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Comparative Evaluation of Direct Displacement Based Design and Performance Based Plastic Design of regular Moment Resisting Steel Frame

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Abstract: Structures designed by current seismic code uses Force-based design approach, which consider inelastic response of structure in rather indirect manner by using factors like Response Reduction Factor (R) over which debate already, exist. Hence, a need arises to use a rational method, Performance based design is one such method, which uses Displacement-based designed approach. Performance based Plastic Design (PBPD) and Direct Displacement based Design (DDBD) are two such Performance based methodologies which target drift and yield mechanism are used as performance criteria, so a comparative evaluation of Base shear, Storey drift and Failure pattern is done for a moment resisting steel frame of 10 storey by using the above stated method.

Keywords: Performance Based Plastic Design methodology; Direct Displacement Based Design methodology; Ductility factor; Yield mechanism; Target drift.

I. INTRODUCTION

Direct Displacement Based Design method is the most recently methods proposed for performance-based design of structures. During last decade, different methods have been proposed based on displacement-based design of structures but only a few are suitably applicable within modern design codes. This study focuses on direct displacement based design of regular steel moment resisting structures.

The concepts of Gulkan & Sozen (1974) [21] which are recently developed by Priestley & Kowalski (2000) [11] for direct displacement based design of regular and ductile RC structures are used to estimate nonlinear response of elastic models with equivalent damping values.

PBPD method uses pre-selected target drift and yield mechanisms as key performance objectives. These two design parameters are immediately associated to the degree and distribution of structural damage, respectively.

The design base shear for a specified hazard, which is generally given as design spectrum in the codes, is estimated by equating the work required to push the structure monotonically up to the target drift to the energy required by an equivalent EP-SDOF to achieve the same state. Likewise, a new distribution of lateral design forces is utilized that is based on the proportional distribution of maximum storey shears consistent with inelastic dynamic response results (Chao et al., 2007) [22].

Plastic design is then performed to detail the frame members and connections in order to achieve the intended yield mechanism and behavior. Therefore, determination of design base shear, lateral force distribution and plastic design are three principal parts of the PBPD method [5].

The main objective of the present study is to carry out seismic design of regular steel moment resisting frame using displacement-based design approach and performance based plastic design instead of codal-based force-based design.

Also, comparison among DBD, PBPD and FBD method is of prime concern. The specific objectives are as follows:

1) To understand need of displacement-based seismic design.
2) To evaluate lateral forces using IS code, PBPD and DDBD method.
3) To determine the failure pattern and the location of hinge formation so as to know weather “Strong Column - Weak Beam” holds true by performing pushover analysis.

Scope of the study will be limited to Selected moment resisting steel frame resting on medium soil. As there is a need of more rational method for analysis of earthquake, Performance based design provides with it.

PBPD and DDBD are two such methodologies, which are widely accepted, hence there is a need of evaluating the lateral forces and their failure pattern by both the methods.

II. DESIGN METHODOLOGY

A. Design Methodology of Direct Displacement Based Design

As mentioned by Priestly (2003) it is shown that the damage limit can be touched on to strain which can be translated into equivalent displacements. It is not practically possible to directly relate the damage limit to force-level.

In DDBD methodology, the original MDOF structure is replaced with an equivalent SDOF system. This equivalent system is represented by a secant stiffness k_e(fig.1 (a)) at maximum displacement Δ_d and an equivalent viscous damping including both the viscous and hysteretic damping of structure. Thus, as shown in fig.1(c). The approach used to characterize the structure is based on the “substitute structure” analysis procedure developed by shibata and sozen in the 1970’s.

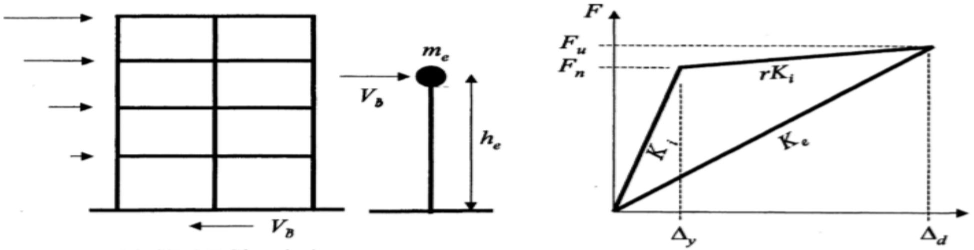
With the design displacement Δ_d calculated, as discussed subsequently, and the damping estimated from the expected ductility demand, the effective period $T_e$at maximum displacement response can be read from set of design displacement spectra, as shown in figure1d. Fig.3-1-a represents the structure as an equivalent SDOF oscillator, the effective stiffness k_e at maximum response displacement can be found by inverting the equation for natural period of a SDOF oscillator.

$$k_e = 4\pi^2 \left({m_e}/{T_e}^2 \right) \qquad \text{[Eq. 1]}$$

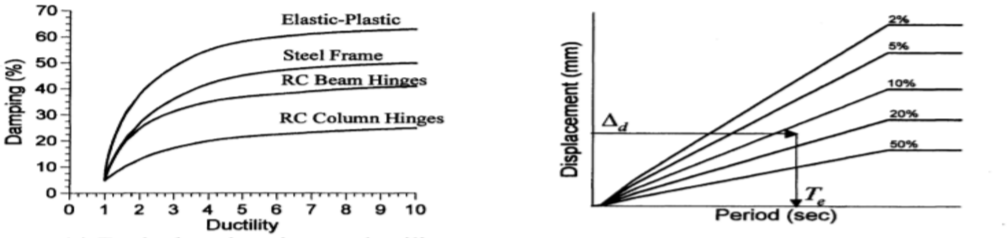
Where Ke and me are respectively effective mass and effective stiffness of SDOF structure.

