On Assessment of Nonlinear responses of Medium-rise 3D RC Frames using Performance Evaluation Methods suggested in FEMA 440

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Abstract - It has become necessary to evaluate the seismic capability of medium-rise buildings to quantify the associated repairs, casualties and downtime of usage of property for a predefined seismic hazard. This ensures safety for both structure and occupants life. The present study focuses on a parametric analysis of nine storied 3D-bare frame using performance-based seismic evaluation procedures. The example building is designed as per the guidelines of IS 456 and IS 1893 for gravity loads. The engineering demand parameters used in the performance evaluation are obtained from nonlinear static analysis performed on the example frame subjected to different lateral load patterns. The obtained results have been validated through correlation with the permissible values documented in FEMA 440. The entire procedure used in the study provides a short hand procedure for stakeholders engaged in seismic design engineering.

Keywords* — *Medium-rise building, Performance-evaluation techniques, Pushover analysis, Parametric study

I. INTRODUCTION

The modern infrastructural development demands for the multi-storied structures. These multistoried structures show multiple performances, both at the global and local levels when subjected to inertia loads. The inertia loads acting on the structure are due to wind and seismic forces [1-6].

Contribution of research and experimental studies done in the field of seismic engineering has improved the seismic design and assessment procedures. With the intention to communicate the safety-related decisions to the stakeholders, the design engineers have shifted their focus towards the predictive methods of the seismic design. This resulted into the development of PBSD [4].

Present seismic design codes are incapable to describe the nonlinear modeling parameters for a reinforced concrete structure, but they provide the limits for strength and serviceability [1-6]. Federal Emergency Management Agency (FEMA) in association with Applied Technical Council (ATC) has put forth various nonlinear modeling methods and performance evaluation techniques for the assessment of seismic capability of reinforced concrete structure or member [7-10].The framework is known as, Performance-based Seismic Design (PBSD). PBSD document has presented various performance evaluation techniques using linear and non-linear analysis procedures [11]. The various building performance levels and structural performance levels described in PBSD framework are illustrated in Table 1-3.

This performance evaluation methodology is a three step procedure. In the first step, monotonically increasing lateral loads are applied up to the target displacement. From this capacity spectrum of the structure is obtained. Secondly the demand spectrum of the structure is obtained based on inelastic demand, soil parameters and assumed damping. In Third step the intersection of capacity spectrum and demand spectrum is done to evaluate the performance point, which validates the inelastic capability of the structure at global level. The local level performance is assessed through moment-curvature relation of structural components.

Table 1: Building performance levels as per ATC 40[7]

Non-structural Performance Levels	Structural Performance Levels					
	SP1 IO	SP2 DCR	SP3 LS	SP4 LSR	SP5 SS	SP6 NC
NP-A (Operational)	1-A	-	NR	NR	NR	NR
NP-B (Immediate Occupancy)	1-B	-	-	NR	NR	NR
NP-C (Life Safety)	-	-	3-C	-	-	-
NP-D (Hazard Reduced)	NR	-	-	-	-	-
NP-E (Not Considered)	NR	NR	NR	-	5-E	NR

NR = Not Recommended performance levels

The various performance evaluation method documented in FEMA are; (1) Capacity Spectrum Method (CSM), (2) Displacement Coefficient Method (DCM), (3) Improved

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Capacity Spectrum Method (ACSM) and (4) Improved Displacement Coefficient Method (ADCM).

First two methods are said to be first- and second-generation procedures. While third and fourth are said to be next generation procedures, which are documented in FEMA 440 [12-16]. While nonlinear static procedures are simple and their results are closer to that of dynamic procedures. PBSD document has proposed various performance evaluation procedures based on nonlinear static procedures (pushover analysis, POA), namely the capacity spectrum method and displacement coefficient method [7].

