International Journal of Scientific & Engineering Research, Volume 5, Issue 10, October-2014 284
ISSN 2229-5518

Seismic Evaluation of Ten Storeyed RC Buildings with Openings in Infill walls

Praveen Rathod, Dr.S.S.Dyavanal

Abstract— Infill walls are commonly used in the RC frame structure buildings in India. Openings in infill walls are unavoidable part. Openings in infill walls are considerably decrease the lateral strength and stiffness of RC frames. In the paper two-dimensional ten storeyed reinforced concrete (RC) building models are considered with different sizes of openings (15%, 25%, and 35%). Bare frame and soft storey buildings are modeled considering special moment resisting frame (SMRF) for medium soil profile under zone III. Brick masonry infill walls are modelled as pin-jointed single equivalent diagonal strut. Pushover analysis is carried out for both default and user defined hinge properties as per FEMA 440 guidelines using SAP2000 software. Results of default and user defined hinge properties are studied by nonlinear static analysis. The results of ductility ratio, safety ratio, global stiffness, and hinge status at performance point are compared with the models. Authors conclude that user-defined hinge model is better than the default-hinge model in reflecting nonlinear behavior compatible with the element properties the user needs to be careful. The misuse of default-hinge properties may lead to unreasonable displacement capacities for existing structures. However, if the default-hinge model is preferred due to easy way, the user should be aware of what is provided in the program and should ignore the misuse of default-hinge properties.

Index Terms— Openings, Default and User defined hinges, Pushover analysis, Performance levels, Ductility ratio, Safety ratio, Global stiffness.

1 Introduction

Reinforced concrete (RC) frame buildings with masonry infill walls have been broadly constructed for commercial, industrial and multi-family residential uses in seismic-prone regions worldwide. Masonry infill typically consists of brick masonry or concrete block walls, constructed between columns and beams of a RC frame [1]. These panels are generally not considered in the design process and treated as non-structural components [1]. In country like India, Brick masonry infill walls have been broadly used as interior and exterior partition walls for aesthetic reasons and functional needs. Though the brick masonry infill is considered to be a non structural element, but it has its own strength and stiffness. Hence if the effect of brick masonry is considered in analysis and design, considerable increase in strength and stiffness of overall structure may be observed [1]. The particular characteristic 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 unavoidable part of the infill walls. However, the presence of openings in masonry infill walls reduces 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.

2 Description of The Building

Two-dimensional ten storeyed RC frame buildings are considered for the present study. 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 floors 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 density of concrete and brick masonry is 25 and 20 kN/m^3 [7]. Young's modulus of concrete and brick masonry is 25000 MPa [6] 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 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 - 15%.

Model 3 - 25%

Model 4 - 35%

Models are designed for 1.2(*DL*+*LL*+*EQ*) and 1.2(*DL*+*LL*+*RS*) are carried out for equivalent static and response spectrum analysis respectively [4].

