International Journal of Advances in Engineering and Management (IJAEM)
Volume 3, Issue 7 July 2021, pp: 465-473 www.ijaem.net ISSN: 2395-5252

Earthquake Performances and Evaluation of Existing Building Design as Per Past Code of Practice

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Submitted: 25-06-2021 Revised: 01-07-2021 Accepted: 03-07-2021

ABSTRACT
Codes of practice for plain and reinforced concrete and earthquake resistant design are always revised periodically. Assessing the capacity of present building as per the requirement of new codes of practice is an important task. In this paper, three typical designs of a 6-Storey building are carried out as per past codes of practice for three load cases and they are, i) Case–1: For gravity load plus earthquake load as per IS: 456- 1964 and IS: 1893-1966 (Working stress method), ii) Case-2 For gravity load plus earthquake load as per IS: 456-1978 and IS: 1893- 1984(Limit state method), iii)Case-3: For gravity load plus earthquake load as per IS: 456-2000 and IS:1893-2002 (Limit state method).With theseload cases the performance evaluation of the building is carried out with nonlinear static analyses and the capacity curves are generated. From these curves, the variation in maximum base shear and roof displacement capacities for the three different load cases are brought out clearly. The performance points are obtained and the corresponding base shear and roof displacements are arrived for IS 1893 – 2002, Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE). All the three designs are found to meet the design basis earthquake demand. However, only case-3, is found to meet the performance point for maximum considered earthquake.

KEYWORDS:Working stress method, Limit state method, Non-linear static Analysis, Push over curve, Performance point.

I. INTRODUCTION

In general Life safety of buildings has become an important big problem. The strength and ductility of the buildings designed and detailed using earlier versions of the codes are becoming important issuesfor assessing their safety prescribed by the present earthquake codes of practice. In present study nonlinear static analysis is used to evaluate the performance of the buildings. Presently, there are two nonlinear static analysis procedures available, one termed as the Displacement Coefficient Method (DCM) included in the FEMA-356 document and the other termed as the Capacity Spectrum Method (CSM) included in the ATC-40 document. Both of these methods depend on the lateral load –deformation variation obtained by using the nonlinear static analysis under the gravity loading and idealized lateral loading due to seismic action. In the present work an attempt is pursued to establish guidelines for strengthening/retrofitting the existing buildings designed as per the past codes of practice to the present revisions of codes of practice. For seismic performance evaluation the existing building, a 6-Storey building is taken from, IITK-GSDMA-EQ26-V3.0. This is a typical beam-column RC frame building with no shear wall. The building considered does not have any vertical plan irregularities and it is a 6- storey office building. The building is analysed for three cases. They are, i) Case–1: For dead load plus earthquake load as per IS: 456- 1964 and IS: 1893-1966. ii) Case 2: For dead load plus earthquake load as per IS: 456-1978, and IS: 1893- 1984, iii) Case-3: For dead load plus earthquake load as per IS: 456-2000 and IS: 1893-2002.

The analysis of building for the three cases is carried out with STAADPro package and spread sheets are developed to design the cross sections. The building is designed for the three load cases using the spread sheets. The section details are arrived by working stress method for case-1 and by limit state method. SAP-2000 nonlinear analysis program is used to obtain the capacity of the buildings by push over analysis for the three cases.

1.1 Details of 6-storey Building

The building studied is a 6-storey office building. The plan and elevation of the building are

International Journal of Advances in Engineering and Management (IJAEM)
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shown in Fig.3.1.The soil type is medium soil and the plan is regular in nature it is a symmetrical one there are four cases are studied They are i) Case–1: For gravity load plus earthquake load as per IS: 456- 1964 and IS: 1893-1966. ii) For gravity load plus earthquake load as per IS: 456-1978 and IS: 1893- 1984. iii) Case-3: For gravity load plus earthquake load as per IS: 456-2000 and IS: 1893-2002. Pushover analysis of this problem is carried out using SAP-2000 software package.