Table 2: Building performance levels as per FEMA 273/356 [8-9]

Non-structural Performance Levels	Structural Performance Levels					
	SP1 IO	SP2 DCR	SP3 LS	SP4 LSR	SP5 SS	SP6 NC
NP-A (Operational)	1-A	2-A	NR	NR	NR	NR
NP-B (Immediate Occupancy)	1-B	2-B	3-B	NR	NR	NR
NP-C (Life Safety)	1-C	2-C	3-C	4-C	5-C	6-C
NP-D (Hazard Reduced)	NR	2-D	3-D	4-D	5-D	6-D
NP-E (Not Considered)	NR	NR	3-E	4-E	5-E	NR

NR = Not Recommended performance levels

Table 4: Recommended implications of performance levels with discrete levels overlay [20]

Performance level	Building usability	Damage description
Life safety	Reoccupation of the building is unlikely and it will need to be replaced	Collapse prevention
Interrupted occupancy and Interrupted operations	Reoccupation of the building is delayed and repairs may be costly	Significant or Substantial damage
Continued occupancy and interrupted operations	Reoccupation of the building is almost immediate and the cost of repair is modest	Limited damage
Continued occupancy and Continued operations	The building can continue its operation almost immediately	Minimal to no damage

In capacity spectrum method, the structure is subjected to predefined lateral load pattern with monotonically incremental steps till a target displacement is reached. The response of structure is plotted for rooftop displacement and base shear known as capacity spectrum. The inelastic demand spectrum is obtained for the stated time period and damping coefficient. The intersection of this curve provides performance point which defines the level of seismic performance. Figure1 describes the CSM procedure [1-3]. While in the displacement coefficient method (DCM) is the simplest method of obtaining target displacement. The method does not involve the conversion of capacity curve into corresponding spectral coordinates. The linearization of capacity curve is done to obtain performance point. Figure2 illustrated DCM method.

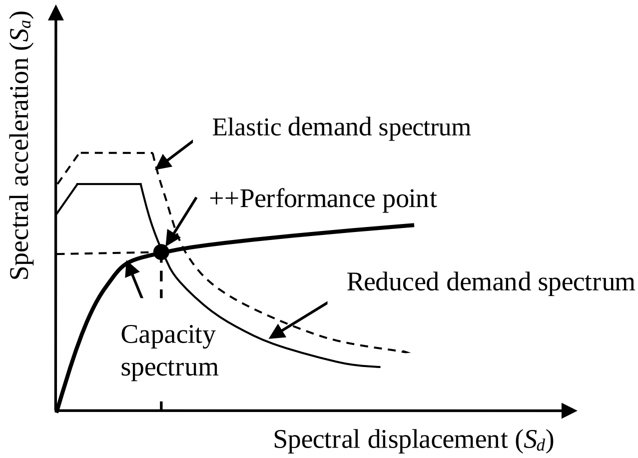

Fig. 1:Capacity Spectrum Method [7]

These procedures provide the information about nonlinear responses using the collapse mechanism and transfer of plastic hinges from on performance levels to another, but fail to provide any associated damage values. In this study, we had attempted to identify the engineering demand parameters showing the loss or damage to the structure. The statistical data will provide a rational approach to designers to predict damage state level at the iterative stage of the design process.

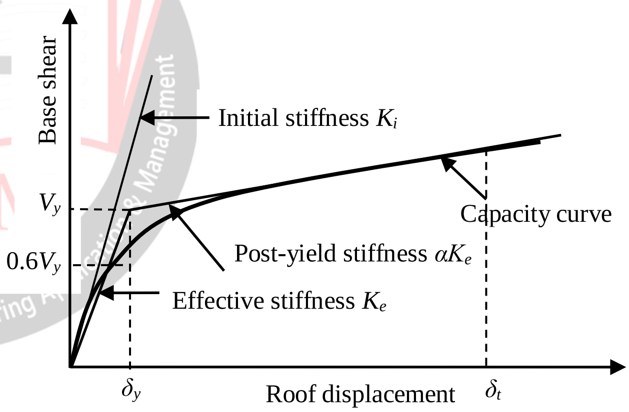

Fig. 2:Displacement Coefficient Method [7]