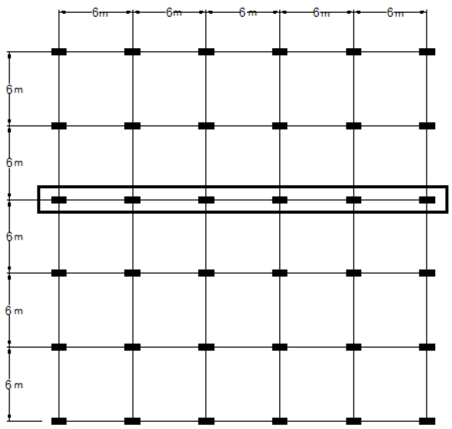

Fig. 1. Plan of the building

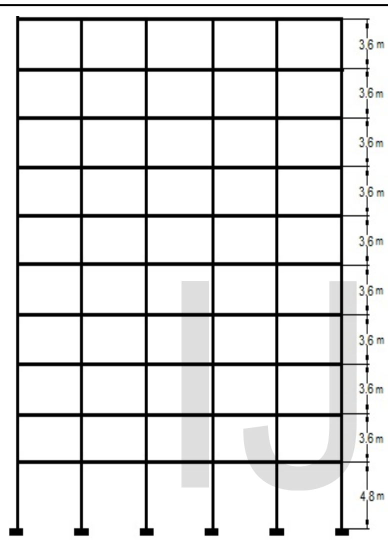

Fig. 2. Elevatin for bare frame building

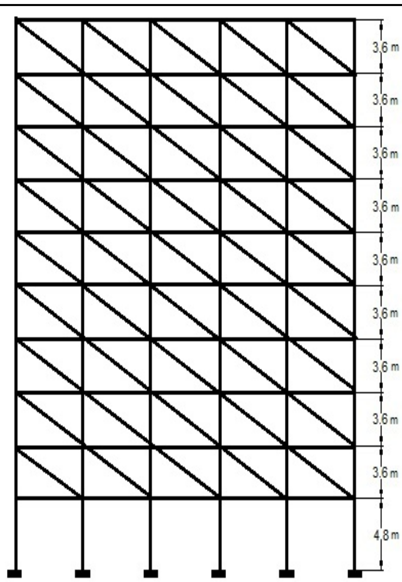

Fig. 3. Elevatin for soft storey building

3 Methodology of The Study

3.1 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 wall. 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 walls are assigned. The calculated moment-curvature values for beam (*M3 and V3*), column (*PM3*), and load deformation curve values for wall (*P*) are substituted instead of default hinge values in SAP2000.

3.1.1 Moment Curvature for Beam

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 [6].

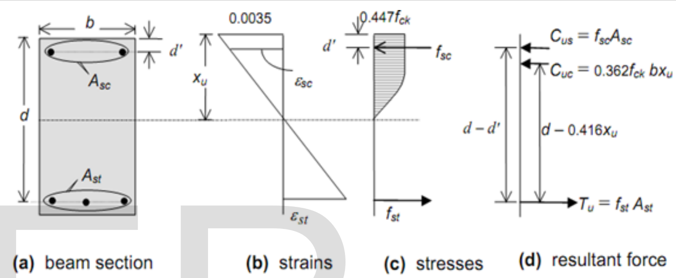

Fig. 4. Stress strain block for beam [9]

1. Calculate the neutral axis depth by equating compressive and tensile forces.
2. Calculate the maximum neutral axis depth xumax from equation 1.

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

3. Divide the xumax in to equal laminae.
4. For each value of xu 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.

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

8. Plot moment curvature curve. Figure 5 shows the moment curvature curve for beam.

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.

ISSN 2229-5518

[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,θu with that [10].

[5] The point 'E' defines the maximum deformation capacity and is taken as 15θy 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]

