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PERFORMANCE BASED PLASTIC DESIGN OF L-SHAPED RCC FRAME

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Abstract — *If structure is designed for seismic loads as per current code method, it generally satisfies the strength and serviceability criteria. But during strong ground shaking, it may undergo total collapse. The PBPD method can prove better as it prevents total collapse of the structure by designing it for a predetermined failure and yield mechanism (strong column-weak beam principle). As total collapse is prevented in this method, it is gaining popularity worldwide. In our country, the method still needs recognition and hence the study aims on proposing a PBPD method for RCC frames attuned with our code. The PBPD method differs from current code method mainly in terms of analysis, i.e. calculation of forces and moments. As the analysis method in PBPD method is based on basic equation of equilibrium. It can be directly implemented. The design can then carry out satisfying the IS 456:2000 code. The PBPD design proposed for Indian designers is further made clear by designing a 15 story L-shaped RCC frame.*

Keywords- *PBPD; FBD; ENERGY BALANCED CONCEPT; SEISMIC EVALUATION; PUSHOVER ANALYSIS.*

I. INTRODUCTION

Performance-Based Plastic Design (PBPD) method is based on "STRONG COLUMN-WEAK BEAM" principle. This design concept uses pre-selected target drift and yield mechanism as performance criteria.

In the current Indian design practice the structure is designed for forces and then checked for displacements. The current force-based approach using an elastic design spectrum to obtain the seismic strength of a structure starts with an estimate of the lateral stiffness, or equivalently the period of the structure. Then, the elastic strength is obtained from the elastic design spectrum, based on the perceived period by taking ductility factor μ=1 (μ=(μ-max)/(μ-yield)=(Ө-max)/(Ө-yield)). The elastic strength is then modified by a reduction factor R to arrive at the design base shear. In this approach, it is assumed that the original stiffness estimate will not be affected when the design strength is reduced substantially from the elastic strength.

The design base shear is usually a fraction of the elastic strength. For a system where its strength and stiffness are coupled parameters, a reduction in design strength from elastic strength would lead to a corresponding reduction in stiffness. This would in turn increase the period of the system. Therefore, the actual elastic demand to design base shear ratio will be less than R as originally envisioned. Our current IS 1893:2002 provides the "Sa/g" values for the elastic response of the structure, i.e. ductility factor μ=1. The base shear is obtained by reducing it for reduction factor R. This calculation of base shear is approximate. These response reduction factors do not predict the exact inelastic response of the structure (Victor and Federico, 2003) because same value of "R" is assigned to all the system falling under a defined category for available ductility. Although two systems may have the same value of "R", it may be appropriate to assign a higher reduction factor to a system with greater ductility (SEAOC, 2008). In the PBPD method, the Sa/g values are calculated for desired value of μ and hence no reduction factor is required.

II. PBPD METHOD AND ITS ADVANTAGE

The design base shear for a specified hazard, which is generally given as design spectrum in the codes, is calculated by equating the work needed to push the structure monotonically up to the target drift to the energy required by an equivalent EP-SDOF to achieve the same state (Figure 2.1). Also, a new distribution of lateral design forces is used that is based on relative distribution of maximum storey shears consistent with inelastic dynamic response results (Chao et al.2007). Plastic design is then performed to detail the frame members and connections in order to achieve the intended yield mechanism and behaviour. Thus, determination of design base shear, lateral force distribution and plastic design are three main components of the PBPD method.

It should be noted that in this design approach, the designer selects the target drifts consistent with acceptable ductility and damage, and a yield mechanism for desirable response and ease of post-earthquake damage inspection and reparability. The design lateral forces are determined for the given seismic hazard (design spectrum) and selected target drift. Thus, there is no need for factors, such as Z, I and R etc., as required in the current design codes.

