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Application of Direct Displacement Based Design of Reinforced Concrete Frames Subjected To Earthquake Loads

Channabasaveshwar Chikmath, Kamatchi P, S A Vasanwala

Abstract: ***Numerous studies are reported in literature on performance evaluation and rehabilitation of building however, limited studies are reported on performance based design of new buildings. Displacement based design procedure is a new method which is not available in Indian building design codes. An effort has been done to investigate the Direct Displacement Based Design (DDBD) for four, eight and twelve story regular RC frame buildings proposed by Priestley et al, using Indian code Response Spectrum for Zone V which is considered as a very high intensity seismic risk zone for life safety and collapse prevention performance levels. Nonlinear time history analysis is carried out for available ground motion and compared with the performance levels (in terms of drift, displacement). Observations show that design displacement reduction factor should be different for life safety and collapse prevention levels. The effective damping increases as the height of the building increases and is higher for collapse prevention.***

Keywords: Direct Displacement Based Design, Design Displacement Spectrum, Performance levels, nonlinear time history analysis, hysteretic damping.

I. INTRODUCTION

Conventional force based design indirectly accounts for the inelastic behavior of building by limiting forces and stresses. To bring in more transparency and to bridge the gap between expected and actual performance of the building, displacement based design is suggested in literature. The Indian codes for earthquake design[1] are based on controlling the forces and stresses in the structural members and finally check the drift for the estimated forces and stresses which is called as Force-Based Design(FBD). This method is not adequate. Since the damage are measured in terms of displacement and strains after the earthquake, the researchers are more interested in developing new methods which consider displacements as a initial criteria, which gave rise to Displacement Based Design [2].Traditional earthquake resistant design of buildings in the Indian code aims to provide minimum amount of lateral resistance to buildings. It was realized in 1970's that not only the strength, the ductility is also important [3].

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In conventional seismic design, a structure is analyzed for equivalent lateral forces and designed as per the load combinations given in reinforced concrete design codes. Semi-empirical expressions were developed at ultimate and yielding by conducting tests on more than thousand reinforced concrete specimens mainly beams, columns and walls which gave good results with the experiments[4]. Many researchers have carried out the comparison of FBD and DDBD[5–7], and proved that DDBD has advantage over FBD in terms of achieving performance goals, material sizes. But real nonlinear behavior of the structure and the failure mechanism has not been explicitly brought in FBD procedure. In the conventional dual design strategy the buildings are designed for life safety for moderate earthquakes and collapse prevention for severe earthquakes. This design philosophy has resulted in severe damage of buildings. Considering the huge time and cost for repair and rehabilitation of damaged structures, different performance levels and associated performance objectives came into picture [7,8]. Plenty of information is available in open source for the performance evaluation of buildings and less work has gone into the performance based design of new buildings.

Studies on the development and use of displacement design spectra and direct displacement based design of steel, reinforced concrete buildings are reported in literature[9,10]. There are many methods available in for displacement based designs but very few can be used to design in codes. Lot of research has been carried out on displacement based design of bridges[11] since it is a single degree of freedom system than any other structures where in it is necessary to convert multi degree of freedom system (MDOF) to equivalent single degree of freedom system (SDOF). The present procedure was developed by Priestly et al[12] for RC structures and applied to Indian context.

To verify the applicability of this method to Indian context, Response spectrum of IS 1893-2016 is converted into Design Displacement Spectra for Zone V, also considering higher modes contribution. To investigate the dynamic behavior of the building, nonlinear time history analysis for Uttarkashi, Bhuj, Northridge and El-Centro earthquake ground motions are carried out.

