Non-Linear Static Analysis of G+3 Storeyed RC Buildings With openings in Infill Walls

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Abstract— **The brick masonry infilled frame structures are frequently used in multistoreyed buildings in India. Window and door openings are inevitable part of the infill walls. The presence of openings significantly reduces the stiffness and lateral strength of reinforced concrete frame infill buildings. The present paper deals with the behavior of performance based seismic vulnerability of two-dimensional reinforced concrete (RC) multistorey building models, with the varying percentage of central openings (15%, 25%, and 35%) in brick masonry infill walls. The brick masonry infill walls are modeled as pin-jointed single equivalent diagonal struts. Pushover analysis is carried out for both default-hinge and user-defined hinge properties as per the FEMA 440 guidelines. This paper investigates the possible differences in the results of pushover analysis due to default and user-defined nonlinear component properties. The results of ductility ratio, safety ratio, global stiffness, and hinge status at performance point are compared amongst the models. Authors conclude that increase in openings in infill walls increases the vulnerability of building models and also the user-defined hinge models are more successful in capturing the hinging mechanism compared to the models with the default hinge.**

Keywords— Openings; Default and User defined hinges; Pushover analysis; Performance levels; Ductility ratio; Safety ratio; Global stiffness.

I. INTRODUCTION

Earthquake causes the random ground motions in all directions, radiating from epicenter. These ground motions causes structure to vibrate and induces inertia forces in them [1]. In India majority of the existing reinforced concrete structures in this seismic region do not meet the current seismic code requirements as these are primarily designed for gravity loads only. However they can resist certain amount of lateral forces due to earthquake of small magnitude, due to the effects of stiffness of the masonry infill walls [2].

In India buildings with masonry infill RC frames are the most common type of structures used for the multistoreyed building construction. The presence of infills in RC frame structures significantly increases the strength and stiffness [3]. In general infill walls are considered as non-structural elements during deign. The special feature in many buildings constructed in urban India is that they have open ground storey to facilitate the vehicle parking, i.e. there are no partition walls for the columns in the ground storey, and such buildings are called as soft storey buildings. Thus the upper storeys of the building with infill walls have more stiffness than the open ground storey, most of the lateral displacement of the building occurs in the open ground storey. Collapse of many buildings with the open ground storey during the 2001 Bhuj earthquake emphasizes that such buildings are extremely vulnerable under the earthquake shaking [2]. Window and door openings are inevitable part of the infill walls. However, the presence of openings in infill walls decreases the stiffness and lateral strength of the RC frame building [3]. Further if the openings are provided in the infill walls of the soft storey building, it proves to be critical condition [2]. 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.

II. DESCRIPTION OF THE BUILDING MODELS

In the present study two-dimensional four storeyed RC frame buildings are considered. The plan and elevation of the building models are shown in Figure 1, Figure 2, and Figure 3. The bottom storey height is 4.8 m and upper storey height is 3.6 m [2]. 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 [6]. 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 load combinations of equivalent static and response spectrum analysis are considered as 1.2 (DL+LL+EQX) and 1.2 (DL+LL+RSX) respectively [4]. The density of concrete and brick masonry is 25 [7] and 20 kN/m^3 [7]. Young's modulus of concrete and brick masonry is 25000 MPa [4] and 3285.9 MPa [8]. Poison's ratio of concrete is 0.3 [9]. 15%, 25% and 35% [2] of central openings are considered and four analytical models are developed as mentioned below,

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

Building has no walls in the first storey and unreinforced masonry infill walls in the upper storeys, with varying central opening, however stiffness and masses of the walls are considered.

Model 2 - 15% of the total area of infill.
Model 3 -25% of the total area of infill.
Model 4 - 35% of the total area of infill.

