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Analytical Study on Performance Evaluation of Multi-Storey RC Building by Direct Displacement Approach

Mr. Ganesh S. Bhagure
PG Student
Department of Structural Engineering
Government College of Engineering, Aurangabad, MS, India

Dr. R. S. Londhe
Associate Professor
Department of Structural Engineering
Government College of Engineering, Aurangabad, MS, India

Abstract

Now-a-days structures are designed based on Force Based Design (FBD) using traditional seismic design codes (IS 1893:2002). Inadequate response reduction factor, Initial-stiffness categorization of structures, and design fundamental time period is based on height dependent formula this problems are associated when building subjected to seismic loads. For finding the solution of those problem need to develop alternatives design approach which based on Performance Based Seismic Design (PBSD) is Direct Displacement Design Approach (DDBD). DDBD is the new concept for seismic design of structures. In PBSD procedure Direct Displacement-Based Design (DDBD) approach is one of the available approach, which is implemented in this study. Design and analysis is done for regular R. C. building frame of 4, 8 and 12 storey based on IS 1893:2002 by using two design approaches. The performance evaluation of buildings designed by FBD and DDBD is done using nonlinear static pushover analysis in accordance with FEMA 356 using SAP 2000 v.18.2.0. The results of analysis are compared with FBD and DDBD approach in terms of base shear, storey drift, number of hinge formation for achieving same performance level.

Keywords: Force based Design (FBD), Direct Displacement based Design (DDBD), Performance based Seismic Design (PBSD), Nonlinear Static Pushover Analysis, Pushover Curve and Storey Drift

I. INTRODUCTION

The ultimate aim of structural engineer is to design structures to sustain various types of loads imposed by their service requirements and natural hazards. Presently, guided by codes and standards. Structures designed with current seismic design codes and standards, should be able to satisfy specific performance level, known as life safety performance level, for a specific intensity of ground motion. However, occupancy interruptions and economic losses are not provided. In addition, although life safety performance level is obtained for different structures, the responses of various structures are different in terms of damages for the same earthquake hazard levels [01].

Building performance is an indicator of how well a structure supports the defined needs of its users [01]. Acceptable performances indicate acceptable levels of damage or condition that allow uninterrupted facility operation. Consequently, performance-based design is the process used by design professionals to create buildings that protect functionality and the continued availability of services [01].

Performance-Based Seismic Design (PBSD) is a modern approach for the design of new structures and evaluation and retrofitting of existing structures, recently. Structures designed through PBSD approach, would be able to show different performance levels for different earthquake ground motions [01, 05].

PBSD is an iterative procedure always under development. Direct displacement approaches can be used to gain the performance objectives. Performance objective is the combination of performance levels and hazard levels and performance levels can be determined by damage states of the structural and non-structural components and content systems [02]. Since the damages are related directly to displacements, therefore, in this study Direct Displacement-Based Design (DDBD) approach in the content of PBSD is implemented.

II. PERFORMANCE-BASED SEISMIC DESIGN (PBSD)

PBSD is starts with the selection of design criteria which is in the form of one or more performance objectives [01].