II. METHODOLOGY FOR DIRECT DISPLACEMENT BASED DESIGN

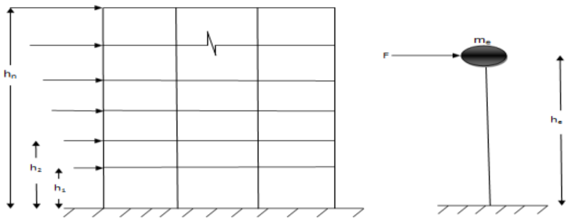
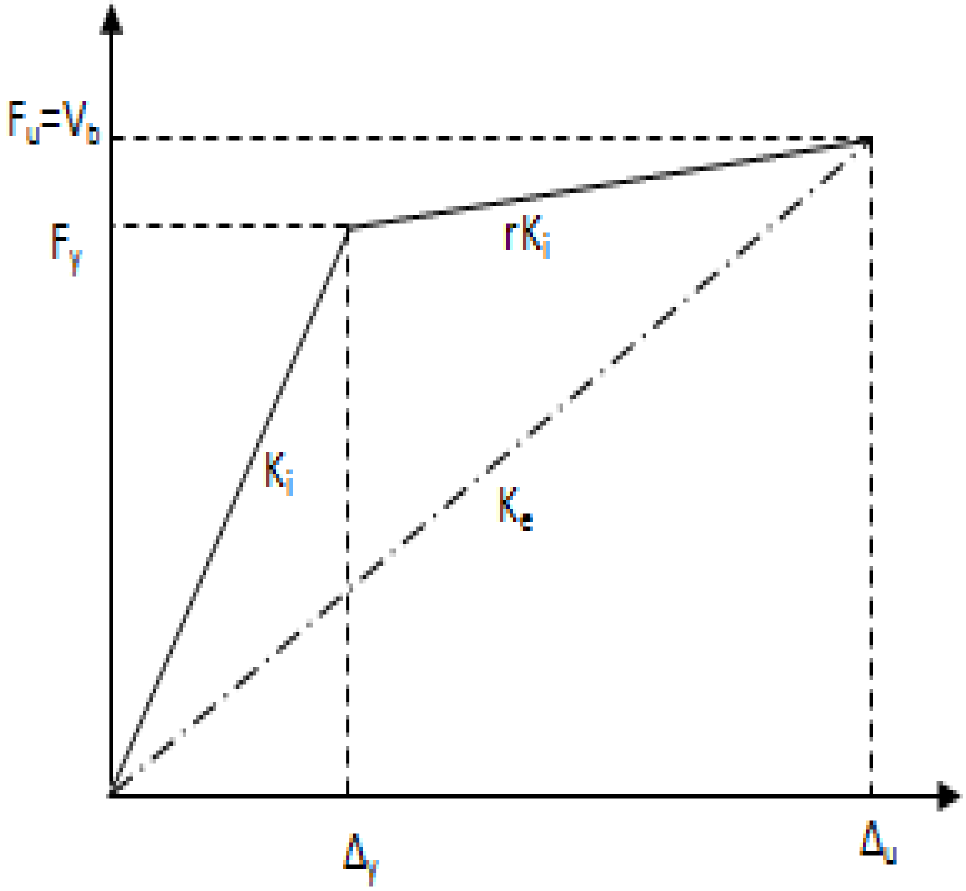