1.2 Design Details

The building is assumed to have only external walls of 230mm thick with 12mm plaster on both sides and no internal walls are assumed. At ground floor only tie beams are provided. M20 grade concrete and F415 grade steel are used for design. The sizesof all columns are kept equal and to be equal to 500mm x 500mm. The sizes of all beams are kept equal to 300mm x 600mm. At ground floor slabs are not provided and the floor will directly rest on ground. Therefore, only ground beams passing through columns are provided as tie beams. The design data considered.

Different load cases studied and design methodology adopted are given in Table- 1 For seismic performance evaluation the 6-Storey building, is designed with different revisions of codes of practice with respective seismic zones.

Table-1 The Different Cases Studied

	Case-1	Case-2	Case-3
Codes	IS: 456-1964 and IS: 1893-1966	IS:456-1978 and IS:1893-1984	IS: 456-2000 and IS: 1893-2002.
Load cases with factors	(DL+EQ)	1.5(DL+EQ)	1.5(DL+EQ)
Design approach	WS method	LS method	LS method

1.3 Estimation of base shear calculation

The design base shear for the various cases studied as per the revisions of IS: 1893.

1.4Analysis of the building

The analysis of the building is carried out by using STADD PRO software package for the four cases. The Fig-1 shows the frame studied under gravity loads and lateral loads considered in each case is calculated. The values for axial forces and Moments for column members and Moments and Shear force for beam members respectively are given in Table-B1-B6.

1.5 Reinforcement Details

The axial force and moments found from the analysis packages (STADD PRO) of are used for designing column members as per IS: 456-1964 for case 1 and SP-16 for case-2 and 3, and they are given in Table-2 (exterior columns) and Table-3 (interior columns). Considering the moments and shear forces the beam members are designed as per IS: 456-1964 for case 1 and SP-16 for case-2 and 3, and given in Table-4

Fig.-1 Gravity Loads: Frame AA

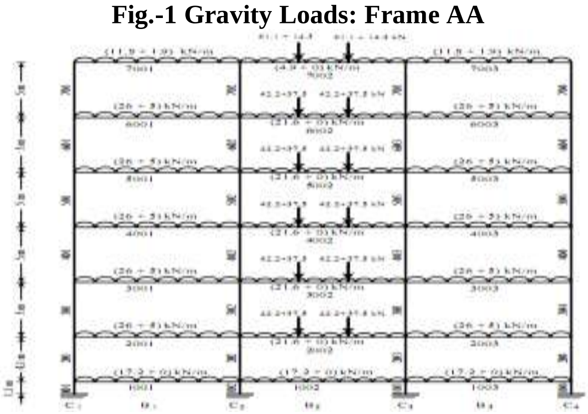

Table-2 Forces and Reinforcement Details

		Case-1 (DL+EQ) IS:456-1964, 1893-1966 WS	Case-2 1.5(DL+EQ) IS:456-1978, 1893-1984	Case-3 1.5(DL+EQ) IS:456-2000, 1893-2002
C101,C401, SPAN = 1100	Force (kN)	1093	1639	1799
	Moment (kNm)	143	214.5	314
	Section-1	600x600	600x600	600x600
	Longitudinal	3-25 Φ T/B	4 -25Φ T/B	8-25Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C112, C412 SPAN = 4100	Force (kN)	992	1488	1638.4
	Moment (kNm)	985	273	356
	Section-1	500x500	500x500	500x500
	Longitudinal	3-25Φ T/B	4 -25Φ T/B	8-25 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C123,C423 SPAN = 5000	Force (kN)	817.6	1226.4	1347
	Moment (kNm)	171	256.2	336
	Section-1	500x500	500x500	500x500
	Longitudinal	3-25Φ T/B	4 -25Φ T/B	4 -25 Φ T/B, 4-22 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C134, C434 SPAN = 5000	Force (kN)	630	945	1031
	Moment (kNm)	162.4	244	315.2
	Section-1	500x500	500x500	500x500
	Longitudinal	3-25Φ T/B	4 -25Φ T/B	4 -25 Φ T/B, 4 -22 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C145,C445 SPAN = 5000	Force (kN)	445	667	720
	Moment (kNm)	158	236.3	303.3
	Section-1	500x500	500x500	500x500
	Longitudinal	3-25Φ T/B	4 -25Φ T/B	4 -25 Φ T/B, 4 -22 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C156, C456 SPAN = 5000	Force (kN)	266	399	425
	Moment (kNm)	148	222	279
	Section-1	500x500	500x500	500x500
	Longitudinal	3-25Φ T/B	3-25Φ T/B	4 -25 Φ T/B, 4 -22 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C167, C467 SPAN = 5000	Force (kN)	98	147	155
	Moment (kNm)	110	165	198
	Section-1	500x500	500x500	500x500
	Longitudinal	3-25Φ T/B	3-25Φ T/B	4 -25 Φ T/B, 4-22 Φ T/B

