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Seismic Analysis on Ten Storey building Located in Seismic Zone III of India

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Abstract ***Reinforced concrete buildings represent major type of concrete structures all over the world. It is observed from the past major earthquakes in India, Turkey, China, Nepal and other countries, numerous medium to high rise buildings suffered major damages including life and safety of people. This clearly implies that the design capacity of buildings designed as per codes was not enough to withstand earthquake demand. In the regions of high seismic activity, safety of these buildings occupying major population is of growing concern, as most of them are seismically deficient, as many low to medium rise buildings are designed only for gravity loading, or they are designed for earthquake forces by performing elastic analysis. When elastic analysis is performed, building structure is assumed as a linear system. This analysis is sufficed to withstand mild earthquakes but a major earthquake is expected to cause significant structural damage resulting in loss of structural stiffness. A linear elastic analysis is not applicable in this situation since these analyses do not account for change in structural properties nor they give any idea about the location and extent of damage or inelasticity. The true response is therefore determined only by non-linear analysis considering the changing stiffness of the various members and moment distribution, when structures are subjected to moderate or major earthquakes. Thus to mitigate the effects of future earthquake damages, the seismic design should be carried out incorporating non-linear analyses to calculate structural response in order to assess and design seismic retrofit solutions for existing buildings and design new buildings based on above approach. In the present study three different building models are designed for critical load combinations by considering lateral Earthquake load by Equivalent Static Analysis (ESA) and Response spectrum analysis (RSA) and then pushover analysis is carried out to assess their performance. The performance of building models, displacements corresponding to performance point (PP), and hinging pattern at PP are presented and studied. It is concluded that ESA method of analysis has satisfactory performance and RSA method of analysis design exhibits over strength using pushover analysis method for evaluation.***

Keywords Pushover analysis, performance point

1. Introduction

For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required, to determine force and displacement demands in various components of the structure. Possible analysis approaches include various elastic and inelastic methods to predict the seismic performance of the structure depending upon the method of lateral load analysis, two approaches are used namely, Push over Analysis or Non-linear static procedure (NSP) and Non-linear Dynamic analysis (NDA).current structural engineering practice uses the nonlinear static procedure (NSP) or pushover analysis, for the seismic evaluation of existing buildings as well design of new buildings, due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. The practical objective of non linear seismic analysis is to predict the expected behavior of the structure in future earthquake shaking. In NSP ñpushoverò or ñcapacityò curves are generated by subjecting a detailed structural model to one or more lateral load patterns (vectors) and then increasing the magnitude of the total load to generate a non linear inelastic force deformation relationship for the structure at global level. The load vector is usually an approximate representation of the relative accelerations associated with the first mode of vibration for the structure (FEMA-440, 2005).Currently; the structural engineering profession uses the non-linear static procedures or pushover analysis described in FEMA-356 and ATC-40 documents to estimate seismic demands.

2. Analytical Modeling

Three dimensional reinforced concrete moment resisting frame building with open first storey and unreinforced brick infill walls in the upper stories was chosen for this study. The plan of the Building is shown in Fig.1. The building is considered having G+9 stories, which the ground storey is intended for parking. The building is kept symmetric in both orthogonal directions in plan to avoid torsion effect under pure lateral forces. The columns are assumed to be fixed at the base. The height of the ground floor is 4.5m and upper storey heights are 3.2m. Columns and beams are assumed having cross section of 0.5m x 0.5m and 0.25m x 0.6m respectively. Solid slabs are modeled as membrane element of 0.15m thickness for all stories. Live load on floor is taken as 3kN/m2 and on roof as 1.5kN/m2. Floor finish on the floor is 1.5kN/m2. Weathering course on roof is 2kN/m2. In the seismic weight calculation only 25% of floor live load is considered. The unit weights of concrete and masonry are taken as 25kN/m3 and 20kN/m3 respectively. Modulus of elasticity of concrete is 22360MPa and that of masonry is 2100MPa. The building is considered to be situated in seismic zone III and intended for office use. following models were considered:

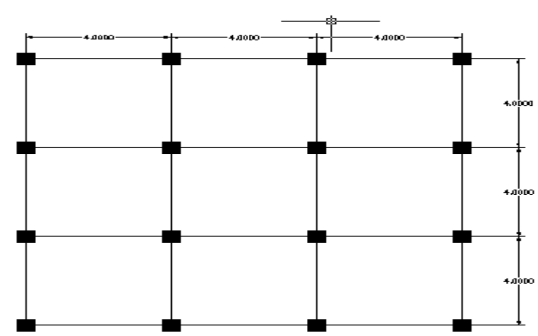

Figure 1. Plan of the Building

Model I: Bare frame model, in this Building has no walls in the first storey and 250mm thick external walls and internal walls in the upper stories. Stiffness of the walls is ignored; however, masses of infill walls are included in the model.