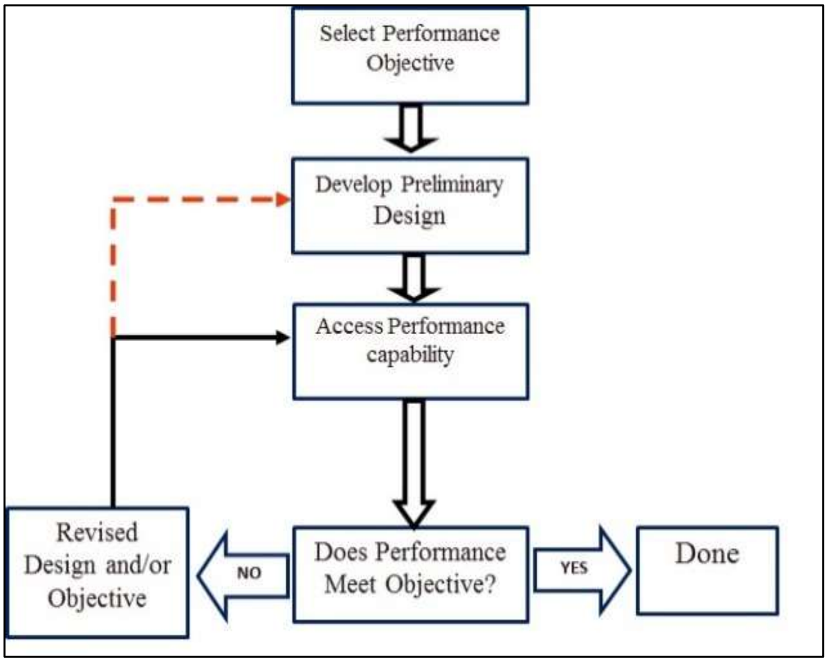

Fig. 1: Performance-Based Flow Diagram [1]

III. DIRECT DISPLACEMENT-BASED SEISMIC DESIGN PROCEDURE [04]

The regular R. C. building frames are used in this study, according to Priestly et al. 2007 [04]. The following procedure is to be used for analysis and design [20].

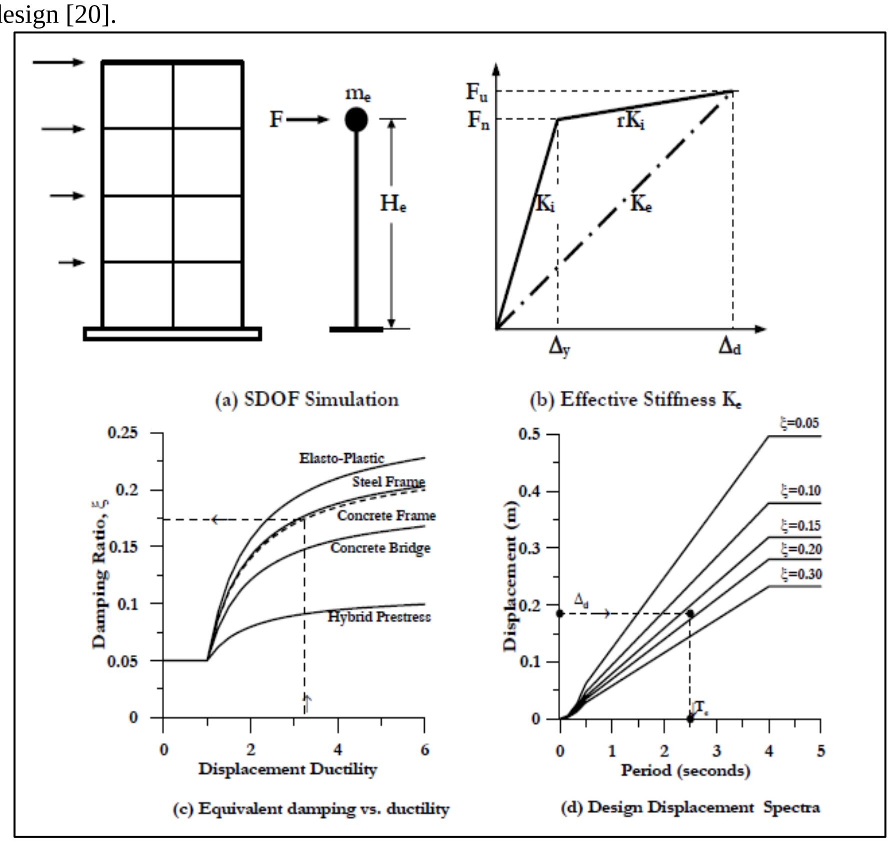

Fig. 2: Fundamentals of DDBD [4]

A. Representation of MDOF System by an SDOF System

MDOF system is presented by an equivalent SDOF system. The equivalent SDOF system has equivalent mass and height [04]. The first mode of inelastic response of the structure is taken into account. The following steps are needed [04, 20, and 27].

