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Characteristics of FBD and DDBD techniques for SMRF buildings designed for seismic zone-V in India

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Abstract

This paper presents the results of the nonlinear time history analysis of six different reinforced concrete moment frames. The frames were designed using two major seismic design methods as Force Based Design (FBD) and Direct Displacement Based Design (DDBD) in which former is a conventional method while later one is a performance based approach of design. However, in this study, 4, 8 and 12 stories RC framed building with two×three bays were designed according to Indian standard and the Federal Emergency Management Agency (FEMA) considering two design approaches (FBD and DDBD) studied. Analysis and design for this study performed using SAP2000 v15.1 tool. Both design approaches performed using nonlinear time history analysis for five different real recorded ground motions of north-eastern region, India. The parameters such as inter-storey drift, displacement, material strain, and ductility demand obtained from the analyses of the frames designed using both design methods are noted and compared. This study examined, discussed and shown about the applicability of existing methods adopted considering the FBD and DDBD in a cost-effective manner to achieve desired level of performance.

Keywords Force based design · Direct displacement based design · Inter storey drift · Nonlinear time history analysis · Performance based design

1 Introduction

Among all the natural hazards, earthquakes have the most potential for causing the maximum damages. The structures that are designed appear to be strong enough, may collapse during an earthquake and deficiencies are exposed. However, earthquake forces are random in nature and unpredictable, the engineering tools need to be sharpened for analyzing structures under the action of these forces. The major cause for building damage is due to seismic effect caused by shear deformation and eccentric moment additionally generated. As the ground shakes, building loses its stability and gets collapsed. So for any structure, seismic analysis is mandatory as it resists the structure against the seismic forces. In different parts of world, different methods of seismic analysis are practiced. Designing process for making structure seismic resistance has been undergoing a critical reappraisal in recent years, with the emphasis changing from strength to

performance. During the mid of the twentieth century, an era when seismic resistant designing of structures were actually taken into consideration by codes, strength and performance criteria were considered to be parallel and compatible to each other. However, over the past 25 years there has been a gradual shift from this position with the realization that, increasing strength may not actually increase safety, neither necessarily reduce damage [2, 3]. This lead to an approach toward a new design concept called "performance-based seismic design (PBSD)". However, the PBSD is a modern designing concept of seismic resistant structure [1]. Performance-based design is a more general design philosophy in which the design criteria are expressed in terms of achieving stated performance objectives when the structure is subjected to stated levels of seismic hazard [1, 2]. During the last decade, seismic design of structures has matured with revaluation due to the evolution of PBSD methodologies and the encouraging analytical results. Current seismic codes are based on FBD. Current FBD (spectral acceleration-based design) is considerably improved compared with procedures used in earlier years; there are many fundamental problems with the procedure, particularly when applied to reinforced concrete structures. Although the structure is designed to

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yield during the design earthquake, only the elastic part of the response, up to yield, is examined [2]. The analysis is based on the corresponding secant stiffness. To overcome the problems with FBD, a newly design concept has been developed known as DDBD which promise a more rational design philosophy compared to the conventional FBD. Here are some reviews by different authors regarding FBD and DDBD given below [1–3].

The PBD concept that considers rotation ductility factors, pre-defined target damage indices and beam side sway mechanism as the key performance objectives to estimate curvature ductility demands. A 12 story RC moment resisting frame was chosen to observe the performance in terms of damage considering P-Delta effect. Pushover analysis using SAP2000 carried out for the frame to verify the results of the ductility based damage index method. However, the DDBD of single moment resisting frame dual system and dual steel-braced systems considered to investigate damage through lateral drift profile. Some deviations, especially in tall models, from design values are mainly due to the complex and highly varying nature of frequency content of near-fault records. Another important finding of the study is that, the DDBD methodology is able to design structures with quite controlled residual behavior. Assessment of reinforced concrete building frame structures designed according to the DDBD. A comparison of frames characterized by a same overall geometry (number of storey, bay length and storey height). It is seen that requirement for anchorage of reinforcement for beams lead to an over dimensioned structures, influencing the final performance of the designed structures that comply this rule. It was observed that designed structures according to FBD imply larger sections to fulfill both, the capacity design rules and the limit values of reinforcement ratio [4, 5]. In this context a multi storey frame shear wall buildings were designed in DDBD method considering elastic properties such as initial stiffness, strength and period to check the performance [6]. Some of the more controversial issues identified in a book. Further, DDBD is based on sound engineering principles. The approach has also been developed as an assessment tool for existing structures to complement the design approach. The state of development of the approach is applicable on MDOF systems, torsional response, irregularity of structural layout, $P - \Delta$ effects, and a wide range of different structural types including walls, frames, dual systems, bridges and seismic isolated structures [7]. A steel frame-RC wall structures designed by DDBD method has been proposed. For the seismic intensity and modeling, assumptions considered in this work, it is found that the proposed design methodology controls the lateral displacements of the buildings well. For the ground motion intensity and modeling assumptions considered in this work, the design methodology effectively controlled the deformations and therefore likely damage found of the case study

buildings [8]. Similarly, a full-scaled wood frame building was designed according to DDBD method to examine the progress, current state, and challenges for PBSD of mid-rise wood frame buildings. This approach relies on the simplified DDD (Direct Displacement Design) procedure for shear wall selection and a combination of software and basic statics for shear transfer and uplift control. The largest interstory drift was 3.8% at one corner of one story, which was less than the 4% limit [9]. Five different case studies on seismic design was performed to find out the use of eight different displacement based design (DBD) methods. Some significant limitations with the eight methods have been identified through their application to realistic design examples. The performance assessment indicates that each of the eight DBD methods provide designs, that ensure limit states are not exceeded. These limitations could be considered as minor in some instances and rather major in others. However, it is also considered that all of these limitations can easily be overcome now that they have been identified. [10]. Seismic vulnerability of a real RC irregular hospital buildings was assessed in terms of torsional response. A proposed pushover procedure accounts for mass distribution, higher modes contribution and mode-shapes correlation. Furthermore, due to its non-iterative feature, it avoids problems of non-convergence and multiple solutions of the conventional capacity spectrum method [11]. A DBD methodology for structures that are comprised of both frames and walls has been proposed. The various steps required representing a frame-wall structure as a SDOF system identified and a DDBD process then adopted to set the required strength level. To test the design methodology, two sets of 4, 8, 12, 16 and 20-storey reinforced concrete structures are designed, with different proportions of the total strength assigned to the frames [12]. Likewise, a RC frame structure was designed as per Euro code-8. Furthermore, results obtained using DDBD method compared to the result obtained using multimodal response spectrum method. Among other things, significant differences highlighted in regard to current design regulations. Unlike the force based method, in DDBD the limit states are not checked, rather those used as an input data. Strength is not the key parameter in seismic design with this in mind; it is shown that the DDBD becomes more straightforward and economical than the FBD in regards to highlighting one of the key aspects in seismic design: the weak beam-strong column principle [13]. A comparison between different seismic design procedures for steel moment resisting frames is presented. A set of steel moment resisting structures designed according to Euro code 8 (EC8) and DDBD to compare the results [14]. A 15 storey RC framed building was designed by both FBD and DDBD methods located in zone-V in India. The base shear calculated by both the methods compared that defined as per IS code 1893-2002 and reinforcement percentage for beams and columns are shown. It is observed

that base shear for frame designed by DDBD becomes less than that of frame design by FBD and inter-storey drift (IDR) evaluated to assess whether the target drift demand achieved [15]. On the other hand, it has been observed that DDBD becomes more suitable for moment resisting frame type buildings with number of storey more than 8 subjected to ground accelerations exceeding 0.5 g [16–18].

The objective of this paper is to obtain the target drift from both DDBD and FBD methods for different structural section and final section obtained after simultaneous iteration. Furthermore, to compare important design parameters such as IDR, displacements, material strains and ductility demand obtained from the nonlinear time history analysis (NLTHA) of RC frames designed using DDBD and FBD. Reinforced concrete frame buildings of 4, 6, 8, and 12 storey with two and three bays were designed using both methods. However, IS 456 [19] and IS 1893 2002 [20] codes were used to design the frames using the force based approach.

1.1 Force based design

The Force Based Design is based on calculating the base shear force resulting from the earthquake dynamic motion using the acceleration response spectrum and the expected elastic period of the building. In this procedure the static loads are applied on a structure with magnitudes and directions that closely approximate the effects of dynamic loading caused by earthquakes. Concentrated lateral forces due to dynamic loading tend to occur at each floor in buildings, where concentration of mass exists. It also tends to follow the fundamental mode shape of the building where it is larger at higher elevations in structure. Thus, the greatest lateral displacements and the largest lateral forces often occur at the top level of a structure. These effects are modeled in equivalent static lateral force procedures of most design codes by placing a force at each storey level in the structure, which is directly proportional with the height.

1.2 Limitation of FBD

The distribution of design forces between different structural elements are estimated based on initial stiffness of the members. The stiffness of a member could be determined only after the design process completed based on the type of forces (P-M-M, P-M, M, P etc.) and the reinforcement details finalized. The basic assumption in the FBD is member stiffness remains constant. Thus $EI = Constant = M/\Phi$, i.e. yield curvature is proportional to Strength of the Member. Hence till the member strength is determined neither its elastic stiffness nor the elastic time period could be determined. This process is iterative as well as demands incorporating nonlinear behavior.