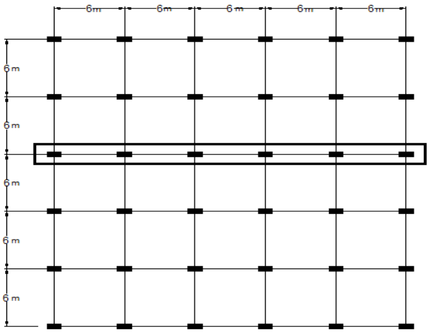

Fig. 1. Plan of the building

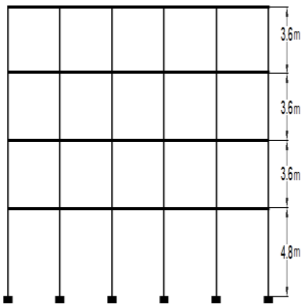

Fig. 2. Elevation of bare frame building

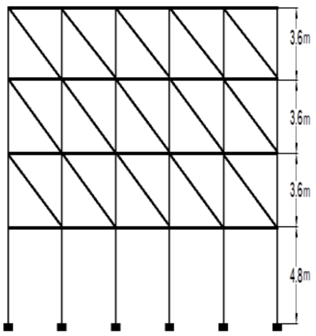

Fig. 3. Elevation of soft storey models

III. METHODOLOGY OF THE STUDY

A. 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

1) Moment Curvature for Beam Section

Following procedure is adopted for the determination of moment-curvature relationship considering unconfined concrete model as given in stress-strain block as per IS 456 : 2000 [6].

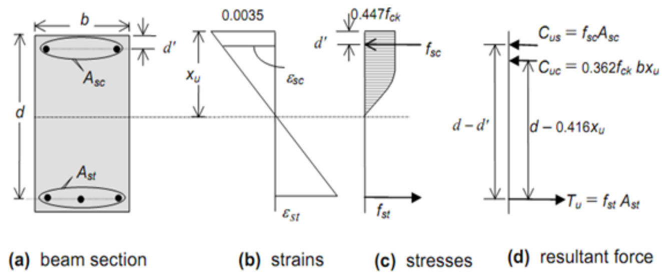

Fig. 4. Stress-Strain block for beam [9]

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

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

3. Divide the x_{umax} 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×Z).
7. Now calculate the curvature from equation 2.

$$\phi = \frac{\varepsilon_s}{d - x_u} \quad \ldots\ldots\ldots\ldots\ldots (2)$$

8. Plot moment curvature curve.

Assumption made in obtaining moment curvature curve for beam and column

[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 [6].
[4] The point 'D' is usually limited to 20% of the yield strength, and ultimate curvature,θ_u with that [10].
[5] The point 'E' defines the maximum deformation capacity and is taken as $15\theta_y$ whichever is greater [10].
[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) [6]

TABLE I. MOMENT CURVATURE VALUES FOR BEAM

Points	Moment/SF	Curvature/SF)
A (Origin)	0	0
B (Yeilding)	1	0.0145
C (Ultimate)	1.4387	0.1742
D (Strain hardening)	0.2	0.1742
E (Strain hardening)	0.2	0.2169
Note: Scale factors (SF) for curvature is taken as unity while a scale factor (SF) for moment capacities is taken as yield moment (SAP 2000 Manual).		

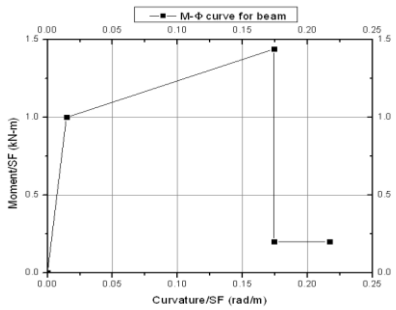

Fig. 5. Moment curvature curve for beam

2) *Moment Curvature for Column Section*

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

1. Calculate the maximum neutral axis depth x_{umax} from equation 3.

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

2. NA depth is calculated by assuming the neutral axis lies within the section.
3. The value of x_u is varied (trial and error) until the value of load tends to zero. At P = 0 kN the value of x_u obtained is the initial depth of NA.
4. Similarly N.A 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 for the obtained value of load and Moment.
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 the equation 4,

