (b) Intermediate step (c) Final step

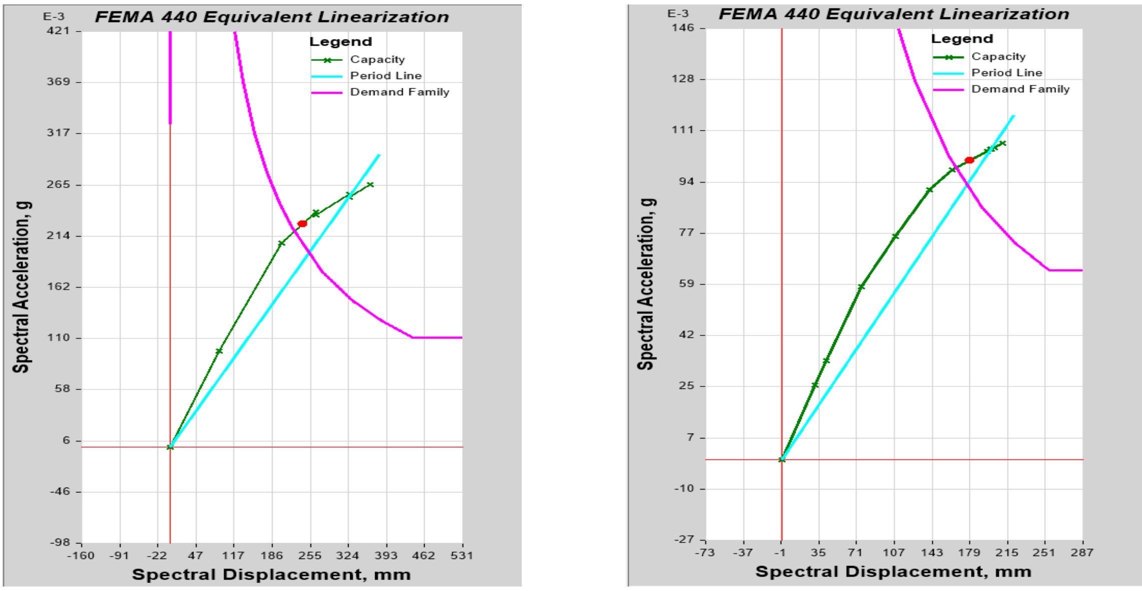
Fig. 10 Hinge progression for Frame type R5



(a) Initial step (b) Intermediate step (c) Final step

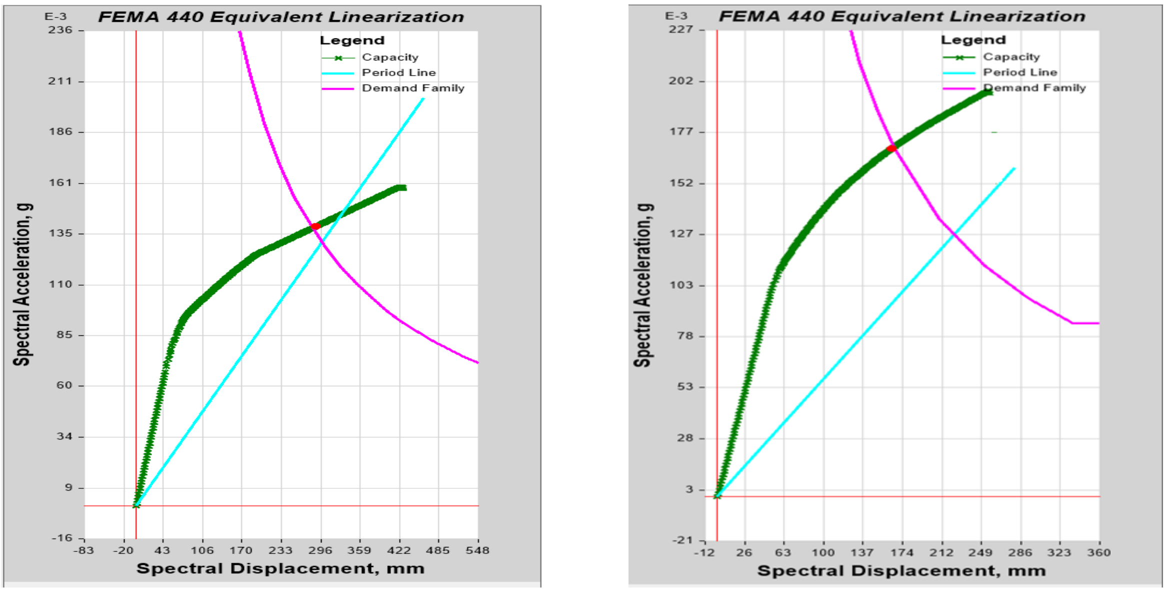
Fig. 11 Hinge progression for Frame type S5-1

Figure 12 (a) and Figure 12 (b) respectively show the performance point for 5 storey models R5 and S5-1.



