SEISMIC PERFORMANCES AND EVALUATION OF PRESENT R.C.C BUILDING DESIGN AS PER REVISED CODE OF PRACTICE

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ABSTRACT

Regular Codes of practice of plain and reinforced concrete and earthquake resistant design are always changed periodically with time. Calculating the capacity of present building as per the requirement of current codes of practice is an important task. In this study, three typical designs of a six-Storey building are taken out as per revised codes of practice for three load cases that is 1) Case–1: For Gravity load plus EQL as per IS: 456- 1964 and IS: 1893-1966 (WSM), 2) Case-2 Gravity load plus EQL as per IS: 456-1978 and IS: 1893- 1984(LSM), 3)Case-3: For Gravity load plus EQL as per IS: 456-2000 and IS:1893-2002 (LSM). With these different load cases the performance evaluation of the R.C.C Building is determined by the nonlinear static analyses and the capacity curves are generated. The variation in maximum base shear and roof displacement capacities for the three different load cases are came out clearly. All the three designs are found to meet the design basis earthquake demand. However, the Case-3 is only found to meet the performance point for Maximum considered earthquake.

KEYWORDS: Working stress method, Limit state method, Push over Analysis, Push over curve, Performance point.

1. INTRODUCTION

In general the Life safety of buildings has become an important big Issue. The strength and ductility of the buildings designed and detailed using earlier versions of the codes are becoming important issues for 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 R.C.C buildings. Presently, there are two nonlinear static analysis procedures are available, one is Displacement Coefficient Method (DCM) it is included in the FEMA-356.and the another on that is termed as the Capacity Spectrum Method (CSM) included in the ATC-40 . 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 work. In the present work an attempt is considered to establish the guidelines for strengthening/retrofitting of the existing or present buildings designed as per the past codes of practice to the present revisions of codes of practice that is IS 456-2000. For seismic performance evaluation of the existing building, a 6-

Storey building is taken. 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 six- storey office building. The building is analysed for three different load cases. Case–1: For DL and LL plus EQL as per IS: 456- 1964 and IS: 1893-1966 (WSM), ii) Case-2 DL and LL plus EQL as per IS: 456-1978 and IS: 1893-1984(LSM), iii)Case-3: For DL and LL plus EQL as per IS: 456-2000 and IS:1893-2002 (LSM).

The analysis of building for the three cases is carried out with STAADPro software package and spread sheets are developed manually to design the cross sections of the member. The building is designed for the three different load cases using the spread sheets. The section details are calculated by using WSM for case-1 and LSM. SAP-2000 software is to be used for nonlinear static analysis to determine the capacity of the buildings by push over or Non Linear static analysis for the three different load cases.

1.1 Details of Six-Storey R.C.C Building

The building studied is a six-storey office building. The plan and elevation of the building are 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 three cases are carried out They are i) Case-1: For DL and LL plus EQL as per IS: 456- 1964 and IS: 1893-1966 (WSM), ii) Case-2 DL and LL plus EQL as per IS: 456-1978 and IS: 1893- 1984(LSM), iii)Case-3: For DL and LL plus EQL as per IS: 456-2000 and IS:1893-2002 (LSM). Pushover or Non Linear static analysis of this problem is carried out by using SAP-2000 software package.

1.2 Design Details

The building is assumed to have only external walls of thickness 230mm and with 12mm plaster on both sides and there is no internal walls are assumed. At ground floor only tie beams are provided. M20 grade concrete and F415 grade steel are considered for design. The sizes of 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 and evaluation of a six-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
List of 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 Load factors	(DL+EQ)	1.5(DL+EQ)	1.5(DL+EQ)
Design	WS	LS method	LS method

approach	method		

1.3 Estimation of base shear calculation

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

1.4 Analysis of the building

The analysis of the building is carried out by using STADD PRO software package for the three cases. The Fig-1 shows the building frame is carried out for under gravity loads and lateral loads considered in each case is calculated. The values for axial loads and Moments for column members and Bending Moments and Shear force for beam members are calculated respectively are given in Table-2.