Model II: This Building model has no walls in the first storey and 250mm thick external and internal walls in the upper stories. Stiffness and mass of the walls is included in the model.

Model III: This Building model has 250mm thick external and internal walls in the upper stories. Further, 250mm thick masonry infill is provided at corner panels in first storey. Stiffness and mass of the walls is included in the model.

In the present work, infill panel is modelled as single equivalent diagonal strut connected between two compressive diagonal corners. The diagonal compression struts is assumed to be pin-connected to the corners of frame at both ends. The key to the equivalent diagonal strut approach lies in the determination of the effective width of the strut. In this work effective width is calculated by using Hendryȏs (1998), formula

2.1. Analysis of building

In this work three distinct analysis is carried out on all three models namely, Gravity analysis as per IS: 456-2000 code provisions, Equivalent Static Analysis and Response Spectrum Analysis as per IS: 1893(Part 1)-2002, using ETABS v9 software. Their vulnerability is evaluated using Nonlinear Static Pushover (NSP) analysis as per ATC-40 guidelines at performance levels defined in FEMA-356.

2.1.1. Equivalent Static Analysis (ESA)

The natural period of the building is calculated by the empirical expressions prescribed in the code. The total design seismic base shear calculation and its distribution along the height are done. The seismic weight is calculated using full dead load plus 25% of live load. Seismic data and weight calculations and distribution of lateral forces are appended in Appendix A. Design was carried out using following load combinations:

1.5 (DL+LL) COMB 1
1.2 (DL+LL+EQX) COMB 2
1.2 (DL+LL-EQX) COMB 3
1.2 (DL+LL+EQY) COMB 4
1.2 (DL+LL-EQY) COMB 5

Where,
DL= Dead load
LL= Live load
EQX, EQY= Earthquake load in the X- and Y- directions, respectively.

2.1.2. Response Spectrum Analysis (ESA)

RSA is carried for medium soil sites with damping ratio of 5% as per: 1893(Part 1)-2002, using ETABS v9 software. The natural period is calculated by Eigen value analysis. Spectral acceleration coefficient Sa/g for medium soil sites is determined from the equation 3.1 (IS: 1893 Part 1)-2002). The design seismic base shear, displacement, and member forces are computed as per complete quadratic combination method. Design was carried out using following load combinations:

1.2(DL+LL+SPX) COMB 6
1.2(DL+LL-SPX) COMB 7
1.2(DL+LL+SPY) COMB 8
1.2(DL+LL-SPY) COMB 9

Where, DL= Dead load, LL= Live load
SPX, SPY= Earthquake acceleration load in the X- and Y- directions, respectively.

2.1.3. Nonlinear static Pushover Analysis

Performance based seismic evaluation of building models is carried out by pushover analysis. The entire gravity load is applied first. Consequently two lateral load cases are distributed along the height of the building. Firstly, lateral load calculated as per equivalent static method (EQX) is incrementally increased along X-direction and secondly lateral load calculated as per Response spectrum method (SPX) is incrementally increased along X-direction, as shown in Table 1.1. The buildings are analysed and designed separately for the two cases for evaluation.

Table 1 Load Cases for Pushover Analysis

Pushover cases	Load,	Controlled by	Previous case
1	DL+LL	Forces	-
2	EQX	Displacements	GRAV
2	SPX	Displacements	GRAV

The analysis is carried using ETABS v9 software. The geometric nonlinearity, that is, P-delta effect is included in the analysis. Default hinge property assignment option available is used to assign non linear hinge properties to the frame members. Flexural moment (M3), axial force biaxial moment (PM2M3), and axial compressive force (P) hinges being assigned at the beam, column, and strut joints respectively. Moment (M3) hinges are considered for beam element, axial with biaxial moment (P-M-M) hinges are considered for column element and Axial P hinge is considered for diagonal strut.

