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Performance Based Seismic Design of Reinforced Concrete Moment Resistant Frame with Vertical Setback

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***Abstract*— A performance-based seismic design (PBSD) method is aimed at controlling the structural damage based on precise estimations of proper response parameters. PBSD method evaluates the performance of a building frame for any seismic hazard, the building may experience. This paper gives a comparison between Performance based Seismic design and conventional design method (using I.S 1893; 2002) for irregular RC building frames (10 storeys) and evaluates performance using pushover and Time History analysis.**

I. INTRODUCTION

Earthquakes have the potential for causing the greatest damages, among all the natural hazards. Since earthquake forces are random in nature & unpredictable, need of some sophisticated methods to analyze our structures for these forces. Performance based design can relate to a new dimension in the seismic design philosophy. We need to carefully understand and model the earthquake forces to study the actual behavior of structure so that structure faces a controlled damage. India has witnessed more than 690 earthquakes of Richter magnitude (‘M’) greater than 5 during 1828 to 2010. Damage survey reports show that life and property losses occur in urban and semi-urban areas. It is uneconomical to design a building so as not to suffer any damage during strong earthquake. An engineering approach aims for achieving balance in cost and performance through controlled damage. The goal of performance-based seismic design is to ensure that performance objectives are satisfied. A successful conceptual design could hopefully reduce the impact of uncertainties on the real structural behaviour.

II. PERFORMANCE BASED SEISMIC DESIGN OF REINFORCED CONCRETE MOMENT RESISTANT FRAME:

Reinforced Concrete Building stock in India is mainly classified from low to medium rise buildings. Approach of I.S 1893 is in tune with typical code practice followed by many other countries. In spite of knowing drawbacks of force based seismic design procedures, the practice is in vogue due to its simplicity and non-availability of the alternative. We can use guidelines given by FEMA and ATC documents by modifying them for Indian condition. The objective of this study is to develop and validate a seismic design methodology for Reinforced Concrete Moment Frame which enables us to produce structures of seismic performance which is predictable and intended. Based on performance limit states of target drift and desired yield mechanism, this design methodology accounts for inelastic structural behaviour directly, and practically eliminates the need for assessment or iteration by nonlinear static or time-history analysis after initial design. The methodology for steel frames has been developed by Goel et al., in recent years (1999~2008). It is called Performance-Based Plastic Design (PBPD) method.An outline of the step-by-step Performance-Based Seismic Design (PBSD) procedure is given in the following.

A. Design Procedure

1. Select a desired yield mechanism and target drift for the structure for the design earthquake hazard.

2. Estimate the yielding drift, ‘θy’, the fundamental period, ‘T’, of the structure and determine an appropriate vertical distribution of design lateral forces.

3. Determine the elastic design spectral acceleration value, ‘Sa’ (Fig 1), by multiplying seismic response coefficient, ‘C s’, with $\frac{R}{I}$

4. Calculate the design base shear, ‘V’. In order to estimate the ductility reduction factor and the structural ductility factor, an inelastic seismic response of EP-SDOF is needed, such as idealized inelastic response spectra by Newmark-Hall (1985) used in this study.

5. Modify ‘V’ for Reinforced Concrete MF as needed since the force-deformation behavior is different from the assumed EP behavior and P-Delta effect is not considered in the calculation of ‘V’ in Step 4.

6. Use plastic method to design the designated yielding members (DYM), such as beams in Reinforced Concrete Moment Frames. Members that are required to remain elastic (non-DYM), such as columns, are designed by a capacity design approach.

B. *Determination of Fundamental Period*

The fundamental period, 'T', in seconds, for Reinforced Concrete MF can be determined from the following equation, as given in ASCE 7-05 (2006)

$$T = C_u . T_a = C_u . C_t . h_n^x$$

$$T_{actual\ /model} > C_u . C_t . h_n^x \qquad (1)$$

where 'Ta' is the approximate fundamental period per ASCE 7-05 (2006) section 12.8.2.1; 'Cu' represents the coefficient for upper limit on calculated period, and for SD1 ≥0.3g , 'Cu' is 1.4 (Table 12.8-1 in ASCE 7-05); 'hn' is the height in feet above the base to the highest level of the structure and the coefficient 'Ct' and 'x' for concrete moment resistant frames are 0.016 and 0.9

(Table 12.8-2 in ASCE 7-05), respectively.

