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A Comparison of Basic Pushover Methods

N.K. Manjula[1], Praveen Nagarajan[2], T.M. Madhavan Pillai[3]
[1]*Research Scholar, Department of Civil Engineering, NIT Calicut, India*
[2]*Associate Professor, Department of Civil Engineering, NIT Calicut, India*
[3]*Professor, Department of Civil Engineering, NIT Calicut, India*

ABSTRACT: *Nonlinear static (pushover) methods for seismic analysis of structures are being widely used due to their inherent simplicity in modeling and low computational time. Advanced procedures for such methods are being developed in recent years, but the procedures described in international standards and building codes are mostly used in the design field. This work aims to compare and identify the differences among the pushover analysis methods given in international standards, considering one reinforced concrete (RC) building frame, designed as per IS 1893-2002provisions. The performance of the building which is designed based on strength based method with sufficient ductile detailing is also evaluated.*

Keywords - *Nonlinear, Performance based , Pushover, Seismic analysis, Strength based.*

I. INTRODUCTION

In recent decades, the civil engineering practices worldwide has come a long way in the analysis and design of structures against seismic actions. Structures built prior to this scenario may need repair or retrofit, and their analyses require much effort, as assumptions about their strength, stiffness or ductility may not be dependable. Design of new buildings or performance enhancement of existing buildings can be commenced with an elastic or inelastic analysis of the structure either in the static or in the dynamic analysis domains. Displacement based analysis techniques are most popular today in this regard, which includes a range of nonlinear static (pushover) [1,2,3]analysis methods. The basic principles behind them are all alike, but each of them differs in their procedures and hence, in their results.

II. OBJECTIVES OF PERFORMANCE BASED DESIGN

The strength based design procedures in the building codes are considered insufficient to determine the performance of a structure under seismic loads. Displacement based procedures are more suitable for nonlinear performance stages, as it is uneconomic to design buildings to remain elastic during earthquakes. The performance based design assumes different displacement (damage) levels that satisfy specific performance criteria during certain specific levels of seismic actions.

Performance levels

A limit state of damage which may be considered satisfactory for a given building and a given ground motion intensity is known as a performance level. It contains structural and non structural performance levels which should consider

1. the substantial damage within the building,
2. the safety hazard caused by the damage and
3. the post earthquake serviceability of the building.

Description of structural and non structural performance levels vary slightly in different standards and the ones (Immediate Occupancy, Life Safety, Collapse prevention etc) given in FEMA356[2]are used throughout this work.

Nonlinearity: The Plastic hinge model

Even though it is advisable to check for the position of plastic hinges by trial and error modelling, researches show that the possibility of hinge formation is maximum at beam and column end zones during an earthquake. If the gravity loads are large, hinges may form near the mid span in beams. In such cases, cyclic loads increases the rotation of hinges progressively, causing the beam to sag. The required hinge properties can be developed from experimental results. In the present work, plastic hinges (lumped or point plasticity)[5] for beams are modelled near the ends only, because gravity loads are not very large.

For beams, as the axial forces are not taken in to account, the Moment- curvature (M-ϕ) relationship[5] is sufficient to model the nonlinearity (hinge) point. Concrete beams are usually brittle in shear and hence designed for flexural strength. The inelastic shear, which can be modelled by a shear hinge, is not taken in to account in this work, as it is unimportant for the building considered because of the ductility assumptions and shear reinforcement specifications followed in its design.

But for columns, the Axial force (P)- Moment(M) interactions are required in addition, because the flexural strength depends on the axial force and vice versa. Also, the moments and shears (V) acts with respect to two axes and the P and M values and interactions affects the shear strength too. Because of these inter-relations, the hinge behaviour is complicated, not simple rotation. But, even though the column hinges have both axial and bending deformations, the demand/capacity ratio is usually calculated from bending deformation only, not including axial deformations. However, the rotation capacity of a hinge can depend on P and V because the bending ductility is smaller for larger values of these parameters. Inelastic shear in columns is not considered in the present study.

