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PERFORMANCE BASED SEISMIC DESIGN OF RC FRAMED BUILDING

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ABSTRACT

Presently, the Indian codes utilize a force based methodology for seismic design of structures. This approach has several shortcomings which has led to the development of alternative design methodologies. The performance based approach aims at developing designs that meet certain performance criteria for a specified hazard level. This paper introduces a displacement based approach based on Indian conditions to obtain earthquake resistant design for RC framed structures. The effectiveness of direct displacement based design (DDBD) with respect to Indian code based design is compared. Three sets of RC frames of 3, 5 and 8 storey are designed according to latest Indian codes and DDBD approach. Pushover analysis of the structure is then carried out to evaluate the performance of each design methodology. It is observed that the base shear calculated by DDBD is significantly lesser than calculated according to IS 1893:2002. The DDBD approach results in a more economical design. Further, as the height of structure increases the force based method becomes more conservative and uneconomical.

Key words: Performance Based Design, Direct Displacement-Based Design, Pushover Analysis, Plastic Hinges, Non-Linear Analysis.

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1. INTRODUCTION

Earthquakes remain one of the most devastating natural disasters that lead to catastrophic losses of lives and property around the world. The past earthquake experiences have shown that the major cause for loss of life and property is the unsafe structures built by man [1]. The earthquake resistant design philosophy follows that the structures should be able to resist minor earthquakes without damage, moderate earthquakes without significant structural damage and major earthquakes without collapse [2, 3].Hence, we need to build structures that are capable of withstanding earthquakes with acceptable damage. The current seismic design codes in India and most of the other countries follow force-based design (FBD) method [4].

The force based design is a traditional approach that has many shortcomings [5]. Firstly, the initial stiffness used to determine the displacement response of structure is incorrect. The ultimate displacement of structure depends upon the nonlinear behavior of the structure where stiffness varies periodically [5]. Secondly, the force reduction factor(R) taken to account for nonlinearbehavior of structure is taken empirically for a group of structures. However, R depends on a number of factors such as geometry, material used, foundation conditions, reinforcement ratio, axial loads, etc. Hence, the value of R is unique and differs for every structure [5]. Moreover, it is widely accepted that damage is more related to strain and displacement rather than strength [6, 7]. Hence, use of strain parameters for design of structures would be a more rational approach.

Due to the fallacies in FBD, it became necessary to develop a more robust design methodology. The concept of performance-based seismic engineering was first initially utilized in the retrofitting of old structures in the USA in early 1990's [8]. However, with passage of time the benefits of this approach over traditional design approach were evidently clear and use of PBSD for the design of new structures started as well. PBSD is more of a design philosophy incorporating a vast number of issues [9] such as life safety, downtime for structures, economic losses, etc. This paper only deals with the structural safety aspect of PBSD. Out of several methods proposed that follow principles of PBSD, the DDBD method has been discussed here. Most of the methods are iterative in nature. The design methodology involves developing a preliminary design which is checked for performance criteria and subsequent modifications are done if performance objectives are not met.

2. INDIAN SEISMIC DESIGN PRACTICE

The earthquake resistant design methodology adopted in the Indian seismic code of IS 1893:2002 follows a traditional force based approach. The fundamental time period of the structure is determined from empirical equations. The seismic force is calculated from acceleration response spectrum based on the fundamental time period. This seismic force is now distributed along the height of the structure [1]. The structure is analysed for different load combinations to obtain the forces and moments in the structural elements. These elements are then designed according to IS 456: 2000 and IS 13920:2016 so that they possess strength greater than required by the demand. At a joint, the concept of strongcolumn weak beam is followed by having sum of moment capacity of columns 1.4 times greater than that of sum of moment capacity of beams [10].