C. *Design Base Shear*

Assuming an idealized E-P force-deformation behavior of the system as shown in figure, the work-energy equation can be written as:

$$(E_e + E_p) = \gamma.\left(\frac{1}{2} M. S_v^2\right) = \frac{1}{2}\gamma. M. \left(\frac{T}{2\pi} S_a g\right)^2 \qquad (2)$$

where Ee and Ep are, respectively, the elastic and plastic components of the energy (work) needed to push the structure up to the target drift. 'Sv' is the design pseudo-spectral velocity; 'Sa' is the pseudo spectral acceleration, which can be obtained from the seismic design response spectrum in ASCE 7-05 (2006) 'With the assumed yield drift 'θy' for different structural systems (Table 1), the energy modification factor, 'γ' , depends on the structural ductility factor ('μs') and the ductility reduction factor (' Rμ') and can be obtained from the following relationship.

Table No .1 Assumed design yield drift ratios as given in ASCE7

Frame Type	Reinforced Concrete	Steel			
	SMF	MF	EBF	STMF	CBF
Yield Drift ratio 'θ_y'(%)	0.5	1	0.5	0.75	0.3

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \qquad (3)$$

Plots of energy modification factor 'γ' as obtained from Equation 3 are also shown in Figure 3.3(b) (Lee and Goel, 2001).. Other inelastic spectra for EPSDOF systems can also be used as preferred, such as those by Miranda and Bertero (1994).

Table No 2 Ductility reduction factor and its corresponding structural period range

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

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

III. C2 METHOD FOR MODIFICATION OF TARGET DRIFT

After studying the hysteretic (degradation of strength and stiffness) it is revealed that the Peak displacements for non-degrading frames are large for short periods but are equal for longer periods as that of degrading frames. The coefficient 'C2' is a modification factor to represent the effect of pinched shape of hysteretic loops, stiffness degradation, and strength deterioration on the maximum displacement response according to FEMA 356. The equations of simplified linear regression trend line of 'C2'for different force reduction factor, R, are summarized in Table below.

Table No 3 C2 factor

	$0.2 <= T < 0.4$	$0.4 \le T < 0.8$	$0.8 \le T$
R= 3.0 ~ 6.0	$3.0 - 7.5 \cdot (T - 0.2$	$1.5 - 1.0 \cdot (T - 0.4)$	$1.1 - 0.045 \cdot (T - 0.8)$
R= 2.0	$2.5 - 6.5 \cdot (T - 0.2)$	$1.1 - 0.077 \cdot (T - 0.4)$	

After determining the value of 'C2' , the modified target design drift 'θ u' , ductility 'μ_s'

Ductility reduction factor 'Rμ'and energy modification factor 'γ' can be calculated as follows:

$$\theta u = \frac{\theta t}{C\,2} \qquad (4)$$

$$\mu_s = \frac{\theta u}{\theta y} \qquad (5)$$

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \qquad (6)$$

A. *Design lateral forces*

Shear distribution factor for the respective story factor for the respective story is calculated by using following equation;

$$\frac{V_i}{V_n} = \beta_i = \left(\frac{\sum_{j=1}^{n} w_j h_j}{w_n h_n}\right)^{0.75T^{-0.2}} \qquad (7)$$

Vi = shear force at ith level

β_i = Shear distribution factor at ith level
w_j = Seismic weight atlevel j
h_j = height of level j from the base
w_n = Seismic weight at top level
h_n = height of roof level from the base

Then, the lateral force at level i, Fi , can be obtained as,

$$F_i = (\beta_i - \beta_{i+1}).V_n$$

Fi = Lateral force at ith level

V_n = Story shear at roof level

Vy= Design base shear

Substituting the values of Vn we get following equation

$$F_i = (\beta_i - \beta_{i+1})\left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}}.V_y \quad (8)$$

B. Design of Designated Yielding Members (DYM)

When using the target yield mechanism for moment frames as shown in fig 5 beams become the primary designated yielding members (DYM). The required beam moment capacity at each level can be determined by plastic design approach (external work equals internal work).For Reinforced Concrete moment frames, in general, because of strength contribution from slabs and non-rectangular beam shapes (ie, T shape beam), as well as the use of different amounts of top and bottom reinforcement, plastic moments in positive and negative direction of DYM may be different.

$$\sum_{i=1}^{n} F_i h_i \theta_p = 2.M_{pc}\theta_p + \sum_{i=1}^{n} \beta_i.(M_{pb-positive} + M_{pb-negative})\gamma_i \quad (9)$$

$$\sum_{i=1}^{n} F_i h_i \theta_p = 2.\sum_{i=1}^{n} F_i h_i \theta_p + \sum_{i=1}^{n}(1+x)\beta_i.(M_{pb-positive})\gamma_i \quad (10)$$

$$\beta_i\left(M_{pb-positive}\right) = \beta_i \frac{\sum_{i=1}^{n} F_i h_i - 2M_{pc}}{(1+x)\sum_{i=1}^{n} \beta_i \frac{L}{L'_i}} \quad (11)$$

Where x is the ratio of the absolute value of negative Bending moment to positive Bending moment.

