

# Performance Based Seismic Design of Symmetrical and Unsymmetrical Building by Using SAP2000

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**ABSTRACT:** In this present study two R.C buildings, one symmetrical and one unsymmetrical in plan (designed according to IS 456:2000) are analysed using Pushover Analysis and redesigning by changing the main reinforcement of various frame elements and again analysing. The pushover analysis has been carried out using SAP2000, a product of Computers and Structures International. A total of 24 cases for a particular four storey building located in Zone-IV have been analysed, changing reinforcement of different structural elements, i.e. Beams and Columns, in different combinations as well as at different storey levels. The results of analysis are compared in terms of base shear, storey drift, spectral acceleration, and spectral displacement and storey displacements. The best possible combination of reinforcement that is economical, effective and whose damage is limited to Grade 2 (slight structural damage, moderate non-structural damage) in order to enable Immediate Occupancy is determined and is termed as Performance Based Design. The effect of providing shear walls, on the performance of RC framed building, is also studied using pushover analysis.

**KEYWORDS:** Push over Analysis, SAP2000, Symmetrical and Unsymmetrical.

## I. INTRODUCTION

Performance-based design begins with the selection of design criteria stated in the form of one or more performance objectives. Each performance objective is a statement of the acceptable risk of incurring specific levels of damage, and the consequential losses that occur as a result of this damage, at a specified level of seismic hazard. Losses can be associated with structural damage, non-structural damage, or both. They can be expressed in the form of casualties, direct economic costs, and downtime (time out of service), resulting from damage. Methods for estimating losses and communicating these losses to stakeholders are at the heart of the evolution of performance-based design.

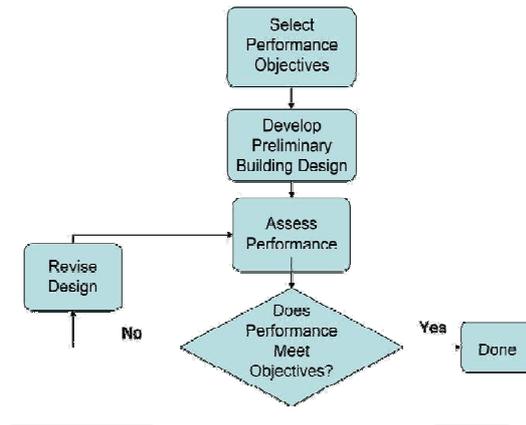
Once the performance objectives are set, a series of simulations (analyses of building response to loading) are performed to estimate the probable performance of the building under various design scenario events. In the case of extreme loading, as would be imparted by a severe earthquake, simulations may be performed using nonlinear analysis techniques. If the simulated performance meets or exceeds the performance objectives, the design is complete. If not, the design is revised in an iterative process until the performance objectives are met. In some cases it may not be possible to meet the stated objective at reasonable cost, in which case, some relaxation of the original objectives may be appropriate.

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Vol. 8, Issue 1, January 2019



**Figure 1.1 Performance-based design flow diagram [4]**

PBSD permits design of new buildings or upgrade of existing buildings with a realistic understanding of the risk of casualties, occupancy interruption, and economic loss that may occur as a result of future earthquakes.

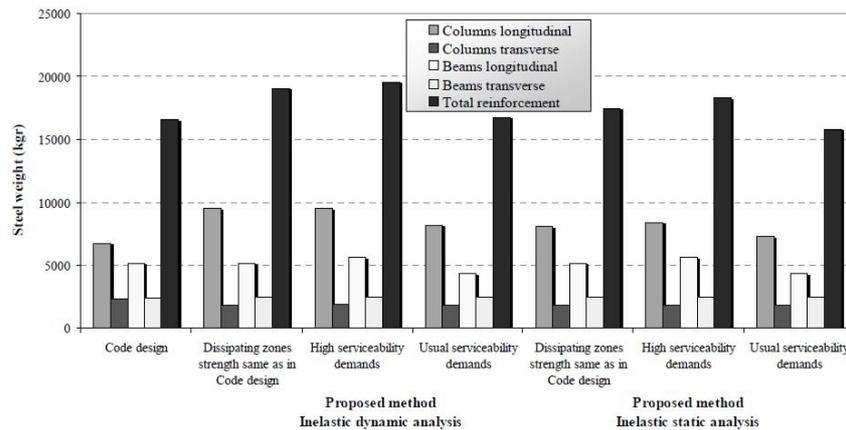
## ADVANTAGES OF PERFORMANCE-BASED SEISMIC DESIGN