3. DDBD PROCEDURE FOR RC FRAMES

Direct Displacement based Design (DDBD) is a relatively simple non-iterative method for design of structures proposed by Priestley et al. [11]. The aim of DDBD is to obtain a performance level for a given earthquake level. Eq. 1 through Eq. 12, explains the design procedure of DDBB. The performance level is governed by the displacement limits (strain, inter-story drift limits) of the structure. This method of design uses displacement response spectrum, contrary to acceleration response spectrum used in force based design to determine base shear force. The procedure is explained in brief here with the aid of equations 1 to 12. The MDOF structure is idealized into a simpler SDOF structure, whose equivalent properties are found out

Step 1: Determine displacement profile

A deformation level (generally inter-storey drift ratio) is chosen as performance criteria for given level of earthquake [6]. This ratio depends upon the level of earthquake hazard and building material.

The design storey displacement (Δ_i) is calculated using the normalized inelastic mode shape.

$$\Delta_i = \omega_\theta \delta_i \frac{\Delta_c}{\delta_c} \tag{1}$$

$$\omega_\theta = 1.15 - 0.034 H_n \le 1$$

$$\Delta_C = \theta_d H_1$$

$$\delta_i = \frac{H_i}{H_n} \text{ for } n \le 4 \quad ; \qquad \delta_i = \frac{4}{3}\left(\frac{H_i}{H_n}\right)\left(1 - \frac{H_i}{4H_n}\right) \text{ for } n > 4$$

where,

ω_θ is drift reduction factor to compensate for higher mode effects

Δ_c is the design displacement of the critical storey (In most cases the ground storey)

δ_c is the inelastic mode shape of the critical storey

H_i is the height of ith storey

H_n is the total height of the structure

n is the number of stories

θ_d is the inter-storey drift ratio chosen for a give earthquake level

Step 2: Calculate the design displacement (Δ_d)

$$\Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \tag{2}$$

where, m_i is the mass of ith storey and Δ_i is the displacement of ith storey

Step 3: Determine the effective height (H_e)

$$H_e = \frac{\sum_{i=1}^{n} m_i \Delta_i H_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{3}$$

Step 4: Calculate the design displacement ductility factor (μ)

$$\mu = \frac{\Delta_d}{\Delta_y} \tag{4}$$

$$\Delta_y = \frac{\sum_{i=1}^{n} M_i \theta_{yi}}{\sum_{i=1}^{n} M_i} H_e \tag{5}$$

where, Δ_y is the yield displacement of structure and θ_{yi} is yield drift and is give as

$$\theta_{y_i} = 0.5\varepsilon_y \frac{L_{bi}}{h_{bi}} ; \varepsilon_y = 1.1\frac{f_y}{E_s}$$

where ε_y is the yield strain of rebar material,

f_y is the yield strength of rebar and

E_s is the young's modulus for steel

Step 5: Determine the equivalent viscous damping (ξ_{eq})

$$\xi_{eq} = 0.05 + 0.565\left(\frac{\mu - 1}{\mu\Pi}\right) \tag{6}$$

Step 6: Develop displacement response spectrum for given level of seismic hazard

The DDBD approach uses displacement response spectrum in order to calculate the seismic demand. The displacement spectrum is location dependent. The displacement spectra for 5% damping radio is developed from acceleration spectra in IS 1893:2002 using the procedure given in Eurocode 8[12].Figure1 shows the displacement response spectrum for Indian conditions.

$$S_{D,5} = \mathrm{S_{ae}(T)}\frac{T^2}{4\Pi^2} \tag{7}$$

where, $\mathrm{S_{ae}(T)}$ is the elastic acceleration response spectrum for 5% damping which according to IS 1893:2002 is given as:

$$\mathrm{S_{ae}(T)} = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g} * g$$

where, Z is the zone factor

I is the importance factor taken as 1 for residential building

R is the reduction factor taken as 5 for SMRF type

$\frac{S_a}{g}$ is the spectral acceleration coefficient and g is the acceleration due to gravity