(a) Frame type R5 (b) Frame type S5-1

Fig. 12 Capacity and demand curves for 5 storey buildings



(a) Frame type R10 (b) Frame type S10-1

Fig. 13 Capacity and demand curves for 10 storey buildings

Figure 13 (a) and Figure 13 (b) respectively show the performance point for 10 storey models R10 and S10-1.

From Table 7 and Table 8 it is clear that the base shear and roof displacement for the irregular frames are smaller than the corresponding values of the regular frames, at performance point. But conclusions can only be drawn after looking at the hinge progression of each frame throughout the analysis. From Figure 10 and Figure 11 it is revealed that the damage in the step-back frame S5-1 is higher as compared to the regular frame R5. This happens due to the way the base shear is distributed throughout the structure in case of nonlinear analysis. Due to lower stiffness at the level of step-back, the damage and number of critical hinges is higher beyond the CP level in all step-back buildings.

LATERAL STIFFNESS

Lateral stiffness of a storey is defined as the ratio of the storey shear to storey drift. Higher drift indicates lower storey stiffness. The stiffness of the storeys with step-backs is lower than that of the corresponding regular frames. Storey stiffness is affected by distribution the stiffness of all the lateral force resisting members in a particular storey. As the floor area decreases the number of members resisting the lateral forces also decreases. Although the weight of the floor decreases, the force distribution is such that it increases the drift in the storeys with step-back. This results in the reduction of the overall storey lateral stiffness. Table 9 and Table 10 show the variation in storey stiffness for 5 storey and 10 storey building configurations, respectively. It is observed that maximum reduction in storey stiffness is in configurations S5-1 and S10-1.

Table 9: Variation in storey stiffness for 5 storey models

Storey	S5-1		S5-2		S5-3	
	% Reduction in		% Reduction in		% Reduction in	
	Floor Area	Storey Stiffness	Floor Area	Storey Stiffness	Floor Area	Storey Stiffness
5	60	37.1	80	22.0	80	24.5
4	60	**44.8**	40	**23.8**	60	**24.7**
3	0	39.6	40	22.4	40	20.9
2	0	8.3	0	12.6	20	14.1
1	0	1.2	0	3.7	0	3.5

Table 10: Variation in storey stiffness for 10 storey models

Storey	S10-1		S10-2		S10-3	
	% Reduction in		% Reduction in		% Reduction in	
	Floor Area	Storey Stiffness	Floor Area	Storey Stiffness	Floor Area	Storey Stiffness
10	60	37.4	80	29.8	80	28.6
9	60	38.0	80	36.1	80	32.1
8	60	49.1	60	**38.7**	60	**37.6**
7	60	**55.6**	60	36.2	60	33.5
6	60	34.5	40	25.1	40	24.6
5	60	17.1	40	15.4	40	12.7
4	0	10.2	20	12.0	20	13.4
3	0	4.9	20	3.3	20	3.9
2	0	4.7	0	2.8	0	2.2
1	0	2.3	0	1.7	0	2.1