II. PUSHOVER ANALYSIS

In PBSD various performance levels are defined in terms of damages sustained by the structural and non-structural components during a seismic event. Namely, Operational level (OP), Immediate Occupancy level (IO), Life-safety range (LS), Collapse prevention (CP) and Collapse (C). The attainment of these performance levels are identified on the basis of drift. For global performance identification storey drift is referred, whereas; for local level inter-storey drifts are used. The accuracy and efficacy of these performance levels depend on modeling of the plastic hinges and their locations [7, 9].

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The performance evaluation procedures provided by PBSD aims to; (1) to verify measured structural strength, (2) to interpret the effects of an inelastic incursion on the overall behavior of structural and (3) to interpret the displacement histories and maxima.

In this study performance evaluation procedures are used to evaluate the nonlinear responses of 3-D RC bare frames with soft storey subjected to seismic loads. The engineering demand parameters resulted in the output of these procedures was used to assess the damage or loss to the example frame. The damage states are correlated with the performance levels stated in PBSD. The parametric study illustrates a rational approach to identify the damage value along with the performance evaluation process, which is a need of the hour.

The accuracy of the results depends on the applied lateral load patterns. To obtain realistic results a set of lateral load patterns are used, which defines upper bound and lower bound values of structural capacity. In the present study, we have considered three lateral load patterns, namely IS 1893, uniform load and elastic-first mode lateral loads [9-16].

For POA dead loads contributing from the slab, beams, and columns (including finishes loads) were used. Live load of intensity 3 kN/m^2 was applied on the slabs. The seismic design loads used in the design include 100 percent dead load and 25 percent live load contribution on a floor. Auto hinges of type P-M3-M2 for the columns and M3 for beams were assigned at both ends. The default properties for moment-curvature and stress-strain distribution defined in software are used to develop the nonlinear parameters of RC sections. The target displacement against lateral loads was considered to be 4 % of the total height of the building.

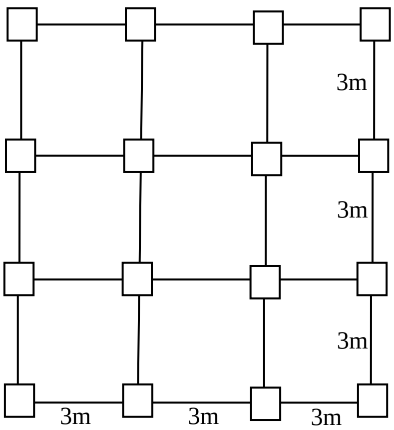
III. PUSHOVER ANALYSIS

For the present study, a 9 storey 3D bare MRF is subjected to different lateral load patterns. Table 5, details the different lateral load pattern used in POA.The design of the buildings is made as per the guidelines of IS 1893, IS 456 and IS13920 [17-19]. Table 6 provides details of material used and structural design of example MRFs. The plan of building is symmetric about X and Y axis to avoid torsion. Figure 3 shows typical plan and elevation of example MRF.

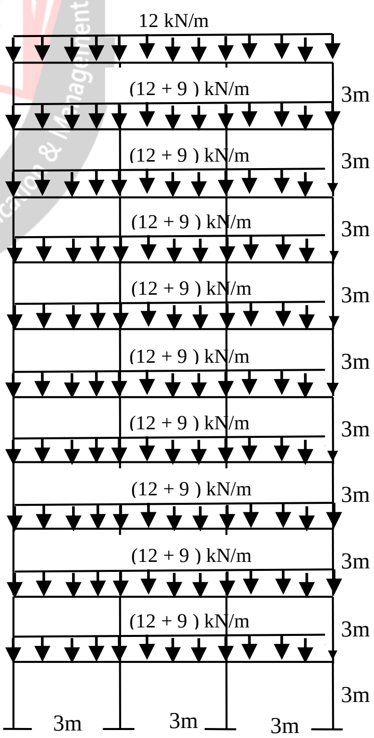
The pushover curve obtained for example MRF for different lateral load pattern is shown figure 4. The intersection of capacity spectrum with the demand spectrum is known as performance point. The value at performance point evaluates nonlinear responses of example MRF. The efficacy of performance evaluation procedures can be seen from the comparison of base shear and displacement values obtained at performance point for different PBSE methods as described in the figure 4.