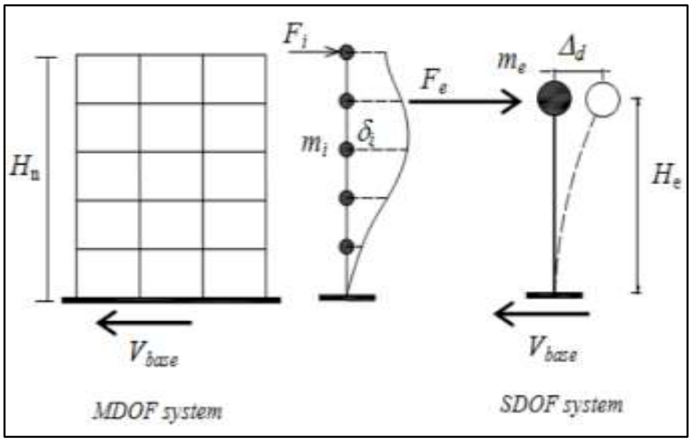

Fig. 3: Simplified Model of MDOF System Represented by SDOF System [20]

1) *Step1: Determine design displacement profile (Δ_i)*

The assumed design displacement profile, corresponding to the normalized inelastic mode shape δ_i at the design drift limit $\boldsymbol{\theta}_d$. Where i = 1 to n storeys [20].

$$\Delta i = \delta i \frac{\Delta c}{\delta c} \omega\theta \quad (1)$$

Where the normalized inelastic mode shape $\boldsymbol{\delta_i}$ depends on the height (H_i) and roof height (H_n) according to the following relationships:

$$\text{For } n \leq 4: \boldsymbol{\delta i} = \frac{\text{Hi}}{\text{Hn}} \quad (2a)$$

$$\text{For } n > 4: \delta i = \frac{4}{3}\left(\frac{\text{Hi}}{\text{Hn}}\right)\left(1 - \frac{\text{Hi}}{4\text{Hn}}\right) \quad (2b)$$

This normalized inelastic mode shape implies that maximum drift occurs between ground and first floor. So first storey is the critical storey [20].

$$\boldsymbol{\omega}\theta = 1.15 - 0.0034H_n \leq 1.0 \ (H_n \text{ in m})$$

= Drift reduction factor for controlling higher mode effect.

2) *Step2: Calculate design displacement (Δ_d):*

$$\Delta d = \frac{\sum_{i=1}^{n}(m_i\Delta_i^2)}{\sum_{i=1}^{n}(m_i\Delta_i)} \quad (3)$$

3) *Step3: Calculate effective height (H_e) and effective mass:*

$$He = \frac{\sum_{i=1}^{n}(m_i\Delta_i H_i)}{\sum_{i=1}^{n}(m_i\Delta_i)} \quad (4)$$

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

4) *Step4: Calculate yield displacement (Δ_y):*

$$\Delta y = \theta y \text{ x He} \quad (6)$$

For reinforced concrete frame structure,

$$\theta y = 0.5 \text{ x } \varepsilon y \text{ x } \frac{L_b}{h_c} \quad (7)$$

5) *Step5: Calculate design ductility (μ) and equivalent viscous damping (ξ_{eq}):*

For estimation of equivalent viscous damping (ξ_{eq}), the displacement ductility μ must be known. The displacement ductility is the ratio between the equivalent design displacement and the equivalent yield displacement Δ_y. The equivalent yield displacement is estimated according to the considered properties of the structural elements, for example through the use of approximated equations proposed and based on the yield curvature [20].

$$\mu = \frac{\Delta d}{\Delta y} \quad (8)$$

$$\xi eq = 0.05 + 0.565 \text{ x } \frac{\mu - 1}{\mu\pi} \quad (9)$$

6) *Step6: Determine effective time period (T_e) of substitute structure and Effective Stiffness (K_e)*

It is the effective time period of the equivalent SDOF system and it is obtained from displacement spectra corresponding to the curve for equivalent damping (ξeq) and the value of design displacement (Δ_d) for IS 1893:2002 [06] as shown in chart-1.