The displacement spectrum at 5% damping thus obtained is transformed into design displacement response spectrum at any % of damping using the following relation:

$$S_{D,\xi} = S_{D,5}\left(\frac{0.10}{0.05 + \xi_{eq}}\right)^{0.5} \tag{8}$$

The corner period (T_c) is the time period at which displacement spectrum has maximum displacement and beyond T_c, the displacement remains constant. The corner period can be calculated using expression given by *Faccioli et.al.*[13]

$$T_c = 1+2.5(M_w-5.7) \tag{9}$$

where, M_w is the moment magnitude

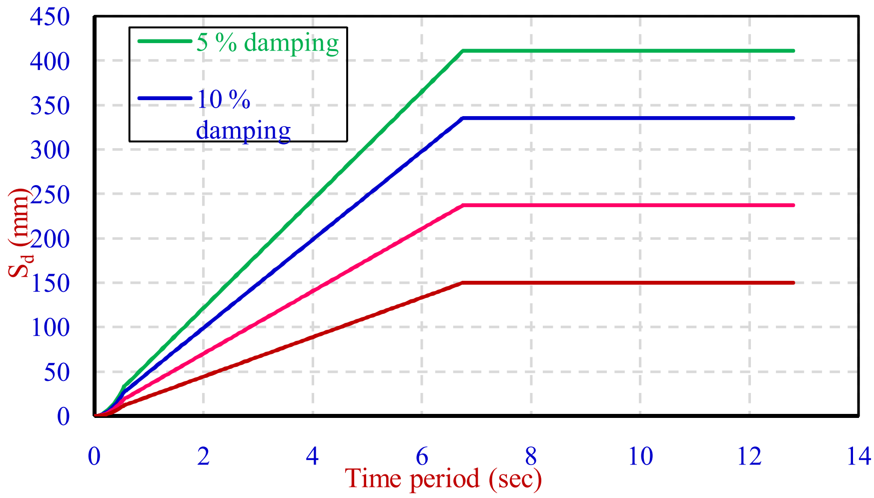


Figure 1 Displacement Spectra corresponding to DBE in zone V of medium soil type IS-1893

Step 7: Calculate the effective stiffness (k_{eff})

$$k_{eff} = \frac{4\Pi^2}{T_{eff}^2} m_e \qquad (10)$$

where, T_{eff} is the effective time period of the structure that is read from the design displacement response spectrum.

Step 8: Determine base shear and distribute it along the height of structure

$$V_{base} = k_{eff} \Delta_d \qquad (11)$$

$$F_i = V_{base} \frac{m_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i} \quad \text{for } n<10 \qquad (12)$$

where, n is number of stories.

Step 9: Analysis of Structures

The structure can be analysed manually based on relative stiffness of based in equilibrium considerations [6]. Here, SAP 2000, a finite element program is used to determine forces and stresses at member level. For design of potentially plastic hinge regions, the gravity and seismic moments cannot be added directly [6], since different stiffness is used for the calculation of two moments. It would reduce the seismic displacement below intended level and result in unnecessarily conservative design. Hence, the plastic hinge locations shall be designed for larger of factored gravity or seismic moments alone. Or if the moments are added directly, a 30% redistribution approach can also be adopted for realistic estimate of moments [6]. The former approach is more conservative and is used here.

Step 10: Design of Structure

The capacity based design principle was first introduced by Park and Paulay [3, 14] and has since been used in progressive design methodologies. DDBD also advocates a capacity based design in order to ensure ductile failure [15]. The beam and columns are designed according to procedures given in IS 456:2000. The capacity based provision for shear demand in beams, columns, and beam column (b/c) moment ratio of 1.4 according to IS 13920:2016 is followed.

4. STRUCTURAL MODELLING AND ANALYSIS

Both linear and nonlinear model of the frame is developed using SAP2000 [16]. The linear model is used to analyse the structure for forces. The forces are used to develop designs according to IS codes and DDBD respectively. The performance of the designed frame is then evaluated using the non-linear model.

4.1. Description of Structure and Loading Conditions

An interior frame, as shown by the shaded part in Figure2of typical RCC building of 3, 5 and 8 storey used for residential purpose is considered here. The different frames used in study here are named in Table 1. The frames have regular bay width of 4m and storey height of 3 m. The column is assumed to be fixed on the ground. In addition to the self-weight, the frame is subjected to various other loads that are listed in Table 2. The structure is of special moment resisting frame type, building situated in zone V on medium soil. The materials used are M25 grade concrete and Fe500 HYSD rebar.