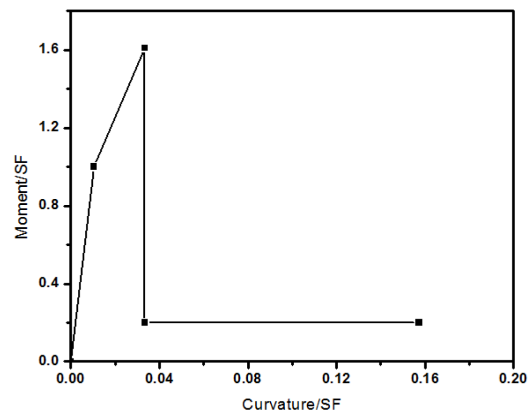

Fig. 5. Moment curvature curve for beam

3.1.2 Moment curvature for column section

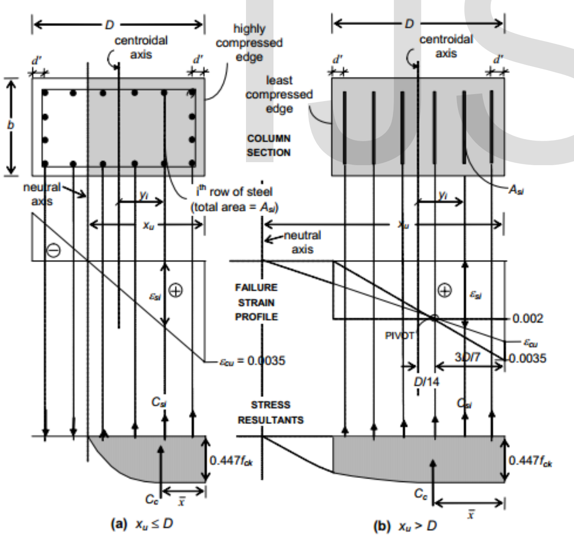

Fig. 6. Analysis of design strength of rectangular section under compression [9]

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.

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

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

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

8. Plot the moment curvature curve. Moment curvature curve shown in Fig 8.

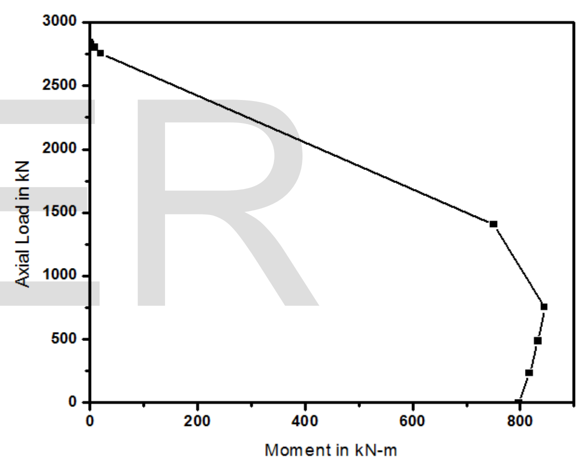

Fig. 7. P-M interaction curve

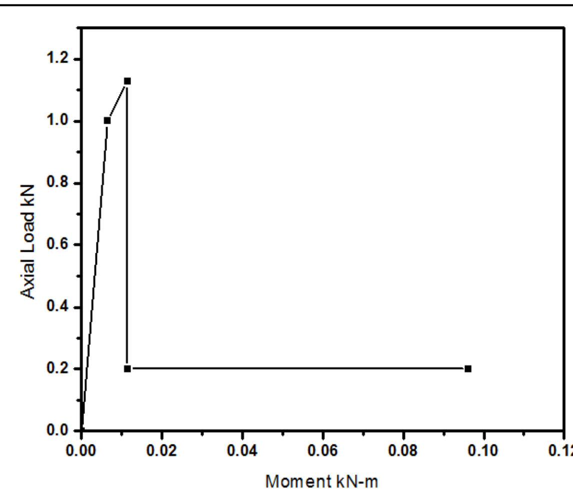

Fig. 8. Moment curvature curve for column

ISSN 2229-5518

3.2 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.

4.0 Results and Descussion

4.1 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.

4.1.1 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 Table 1 to Table 4.

The base force at performance point and ultimate point of the building depends on its lateral strength. It is seen in Table 1, Table 2, Table 3, and Table 4 that, as the openings increase the base force at ultimate point reduces by 1.012 and 1.019 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 1.017 and 1.019 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.

