

Pushover analysis of ten storeyed RC buildings with openings in infill walls

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ABSTRACT

The frame structures with brick masonry are commonly used in RC multistoreyed buildings in recent past. Window and door openings are unavoidable part of the infill walls. The presence of openings in infill walls significantly reduces the lateral strength and stiffness of RC frames. An attempt has been made in the present paper the behavior of performance based seismic vulnerability of two-dimensional RC multistorey building models, with the varying percentage of central openings in brick masonry infill walls ranging from 10 to 35%. The brick masonry infill walls are modeled as pin-jointed single equivalent diagonal struts. Equivalent and response spectrum analysis was performed using SAP2000 V14.2 software. Nonlinear static pushover analysis is carried out using user defined hinge properties as per the FEMA 440 guidelines. The results are compared with the natural period, base shear, lateral displacement, storey drift, ductility ratio, safety ratio, global stiffness, and hinge status at performance point amongst the models. Authors conclude that increase in openings in infill walls increases the vulnerability of building models. Earthquake code procedures should be considered during design of structures.

Key words: Openings, User defined hinges, Non-linear static analysis, Performance levels, Ductility ratio, Safety ratio, Global stiffness.

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INTRODUCTION

In India large numbers of buildings are constructed with masonry infill walls. These infill walls significantly enhance the stiffness and strength of the infilled frame [1]. In the current practice of structural design in India, masonry infill panels are treated as non-structural element and their strength and stiffness contributions are neglected [2]. The RC frame action behavior with masonry infill walls illustrates the truss action, where the infill wall behaves as the diagonal strut and absorbs the lateral load under compression [3]. Several buildings constructed in India and across the world have the ground storey frames without infill walls leading to soft open ground storey. Thus, upper floors move almost together as a single block and most of the lateral displacement of the buildings occurs in the open ground storey to earthquake excitation.

Door and window openings are unavoidable parts of any structure. However, the presence of openings in infill walls decreases the stiffness and strength of the RC frame. Indian seismic code recommends no provision regarding the stiffness and openings in the masonry infill wall. Whereas, clause 7.10.2.2 and 7.10.2.3 of the “Proposed draft provision and commentary on Indian seismic code IS 1893 (Part 1) : 2002” [4], [Jain and Murty] [5] defines the provision for calculation of stiffness of the masonry infill and a reduction factor for the opening in infill walls.

Seismic responses are estimated by nonlinear static pushover analysis, frequently utilized in engineering applications. Non-linear static pushover analysis recommended in ATC 40 [6] and FEMA 356 [7] was accepted by most of the engineers. These procedures are based on monotonically increasing the predefined load patterns until the defined target displacements are achieved. Modal pushover analysis (MPA) procedure proposed by Chopra and Goel attempt to account for higher mode effects and use elastic modal combination rules [8]. The modified modal pushover analysis (MMPA), which has been recently developed by Chopra et al. [9] is an extension of MPA, combines the elastic influence of higher modes with the inelastic response of a first mode pushover analysis using modal combination rules. This paper considers pushover analysis as per FEMA 440 [10] guidelines, since SAP2000 is widely employed by structural designers throughout the world in recent past.

ANALYTICAL MODELING

In the present study two-dimensional ten storeyed RC frame buildings are considered. The plan and elevation of the building models are shown in Figure 1 and Figure 2. The bottom storey height is 4.8 m and upper floors height is 3.6 m [11]. The building is assumed to be located in zone III. M25 grade of concrete and Fe415 grade of steel are considered. The stress-strain relationship is used as per IS 456 : 2000 [12]. The brick masonry infill walls are modeled as pin-jointed equivalent diagonal struts. M3 (*Moment*), V3 (*Shear*), PM3 (*axial force with moment*), and P (*Axial force*) user defined hinge properties are assigned at rigid ends of beam, column, and strut elements. The density of concrete and brick masonry is 25 [13] and 20 kN/m³ [13]. Young's modulus of concrete and brick masonry is 25000 MPa [12] and 3285.9 MPa [14]. Poisson's ratio of concrete is 0.3 [15]. 10% to 35% [11] of central openings are considered and analytical models developed are,

Model 1 - Building has no walls and modeled as bare frame, however masses of the walls are considered.

Building has no walls in the first storey and walls in the upper floors and modeled as soft storey with varying central opening of the total area, however stiffness and masses of the walls are considered.

Model 2 - 10%.

Model 3 - 15%

Model 4 - 20%

Model 5 - 25%

Model 6 - 30%

Model 7 - 35%

Models are designed for $1.2(DL+LL+EQ)$ and $1.2(DL+LL+RS)$.