C. Design of Non Designated Yielding Members (NON-DYM)

Members that are not designated to yield (Non-DYM), such as columns in, must be designed to resist the combination of factored gravity loads and maximum expected strength of the DYM by accounting for reasonable strain-hardening and material over strength.. According to the concept of “column tree” is used to design the columns. The columns must be designed for maximum expected forces by including gravity loads on beams and columns and by considering a reasonable extent of strain-hardening and material over strength in the beam plastic hinges.

$$\text{Mpr} = \xi\,\text{Mpb} = 1.25\text{Mpb}. \quad (12)$$

The over-strength factor (‘ξ’) was taken as 1.25 which was established recognizing all these effects in ACI 318 (Moehle et al, 2008).when the frame reaches its target drift the shear force and bending moment at the desired beam plastic hinge locations at all levels are assumed to reach the expected strengths, Hence they are calculated as following equations;

$$Mpc_{pc} = \frac{\varphi V' h_1}{4} \quad (13)$$

$$V_i = \frac{\text{M PR POSITIVE} + \text{M PR NEGATIVE}}{L'} + \frac{\text{W i tributary L}}{2} \quad (14)$$

$$V_i = \frac{\text{M PR POSI TIVE} + \text{M PR NEGATIVE}}{L'} - \frac{\text{W i tributary L}}{2} \quad (15)$$

h 1= height of first story

IV. PERFORMANCE BASED SEISMIC DESIGN OF REINFORCED CONCRETE MOMENT RESISTANT IRREGULAR FRAME:-

In our study we have considered one regular 10 storey frameand compared our seismic design with Performance based Seismic design Methodology.Also to study the effect of vertical Geometric Irregularity we have compared two 10 storey frames with one step and two step setbacks with conventional and Performance based Seismic design method.We have shown a detail design calculation procedure for frame with one step setback.And compiled the results of all the three frames (10 storey regular & 10 storey irregular with two step setback designed in similar manner .Following are the three frame models considered for the study. Basic Dimensions for the frames and general design parameters were taken coomonly as follows.

Type of frame:Moment Resistant frame
Size of Column = 450 x 450mm
Size of Beam = 350 x 500 mm
Thickness of Slab = 125mm thick
Wall thickness = 150mm
Floor Finish = 1 KN/m2
Live load at all floor levels = 2 kN/m2
Zone III, Medium type of soil.

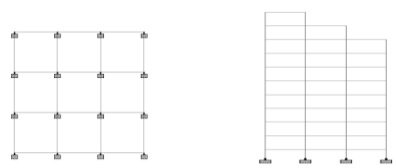


Figure 1Plan &Elevation of 10 storied regular and irregular frames considered for study

Table 4 Seismic parameters considered for design

Seismic zone factor '*Z*'	0.16
Soil Profile Type	Type 2 Medium
Importance factor, '*I*'	1
Sa Inelastic	0.1875 g
'*T*'	0.8s
Yield drift ratio 'θ_y'	0.5%
Target drift ratio 'θ_u'	2%
Inelastic drift ratio '$(\theta_u - \theta_y)$'	1.5%
Ductility factor	4
Reduction Factor due to Ductility '$R\mu$'	4
Energy Modification Factor 'γ'	0.43
Design Base shear	816.832

Table No.5 Steel area calculation for beams

Story	B	d	Mpr +ve	Ast (mm2)	Mpr -ve	Ast (mm2
10	350	500	867	6015	975	6131
09	350	500	865	6003	977	6143
08	350	500	891	5859	1001	6287
07	350	500	855	5943	986	6197
06	350	500	848	5901	994	6245
05	350	500	838	5841	1004	6305
04	350	500	819	5728	1022	6413
03	350	500	791	5560	1051	6586
02	350	500	739	5249	1102	6891
01	350	500	760	5375	1082	6771