The check for structural displacement could be directly related to damage potential through drift is carried using coarse and unreliable approach at the end of design process. As per IS 1893 2002, the storey drift in any storey due to minimum specified design lateral force, with partial load factor of 1.0, shall not exceed 0.004 times the storey height. For the purpose of displacement requirements only, it is permissible to use seismic force obtained from computed fundamental period (T) of the building without the lower bound limit on design seismic force". Also it has no consensus among code on response reduction factor and uncertainty/non-uniformity in evaluation of ductility of the structure.

1.3 Direct displacement based design (DDBD)

The DDBD method was developed with the aim of providing a greater emphasis on displacement in contrast to conventional FBD by a variety of performance limit states. A structure is designed to achieve a pre-defined level of displacement when subjected to a given level of seismic intensity by selecting appropriate value of drift limit. It calculates base shear corresponding to secant stiffness at effective displacement of an equivalent SDOF system using substitute structure approach. It facilitates the use of elastic displacement spectra at equivalent damping when structure behaves inelastic under the design earthquake. The characterization of the structure using the substitute structure avoids many problems inherent in FBD, where initial stiffness is used to determine an elastic period which is a drawback that is present in most of the building codes.

2 Design procedure for direct based displacement design

A full description of the steps needed for applying the DDBD method in the design of reinforced concrete moment resisting frame buildings are as follows:

(i) Determine Displacement shapes.

$$\Delta_i = \omega_\theta \theta_d h_i \frac{4H_n - h_i}{4H_n - h_1} \tag{1}$$

where Δ_i = Displacement at level i. ω_θ = Drift reduction factor to include allowance for higher mode amplification of drift by reducing the design floor displacement. H_n, h_i are the total building height and height of floor i.

(ii) Determine the design displacement, Δ_d:

$$\Delta_d = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i} \tag{2}$$

where m_i and Δ_i are Mass and displacement at significant mass location.

(iii) Calculate the effective height.

$$H_e = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i} \tag{3}$$

(iv) Calculate yield drift slope θ_y:

$$\theta_y = 0.5\varepsilon_y \frac{l_b}{h_b} \tag{4}$$

where l_b is the beam span between column centerline. h_b is the overall beam depth. θ_y is the yield strain of flexure reinforcement.

(v) Calculate yield displacement Δ_y:

$$\Delta_y = \theta_y H_e \tag{5}$$

(vi) Calculate the displacement ductility, μ:

$$\mu = \Delta_d / \Delta_y \tag{6}$$

(vii) Estimate the equivalent viscous damping:

$$\xi = 0.05 + 0.565(\mu - 1)/\mu\pi \tag{7}$$

(viii) Plot the displacement response spectrum for ($\xi = \xi_d$): The displacement spectra for other different levels of ξ than 5% can be found from

$$SD, \xi = SD, 5\%(10/(5 + \xi)0.5 \tag{8}$$

(ix) Calculate the effective period, T_e:
The effective period could be obtained from the displacement response spectrum (using the design displacement calculated from Eq. 2).

(x) Calculate the effective mass, m_e (mass of the substitute structure):

$$m_e = \left(\sum m_i \Delta_i\right)^2 \bigg/ \left(\sum m_i \Delta_i^2\right) \tag{9}$$

(xi) Calculate the effective stiffness of the building:

$$K_e = 4\pi^2 m_e / T_e^2 \tag{10}$$

(xii) Calculate the design base shear force:

$$V = K_e \Delta_d \tag{11}$$

(xiii) Distribute the base shear force at different levels of the building using the following equation:

$$F_i = V \, m_i h_i \bigg/ \sum m_i h_h \tag{12}$$

Calculate the displacement then the forces are distributed at each floor level. The DDBD method [21, 22] is illustrated in Fig. 1.

3 Application of FBD and DDBD methods to RC frames buildings

The FBD and DDBD methods are illustrated with the design of several RC frames with 4, 8 and 12 stories for two bays and three bays in each three cases. The response spectrum analysis is used in FBD method as described in IS 1893 2002 to design the same RC frames. These frames were designed to deform according to the beam–sway mechanism, in which flexural plastic hinges at the ends of the beams and at the column base of the first floor are expected. All RC sections have distributed reinforcing steel around the perimeter. The target drift used for designing a RC frame of ($\theta = 2\%$) as Life Safety category as per given in FEMA 356 and 445 [23, 24]. Therefore, now by taking rebar as Fe-415, the base shear is calculated using DDBD method and the displacement obtained by using SAP2000 v15.1 software [25]. The buildings are designed by taking seismic zone-V and soil sites as a medium soil given in IS 1893 (Part 1) 2002 code. The displacement for the force based designed frames and the sections were modified to satisfy the allowable drift limit of 2%. Finally after simultaneous iteration the dimension of beams and columns finalized for both the methods. The dead loads and live loads are taken as per IS 875 code [26] for 4, 8 and 12 stories of bay 2 and 3 RC frame building shown on Table 2.

The DDBD method could be used to account for different performance limit states such as serviceability, damage control and survival, among others. As the purpose of the examples is to compare the results with the buildings designed using FBD, several limits were not considered as part of the initial comparison. In order to compare both methods, the frames were also designed for 2% target drift using the DDBD method.

The three dimensional 4, 8 and 12 storey frames were modeled by assigning the beam and column dimensions. A series of iteration carried out for the structure to get safe and economic section shown in Table 1. Load intensity is given in Table 2.

The various load combinations used by FBD method as in IS 1893 (part-1) 2002 are-

1. 1.5(DL + LL).
2. 1.2(DL + LL ± EL).
3. 1.5(DL ± EL).
4. (0.9DL ± 1.5EL.

Fig. 1 Basic steps for DDBD [21]
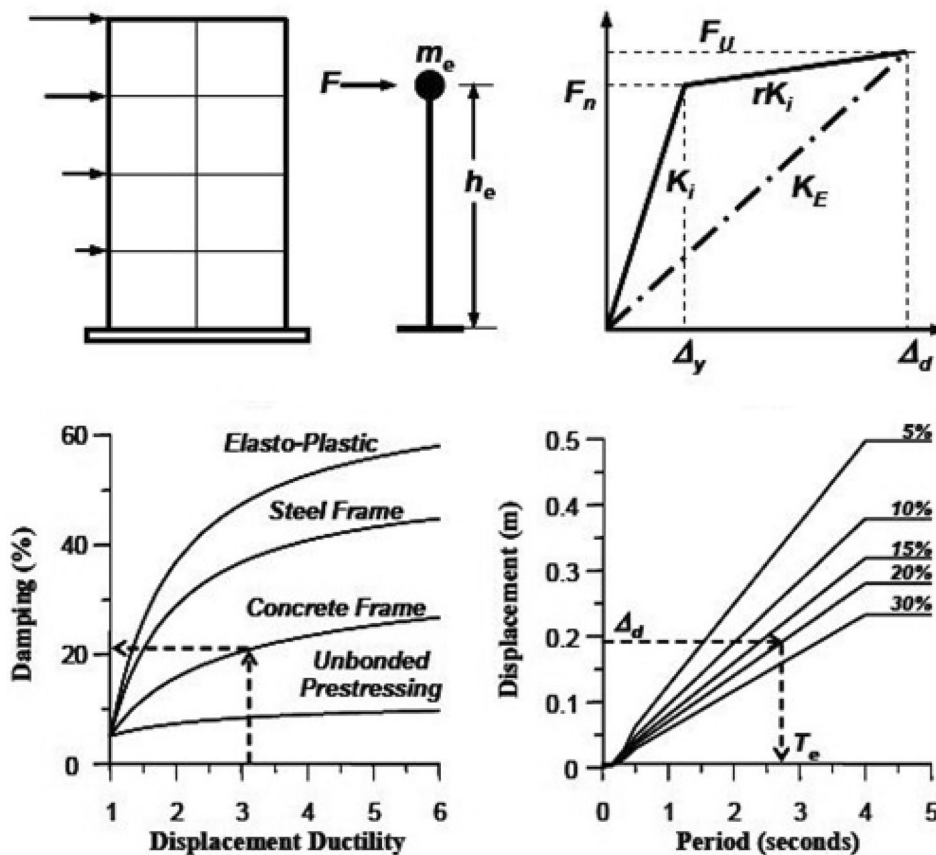
Table 1 Properties of frame building design using DDBD and FBD (revised)

No. stories	No. bays	Floor height (m)	Beam length (m)	Beam section (mm × mm)		Column section (mm × mm)		DDBD target drift 2%	FBD target drift 2%
				DDBD	FBD	DDBD	FBD		
4	2	3.2	5	400×500	450×600	500×500	650 × 650	2	2
4	3	3.2	5	400 × 500	400 × 550	500 × 500	650 × 650	2	2
8	2	3.2	6	400 × 500	450 × 600	500 × 500	600 × 600	2	2
8	3	3.2	6	400 × 500	400 × 550	500 × 500	600 × 600	2	2
12	2	3.2	5.5	450 × 600	600 × 750	600 × 600	850 × 850	2	2
12	3	3.2	5.5	450 × 600	550 × 700	600 × 600	850 × 850	2	2

Table 2 Load values for the frames

Types of loads	Values
Dead Load	Self-Weight
Dead load at floor finish	3 kN/m
Live load at floor level	3 kN/m
Live load at roof	1.5 kN/m
Earthquake load	
Zone	V
Soil type	Medium soil

where DL stands for dead load, LL for imposed load and EL for Earthquake load along the frame.

Similarly for DDBD method the various load combinations are-.