Fig. 1 Simulation of MDOF to SDOF

Fig. 2 Effective stiffness



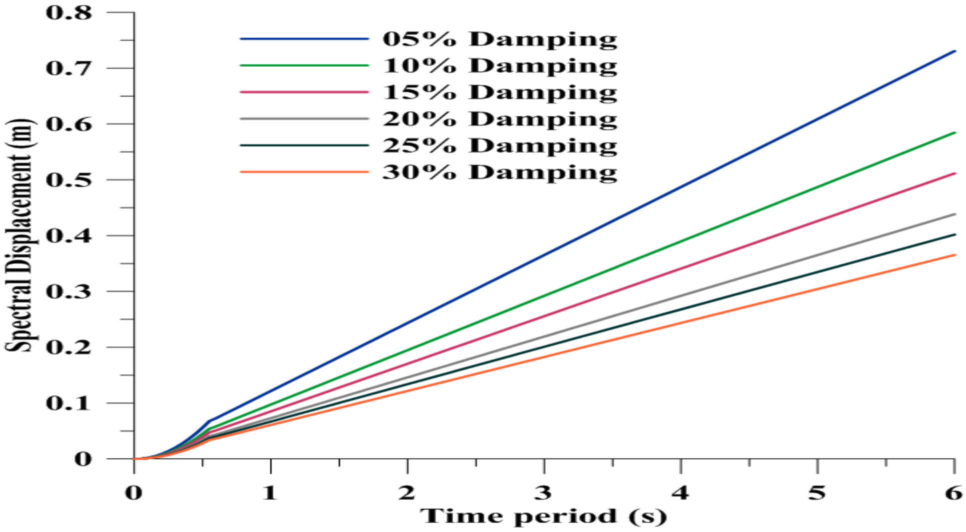


Fig. 3 Design displacement spectrum for IS 1893-2016 for Zone V

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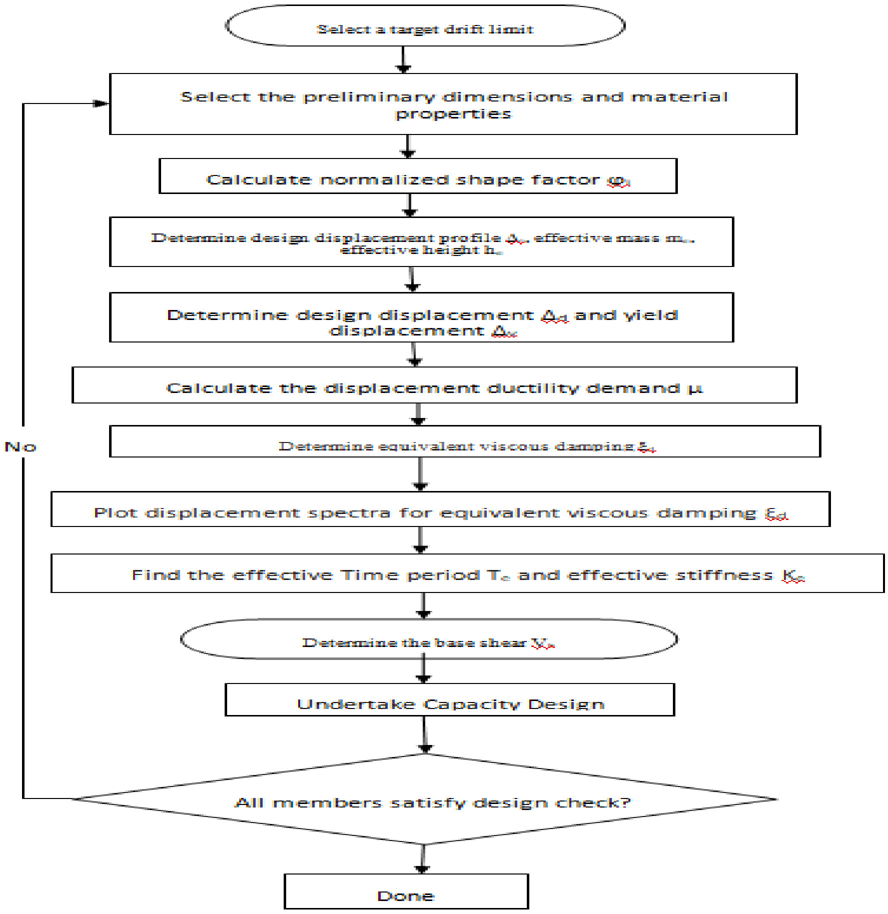

Fig. 4 Flowchart for DDBD

The Direct Displacement Based Design is based on the representing the multi degree of freedom system into an equivalent single degree as shown in Fig. 1[3]. This equivalent single degree of freedom system has secant stiffness related at ultimate displacement shown in Fig. 2, equivalent viscous damping, effective mass and effective height shown in Fig. 3. The steps involved are as follows:

1. Find the first mode normalized shape factor ϕ_i using Eq 1

   For n≤4 $\emptyset_i = \frac{h_i}{h_n}$ (1a)