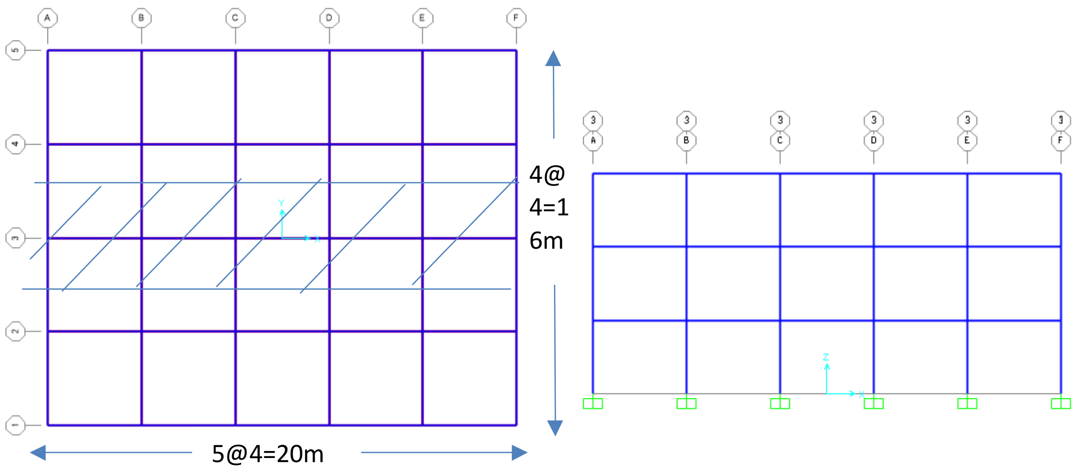


Figure 2 Plan and elevation of building

Table 1 Nomenclature of Building Models

Name	Design Method	No. of storey
IS-3	IS Code	3
DDBD-3	DDBD	3
IS-5	IS Code	5
DDBD-5	DDBD	5
IS-8	IS Code	8
DDBD-8	DDBD	8

Table 2 Loadings Considered

Type of load	Loading Values
1. Dead	
a. Slab	4 kN/m^2 (5” thick)
b. Finish	1.5 kN/m^2
c. Wall	4.5 kN/m for internal walls
2. Live	2 kN/m^2
3. Earthquake	According to IS 1893:2002

4.2. Non-linear Properties

The beam and column are modelled as 2D line elements whose non-linearity is represented by the lumped plasticity at start and end of each frame element [17].Default M3 (primary moment) and PMM (axial load and biaxial moment) hinges are used for beam and column elements respectively [17]. The hinges represent the inelastic deformations in the frame. Due to cracking in hinge regions, the effective stiffness of the frame members is reduced. The effective stiffness is taken as $0.5EI_{eff}$ for rectangular beam and 0.7 EI_{eff} for column [18]. The default hinge properties are calculated by program automatically by using provisions from ASCE 41-13 [19] depending upon material used and section details (sizes and reinforcement).

A typical load deformation graph is shown in Figure3.The Figure 3shows moment rotation capacity of a beam. The points A, B, C, D and E represent various stages in loading [19, 20]. The beam remains elastic in AB with initial stiffness. At B, yielding occurs and BC represent the non-linear phase where the stiffness of the beam gets reduced. IO, LS and CP are the performance levels whose values are 0.2Δ, 0.5Δ and 0.9 Δ respectively where, Δ is the length of deformation plateau [17]. Along CD, there is sudden decrease in the load carrying capacity at same deformation level. After D, the beam continues to deform and eventually loses all of it's strength.