III. THE PUSHOVER ANALYSIS METHOD

In general, it is the method of analysis by applying specified pattern of direct lateral loads on the structure, starting from zero to a value corresponding to a specific displacement level, and identifying the possible weak points and failure patterns of a structure. The performance of the structure is evaluated using the status of hinges at target displacement[2] or performance point[1] corresponding to specified earthquake level (the given response spectrum). The performance is satisfactory if the demand is less than capacity at all hinge locations.

As the loading and evaluation procedures are only virtually correct with respect to the real earthquake events, it differs from the rigorous dynamic analysis in many ways.

IV. EVALUATION PROCEDURES

Although the procedures for building evaluation are different from one another, their basic principles are all the same and they all use the bilinear approximation of the pushover curve. This static procedure equates the properties of every Multi degree of freedom (MDOF) structures to corresponding Single degree of freedom (SDOF) equivalents, and approximates the expected maximum displacement using the Response spectrum of relevant earthquake intensity.

a. ATC 40*[1] *- 1996 - Capacity Spectrum Method(CSM)

This method is based on the equivalent linearization of a nonlinear system. The important assumption here is that inelastic displacement of a nonlinear SDF system will be approximately equal to the maximum elastic displacement of linear SDF system with natural time period and damping values greater than the initial values for those in nonlinear system. ***ATC 40*** describes three procedures (A,B and C) for the ***CSM*** and the second one is used in this study.

b. FEMA 356* [2]*- 2000 - Displacement Coefficient Method(DCM)

Here, the nonlinear MDF system's displacement is obtained from the linear elastic demand spectrum, using certain coefficients which are based on empirical equations derived by calibration against a large number of dynamic analyses.

c. FEMA 440* [3]*- 2005 - Equivalent Linearization - Modified CSM

This improved version of equivalent linearization is derived from the statistical analysis of large number of responses against different earthquake ground motions. The assumption in CSM that the equivalent stiffness of inelastic system will be the same as its secant stiffness is not used here. Instead, the equivalent stiffness is obtained from effective time period and damping properties derived using equations from statistical analyses.

d. FEMA 440* [3]*- 2005- Displacement Modification- Improvement for DCM

This improvement for the earlier Displacement coefficient method uses advanced equations for different coefficients. Coefficient for $\mathrm{P}-\Delta$ effects is replaced with a lateral dynamic instability check by defining a maximum value of lateral strength R, such that

$$R_{max} = \frac{\Delta_d}{\Delta_y} + \frac{(\alpha_e)^{-t}}{4} \quad (1)$$

where, the terms are as described below:

Δ_d and Δ_y are the displacements corresponding to maximum base shear V_d and effective yield strength V_y respectively

If K_e is the effective stiffness of the building, which is the slope of the line joining zero base shear point and the point at 60% of idealized yield strength, obtained from idealization of pushover curve in to linear portions,

$\alpha_1\ K_e$ = effective post yield stiffness with positive slope,

$\alpha_2\ K_e$ = maximum (negative) post -elastic stiffness, which is the slope of the line connecting points of maximum base shear and 60% yield strength on the post- elastic curve,

$\alpha_{\mathrm{P}-\Delta}\ K_e$ = Slope of the tangent at the point of maximum base shear,

$\alpha_e\ K_e$ = effective post elastic (negative) stiffness, where, $\alpha_e = \alpha_{\mathrm{P}-\Delta} + \lambda(\alpha_2 - \alpha_{\mathrm{P}-\Delta})$ (2)

λ, a factor representing ground motion effects, = 0.2 for far field motions and 0.8 for near field motions

If T = fundamental time period of the building, $t = 1 + 0.15 \ln T$ (3)

$$R = \frac{S_a}{V_y/W} C_m \tag{4}$$

Where, V_y = Yield strength calculated using results of the pushover analysis for the idealized nonlinear force displacement curve,
S_a = Spectral acceleration obtained from the demand spectrum with specified damping, corresponding to the effective time period T_e, obtained from the idealized pushover curve,
W = Effective seismic weight of the building including the total dead load and applicable portions of other gravity loads as given in FEMA 356, and
C_m = Effective mass factor which is taken as the effective modal mass for 1st mode of the structure.