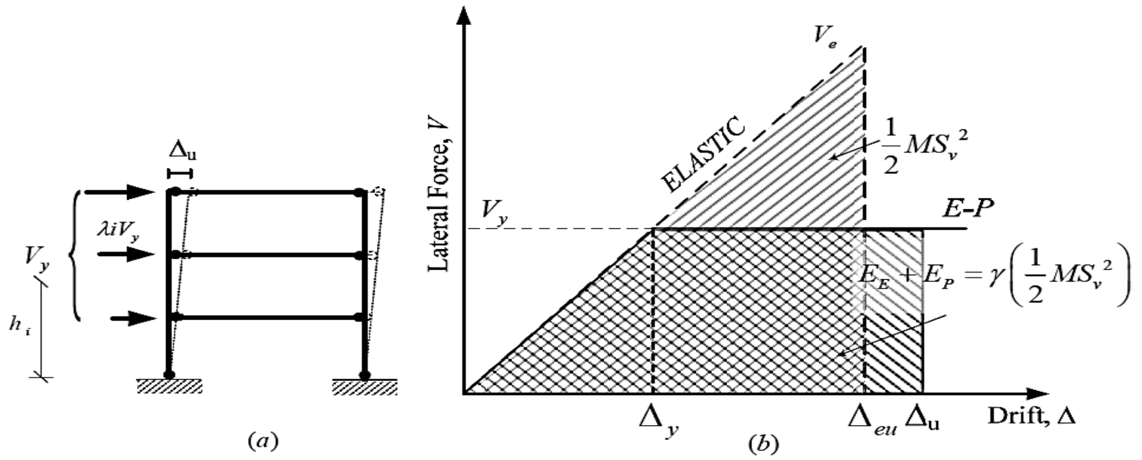

Figure 2.1: PBPD Concepts.

It is important to note that in the PBPD method, control of drift and yielding is built into the design process from the very start, eliminating or minimizing the need for lengthy iterations to arrive at the final design. Other advantages include the fact that innovative structural schemes can be developed by selecting suitable yielding members and/or devices and placing them at strategic locations, while the designated non-yielding members can be detailed for no or minimum ductility capacity. All of these would translate into enhanced performance, safety and economy in life-cycle costs.

The method has been successfully applied to steel moment frame (Lee et al.), buckling restrained braced frame (BRBF) (Dasgupta et al.), Eccentrically braced frame (EBF) (Chao and Goel.), Special truss moment frame (STMF) (Chao and Goel.), and concentric braced frame (CBF) (Chao et al.). The main advantage of PBPD method is that, it is the direct design method without the need for iteration to achieve the desire targeted performance in terms of drift and yield mechanism control. Other advantage include the fact that innovative structure schemes can be developed by selecting suitable yielding members device and placing them at strategic location, while the designed non yielding members can be detailed for no or minimum ductility capacity.

Reinforced concrete moment resisting frames (RC MRF) comprise of horizontal framing components (beams and slabs), vertical framing components (columns) and joints connecting horizontal and vertical framing components that are designed to meet the special requirements given in seismic codes (e.g., IS 1893(Part1):2002, IS 13920:2002). Those special proportioning and detailing requirements are intended to make the frames capable of resisting strong earthquake shaking without significant loss of stiffness or strength. However, the losses due to structural and nonstructural damage in code compliant buildings have led to the awareness that current seismic design methods are not always able to provide the desired and satisfactory performance.

III. PBPD METHODOLOGY

3.1. Lateral force calculation and distribution.[13]

Step 1: Calculation of shear distribution factor (β_i).

$$\beta_i = \left(\frac{V_i}{V_n}\right) = \left(\frac{\sum_{i=1}^{n} W_j h_j}{W_n h_n}\right)^{0.75T^{-0.2}} \quad \text{(Chao, 2007).}$$

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.

Step 2: Calculation of horizontal seismic coefficient (A_h).

$$A_h = \frac{h_* \, \Theta_p \, 8\pi^2}{T^2 \, g}$$

(Chao, 2007).

Θ_p = Plastic component of target drift ratio. = $\Theta_u - \Theta_y$.

$h* = \sum_{i=1}^{n} (\lambda_i h_i)$.

λ_i = 0.78. (Wen-Cheng 2010).

Step 3: Calculation of Base Shear (V_b).

$$V_b = \frac{-A_h + \sqrt{A_h{}^2 + 4\gamma S a^2}}{2}$$

Where,

$\gamma = \frac{2\mu_s - 1}{R_\mu{}^2}$ = 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.

Table 3- 1 Ductility reduction factor and its corresponding structural period range.[13]

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

Note: $T_1 = 0.57$ sec.; $T_1' = T_1 \cdot \left(\sqrt{2\mu_s - 1} / \mu_s\right)$ sec.