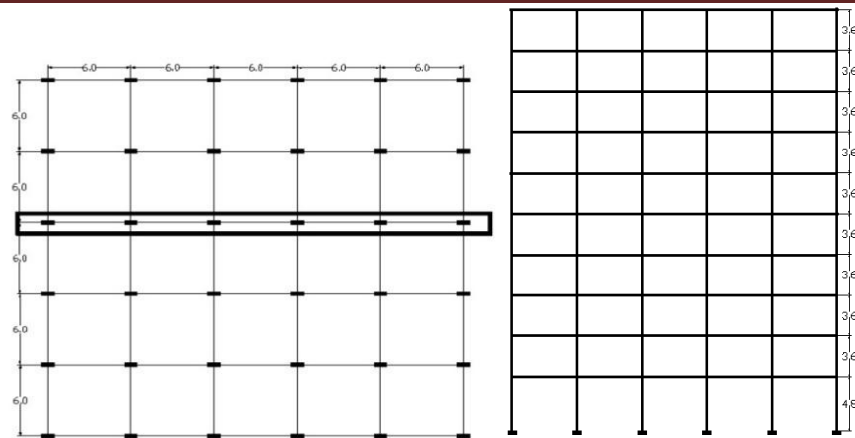


Fig 1. Plan and elevation of bare frame building

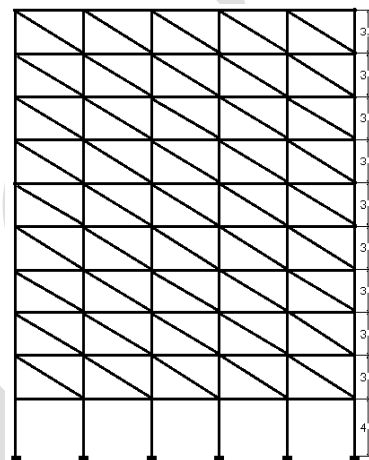


Fig 2: Elevation of soft storey models

METHODOLOGY OF THE STUDY

In this present study, equivalent static (ESM) and response spectrum methods (RSM) as per the seismic code IS 1893 (Part 1) : 2002 [4] for the bare frame and soft storey models are carried out. An effort is made to study the linear static and dynamic responses to the base shear distribution along the height of buildings as computed by ESM and RSM.. Their performance point and location of hinges are evaluated using nonlinear static pushover analysis.

USER DEFINED HINGES

The definition of user-defined hinge properties requires moment–curvature analysis of beam and column elements. Similarly load deformation curve is used for strut element. For the problem defined, building deformation is assumed to take place only due to moment under the action of laterally applied earthquake loads. Thus user-defined M3 and V3 hinges for beams, PM3 hinges for columns and P hinges for struts are assigned. The calculated moment-curvature values for beam (*M3 and V3*), column (*PM3*), and load deformation curve values for strut (*P*) are substituted instead of default hinge values in SAP2000.

MOMENT CURVATURE FOR BEAM SECTION

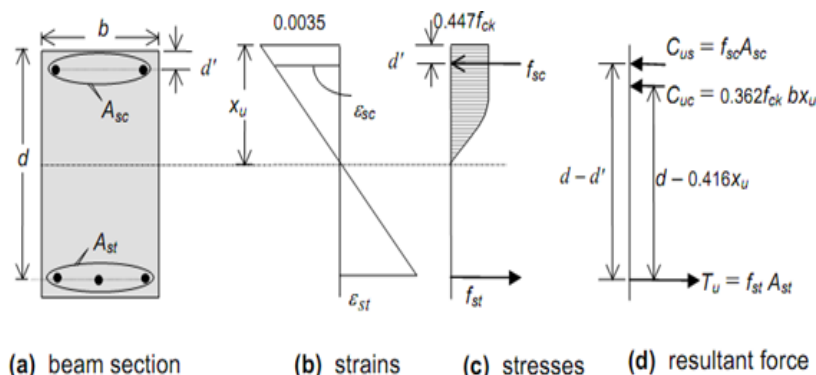


Fig 3: Stress-Strain block for beam [15]

Following procedure is adopted for the determination of moment-curvature relationship considering unconfined concrete model given in stress-strain block as per IS 456 : 2000 [12].