From fig 1b, the design base shear at maximum response is the

$$V_B = F_u = k_e \Delta_d \qquad \text{[Eq. 2]}$$



(a) SDOF Simulation **(b) Effective stiffness, K_e**



(c) Equivalent damping vs. ductility **(d) Design displacement response spectra**

Fig.-1 fundamentals of direct displacement based design

Stepwise procedure for DDBD

1) Step-1 Determination of design displacement Δ_d

Maximum displacement profile of stories determined, design displacementΔ_d, effective mass m_e and effective height $H_e$of equivalent SDOF system are calculated as the following,

$$\Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \qquad \text{[Eq. 3]}$$

$$m_e = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} \qquad \text{[Eq. 4]}$$

$$H_e = \frac{\sum_{i=1}^{n} m_i h_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i}$$ [Eq. 5]

2) *Step-2 Determination of maximum displacement profile* Δ_i

$$\Delta_i = P_1 \theta_d h_i \left(1 - \frac{P_2 h_i}{H}\right)$$ [Eq. 6]

The calculation of the parameters P1 and P2 is done with the aid of Table 1,as a function of the number of stories of the frame and the desired response range (elastic or inelastic) as stated by kravalisis.

Table 1

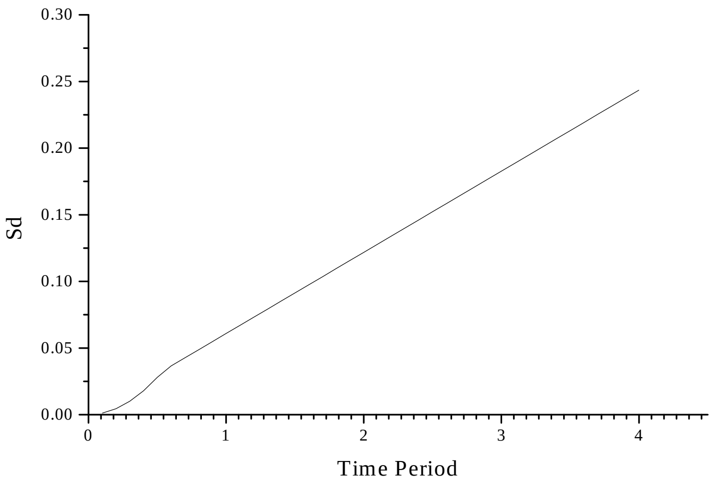
Values of the Parameters of The Proposed Maximum Displacement Profile

Stories	Elastic response		Inelastic response	
	P1	P2	P1	P2
1	1.00	0.00	1.00	0.00
3	1.00	0.18	1.00	0.10
6	0.85	0.20	0.90	0.20
9	0.70	0.21	0.75,0.80,0.85	0.30
12	0.62	0.22	0.70,0.75,0.80	0.35
15	0.55	0.24	0.65,0.70,0.75	0.40
18	0.52	0.25	0.66,0.65,0.70	0.40
20	0.50	0.25	0.55,0.60,0.65	0.40

3) *Step-3 Design displacement spectra*

T_e Can be obtained from design displacement spectra. In IS 1893(part-1):2016 Response spectra for 5% damping is given. Response spectra can be converted in to displacement spectra as shown here. Spectral acceleration for 5% damping can be obtained from response spectra and spectral displacement can be obtained from spectral acceleration as follows (fig-2).

$$S_d = \frac{S_a}{\omega^2}$$ [Eq. 7]



Error! No text of specified style in document. -2 Displacement spectra

Spectral displacements for damping other then 5% can be determine by,

$$\Delta(T,\xi)= \Delta(T,5)\left(\frac{7}{2+\xi}\right)^{\left(\frac{1}{2}\right)} \qquad \text{[Eq. 8]}$$

4) Step-4 Calculation of base shear

$$k_e = 4\pi^2\left({m_e}/{T_e}^2\right) \qquad \text{[Eq. 9]}$$

$$m_e = \frac{\sum_{i=1}^{n} m_i\,\Delta_i}{\Delta_d} \qquad \text{[Eq. 10]}$$

Where m_e and are k_e respectively effective mass and effective stiffness of SDOF structure.

The design base shear V_B, at maximum response can be expressed as below:

$$V_B = F_u = k_e\Delta_d \qquad \text{[Eq. 11]}$$

5) Step-5 Vertical distribution of base shear

Determined base shear in accordance with the Eq. 12 is vertically distributed in proportion to vertical mass and displacement profile.

$$F_i = V_B\frac{m_i\Delta_i}{\sum_{i=1}^{n}(m_i\,\Delta_i)} \qquad \text{[Eq. 12]}$$

Where m_i, Δ_i are respectively related mass and design displacement at different story's (i)

6) Design of beams and columns

Columns are designed as per clause 7.1.2, 8.2.2, 9.3.1.3 and are checked for Combined Axial and Biaxial Bending as mentioned in clause 9.3.2.2 of IS 800:2007.