1. Multi-level seismic hazards are considered with an emphasis on the transparency of performance objectives.
2. Building performance is guaranteed through limited inelastic deformation in addition to strength and ductility.
3. Seismic design is oriented by performance objectives interpreted by engineering parameters as performance criteria.
4. An analytical method through which the structural behavior, particularly the nonlinear behaviour is rationally obtained.
5. The building will meet the prescribed performance objectives reliably with accepted confidence.
6. The design will ensure the minimum life-cycle cost.

## II. RELATED WORK

**Andreas J. Kappos et al (2004)** proposed a performance-based design procedure for realistic 3D reinforced concrete (R/C) buildings, which involves the use of advanced analytical tools. The proposed method was then applied to a regular multi-storey reinforced concrete 3D frame building and was found to lead to better seismic performance than the standard code (Eurocode 8) procedure, and in addition led to a more economic design of transverse reinforcement in the members that develop very little inelastic behaviour even for very strong earthquakes.

The proposed procedure resulted, in an increase in longitudinal reinforcement of columns, at the lower storeys. This increase was more significant (about 20% in the usual serviceability case and 40% in the high serviceability case, compared to Code design) when the design was carried out using time-history analyses; increases of only 8% to 25% were found when inelastic static analysis was used. On the contrary, the transverse reinforcement was significantly reduced (from 17% to 23%). A complete picture of the reinforcement requirements in each alternative design can be obtained from Fig. 2.2.



**Figure 2.1 Required amount of steel in beams and columns, for all designs**

X.-K. Zou et al (2005) present an effective computer-based technique that incorporates pushover analysis together with numerical optimization procedures to automate the pushover drift performance design of reinforced concrete (RC) buildings. Performance-based design using nonlinear pushover analysis, is a highly iterative process needed to meet designer-specified and code requirements. This paper presents an effective computer-based technique that incorporates pushover analysis together with numerical optimization procedures to automate the pushover drift performance design. Steel reinforcement, as compared with concrete materials, appears to be the more cost-effective material that can be effectively used to control drift beyond the occurrence of first yielding and to provide the required ductility of RC building frameworks.

In this study, steel reinforcement ratios are taken as design variables during the design optimization process. Using the principle of virtual work, the nonlinear inelastic seismic drift responses generated by the pushover analysis can be explicitly expressed in terms of element design variables. An optimality criteria technique is presented in this paper for solving the explicit performance-based seismic design optimization problem for RC buildings. Two building frame examples are presented to illustrate the effectiveness and practicality of the proposed optimal design method.

### III. NUMERICAL MODELLING

#### PERFORMANCE OBJECTIVE

The following two-level performance objective is suggested for new ordinary structures.

- Under DBE, damage must be limited to Grade 2 (slight structural damage, moderate nonstructural damage) in order to enable Immediate Occupancy after DBE.
- Under MCE, damage must be limited to Grade 3 (moderate structural damage, heavy nonstructural damage) in order to ensure Life Safety after MCE.

#### DESCRIPTION OF BUILDING

In the present work, a four storied reinforced concrete frame building situated in Zone IV, is taken for the purpose of study. The plan area of building is 10 x 8 m with 3.5m as height of each typical storey. It consists of 2 bays of 5m each in X-direction and 2 bays of 4m each in Y-direction. Hence, the building is symmetrical about both the axis. The total

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(A High Impact Factor, Monthly, Peer Reviewed Journal)

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Vol. 8, Issue 1, January 2019

height of the building is 14m. The building is considered as a Special Moment resisting frame. The plan of building is shown in fig. 4.1 and the front elevation is shown in fig. 4.2.