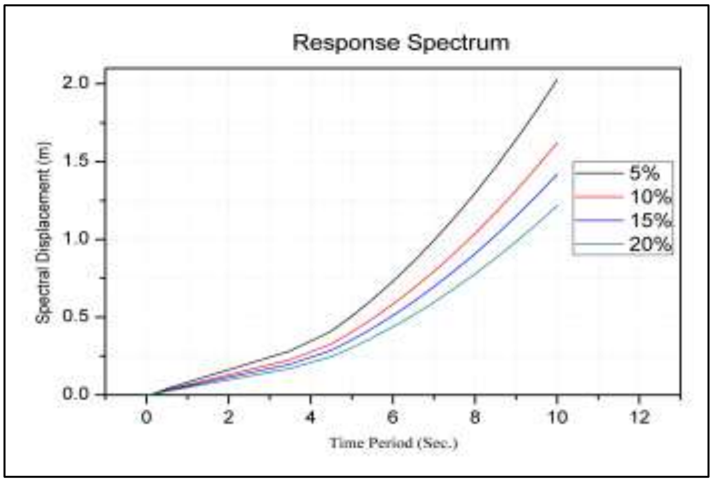

Fig. 1: Response Spectrum

From consideration of the mass participating in the first inelastic mode of vibration, the effective system mass for the substitute structure is,

$$K_e = \frac{4\pi^2 m_e}{T_e^2} \qquad (10)$$

7) *Step7: Calculation of design base shear (Vb):*

$$Vb = Ke \text{ x } \Delta d \qquad \ldots. \quad (11)$$

8) *Step 8: Distribution of base shear force to floor levels*

The base shear force is distributed to the floor levels in proportion to the product of mass and displacements, given in Indian Seismic Design Code (IS 1893:2000) [06], is adopted

$$Fi = Vb \text{ x } \left[\frac{m_i \Delta_i}{\sum m_i \Delta_i}\right] \qquad (12)$$

9) *Step 9: Calculation of the Total Overturning Moment (M_{OTM})*

Once the forces at the top of each story are obtained, then total overturning moment at the base of the building can be found from the following equation [20]:

$$M_{OTM} = \sum_{i=1}^{n} F_i H_i \qquad (13)$$

10) *Step 10: P-Delta Effects*

P-Delta effects should also be included if they are being required. The stability index is calculated by following relation [4, 20]:

$$\theta_\Delta = \frac{P\Delta_{max}}{M_{otm}} \qquad (14)$$

Where

$M_D = M_{OTM}$ and $\Delta_{max} = \Delta_d$

P is the total seismic weight of the building considering 100% of live load [20].

If $0.1 \leq \theta_\Delta \leq 0.33$ P-Delta effects should be considered. If, $\theta_\Delta > 0.33$, then the structure must be made stiffer [58], and the calculations should be revised. Further, if $\theta_\Delta < 0.1$, then there is no need to take into account the P-Delta effects. The base shear force is amplified (if $0.1 \leq \theta_\Delta \leq 0.33$) and will be found as follows [6, 28]:

$$V_{base\,(modified)} = K_{eff}\Delta_d + c \text{ x } \frac{P\Delta_{max}}{H_e} \qquad (15)$$

C is a constant and for reinforced concrete structures, C = 0.5 is used.

After calculating base shear considering P-delta effects, the base shear is distributed again using Equation 12.

IV. System development

A. *Building Geometry*

A regular reinforced concrete moment resisting building frame have been selected. They are similar in plans, but different in height 4, 8 and 12 stories with storey height 3.0m, and regular reinforced concrete frames are the only earthquake resistance system of these buildings. From each of these buildings, one frame, critical in terms of gravitational loads, is chosen. In Figure 4, a typical plan of the building has been shown, the dimensions are all in *m*, and frame 2-2, surrounded by shaded area is the frame of interest. The design of all frames was according to the Indian standard Seismic code IS 1893:2002 [06]. The reinforced concrete moment resistance frames chosen are having 3 bays. Each structure are designed by using FBD and DDBD approach.