3. Results and Discussions

The results are presented for each of the building model considered, for different type of analysis carried out namely analysis for gravity loading, equivalent static analysis and response spectrum analysis. The results of natural period of vibration, Base shear capacity, column design forces and lateral roof displacements for the different building models for each of the above analysis are presented and compared. Performance evaluation was then carried out by carrying out pushover analysis..

3.1. Natural Periods

The codal (IS: 1893-2002) and analytical (Eigen Value Analysis by ETABS v9 software) fundamental natural periods of the building models are shown in Table 2. It is seen that the analytical natural periods do not tally with the natural periods obtained from the empirical expressions of the code. It can be observed that the presence of stiffness of masonry infills modelled as equivalent diagonal strut significantly affects the fundamental periods of vibration of the building. The natural period obtained using EVA in case of bare frame model is 1.64 and 2.03 times more compared to model 2 and model 3 respectively. The graphical representation of the same is shown in Fig. 2. In Table 2, Base shear using ESA for each model is also presented and compared with that obtained from RSA. It is observed that although the seismic weight of all three models is same the base shear varies because of the lateral load pattern in the two methods. It is observed that in case of bare frame model 1, the shear obtained using ESA is 1.56 times more than that obtained by RSA, in case of model 2 it is 1.09 times more compared to RSA and in model 3 the base shear obtained by RSA is greater by 1.56 times compared to ESA. Now, it is seen that RSA base shear values are less conservative in model 1 and model2 and model 3 it is more compared to ESA. RSA though accepted as accurate and less conservative method of analysis, the code restricts ourselves to adopt the values obtained using ESA based on empirical equations (EE) of the code to be on conservative side. Therefore as per clause 7.6 and 7.8.2 of IS 1893 part 1, 2002, the base shear obtained using RSA are scaled upwards if found less than ESA by applying scaling factors so that the base shear obtained using RSA matches with that obtained using ESA. If base shear obtained using RSA is more compared to ESA base shear, than no scaling is carried out. In the present Example building, RSA base shear for model 1 and model 2 are scaled upwards and model 3 does not require scaling as RSA base shear is more than ESA base shear.

3.2. Lateral Roof displacement

Table 3 give the values of displacement at each storey for the three models. The roof displacement obtained from equivalent static method decreases by 60% and 70% in model 2 and model 3 respectively, compared to the bare frame model 1. In the response spectrum method, roof displacements obtained decreases by 58% and 69% in model 2 and model 3 respectively, compared to the bare frame model 1. The displacement profiles are shown in fig. 1.5 and Fig.1.6 by ESA and RSA. It is also observed that RSA method gives 12 to 20% lesser roof displacement compared to ESA method analysis.

Table 2 Fundamental Natural Period (FNP) and Base shear

Model	Seismic Weight in KN	FNP in Secs		Base Shear VB		SF
		EE	EVA	ESA	RSA	
1	29898	1.0396	1.895	1040.5	663.71	1.56
2		0.865	1.157	1252.7	1146.2	1.09
3		0.865	0.937	1252.7	1323.5	Nil

3.3 Member Forces in Ground storey columns

Table 4, shows maximum bending moment about two axis, M2 (Minor Bending), M3 (Major Bending) and maximum Axial forces (Pmax) for the first storey columns of building models designed for worst load combination considering lateral forces obtained from ESA and RSA. Axial forces and Bending Moments (BM) are compared for the End column, Interior and Corner column for the three models.