Beams are designed for Moments and Shear Forces obtained in above calculations as per clause 8.2.1.2, 8.4.1 and 9.2.2b of IS 800:2007 respectively. They are checked for deflection as mentioned in clause 5.6.1 of IS 800:2007.

B. Design Methodology of Performance Based Plastic Design

Design spectral acceleration

The design spectral acceleration “Sa/g” in the PBPD method is calculated from the Inelastic Design Spectra which is based on the inelastic behaviour of the structure. The inelastic behaviour of the structure is quantified in terms of the ductility factor “µ” ($\mu = \Theta_u / \Theta_y$) which is predefined.

For inelastic spectra spectral acceleration can be derived by below mention formula,

$$S_a=\frac{S_{ae}}{R_\mu} \qquad \text{[Eq.13]}$$

Table 1 Ductility Reduction Factor and its corresponding Structural Period Range

Period range	Ductility Reduction factor
$0 \le T < \frac{T_1}{10}$	$R_\mu = 1$
$\frac{T_1}{10} \le T < \frac{T_1}{4}$	$R_\mu = \sqrt{(2\mu_s - 1)}*\left(\frac{T_1}{4T}\right)^{2.513*\log\left(\frac{1}{\sqrt{(2\mu_s-1)}}\right)}$
$\frac{T_1}{4} \le T < T_1{}'$	$R_\mu = \sqrt{(2\mu_s - 1)}$
$T_1{}' \le T < T_1$	$R_\mu=\frac{T\mu_s}{T_1}$
$T_1 \le T$	$R_\mu=\mu_s$

Note: T_1 = 0.57 sec.(characteristic time period) ; $T_1{}' = T_1 \left(\frac{\sqrt{(2\mu_s - 1)}}{\mu_s}\right)$

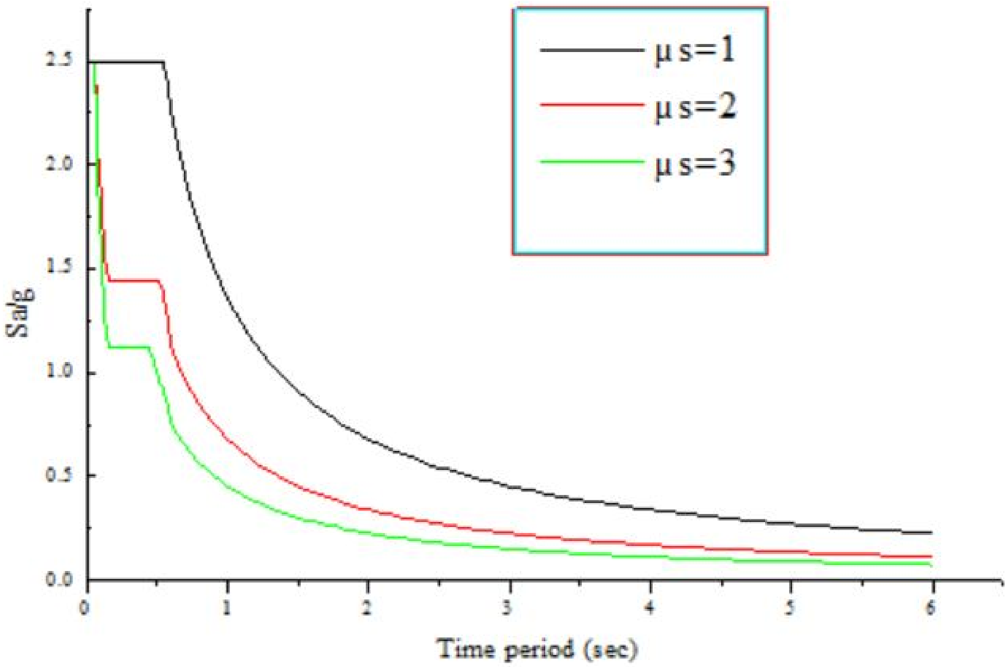


Fig.-3 Inelastic Design Spectra of varying ductile ratio for Medium Soil Site

The detailed stepwise procedure for PBPD has been mentioned below

1) *Step-1 Calculation Of Shear Distribution Factor (βI)*

$$\beta_i = \frac{V_i}{V_n}\left(\frac{\sum_{i=1}^{n} W_j\, h_j}{W_n\, h_n}\right)^{0.75T^{-0.2}} \qquad \text{[Eq. 14]}$$

Where,

V_i = Story shear force at floor i

V_n = Story shear force at roof

W_j= Seismic weight at floor j

h_j = Height of floor j from base

W_n = Seismic weight at the top floor

h_n = Height of roof from base

2) *Step-2 Calculation Of Horizontal Seismic Coefficient (α)*

$$\alpha = \sum(\beta_i - \beta_{i+1}).* h_i * \left(\frac{W_n\, h_n}{\sum_{i=1}^{n} W_j\, h_j}\right)^{0.75T^{-0.2}} * \frac{\theta p\, 8\pi^2}{T^2\, g} \qquad \text{[Eq. 15]}$$

Where,

θp = Plastic component of target drift ratio = θu - θy

3) Step-3 Calculation of Base Shear (V_B)

Base shear is calculated by Equating the work needed to push the structure monotonically up to the target drift to that of EP-SDOF system to reach the same state. For evaluating pinched hysteretic response of a structure energy modification factor needs to be considered [6]

$$V_b = W\frac{(-\alpha+\sqrt{\alpha^2+4\gamma Sa^2})}{2} \qquad \text{[Eq. 16]}$$