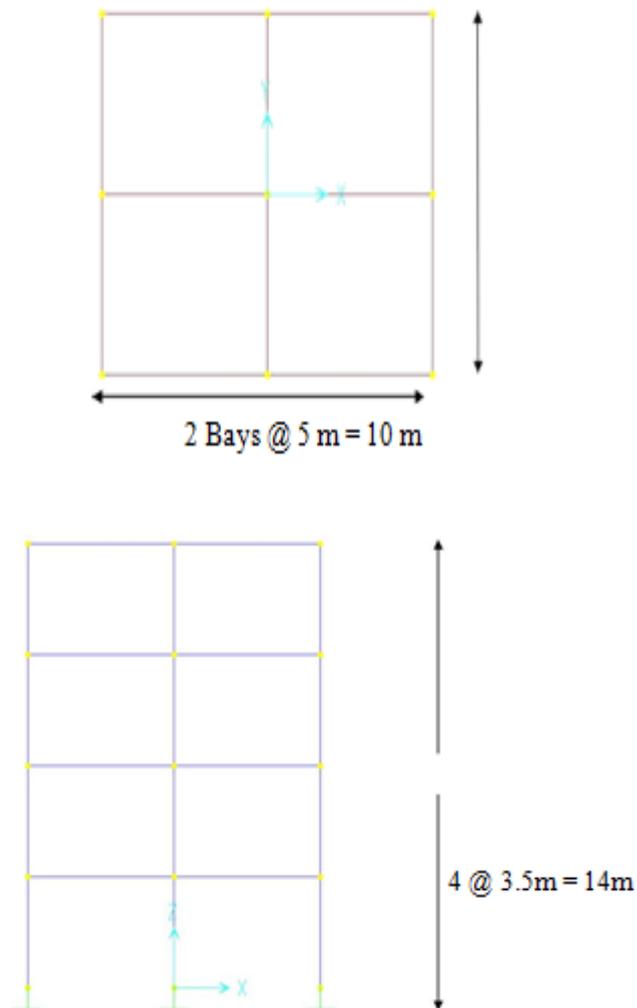


Figure 3.1 Elevation of Building

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Vol. 8, Issue 1, January 2019

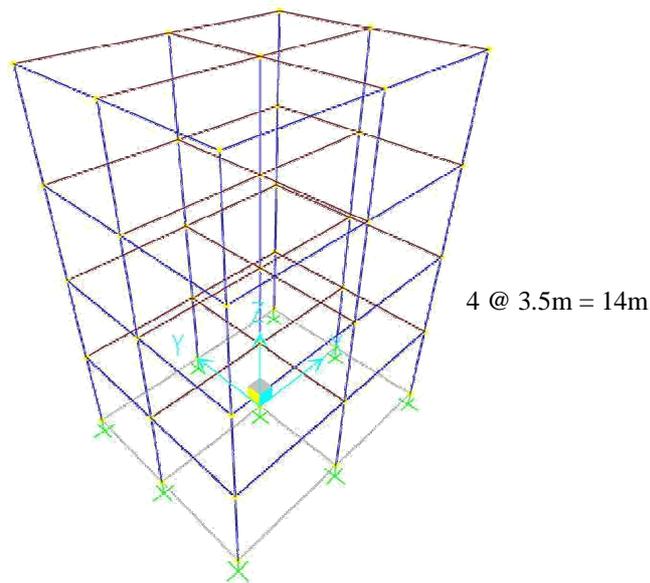


Figure 3.2 3D View of Building

## IV. SECTION PROPERTIES AND LOADS CONSIDERED

The sectional properties of elements in case of the Basic structure are taken as follows:

Size of Column= 345 x 345mm

Size of Beam= 345 x 500 mm

Thickness of Slab= 125mm thick

### LOADS CONSIDERED

The following loads were considered for the analysis of the building. The loads were taken in accordance with IS: 875.

#### 4.5.1 Gravity Loads

The intensity of dead load and live load at various floor levels and roof levels considered in the study are listed below.