   For n>4 $\emptyset_i = \frac{4}{3}.\left(\frac{h_i}{h_n}\right).\left(1 - \frac{1}{4}\frac{h_i}{h_n}\right)$ (1b)

   where h_i= height of i^{th} storey from base,
   h_n=total height of the structure
2. The design displacement profile Δ_i for i^{th} storey is developed using the Eq 2 and are related to normalized mode ϕ_i in the form of critical storey displacement

   $\Delta_i = \emptyset_i.\left(\frac{\Delta_c}{\emptyset_c}\right)$ (2)

   Δ_c=displacement of a critical storey which occurs at lower storeys, in this study taken at first storey[13],
   ϕ_c=displacement profile of a critical storey for the assumed drift.
   To consider the effect of higher modes which play significant role in tall structures, the design displacement profile multiplied by reduction factor ω_θ.

   $\Delta_{i,\omega} = \omega_\theta \Delta_i$ (3)

   $\omega_\theta = 1.15 - 0.0034H_n \le 1$ (4)

   ω_θ=Design displacement reduction factor taken as 0.85 as an initial average value in this study as suggested by Priestley[3].
3. Design displacement of equivalent SDOF system Δ_d, effective mass m_e, effective height h_e are given by Eq 5-7 respectively

   $\Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i}$ (5)

   $m_e = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} = \frac{[\sum_{i=1}^{n} m_i \Delta_i]^2}{\sum_{i=1}^{n} m_i \Delta_i^2}$ (6)

   $h_e = \frac{\sum_{i=1}^{n} m_i \Delta_i h_i}{\sum_{i=1}^{n} m_i \Delta_i}$ (7)

   where m_i, h_i are mass and height of i^{th} storey.
4. Find the yield displacement Δ_y followed by design ductility μ from Eq 8-9

   $\Delta_y = 0.5\epsilon_y \frac{l_b}{h_b}$ (8)

   $\mu = \frac{\Delta_d}{\Delta_y}$ (9)

   l_b=beam length and
   h_b=beam depth
5. Hysteretic damping is added to elastic viscous damping to include energy dissipated by RC members during earthquake and is called as equivalent viscous damping[14].

   $\xi_d = (5 + \xi_{hyst})\%$

   $\xi_d = 5 + 120\left(\frac{1-\mu^{-0.5}}{\pi}\right)\%$ (10)

6. The displacement spectra for damping levels other than 5% were calculated using the EC8 expression[15]:

$$\Delta_{(T,\xi)} = \Delta_{(T,5)}\sqrt{\frac{10}{5+\xi_d}} \qquad (11)$$

7. The effective time period is established by entering the displacement spectra set shown in Fig. 3 with the design displacement Δ_d and the equivalent viscous damping Eq 9.
   If the design displacement is greater than the corner displacement, effective time period is given by Eq 11 [16]

$$T_e = \frac{\Delta_d}{\Delta_{(T,\xi)}} T_D \qquad (12)$$

   T_D=corner time period of the spectral displacement.
8. The effective stiffness K_e for the design displacement of the equivalent SDOF system using

$$K_e = 4\pi^2 \frac{m_e}{T_e^2} \qquad (13)$$

9. This effective stiffness is then multiplied by the design displacement, Δ_d, to obtain the design base shear, V_b, as

$$V_b = K_e \Delta_d \qquad (14)$$

10. Distribute the base shear along the height of the building is done according to Indian code IS 1893 [1]

$$Q_i = \left[\frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2}\right] V_b \qquad (15)$$

III. NON-LINEAR TIME HISTORY ANALYSIS

The selected ground motions should be scaled to match the earthquake intensity for that particular site under study. Therefore, the ground motion records considered should be scaled to match the response spectrum for zone-V in the code[1] using SeismoMatch 2018 version. The procedure used was an iterative one[17]. The matched accelerations spectra with acceleration spectrum of Zone V are shown in Fig. 5. Table 1 shows the earthquake ground motions considered for nonlinear time history analysis. Two near field ground motions with one short and long time periods and two far field ground motions with one short and long time periods are considered[18,19].