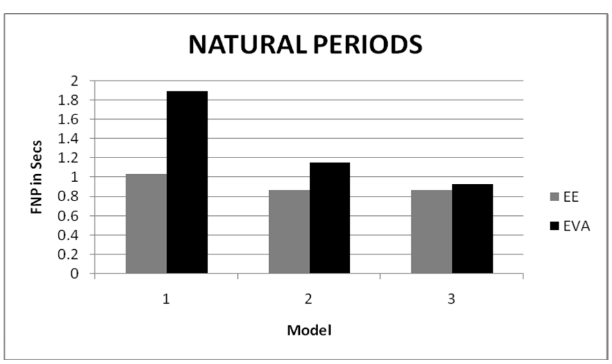

Figure 2 Natural Period

Table 3 Lateral Displacements

Storey	Displacement in mm using ESA			Displacement in mm using RSA		
	M1	M2	M3	M1	M2	M3
10	43.7	17.5	13.2	35.2	14.7	11.6
9	42.1	16.6	12.4	34.1	14.1	10.9
8	39.6	15.5	11.3	32.3	13.4	10.1
7	36.1	14.3	10.1	30	12.6	9.2
6	32	13.1	8.9	27.2	11.7	8.2
5	27.4	11.8	7.6	23.9	10.8	7.2
4	22.5	10.5	6.3	20.2	9.9	6.1
3	17.4	9.3	5.1	16.2	8.9	5.1
2	12.2	8.1	3.9	11.7	8	4
1	7	6.9	2.8	6.9	6.8	2.9

Interior Column: in case of model 1, critical load combination for ESA method is comb 3 whereas for RSA method it is comb 1, indicating that lateral forces generated due to SPX are less critical compared to gravity loading. In ESA method it observed that Axial force in the model 2 decreases slightly by 9.5% and increases slightly by 6.7% compared to Bare frame model 1 and RSA method in model 2 there is reduction in axial force by 18% and model 3 it is same as model 1. This shows that the axial forces are almost uniform for the interior columns of all models as axial force is dependent on gravity loading. Whereas the major BM M3 obtained using both the methods is almost same but there is significant variation in its values across the models. There is slight increase in BM M3 by 6.3% for model 2 compared to model 1, while in model 3 critical load combination is due to gravity load. Accordingly there will be variation in the reinforcement percentages.

End Column: axial forces are almost uniform for the End columns of all models across both the methods as axial force is dependent on gravity loading. Whereas the major BM M3 is having almost same values across the two methods of design while there is significant change in values across the models. In case of model 2 major BM M3 increases slight by 11% and that of model 3 decreases by 27%.

Corner Column: there is difference in axial force values across the two methods as well as across the models. But the variation across the methods is very less of about 4 to 5%. Axial force in the model 2 increases by 18% and in model 3 it increases by 19% compared to Bare frame model 1..there is slight variation of about 4 to 6% in the values of major BM m3 across the two methods of design while there is significant variation in the values across the models. The BM M3 of model 2 increases slight by 12 to 13% and that of model 3 decreases by 25 to 29%.

From the above observations it is obvious that the variation in design forces across the two methods namely ESA and RSA may be attributed to the difference in pattern of distribution of lateral forces along the height of the building in spite of same base shear. There is difference in design forces mainly the BM of model 1, model 2 and model 3. Model 2 and model 3 are having higher base shear compared to bare frame model 1 due to stiffness of walls in model 2 and model 3. Due to increased base force higher forces are generated in the top storeys of model 2 as seen from Table A.9 appended in Appendix A also

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there is abrupt change in stiffness in the bottom storey compared to upper storeys of model2. Due to this reason slightly larger BM are generated in the End column and corner columns of model 2 as interior columns are largely governed by gravity loads.. In case model 3 there is additional stiffness of walls present in the first storey, due to this stiffness of the first storey increase compared to model 2. It is also noticed that for End columns and Corner columns, reinforcement percentage in case of model 2 is highest compared to model 1 and model 3. Interior columns are largely governed by gravity loading and are thus less vulnerable to earthquake forces

3.4. Pushover Analysis by lateral forces generated from ESA method (EQX) as lateral load

The performance of building models, displacements corresponding to performance point (PP), and hinging pattern at PP are presented in Table 5. It is seen from the results that Base force at performance point for model 1 is greater than Design Base Shear by 55% and has performed well and possesses over strength than required at demand earthquake. It is also observed that 73% hinges formed are within elastic range of A-B, 5% are within Immediate occupancy (IO), 7% within life safety (LS) and 15% of hinges formed are within Collapse prevention (CP) levels out of total 800 hinges

Model 2: In this model the Base force at Performance Point (PP) is very high by 2.8 times compared to design base Shear. Possessing large over strength than which is required. It is also observed that roof displacements in model 2 are significantly reduced by 57% compared to BF model 1. It is also observed that 90% hinges formed are within elastic range of A-B, 6% are within Immediate occupancy (IO), 2% within life safety (LS) and 2% of hinges formed are within Collapse prevention (CP) levels out of total 1232 hinges. Model 3: model 3 has the same base shear as model 2 but the Base force at PP is 6.3 times greater than the design Base shear. The roof displacements at PP are significantly reduced by 62% compared to that of BF model 1 It is also observed that 87% hinges formed are within elastic range of A-B, 12.6% are within Immediate occupancy (IO), 1% within life safety (LS) and only 1 hinge formed within Collapse prevention (CP) levels out of total 1240 hinges

3.5. Pushover Analysis by acceleration SPX as lateral load

Performance based seismic evaluation of building models is carried out by pushover analysis by taking accelerations as lateral load in the X-direction. The performance of building models, displacements corresponding to performance point, and hinging patterns at performance levels are presented in Table 5.