1.5 Reinforcement Details

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

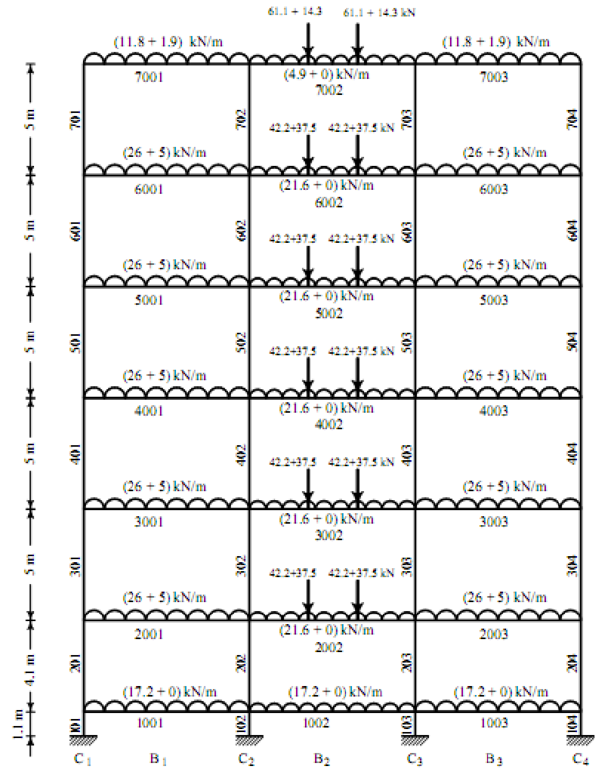

Fig.-1 Gravity Loads: Frame AA

Table-2 AxialForces,B.M and Reinforcement

		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
	Transve	8Φ2L@	8Φ2L@	8Φ5L@2

	rse	200c/c	200c/c	00c/c
C112, C412 SPAN = 4100	Force (kN)	992	1488	1638.4
	Momen t (kNm)	985	273	356
	Section -1	500x500	500x500	500x500
	Longitu dinal	3-25Φ T/B	4 -25Φ T/B	8-25 Φ T/B
	Transve rse	8Φ2L@ 200c/c	8Φ2L@ 200c/c	8Φ5L@ 200c/c
C123, C423 SPAN = 5000	Force (kN)	817.6	1226.4	1347
	Momen t (kNm)	171	256.2	336
	Section -1	500x500	500x500	500x500
	Longitu dinal	3-25Φ T/B	4 -25Φ T/B	4 -25 Φ T/B, 4-22 Φ T/B
	Transve rse	8Φ2L@ 200c/c	8Φ2L@ 200c/c	8Φ5L@ 200c/c
C134, C434 SPAN = 5000	Force (kN)	630	945	1031
	Momen t (kNm)	162.4	244	315.2
	Section -1	500x500	500x500	500x500
	Longitu dinal	3-25Φ T/B	4 -25Φ T/B	4 -25 Φ T/B, 4 -22 Φ T/B
	Transve rse	8Φ2L@ 200c/c	8Φ2L@ 200c/c	8Φ5L@ 200c/c
C145, C445 SPAN = 5000	Force (kN)	445	667	720
	Momen t (kNm)	158	236.3	303.3
	Section -1	500x500	500x500	500x500
	Longitu dinal	3-25Φ T/B	4 -25Φ T/B	4 -25 Φ T/B, 4 -22 Φ T/B
	Transve rse	8Φ2L@ 200c/c	8Φ2L@ 200c/c	8Φ5L@ 200c/c
C156, C456 SPAN = 5000	Force (kN)	266	399	425
	Momen t (kNm)	148	222	279
	Section -1	500x500	500x500	500x500
	Longitu dinal	3-25Φ T/B	3-25Φ T/B	4 -25 Φ T/B, 4 -22 Φ T/B
	Transve rse	8Φ2L@ 200c/c	8Φ2L@ 200c/c	8Φ5L@ 200c/c
C167, C467 SPAN = 5000	Force (kN)	98	147	155
	Momen t (kNm)	110	165	198
	Section -1	500x500	500x500	500x500
	Longitu dinal	3-25Φ T/B	3-25Φ T/B	4 -25 Φ T/B, 4-22 Φ T/B
	Transve rse	8Φ2L@ 200c/c	8Φ2L@ 200c/c	8Φ5L@ 200c/c