Where,

$\Upsilon = \frac{2\mu_s-1}{R_\mu{}^2}$ = Energy modification factor

S_a =Spectral acceleration due to inelastic response calculated by Newmark & Hall factors for different value of µ

μ_s = Structural ductility factor

R_μ = Ductility reduction factor

4) Step-4 Distribution of base shear at each floor (Q_i)

$F_i = F_n\,(\beta_i - \beta_{i+1})$ [Eq. 17]

In PBPD method, first the lateral force at roof (F_n) is calculated. Then the lateral force at each floor (F_i) is distributed with reference to the lateral force of roof.

$$F_n = \frac{V_b}{\sum(\beta_i-\beta_{i+1})} \qquad \text{[Eq. 18]}$$

C. Analysis and Design of Beam

1) *Calculation of Beam Moments by PBPD method:* As beams are to be designed as designated yielding members, the required moment capacity at each floor is determined by plastic design approach with help of figure.

Beam moment at Floor “i”

$$\beta_i\,M_{pb-positive} = \frac{\beta_i(\sum_{i=1}^{n} F_i\,h_i - 2M_{PC})}{(1+X)\,\sum_{i=1}^{n}(\beta_I\,.L/L')} \qquad \text{[Eq. 19]}$$

Where,

$\beta_i\,M_{pb-positive}$ & $\beta_i\,M_{pb-negative}$ = Probable positive and negative moment in beam

M_{PC} = Plastic moment of the columns at the base of the structure = $\frac{\psi V' h_1}{4}$

$\psi = 1.1$

V’ = Base-shear for one bay

h_1 = Height of the first storey

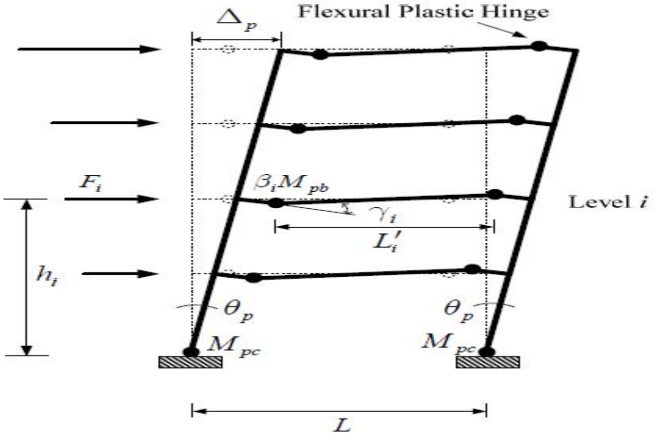


Fig.-4 Pre-Selected Yield Mechanism of 4-Story RC SMF with Beam Plastic Hinges Away From the Column Faces

D. Analysis and Design of Column

Shear forces, axial forces and moments for interior and exterior columns could be determined by free body diagram shown in figure-4. The “column tree” concept and PBPD force distribution, gives a very good estimation of maximum column moment demands when matched with severe ground motions

1) Calculation of column Moments by PBPD method

Calculate the story shear V_i and V_i'

$$V_i = \frac{(|Mu_{\text{positive}}|_i + |Mu_{\text{negative}}|_i)}{L'} + \frac{(W_{i-\text{tributary}} \cdot L')}{2} \quad \text{[Eq. 20]}$$

$$V_i' = \frac{(|Mu_{\text{positive}}|_i + |Mu_{\text{negative}}|_i)}{L'} - \frac{(W_{i-\text{tributary}} \cdot L')}{2} \quad \text{[Eq. 21]}$$

