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Performance based seismic evaluation of G+3 RC buildings with openings in infill walls

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Abstract— The RC frame structures with infill walls are frequently used in multistoreyed buildings in recent past. Window and door openings are inevitable part of the infill walls. The presence of openings in infill walls considerably reduces the lateral strength and stiffness of RC frames. In the present study two-dimensional four 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. Concrete block 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 pushover analysis. The results of ductility ratio, safety ratio, global stiffness, and hinge status at performance point are compared with the models. Authors conclude that as the percentage of openings increases, vulnerability increases in the infill walls. Earthquake code procedure should be considered during the design of the structure. User defined hinge models are more successful in capturing the hinging mechanism compared to the default hinge models.

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

I. INTRODUCTION

In India large numbers of buildings are constructed with brick/or concrete block infill walls. These infill walls significantly increase the stiffness and strength of the infilled frame [1]. In the current practice, masonry infill panels are treated as non-structural element during the design of the structure and their strength and stiffness contributions are ignored [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 reduces the stiffness and strength of the RC frame [1]. 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 Fig. 1, and Fig. 2. The bottom storey height is 4.8 m, upper storey height is 3.6 m, and bay width in longitudinal direction is considered as 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 concrete block 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 and Young’s modulus of concrete block is 22 kN/m^3 and 2272 MPa [7]. Poison’s ratio of concrete is 0.3 [8]. 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.

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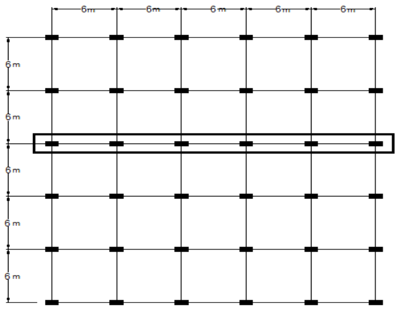

Fig. 1 Plan of building

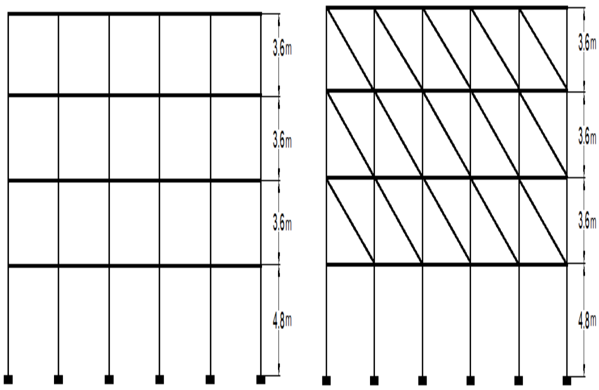

Fig. 2 Elevation of bare frame and soft storey building models

III. METHODOLOGY

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.

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1) Moment Curvature for Beam Section: 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].

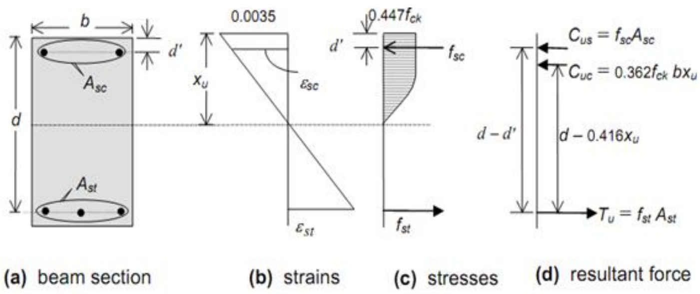

Fig. 3 Stress-Strain block for beam [8]

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

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

3. Divide the x_{umax} in to equal laminae.

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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.

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

8. Plot moment curvature curve. Calculated values of moment-curvatures are presented in Table I.

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 [9].
[5] The point 'E' defines the maximum deformation capacity and is taken as $15\theta_y$ whichever is greater [9].
[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).		

2) Moment Curvature for Column Section: Following procedure is adopted for the determination of moment-curvature relationship *for* column.