Table-3 Forces and Reinforcements

		Case-1 (DL+EQ	Case-2 1.5(DL+	Case-3 1.5(DL+

		) IS:456-1964, 1893-1966 WS	EQ) IS:456-1978, 1893-1984	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, 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

This chapter summarizes the design guidelines and features as per the revisions of IS: 456-1964, 1978 and 2000 and Calculation of design seismic base shear (seismic coefficient method) as per the revisions of IS: 1893-1966, 1984 and 2002 are considered. Apart from that the general analysis and design guidelines, the problem definition and methodology adopted for analysis and design of four three cases studied also presented. The six-Storey office building with different load cases with reinforcement details for column and beam members as per the three cases are also discussed.

2. PUSH OVER OR NON LINEAR STATIC ANALYSIS

2.1 Capacity

The overall capacity of a structure depends upon the strength and deformation capacities of individual members of the structure. In this way to determine the capacities beyond the elastic limits some form of nonlinear analysis is needed. 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 behaviour 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 of vibration is 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 considered for our study.

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 plastic hinge distribution pattern along the height of the building. By increasing the magnitude of the loads, as a result of the weak links and failure modes of the building will generate. 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 or not. For regular buildings, it can also give a rough idea about the global stiffness of the building.

2.4 Procedure 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

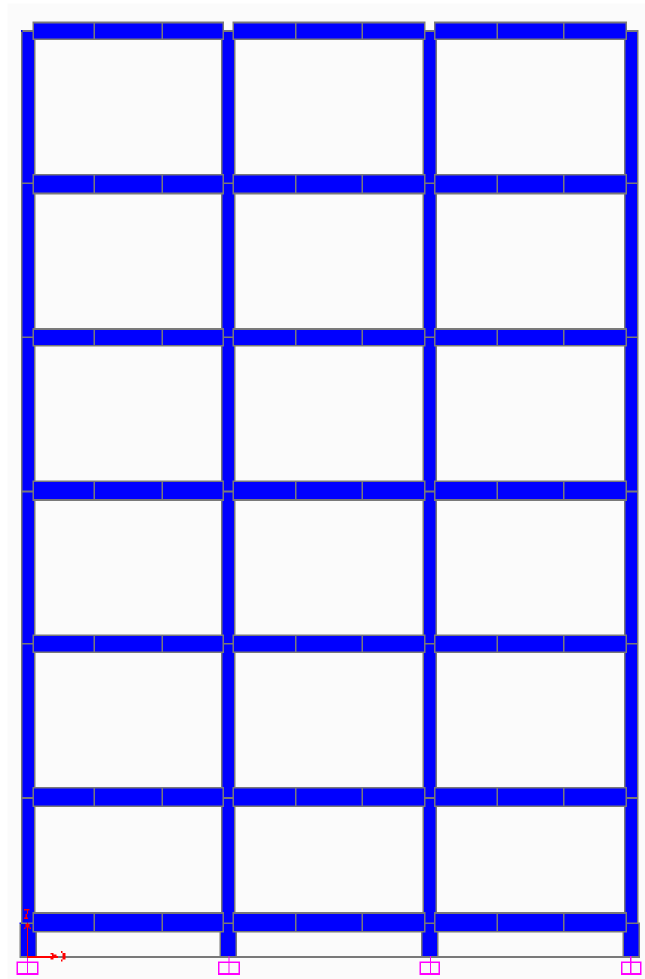

Fig -2 Model of the Building Frame

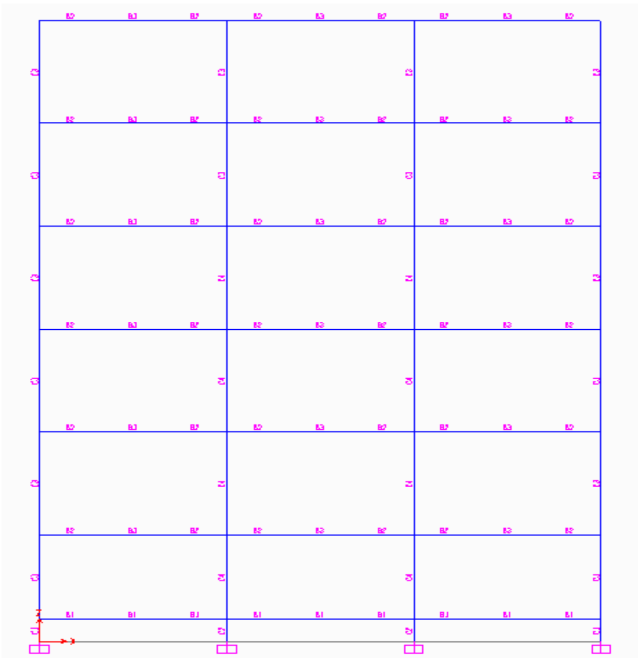

Fig -4 Assigning the Member Sections

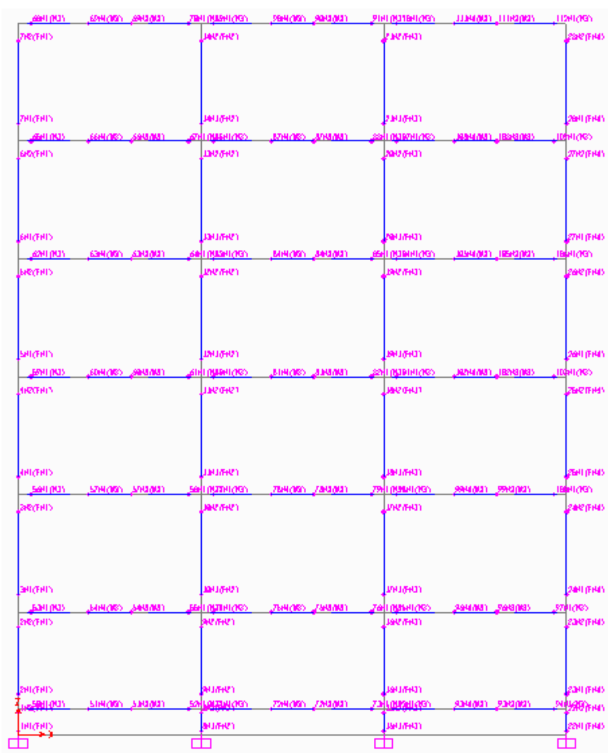

Fig-5 Assigning the plastic hinges

- ✓ 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 Six- Storey Building

The nonlinear static analyses are carried out for the six storey building designed earlier. Considering the symmetry of the building and neglecting torsion effects, the 2D frame model is simulated in SAP2000 software for pushover analysis. The frame is modelled with default PMM hinge properties for columns and M3 hinge properties for beams Members. 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 Six-storey building

Maximum Base shear and Roof displacement		
cases	Base shear (kN)	Roof Displacement (m)
Case-1	897	0.11
Case-2	1095	0.099
Case-3	1334	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	870
Case2	0.030	0.097	0.030	915
Case3	0.030	0.097	0.030	915

Sd : Spectral Displacement, Sa: Spectral Acceleration, g is acceleration due to gravity.

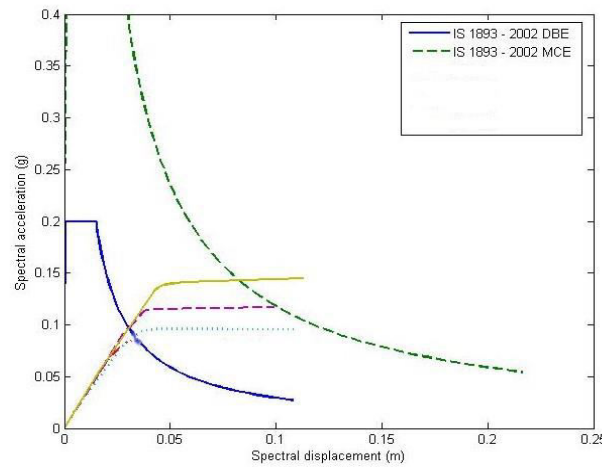

Fig-6 Capacity curve for the three load cases

3. 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.

4. SUMMARY AND CONCLUSIONS

In this study, the evolution of RC design procedure from WSM, to LSM as given in different versions of IS: 456 are discussed. The three typical designs have been carried out as per past and present codes of practice. The nonlinear static analyses are carried out and the capacity curves are found. The variation in maximum base shear and roof displacement capacities for the three 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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