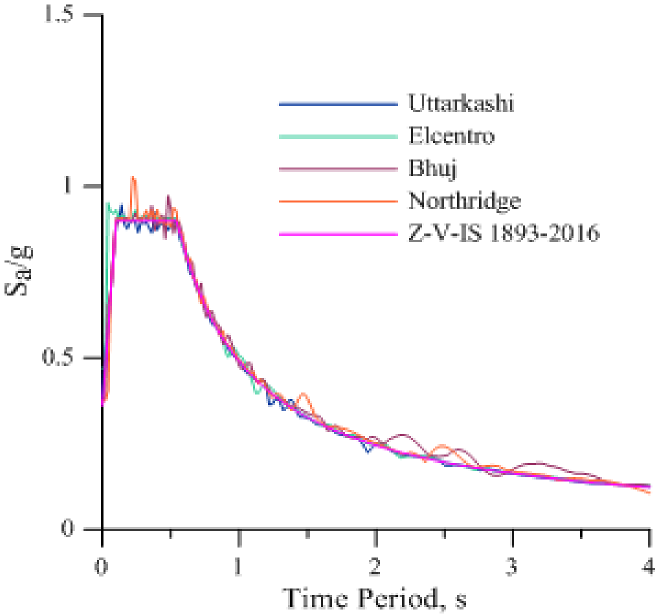

Fig. 5 Matched Acceleration Response Spectra

Table 1 Details of Earthquake ground motions

No	Earthquake Event	Intensity	Year	PGA (g)*	Time duration (s)	
1	Uttarkashi	7	1991	0.385	115.43	Near field
2	El Centro	6.9	1940	0.407	31.02	Far field
3	Bhuj	7	2001	0.368	133.525	Far field
4	Northridge	6.7	1994	0.37	39.88	Near field
						***Peak ground acceleration**

IV. APPLICATION OF THE PROCEDURE

An office building has plan dimensions as shown in Fig. 6 for three different storeys namely-4, 8 and 12[20],[12]. The building is located in medium soil is analyzed with DDBD mentioned in section 2 for Bhuj area which falls under Zone-V according to IS 1893:2016[1]. The building is designed according to Indian design code IS 456:2000[21]. The drift limits are taken as 2% and 4% for two performance levels Life Safety and Collapse Prevention respectively[7,10]. Nonlinear time history analysis is carried out to verify the percentage drift ratios for earthquake ground motions of Uttarkashi, Bhuj, Northridge and Elcentro normalized to Zone-V[1]. The analysis and design were carried out through MIDAS/GEN 2019. The plan is regular in both X and Y directions. The description of the office building considered for the project study is as under:

- The building will be used for office, so that there are no walls inside the building. For simplicity in analysis, no balconies are used in the building. Only external wall of 230mm thick exists.
- The live load is 3kN/m^2.
- The floor diaphragms are assumed to be rigid.
- Seismic loads will be measured in the horizontal direction and not in the vertical direction, since it is not considered to be large.
- Slab is of 150mm thick.
- Grade of main steel if 500N/mm^2.
- Grade of stirrups is 415 N/mm^2.
- Secondary beams are of 300x450mm in dimension.
- The member sizes for Life safety and Collapse prevention for 4, 8, 12 story buildings are shown in Table 2 and Table 3.