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	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c

Table-3 Forces and Reinforcements

		Case-1 (DL+EQ) IS:456-1964, 1893-1966 WS	Case-2 1.5(DL+EQ) IS:456-1978, 1893-1984	Case-3 1.5(DL+EQ) IS:456-2000, 1893-2002
C201,C301 SPAN = 1100	Force (kN)	1796	2694	2709
	Moment (kNm)	145	217.3	320
	Section-1	600x600	600x600	600x600
	Longitudinal	4-25Φ T/B	6-25Φ T/B	8-25Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C212, C312 SPAN = 4100	Force (kN)	1624.5	2436.7	2452
	Moment (kNm)	168	251.4	369
	Section-1	500x500	500x500	500x500
	Longitudinal	4-25Φ T/B	6-25Φ T/B	8-25Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C223, C323 SPAN = 5000	Force (kN)	1338	2007	2018
	Moment (kNm)	195.3	293	452
	Section-1	500x500	500x500	500x500
	Longitudinal	4-25Φ T/B	6-25Φ T/B	4 -25 Φ T/B, 4-22 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C234,C334 SPAN = 5000	Force (kN)	1047.2	1571	1578
	Moment (kNm)	188.6	283	405.2
	Section-1	500x500	500x500	500x500
	Longitudinal	4 -25Φ T/B	5-25Φ T/B	4 -25 Φ T/B, 4-22 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C245,C345 SPAN = 5000	Force (kN)	759	1138	1142
	Moment (kNm)	176.4	265	376.2
	Section-1	500x500	500x500	500x500
	Longitudinal	4-25Φ T/B	5-25Φ T/B	4 -25 Φ T/B, 4-22 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C256,C356 SPAN = 5000	Force (kN)	472.4	709	710
	Moment (kNm)	144	216	305.4
	Section-1	500x500	500x500	500x500
	Longitudinal	3-25Φ T/B	3-25Φ T/B	4 -25 Φ T/B, 4-22 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c
C267,C367 SPAN = 5000	Force (kN)	189	283	284
	Moment (kNm)	125	187	244
	Section-1	500x500	500x500	500x500
	Longitudinal	3-25Φ T/B	3-25Φ T/B	4 -25 Φ T/B,

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				4-22 Φ T/B
	Transverse	8Φ2L@200c/c	8Φ2L@200c/c	8Φ5L@200c/c

Table-4 Forces and Reinforcements

	Case1	Case2	Case3
Support All Beam B212 to B734	300x600 4-25Φat top 4-25Φ at bottom	300x600 4-25Φat top 4-25Φ at bottom	300x600 7-25Φat top 6-20Φ at bottom
Mid Span All Beam B212 to B734	300x600 2-25Φat top 4-25Φ at bottom	300x600 2-25Φat top 4-25Φ at bottom	300x600 2-25Φat top 52-20Φ at bottom
Support Beam B112,B123,B134	300x600 3-25Φat top 3-25Φ at bottom	300x600 3-25Φat top 3-25Φ at bottom	300x600 5-20Φat top 5-20Φ at bottom
Mid Span Beam B112,B123,B134	300x600 3-25Φat top 3-25Φ at bottom	300x600 3-25Φat top 3-25Φ at bottom	300x600 5-20Φat top 5-20Φ at bottom