1. DL+LL.
2. DL +LL±EL$_x$.
3. DL+LL±EL$_y$.

In this study, all beams and columns were modeled as frame elements. The beam and column joints were modeled

by giving end-offsets to the frame elements and assumed to be rigid. For slabs due to their in-plane stiffness, "diaphragm" action at each floor level was assigned. The load contribution of slab modeled separately on the supporting beams and the loads were uniformly distributed. The foundation considered as fixed for all the models. All the frame elements modeled with nonlinear properties at the possible yield locations. In Fig. 2, it shows that the FBD and DDBD method have quite similar displacement on fourth storey for bay-2 and bay-3 after certain iteration and the same trend we could see on eight and twelve storey building.

In Fig. 3, it could be clearly seen that the inter-storey drifts are similar for both the methods on fourth storey having similar trend, however, we could see on the eight and twelve storey respectively. The results showing displacement, inter-storey drift profiles for four, eight and twelve storey with two bay and three bay frames shown in Table 3.

Further, in Fig. 3 and Table 3, the displacement and drift values for FBD are approximately similar than that of the

DDBD comparing on their respective storey after certain iterations. But the section considered for FBD becomes higher than that of DDBD method, this shows that the similar in the displacement signifies that the structure becomes more rigid and as their flexibility is less makes the structure to collapse easily. In case of structure with DDBD, shows similar drift values makes the structures flexibility high and these values are achieved nearer to its maximum drift. Also from Fig. 3, the inter-storey drifts are more at the lower storey as the structure is more rigid than that of higher storey. The base shear values of DDBD structures are less compared to FBD structures; this explains that the displacement approach is preferred which results in lower stiffness and attraction of lower floor accelerations than FBD. As in displacement approach design, required base shear is directly proportional to the square of seismic intensity, whereas in FBD, the base shear relates linearly to seismic intensity, so displacement approach base shear values are preferred to that of the FBD approach values.

Fig. 2 Displacement profiles for the FBD and DDBD for **a** Four storey two bay, **b** four storey three bay, **c** eight storey two bay, **d** eight storey three bay, **e** Twelve storey two bay, **f** twelve storey three bay
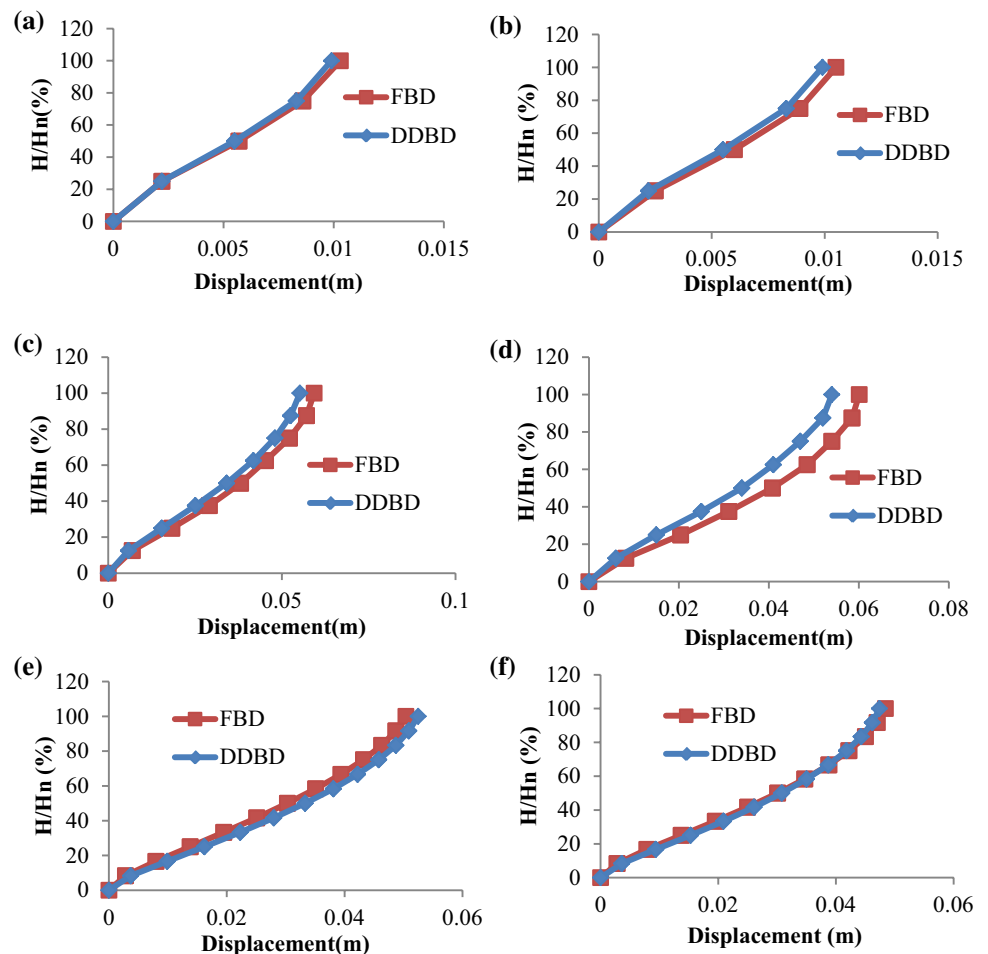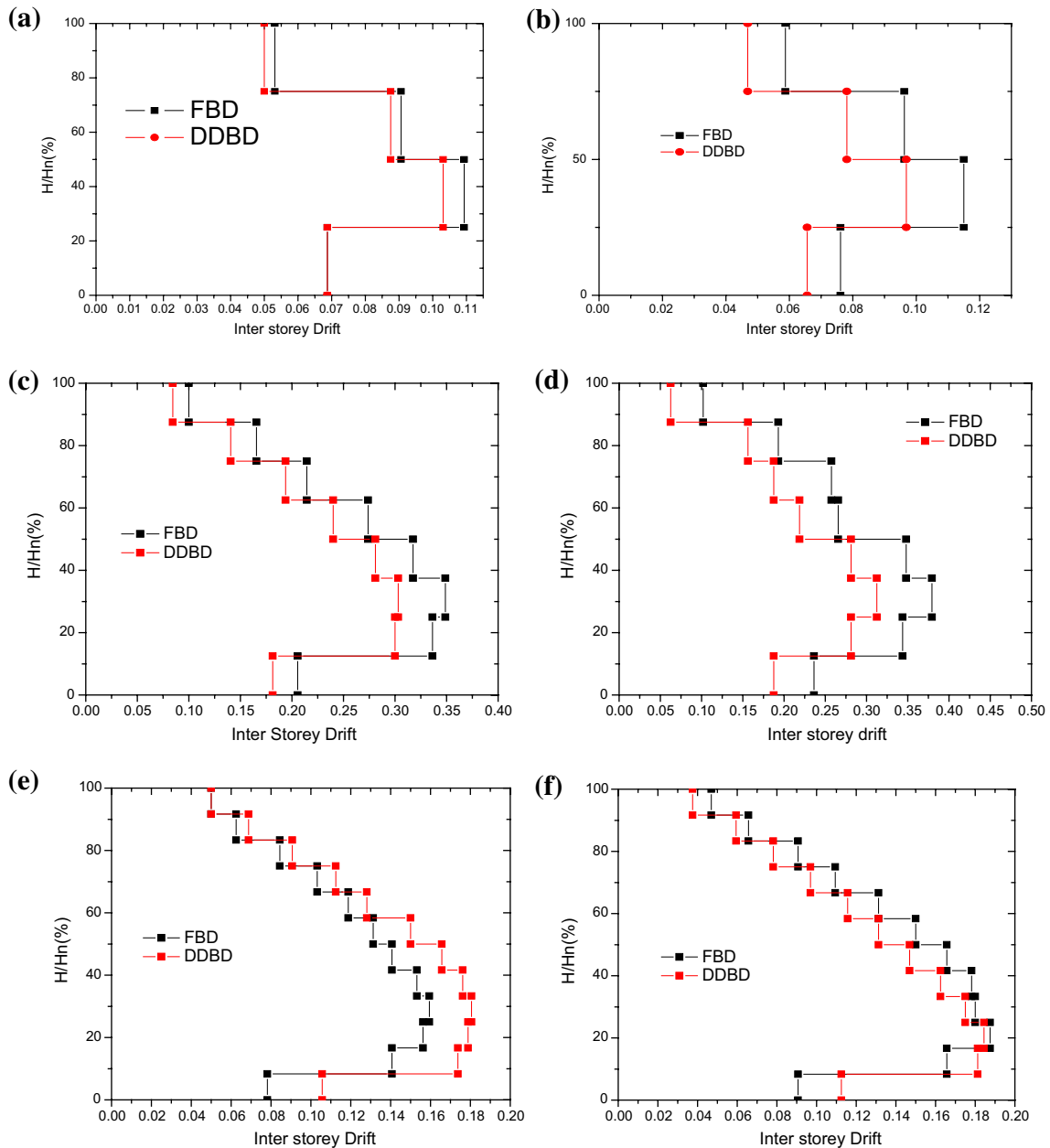
Fig. 3 Inter-storey drift profiles for the FBD and DDBD for, **a** four storey two bay, **b** four storey three bay, **c** eight storey two bay, **d** eight storey three bay, **e** twelve storey two bay, **f** twelve storey three bay

Table 3 Displacement, Inter-storey drift, Base Shear for frames designed using FBD and DDBD methods

Storey bay	Displacement (m)		Inter-storey drift		Base shear	
	FBD	DDBD	FBD	DDBD	FBD	DDBD
4-2 bay	0.010	0.010	0.110	0.105	659	424
8-2 bay	0.0593	0.055	0.345	0.315	1522	980
12-2bay	0.0504	0.0525	0.180	0.162	1918	1289
4-3 bay	0.0105	0.010	0.116	0.100	1178	749
8-3 bay	0.0601	0.054	0.360	0.325	2874	1867
12-3 bay	0.048	0.047	0.190	0.182	3111	2359

4 Nonlinear time history analysis of the RC frames

4.1 Nonlinear time history analysis

Nonlinear Time history analysis is a step by step analysis of dynamic response of a structure subjected to a specified ground motion. Hilber–Hughes–Taylor alpha (HHT) method was used for performing direct integration time history analysis where mass-stiffness proportional Rayleigh damping was considered. The HHT method uses a single parameter (alpha) whose value is bounded by 0 and − 1/3. These coefficients were computed by specifying equivalent fractions of critical modal damping at two different periods. The dynamic input has been given as a ground acceleration time history which was applied uniformly at all points of the base of the structure and only one horizontal component of ground motion has been considered.