Model 1: It is seen from the results that Base force at performance point for model 1 is greater than Design Base Shear by 90% and has performed well and possesses over strength than required at demand earthquake. It is also seen that Base force is increased by 22% compared to base force obtained by ESA load distribution. Roof displacement is reduced by 17% compared to ESA.

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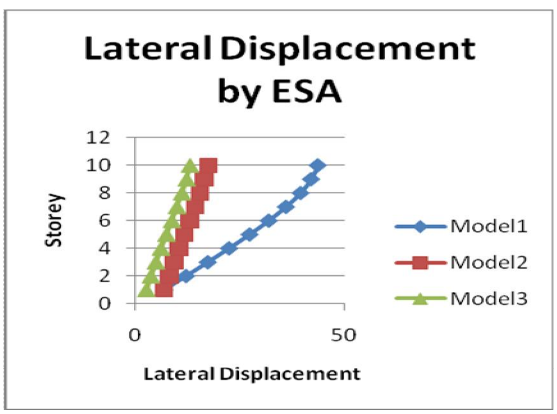

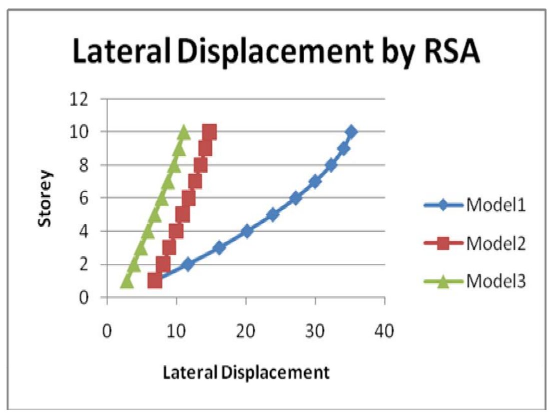

Figure 3. Lateral storey displacement by ESA and RSA

Table 4. Ground storey Column design forces

End Column										
Model	ESA					RSA				
	P (kN)	M2 (kNm)	M3 (kNm)	% reinf	Critical Comb	P (kN)	M2 (kNm)	M3 (kNm)	% reinf	Critical Comb
1	2751.7	-67.3	-246	2.67	3	2650	-64.8	-248.5	2.59	7
2	2850	-69.7	-274.1	2.95	3	2723	-66.6	-275.3	2.85	7
3	2834	-85.5	-180.8	2.35	3	2724	-84.8	-179.5	2.2	7

Interior Column										
Model	ESA					RSA				
	P (kN)	M2 (kNm)	M3 (kNm)	% reinf	Critical Comb	P (kN)	M2 (kNm)	M3 (kNm)	% reinf	Critical Comb
1	2980	-72.9	-263.5	3.01	3	3244	-30	-71	2.14	1
2	2698	-66	-280.2	2.85	3	2660	-65	-283	2.84	7
3	3180.5	-85	-85	2.17	1	3180.5	-85	-85	2.17	1
Corner Column										
Model	ESA					RSA				
	P (kN)	M2 (kNm)	M3 (kNm)	% reinf	Critical Comb	P (kN)	M2 (kNm)	M3 (kNm)	% reinf	Critical Comb
1	2217.5	-58	-223.8	1.88	3	2103	-51.4	-237.5	1.85	7
2	2593.6	-67.4	-254.7	2.59	3	2466.5	-61.5	-265	2.53	7
3	2648.7	-85	-167.6	2	3	2542.5	-84.2	-168.8	1.86	7