$$M_u = \xi \cdot M_{pb} = 1.5 \cdot M_{pb} \quad \text{[Eq. 22]}$$

Where, $V_i = V_i'$=Positive and negative shear force of column respectively

M_u = Design beam moment

ξ = Factor of safety

$W_{i-\text{tributary}}$= The calculated udl (as per IS 875) in the tributary

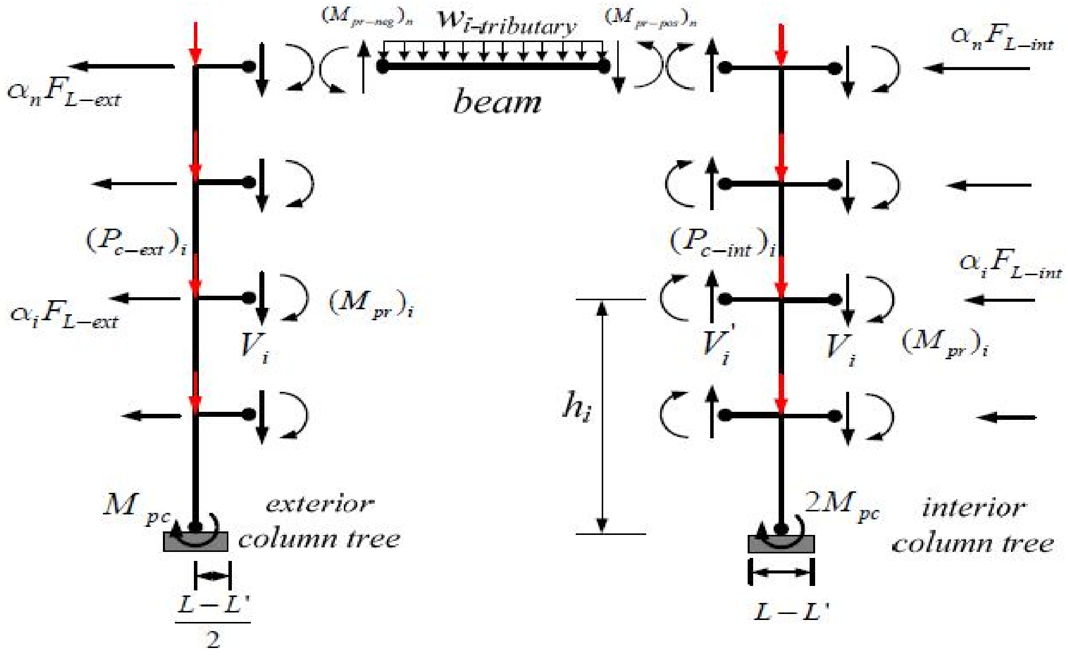


Fig.-5 The Free-Body Diagrams of Beam, Exterior Column Tree and Interior Column Tree

2) Calculation of column forces

a) External column tree

$$FL_{\text{ext}} = \frac{\sum_{i=1}^{n} (M_{u-negative})_i + \sum_{i=1}^{n} V_i \cdot (L-L'/2)_i + M_{pc}}{\sum_{i=1}^{n} \alpha_i h_I} \quad \text{[Eq. 21]}$$

b) Internal column tree

$$FL_{\text{int}} = \frac{\sum_{i=1}^{n} (M_{u-negative})_i + (M_{u-positive})_i + \sum_{i=1}^{n} \{V_i + V_i'\} \cdot (L-L'/2)_i + 2M_{pc}}{\sum_{i=1}^{n} \alpha_i h_I} \quad \text{[Eq. 22]}$$

Where,

F_L= Lateral force in column

$$\alpha_i = \frac{(\beta_i - \beta_{i+1})}{\sum_{i=1}^{n} (\beta_i - \beta_{i+1})}$$

III.PROBLEM STATEMENT AND DESIGN OF STUDY FRAME

A. Problem Statement

Design details of study frame

The study frame selected is symmetrical in both the direction having 4 bays of 5m each.

The Important design parameters are as follows:

Number of story:- 10
Outer Wall thickness:- 230mm thick
Partition Wall thickness:- 150mm thick
Thickness of slab:- 150mm
Parapet Wall thickness:- 150mm thick 1m height
Live load:- 3 kN/m^2 (Roof 2 kN/m^2) (IS: 875 (PART 2)-1987)
Floor finish:- 1.0 kN/m^2 (IS: 875 (PART 2)-1987)
Water proofing :- 1 kN/m^2
Soil Type:- Medium
Yield strength of steel :- 410Mpa

A. Design Parameters for DDBD method

Table 2
Design Parameters for Determination of Base Shear

Design Parameter			
$S_a(g)$	0.36	Ξ	14.49
T(sec)	3.75 Sec.	Δd (mm)	0.3004
Yield drift ratio Θ_y	0.00625	Δy (mm)	0.11043
target drift ratio Θ_u	0.025	Me (tonne)	1120.63
$\Theta_p = \Theta_u - \Theta_y$	0.01875	He (m)	20.26
L (m)	5	Ke	3142.06
M	2.72	V (kN)	943.89

Table 3
Displacements and Lateral Forces for determination of Base Shear

Floor	Height (m)	mi (kN)	Δi	$(\Delta i)^2$	mi.Δi	mi.$(\Delta i)^2$	mi.Δi.hi	Fi (kN)
1	3	1338	0.056	0.003	76.091	4.32	228.27	21.33
2	6	1338	0.110	0.012	147.21	16.19	883.29	41.28
3	9	1338	0.159	0.025	213.37	34.02	1920.38	59.84
4	12	1338	0.205	0.042	274.56	56.34	3294.83	77
5	15	1338	0.247	0.061	330.79	81.78	4961.95	92.77
6	18	1338	0.285	0.081	382.05	109.09	6877.05	107.14
7	21	1338	0.320	0.102	428.35	137.13	8995.44	120.13
8	24	1338	0.351	0.123	469.68	164.87	11272.42	131.72
9	27	1338	0.378	0.143	506.04	191.39	13663.3	141.92
10	30	1338	0.401	0.161	537.44	215.88	16123.4	150.72
		SUM	2.515	0.755	3365.6	1011.06	68220.35	943.89

B. *Design of Study Frame analysed by DDBD method*

1) *Design of Columns:* Columns are designed as per clause 7.1.2, 8.2.2, and 9.3.1.3 and are checked for Combined Axial and Biaxial Bending as mentioned in clause 9.3.2.2 of IS 800:2007.

2) *Design of Beams:* Beams are designed for Moments and Shear Forces obtained in above calculations as per clause 8.2.1.2, 8.4.1 and 9.2.2b of IS 800:2007 respectively. They are checked for deflection as mentioned in clause 5.6.1 of IS 800:2007.