4.2 Modeling approach and hinges

The frame models were developed using the program SAP 2000 software. Hinges are failure points, that occur when the plasticity is assumed at critical locations during the inelastic failure of the structure. There are two approaches for specifying the hinge properties:

  i. Distributed plasticity model.
 ii. Lumped plasticity model.

In the distributed plasticity model, the zone of yielding is assumed to be spread over a certain length whereas in the lumped plasticity model, the zone of yielding assumed to be concentrated at critical points in the element. The modeling approach used in this study is based on the lumped plasticity model, as implemented in SAP2000. This approach is favored in FEMA356, which directly refers to plastic hinge rotations rather than local strains or curvatures. Plasticization of the section is assumed to occur suddenly and not gradually. There are three types of hinges namely moment hinges, axial hinges and shear hinges.

The flexural hinges in beams are modeled with uncoupled moment (M3) hinges whereas for column elements the flexural hinges are modeled with coupled P-M2-M3 properties that include the interaction of axial force and bi-axial bending moments at the hinge location. Each plastic hinge is modeled as a discrete point hinge. Therefore, the plastic hinge length is considered while generating the hinge characteristics.

4.3 Ground motions used in the NLTHA

Nonlinear time history analysis (NLTHA) of the frames designed with DDBD and FBD methods were performed using spectrum compatible records. A combined frequency–time domain scaling type was used to match the target spectrum. Scaling of natural ground motions was done consistent with general design spectrum given in Indian Standard IS 1893 (2002) for corresponding zone and damping ratio of 5% for each of frames. The ground motions downloaded from a ground motion database in the website of Cosmos Virtual Data Center. Five earthquakes were used in the analysis shows the name, year, and station of the earthquakes used in the NLTHA. These ground motions selected were shown in Table 4 in order to have a good range of different characteristics including strong ground motion duration, and number of cycles.

All the earthquakes taken are from north-eastern region due to the building site chosen for north-eastern India as it is earthquake prone area with seismic zone-V.

Saitsama station has a focal depth of 43 km and surface wave Magnitude (M_s) of 4.5, Berlongfer station has a focal depth of 90 km and Ms of 7.2, Dauki station has a focal depth of 15 km and Ms of 5.8, Diphu station has a focal depth of 117 km and Moment Magnitude (M_w) of 6.4, Silchar station has a focal depth of 34 km and Ms and Mw of 5.6 and 6.0 respectively.

5 NLTHA results and comparison

5.1 Displacement and inter-storey drift profiles for the FBD and DDBD frames

The results of the nonlinear time history analysis compared in terms of the displacement and inter-storey drift which becomes the maximum drift of the structure. Inter-storey drift is defined as the difference in the displacement values of adjacent storey divided by the storey height. Inter-storey drift calculated from the time history by finding the drift for each time for each storey and then the absolute maximum

Table 4 Earthquakes used in NLTHA

Number	Earthquake name	Year	Station	Peak acceleration m/s^2
Equation 1	NE-India EQ	1986	Saitsama	1.36
Equation 2	NE-India EQ	1988	Berlongfer	− 3.37
Equation 3	NE-India EQ	1988	Dauki	0.379
Equation 4	NE-India EQ	1995	Diphu	1.003
Equation 5	NE-India EQ	1997	Silchar	− 1.4882

value is taken as the inter-storey drift of that particular storey.

$$\text{Inter storey drift}(d) = ((\partial n + 1 - \partial n)/(h)) \qquad (13)$$

where $\partial n + 1$ displacement at $n + 1$ storey. ∂n displacement at n storey. h storey height.

Drift is an important parameter used in both design approaches. The inter-storey drift parameter considered in comparing the results which explains the non-structural damage of the structure as FBD is strength based design approach and base shear becomes the fundamental parameter for the design of structures. The structural damage of a building cannot be evaluated using the above parameter alone; hence we considered drift as a parameter. Damage is directly related to deformation and DDBD considers damage control limit state as that this limit state can be defined by strain and drift limits.

The four, eight and twelve stories frames was modeled as per procedure and nonlinear time history analysis done for the structure and the drift values predicted. Plots have been drawn showing displacement and inter-storey drift of the time history analysis for all the ground motions and geometric mean (GM) of the same taking displacement and drift values in × axis while H/Hn (%) in Y axis. Displacement and inter-storey drift values for four, eight and twelve stories are shown below in figure. However, from Fig. 4, it could be seen that the Berlongfer EQ (Eq. 5) shows maximum displacement and the Dauki EQ (Eq. 3) becomes the minimum displacement. The Eqs. 3 and 5 has maximum and minimum displacement owing to the fact that they have maximum and minimum peak acceleration among the five different earthquakes taken.

So, we have taken average value from those five different earthquakes as in the figure (orange line) to consider average displacement we could expect during earthquake in that region. Moreover, also from the figure it could be seen that the DDBD method shows maximum displacement in respect to FBD method for the four storey frame and the similar trend can be notice on eight and twelve storey respectively. Also it could be notice that with increasing storey, the displacement get increased. The results showing displacement, inter-storey drift profiles for four, eight and

twelve storey of two bay and three bay frames are shown in Tables 4 and 5.

In Figs. 4 and 5 and Tables 5 and 8, it could be explained that drift values gets increased on second storey for four storey structure and third storey for eight and twelve stories, and then gets decreased as the storey height increases, thus the maximum drift occurs at the bottom of the storey (Tables 6, 7). The drift value gets decreased as the storey height increased due to shear deflection of the structure. In Table 8 the displacement and drift values for FBD exhibits lesser than that of the DDBD comparing on their respective storey. This shows that the decrease in the displacement signifies that the structure becomes more rigid and as their flexibility is less makes the structure to collapse easily. In case of structure with DDBD, shows high drift values makes the structures with high flexibility and these values are achieved nearer to its maximum drift. Also in this study we have seen that the dimensions of beam and column of DDBD frames are smaller than that of FBD frames, thus DDBD frames becomes more suitable in respect to collapse prevention, life safety and economical aspects.

5.2 Ductility demand

Ductility is the capacity to undergo large inelastic deformations without significant loss of strength. Reduction in ductility value results in better strength. Ductility demand is calculated from the time history by finding the displacement for each time for each storey and then the absolute maximum value is taken as the ductility ratio with the yield displacement of that particular storey.