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

$$Q_i = Q_n (\beta_i - \beta_{i+1})$$

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

$$Q_n = \frac{V_b}{\Sigma(\beta_i - \beta_{i+1})}$$

3.2 Calculation of beam and column moments.

The analysis of RC Frame in PBPD method is done by assuming that a hinge will be formed in beams only at distance 0.1L to 0.2L from column face. The calculation of beam moments is done by simple equation of equilibrium.

Step 1: Calculate the required beam moment strength at Floor "i".

$$\beta_i\, M_{pb\text{-}positive} = \frac{\beta_i\,(\sum_{i=1}^{n} Q_i h_i - 2M_{pc})}{(1+X)\sum_{i=1}^{n}(\beta_i \cdot \frac{L}{L'})}$$

Where,

$M_{pb\text{-}positive}$= $M_{pb\text{-}negative}$= Probable positive and negative moment in beam.

M_{pc} = Plastic moment of the columns at the base of the structure = $\frac{\psi V' h1}{4}$.

ψ = 1.1

V' = Base-shear for one bay.

h_1 = Height of the first storey.

X = Ratio of $M_{pb\text{-}negative}$ to $M_{pb\text{-}positive}$ = 2.1.

Step 2: Calculate the story shear V_i and V_i'.

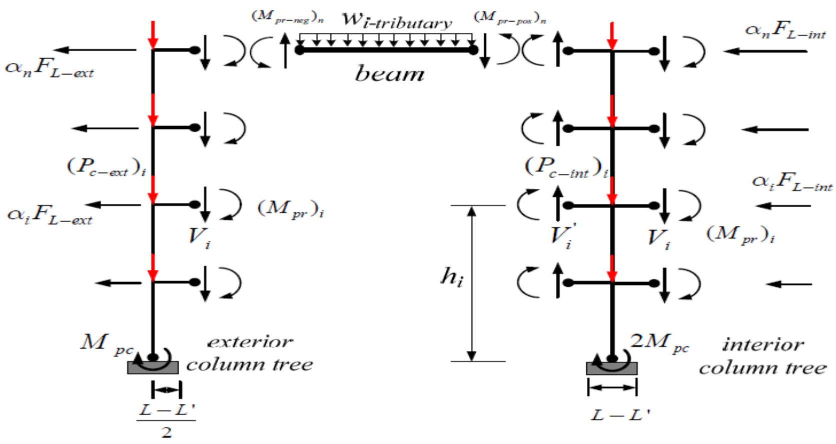

Figure: 3.1 The free-body diagrams of beam, exterior column tree and interior column.

$$V_i = \frac{|M_{u-positive}|_i + |M_{u-negative}|_i}{L'} + \frac{W_{i-tributary}}{2} L'$$

$$V_i' = \frac{|M_{u-positive}|_i + |M_{u-negative}|_i}{L'} - \frac{W_{i-tributary}}{2} L'$$

$$M_u = \xi \cdot M_{pb} = 1.5 \cdot M_{pb}$$

Where,
V_i = V_i'=Positive and negative Shear force of column respectively.
M_u = Design beam moment.
ξ = Factor of safety (1.5 as per our code and 1.25 as per NEHRP, FEMA356).
$W_{i\text{-}tributary}$ = The calculated udl (as per IS 875) in the tributary.
The design of beam for above values of moment and shear is done as per IS 456:2000.

Step 3: Calculation of column Forces.

(a) External column tree.

$$F_{L\text{-ext}} = \frac{\sum_{i=1}^{n} (M_{u-negative})_i + \sum_{i=1}^{n} V_i (\frac{L-L'}{2})_i + M_{pc}}{\sum_{i=1}^{n} \alpha_i h_i}$$

(b) Internal column tree.

$$F_{L\text{-int}} = \frac{\sum_{i=1}^{n} |M_{u-negative}|_i + |M_{u-positive}|_i + \sum_{i=1}^{n} \{V_i + V_i'\}(\frac{L-L'}{2})_i + 2M_{pc}}{\sum_{i=1}^{n} \alpha_i h_i}$$

Where,
F_L= Lateral force in column.
$\alpha_i = \frac{(\beta_i - \beta_{i+1})}{\sum_{i=1}^{n} (\beta_i - \beta_{i+1})}$.

The design of Column for above values of moment and forces is done as per IS 456:2000.