1. Calculate the neutral axis depth by equating compressive and tensile forces.
2. Calculate the maximum neutral axis depth $x_{u\max}$ from equation 1.

$$\frac{0.0035}{x_u} = \frac{\left(\frac{f_y}{E_s} + 0.002\right)}{(d - x_u)} \dots \dots \dots (1)$$

3. Divide the $x_{u\max}$ in to equal laminae.
4. For each value of x_u get the strain in fibers.
5. Calculate the compressive force in fibers corresponding to neutral axis depth.
6. Then calculate the moment from compressive force and lever arm ($C \times Z$).
7. Now calculate the curvature from equation 2.

$$\phi = \frac{\epsilon_s}{d - x_u} \dots \dots \dots (2)$$

8. Plot the moment curvature curve which is shown in Fig 4 and calculated values of moment-curvatures are presented in Table 1.

Assumptions made in obtaining moment curvature curve for beam and column is:

- 1) The strain is linear across the depth of the section ('Plane sections remain plane').
- 2) The tensile strength of the concrete is ignored.
- 3) The concrete spalls off at a strain of 0.0035 [12].
- 4) The point 'D' is usually limited to 20% of the yield strength, and ultimate curvature, θ_u with that [6].
- 5) The point 'E' defines the maximum deformation capacity and is taken as $15\theta_y$ whichever is greater [6].
- 6) The ultimate strain in the concrete for the column is calculated as 0.0035-0.75 times the strain at the least compressed edge (IS 456 : 2000) [12]

Table1. Moment curvature values of beam

Moment	Curvature
0	0
1	0.01048
1.61	0.03345
0.2	0.03345
0.2	0.15727

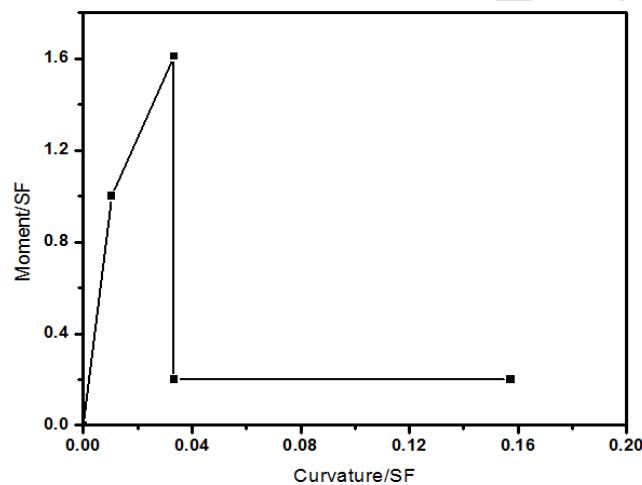


Fig 4: Moment curvature curve for beam

MOMENT CURVATURE FOR COLUMN SECTION

Following procedure is adopted for the determination of moment-curvature relationship for column.

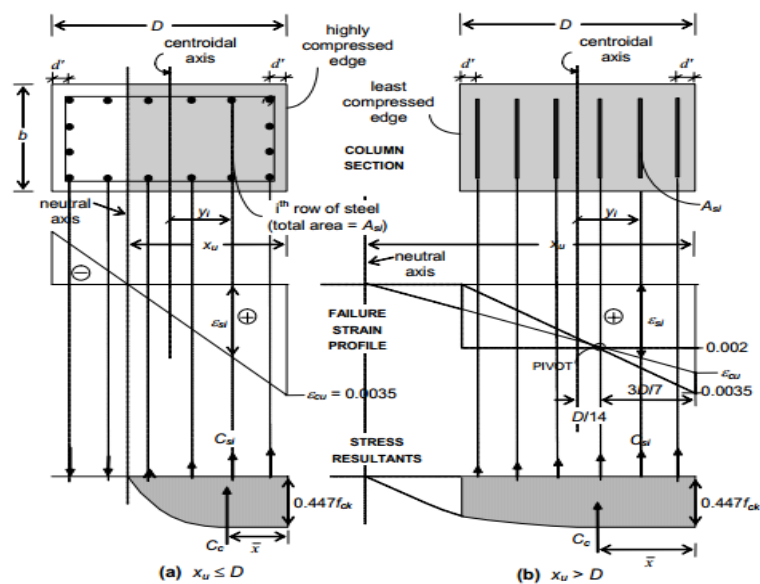


Fig 5: Analysis of design strength of a rectangular section under compression [15]

1. Calculate the maximum neutral axis depth $x_{u\max}$ from equation 3.

$$\frac{0.0035}{x_u} = \frac{\left(\frac{f_y}{E_s} + 0.002\right)}{(d - x_u)} \dots\dots\dots (3)$$

