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Seismic Performance Evaluation Of Reinforced Concrete Frames

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Abstract: *Ten storied -3bays reinforced concrete bare frame designed for gravity loads as per IS 456: 2000 (rev) and IS 13920: 1993 for ductility is subjected to seismic loads. Seismic loads considered is earthquake loads determined from IS 1893-2002 (part 1) response spectra for 5% damping (for hard soil). Structural elements are modeled as two node element (three degrees of freedom at each end). Plastic hinges are used to represent the failure mode in the beams and columns when member yields. Pushover analysis is performed using SAP 2000 V 14.0 commercial software in reference to various performance levels suggested in first, second and next generation performance based design procedures (ATC 40, FEMA 356, FEMA 440). Base shear versus top displacement curve of structure, known as pushover curve is obtained for force-displacement (brittle) and Deformation-controlled (ductile) actions of plastic hinge. Lateral deformation at performance point proves the building capability to sustain certain level of seismic loads. The failure mechanism indicates structural performance levels in accordance first, second and next generation PBSD procedures. The study aims towards understanding the first, second and next generation PBSD evaluation procedures.*
Keywords: *PBSD evaluation procedures, Building frames, Plastic hinges, Pushover analysis, Seismic Performance*

I. Introduction

Present seismic design codes, describes forced–based design procedures for lateral load resistance structures [1]. In static case loads on structure are low resulting in elastic behavior. During strong seismic event, these structures are subjected to loads beyond its elastic limits [2]. Though the present code can provide reliable indication of expected performance for life safety (strength and ductility) and damage control (serviceability drift limits) but are incapable to describe the expected performance, under large forces [3]. Performance based seismic design has emerged as best alternative towards present seismic code design procedure which is capable to describe the inelastic behavior of structure. PBSD, where inelastic structural analysis in combination to defined seismic hazard level is used to obtained expected performance of structure [1]. Second generation procedure recommends four analysis procedures to estimate seismic demands as presented in table 1[4]. Next generation procedure presents improvements in first and second generation procedures [8]. Amongst this Nonlinear Dynamic Procedure (Time history, NLTH) is capable to calculate seismic responses under strong earthquakes, but results in large amount of data and time consuming process hence not considered practical.

Practicing field engineers prefers nonlinear static procedure (Pushover, NLSP) procedure due to easy and compatible compared to results obtained through NLTH [5-9]. First, second and next generation procedure provides various building performance levels based on contribution of structural and non structural performance levels in reference to transient and permanent drift as presented in table 2 [10]. The various structural performance levels and damage for vertical elements described by various PBSD procedures are presented in table 3 [7] and used for modeling of example building frame in present study.

Table 1: Various analysis procedures to estimate seismic demands suggested by second generation procedure [4]

Type of Analysis	**Usual Name**	**Dynamic effects**	**Material Non-linearity**
Linear static	Equivalent static	No	No
Linear dynamic	Response spectrum	Yes	No
Nonlinear static	Pushover	No	Yes
Nonlinear dynamic	Time history	Yes	Yes

Table 2: Building Performance Levels [10]

FEMA 273/356		**SEAOC vision 2000**		**ATC 58**	
Rating	**Performance Levels**	**Rating**	**Performance Expectation**	**Anticipated damage**	**Color code**
S-1	Immediate occupancy	10	Fully operational	Negligible	Green
	Damage control	9			
S-2		8	Operational	Light	
		7			
S-3	Life safety	6	Life safe	Moderate	Yellow

		5			
S-4	Limited safety	4	Near collapse	Severe	Red
S-5	Collapse	3			
	prevention	2	Partial collapse	Complete	
		2	Partial collapse-assembly areas		
		1	Total collapse		

Table 3: Performance levels corresponding to damage states and drift limits [7]

Structural Performance level	Damage state	Transient Drift	Permanent drift
Immediate occupancy	Minor hair line cracking	1%	Negligible
Life safety	Extensive damage to beams, spalling of cover and shear cracking	2%	1%
Collapse	Extensive cracking and hinge formation	4%	Permanent

Performance based design procedures

Interest in performance based seismic design initiated in the 1980s amongst engineers engaged in seismic design and retrofit of existing buildings [1-3]. The roots of PBSD can be traced from the development of recommendation by the Structural Engineers Association of California (SEAOC, 1960), and the publication by the Portland Cement Association (PCA). Later Joint efforts of Federal Emergency Management Agency (FEMA) and Applied Technical Council (ATC) results in publication of document named ATC 40 (1996) [5].