Table No6 Steel for columns

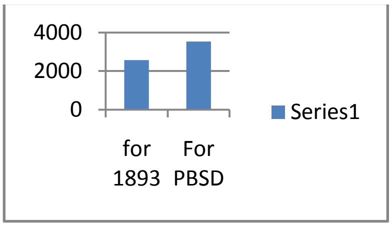
Width	Depth	Ac(mm2)	Fck(N/mm2)	fy N/mm2)	Axial force	Ast(mm2)
450	450	202500	20	415	624.7971	1620
450	450	202500	20	415	1260.861	1620
450	450	202500	20	415	1896.925	1620
450	450	202500	20	415	2532.988	3402
450	450	202500	20	415	3169.052	5811
450	450	202500	20	415	3805.116	8344
450	450	202500	20	415	4441.179	10602
450	450	202500	20	415	5077.243	12966
450	450	202500	20	415	5713.307	15208
450	450	202500	20	415	6349.37	17621

V. COMPARATIVE PERFORMANCE EVALUATION OF REINFORCED CONCRETE MOMENT RESISTANT FRAME

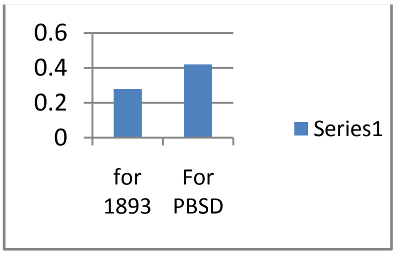
Capacity spectrum curve is actual plot representing the performance point i.e intersection point of spectral displacement and spectral acceleration. It is clear that in PBSD method performance point (intersection of demand and capacity curves) shifts due to extra confined steel which is normally incorporated in design. Hence provision for extra ductility is avoided since this care is already taken while designing.

Table No 7 Performance point comparison for Irregular frame with one set back

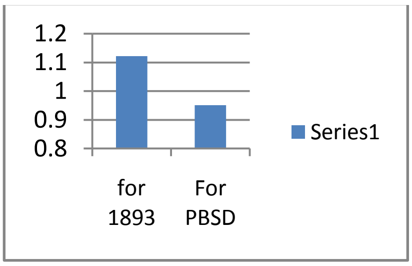
Performance point parameters	I.S 1893 method	PBSD method
Base shear vs Displacement	2575	3535
Spectral acceleration vs Spectral displacement	0.278	0.421
Effective Time	1.122	0.951



Performance point (V ,D)



Performance point (Sa, Sd)

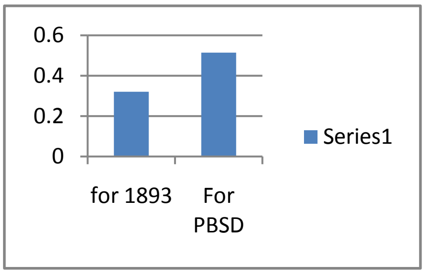


Performance point (Teff)

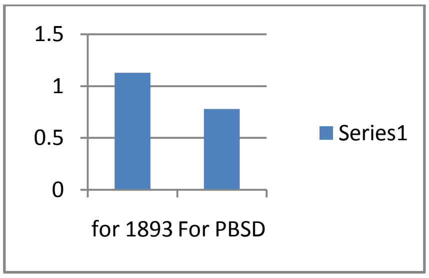
Fig 2 Push over curve comparison for I.S 1893 method and PBSD method for irregular frame with one step back.

Table No 8 Performance point comparison for Irregular frame with two step setback

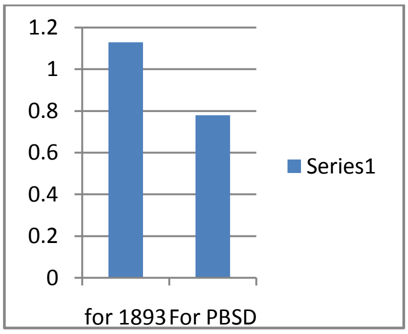
Performance point parameters	I.S 1893 method	PBSD method
Base shear vs Displacemet	2770	3990
Spectral acceleration vs Spectral displacement	0.32	0.514
Effective Time	1.13	0.78



Performance point (V, D



Performance point (Sa, Sd)



Performance point (Teff)

Fig3Push over curve comparison for I.S 1893 method ad PBSD method for irregular frame with two steps back

VI. Time History Analysis

In order to get a validation of performance with nonlinear static analysis this study includes nonlinear time history analysis and comparison of all the three frames designed by both methods ie (By I.S 1893;2002 method and Performance based Seismic design method). We have considered 4 standard ground motions(Superstition Hills1987 (Brawley), Imperial Valley, 1940(El Centro), 1989 Loma Prieta (Corralitos Station), 1994 Northridge (Santa Monica City Hall), Imperial Valley, 1940 (El Centro) Intensity factor=2.0). These ground motions are taken considering their maximum intensity and peak ground acceleration. After performing the time history analysis the major aspect considered is displacement. Hence this aspect is studied with reference to height of the structure. Since the building is 10 story, we had considered 6 intervals as shown. Time history results for regular, and two irregular frames designed by I.S 1893; 2002 and PBSD method are shown below.