2. NA depth is calculated by assuming the neutral axis lies within the section.
3. The value of x_u is varied until the value of load (P) tends to zero. At $P = 0$ kN the value of x_u obtained is the initial depth of NA.
4. Similarly, NA depth is varied until the value of moment tends to zero. At $M = 0$ kN-m the value of x_u obtained will be the final depth of NA.
5. The P-M interaction curve is plotted in Figure 6 for the obtained value of load and moment. These values are presented in Table 2.
6. For the different values of x_u , the strain in concrete is calculated by using the similar triangle rule.
7. The curvature values are calculated using equation 4,

$$\phi = \frac{\epsilon_c}{x_u} \dots\dots\dots (4)$$

8. Plot the moment curvature curve shown in Figure 7 and moment curvature values are presented in the Table 3.

Table2. Axial load and moment values for P-M interaction curve

X_u	P_u	M_u	Strain in concrete	Curvature
217.4	0	797.46	0.00248	0.01141
247.4	235.34	816.94	0.00282	0.01142
277.4	487.34	833.19	0.00317	0.01141
306.63	752.97	845.23	0.00350	0.01141
700	1408.18	750.74	0.00449	0.00641
1000	2756.275	19.59	0.00255	0.00255
1300	2802.743	8.81	0.00239	0.00184
1600	2828.091	2.803	0.00231	0.00144
1850	2841.84	0	0.00226	0.00122

Table3. Moment-curvature values for column

Points	Moment	Curvature
A (Origin)	0.00	0.00000
B (Yielding)	1.00	0.00641
C (Ultimate)	1.126	0.01141
D (strain hardening)	0.20	0.01141

E (strain hardening)	0.20	0.09621
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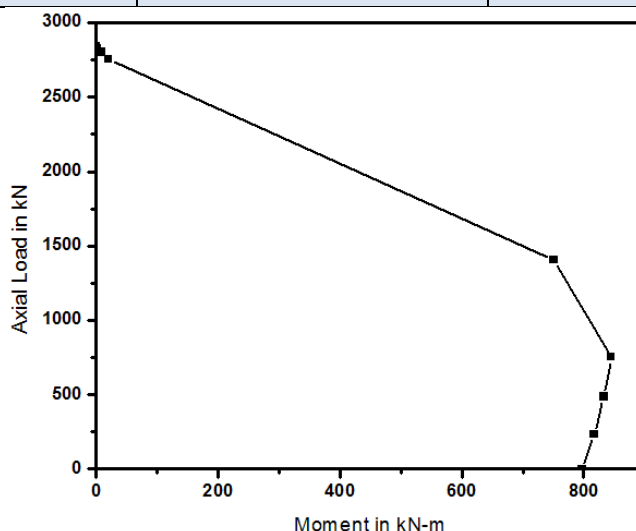


Fig 6: P-M interaction curve

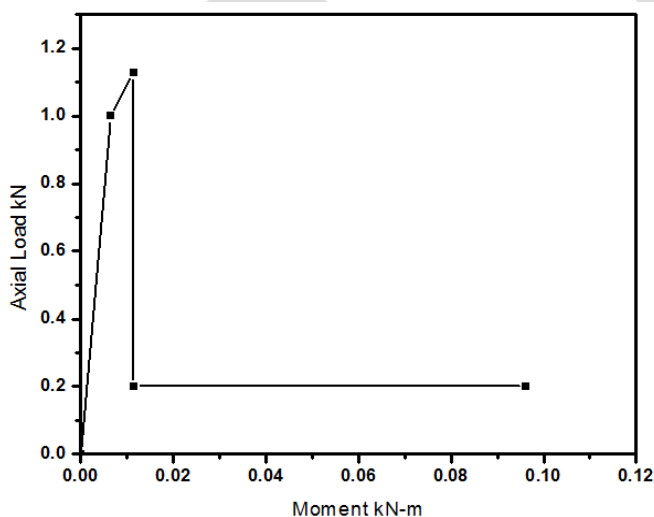


Fig 7: Moment curvature curve for column

PUSHOVER ANALYSIS

Pushover analysis is one of the methods available to understand the behavior and vulnerability of structures subjected to earthquake forces. As the name implies, it is the process of pushing horizontally with a prescribed loading pattern incrementally until the structure reaches a limit state [6]. The static approximation consists of applying a vertical distribution of lateral loads to models which captures the material non - linearity of an existing or previously designed structure. The loads are monotonically increased until the peak response of the structure is obtained on a base shear vs. roof displacement plot. Researchers have developed several push over analysis methods. This paper considers the procedures prescribed by FEMA 440 [10]. Maximum displacement equal to 4% of the height of building [6] at roof level and

number of steps in which this displacement must be applied, are defined. The global response of models at each displacement level is obtained in terms of the base shear presented by pushover curve. Pushover curve is a base shear versus roof displacement curve, which enlightens about the shear force developed at the base of the structure at any stage of push.