TABLE 1
PERFORMANCE POINT AND LOCATION OF HINGES BY ESPA WITH DEFAULT HINGES

Model No.		Performance Point		Location of Hinges					
		Displacement mm	Base Force kN	A-B	B-IO	IO - LS	LS-CP	CP to E	Total
1	Yield	70.85	538.76	270	50	0	0	0	320
	Ultimate	310.68	728.96	232	52	29	0	7	320
2	Yield	28.47	1753.21	395	15	0	0	0	410
	Ultimate	120.45	2161.47	388	10	5	6	1	410
3	Yield	29.43	1718.53	395	15	0	0	0	410
	Ultimate	125.73	2148.36	385	10	5	6	4	410
4	Yield	30.59	1703.14	395	15	0	0	0	410
	Ultimate	131.44	2134.90	388	8	4	5	5	410

TABLE 2
PERFORMANCE POINT AND LOCATION OF HINGES BY RSPA WITH DEFAULT HINGES

Model No.		Performance Point		Location of Hinges					
		Displacement mm	Base Force kN	A-B	B-IO	IO - LS	LS-CP	CP to E	Total
1	Yield	73.14	550.26	274	46	0	0	0	320
	Ultimate	321.54	742.69	231	50	31	0	8	320
2	Yield	27.48	1769.89	396	14	0	0	0	410
	Ultimate	116.83	2181.48	388	10	5	6	1	410
3	Yield	28.43	1730.68	395	15	0	0	0	410
	Ultimate	122.64	2152.62	386	8	6	7	3	410
4	Yield	29.59	1718.86	395	15	0	0	0	410
	Ultimate	128.12	2140.36	385	10	2	7	6	410

TABLE 3
PERFORMANCE POINT AND LOCATION OF HINGES BY ESPA 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 to 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	40.53	1669.7	378	20	4	0	8	410
	Ultimate	118.84	2058.5	361	15	12	8	14	410
3	Yield	41.35	1644.5	380	20	6	2	2	410
	Ultimate	125.24	2039.6	356	22	12	2	18	410
4	Yield	42.17	1614.3	380	16	10	2	2	410
	Ultimate	131.64	2023.6	354	16	8	10	22	410

ISSN 2229-5518

TABLE 4
PERFORMANCE POINT AND LOCATION OF HINGES BY RSPA 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 to 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	39.21	1689.5	378	16	2	2	12	410
	Ultimate	112.05	2083.3	361	14	12	10	13	410
3	Yield	40.16	1665.5	380	20	6	2	2	410
	Ultimate	119.65	2062.9	356	20	10	5	19	410
4	Yield	41.3	1641.5	380	15	10	5	0	410
	Ultimate	127.25	2043.8	354	14	6	12	24	410

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 1 and Table 2 it is observed that, in default hinges the hinges are formed within the life safety range at the ultimate state is 97.81%, 99.76%, 99.02%, and 98.78% in model 1 to 4 respectively by equivalent static pushover analysis (ESPA). Similarly 97.50%, 99.76%, 99.27%, and 98.54% hinges are developed in the models 1 to 4 respectively by response spectrum pushover analysis (RSPA). Similarly from the Table 3 and Table 4 it is observed that, in user defined hinges the hinges are formed within the life safety range at the ultimate state is 93.13%, 96.59%, 95.61%, and 94.63% in model 1 to 4 respectively by equivalent static pushover analysis (ESPA). Similarly 91.88%, 96.83%, 95.37%, and 94.15% 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 scrutinized that in default hinges, the hinges formed beyond the CP range at the ultimate state is 2.19%, 0.24%, 0.98%, and 1.22% in the models 1 to 4 respectively by ESPA. Similarly 2.5%, 0.24%, 0.73%, and 1.46% 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 6.87%, 3.41%, 4.39%, and 5.37% in the models 1 to 4 respectively by ESPA. Similarly 8.12%, 3.17%, 4.63%, and 5.85% 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 the program and should avoid the misuse of default hinge properties.