Table 5: Different lateral load pattern used in POA

Storey height h_i (m)	Seismic Weight Wi (KN)	IS 1893 Lateral Load Pattern: Push 1 (KN)	Uniform Lateral Load Pattern: Push 2 (KN)	Mode 1 Lateral Load Pattern: Push 3 (KN)	Lateral force from software
3	3908.2	3.05	77.14	9.24	2.82
6	3908.2	12.23	77.14	27.27	11.33
9	3908.2	27.52	77.14	47.58	24.58
12	3434.5	43.00	67.77	62.00	41.50
15	3434.5	67.19	67.77	82.51	65.00
18	3434.5	96.76	67.77	101.53	89.80
21	2936.2	112.59	57.94	102.55	115.00
24	2936.2	147.00	57.94	115.46	150.00
27	2936.2	186.12	57.94	124.30	160.67



Typical Plan



Typical Elevation

Fig. 3: Modeling details of example building

The collapse mechanism resulting from POA is shown in terms of formation of plastic hinges and their fall from one performance level to other performance range.

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Table 6: Material used and structural details

Sr. No	Particulars	Assumptions
1	Type of structure	Multi-storied Special Moment Resisting Frame
2	Seismic zone	III (table 2; I.S. 1893:2002)
3	No. of stories	Nine storied (G+8)
4	Floor height	3m
5	Tributary width	3m
6	Imposed load	3 kN/m^2
7	Materials	**Concrete:** a. Weight per unit volume 25 kN/m^3 b. Mass per unit volume 2.5485 Kg/m^3 c. Modulus of elasticity (E_c)= 5000$\sqrt{f_{ck}}$ = 25000 kNm d. Poisson ratio (μ) 0.20 e. Coefficient of thermal expansion (α) = 5.50 E-06 f. Shear modulus (G) 1041667 kN/m^2 g. Characteristic strength (f_{ck}) = 25000 kN/m^2 **Reinforcement:** a. Weight per unit volume 76.9729 kN/m^3 b. Mass per unit volume 7.849 Kg/m^3 c. Modulus of elasticity(E_s) = 2E+08 kNm d. Poisson ratio (μ) 0.30 e. Coefficient of thermal expansion (α) 1.17 E-05 f. Shear modulus (G) 76923077 kN/m^2 g. Yield strength (f_y) 41500 kN/m^2 h. Minimum tensile stess (f_u) 485000 kN/m^2 i. Expected yield strength (f_e) 456500 kN/m^2 j. Expected tensile stess (f_{ue}) 533500 kN/m^2
8	Size of columns	(obtained from gravity analysis) **Floors Size of columns Main bars (Tor) and Shear bars (Tor)** 01-03; 750 mm x 750 mm; 10No-25 mm; 8 mm@ 150 mm c/c 04-06; 680 mm x 680 mm; 08No-25 mm; 8mm@ 150 mm c/c 07-09; 600 mm x 600 mm; 06No-25 mm; 8mm@ 150 mm c/c
9	Size of beams	Both longitudinal and lateral (obtained from gravity analysis) **Floors; Size of Beams; Top bars(Tor); Bottom bars (Tor); and Shear bars (Tor)** 01-9; 300 mm x 530 mm; 705 mm^2; 625 mm^2; and 8mm@ 110 mm c/c
10	Depth of slab	150 mm thick
11	Type of soil	Medium soil
12	Seismic Zone-II	Z =0.16
13	Response spectra	As per IS 1893:2002(part1) compatible for 5 % damping