ELEMENT DESCRIPTION OF SAP2000

Frame element in SAP2000 is modeled as a line element having linearly elastic properties and nonlinear force-displacement characteristics, individual frame elements are modeled as hinges represented by a series of straight line segments.

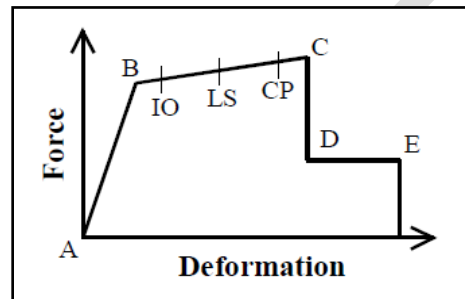


Fig 8: Force-Deformation for Pushover Hinge [16]

Point A corresponds to unloaded condition and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads [16]. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained. There are three types of hinge properties in SAP2000. They are default hinge properties, user-defined hinge properties and generated hinge properties. In this present paper only user defined hinge properties are assigned to the frame elements.

RESULTS AND DISCUSSIONS

FUNDAMENTAL NATURAL PERIOD

It is the longest modal time period of vibration [4]. The analytical (SAP 2000) and empirical formula mentioned in IS 1893 (Part1) : 2002 natural periods of the building models are presented in the Table 4.

Table4. Analytical and codal natural periods

Model No.	Analytical (sec)	Code (sec)
1	2.402	1.129
2	1.1971	0.611
3	1.2032	0.611

4	1.2100	0.611
5	1.2175	0.611
6	1.2204	0.611
7	1.2253	0.611

Stiffness of the building is directly proportional to its natural frequency and inversely proportional to the natural period. If the stiffness of the building decreases, the natural periods are longer. From the results it is observed that, the natural period of model 1 is longer by 2.01 times compared to the model 2. This is because the stiffness of masonry infill being ignored during design. As the percentage of openings increases the fundamental natural periods are longer indicating enhance in ductility.

BASE SHEAR

It is the total design lateral force at the base of the structure [4]. The base shear for equivalent static and response spectrum methods as per IS 1893 (Part 1) : 2002 and the scale factor (SF) for the building models are listed in the Table 5.

Table5. Base shear

Model No.	(kN)	V_B (kN)	Scale Factor
1	508.4	223	2.28
2	976.93	475.6	2.05
3	967.66	472.73	2.04
4	937.62	458.02	2.04
5	907.59	444.95	2.03
6	877.55	432.5	2.02
7	847.51	419.98	2.01

The base shear is function of mass, stiffness, height, and natural period of the building. In the equivalent static method horizontal acceleration obtained is adopted and basic assumption in the equivalent static method is that only first mode of vibration of building governs. Higher modes are not considered, therefore base shear determined from equivalent static method are larger than the dynamic response spectrum method as all the modes are considered. It is observed that, as the percentage of central openings increases the base shear decreases by 1.16 and 1.13 times the model 2 and model 7 for equivalent static and response spectrum method respectively. These results reveal that, as the percentage of openings increases the strength in the buildings decreases thereby indicating a lesser amount of earthquake carrying capacity.

LATERAL DISPLACEMENT

The profile of lateral displacements for the building models obtained by equivalent static (ESM) and response spectrum method (RSM) is shown in Figure 9. The lateral displacement of a building is a function of the stiffness. Lateral displacement of the building increases with the decreases in the lateral stiffness. Figure 9 show that, lateral displacements of model 2 to model 7 are less than model 1. From these results it is observed that, there is decrement in the

lateral displacement of model 2 compared to model 1 by 60.32% and 52.02% for ESM and RSM. As the percentage of openings increases, lateral displacements increases nearly by 2.00 and 1.6 times from model 2 to model 7 for ESM and RSM respectively. These results lead to higher flexibility in the buildings with increase in openings.