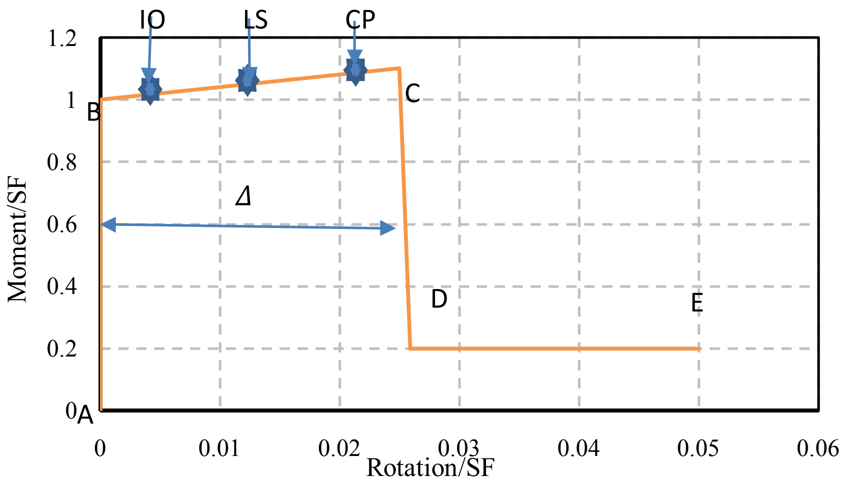


Figure 3 Typical moment rotation

4.3. Assessment of Seismic Performance

The performance of the frames is evaluated using static pushover analysis. Pushover analysis is a tool that can be used to predict the nonlinear behaviour of the structure and find out progressive failure that would occur in case of seismic loads [21]. The structure is subjected to monotonically increasing load (force or deformation) in order to obtain a pushover curve. The pushover curve consists of capacity and demand curve in Acceleration Displacement Response Spectrum (ADRS) format. Several pushover methods exist [22], out of which ATC-40 method is used here. The capacity curve depicts the global strength and deformation capacity of the structure. The demand curve represents the level of seismic hazard that is to be expected. In ATC-40 demand curve is represented by two variables C_a and C_v [18, 20]. The equivalent values of C_a and C_v that would represent seismic demand in zone V for medium soil type is used here. The intersection point of capacity and demand curve is the performance point of the structure. The performance of structure is deemed satisfactory if a performance point exists, and the damage sustained at that point is acceptable [17].

5. RESULTS AND DISCUSSIONS

The seismic demand according to IS 1893:2002 and DDBD approach is calculated respectively as per procedure explained in section 2 and 3. The frames are analysed in SAP2000 for member forces under loads given in Table 2. The designs are then carried out obtained manually. Finally, the performance of the structure is evaluated using pushover analysis.

5.1. DDBD Parameters for Design

The frames in Table1 are idealized into SDOF structure and their equivalent properties are found out using equations given in section 3. Table 3shows the various parameters of DDBD obtained for the frames. With increase in the number of stories, the ductility factor, equivalent damping and effective stiffness of structure decreases.

Table 3 DDBD Parameters

Frame	H_n (m)	Δ_{top} (m)	H_e (m)	M_e (kN)	Δ_d (m)	Δ_y(m)	μ	δ (%)	T_{eff}(sec.)	K_{eff}	V_{base} (kN)
3 storey	9	0.18	6.93	1444.165	0.138	0.108	1.272	0.088	2.66	820.547	113.723
5 storey	15	0.3	10.623	2150.759	0.178	0.167	1.072	0.062	3.1	899.743	161.034
8 storey	24	0.48	16.453	3795.594	0.273	0.258	1.058	0.0599	4.7	690.77	189.07

5.2. Base Shear Comparison

The DDBD method uses secant stiffness which is considerably lesser than initial stiffness used in FBD. Due to lower stiffness, base shear for DDBD is much lesser as depicted in Table4. The percentage reduction in base shear increases with the height of the structure.

Table 4 Base shear values

Frame	V_{base} by IS method	V_{base} by DDBD	% Reduction in V_{base}
3	151.873	113.723	25.12
5	225.54	161.034	28.60
8	409.847	189.07	53.87

5.3. Design Data

The lower seismic demands result in lower member forces. Hence, the frames designed by DDBD method have lighter concrete sections and lesser reinforcement values. Table 5 shows the quantities of construction material that would be saved if DDBD approach is used for design. The percentage reduction in use of concrete and rebar is quite significant for the 8 storey frame.