First Generation procedures

ATC-40 procedure involves comparison between capacity of structure (Pushover curve) and demands on thestructure (Demand spectrum) and their graphical instruction is termed as performance point. The target displacement is computed as maximum displacement of a linearly ESDOF system with time period T_{eq} and effective damping

ratio β_{eq} [15]

Second Generation Procedure

FEMA 273 (1997) [6] provides simplest and straight forward method of estimating target displacement usingductility. It does not require converting the capacity curve to spectral coordinates. The nonlinear force-displacement relationship between base shear and displacement is replaced with an idealized bilinear relationship to calculate the effective initial lateral stiffness K_e and post yield stiffness K_s and effective yield strength V_y of the structure. The target displacement in displacement coefficient method (DCM) is computed as,

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

C_0= modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement ofthe building MDOF system.

C_1= modification factor to relate the expected maximum inelastic displacement calculated for linear elastic range.

C_2= modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strengthdecoration.

C_3= modification factor to represent increased displacement due to dynamics $P-\Delta$ effects.

Later FEMA 356 (2000), presents an incremental improvement to the first generation procedure i.e. FEMA 273 in respect of technical updates to the analytical requirement and acceptance criteria based on the information gained from the physical application in engineering practice and case study given by FEMA 349 [4, 7].

Table 4: Assumed preliminary data required for analysis of frame

Sr. No	Particulars	Assumptions			
1	Type of structure	Multi-storied rigid frame (moment resisting frame)			
2	Seismic zone	IV (table 2 I.S. 1893:2002(part 1)			
3	No. of stories	Ten storied (G+9)			
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 stress (f_u) 485000 kN/m^2			
		i. Expected yield strength (f_e) 456500 kN/m^2			
		j. Expected tensile stress (f_{ue}) 533500 kN/m^2			
8	Size of columns	(obtained from gravity analysis)			
		Floors	Size of columns	Main bars(Tor)	Shear bars (Tor)
		01-03	650 mm x 650 mm	16 No-12 mm	10mm@ 110 mm c/c
		04-06	500 mm x 500 mm	10 No-12 mm	08mm@ 100 mm c/c
		07-10	400 mm x 400 mm	08 No-12 mm	08mm@ 100 mm c/c
9	Size of beams	Both longitudinal and lateral (obtained from gravity analysis)			
		Floors	Size of columns	Main bars(Tor)	Shear bars (Tor)
		01-03	300 mm x 450 mm	4No-12 mm	8mm@ 110 mm c/c
		04-06	300 mm x 450 mm	4 No-12 mm	8mm@ 100 mm c/c
		07-10	300 mm x 450 mm	3 No-12 mm	8mm@ 100 mm c/c
10	Depth of slab	150 mm thick			
12	Type of soil	Rock			
13	Response spectra	As per I.S. 1893:2002(part1) compatible for 5 % damping			

Next generation procedures

Both first generation (ATC 40, FEMA 273) and second generation (FEMA 356) methods have shortcomings as [11];

1. Current procedure predict structural response and demand based on the global behaviour but evaluates performance with reference to damage sustained by individual components resulting (poorest performing element) leading towards misunderstanding of actual structural response.
2. Much of the acceptance criteria contained in document are based on judgement irrespective of laboratory test results or evidence leads towards questioning the reliability of procedures.
3. The guidelines are extensively conservative compared to perspective design criteria.
4. Performance levels defined in document does not directly address primary concerns of owner and designer i.e. probable repair costs, downtime and casualties

Based on these shortcomings FEMA 440 proposed some improvements in capacity spectrum and displacement coefficient method. The improved capacity spectrum method presented in FEMA-440 document suggested empirical expression to calculate effective period and effective damping independent of hysteretic model type and Post-yield slopes. Improved displacement coefficient presented in FEMA 440 is accepted by ASCE-41. Modification of coefficient C_1, C_2, and C_3 is done [12].