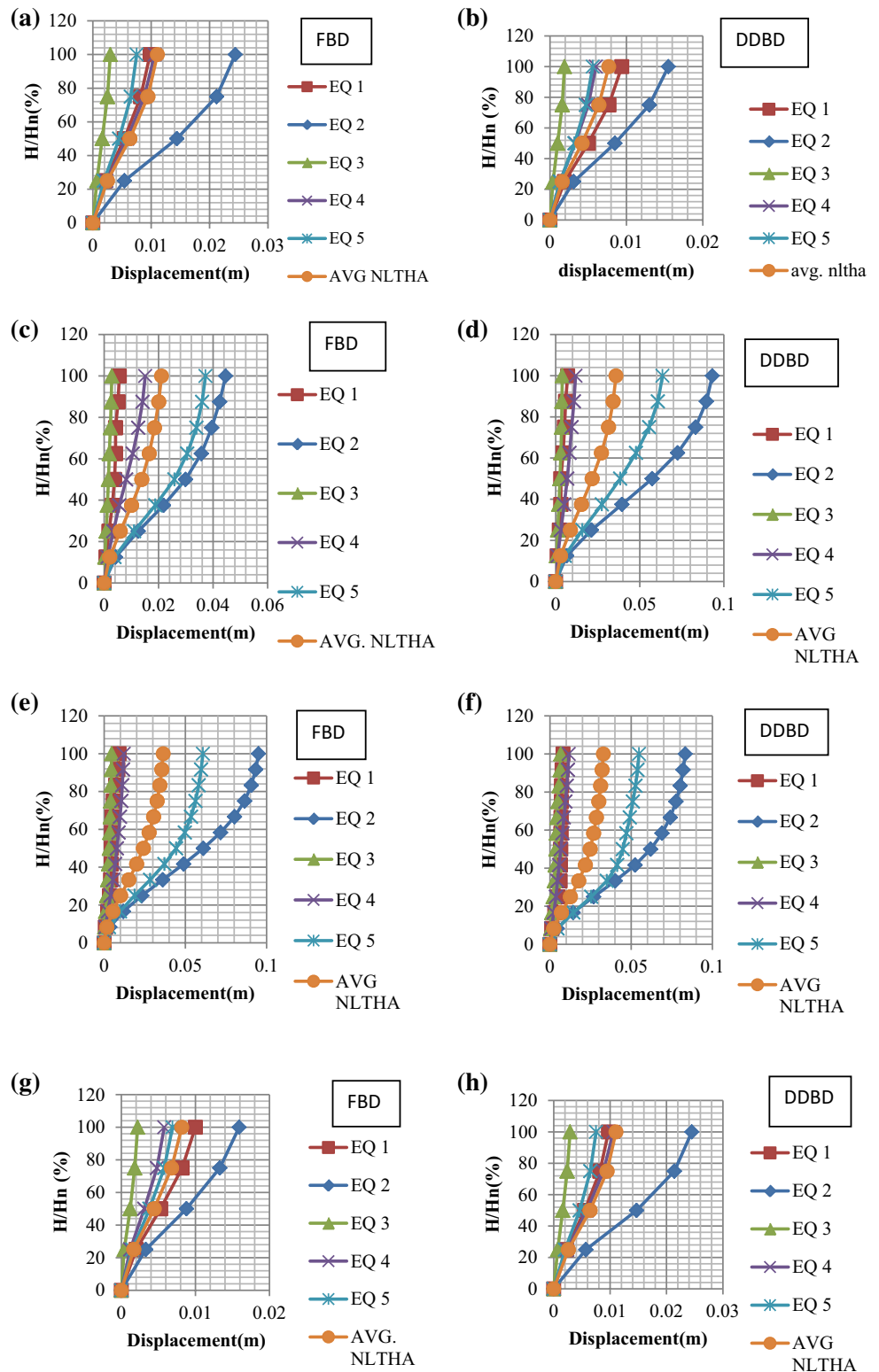$$\text{Ductility demand} = (\Delta m)/(\Delta y) \qquad (14)$$

Δm = maximum displacement. Δy = yield displacement.

The fact that ductility of the structure has major contribution to response reduction factor for the structure which highlights the structural ductility in the seismic analysis process as depending on the level of elasticity, the base condition of structure could be obtained. Drift calculated within adjacent stories alone cannot be validated for seismic design, however, the relationship between all stories displacement must be taken in account as the performance check have to

Table 5 Displacement in m. for frames designed using FBD

| Storey-bay	Displacement (m)					
	EQ-1	EQ-2	EQ-3	EQ-4	EQ-5	Avg
4-2 bay	0.009	0.016	0.002	0.006	0.006	0.008
8-2 bay	0.006	0.045	0.003	0.015	0.037	0.021
12-2bay	0.009	0.095	0.005	0.012	0.061	0.036
4-3 bay	0.010	0.016	0.002	0.006	0.007	0.008
8-3 bay	0.006	0.053	0.003	0.011	0.052	0.025
12-3 bay	0.009	0.097	0.012	0.005	0.066	0.038

Fig. 4 Displacement profiles for **a** FBD four storey 2 bay, **b** DDBD four storey 2 bay, **c** FBD eight storey 2 bay, **d** DDBD eight storey 2 bay, **e** FBD twelve storey 2 bay, **f** DDBD twelve storey 2, **g** FBD four storey 3 bay, **h** DDBD four storey 3 bay, **i** FBD eight storey 3 bay, **j** DDBD eight storey 3 bay, **k** FBD twelve storey 3 bay, **l** DDBD twelve storey 3 bay

be done for whole structure. Hence, checking the margin displacement of each story becomes a necessary one, as ductility demand is chosen as one of the design parameter.

The below graphs explain that the ductility value gets decreased as the storey height increases since the rigidity will be more at the lower storey. Increase in ductility reduces acceleration demand. Plots have been drawn showing

Fig. 4 (continued)
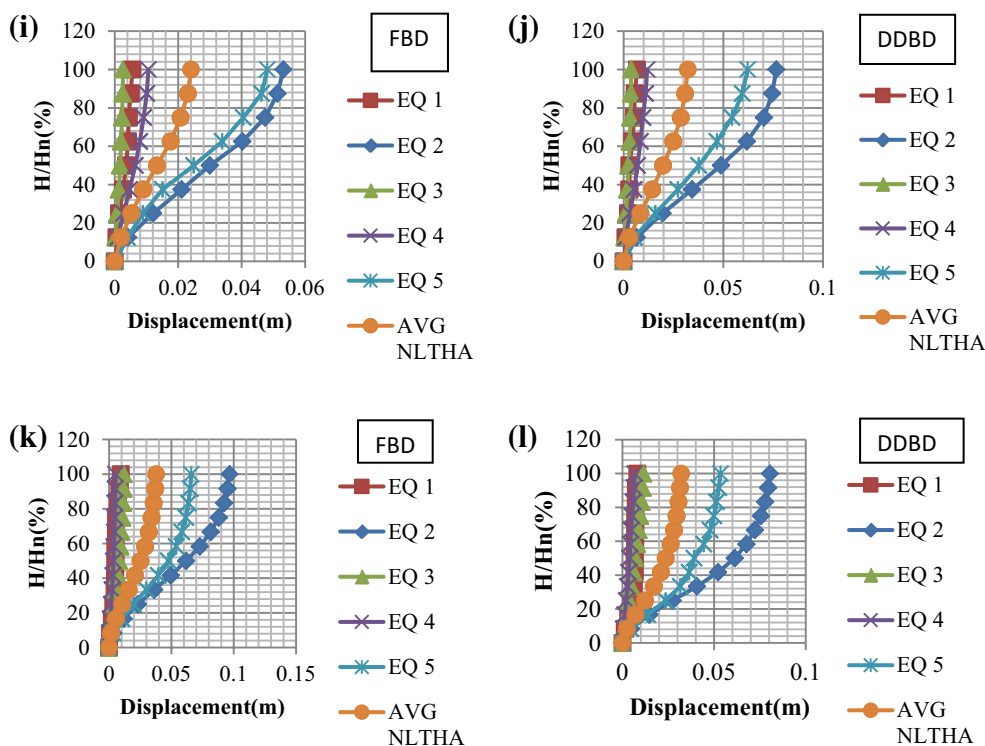
ductility demand of the time history analysis for all the ground motions and GM of the same taking ductility values in Y axis while number of storey in × axis. Ductility demand values for four, eight and twelve stories are shown in Fig. 6.

In Fig. 6 it shows that Berlongfer has maximum value and Dauki has minimum value whereas orange line shows the average value due to Berlongfer has max ground acceleration and Dauki has min. ground acceleration. Here it shows that DDBD has less ductility value than FBD method and trend continues at 8 and 12 stories respectively. From Fig. 6 lower stories have more displacement value then the higher stories; the ductility value gets decreased while the story height increases as the drift will be maximum at the bottom stories.

The Table 9 shows the average value taken from five different earthquakes where DDBD structure shows lesser ductility demand comparing to that of the FBD structures, this explains ductility will be maximum for DDBD structures as it undergoes deflection easily and flexibility will be high (Tables 10, 11).

6 Serviceability limit for DDBD frames

The validity of the DDBD method to design for any limit state is explored in this section. The serviceability limit state based on material strains selected to design the same frames previously discussed. The serviceability concrete compression strain is defined as the strain at which crushing is expected to begin (usually 0.004), while the serviceability