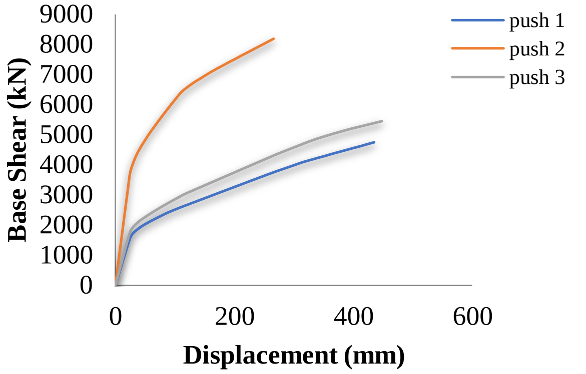
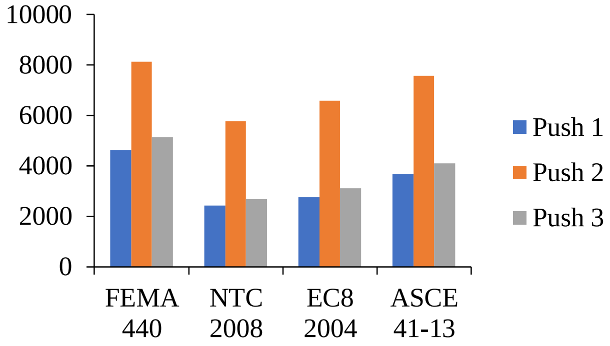
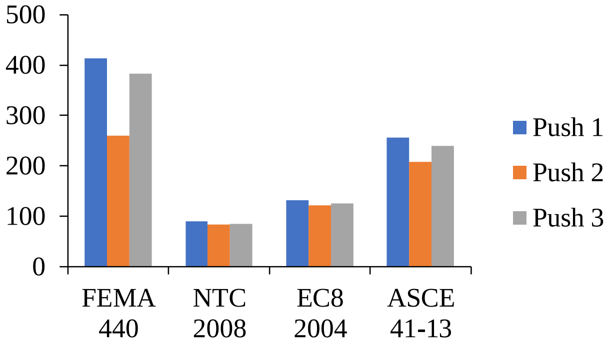


Fig 4: Pushover curve for different push load cases



(a) Base Shear at performance point



(b) Displacement at performance point

Fig 5: Comparison of base shear and displacement values at performance point for different PBSE methods

Figure 6 shows the collapse hinge mechanism of example frame for different POA load cases.

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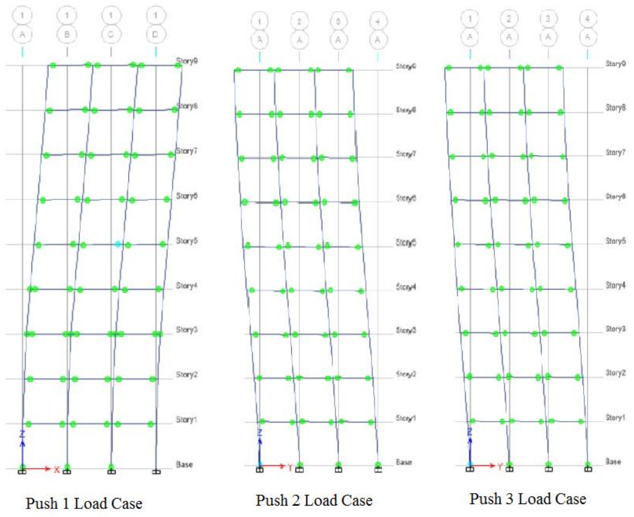

Fig. 6: Collapse hinge mechanism of example MRF

With the incremental displacements fall in stiffness of the structure is observed. This reduction in stiffness value may be attributed towards the damages to structural components.