Table 5 Comparison of reinforcement quantities

Frame	Consumption of concrete (m3)	Difference (%)	Consumption of steel (kg)	Difference (%)
IS-3	10.25	5.46	1711.3	34.40
DDBD-3	9.69		1122.55	
IS-5	18.164	8.00	2975.15	24.91
DDBD-5	16.71		2234.11	
IS-8	43.46	34.62	6213.275	36.70
DDBD-8	28.416		3932.85	

5.4. Capacity Curves and Damage States

Figure4shows the pushover curves for RCC frames corresponding to DBE and MCE level of seismic demand. The frame designed by FBD method is more conservative and the damage inflicted is less than intended by the design philosophy of earthquake resistant structures. Hence, the ductility of the structure is not utilized properly in the FBD method.

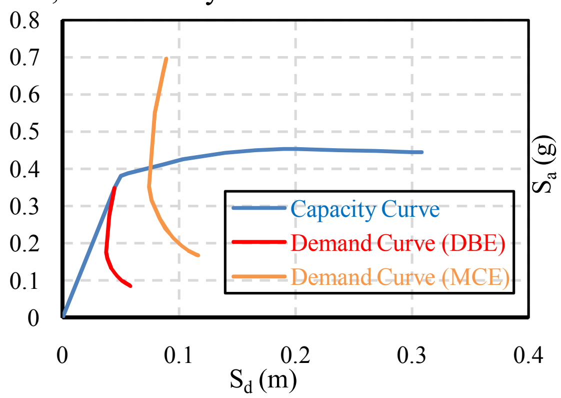


Figure 4 (a) IS-3

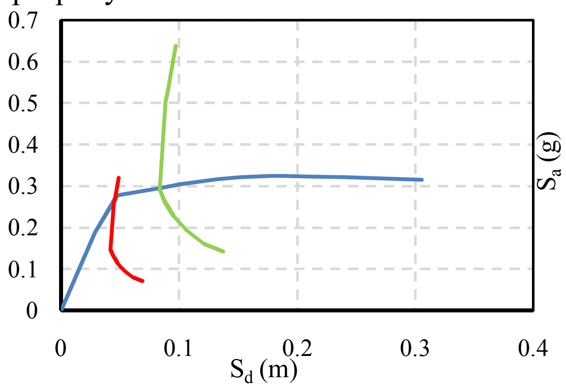


Figure 4 (b) DDBD-3

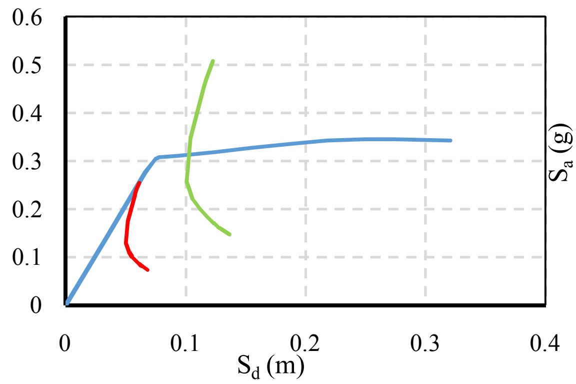


Figure 4 (c) IS-5

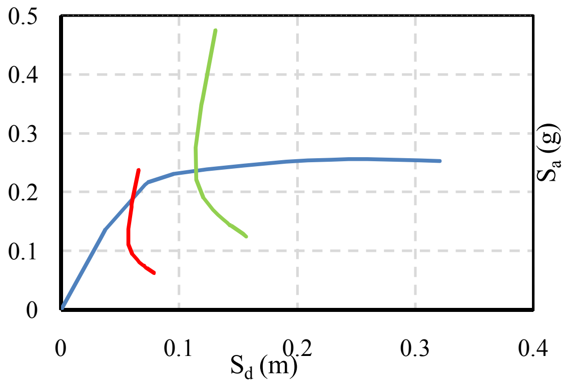


Figure 4 (d) DDBD-5

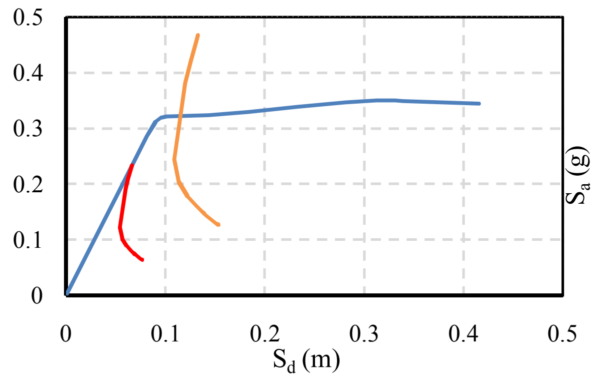


Figure 4 (e) IS-8

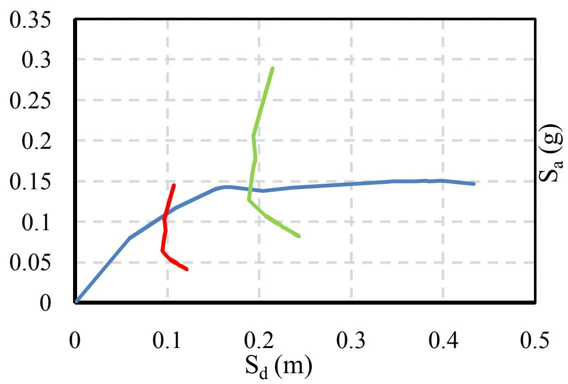