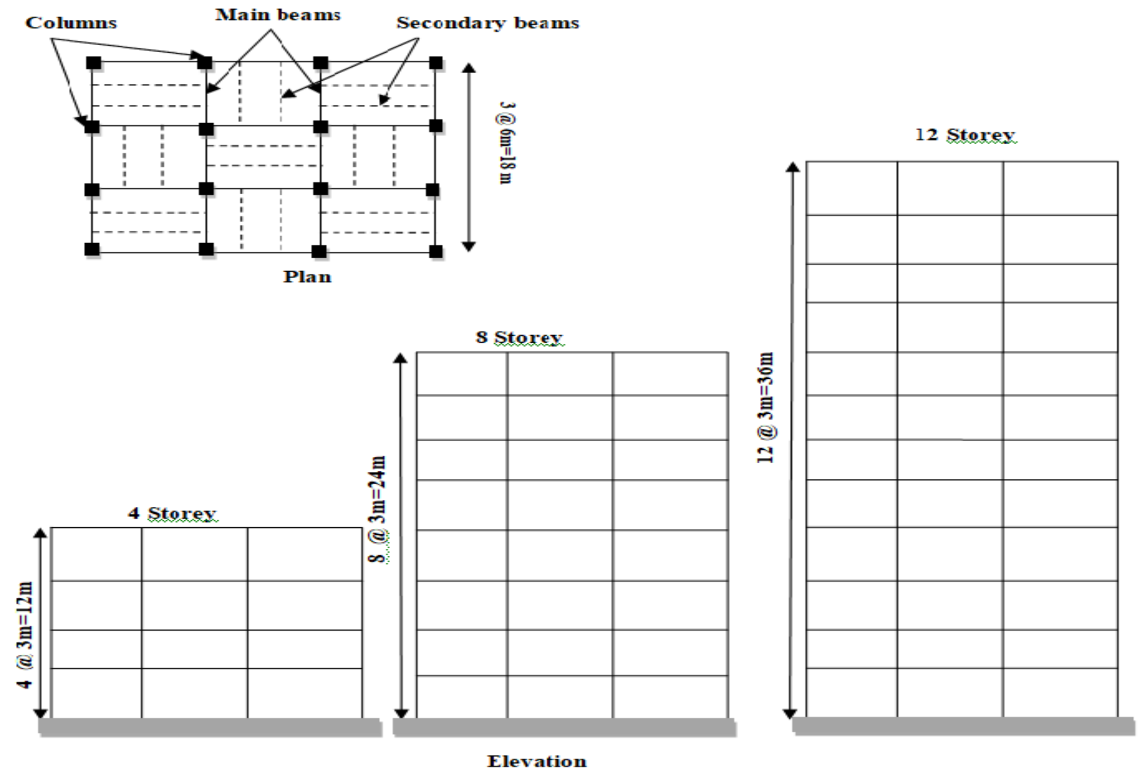

Fig. 6 Geometry of the buildings under study

Table 2 Properties of frame building for Life Safety

	Member	Floor	Width (mm)	Depth (mm)	Grade of concrete cube (N/mm2)
	Beam		350	450	20
	Column	2-Jan	900	900	25
	Beam		350	450	20
4-Storey	Column	4-Mar	700	700	25
	Beam		425	600	25
	Column	1	1200	1200	30
	Beam		425	600	25
	Column	4-Feb	1100	1100	30
	Beam		425	600	25
8-Storey	Column	8-May	900	900	30
	Beam		425	600	25
	Column	4-Jan	1400	1400	35
	Beam		425	600	25
	Column	8-May	1100	1100	35
	Beam		425	600	25
12-Storey	Column	12-Sep	975	975	35

Table 3 Properties of frame building for Collapse Prevention

	Member	Floor	Width (mm)	Depth (mm)	Grade of concrete cube (N/mm2)
	Beam		350	450	20
4-Storey	Column	4-Jan	450	450	25
	Beam		375	600	25
	Column	4-Jan	700	700	30
	Beam		375	600	25
8-Storey	Column	8-May	550	550	30
	Beam		425	600	25
	Column	4-Jan	900	900	35
	Beam		425	600	25
	Column	8-May	750	750	35
	Beam		425	600	25
12-Storey	Column	12-Sep	700	700	35

V. RESULTS AND DISCUSSIONS

Table 4 Design Parameters for DDBD

Performance Level	Story	ω_θ Eq. 4	Δ_d (m) Eq 5	m_e (T) Eq 6	h_e (m) Eq 7	Δ_y (m) Eq 8	μ Eq 9	ξ_d (%) Eq 10	T_e (s) Eq 12	K_e (kN/m) Eq 13	V_b (kN) Eq 14
Life Safety	4	0.95	0.158	1110	8.72	0.09	1.81	15	1.3	26131	4122
	8	1	0.27	3548	16.2	0.11	2.41	19	2.22	28427	7684
	12	1	0.387	5691	23.29	0.14	2.77	20	3.13	22925	8879
Collapse Prevention	4	0.85	0.256	985	8.79	0.07	3.64	23	2.1	8832	1345
	8	0.85	0.389	2590	16.12	0.1	4.02	24	3.23	9833	3828
	12	0.85	0.569	4545	23.76	0.14	3.99	24	4.29	9758	5557