Design parameters for PBPD method

Table 4
Design Parameters for Determination of Base Shear

Design Parameteer			
Sa(g)	0.36	W (kN)	13380
T(sec)	1.089	μ	4
yeild drift ratio Ѳy	0.00625	R_{μ}	4
target drift ratio Ѳu	0.025	ϒ	0.43
Ѳp = Ѳu -Ѳy	0.018	A	2.87
L (m)	5	V/W	0.019
L'(m)	4.2	V w/o P-Δ (kN)	261.74
$W_{tributary}$ (kN/m)	40.953	V with P-Δ (kN)	596.24

Table 5
Calculated Shear Distribution Factor

Floor	hi	Wj	hiwj	Σhiwj	Bi	βi-βi+1	(βi-βi+1)hi
10	30	1338	40140	40140.00	1.00	1.00	30.00
9	27	1338	36126	76266.00	1.60	0.60	16.33
8	24	1338	32112	108378.00	2.08	0.47	11.39
7	21	1338	28098	136476.00	2.46	0.39	8.09
6	18	1338	24084	160560.00	2.78	0.31	5.64
5	15	1338	20070	180630.00	3.03	0.25	3.78
4	12	1338	16056	196686.00	3.23	0.20	2.35
3	9	1338	12042	208728.00	3.37	0.14	1.30
2	6	1338	8028	216756.00	3.47	0.10	0.57
1	3	1338	4014	220770.00	3.51	0.05	0.14
Total		13380	220770	1545390.00	26.53113	3.51275	79.593393

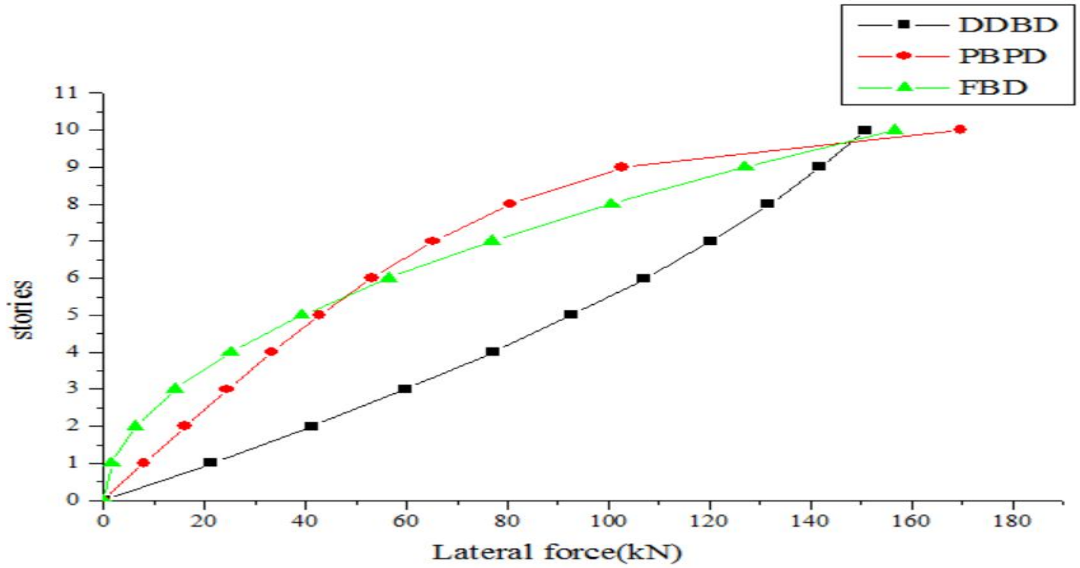
C. *Design of Study Frame analysed by PBBD method*

1) *Design of Columns:* Columns are designed as per clause 7.1.2, 8.2.2, and 9.3.1.3 and are checked for Combined Axial and Biaxial Bending as mentioned in clause 9.3.2.2 of IS 800:2007.
2) *Design of Beams:* Beams are designed for Moments and Shear Forces obtained in above calculations as per clause 8.2.1.2, 8.4.1 and 9.2.2b of IS 800:2007 respectively. They are checked for deflection as mentioned in clause 5.6.1 of IS 800:2007.

IV. RESULTS AND CONCLUSIONS

A. lateral load Comparison

The base shear obtained for 10 storey are 943.89 kN, 596.25 kN and 603.64 kN for DDBD, PBPD and FBD respectively and in accordance to that lateral forces are found which are plotted in fig-6



Fig**Error! No text of specified style in document.**-6 Lateral load comparison

B. Failure Pattern

Pushover analysis for 10 storey building is done in SAP 2000-V19 to ensure strong column weak beam mechanism. The failure pattern for frame analysed by PBPD and DDBD method has been shown in figure 7 & figure 8 respectively.