Codes of practice for plain and reinforced concrete, IS: 456 and the code for criteria for earthquake resistant design IS: 1893 are revised periodically. This chapter summarizes the design guidelines and features as per the revisions of IS: 456-1964, 1978 and 2000 and estimation of design seismic base shear (seismic coefficient method) as per the revisions of IS: 1893-1966, 1984 and 2002. Apart from the general analysis and design guidelines, the problem definition and methodology adopted for analysis and design of four load cases studied also presented. The 6-Storey office building with different load cases with reinforcement details for column and beam members as per the three cases are also discussed.

II. NON LINEAR STATIC ANALYSIS

2.1 Capacity

The overall capacity of a structure depends on the strength and deformation capacities of individual components of the structure. In order to determine capacities beyond the elastic limits some form of nonlinear analysis is required. This procedure uses a series of sequential elastic analyses superimposed to approximate a force-displacement capacity diagram of the overall structure. The capacity curve is generally constructed to represent the first mode response of the structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure. This is generally valid for buildings with fundamental periods of vibration up to 1 second. For more flexible buildings with fundamental period greater than one second, higher modes need to be considered.

2.2 Demand

Demand is the representation of earthquake ground motion and capacity is a representation of the structure's ability to resist the seismic demand. There are three methods to establish the demand of the building. They are i) Capacity spectrum method, ii) Equal displacement method and iii) Displacement coefficient method. Out of these three methods capacity spectrum method is widely used and it is adopted here.

2.3 Evaluation Based on Nonlinear Pushover Analysis

Push over analysis is a nonlinear static analysis in which the magnitude of the lateral load is gradually incrementally increased, maintaining a predefined distribution pattern along the height of the building. By increasing the magnitude of the loads, as a result in weak links and failure modes of the building will occur. In pushover analysis one 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. It gives an idea of the maximum base shear that the structure is capable of resisting. For regular buildings, it can also give a rough idea about the global stiffness of the building.

2.4Procedure Adopted for Pushover Analysis

Create the basic computer model (without the pushover data) in the usual manner using the graphical interface of SAP2000 makes this quick and easy task as shown in the Figure -2