$$\phi = \frac{\varepsilon_c}{x_u} \quad \ldots\ldots\ldots (4)$$

8. Plot the moment curvature curve.

TABLE II. AXIAL LOAD AND MOMENT VALUES FOR PM INTERACTION CURVE

Xu	Pu	Mu in kN-m	Strain in concrete	Curvature rad/m
138.5	0	237.83	0.002248	0.01588
168.5	153.54	249.6	0.002735	0.01602
198.5	318.11	258.46	0.003222	0.01612
215.6	390.92	261.13	0.0035	0.01623
500	893.19	227.14	0.004616	0.00923
800	2695.09	32.779	0.00273	0.00341
1100	2775	15.18	0.00248	0.00225
1400	2812.54	6.47	0.00236	0.00169
1700	2834.05	1.36	0.00229	0.00135
1850	2841.71	0	0.00226	0.00122

TABLE III. MOMENT CURVATURE VALUES FOR COLUMN

Points	Moment/SF	Curvature/SF
A (origin)	0	0
B (yeilding)	1	0.00923
C (ultimate)	1.0462	0.01623
D (Strain hardening)	0.2	0.01623
E (Strain hardening)	0.2	0.13845

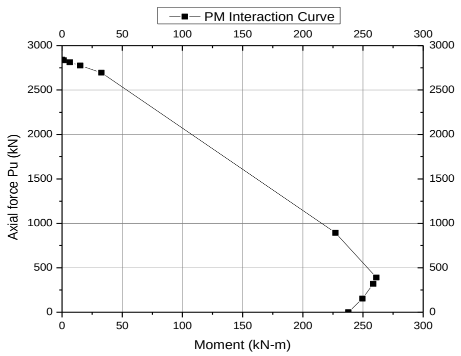

Fig. 6. P M Interaction curve

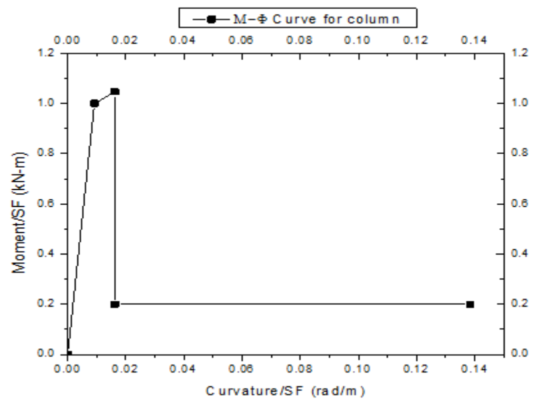

Fig. 7. Moment curvature relations for column

B. *Pushover analysis*

Pushover analysis is a static non-linear procedure in which the magnitude of the lateral load is incrementally increased maintaining a predefined distribution pattern along the height of the building. With the increase in the magnitude of loads, weak links and failure modes of the building can be found. Pushover analysis can determine the behavior of a building, including the ultimate load and the maximum inelastic deflection. At each step, the base shear and the roof displacement can be plotted to generate the pushover curve for that structure. Pushover analysis as per FEMA 440 [11] guide lines is adopted. The models are pushed in a monotonically increasing order in a particular direction till the collapse of the structure. The models are pushed in a monotonically increasing order in a particular direction till the collapse of the structure. 4% of height of building [10] as maximum displacement is taken at roof level and the same is defined in to several steps The global response of structure at each displacement level is obtained in terms of the base shear, which is presented by pushover curve. Pushover curve is a base shear versus roof displacement curve. The peak of this curve represents the maximum base shear, i.e. maximum load carrying capacity of the structure; the initial stiffness of the structure is obtained from the tangent at pushover curve at the load level of 10% [12] that of the ultimate load and the maximum roof displacement of the structure is taken that deflection beyond which the collapse of structure takes place.

IV. RESULTS AND DESCUSSIONS

A. *Performance Evaluation of Building Models*

Performance based seismic evaluation of all the models is carried out by non linear static pushover analysis (i.e. *Equivalent static pushover analysis and Response spectrum pushover analysis*). Default and user defined hinges are assigned for the seismic designed building models along the longitudinal direction.

1) *Performance point and location of hinges*

The base force, displacement and the location of the hinges at the performance point for both default and user defined hinges, for various performance levels along longitudinal direction for all building models are presented in the below Table IV to Table VII.