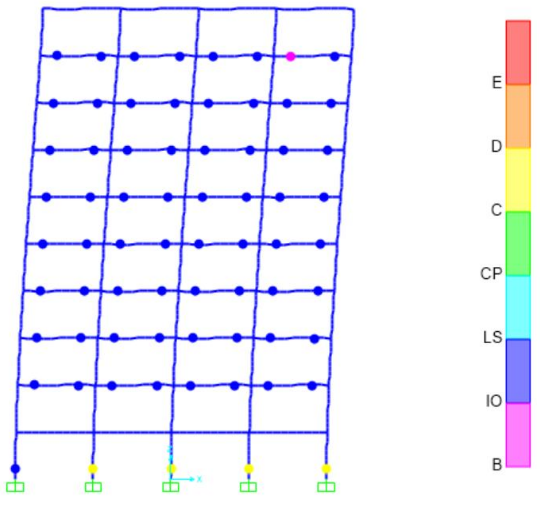


Fig.-8 formation of hinges by pushover analysis in frame designed by DDBD method

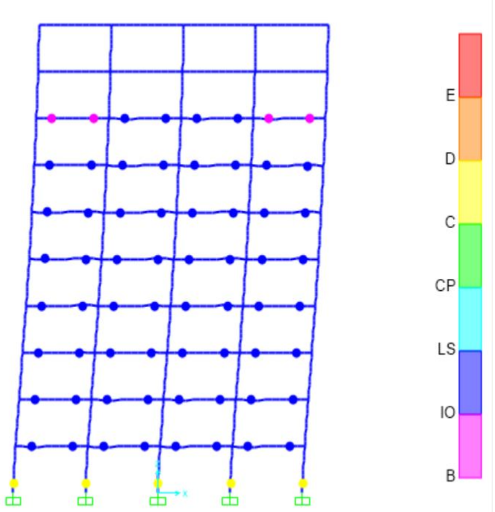


Fig.-7 Formation of hinges by Pushover analysis in frame designed by PBPD method

C. Weight comparison for frame designed by DDBD and PBPD method

The material weight comparisons of beams and columns have been done after the frame is designed in accordance with DDBD and PBPD method.

Table 6
Weight Comparison For 10 Storey frame

	Weight (Kg)		
	DDBD	PBPD	DDBD/PBPD
Beams	1622626	1252075	1.29
Columns	4480376	4541681	0.98
Total	6103002	5793756	1.05

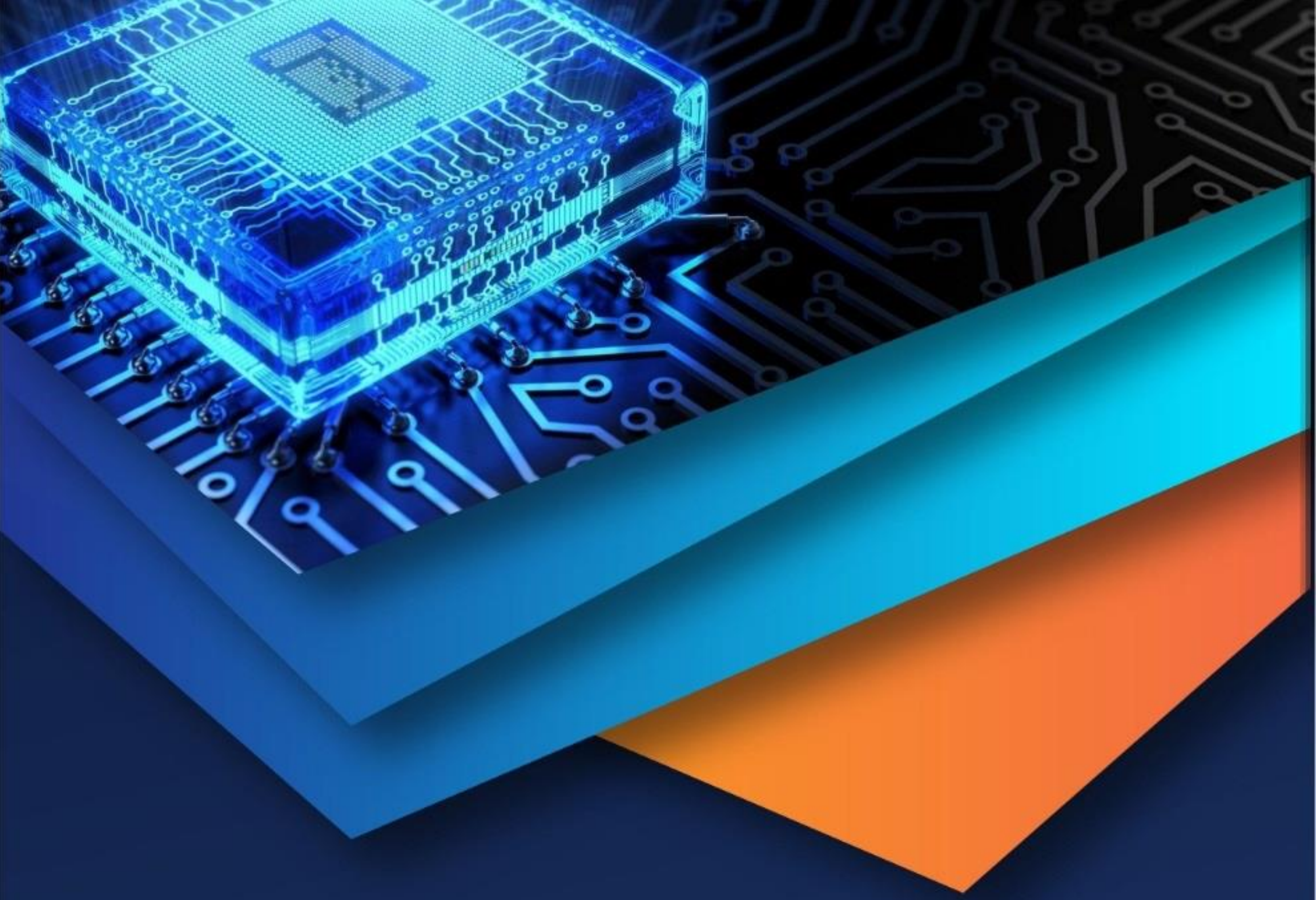
V. CONCLUSION

DDBD and PBPD methods have been successfully applied to 10 storey regular Moment Resistant frame resting on medium soil, which leads to following conclusions: -

1) Base Shear obtained by PBPD and DDBD Method for tall buildings are much more than that of IS Code method which indicates that seismic demands are sometimes underestimated in IS Code method.
2) In frames designed by DDBD and PBPD method, Hinges are formed only in beams and not in columns indicating Strong column - Weak Beam holds true as expected and total collapse of building is prevented.
3) Weight comparison of frames designed by DDBD and PBPD is done which shows that material weight for Steel moment resistant frames are almost equal.