The design parameters obtained for Direct Displacement Based procedure is tabulated in Table 4.The value of displacement reduction factor ω_θ shown in Table 4 was arrived after number of iterations starting with the initial value of 0.85 so that the displacements due to nonlinear time history analysis are within the displacements obtained from present procedure. The displacement profile matched well for the design displacement at the effective height h_e for all the buildings for both the performance levels and the maximum drift was within the design limits for the present procedure. The displacement profiles for the time history considered are within the design displacement at the effective height for 4, 8 and 12 storey buildings refer Fig. 7 and Fig. 9.In the present study, the nonlinear time history analysis for the ground motions considered, shows that the displacement drifts are approximately within the design drift limits except for 12 storey buildings which was exceeding design drift limit for Elcentro ground motion indicating that Eq. 4 need to be revised in taller buildings in Fig. 10 in this case. The interstorey drift ratio for Life Safety and Collapse Prevention were within the design drift limits except for 12 storey Life Safety case. As seen in Fig. 8 and Fig. 10, the critical storeys are different than the first storey assumed in the this procedure.

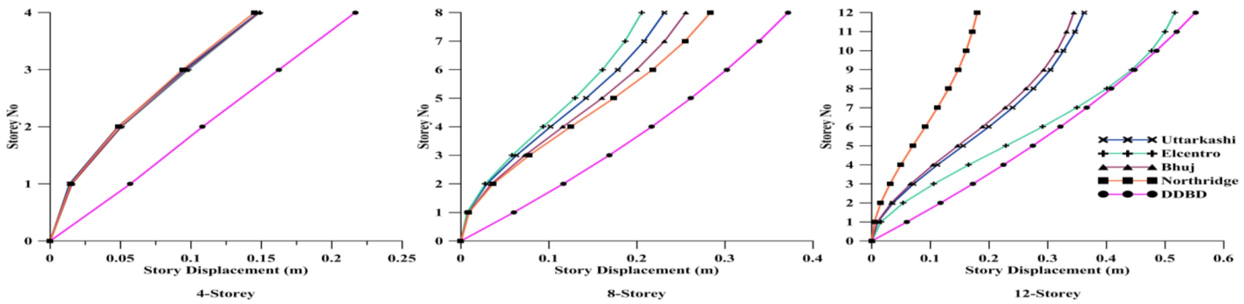

Fig. 7 Displacement envelopes for Life safety

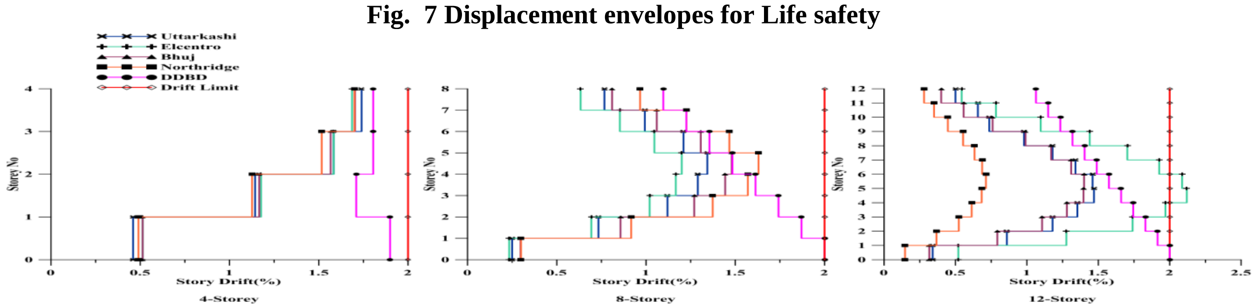

Fig. 8 Inter-storey drift profiles for Life safety

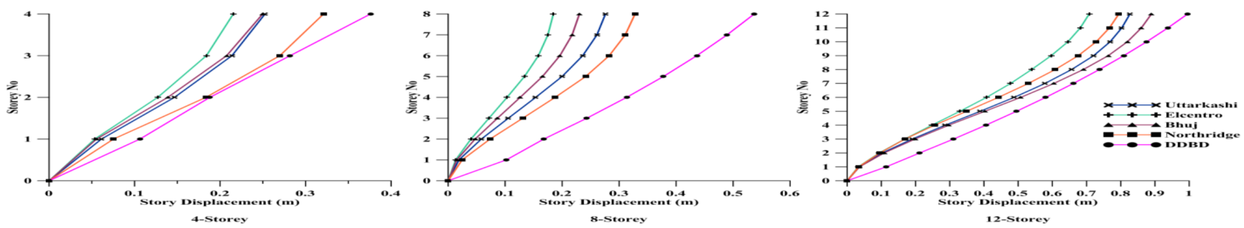

Fig. 9 Displacement envelopes for Collapse prevention