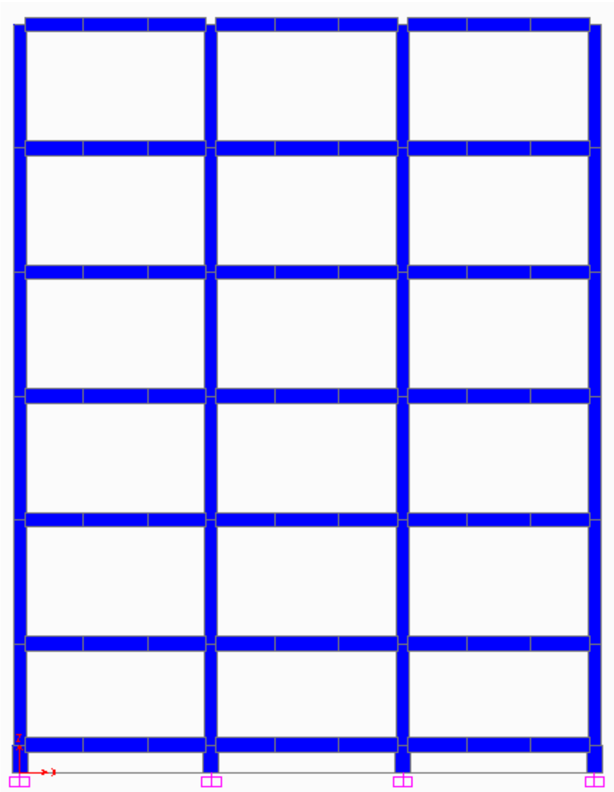

Fig -2 Model of the Frame

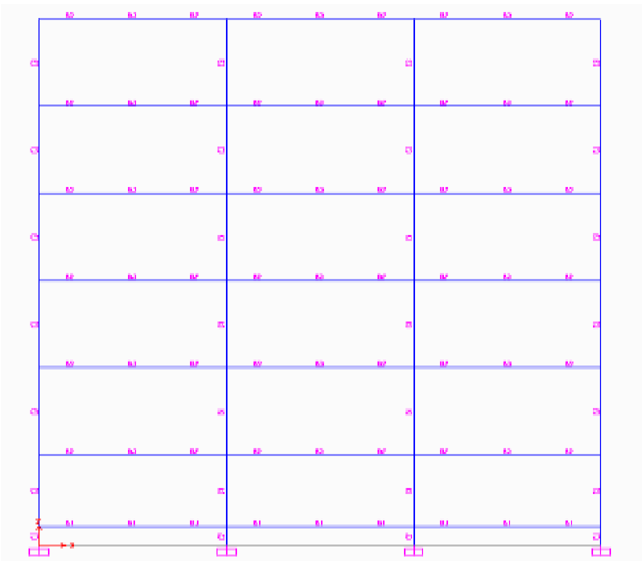

Fig -4 Assigning the Member Sections

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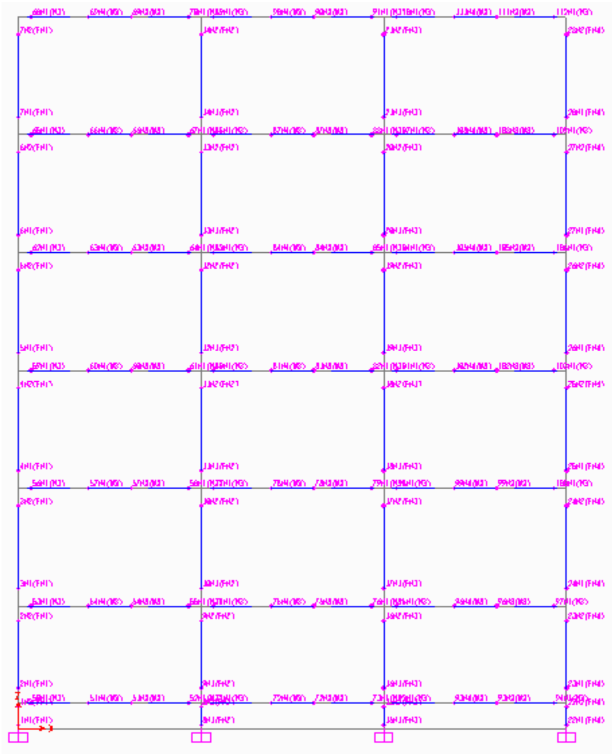

Fig-5 Assigning the plastic hinges

Define the pushover load cases. In SAP2000 more than one pushover load case can be run in the same analysis. Also a pushover load case can start from the final conditions of another pushover load case that was previously run in the same analysis.

Typically the first pushover load case is used to apply gravity load and then subsequent lateral pushover load cases are specified to start from the final conditions of the gravity pushover. Pushover load cases can be force controlled, that is, pushed to a certain defined force level, or they can be displacement controlled, that is, pushed to a specified displacement.

Typically a gravity load pushover is force controlled and lateral pushovers are displacement controlled. SAP2000 allows the distribution of lateral force used in the pushover to be based on a uniform acceleration in a specified direction, a specified mode shape, or a user-defined static load case. Here how the displacement controlled lateral pushover case that is based on a user-defined static lateral load pattern named PUSH is defined for our case.