V. BUILDING DESCRIPTION

Loading details (Table I) of the regular Reinforced Concrete 2D frame which is a part of the 3D building designed according to the provisions of IS 1893-2002[4] are selected from the IITK - GSDMA [6] project on building codes. The frame geometry is slightly modified for simplicity and reinforcements are provided based on IS 1893-2002[4]. The width of each bay is 7.5m and height of each storey is 5m. 50% of live load is considered for seismic weight calculation. Strong column -weak beam philosophy with a moment capacity ratio of 1.1 is used for the calculation of minimum joint reinforcement[7,8]. The generalised reinforcement details which are obtained for a gravity case of dead load plus full live load during the earthquake are shown in Tables II and III. Shear reinforcements as per latest Indian standards [4,7,9]are provided

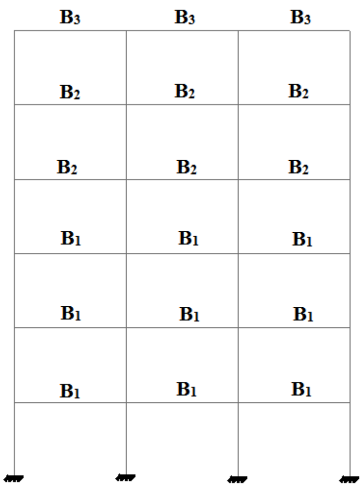


Figure I : Building geometry

Table I : Dead and Live Load Details of Beams

Storey no	Beam Location	Uniformly distributed Load (kN /m)		Concentrated Loads (kN) at 2/3 L locations	
		Dead Load (DL)	Live Load (LL)	DL	LL
Stories 1 to 5	Exterior	26	5	-	-
	Interior	21.6	0	42.2	37.5
Roof	Exterior	12	2	-	-
	Interior	5	0	61.1	14.3

Table II : Reinforcement details - beams

B_1 (300 × 600 mm^2)		
At left and right supports	Top bars	A_s = 3500 mm^2
	Bottom bars	A_s = 2100 mm^2
B_2 (300 × 600 mm^2)		
At left and right supports	Top bars	A_s = 3200 mm^2
	Bottom bars	A_s = 1700 mm^2
B_3 (300 × 600 mm^2)		
At left and right supports	Top bars	A_s = 1570 mm^2
	Bottom bars	A_s = 1570 mm^2

Table III : Reinforcement details - columns

Floor level	*Interior columns*		*Exterior columns*	
	Size (mm × mm)	Reinforcement	Size (mm × mm)	Reinforcement
Ground floor	600 × 600	16 - #25	600 × 600	14 - #25
First floor	600 × 600	14 - #25	600 × 600	14 - #25
Second, third & fourth floor	500 × 500	12 - #26	500 ×500	12- #25
Fifth floor	500 × 500	12 - #25	500 × 500	10- #25

Material properties:

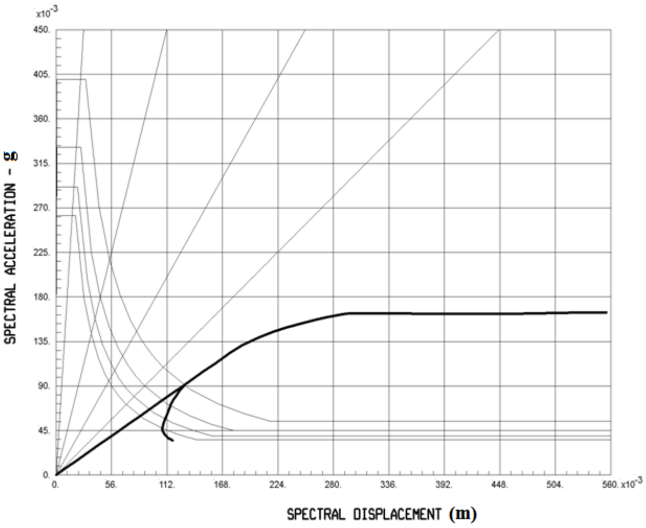
Concrete
All components unless specified are of uniaxial compressive strength $25 N/mm^2$
For columns up to first floor, concrete of compressive strength$30 N/mm^2$ is used.
Steel
HYSD reinforcement bar of uniaxial tensile yield strength $415 N/mm^2$ conforming to IS 1786[10] is used throughout.
Partial material safety factor for steel and concrete are respectively 1.15 and 1.5.
RCC is modelled using Mander's stress- strain curve for confined concrete.

VI. EVALUATION USING THE SPECTRUM OF IS 1893-2002

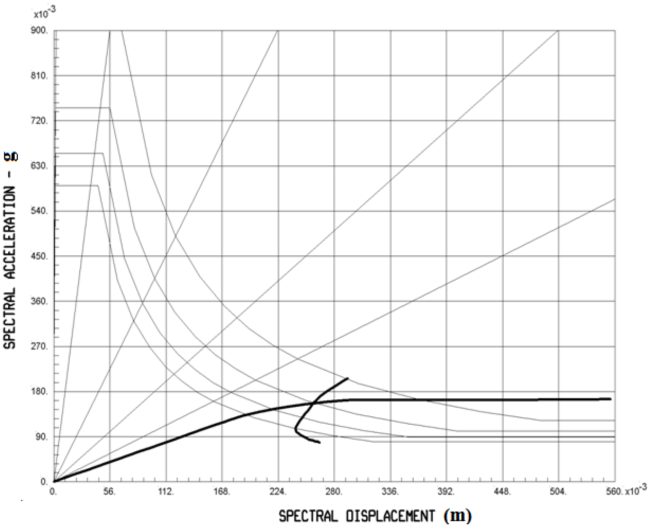
The building based on IS 1893 - 2002 design, considering seismic zone III is evaluated for two cases. One, its Maximum considered earthquake of design (Case I) and the other a higher level earthquake (Case II). Ductile detailing requirements of latest Indian standards are assumed. The results of each case obtained using different pushover methods are shown in fig.II a,b,c and d.