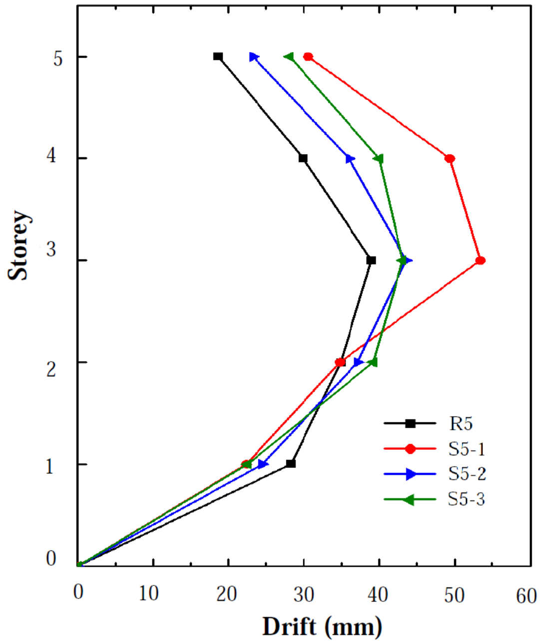
STOREY DRIFT

Inter storey drifts by time history analysis using Bhuj earthquake data and by pushover analysis are found out. The maximum drift values for each storey in all frames are presented in Table 11 for 5 storey models and in Table 12 for 10 storey models. The results are also presented graphically in Figure 14 and

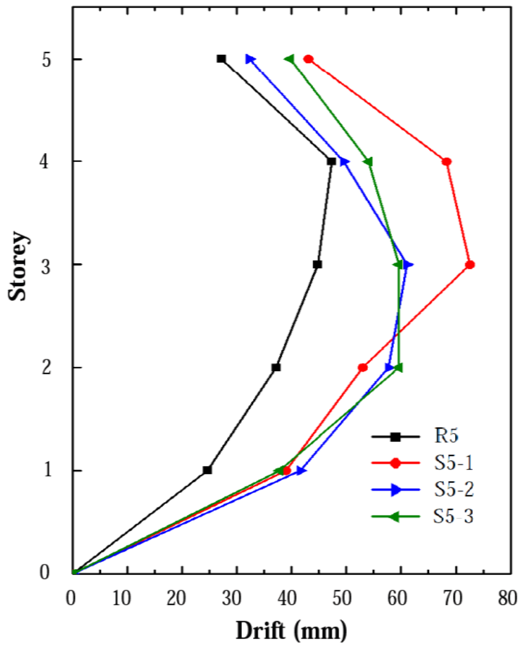
Figure 15, for 5 storey and 10 storey models respectively. It is revealed that storey drifts obtained by time history analysis are higher than those obtained by pushover analysis for all 5 storey models.

Table 11: Maximum storey drifts for 5 storey models

Storey	Drift (mm) : POA				Drift (mm) : THA			
	R5	**S5-1**	**S5-2**	**S5-3**	**R5**	**S5-1**	**S5-2**	**S5-3**
5	18.64	30.52	23.21	28.09	27.14	43.02	32.25	39.73
4	29.91	49.36	35.96	39.94	**47.33**	68.25	49.48	54.02
3	**38.92**	**53.36**	**43.51**	**43.09**	44.79	**72.57**	**61.06**	**59.60**
2	34.85	34.74	37.1	39.17	37.12	52.98	57.66	59.53
1	28.26	22.34	24.48	22.49	24.56	38.92	41.56	37.78