Table 5. Base force, Displacement and hinging pattern at PP

Model	Performance levels	Base Force (kN)	Displacement in mm	Location of Hinges								
				A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
ESA												
1	PP	1611.4	310	582	43	55	120	0	0	0	0	800
2	PP	4727.6	129	1111	77	28	16	0	0	0	0	1232
3	PP	7994.8	117	1077	144	18	1	0	0	0	0	1240
RSA												
1	PP	1973.2	255	616	40	28	116	0	0	0	0	800
2	PP	5115	116	1138	47	25	22	0	0	0	0	1232
3	PP	9419.2	94	1117	101	22	0	0	0	0	0	1240

By studying the hinging pattern It is observed that 77% hinges formed are within elastic range of A-B, 5% are within Immediate occupancy (IO), 3.5% within life safety (LS) and 14.5% of hinges formed are within Collapse prevention (CP) levels out of total 800 hinges indicating higher percentage of hinges formed in elastic range and lesser in IO, LS and CP levels compared to ESA.

Model 2: In this model the Base force at Performance Point (PP) is very high by 4 times compared to design base Shear. It is also seen that Base force is increased slightly by 8% compared to base force obtained by ESA load distribution Possessing over strength than which is required. It is also observed that roof displacement is reduced by 10% compared to ESA. It is seen that the building collapses soon after the PP. It is also observed that 92% hinges formed are within elastic range of A-B, 4% are within Immediate occupancy (IO), 2% within life safety (LS) and 2% of hinges formed are within Collapse prevention (CP) levels out of total 1232 hinges.

Model 3: model 3 has the same base shear as model 2 but the Base force at PP is 7.5 times greater than the design Base shear. It is also seen that Base force is increased by 18% compared to base force obtained by ESA load distribution The roof displacements at PP are significantly reduced by 20% compared to that obtained using ESA. It is also observed that 90% hinges formed are within elastic range of A-B, 8% are within Immediate occupancy (IO), 2% within life safety (LS) and no hinges formed within Collapse prevention (CP) levels out of total 1240 hinges.

4. Conclusion

In the present work performance evaluation of 10 storey OMRF building with OGS designed based on IS 456-2000 code is carried out using NSP for the three analytical models for the Example buildings considered in the study. The main of the study is to assess the vulnerability of these building models and suggest which model is best suited for better performance in the event of any expected earthquake. The building models are analyzed by equivalent static and response spectrum analyses as per IS: 1893 (Part 1)-2002 and pushover analyses as per ATC-40. The results in terms of Natural period, Base shear, Lateral displacements, frame design forces, performance point, are studied and discussed. Based on the results studied for the Example building following conclusions can be drawn:

It is seen that the analytical natural periods do not tally with the natural periods obtained from the empirical expressions of the code. It can be observed that the presence of stiffness of masonry infills modelled as equivalent diagonal strut significantly affects the fundamental periods of vibration of the building.Therefore, stiffness of infill wall is to be considered in for accurate estimation of FNP of buildings having unreinforced masonry infill walls.

By inclusion of infill wall stiffness in the analyses, Lateral displacements of the building are significantly reduced when compared to bare frame displacements. This in turn increases the lateral stiffness of the buildings studied.It is also observed that RSA method gives lesser roof displacement compared to ESA method of analysis. Therefore it is obvious RSA method is less conservative method of analysis and design.

By studying the design forces of the ground storey column, it can be concluded that end columns and corner columns are more vulnerable to lateral earthquake forces compared to interior columns in an OGS building designed without considering the stiffness of infill walls. This shows that increased design forces in the ground storey columns of the OGS buildings are not captured in the conventional bare frame analysis. Thus the appropriate way to analyse the OGS building is to model the strength and stiffness of infill walls.

It is also seen that RSA design is more economical compared to ESA design as the forces are obtained is less in case of RSA

From the results ESA pushover analysis it is observed that model 2 and model 3 has higher base force and lesser roof displacements compared to Bare Frame model 1 which indicates that strengthening by means of stiffening has increased the capacity of the building model in sustaining base force more than that expected at demand earthquake. By studying the hinging patterns observed in all models it is noticed that model 3 has least no of hinges formed at LS and CP levels indicating good performance compared to model 1 and 2.

It is also observed that RSA lateral load distribution method has increased amount of total lateral force required to initiate damage events within the structure. Also lesser roof displacements at PP were observed. Also percentage of hinges formed within elastic range is higher compared to ESA indicating over strength performance in the models analysed using RSA

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