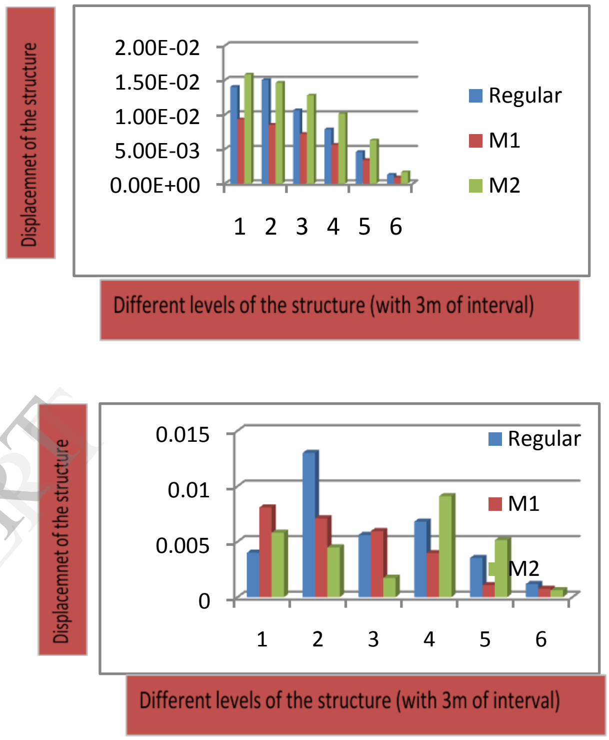


Fig4 Comparative summarization of the three frames designed by 1893; 2002 and PBSD method

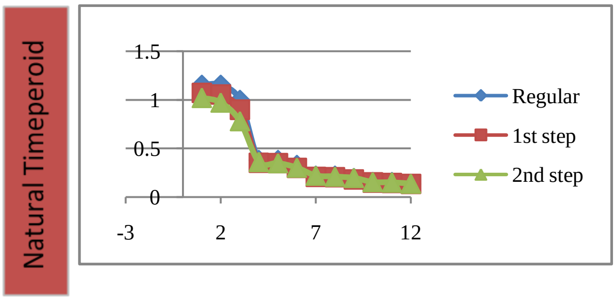


Figure 5 Time period and mode shape variation Curve for frames designed by PBSD method

observed from the table and graph it is For irregular frame with two step setback at top it is seen that the time period decreases initially up to 4th mode and then follows same trend as that of other irregular frame and regular frame. This indicates that for irregular frame, if designed by PBSD method it is more efficient than conventional I.S.1893; 2002 method.

VII. CONCLUSION

Following points are observed during whole design process;

The Performance Based Seismic Design method is based on the "strong column weak beam" concept in which the beams are designed as per plastic moments calculated .And columns are designed which ensures larger life safety of the structure. Performance objective was first decided and lateral forces are determined using inelastic design spectra which incorporate to actual behavior of the structure. These lateral forces are distributed according to new distribution factor which is defined on basis of real ground motion. Basic difference between regular and irregular frame design is for upper storey the calculations for base shear decreases due to asymmetry. This method requires little or no evaluation after the initial design because the nonlinear behavior and key performance criteria are built into the design process from the start. Performance point of the frames designed by PBSD method is enhanced than for all frames designed by conventional method. For the irregular frame with two step setback when designed by conventional method (I.S 1893;2002) method displacement is maximum than other two frames after performing time history analysis. For the irregular frame with two step setback when designed by PBSD method the displacement is lowest after time history analysis compared to the irregular frame with one step setback and regular frame. This proves the degree of reliability of Performance based seismic design method. Time period is one of the effective means to check the reliability of PBSD method. Time period for the irregular frame with two step setback is lowest than other two frames. The Performance Based Seismic Design method can be successfully applied to the design of Reinforced Concrete Moment Resistant Frames

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