Figure 4 (f) DDBD-8

Figure 4 Pushover Curves

Figure 5 shows the hinge formation patterns in the frames for maximum considered level of earthquake (MCE). We see that hinges are formed mostly in beams and base of ground storey columns. The capacity design principle helps to ensure damage first occurs in beam. The performance of same frame is evaluated for beam column ratio of 1 to 1.5. It is observed that b/c ratio of 1.4 and above performs better i.e.no damage in columns for MCE level of earthquake in zone V. Also the hinge formation start at lower stories and then propagates to the upper stories. The damage is mostly concentrated in the lower stories of beams whereas the beams of upper stories and columns are not damaged.

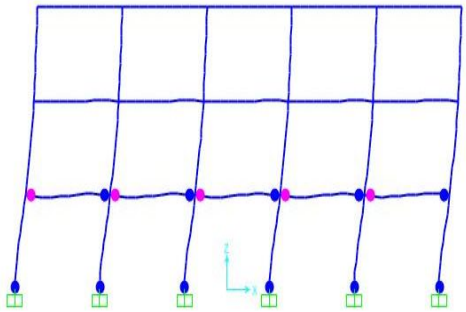


Figure 5 (a) IS-3

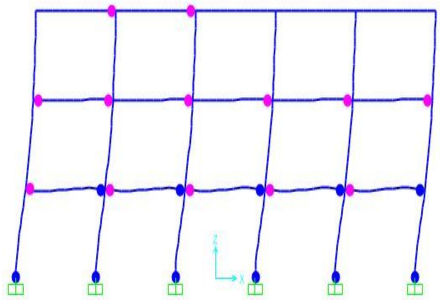


Figure 5 (b) DDBD-3

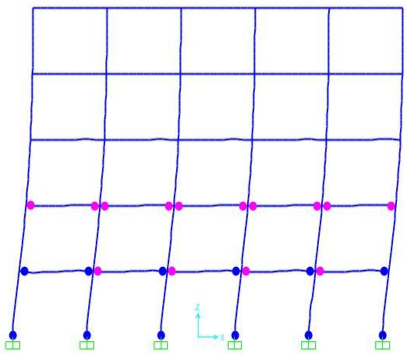


Figure 5 (c) IS-5

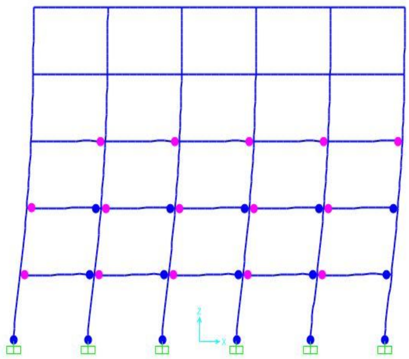
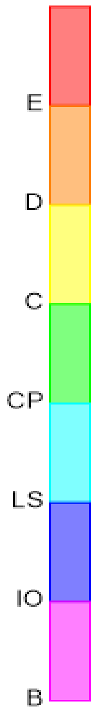