REFERENCES

[1] Aidcer L. Vidot-vega , Mervyn J. Kowalsky 2013. “Drift, strain limits and ductility demands for RC moment frames designed with displacement-based and force-based design methods”. ELESEVIER, engineering structures 51 (2013) 128-140.
[2] Chao, S.-H., Goel, S. C., and Lee S.-S., August 2007 “A Seismic Design Lateral Force Distribution Based on Inelastic State of Structures,” Earthquake Spectra, Earthquake Engineering Research Institute, Vol. 23, No. 3.
[3] Eurocode 8: Seismic Design of Buildings Worked examples.
[4] G.M. Calvi, M.J.N. Priestley, and M.J.Kowalsky.2008,“Displacement–Based Seismic Design of Structure ”
[5] Goel S. C., Liao W.-C., Mohammad, R. B and Chao, S.-H, “Performance-Based Plastic Design (PBPD) Method for Earthquake-Resistant Structures: An Overview”, Structure Design Tall and Special Buildings in Wiley Interscience, October 2009.
[6] Gulkan, P. & Sozen M.A. 1974. Inelastic responses of reinforced concrete structures to earthquakes motions. Proceedings of the ACI 71 (12).
[7] Gupta, A. & Krawinkler, H. 2002. Relating the seismic drift demands of SMRFs to element deformation demands. Engineering Journal 39(2), 100-108.
[8] IS 1893:2000, “Criteria for Earthquake Resistant Design Of Structures”, Bureau Of Indian Standards, New Delhi, India.
[9] Liao, W.C. Goel, S.C, 2012. “Performance Based Plastic Design and Energy Based Evaluation of Seismic Resistant RC Moment Frame”, Journal of marine science and technology, Vol. 20 ,pp304-310
[10] M J N PRIESTLEY. 2000. “PERFORMANCE BASED SEISMIC DESIGN” 14th world conference of earthquake engineering,. University of California, San Diego.
[11] M.J.N. Priestley and M.J. Kowalsky, ”Direct displacement-based seismic design of concrete buildings”,bulletin of the new zealand society for earthquake engineering,vol. 33,no. 4,december 2000
[12] M. R. Banihashemi, A. R. Mirzagoltabar and H. R. Tavakoli “The Effects of Yield Mechanism Selection on the Performance based Plastic Design of Steel Moment Frame”, Indian Journal of Science and Technology, Vol 8(S9), 158–171, May 2015
[13] M. Reza Banihashemi, A. R Mirzagoltabar, and H. R Tavakoli 2015. “Development of the Performance Based Plastic Design for Steel Moment Resistant Frame”,International Journal of Steel Structures 15(1): 51-62 (2015)
[14] Mondal, A. Ghosh, S. and Reddy, G.R, July 2013. “Performance-based evaluation of the response reduction factor for ductile RC frames”, Engineering Structures, Vol. 56, pp1808-19.
[15] Peter Fajfar, M.EER, “A Nonlinear Analysis Method for Performance Based Seismic Design” Earthquake Spectra, Vol.16, No.3, pp.573-592, August 2000.
[16] Priestley, M.J.N. 2003. Myths and fallacies in earthquake engineering, revisited. The Mallet Milne Lecture. IUSS Press, Pavia, Italy.
[17] S. MALEKPOUR, H. GHAFFARZADEH, and F. DASHTI “Direct Displacement Based Design of Regular Steel Moment Resisting Frames”, ELESEVIER. ScienceDirect, Procedia Engineering 14 (2011) 3354–3361
[18] S. Malekpour, H. Ghaffarzadeh, and F. Dashti 2011 “Direct Displacement Based Design of Regular Steel Moment Resisting Frames ”, ELESEVIER . ScienceDirect,Procedia Engineering 14- 3354–3361
[19] Saleh Malekpour and Farhad Dashti. June 2013. “Application of the Direct Displacement Based Design Methodology for Different Types of RC Structural Systems” International Journal of Concrete Structures and Materials Vol.7, No.2, pp.135–15
[20] Sejal P. Dalal, Sandeep A. Vasanwala, Atul K. Desai 2012. “Applying Performance Based Plastic Design Method to Steel Moment Resisting Frame in Accordance with the Indian Standard Code”,International Journal of Engineering and Technology Volume 2 No. 3, March, 2012
[21] Shibata,A. and Sozen,M. “Substitute Structure Method for seismic design in reinforced concrete.”Journal, Structural Division, ASCE Vol. 102(6) 1976.
[22] shih-ho chao, meeri, subhash c. goel , m.eeri, and soon- sik lee, a seismic design lateral force distribution based on inelastic state of structures, earthquake spectra, volume 23, no. 3, august 2007,earthquake engineering research institute.
[23] T.L. KARAVASILIS,N. BAZEOS,D.E.E BESKOS “Maximum displacements profiles for the performance based seismic design of plane steel moments resisting frames”, ELESEVIER. Engineering Structures 28 (2006) 9–22



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