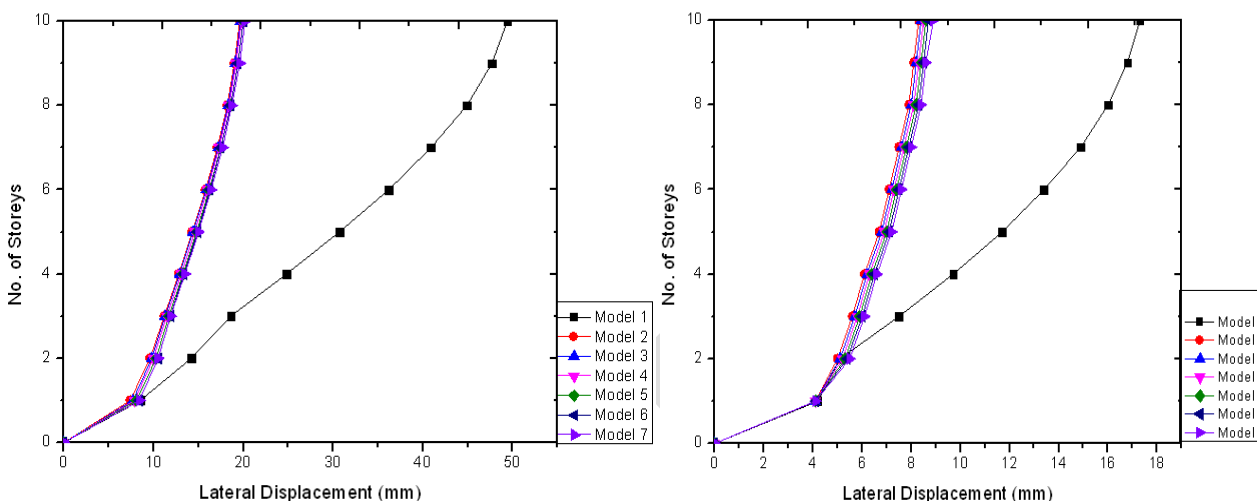


Fig 9: Lateral displacements by ESM and RSM

STOREY DRIFT

The profile of storey drift by ESM and RSM is shown in Figure 10. As per the clause 7.11.1 of IS 1893 (Part 1) : 2002, the storey drift should be within the 0.004 times the storey height [4]. Lateral displacements are 19.2 mm and 14.4 mm for the bottom and the upper floors respectively. The storey drift at all the floors are within the limit for all the models. From the results it is observed that, there is increase in the storey drift of model 1 compared to the model 2 by 2.83 and 2.5 times respectively by ESM and RSM. The drift at the first storey is more compared to the upper storeys as the first storey is designed without constructing infill walls resulting to soft storey. These static and linear results recommend that the civil engineering professionals should follow earthquake code procedures during design of multistorey buildings.

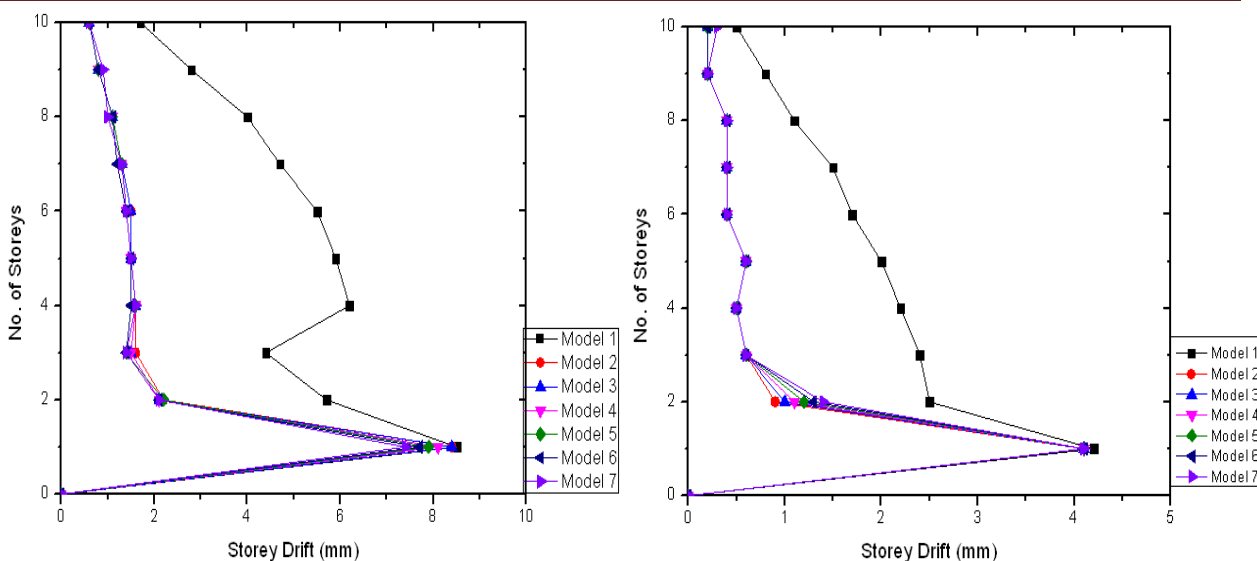


Fig 10: Storey drift for bare and soft storey buildings by ESM and RSM

PERFORMANCE EVALUATION OF BUILDING MODELS

Performance based seismic evaluation of the models is carried out by non linear static pushover analysis. Base force distribution along the height of buildings calculated as per ESM and RSM are considered during pushover analysis. User defined hinges are assigned to the models.