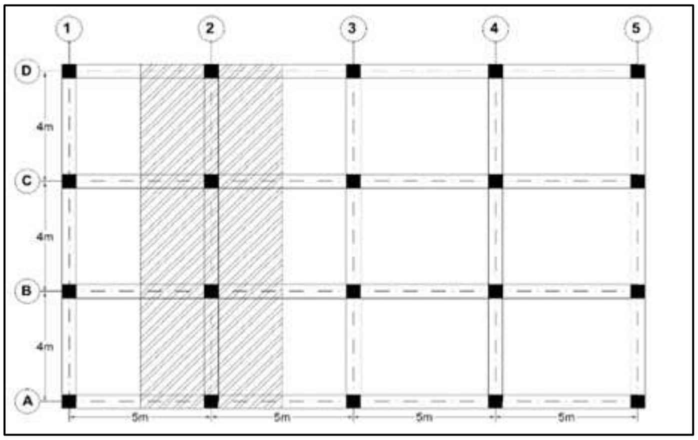

Fig. 4: Typical Plan of the Building

B. Material Properties

- Young modulus of concrete (E_c) = 25 x 10^3
- Young modulus of steel (E_s) = 2 x 10^5
- Poisson's ratio of concrete (μ_c) = 0.15
- Poisson's ratio of steel (μ_s) = 0.3
- Grade of concrete (f_{ck}) = 25 N/mm^2
- Grade of steel (f_y) = 415 N/mm^2
- Steel density = 7850 kg/m^3

C. Column, Beam & Slab Sizes for Modelling of Building

Table - 1
Dimension of Structural Members

Element	*No. of storey*	*Size (mm)*
Beam	*4*	*300 x 450*
	8	*300 x 450*
	12	*300 x 450*
Column	*4*	*450 x 450 (1st to 4th storey)*
	8	*550 x 550 (1st to 4th storey)* *450 x 450 (5st to 8th storey)*
	12	*650 x 650 (1st to 4th storey)* *550 x 550 (5st to 8th storey)* *450 x 450 (9st to 12th storey)*
Slab	*-*	*150 mm thick*

D. Load Values

All the load values are shown in following table are taken from IS 875:1987.

Table – 2
Loading Data

Types of load	***Load values***	
	Typical floor level	***At terrace level***
Dead load	*Self-weight*	
Floor finish	*2 kN/m^2*	*1.5 kN/m^2*
Wall load	*13 kN/m on all beams*	*6 kN/m on peripheral beam*
Live load	*3 kN/m^2 on Eall floor*	
Earthquake load	*As per is 1893:2002 corresponding Zone Z = V, Type of soil = Medium soil. Importance factor I = 1, Response reduction factor R = 3*	

V. RESULT AND DISCUSSION

A. Pushover Analysis

Pushover analysis is a useful and practical tool for the performance evaluation of existing structures and newly designed structures [27]. It is one of the simplest analysis methods and it is used for inelastic analysis of the structure, under a vector of forces or a vector of displacements [30]. The vector of forces presents the expected inertial forces in structures, due to earthquake loads while displacement vector presents the expected displacement response in the structures. In pushover analysis the vector of forces or displacements is increasing monotonically on the structure (i.e. the vector of forces or displacements are incrementally applied) [27]. The main outcome of pushover analysis is a capacity curve (Base shear force vs. top displacement), from which target displacement, a displacement induced by design earthquake, can be determined. The slope of the capacity curve will change, which shows a nonlinear behavior of the structure [30].

Table - 4
Drift value for various performance level [5, 22]

Performance level	*Drift value*
Immediate occupancy (IO)	*1%*
Life safety (LS)	*2%*
Collapse prevention (CP)	*4%*

For implementation of nonlinear static pushover analysis of the designed frames, as mentioned earlier, SAP2000 v18.2.0 [28] is utilized.

1) Results of Nonlinear Static Pushover Analysis

The results from nonlinear static pushover analysis are presented in terms of capacity curves, storey drift and sway mechanism for Immediate Occupancy (IO), life safety (LS) and Collapse Prevention (CP) performance level, and inter-storey drift ratios. In these curves limits for different performance levels are also shown.

From the capacity curves of the frames, shown in Figure 5, it can be seen that the limits for life safety performance levels in terms of top displacements are approximately 175mm, 300mm and 385mm which correspond to 266.09kN, 328.52kN and 380kN base shear forces, for 4, 8 and 12 story frames, respectively.