2.5 Nonlinear Static Analysis of the 6- Storey Building

Towards the performance evaluation of building designed as per past codes of practice nonlinear static analyses are carried out for the 6 storey building designed earlier. Considering the symmetry of the building and neglecting torsion effects, the 2D model of frame AA is simulated in SAP2000 for pushover analysis. The frame is modelled with default PMM hinge properties for columns and M3 hinge properties for beams. Displacement controlled nonlinear static pushover analyses are carried out for the different load cases studied. The capacity curves for the three load cases are shown in Fig-6 and the Maximum Base shear and roof Displacement are given in Table 5. The capacity curves are transformed to capacity spectra in ADRS format.

The demand spectra as per IS 1893 – 2002 (Zone III) 5% response spectra for design basis earthquake (DBE) is obtained and converted to ADRS format. The capacity curves, demand curves and performance points are calculated. The base shear and roof displacement corresponding to the performance points as per IS 1893 – 2002 (Zone III) DBE earthquake are given in Table -6

Table-5 Maximum Base shear and Roof displacement for the 6-storey building

Maximum Base shear and Roof displacement		
cases	Base shear (kN)	Roof Displacement (m)
Case-1	896.99	0.11
Case-2	1094.97	0.099
Case-3	1332.675	0.113

Table -6 Performance Points for IS 1893 -2002 DBE Medium soil

Performance Points for IS 1893 -2002 DBE Medium soil				
Cases	Sd (m)	Sa(g)	Displacement(m)	Base Shear(kN)
Case1	0.032	0.092	0.032	862.146
Case2	0.030	0.097	0.030	912.797
Case3	0.030	0.097	0.030	912.797
Sd : Spectral Displacement, Sa: Spectral Acceleration, g is acceleration due to gravity				

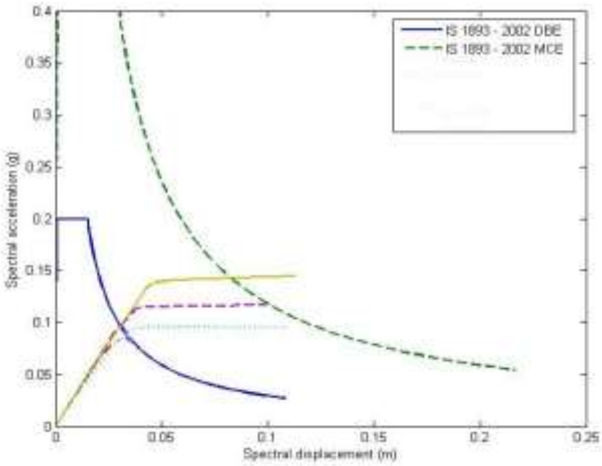

Fig-6 Capacity curve the three load cases

III. RESULT

From the pushover analysis results, it is seen that the performance point for case 1 are observed near the yield point of their capacity spectra for the demand of IS 1893 DBE earthquake (Zone III). Performance points are not obtained for case 1 for the demand of IS 1893 MCE earthquake (Zone III). Performance points for case 2 and case 3 are observed in the elastic region for the demand of IS 1893 DBE earthquake (Zone III). Hence the necessity to convert the 5% demand spectra for higher effective damping did not arise. However for case 3, performance point for MCE earthquake is observed in the inelastic region of the capacity curve. Necessary correction for effective damping needs to be carried out and the performance point can be obtained by trial and error method accordingly. The base shears and maximum displacements corresponding to the performance points reveal the inelastic capacity of existing building designed as per past codes of practice.

IV. SUMMARY AND CONCLUSIONS

In this thesis, the evolution of RC design procedure from working stress method to limit state method as given in different versions of IS: 456 are discussed. The four typical designs have been carried out as per old and present codes of practice. The nonlinear static analyses are carried out and the capacity curves are generated. The variation in maximum base shear and roof displacement capacities for the four different cases are brought out clearly. The performance points are obtained and the corresponding base shear and roof displacements are arrived for IS: 1893 – 2002 design basis earthquake and maximum considered earthquake. All the three designs are found to meet the design basis earthquake demand. However, only case 3, is found to meet the performance point for maximum considered earthquake.

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