IV. PROBLEM STATEMENT

L-SHAPED SCHOOL BUILDING.
Number of story 15.
All walls are 115mm thick.
Thickness of slab 150mm.
Live load 2.5kN / m^2 (IS:875 (PART 2)-1987).
Floor finish 1.0kN / m^2 (IS:875 (PART 2)-1987).
Density of brick masonary 20kN / m^3 (IS:875 (PART 1)-1987).
Density of concrete 25kN / m^3(IS:875 (PART 1)-1987).
Medium soil site.
Vadodara.

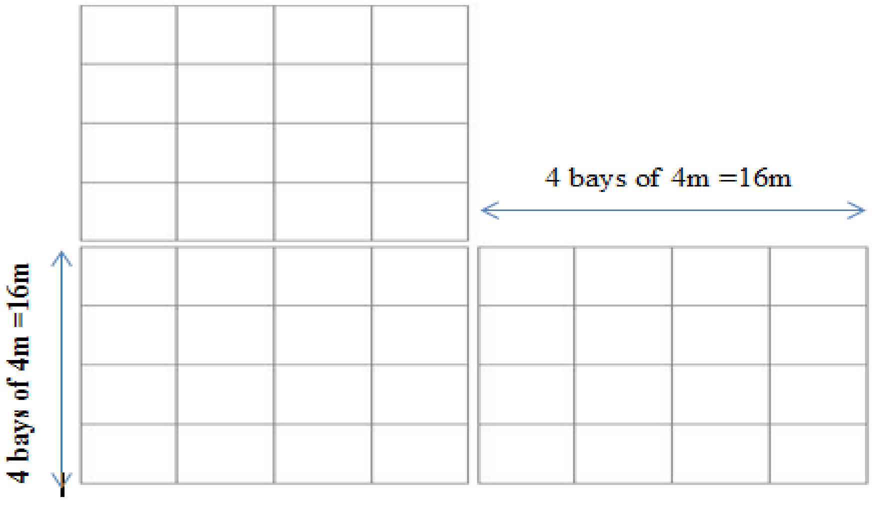

Figure: 4.1 Plan of building.

4.2 LOAD CALCULATION.

Assumed data
Size of column 300mm*600mm.
Size of beam 230mm*450mm.
Weight of Slab = 0.15*16*16*25=960 KN.
Weight of Column= 0.3*0.6*25*25=112.5 KN/m.
Weight of Beam = 0.23*0.45*16*10*25=414 KN.
Weight of Wall = 0.115*16*10*20=368 KN/m.

Lumped mass (w/o LL) = 960+414+(112.5*4)+(368*4) = 3296 KN.
Floor finish = 1*16*16 = 256 KN.
W_{roof} = 3296+256 = 3552 KN.
Live load reduction (**IS 875-PART 2, pg. no.12).**
Live load = 2.5*16*16=640 KN.

Table: 4.1 Load Calculation.

FLOOR	DL(KN)	LL*Reduction factor(KN)	Total load(W_i)(KN)
R	3552	0	3552
14	3552	640*0.5=320	3872
13	3552	640*0.5=320	3872
12	3552	640*0.5=320	3872
11	3552	640*0.5=320	3872
10	3552	640*0.6=384	3936
9	3552	640*0.6=384	3936
8	3552	640*0.6=384	3936
7	3552	640*0.6=384	3936
6	3552	640*0.6=384	3936
5	3552	640*0.6=384	3936
4	3552	640*0.7=448	4000
3	3552	640*0.8=512	4064
2	3552	640*0.9=576	4128
1	3552	640*1=640	4192

Yield Drift Ratio θ_y	Target Drift Ratio θ_u	L(m)	L′(m)	$W_{tributary}$ (KN/m)	W(KN)
0.004 (IS 1893:2002,pg no.27)	0.02	4	3.2	63.5	59040

Table:4.2 The design parameters for 15-story PBPD RC SMF.

TABLE: 4.3 REINFORCEMENT DESIGN OF BEAM.