The design displacements for the roof level of 4 story frame, obtained through DDBD approach, which is 192mm and the corresponding base shear force is almost 328.48kN, is not satisfied. For 8 story frame the top displacement achieved through nonlinear static pushover analysis is less than the one obtained by DDBD procedure which is 300mm, but the corresponding base shear force gained by nonlinear static pushover analysis is little bit greater than the one obtained through DDBD procedure (325.45kN < 328.52kN). The result for top displacement of 8 story frame is 300mm and 373mm, obtained through nonlinear static pushover and DDBD procedures, with corresponding base shear forces of 328.52kN and 325.45kN, respectively.

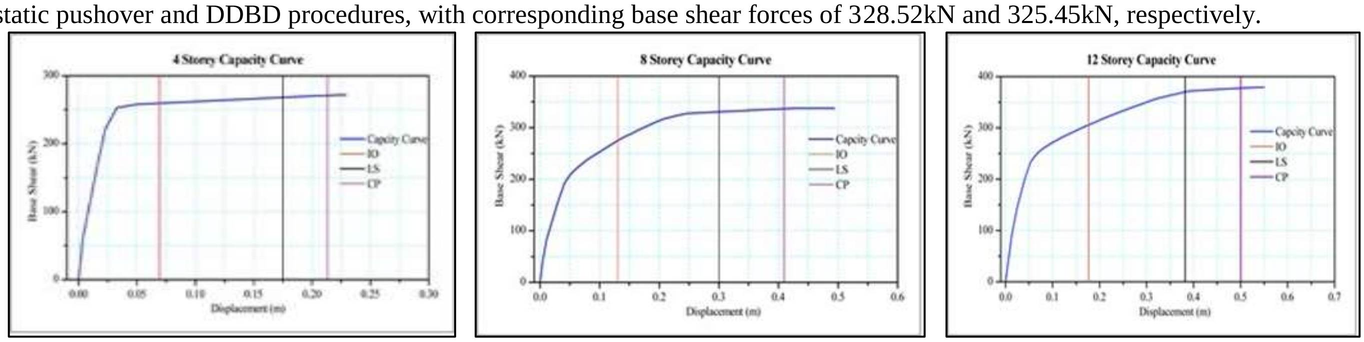

Fig. 5: Capacity Curves for 4, 8 & 12 Storey Irregular RC Frames obtained through Nonlinear Static Pushover Analysis

The inter-storey drift ratios, shown in Figure 6, are for three different steps of the nonlinear static pushover analysis, which correspond to the immediate occupancy, life safety, and collapse prevention performance levels. The story drift ratios for life safety performance level in this figure for frames are just to check if they meet the one chosen for the corresponding life safety performance level.

The story drift ratios for all frames are larger than target drift ratio in life safety performance level. To compare, the 4, 8 and 12 story frame, the maximum inter-storey drift ratio occurs in 8 story frame. This is due to larger displacement obtained using nonlinear static pushover analysis than the design target displacement obtained through DDBD approach.

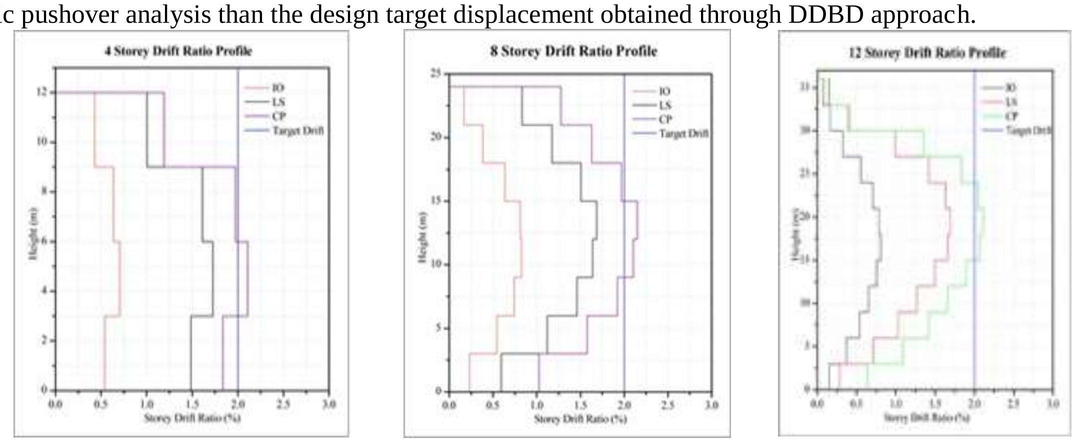

Fig. 6: Inter-storey Drift Ratios of for 12 Storey Irregular RC Frames obtained through Nonlinear Static Pushover Analysis