steel tensile strain is defined as the strain (0.015) at which residual crack widths would exceed 1 mm. The serviceability limit is controlled by the strain limit that is reached first. Target drifts according to those values of strain were found using Eqs. (15) and (16) [27]. These equations could be used to directly correlate material strains to inter-storey drifts in RC frame buildings. The steel tensile strain controls the serviceability limit in these examples. Therefore a steel strain of 0.015 was used to compute target drifts in this section. In these equations, θ_{by} is the yield beam drift Eq. (17), L_p is the plastic hinge length Eq. (19), d_{bl} is the assumed diameter of the reinforcing bar, ρ_{long} is the assumed longitudinal reinforcing ratio, L_b is the beam length, H_b is the beam depth, ε_s is the steel strain, ε_c is the concrete strain, concrete compression strength (f_c'), and F_y and F_u are the yield and ultimate strength, respectively.
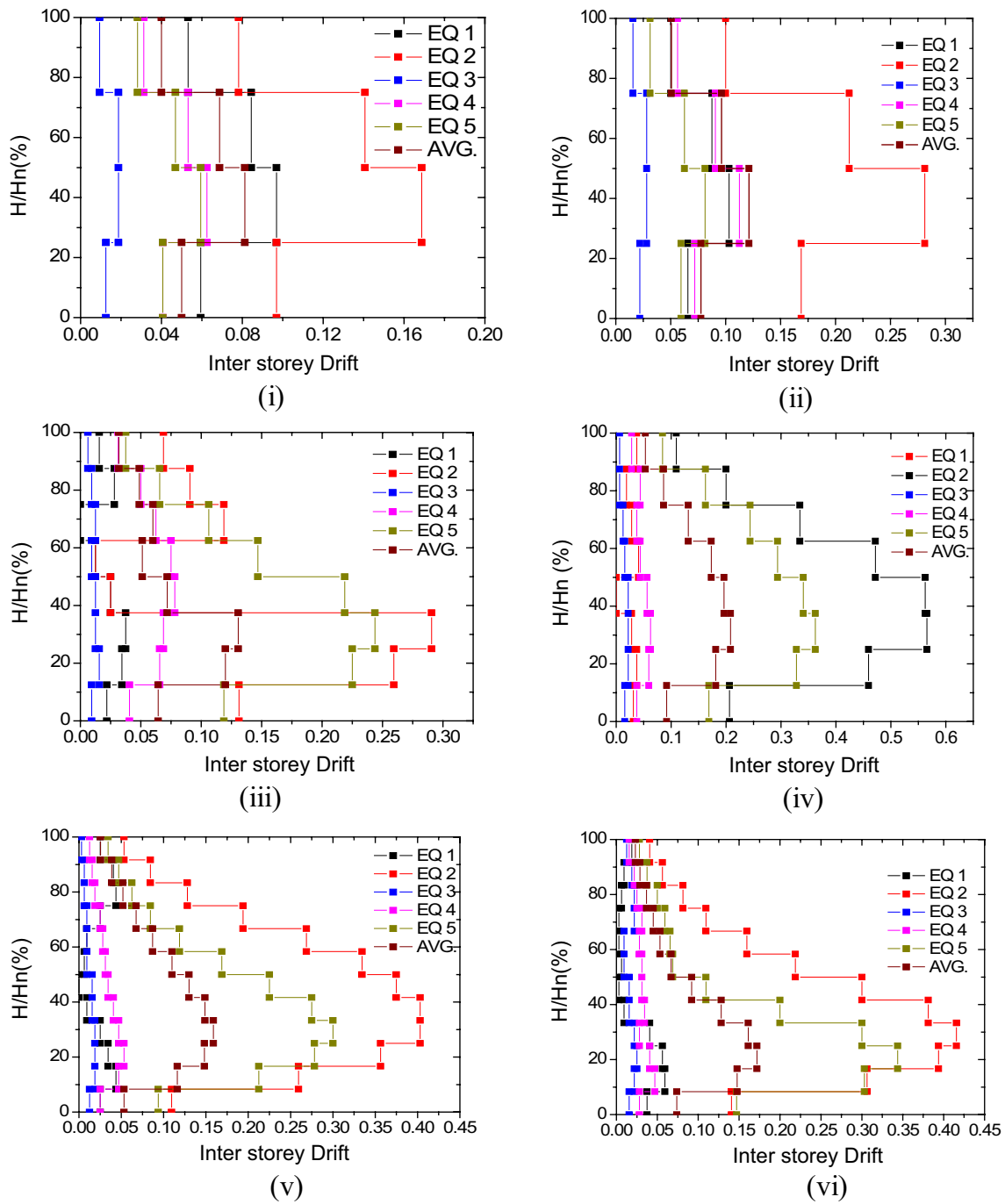
Using steel tensile strains (ε_s):

$$\theta_{target} = \left[\theta_{by} + \left(1.75\varepsilon_s\omega_s^{0.15} - 2.1\varepsilon_y\right)\left(L_p/H_B\right)\right]0.7\varepsilon_s^{-0.08} \tag{15}$$

Using concrete compression strain (ε_c):

$$\theta_{target} = \left[\theta_{by} + \left(\left(-1.6\omega_s + 4\right)\varepsilon_c + 0.004 - 2.1\varepsilon_y\right)\left(L_p/H_B\right)\right] \\ \times 0.85\varepsilon_c^{-0.065} \tag{16}$$

$$\theta_{by} = 2.1\varepsilon_y\left(L_B/6H_B\right) \tag{17}$$

$$\omega_s = \rho_{long}\left(F_y/f_c'\right) \tag{18}$$

Fig. 5 Inter-storey drift for **a** FBD four storey 2 bay, **b** DDBD four storey 2 bay, **c** FBD eight storey 2 bay, **d** DDBD eight storey 2 bay, **e** FBD twelve storey 2 bay, **f** DDBD twelve storey 2, **g** FBD four storey 3 bay, **h** DDBD four storey 3 bay, **i** FBD eight storey 3 bay, **j** DDBD eight storey 3 bay, **k** FBD twelve storey 3 bay, **l** DDBD twelve storey 3 bay

$$L_p = \max\left\{kL + 0.022F_y d_{bl} \text{ or } 0.044F_y d_{bl}\right\};$$
$$k = 0.2\left(F_u/F_y - 1\right) \leq 0.08; \tag{19}$$

$$L = L_B/2(\text{double bending}) \tag{20}$$

The frames designed for the serviceability limit were analyzed using SAP, however, NLTHA of the frames performed using the same earthquake data as given on Table 4. The focus of this analysis was to determine if the strains obtained from the analysis correlated well with the target strains. Steel tensile strains were obtained in the beams at critical stories (usually first or second storey). The strain results for the frames designed for serviceability limit using DDBD and FBD (initial and revised) were plotted in the same figures.
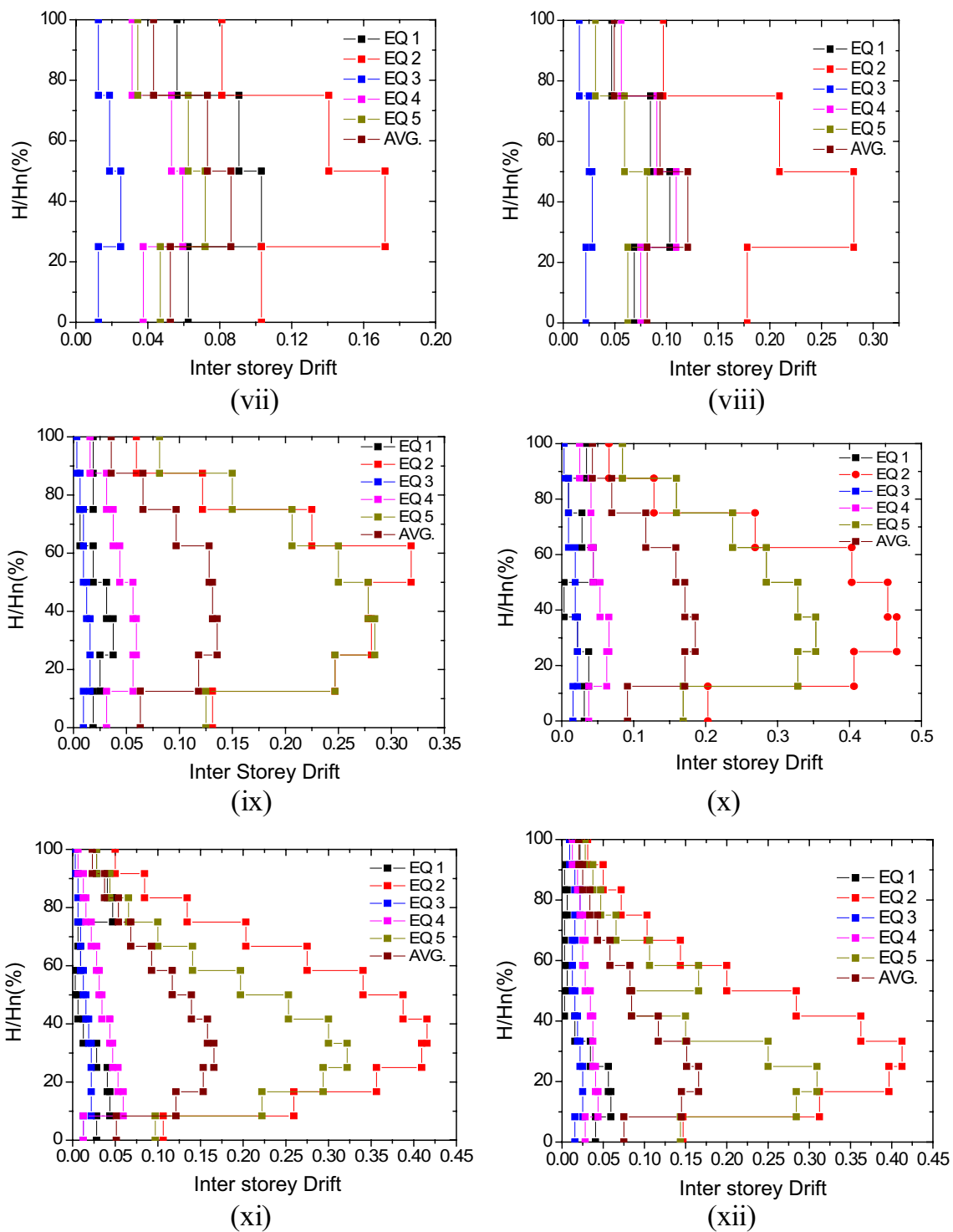
(vii)

(viii)

(ix)

(x)

(xi)

(xii)

Fig. 5 (continued)

Figure 7 shows the average maximum tensile strain in steel at the beams for the frames designed using both the methods with two and three bays, respectively. Again, the strains were obtained at the beams since the frames were designed to follow the beam sway mechanism (weak beams–strong columns). The steel tensile strains are pretty close to the serviceability limit of 0.015 (green line) for which the frames were designed using both the methods. These results show that at the allowable drift limit of 2% the FBD frames will be close to the serviceability limit based on steel and concrete strains of 0.015 and 0.004, respectively.