PERFORMANCE POINT AND LOCATION OF HINGES

The base force, displacement, and the location of the hinges at the performance point for various performance levels for models are presented in the Table 6 and Table 7. The base force at performance point and ultimate point of the building depends on its lateral strength. It is seen in Table 6 and Table 7 that, as the openings increase the base force at ultimate point reduces by 1.02 times by equivalent static and response spectrum pushover analysis method in model 7 compared to model 2. As the stiffness of infill wall is considered in the soft storey buildings, base force is more than that of the bare frame building. The stiffness of the building decreases with the increase in percentage of central openings.

In most of the models, plastic hinges are formed in the first storey because of open ground storey. The plastic hinges are formed in the beams and columns. From the Table 3 and Table 4 it is observed that, the hinges formed within the life safety range at the ultimate state is 93.12%, 97.31%, 96.58%, 96.09%, 95.61%, 95.12%, and 94.63% in the models 1 to 7 respectively by equivalent static pushover analysis (ESPA). Similarly 91.87%, 97.31%, 96.83%, 96.34%, 95.36%, 5.12%, and 94.14% hinges are developed in the models 1 to 7 respectively by response spectrum pushover analysis (RSPA). These results reveal that, seismically designed multistoreyed RC buildings are safe to earthquakes.

Table6. Performance point and location of hinges by ESPA

Mod el No.	Performance point			Location of hinges					
	Displacement mm		Base force kN	A-B	B-IO	IO – LS	LS-CP	CP-E	Total
1	Yield	76.25	650.21	280	21	4	0	15	320
	Ultimate	281.16	850.32	230	46	20	2	22	320
2	Yield	40.12	1681.45	380	16	5	4	5	410
	Ultimate	115.64	2069.54	360	22	16	1	11	410
3	Yield	40.53	1669.72	378	20	4	0	8	410
	Ultimate	118.84	2058.54	361	15	12	8	14	410
4	Yield	40.94	1657.22	382	15	7	4	2	410
	Ultimate	122.04	2047.8	358	18	12	6	16	410
5	Yield	41.35	1644.52	380	20	6	2	2	410
	Ultimate	125.24	2039.58	356	22	12	2	18	410
6	Yield	41.76	1630.84	384	12	6	5	3	410
	Ultimate	128.44	2030.69	358	18	10	4	20	410
7	Yield	42.17	1614.34	380	16	10	2	2	410
	Ultimate	131.64	2023.6	354	16	8	10	22	410

Table7. Performance point and location of hinges by RSPA

Mod el No.	Performance point			Location of hinges					
	Displacement mm		Base force kN	A-B	B-IO	IO – LS	LS-CP	CP-E	Total
1	Yield	78.65	668.36	280	21	4	0	15	320
	Ultimate	286.31	869.24	230	44	20	0	26	320
2	Yield	38.89	1701.45	380	16	5	4	5	410
	Ultimate	108.25	2093.54	360	22	15	2	11	410
3	Yield	39.21	1689.45	378	16	2	2	12	410
	Ultimate	112.05	2083.34	361	14	12	10	13	410
4	Yield	39.65	1677.45	382	15	7	4	2	410
	Ultimate	115.85	2073.14	358	17	12	8	15	410
5	Yield	40.16	1665.45	380	20	6	2	2	410
	Ultimate	119.65	2062.94	356	20	10	5	19	410
6	Yield	40.72	1653.45	384	12	6	5	3	410
	Ultimate	123.45	2052.74	358	18	8	6	20	410
7	Yield	41.3	1641.45	380	15	10	5	0	410
	Ultimate	127.25	2043.84	354	14	6	12	24	410

It is further observed that, the hinges formed beyond the CP range at the ultimate state is 6.88%, 2.68%, 3.42%, 3.90%, 4.63%, 4.89%, and 5.85% in the models 1 to 7 respectively by ESPA. Similarly 8.12%, 2.68%, 3.17%, 3.65%, 4.63%, 4.90%, and 5.85% hinges are

developed in the models 1 to 7 respectively by RSPA. As the collapse hinges are few, retrofitting can be completed quickly and economically without disturbing the incumbents and functioning of the buildings.