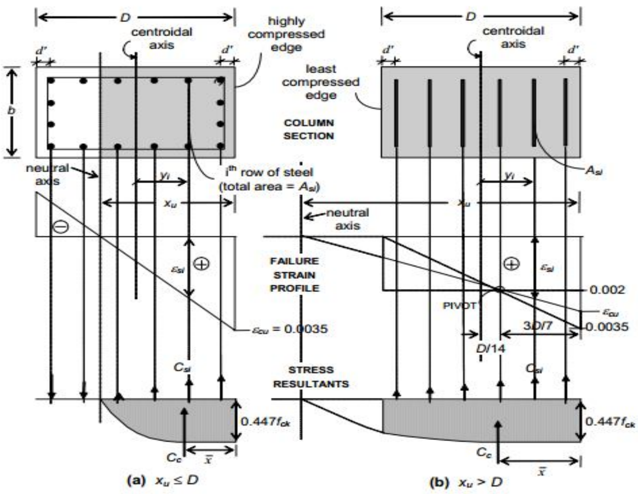

Fig. 4 Analysis of design strength of a rectangular section under compression [8]

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

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

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. Plot the P-M interaction curve for the obtained value of load and moment. These values are presented in Table II.
6. For the different values of x_u, the strain in concrete is calculated by using the similar triangle rule.

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7. The curvature values are calculated using equation 4,

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

8. Plot the moment curvature curve. Moment curvature values are presented in the Table III.

TABLE II
AXIAL LOAD AND MOMENT VALUES FOR PM INTERACTION CURVE

X_u	P_u	Mu in kN-m	Strain in concrete	Curvature rad/m
138.5	0	237.83	0.0022	0.01588
168.5	153.54	249.6	0.0027	0.01602
198.5	318.11	258.46	0.0032	0.01612
215.6	390.92	261.13	0.0035	0.01623
500	893.19	227.14	0.0046	0.00923
800	2695.09	32.779	0.0027	0.00341
1100	2775.00	15.18	0.0024	0.00225
1400	2812.54	6.47	0.0023	0.00169
1700	2834.05	1.36	0.0022	0.00135
1850	2841.71	0	0.0022	0.00122

TABLE III
MOMENT CURVATURE VALUES FOR COLUMN

Points	Moment/SF	Curvature/SF
A (origin)	0.0	0.0
B (yielding)	1.0	0.00923
C (ultimate)	1.0462	0.01623
D (Strain hardening)	0.2	0.01623
E (Strain hardening)	0.2	0.13845

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 [10] 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 [11] 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% [9] 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 DISCUSSIONS

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.

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

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	74.62	530.65	119	5	0	0	0	124
	Ultimate	302.65	736.98	110	4	1	0	9	124
2	Yield	32.469	989.601	151	7	0	0	0	158
	Ultimate	121.26	1171.8	139	6	2	8	3	158
3	Yield	33.699	977.949	150	8	0	0	0	158
	Ultimate	128.64	1166.445	136	10	2	6	4	158
4	Yield	34.02	973.497	148	10	0	0	0	158
	Ultimate	132.86	1156.68	138	8	4	2	6	158

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

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	70.26	542.073	120	4	0	0	0	124
	Ultimate	290.23	741.60	111	1	3	1	8	124
2	Yield	29.36	998.41	148	9	1	0	0	158
	Ultimate	112.36	1196.17	139	5	2	8	4	158
3	Yield	30.58	989.45	150	8	0	0	0	158
	Ultimate	118.63	1191.55	136	10	1	6	5	158
4	Yield	31.56	986.18	152	6	0	0	0	158
	Ultimate	124.03	1188.89	138	8	4	2	6	158

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

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	76.23	480.26	112	6	2	0	8	128
	Ultimate	299.36	698.12	101	6	2	0	19	128
2	Yield	38.85	936.28	126	16	8	4	4	158
	Ultimate	109.72	1116.00	118	9	18	4	9	158
3	Yield	39.65	931.38	126	14	8	4	6	158
	Ultimate	117.92	1110.90	118	10	15	2	13	158
4	Yield	40.45	927.140	126	12	8	4	8	158
	Ultimate	126.12	1101.60	116	10	15	0	17	158