TABLE 5
DUCTILITY RATIO BY ESPA AND RSPA

Model No.	Equivalent Static Pushover Analysis			Response Spectrum Pushover Analysis		
	IY	CY	DR	IY	CY	DR
Default hinges						
1	70.847	310.68	4.39	73.14	321.54	4.40
2	28.47	120.45	4.23	27.48	116.83	4.25
3	29.43	125.73	4.27	28.43	122.64	4.31
4	30.59	131.44	4.30	29.59	128.12	4.33
User defined hinges						
1	76.25	281.16	3.69	78.65	286.31	3.64
2	40.53	118.84	2.93	39.21	112.05	2.86
3	41.35	125.24	3.03	40.16	119.65	2.98
4	42.17	131.64	3.12	41.3	127.25	3.08

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

4.2 Ductility ratio

The ratio of collapse yield (CY) to the initial yield (IY) is called as ductility ratio [13].Ductility ratio (DR) for building models are tabulated in the Table 5.

It is seen in Table 5 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 and model 4 by RSPA. These results reveal that, increase in openings increases the DR more than the target value for both default and user defined hinges.

4.3 Safety ratio

The ratio of base force at performance point to the base shear by equivalent static method is called 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].

It is observed in Table 6 that, in default hinges SR of model 2 to model 4 is 1.56 to 1.76 and 1.54 to 1.73 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 1.28 to 1.43 and 1.26 to 1.41 times safer compared to the model 1 by ESPA and RSPA respectively. Therefore, these results designates that seismically designed soft storey buildings are safer

TABLE 6
SAFETY RATIO BY ESPA AND RSPA

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	728.961	508.40	1.43	742.69	508.4	1.46
2	2161.47	967.66	2.23	2181.48	967.66	2.25
3	2148.36	907.59	2.37	2152.62	907.59	2.37
4	2134.90	847.51	2.52	2140.36	847.51	2.53
User defined hinges						
1	850.32	508.40	1.67	869.24	508.4	1.71
2	2058.54	967.66	2.13	2083.34	967.66	2.15
3	2039.58	907.59	2.25	2062.94	907.59	2.27
4	2023.60	847.51	2.39	2043.84	847.51	2.41

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

than the bare frame buildings for both default and user defined hinges.

4.4 Global stiffness

TABLE 7
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	728.96	310.68	2.35	742.69	321.54	2.31
2	2161.47	120.45	17.94	2181.48	116.83	18.67
3	2148.36	125.73	17.09	2152.62	122.64	17.55
4	2134.9	131.64	16.22	2140.36	128.12	16.71
User defined hinges						
1	850.32	281.16	3.02	869.24	286.31	3.04
2	2058.54	118.84	17.32	2083.34	112.05	18.59
3	2039.58	125.24	16.29	2062.94	119.65	17.24
4	2023.60	131.64	15.37	2043.84	127.25	16.06

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

The ratio of performance base force to the performance displacement is called as global stiffness [14]. Global stiffness (GS) for building models are tabulated in the Table 7.

It is seen in Table 7 that, in default hinges as the openings increases global stiffness reduces slightly by ESPA and ESPA. The global stiffness of model 2 increases 7.63 and 8.08 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 5.74 and 6.12 times compared to the model 1 by ESPA and RSPA respectively.

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

5 CONCLUSION

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

1. During the analysis stiffness of masonry infill walls between frames in RC multi-storeyed buildings should be considering.
2. As the percentage of openings increases the base force at performance point reduces for both default and user defined hinges.
3. A significant variation is observed in hinge formation mechanism by ESPA and RSPA with default and user defined hinges at the ultimate state.
4. The user-defined hinge models are more successful in capturing the hinging mechanism compared to the default hinge models.
5. The default-hinge model is preferred due to simplicity, the user should be aware of what is provided in the program and should avoid the misuse of default-hinge properties.
6. In this present study the models considered are safer, ductile, and stiffer.

REFERENCES

[1] V.P. Jamnekar and P.V. Durge, "Seismic Evaluation of Brick Masonry Infill", *International Journal of Emerging Trends in Engineering and Technology (IJETET)*, Vol. 02, No. 01, 2013

[2] B.G. Prashanta and S.S. Dyavanal, "Performance Based Seismic Evaluation of Multistoreyed Buildings with the Openings in U M Infill Walls", *RDSE-2007, Manipal Institute of Technology, Manipal*. 30-31st August.