TABLE IV. PERFORMANCE POINT AND LOCATION OF HINGES BY EQUIVALENT STATIC PUSHOVER ANALYSIS WITH DEFAULT HINGES

Model No.	Performance Point			Location of Hinges					
	Displacement (mm)		*Base Force (kN)*	*A-B*	*B-IO*	*IO – LS*	*LS-CP*	*CP - E*	*Total*
1	Yield	54.03	498.26	124	4	0	0	0	128
	Ultimate	252.36	726.24	101	18	0	0	9	128
2	Yield	32.41	850.36	152	6	0	0	0	158
	Ultimate	120.36	985.64	137	11	0	6	4	158
3	Yield	33.22	846.35	152	6	0	0	0	158
	Ultimate	124.26	978.65	135	8	0	10	5	158
4	Yield	34.32	842.69	151	7	0	0	0	158
	Ultimate	129.56	974.68	134	6	0	11	7	158

TABLE V. PERFORMANCE POINT AND LOCATION OF HINGES BY RESPONSE SPECTRUM PUSHOVER ANALYSIS WITH DEFAULT HINGES

Model No.	Performance Point			Location of Hinges					
	Displacement (mm)		*Base Force (kN)*	*A-B*	*B-IO*	*IO - LS*	*LS-CP*	*CP - E*	*Total*
1	Yield	50.46	525.64	124	4	0	0	0	128
	Ultimate	240.39	738.96	101	17	0	0	10	128
2	Yield	30.81	932.16	152	6	0	0	0	158
	Ultimate	121.46	1002.61	136	14	0	2	6	158
3	Yield	31.73	925.55	152	6	0	0	0	158
	Ultimate	125.58	998.49	134	8	0	9	7	158
4	Yield	32.28	919.69	151	7	0	0	0	158
	Ultimate	128.14	994.23	134	6	0	10	8	158

TABLE VI. PERFORMANCE POINT AND LOCATION OF HINGES BY EQUIVALENT STATIC PUSHOVER ANALYSIS WITH USER DEFINED HINGES

Model No.	Performance point			Location of Hinges					
	Displacement (mm)		***Base Force (kN)***	***A-B***	***B-IO***	***IO – LS***	***LS-CP***	***CP - E***	***Total***
1	Yield	72.1	450.24	102	14	4	0	8	128
	Ultimate	280.45	690.54	87	6	16	1	18	128
2	Yield	34.65	806.49	125	12	14	3	4	158
	Ultimate	99.34	946.74	120	14	12	4	8	158
3	Yield	35.45	802.24	126	11	17	1	3	158
	Ultimate	107.54	944.48	122	9	12	4	11	158
4	Yield	36.25	797.712	125	13	10	4	6	158
	Ultimate	115.74	942.177	120	10	12	0	16	158

TABLE VII. PERFORMANCE POINT AND LOCATION OF HINGES BY RESPONSE SPECTRUM PUSHOVER ANALYSIS WITH USER DEFINED HINGES

Model No.	Performance point			Location of Hinges					
	Displacement (mm)		***Base Force (kN)***	***A-B***	***B-IO***	***IO - LS***	***LS-CP***	***CP - E***	***Total***
1	Yield	71.25	490.58	102	12	8	0	6	128
	Ultimate	260.12	698.17	88	8	12	0	20	128
2	Yield	32.85	919.5	125	12	13	5	3	158
	Ultimate	97.55	995.88	124	11	14	2	7	158
3	Yield	33.65	914.18	126	12	14	0	6	158
	Ultimate	105.75	991.2	123	10	14	0	11	158
4	Yield	34.45	908.86	125	16	12	0	5	158
	Ultimate	113.95	986.52	119	8	12	4	15	158