#### Dead Load

##### Roof Level

Weight of Slab	0.125 x 25	3.125 kN/m <sup>2</sup>
Weight of Mud Fuska	0.150 x 16	2.400 kN/m <sup>2</sup>
Weight of Tiles	0.040 x 20	0.800 kN/m <sup>2</sup>

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Vol. 8, Issue 1, January 2019

<b>Total Dead Load</b>		<b>6.500 kN/m<sup>2</sup></b>
<b>Floor Levels</b>		
Weight of Slab	0.125 x 25	3.125 kN/m <sup>2</sup>
Weight of Screed	0.065 x 20	1.300 kN/m <sup>2</sup>
Weight of Floor Finish	0.040 x 24	0.960 kN/m <sup>2</sup>
Weight of partition Wall	-	1.500 kN/m <sup>2</sup>
<b>Total Dead Load</b>		<b>7.000 kN/m<sup>2</sup></b>

### Live Load

Live load at all floor levels = **3.5 kN/m<sup>2</sup>**

### DETERMINATION OF LATERAL LOADS FOR PUSHOVER ANALYSIS

The maximum design lateral force, **Qi**, was computed for each storey level and was distributed at each node. The calculation of this force is illustrated below:

#### 4.6.1 Calculation of seismic Weight of Structure

Seismic weight of roof is calculated as under:

$$\text{Slab} = 0.125 \times 4 \times 5 \times 25 \times 4 = 250 \text{ kN}$$

$$\text{Beams} = 54 \times 0.345 \times 0.5 \times 25 = 232.87 \text{ kN}$$

$$\text{Columns} = 0.345 \times 0.345 \times 1.75 \times 25 \times 9 = 46.86 \text{ kN}$$

**Total = 529.73 kN**

Seismic weight of one floor is calculated as under:

$$\text{Slab} = 0.125 \times 4 \times 5 \times 25 \times 4 = 250 \text{ kN}$$

$$\text{Beams} = 54 \times 0.345 \times 0.5 \times 25 = 232.87 \text{ kN}$$

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(A High Impact Factor, Monthly, Peer Reviewed Journal)

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Vol. 8, Issue 1, January 2019

Columns =  $0.345 \times 0.345 \times 3.5 \times 25 \times 9 = 93.73 \text{ kN}$

**Total = 576.60 kN**

**Hence Total Seismic weight = 2259.5 kN**

### 4.6.2 Calculation of base shear

The following parameters were taken:

Zone Factor,  $Z = 0.24$

Importance Factor,  $I = 1.0$

Response Reduction Factor = 5.0

Time Period is calculated from:

$TS = 0.09 \text{ h}/\sqrt{d} = .09 \times 14/\sqrt{10} = 0.4 \text{ seconds}$

Hence,  $Sa/g = 2.5$  (For Medium Soil Conditions)

Hence,  $A_h = (.24/2) \times (1/5) \times 2.5 = .06$

Thus  $V_b = .06 \times 2259.5 = 131 \text{ kN}$

$\sum W_j h_j^2 = 202714$

Now,  $Q_i = V_b \frac{W_i h_i^2}{\sum W_i h_i^2}$

$$\frac{W_i h_i^2}{\sum W_i h_i^2}$$

Hence,  $Q_4 = (131 \times 529.73 \times 14^2)/202714 = 67.10 \text{ kN}$

Similarly,  $Q_3 = 41.08 \text{ kN}$

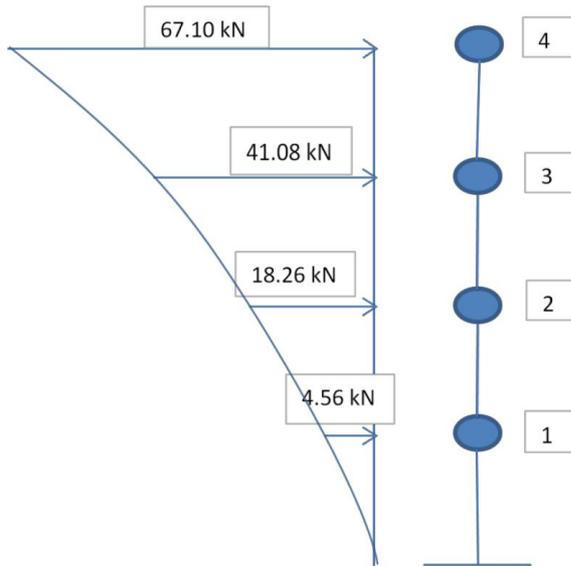
$Q_2 = 18.26 \text{ kN}$        $Q_1 = 4.56 \text{ k}$