				Floor 1 to 10		Floor 11 to 13		Floor 14		roof	
b(mm)				300		300		300		230	
d(effective)(mm)				700		550		450		400	
FLOOR	Mu	Mu/bd^2	P_t	P_c	A_{st}	No of	No of	A_{sc}	No of	No of	No of	No of
	(KN·m)	(N/mm^2)	(%)	(%)	(mm^2)	25mm	20mm	(mm^2)	20mm	16mm	12mm	8mm
R	157.9	4.29	1.456	0.276	1339.52	-	5	253.92	-	-	3	-
14	259.22	4.27	1.445	0.262	1950.75	4	-	353.7	-	2	-	-
13	340.01	3.75	1.285	0.09	2120.25	5	-	148.5	-	-	-	3
12	407.98	4.5	1.517	0.341	2503.05	6	-	562.65	2	-	-	-
11	466.37	5.14	1.72	0.56	2838	6	-	924	3	-	-	-
10	517.74	3.52	1.22	0.025	2562	6	-	52.5		-	-	-
9	562.21	3.82	1.31	0.12	2751	6	-	252	-	-	3	-
8	600.55	4.08	1.39	0.211	2919	6	-	443.1	-	3	-	-
7	633.27	4.31	1.456	0.278	3057.6	7	-	583.8	2	-	-	-
6	660.77	4.5	1.517	0.341	3185.7	7	-	716.1	3	-	-	-

5	683.33	4.65	1.565	0.39	3286.5	7	-	819	3	-	-	-
4	701.44	4.77	1.595	0.435	3349.5	7	-	913.5	3	-	-	-
3	715.12	4.86	1.625	0.465	3412.5	7	-	976.5	4	-	-	-
2	724.31	4.93	1.65	0.48	3465	8	-	1008	4	-	-	-
1	728.96	4.96	1.655	0.485	3475.5	8	-	1018.5	4	-	-	-

Reinforcement criteria for beam as per IS 456:2000.

Tension reinforcement

Minimum reinforcement

$$A_{st} = \frac{0.85 * b * d}{fy}$$

Maximum reinforcement

A_{st} = 0.04*b*D

Compression reinforcement shall not exceed 0.04bD.

Shear Reinforcement Criteria (IS 13920:1993):
Spacing of hoops over a length of 2d at either end of beam shall not exceed

a) d/4 where, d= depth of beam.
b) $8d_b$ d_b = diameter of bar.

It need not be less than 100mm.
Hoop spacing for center of span is not greater than d/2.

Table 4.4 Column Shear and Moments.

	Exterior column			Interior column		
FLOOR	Axial load (KN)	Shear (KN)	Mu (KN.m)	Axial load (KN)	Shear (KN)	M_u (KN.m)
R	131	57.1913319	228.96	262	87.6206525	291.36
14	262	93.1977531	401.57	524	142.969257	524
13	393	121.107195	525.84	786	186.015548	684.32
12	524	143.776485	618.98	1048	221.102346	797.87
11	655	162.46802	690.61	1310	250.141742	878.81
10	786	178.180853	748.28	1572	274.655375	938.35
9	917	191.135062	795.79	1834	294.961194	981.92
8	1048	201.738964	836.91	2096	311.674934	1015
7	1179	210.321389	874.41	2358	325.292287	1041
6	1310	217.157557	910.15	2620	336.227953	1064.52
5	1441	222.483949	945.25	2882	344.837907	1085.5
4	1572	226.564941	980.48	3144	351.527042	1105.4
3	1703	229.548164	1015.59	3406	356.51312	1124.09
2	1834	231.571647	1049.67	3668	359.996074	1139.27
1	1965	232.766035	1081.18	3930	362.15622	1149.85

Table 4.5 Exterior column design.

FLOOR	Exterior column					no of 25mm	b(mm)	D(mm)
	M_u/bD^2f_{ck}	P_u/bDf_{ck}	P_t/f_{ck}	$P_t(\%)$	$A_{st}(mm^2)$			
R	0.07	0.03	0.04	1	2500	5	500	500
14	0.07	0.04	0.04	1	3600	7	600	600
13	0.10	0.07	0.06	1.5	5400	11	600	600
12	0.11	0.09	0.06	1.5	5400	11	600	600
11	0.13	0.11	0.08	2	7200	15	600	600
10	0.14	0.13	0.09	2.25	8100	17	600	600
9	0.15	0.15	0.1	2.5	9000	18	600	600
8	0.10	0.13	0.05	1.25	6125	12	700	700
7	0.10	0.14	0.05	1.25	6125	12	700	700
6	0.11	0.16	0.06	1.5	7350	15	700	700
5	0.11	0.18	0.06	1.5	7350	15	700	700
4	0.11	0.19	0.06	1.5	7350	15	700	700
3	0.12	0.21	0.07	1.75	8575	17	700	700
2	0.12	0.22	0.07	1.75	8575	17	700	700
1	0.13	0.24	0.08	2	9800	20	700	700

Table 4.6 Interior column design.