The larger drift ratios occurred at those levels of the frames, at which beams entered the heavy damage zone (refer to Figure 7). These drift ratios occurred mostly at the lower levels as can be expected from a frame structure. For example, for 4 story frame the maximum inter-storey drift occurred at the second story, if Figure 7 is checked the mid-span beam at the top of first floor has reached to heavy damage region. For 8 story frame, the maximum drift occurred at the 4th story, which corresponds to the level where the upper and lower beams of this level reached to heavy damaged region.

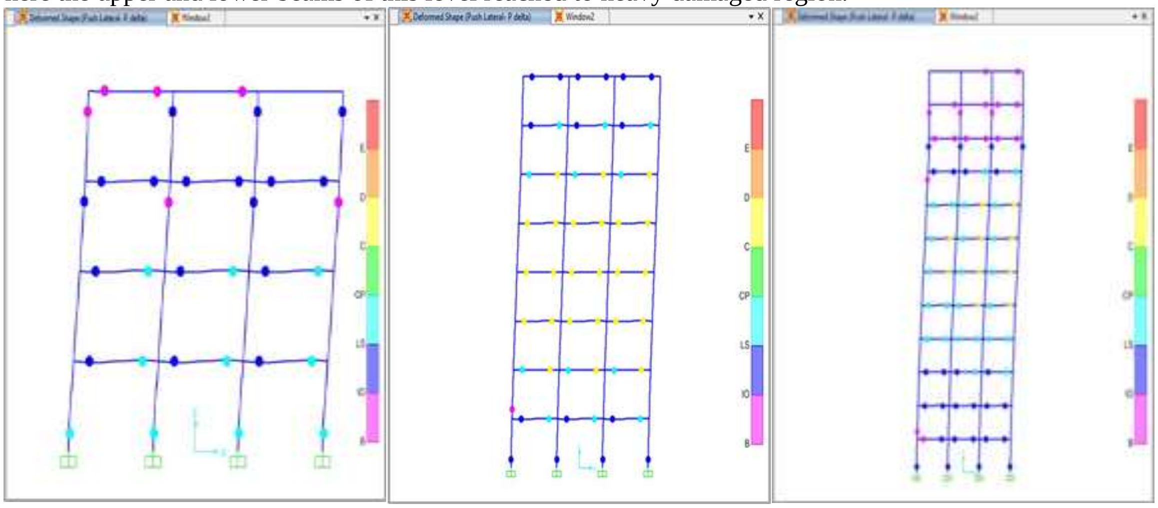

Fig. 7: Sway Mechanisms of 4, 8 & 12 Storey Irregular RC Frames obtained through Nonlinear Static Pushover Analysis

To conclude, the frames are much stiffer than expected, and the stiffest frame amongst these three frames is the 4 story frame, while the 8 story structure is least stiff.

VI. CONCLUSION

1) Direct Displacement Based Design gives lower base shear than Force based method (IS 1893:2002). Base shear obtained by DDBD is 29.31%, 25.83% and 10.60% less for 4, 8 and 12–storey respectively compared to Force Based Method (IS 1893:2002). From this results Force based method is conservative in designing medium rise buildings.
2) As a no. of storey increases, base shear, difference in base shear obtained by FBD and DDBD method decreases. It means that DDBD method converges to FBD method as number of storey increases.
3) For 4, 8 and 12 storey frame the top displacement obtained from nonlinear static pushover analysis is nearly matches to the top displacement through DDBD approach. The corresponding base shear forces from nonlinear static pushover analysis are nearly matches with the base shear obtained through DDBD approach. This concludes that the performances of design frame for life safety performance level are within limit.

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