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Vol. 8, Issue 1, January 2019



**Figure 4.1 Applied Inverted Triangular Loading**

This load was applied to the structure for pushover analysis. This load is similar to the inverted triangular loading suggested for pushover analysis by various codes such as ATC-40, etc.

**Table 4.1 Structural details (as per Analysis and Design on Staad.Pro)**

Element	Dimension (m)	Reinforcement Area in mm <sup>2</sup>
Corner Columns	0.345 x 0.345	452
Mid-face Columns	0.345 x 0.345	804
Interior Column	0.345 x 0.345	1260
Beams 1 <sup>st</sup> storey	0.345 x 0.5	785 (top)
		550 (bottom)
Beams 2 <sup>nd</sup> storey	0.345 x 0.5	678 (top)
		550 (bottom)
Beams 3 <sup>rd</sup> storey	0.345 x 0.5	942 (top)
		550 (bottom)
Beams 4 <sup>th</sup> storey	0.345 x 0.5	678 (top)
		550 (bottom)

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(A High Impact Factor, Monthly, Peer Reviewed Journal)

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Vol. 8, Issue 1, January 2019

## V. ANALYSIS AND RESULTS

**Table 5.1 Comparison of Roof Displacement**

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	ROOF DISPLACEMENTS (mm)	PERCENTAGE CHANGE IN ROOF DISPLACEMENT
Basic structure			152.3	
Beams of 1st STOREY	CASE 1	14.65	137.2	-9.91
	CASE 2	20	133.1	-12.61
Beams of 2nd STOREY	CASE 3	15.78	149.5	-1.84
	CASE 4	32.74	146.5	-3.81
Beams of 3rd STOREY	CASE 5	4.17	152.3	0
	CASE 6	9.03	131.6	-13.59
Beams of 4th STOREY	CASE 7	15.78	152.3	0
	CASE 8	32.74	152.3	0

**Table 5.2 Variation of Roof Displacement with Base Force for all cases**

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	BASE SHEAR (KN)	ROOF DISPLACEMENTS (mm)
Basic structure			599.07	152.3
Beams of 1st STOREY	CASE 1	14.65	605.49	137.2
	CASE 2	20	606.76	133.1
Beams of 2nd STOREY	CASE 3	15.78	579.46	149.5
	CASE 4	32.74	600.53	146.5
Beams of 3rd STOREY	CASE 5	4.17	598.84	152.3
	CASE 6	9.03	599.02	131.6
Beams of 4th STOREY	CASE 7	15.78	599.07	152.3
	CASE 8	32.74	599.07	152.3
Columns of 1st & 2nd STOREY	CASE 9	4.09	600.5	151.4
	CASE 10	39.23	603.2	120.2
Columns of 3rd & 4th STOREY	CASE 11	4.09	584.63	148.7
	CASE 12	39.23	600.97	142.9
Beams & Columns of 1st & 2nd STOREY	CASE 13	11.51	610.35	150.7

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Vol. 8, Issue 1, January 2019

	CASE 14	30.66	614.45	141.7
Beams & Columns of 3rd & 4th STOREY	CASE 15	8.01	602.87	142.9
	CASE 16	27	600.97	132.4
Basic structure with Shear wall	CASE 17		553.83	55.8
Columns of 1 <sup>st</sup> & 2 <sup>nd</sup> STOREY	CASE 9	4.09	151.40	-0.59
	CASE 10	39.23	120.20	-21.08
Columns of 3 <sup>rd</sup> & 4 <sup>th</sup> STOREY	CASE 11	4.09	148.70	-2.36
	CASE 12	39.23	142.90	-6.17
Beams & Columns of 1 <sup>st</sup> & 2 <sup>nd</sup> STOREY	CASE 13	11.51	150.70	-1.05
	CASE 14	30.66	141.70	-6.96
Beams & Columns of 3 <sup>rd</sup> & 4 <sup>th</sup> STOREY	CASE 15	8.01	142.90	-6.17
	CASE 16	27	132.40	-13.07
Basic structure with Shear wall	CASE 17		55.80	-63.36