Figure 5 (d) DDBD-5



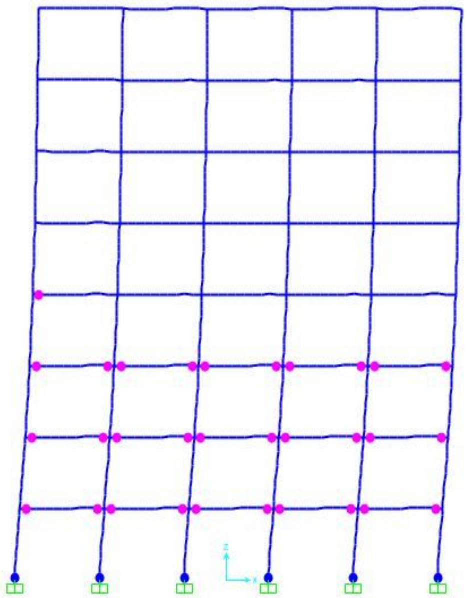


Figure 5 (e) IS-8

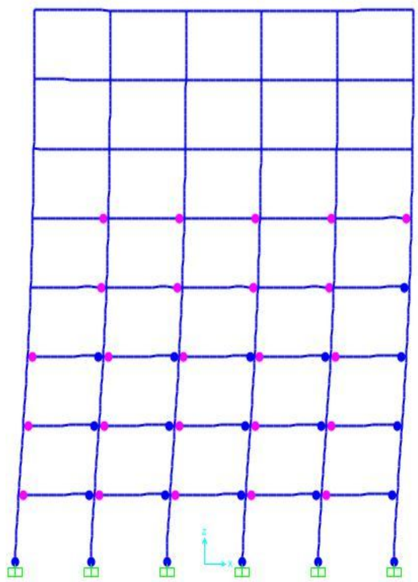


Figure 5 (f) DDBD-8

Figure 5 Hinge formation patterns at MCE level of earthquake

Table 6 shows the performance points for frames at DBE and MCE levels of earthquake. The frames designed by DDBD approach allow more displacement thereby attracting lesser base shear force. The damping forces in frames designed by DDBD are greater as it makes better use of the ductility inherent in the RC frame.

Table 6 Performance points of different frames

Frame	Demand Level	Performance Point				Effective Time Period	Effective Damping
		V (kN)	D(m)	S_a (g)	S_d (m)		
IS-3	DBE	579.597	0.067	0.385	0.053	0.746	0.079
	MCE	622.69	0.094	0.403	0.076	0.866	0.167
DDBD-3	DBE	401.481	0.057	0.268	0.046	0.826	0.079
	MCE	454.821	0.102	0.295	0.084	1.068	0.222
IS-5	DBE	538.851	0.065	0.254	0.061	0.985	0.05
	MCE	687.466	0.111	0.313	0.103	1.146	0.155
DDBD-5	DBE	402.968	0.066	0.191	0.061	1.129	0.085
	MCE	516.528	0.124	0.236	0.114	1.393	0.197
IS-8	DBE	899.134	0.045	0.234	0.066	1.07	0.05
	MCE	1278.281	0.087	0.322	0.116	1.198	0.128
DDBD-8	DBE	391.907	0.072	0.109	0.099	1.901	0.098
	MCE	524.565	0.147	0.14	0.19	2.343	0.202

6. CONCLUSIONS

The paper presents an alternative method for design of RCC framed structures. Both the FBD and DDBD procedures result in design that is safe and has acceptable damage pattern at performance points provided the capacity design principles are followed. The b/c capacity ratio of 1.4 helps ensure no damage in columns for MCE level of earthquake in zone V. The seismic demands and member forces in case of FBD approach is too conservative as the ductility of the frames is under-utilized. Hence, it results in larger sections and higher reinforcement values. Therefore, for achieving same performance level, the DDBD approach would lead to a more economical design. As the height of frame increases, the effectiveness of DDBD approach becomes more evident.

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