Table 6 Displacement in m. for frames designed using DDBD

Storey-bay	Displacement (m)					
	EQ-1	EQ-2	EQ-3	EQ-4	EQ-5	Avg
4-2 bay	0.010	0.024	0.003	0.011	0.008	0.011
8-2 bay	0.007	0.089	0.004	0.012	0.059	0.034
12-2bay	0.008	0.090	0.007	0.012	0.061	0.035
4-3 bay	0.010	0.025	0.003	0.011	0.008	0.011
8-3 bay	0.007	0.077	0.004	0.012	0.062	0.032
12-3 bay	0.008	0.081	0.011	0.007	0.065	0.035

Table 7 Inter storey drift values (%) for frames designed using FBD

Storey-bay	IDR (%)						Limit
	EQ-1	EQ-2	EQ-3	EQ-4	EQ-5	Avg	
4-2 bay	0.09	0.16	0.02	0.06	0.06	0.08	2
8-2 bay	0.04	0.29	0.02	0.08	0.24	0.13	2
12-2bay	0.03	0.40	0.02	0.05	0.28	0.16	2
4-3 bay	0.10	0.17	0.02	0.06	0.07	0.08	2
8-3 bay	0.04	0.32	0.02	0.06	0.28	0.14	2
12-3 bay	0.04	0.41	0.02	0.06	0.32	0.17	2

Table 8 Inter-storey drift values (%) for frames designed using DDBD

| Storey-bay	IDR (%)						Target
	EQ-1	EQ-2	EQ-3	EQ-4	EQ-5	Avg	
4-2 bay	0.10	0.28	0.03	0.09	0.08	0.11	2
8-2 bay	0.03	0.45	0.02	0.06	0.34	0.18	2
12-2bay	0.06	0.40	0.03	0.05	0.34	0.18	2
4-3 bay	0.10	0.28	0.03	0.10	0.08	0.12	2
8-3 bay	0.03	0.46	0.02	0.06	0.35	0.18	2
12-3 bay	0.06	0.40	0.02	0.04	0.31	0.17	2

6.1 Concrete compression strain

Although the DDBD frames designed for a steel tensile strain of 0.015 since this limit controls the design and not a serviceability concrete strain of 0.004, it was also deemed important to obtain the concrete compression strains in the critical beams (where maximum strain occurs). Figure 8 shows average values of maximum compression strain in concrete obtained from the NLTHA as a function of the number of stories for frames with two and three bays, respectively. In these figures the serviceability strain limit of 0.004 also included for comparison purpose. Higher strains were obtained in frames designed using DDBD than in the revised FBD frames. So, the DDBD frames are more suitable as the frames will be more deformed before collapse during earthquake leads to be life safety and the section are smaller than FBD frame so economical. The concrete compression strains for all DBBD frames do not exceed the serviceability limit of 0.004. This outcome was expected since the frames were designed to reach first the steel strain of 0.015 and not the concrete strain of 0.004.

The steel tension strain and concrete compression strains are shown in Fig. 8 for RC frames designed using both the methods.

Fig. 6 Ductility demand for **a** FBD four storey 2 bay, **b** DDBD four storey 2 bay, **c** FBD eight storey 2 bay, **d** DDBD eight storey 2 bay, **e** FBD twelve storey 2 bay, **f** DDBD twelve storey 2, **g** FBD four storey 3 bay, **h** DDBD four storey 3 bay, **i** FBD eight storey 3 bay, **j** DDBD eight storey 3 bay, **k** FBD twelve storey 3 bay, **l** DDBD twelve storey 3 bay
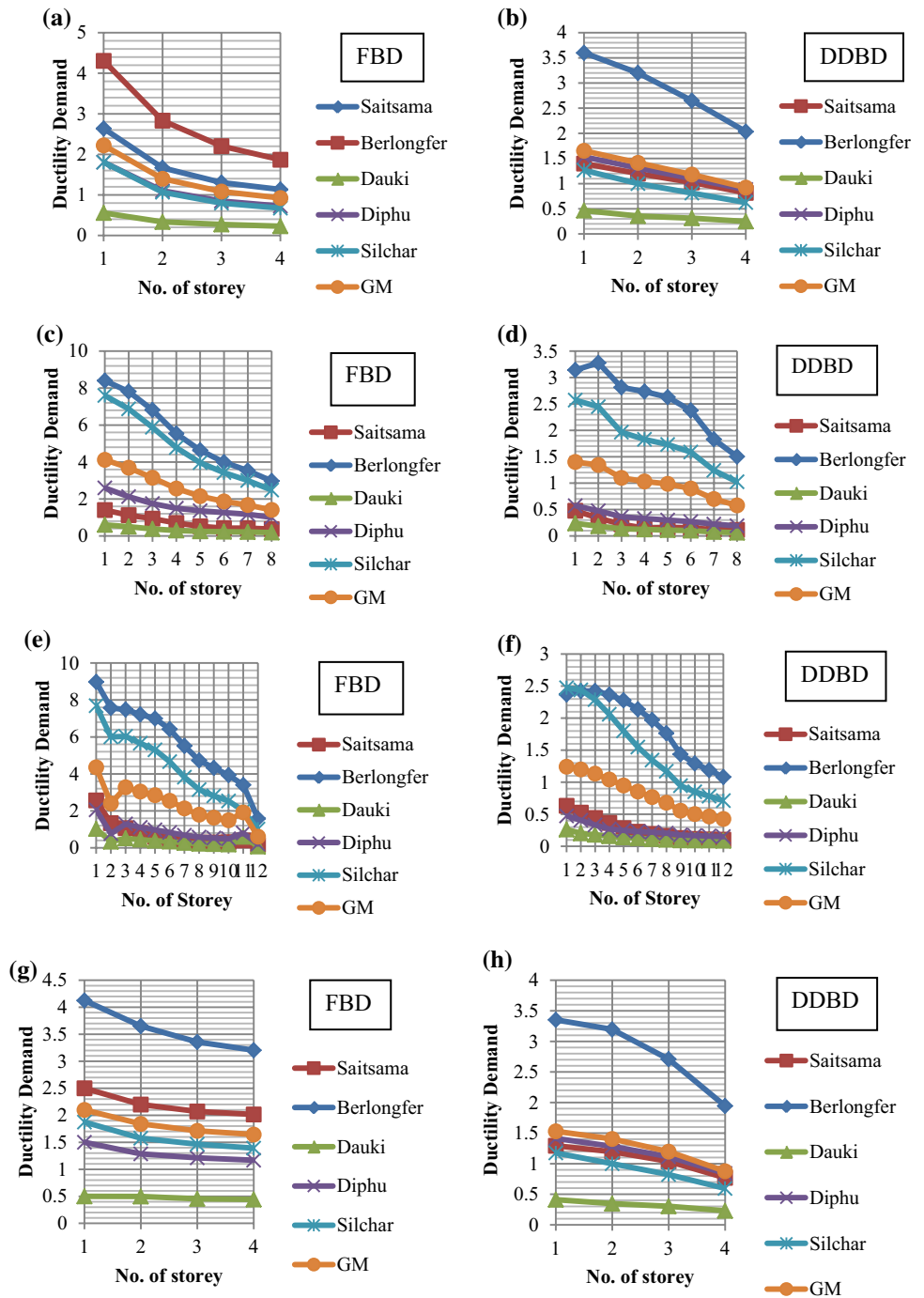
7 Conclusions

This paper explains the FBD and DDBD seismic approaches performed on four, eight and twelve stories for different structural parameters and the design approaches were performed with nonlinear time history analysis and the results are validated, it concludes the following based on the present study as:

1. Compare important parameters such as: drifts, displacements obtained from DDBD and FBD frames.

2. The displacement and inter-storey drift obtained from FBD and DDBD frames were approximately similar. Thus smaller RC sections were obtained using DDBD, resulting in some savings in material.

3. The base shear values of DDBD structures are less compared to FBD structures, this explains that the dis-
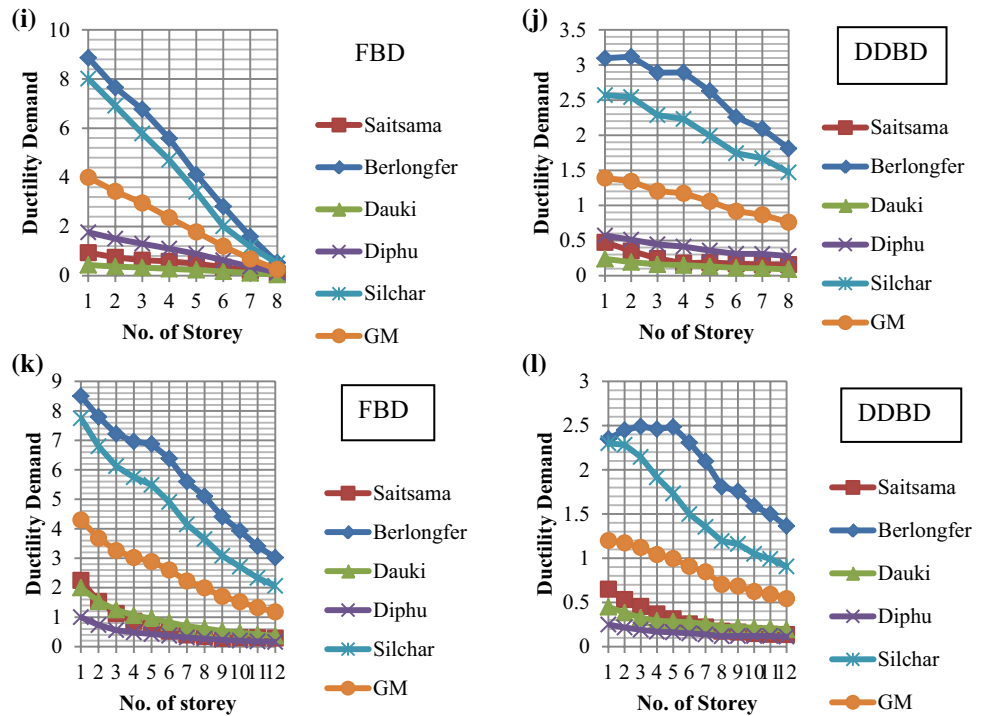
Fig. 6 (continued)