FLOOR	Interior column					no of 25mm	b(mm)	D(mm)
	M_u/bD^2f_{ck}	P_u/bDf_{ck}	P_t/f_{ck}	$P_t(\%)$	$A_{st}(mm^2)$			
R	0.09	0.06	0.05	1.25	3125	6	500	500
14	0.10	0.09	0.06	1.5	5400	11	600	600
13	0.08	0.10	0.04	1	4900	10	700	700
12	0.09	0.13	0.04	1	4900	10	700	700
11	0.10	0.16	0.05	1.25	6125	12	700	700
10	0.11	0.19	0.06	1.5	7350	15	700	700
9	0.11	0.22	0.06	1.5	7350	15	700	700
8	0.08	0.20	0.03	0.75	4800	10	800	800
7	0.08	0.22	0.03	0.75	4800	10	800	800
6	0.08	0.25	0.03	0.75	4800	10	800	800
5	0.08	0.27	0.03	0.75	4800	10	800	800
4	0.09	0.29	0.05	1.25	8000	16	800	800
3	0.09	0.32	0.05	1.25	8000	16	800	800

2	0.09	0.34	0.06	1.5	9600	20	800	800
1	0.09	0.37	0.06	1.5	9600	20	800	800

V. PUSHOVER ANALYSIS

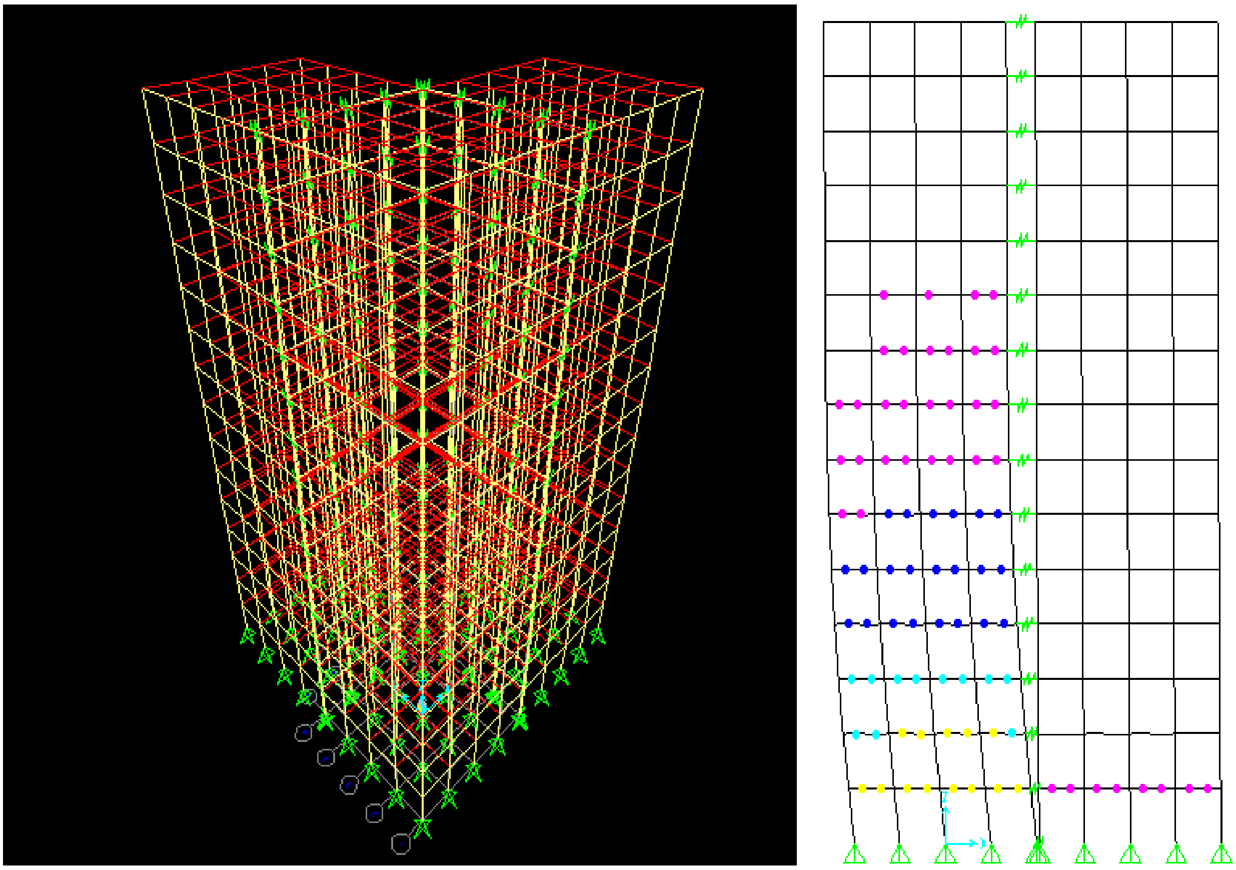

Figure 5.1 Model and failure pattern by pushover analysis using SAP2000.