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Vol. 8, Issue 1, January 2019

**Table 5.3 Variation of Roof Displacement with column reinforcement for all cases**

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	ROOF DISPLACEMENT (mm)	PERCENTAGE DECREASE IN ROOF DISPLACEMENT
Basic structure			155	
Column 1	CASE 1	77.87	144	7.1
	CASE 2	177.87	129.6	16.39
Column 2	CASE 3	77.87	138.9	10.39
	CASE 4	177.87	119.1	23.16
Column 3	CASE 5	77.87	138.1	10.90
	CASE 6	177.87	133.9	13.61
Column 4	CASE 7	77.87	144.5	6.77
	CASE 8	177.87	131.0	15.48
Column 5	CASE 9	77.87	0	15.87
	CASE 10	177.87	0	25.48
Column 6	CASE 11	77.87	144.2	6.97
	CASE 12	177.87	134.5	13.23
Column 7	CASE 13	77.87	144.3	6.90
	CASE 14	177.87	132.8	14.32

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(A High Impact Factor, Monthly, Peer Reviewed Journal)

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Vol. 8, Issue 1, January 2019

Column 8	CASE 15	77.87	143.6	7.35
	CASE 16	177.87	120.5	22.26

**Table 5.4 Comparison of area of reinforcement in mm<sup>2</sup> in beams and columns for all designs**

Element	IS 456:2000	Performance based Design	IS 1893:2002
Corner Columns 1 <sup>st</sup> and 2 <sup>nd</sup> storey	452	1880	1260
Corner Columns 3 <sup>rd</sup> and 4 <sup>th</sup> storey	452	905	1260
Mid-Frame Columns 1 <sup>st</sup> and 2 <sup>nd</sup> storey	804	1880	1260
Mid-Frame Columns 3 <sup>rd</sup> and 4 <sup>th</sup> storey	804	905	1260
Interior Column 1 <sup>st</sup> and 2 <sup>nd</sup> storey	1260	1880	1260
Interior Column 3 <sup>rd</sup> and 4 <sup>th</sup> storey	1260	905	1260
Beams 1 <sup>st</sup> storey	785 (top) 550 (bottom)	1020 (top) 550 (bottom)	1020 (top) 550 (bottom)
Beams 2 <sup>nd</sup> storey	680 (top) 550 (bottom)	942 (top) 550 (bottom)	942 (top) 550 (bottom)
Beams 3 <sup>rd</sup> storey	942 (top) 550 (bottom)	864 (top) 550 (bottom)	864 (top) 550 (bottom)
Beams 4 <sup>th</sup> storey	680 (top) 550 (bottom)	680 (top) 550 (bottom)	785 (top) 550 (bottom)

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Vol. 8, Issue 1, January 2019

Following results are obtained for pushover analysis of Performance based design:

Base Shear = 531.63 kN

Roof Displacement = 91.0mm

Thus Roof displacement is less than target roof displacement.

## VI. CONCLUSIONS

1. Performance increases on increasing reinforcement of columns only resulting into an appreciable decrease in the maximum roof displacement both symmetrical as well as unsymmetrical building. Decrease in roof displacement is maximum interior column and for corner and mid-face columns it is comparable.
2. The increase in reinforcement of columns only results into a nominal increase in base shear, for both symmetrical building and un-symmetrical building. It is observed that changing reinforcement of 1<sup>st</sup> storey affects base shear more than other storeys.
3. Performance of the building decreases when the sectional sizes of beams and columns are reduced while keeping same reinforcement.
4. Increasing reinforcement of beams and columns both result in an appreciable decrease in roof displacement, for both symmetrical building and un-symmetrical building.
5. Provision of shear wall results in a huge decrease in base shear and roof displacement both symmetrical building and un-symmetrical building.

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