a. ATC 40 - Capacity Spectrum Method

Case I

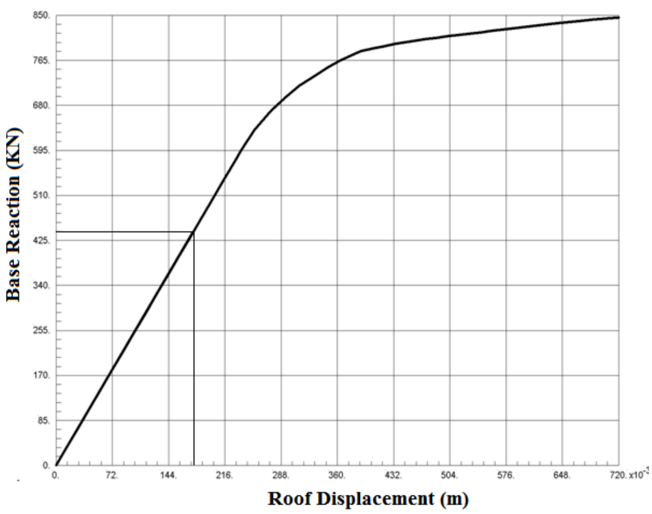


Case II

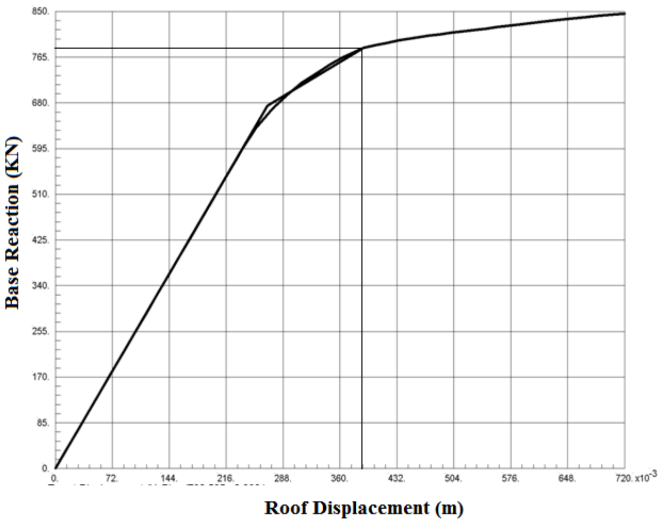


b. FEMA 356 - Displacement Coefficient Method

Case I



Case II



c. FEMA 440- Displacement Modification
Both Case I and Case II results are same as the FEMA 356 -DCM shown in fig.II(b).
d. FEMA 440 - Equivalent Linearization

Case I **Case II**

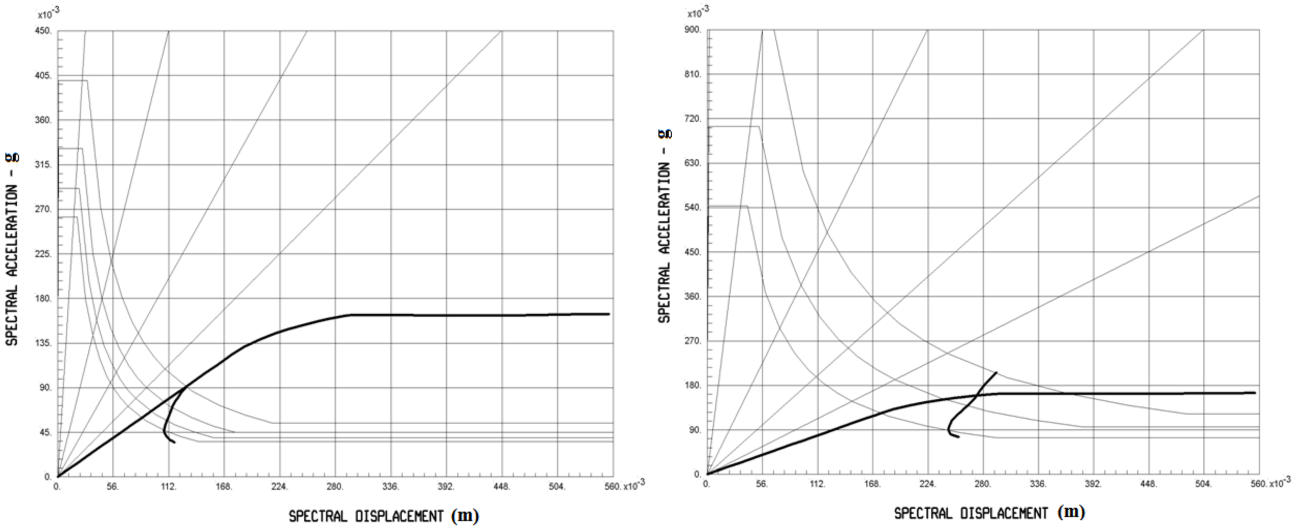


Figure II (a,b,c,d): Target Displacement Results

The lateral load pattern corresponding to IS 1893 - 2002[4] is adopted and applied as auto lateral load pattern in SAP 2000[11]. Total load including DL and LL are applied in the gravity load case and the seismic weight and hence the load pattern is calculated using DL+0.25LL for the EQ load case. This is done to make the analysis method different from the design aspect. The direction of monitoring the behaviour of the building is same as the push direction. The effect of torsion is ignored. In case of columns, program defined auto PM_2M_3 interacting hinges are provided at both the ends, while in case of beams, *M3* auto hinges are provided. Effective stiffness of columns and beams are taken as per NEHRP [2] guidelines for existing buildings.