[3] G. Mondal and S.K. Jain, "Lateral Stiffness of Masonry Infilled Reinforced Concrete (RC) Frames with Central Opening", *Earthquake Spectra*, Vol 24, No 3, pages 701-723, Indian Institute of Technology, India, 2008.

[4] IS 1893 (Part 1): 2002, "*Criteria for Earthquake Resistant Design of Structures*", Bureau of Indian Standards, New Delhi 110002.

[5] S.K. Jain, and C.V.R. Murty, "*Proposed draft provision and commentary on Indian Seismic Code IS 1893 (Part 1)*", Department of Civil Engineering, Indian Institute of Technology Kanpur.

[6] IS 456 : 2000, "*Code of Practice for Plain and Reinforced Concrete*", Bureau of Indian Standards, New Delhi, India

[7] Dr.H.J. Shah, "Reinforced Concrete", 9th ed., vol. 1. Charotar Publication 2012.

[8] Rihan Maaze, "*Seismic Evaluation of Multistorey Buildings with Soft*

Storey Assigned with User Defined Hinges", M.Tech Thesis, B.V. Bhoomaraddi College of Engineering & Technology, Hubli, 2013.

[9] Pillai and Menon, "*Reinforced Concrete Design*", Tata McGraw-Hill Education 2003

[10] Applied Technology Council (1996), "*Seismic Evaluation and Retrofit of Concrete Buildings*", Vol.1& 2, California.

[11] Federal Emergency Management Agency, FEMA-440 (2005), "*Improvement of Nonlinear Seismic Analysis Procedures*". California.

[12] FEMA 356, 2000 "*Pre-standard and commentary for the seismic rehabilitation of buildings*", ASCE for the Federal Emergency Management Agency, Washington, D.C.

[13] R. Park and T. Paulay, "*Reinforced Concrete Structures*," Christ church, New Zealand, Aug, pp. 270-343, 1974.

[14] V.B. Karkatti, "*Seismic Evaluation and Retrofitting of Soft Ground RC Multistorey Buildings*", M.Tech Thesis, B.V. Bhoomaraddi College of Engineering & Technology, Hubli, 2006.

[15] Praveen Rathod, S.S Dyavanal, "Performance Based Seismic Evaluation of G+9 RC Buildings with Openings in Infill Walls", *IJARSE*, Vol. No.3, Issue No.7, July 2014.

[16] Praveen Rathod, S.S Dyavanal, "Non-Linear Static Analysis of G+3 Storeyed RC buildings with Openings in Infill walls", *IJERT*, Vol. No.3, Issue No.8, August 2014.

[17] Praveen Rathod, S.S Dyavanal, "Pushover Analysis of Seven Storeyed RC buildings with Openings in Infill walls", *IJETT*, Volume 14, Number 3, August 2014.

[18] Praveen Rathod, S.S Dyavanal, "Performance Based Seismic Evaluation of G+3 Storeyed RC Buildings with Openings in Infill Walls", *IJIRAE*, Volume 1, Issue 7, August 2014.

[19] Praveen Rathod, S.S Dyavanal, "Non-Linear Static Analysis of G+6 Storeyed RC Buildings with Openings in Infill Walls", *IJERA*, Vol. 4, Issue 9(Version 5), September 2014.

[20] Praveen Rathod, S.S Dyavanal, "Pushovr Analysis of Ten Storeyed RC Buildings with Openings in Infill Walls", *IJAST*, Volume 4, Issue 4, July-August 2014.

[21] Praveen Rathod, S.S.Dyavanal,"Seismic Evaluation of Multistorey RC Building with Openings in Unreinforced Masonry Infill Walls with User Defined Hinges" *Proceedings of 08th IRF International Conference,* 05th July-2014, Bengaluru, India, ISBN: 978-93-84209-33-9.