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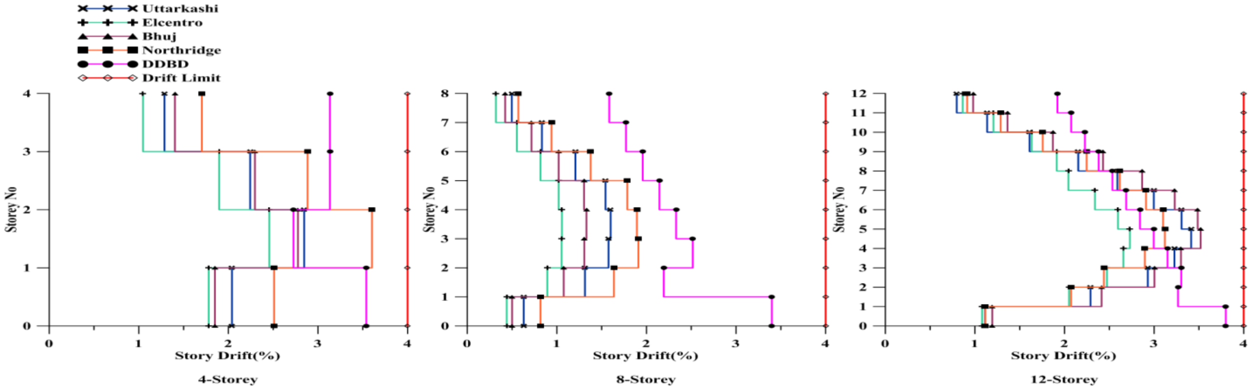

Fig. 10 Inter-storey drift profiles for Collapse prevention

VI. CONCLUSIONS

This paper applies the Direct Displacement Based Design to Indian context in Zone-V considered as a very high intensity seismic region for 4, 8, 12 storey regular reinforced concrete frame buildings for Life safety and Collapse prevention performance levels and the results are compared with inelastic time history analysis. The following conclusions are made from the analysis results:

1. The method proposed has potential to be applied practically for buildings designed according to Indian code which is a force based one.
2. DDBD is easy in implementation and has control over the design objectives.
3. The critical storey in terms of inter-storey drift ratio obtained from nonlinear time history analysis are other than first storey, but the inter-storey drift ratios obtained are less than design drift limits except for 12 storey Life safety case.
4. The effective damping increases as the height of the building increases and is higher for collapse prevention.
5. As the damping increases during the earthquake event, hysteretic damping is also considered in addition to elastic viscous damping in this study.

Even though DDBD is easy in implementation, however, to be specific and flexible for adapting, it is necessary to apply this method for more examples for different configurations and types of buildings for remaining zones prescribed in the Indian code IS 1893:2016. This is a small effort towards adopting DDBD in Indian codes and may require series of modifications in later stages.

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