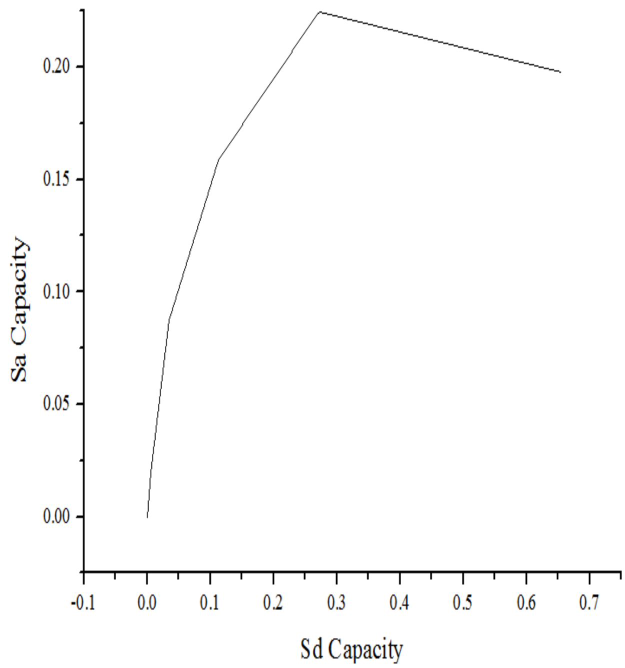

Figure 5.2 Capacity curve for 15 storey building

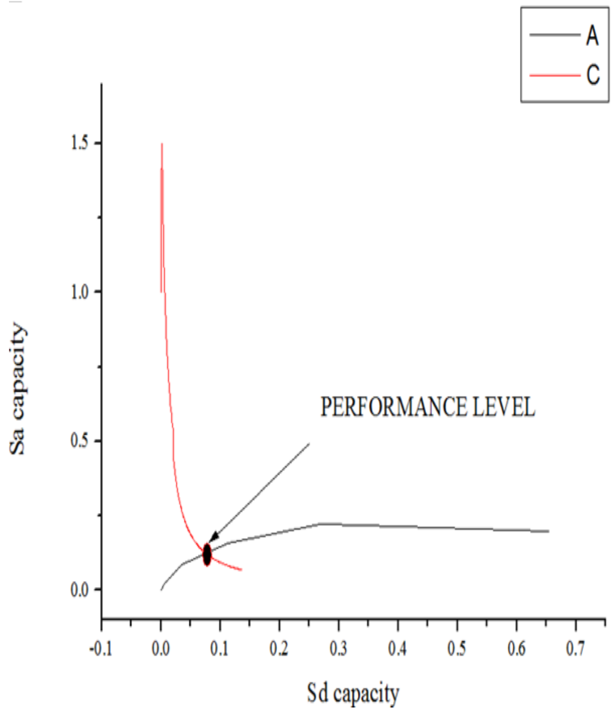

Figure 5.3 Performance level of 15 storey frame.

CONCLUSION

The main weakness of current seismic design code for RC SMF is lack of guidance to provide the engineers as to how to achieve the desired goals such as, controlling drifts, distribution and extent of inelastic deformation, etc. In contrast, the PBPD method is a direct design method, which requires no evaluation after the initial design because the nonlinear behavior and key performance criteria are built into the design process from the start. Following points were observed during the whole research:

1. These lateral forces are distributed according new distribution factor defined on the basic of real ground motions.
2. Values of lateral forces are higher compared to code specified lateral force distribution which gives conservative results and better performance.
3. Columns are designed for higher moments compared to beam which fulfill the “strong column-weak beam” principle.
4. Failure of frame occurs only at predefined beam location, and not in columns, which prevents the total collapse of structure increase life safety.

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