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

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	73.46	516.26	112	8	4	0	4	128
	Ultimate	272.65	705.62	98	8	1	0	21	128
2	Yield	34.08	949.06	125	14	10	4	5	158
	Ultimate	102.32	1140.3	118	10	16	4	10	158
3	Yield	34.88	944.14	124	18	8	2	6	158
	Ultimate	110.52	1135.9	119	8	15	4	12	158
4	Yield	35.68	939.22	124	14	10	2	8	158
	Ultimate	118.72	1131.2	117	10	12	2	17	158

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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 1.013 and 1.006 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 1.013 and 1.008 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 92.74%, 98.10%, 97.47%, and 96.20% in model 1 to 4 respectively by equivalent static pushover analysis (ESPA). Similarly 93.55%, 97.47%, 96.84%, and 96.20% 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 85.15%, 94.30%, 91.77%, and 89.24% in model 1 to 4 respectively by equivalent static pushover analysis (ESPA). Similarly 83.59%, 93.67%, 92.41%, and 89.24% 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 secure to earthquakes.

It is further observed that in default hinges, the hinges formed beyond the CP range at the ultimate state is 7.26%, 1.90%, 2.53%, and 3.80% in the models 1 to 4 respectively by ESPA. Similarly 6.45%, 2.53%, 3.16%, and 3.80% 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 14.85%, 5.70%, 8.23%, and 10.76% in the models 1 to 4 respectively by ESPA. Similarly 16.41%, 6.33%, 7.59%, and 10.76% hinges are developed in the models 1 to 4 respectively by RSPA. As the collapse hinges are few, retrofitting can be completed immediately 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.

B. Ductility Ratio

The ratio of collapse yield (CY) to the initial yield (IY) is called as ductility ratio [12].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	74.62	302.65	4.06	70.26	290.23	4.13
2	32.469	121.26	3.73	29.36	112.36	3.83
3	33.699	128.64	3.82	30.58	118.63	3.88
4	34.02	132.82	3.90	31.56	124.03	3.93
User defined hinges						
1	76.23	299.36	3.93	73.46	272.65	3.71
2	38.85	109.72	2.82	34.08	102.32	3.00
3	39.65	117.92	2.97	34.88	110.52	3.17
4	40.45	126.12	3.12	35.68	118.72	3.33
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 and model 4 are more than the targeted value which is equal to 3 by ESPA. Similar results are observed in model 1, model 2, 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 [13].

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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	736.98	376.16	1.96	741.60	376.16	1.97
2	1171.8	410.68	2.85	1196.17	410.68	2.91
3	1166.445	384.45	3.03	1191.559	384.45	3.10
4	1156.68	358.14	3.23	1188.89	358.14	3.32
User defined hinges						
1	698.12	376.16	1.86	705.60	376.16	1.88
2	1116.02	410.68	2.72	1140.31	410.68	2.78
3	1110.89	384.45	2.89	1135.87	384.45	2.95
4	1101.6	358.14	3.08	1131.15	358.14	3.16
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 1.45 to 1.65 and 1.47 to 1.69 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 1.46 to 1.66 and 1.48 to 1.68 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 [13]. Global stiffness (GS) for ten storeyed building models are tabulated in the below Table X.

TABLE X
GLOBAL STIFFNESS FOR DEFAULT AND USER DEFINED HINGES

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	736.98	302.65	2.44	741.60	290.23	2.56
2	1171.8	121.26	9.66	1196.17	112.36	10.65
3	1166.445	128.64	9.07	1191.559	118.63	10.04
4	1156.68	132.82	8.71	1188.89	124.03	9.59
User defined hinges						
1	698.12	299.36	2.33	705.62	272.65	2.59
2	1116.02	109.72	10.17	1140.31	102.32	11.14
3	1110.89	117.92	9.42	1135.87	110.52	10.28
4	1101.6	126.12	8.73	1131.15	118.72	9.53
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 by ESPA and ESPA. The global stiffness of model 2 increases 3.96 and 4.16 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 4.36 and 4.30 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.

V. CONCLUSIONS

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

1. RCC framed multi-storeyed buildings must be designed considering methods mentioned in earthquake codes to reduce vulnerability to earthquake shaking.
2. The base force at the ultimate state decreases with increases in the percentage of central openings for both default and user defined hinges. Also in base force 4 to 5% variation is observed in-between default and user defined hinges.

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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 models with the default hinge.
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. 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 static analyses.

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