The base force at performance point and ultimate point of the building depends on its lateral strength. It is seen in Table IV, Table V, Table VI, and Table VII that, as the openings increase the base force at ultimate point reduces by 1.011 and 1.008 times by equivalent static and response spectrum pushover analysis method in model 4 compared to model 2 with default hinges. Similarly base force reduces in model 4 compared to model 2 by 1.005 and 1.009 times by equivalent static and response spectrum pushover analysis method with user defined hinges. 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 IV and Table V it is observed that, in default hinges the hinges are formed within the life safety range at the ultimate state is 92.97%, 97.47%, 96.84%, and 95.57% in model 1 to 4 respectively by equivalent static pushover analysis (ESPA). Similarly 85.94%, 94.94%, 93.04%, and 89.87% hinges are developed in the models 1 to 4 respectively by response spectrum pushover analysis (RSPA). Similarly from the Table VI and Table VII it is observed that, in user defined hinges the hinges are formed within the life safety range at the ultimate state is 85.94%, 94.94%, 93.04%, and 89.87% in model 1 to 4 respectively by equivalent static pushover analysis (ESPA). Similarly 84.38%, 94.53%, 93.04%, and 90.50% hinges are developed in the models 1 to 4 respectively by response spectrum pushover analysis (RSPA). These results reveal that, seismically designed multistoreyed RC buildings are safe to earthquakes.

It is further observed that in default hinges, the hinges formed beyond the CP range at the ultimate state is 7.03%, 2.53%, 3.16%, and 4.43% in the models 1 to 4 respectively by ESPA. Similarly 7.81%, 3.80%, 4.43%, and 5.06% hinges are developed in the models 1 to 4 respectively by RSPA. Similarly in user defined hinges, the hinges formed beyond the CP range at the ultimate state is 14.06%, 5.06%, 6.96%, and 10.13% in the models 1 to 4 respectively by ESPA. Similarly 15.62%, 5.47%, 6.96%, and 9.50% hinges are developed in the models 1 to 4 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.

From the above results it can be conclude that, a significant variation is observed in base force and hinge formation mechanism by ESPA and RSPA with default and user defined hinges at the ultimate state. The user-defined hinge models are more successful in capturing the hinging mechanism compared to the models with the default hinge. However, if the default hinge model is preferred due to simplicity, the user should be aware of what is provided in

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the program and should avoid the misuse of default hinge properties.

B. *Ductility ratio*

Ductility ratio means it is the ratio of collapse yield (CY) to the initial yield (IY) [13]. Ductility ratio (DR) for building models are tabulated in the below Table VIII.

TABLE VIII. DUCTILITY RATIO FOR DEFAULT AND USER DEFINED HINGES

Model No.	Equivalent Static Pushover Analysis			Response Spectrum Pushover Analysis		
	IY	***CY***	***DR***	***IY***	***CY***	***DR***
Default hinges						
1	54.03	252.36	4.67	50.46	240.39	4.76
2	32.41	120.36	3.71	30.81	121.46	3.94
3	33.22	124.26	3.74	31.73	125.58	3.96
4	34.32	129.56	3.78	32.28	128.14	3.97
User defined hinges						
1	72.10	280.45	3.89	71.25	260.12	3.65
2	34.65	99.34	2.87	32.85	97.55	2.97
3	35.45	107.54	3.03	33.65	105.75	3.14
4	36.25	115.74	3.19	34.45	113.95	3.31

Note: IY: Initial Yield, CY: Collapse Yield, and DR: Ductility Ratio,

It is seen in Table VIII that, the ductility ratio of the bare frame is larger than the soft storey models, specifying stiffness of infill walls not considered. In default hinges, DR of all models i.e. model 1, model 2, model 3, and model 4 are more than the target value equal to 3 by ESPA. Similar results are observed in all models i.e. model 1, model 2, model 3, and model 4 by RSPA. Similarly in user defined hinges, DR of model 1, model 3, and model 4 are more than the targeted value which is equal to 3 by ESPA. Similar results are observed in model 1, model 3, and model 4 by RSPA. These results reveal that, increase in openings increases the DR slightly more than the target value for both default and user defined hinges.

C. *Safety Ratio*

The ratio of base force at performance point to the base shear by equivalent static method is known as safety ratio. If the safety ratio is equal to one then the structure is called safe, if it is less than one than the structure is unsafe and if ratio is more than one then the structure is safer [14].