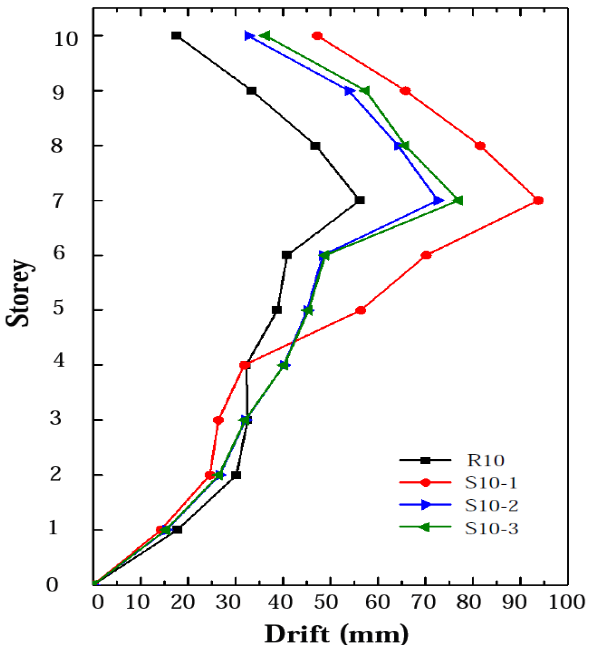


(a) POA

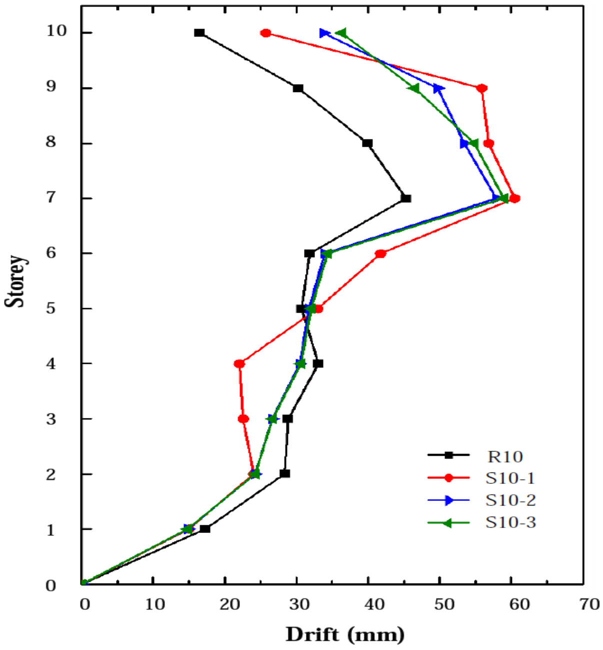


(b) THA

Fig. 14 Maximum storey drift for 5 storey models



(a) POA



(b) THA

Fig 15. Maximum storey drift for 10 storey models

Table 12: Maximum storey drifts for 10 storey models

Storey	Drift (mm) : POA				Drift (mm) : THA			
	R10	S10-1	S10-2	S10-3	R10	S10-1	S10-2	S10-3
10	17.56	47.15	32.56	36.25	16.51	25.7	33.75	36.34
9	33.36	65.69	53.75	57.33	30.26	55.87	49.71	46.48
8	46.83	81.44	64.14	65.67	39.9	56.84	53.43	54.86
7	**56.09**	**93.68**	**72.54**	**76.94**	**45.27**	**60.47**	**57.9**	**58.94**
6	40.84	70.05	48.31	48.8	31.87	41.77	33.94	34.33
5	38.73	56.31	44.99	45.3	30.69	32.95	31.74	32.06
4	32.19	31.74	40.12	40.16	33.07	22.05	30.47	30.61
3	32.48	26.34	32.06	31.98	28.87	22.53	26.62	26.66
2	30.14	24.53	26.58	26.45	28.34	24.03	24.24	24.24
1	17.73	14.25	15.42	15.29	17.26	14.96	14.82	14.79

From Table 10 it is seen that for S10-1 configuration, maximum % reduction is stiffness is at storey 7, and hence in this configuration, maximum drift occurs at storey 7, by POA (93.68 mm) and by THA (60.47 mm).

CONCLUSIONS

Comparative study on seismic performance of eight step-back buildings is presented. The results are presented by performing time history analysis and by pushover analysis. Nonlinear analysis gives large inter-storey drifts, due to the geometric and material nonlinearity, which significantly affects the post-yield load-deformation behaviour of RC members. From the trend of the numerical results obtained, following conclusions are drawn.

1. The twisting moment in all the irregular frames is significantly higher than the regular frame, for all seven earthquakes.

2. The number of hinges exceeding collapse prevention levels is greater in the irregular frames. Hinge progression in the frames shows CP hinges at the level of setbacks.
3. Pushover analysis shows that the base shear and roof displacement at performance points is lower for the step-back frames as compared to the regular frame, but the damage is more. This may be due to the difference in the distribution of storey forces.
4. For 5 storey step-back buildings, compared to the regular frame, the roof displacement is 15% lower in S5-1, 41.3% lower in S5-2 and 10.4% lower for S5-3. Further, compared to the regular frame, the base shear is 50.3% lower in S5-1, 42.3% lower in S5-2 and 35.1% lower in S5-3.
5. It is concluded that the performance of frames S5-2 and S5-3 is superior to the frame S5-1.
6. For 10 storey step-back buildings, compared to the regular frame, the roof displacement is 26.9 % lower in S10-1, 40% lower in S10-2 and 14.5% lower in S10-3. Further, compared to the regular frame, the base shear is 51% lower in S10-1, 46% lower in S10-2 and 42.4 % lower in S10-3.
7. It is concluded that the performance of frames S10-2 and S10-3 is superior to the frame S10-1.
8. In case of step-back buildings, the configurations of type S5-2, S5-3, S10-2 and S10-3 are preferred.

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