DUCTILITY RATIO

Ductility ratio means it is the ratio of collapsed yield (CY) to the initial yield (IY) [17]. Ductility ratios (DR) for models are presented in the Table 8.

Table8. Ductility ratio by ESPA and RSPA

Model No.	ESPA			RSPA		
	IY	CY	DR	IY	CY	DR
1	76.25	281.16	3.69	78.65	286.31	3.64
2	40.12	115.64	2.88	38.89	108.25	2.78
3	40.53	118.84	2.93	39.21	112.05	2.86
4	40.94	122.04	2.98	39.65	115.85	2.92
5	41.35	125.24	3.03	40.16	119.65	2.98
6	41.76	128.44	3.08	40.72	123.45	3.03
7	42.17	131.64	3.12	41.3	127.25	3.08

It is seen in Table 8 that, the ductility ratios of the bare frame is larger than the soft storey models specifying stiffness of infill walls not considered during analysis. DR of model 1, model 5, model 6, and model 7 are more than the target value equal to 3 by ESPA. Similar results are observed in model 1, model 6, and model 7 by RSPA. DR in remaining models is nearer to target value. These results reveal that, increase in openings increases the DR nearer or slightly more than the target value.

SAFETY RATIO

The ratio of base force at the performance point to base shear by ESPA and RSPA is defined as safety ratio (SR). The buildings are safe when SR is equal to one, safer when SR is more than one, and unsafe when SR is less than one [18].

Table9. Safety ratio by ESPA and RSPA

Model No.	ESPA			RSPA		
	Base force at performance point	Base Shear at ESM	SR	Base force at performance point	Base Shear at ESM	SR
1	850.32	508.4	1.67	869.24	508.4	1.71
2	2069.54	976.93	2.12	2093.54	976.93	2.14
3	2058.54	967.66	2.13	2083.34	967.66	2.15
4	2047.8	937.62	2.18	2073.14	937.62	2.21
5	2039.58	907.59	2.25	2062.94	907.59	2.27

6	2030.69	877.55	2.31	2052.74	877.55	2.34
7	2023.6	847.51	2.39	2043.84	847.51	2.41

It is observed in Table 9 that, SR of model 2 to model 7 is 1.26 to 1.43 and 1.25 to 1.41 times safer compared to the model 1 by ESPA and RSPA respectively. Therefore, these results indicate that seismically designed soft storey buildings are more than 1.27 times safer than the bare frame buildings.

GLOBAL STIFFNESS

Table10. Global stiffness by ESPA and RSPA

Model No.	ESPA			RSPA		
	Base Force at Performance point	Displacement at Performance point	GS	Base Force at Performance point	Displacement at Performance point	GS
1	850.32	281.16	3.02	869.24	286.31	3.04
2	2069.54	115.64	17.9	2093.54	108.25	19.34
3	2058.54	118.84	17.32	2083.34	112.05	18.41
4	2047.8	122.04	16.78	2073.14	115.85	17.7
5	2039.58	125.24	16.29	2062.94	119.65	17.24
6	2030.69	128.44	15.81	2052.74	123.45	16.33
7	2023.6	131.64	15.37	2043.84	127.25	16.06

The ratio of base force and displacement at the performance point is known as global stiffness (GS) of the structure. The GS of buildings is computed to study the strength in the building models to sustain earthquakes [18]. GS for building models are tabulated in Table 10. It is seen in Table 10 that, as the openings increases global stiffness reduces marginally by ESPA and RSPA. The global stiffness of model 2 increases 5.93 and 6.36 times compared to the model 1 by ESPA and RSPA respectively.

These results reveal that, RC multistoreyed buildings designed considering earthquake load combinations prescribed in earthquake codes are stiffer to sustain earthquakes.

CONCLUSION

Based on the material properties, building models considered, procedures followed, and results discussed, the following conclusions are drawn.

1. Stiffness of masonry infill walls between frames in RC multi-storeyed buildings should be considering during analysis.
2. RC framed multi-storeyed buildings must be designed considering methods mentioned in earthquake codes to reduce vulnerability to earthquake shaking.
3. As the percentage of openings increases, the fundamental natural periods are longer and earthquake force carrying capacity reduces marginally.
4. The empirical formula mentioned in IS 1893 (Part 1) : 2002 to calculate natural period may be revised.

5. The models considered in this paper are safer, ductile, stiffer, and more than 90% of hinges are developed within life safety level to non linear pushover analyses.
6. The soft storey models considered in this paper are stiffer and safer more compared to bare frame models.
7. Higher percentage of openings in masonry infill walls may be investigated to understand the behaviour of RC framed multi-storeyed buildings by linear and non linear analysis.

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