TABLE IX. SAFETY RATIO FOR DEFAULT AND USER DEFINED HINGES

Model No.	Equivalent Static Pushover Analysis			Response Spectrum Pushover Analysis		
	BF at PP	***BS by ESM***	***SR***	***BF at PP***	***BS by ESM***	***SR***
Default hinges						
1	726.24	357.73	2.03	738.96	357.73	2.07
2	985.64	387.48	2.54	1002.61	387.48	2.59
3	978.65	364	2.69	998.49	364	2.74
4	974.68	340.49	2.86	994.23	340.49	2.92
User defined hinges						
1	690.54	357.73	1.93	698.17	357.73	1.95
2	946.74	387.48	2.44	995.88	387.48	2.57
3	944.48	364.00	2.59	991.20	364.00	2.72
4	942.17	340.49	2.77	986.52	340.49	2.90

Note: BF at PP: Base Force at Performance Point, BS by ESM: Base shear by Equivalent Static Method, SR: Safety Ratio

It is observed in Table IX that, in default hinges SR of model 2 to model 4 is 1.25 to 1.41 times safer compared to the model 1 by ESPA and RSPA respectively. Similarly in user defined hinges SR of model 2 to model 4 is 1.26 to 1.43 and 1.32 to 1.49 times safer compared to the model 1 by ESPA and RSPA respectively. Therefore, these results indicate that seismically designed soft storey buildings are safer than the bare frame buildings for both default and user defined hinges.

D. *Global Stiffness*

The ratio of performance force shear to the performance displacement is called as global stiffness [14]. Global stiffness (GS) for ten storeyed building models are tabulated in the below Table X.

TABLE X. GLOBAL STIFFNESS BY ESPA AND RSPA

Model No.	Equivalent Static Pushover Analysis			Response Spectrum Pushover Analysis		
	BF at PP	***Disp. at PP***	***GS***	***BF at PP***	***Disp. at PP***	***GS***
Default hinges						
1	726.24	252.36	2.88	738.96	240.39	3.07
2	985.64	120.36	8.19	1002.61	121.46	8.25
3	978.65	124.26	7.88	998.49	125.58	7.95
4	974.68	129.56	7.52	994.23	128.14	7.75
User defined hinges						
1	690.54	280.45	2.46	698.17	260.12	2.68
2	946.74	99.34	9.53	995.88	97.55	10.21
3	944.48	107.54	8.78	991.20	105.75	9.37
4	942.17	115.74	8.14	986.52	113.95	8.66

Note: BF at PP: Base Force at Performance point, Disp. at PP: Displacement at Performance Point, GS: Global Stiffness

It is seen in Table X that, in default hinges as the openings increases global stiffness reduces slightly by ESPA and ESPA. The global stiffness of model 2 increases 2.84 and 2.69 times compared to the model 1 by ESPA and RSPA respectively. In user defined hinges as the openings increases global stiffness reduces marginally by ESPA and ESPA. The global stiffness of model 2 increases 3.87 and 3.81 times compared to the model 1 by ESPA and RSPA respectively.

These results reveal that, multistoreyed RC buildings designed considering earthquake load combinations prescribed in earthquake codes are stiffer to sustain earthquakes. It can also conclude that building models with user defined hinge are found stiffer compare to building models with default hinge.

CONCLUSION

Based on the results obtained from different analysis for the various building models, the following conclusion is drawn.

1. The base force at performance point decreases with increases in the percentage of central openings for both default and user defined hinge.
2. A significant variation is observed in base force and hinge formation mechanism by ESPA and RSPA with default and user defined hinges at the ultimate state.
3. The user-defined hinge models are more successful in capturing the hinging mechanism compared to the models with the default hinge.
4. The models considered in this paper are safer, ductile, stiffer, and more than 90% with default and 85% with user defined hinges are developed within life safety level by non linear pushover analyses.
5. The soft storey models considered in this paper are stiffer and safer compared to bare frame models for both default and user defined hinge.
6. Global stiffness is more in the soft storey building models compared to the bare frame building. As the percentage of openings increases, the global stiffness decreases.

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