Table 9 Ductility Demand values

No. of Storey	Bay-2		Bay-3	
	FBD	DDBD	FBD	DDBD
4	2.22	1.65	2.1	1.53
8	4.12	1.4	4	1.39
12	4.36	1.24	4.3	1.2

Table 10 Steel Tension Strain for both FBD and DDBD method for bay-2 and bay-3 building

Serial No.	Storey	Bay-2		Bay-3	
		FBD	DDBD	FBD	DDBD
1	4	0.0113	0.0137	0.0129	0.0136
2	8	0.0140	0.0149	0.0142	0.0149
3	12	0.0117	0.0140	0.0132	0.0143

Table 11 Concrete compression strain for both FBD and DDBD method for bay-2 and bay-3 building

| Serial no.	Storey	Bay-2		Bay-3	
		FBD	DDBD	FBD	DDBD
1	4	0.00145	0.00174	0.00157	0.00174
2	8	0.00185	0.00260	0.00200	0.00220
3	12	0.00143	0.00176	0.00153	0.00177

placement approach is preferred which results in lower stiffness (or provides more flexibility) and has less acceleration demands.

4. Maximum inter-storey drift occurs at bottom of the framed structure as the base will be more rigid. FBD and DDBD shows the drift values less than the actual design drift limit (d = 0.02).

5. The displacement and drift values for FBD is lesser than that of the DDBD comparing on their respective storey in nonlinear time history analysis for five earthquakes data. Thus maximum inter-storey drifts were controlled with good accuracy using DDBD methods.

6. Ductility demand values of DDBD structure decreases by 26, 65, and 72% for four, eight and twelve storeys respectively than that of the FBD structure. Direct displacement based design shows lesser ductility value than the FBD as DDBD undergoes flexible deflection.

7. The tension Steel strain and Compression Concrete strain were obtained by using both FBD and DDBD frames, the DDBD frames have slightly higher value than FBD frame as the frame will be deformed more before collapse during earthquake leads to life safety than FBD frame.
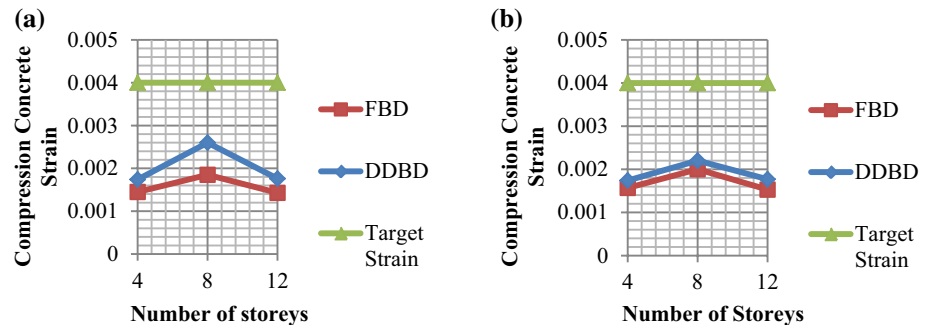
Thus, overall DDBD structure shows good performance over the structural parameters and achieved design exhibits better and safe compared to FBD structures.

Fig. 7 Average tensile strain in steel from the NLTHA for building with **a** two bay, **b** three bay
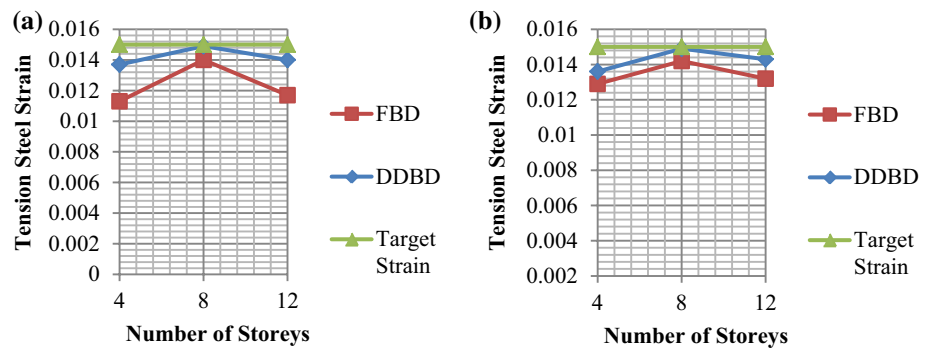
Fig. 8 Average compressive strain in concrete from the NLTHA for building with **a** two bay, **b** three bay

Compliance with ethical standards

Conflict of interest Authors do not have conflict of interest in publishing of the paper as specified in this journal.

References

1. Calvi GM, Priestley MJN, Kowalsky MJ (2008) Displacement based seismic design of structures. Istituto Universitario di Studi Superiori Press (IUSS), Pavia
2. Yang Lu, Hajirasouliha Iman, Marshall Alec M (2018) Direct displacement based seismic design of flexible-base structures subjected to pulse-like ground motions. Eng Struct 168:276–289
3. Loss C, Thomas T, Tesfamariam S (2018) State-of-the-art review of displacement based seismic design of timber buildings. Constr Build Mater 191:481–497
4. Alhaddad MS, Wazira KM, Al-Salloum YA, Abbas H (2015) Ductility damage indices based on seismic performance of RC frames. Soil Dyn Earthq Eng 77:226–237
5. Malekpour S, Dashti F (2013) Application of the DDBD methodology for different types of RC structural systems. Int J Concr Struct Mater 7:135–153
6. Massena B, Bento R, Degée H (2012) Assessment of direct displacement –based seismic design of reinforced concrete frames, 15th WCEE, 2012, Lisbon, Portugal
7. Priestley MJN, Kawalsky MJ (2000) Direct displacement based seismic design of concrete building. Bull N Z Soc Earthq Eng 33:421–444
8. Garcia R, Sullivan TJ, Corte GD (2010) Development of a DBD method for steel frame RC wall buildings. J Earthq Eng 14:252–277
9. van de Lindt JW, Rosowsky DV (2013) Performance-based seismic design of mid-rise wood frame buildings. J Struct Eng ASCE 139:1294–1302
10. Sullivan TJ, Calvi GM, Priestley MJN (2009) The limitations and performances of different displacement based design methods. J Earthq Eng 7:201–241
11. Ferraioli Massimiliano (2015) Case study of seismic performance assessment of irregular RC buildings: hospital structure of Avezzano (L'Aquila, Italy). Earthq Eng Eng Vib 14:141–156
12. Sullivan TJ, Priestley MJN, Calvi GM (2010) Development of an innovative seismic design procedure for frame-wall structures. J Earthq Eng 9:279–307
13. Džakić D, Kraus I, Morić D (2012) DDBD of regular concrete frames in compliance with Euro code-8. Tehnicki vjesnik/Technical Gazette 19:973–982
14. Macedo L, Castro JM (2012) Direct displacement based seismic design of steel moment frames, 15th WCEE, 2012, Lisbon, Portugal
15. Sheth RK, Sheth DP, Soni KN (2014) DDBD for fifteen storey reinforced concrete moment resisting frame as per IS codes. Int J Adv Eng Res Dev 1:179–184
16. El Attar A, Zaghw AH, Elansary A (2014) Comparison between the direct displacement based design and the force based design methods in reinforced concrete framed structures. In: Second european conference on earthquake engineering and seismology, Istanbul, pp 25–29
17. Dohadwala AT, Sheth RK, Dr Patel IN (2014) Comparison of base shear for forced based design method and DDBD Method'. Int J Adv Eng Res Dev (IJAERD) 1:1–9
18. Sil A, Longmailai T (2017) Drift reliability assessment of a four storey residential building under seismic loading considering multiple factors. J Inst of Eng (India) Ser-A 8:245–256
19. Plain and Reinforced Concrete—Code of practice, IS 456 (2000)
20. Criteria for earthquake resistant design of structures IS 1893 (part 2) (2002)

21. Pettinga JD, Priestley M (2005) Dynamic behavior of reinforced concrete frames designed with DDBD. J Earthq Eng 9:309–330

22. Calvi GM, Sullivan TJ (2009) A model code for the displacement based seismic design of Structures". IUSS Press, Pavia

23. FEMA 356 (2000) in Prestandard and commentary for the seismic rehabilitation of buildings. US Federal Emergency Management Agency

24. FEMA 445 (2006) Next-generation performance-based seismic design guidelines. Program plan for new and existing buildings. Federal Emergency Management Agemcy, Washington, DC

25. SAP2000 (2006) v. 14 Nonlinear. Educational version, Computer and structures Inc., Berkley

26. Code of Practice for Design Loads (1997) (Other Than Earthquake) for Buildings and Structures. Public Work IS: 875

27. Vidot-Vega AL, Kowalsky MJ (2013) Drift, strain limits and ductility demands for RC moment frames designed with displacement-based and force-based design methods. Department of Civil, Construction, and Environmental Engineering, Engineering Structures 51:128–140