For Case I , seismic zone III and soil type II, the Response spectrum is obtained corresponding to a zone factor 0.16 as per the IS code provisions. For Case II, seismic zone V and soil type II, the Response spectrum is obtained corresponding to a zone factor 0.36. In both cases, the Maximum Considered Earthquake of the zone is chosen.

The target displacement(δ_t) and base shear(V_b) for the Displacement modification methods are directly obtained from fig.II (**b**). The performance points for Equivalent Linearization methods has to be obtained from fig II (**a** and **d**) by converting the spectral displacement value to the control node displacement value, by multiplying with the factor $PF1\varphi_{roof,1}$, representing the first modal participation at the level of control node.

The table IV shows that all the methods yield similar results when the building is in the elastic range. But when the capacity of the structure is being evaluated for a higher level of earthquake, the results show considerable variations.

The ATC 40 method, the oldest one in these, underestimates the result, compared to the other methods, for the case considered, which was expected as explained in latest American standards[3]. Nonlinear time history analysis for the spectrum compatible set of earthquakes may be used to validate the results, which is not done in the present study.

Table IV: Target Displacement values

		ATC 40	FEMA 356	FEMA 440 EL	FEMA 440 DM
Case I	**V_b (KN)**	442.12	439.1	442.12	439.1
	δ_t (m)	0.176	0.175	0.176	0.175
Case II	**V_b (KN)**	748.97	780.6	764.5	780.6
	δ_t (m)	0.346	0.388	0.364	0.388

Hinge status at a roof displacement of 0.39m (displacement greater than δ_t value from all the methods) is shown in fig III. It shows the strong column- weak beam failure pattern and a satisfactory distribution of hinges. But the limit state of collapse prevention stage at the MCE is not reached in this case, showing a conservative design.

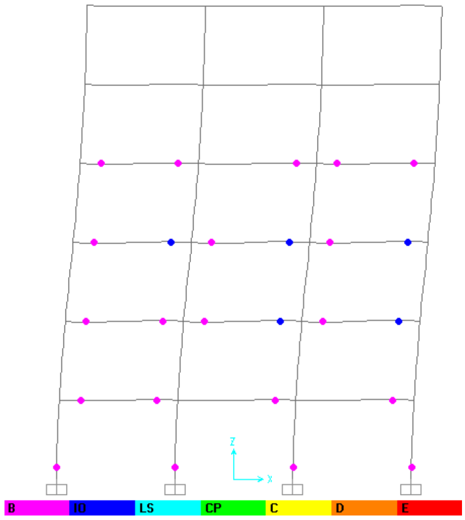


figure III: Hinge status at target displacement

VII. DISCUSSION

Even though the limited analysis using only one analytical model is not sufficient to lead to any conclusion, the following points can be drawn from this study.

1. Code based methods with good ductile detailing for low seismicity normally ensures good performance under higher level earthquakes too as indicated from the hinge status at a roof displacement near the target displacement for higher level seismicity. Such methods are not uneconomical for low seismic regions, as buildings are not expected to yield much during such events.
2. The performance based analysis may be done for retrofitting of structures and design of structures for higher levels of seismicity, as the inevitable nonlinearity in such events cannot be properly accounted by the force based methods.
3. Also, the nonlinear static analysis must be used with caution for complex and large structures, as the results vary considerably from one another, as seen from table IV. Nonlinear time history analysis is essential for such cases.
4. Similar studies on complete 3D models with dynamic analysis validation may lead to better conclusion.

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