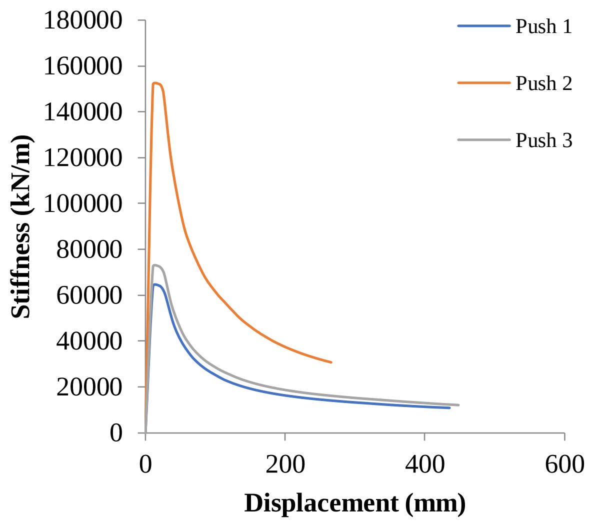

Fig 7: stiffness vs displacement curve for different push load cases

Table 7 A: Stiffness of example MRF at performance point for Push 1 load case

PBSE Methods	Base shear at performance point (kN)	Displacement at performance point (mm)	Stiffness at performance point (kN/m)
FEMA 440	4624.36	413.27	11196
NTC 2008	2430.86	90.00	27000
EC8 2004	2756.13	131.98	20881
ASCE41-13	3678.09	256.03	14356

Table 7A-7D provides the loss in stiffness at performance point and performance levels. Stiffness values resulted at various performance levels when compared to each other, reflects the loss in stiffness at global level. The values obtained at operational level can be treated as elastic stiffness assuming undamaged state of example MRF. The stiffness value showed percentage loss at Immediate Occupancy 0.05; life safety range 74; 78.96 for collapse prevention and at collapse 82.88.

Table 7B: Stiffness of example MRF at performance point for Push 2 load case

PBSE Methods	Base shear at performance point (kN)	Displacement at performance point (mm)	Stiffness at performance point (kN/m)
FEMA 440	8127.85	260.26	31260
NTC 2008	5761.51	84.17	68583
EC8 2004	6598.00	122.28	54081
ASCE41-13	7584.28	207.95	36637

Table 7C: Stiffness of example MRF at performance point for Push 3 load case

PBSE Methods	Base shear at performance point (kN)	Displacement at performance point (mm)	Stiffness at performance point (kN/m)
FEMA 440	5131	383	13369
NTC 2008	2692	85.05	31670
EC8 2004	3111.22	125.73	24888
ASCE41-13	4096.28	240	17066

Table 7D: Stiffness of example MRF at various performance levels

Performance Levels	Stiffness (kN/m)		
	Push 1	Push 1	Push 1m)
OP	63851	151666	72438
IO	63818	151200	72103
LS	16502	38724	19630
CP	13428	32301	14371
C	10928	30432	12200

IV. CONCLUSION

Assessing the performance of an RC structure under seismic loads is a critical process. RC structures exhibit inelastic behaviour under seismic load which leads towards inaccurate estimates of engineering demand parameters. Next generation procedures have provided various performance based seismic design and evaluation procedures, using nonlinear static analysis. When these procedures were used to analyze an example MRF there has been disagreement in base shear and displacement value. This may be attributed towards different bi-linearization techniques, time period and damping effects adopted in individual methods. Thus, questioning the adequacy of these procedures. To arriving the promising results, it needs to properly model RC members.

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Different lateral load patterns were applied to an example MRF, resulting in a capacity spectrum with upper and lower bond values for the push 1 and push 3 load cases, respectively, and a median response for the push 2 load case. With the increase in inelastic displacement, a fall in stiffness has been observed. If these values are reflected with limiting drift at various performance levels it may help in identifying failure zones, which can be used for optimization of RC section design.

The present study aims to illustrate all pros and cons of performance based seismic evaluation techniques and highlights the grey areas for structural optimization.

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