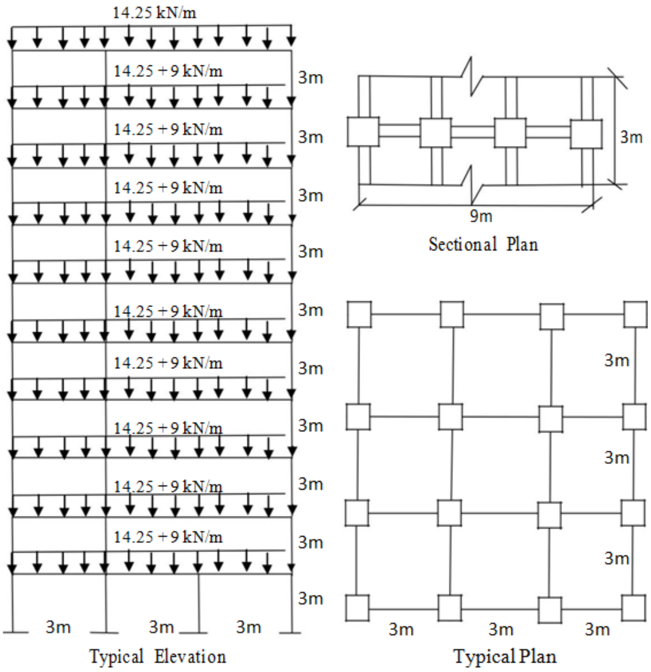


Figure 1: Typical layout of example building

Building frame

The example 2D- RC frame is 3 bays, 10 storied bare frame representing high rise RC building frame. The building is designed in accordance to IS 456: 2000(rev) [13] and IS 13920:1993 [15] ductility provision. The width of bay is 3m and height of each storey is 3m. Figure 1, describes dimension of building and member designation. The preliminary data assumed and gravity design result is presented in table 4. For the example building all floors carries a dead load of 4.75 kN/m^2 and live load of 3kN/m^2 except for roof level. The earthquake forces are calculated for the dead load plus the percentage of imposed load as defined in Table 8 of I.S 1893 (Part 1): 2002 [14]. The imposed load on roof is assumed to be zero as per clause No.7.3.2 of I.S 1893 (Part 1): 2002. The lumped masses of each floor, base shear force and vertical distribution of base shear (obtained as per clause no. 7.7.1 of I.S. 1893(part 1): 2002) are presented in table 5.

Table 5: Structural parameters of example building

Storey Level	Weight (kN)	Mass (x 10^3 kgs)	Vertical shear distribution (kN)	Modal time period (secs)	Modal frequency (rad/sec)	Lumped Stiffness (x 10^3 kNm)
Roof	441.45	45.00	76.57	0.0309	203.11	130.06
9th floor	499.50	50.92	70.17	0.0344	182.48	130.06
8th floor	499.50	50.92	55.45	0.0366	171.60	130.06
7th floor	553.16	56.39	47.01	0.0420	149.47	130.06
6th floor	593.33	60.48	37.05	0.0526	114.84	317.57
5th floor	593.33	60.48	25.73	0.0718	119.34	317.52
4th floor	694.41	70.84	19.27	0.0718	87.471	317.52
3rd floor	788.06	80.33	12.30	0.1047	59.972	906.90
2nd floor	788.06	80.33	5.47	0.1857	33.825	906.90
1st floor	788.06	80.33	1.37	0.4949	12.695	906.90

Pushover Analysis

Since mid 90s, pushover analysis finds its way to seismic guidelines viz. SEAOC (1996), ATC 40 (1996), FEMA 273/274 (1997), FEMA 356/357 (2000), ATC 55 (2005) and FEMA 440 (2005). Procedure consists of applying vertical distribution of lateral loads to a model which captures the material non-linearity of an existing or newly design structure. Loads are increased monotonically until the peak response of the structure is obtained on a base shear versus roof displacement plot.

Performance based evaluation procedures (ATC 40, FEMA 356, FEMA 440) documents have published modeling parameters, acceptance criteria, and procedures for pushover analysis. In present study second generation procedure FEMA 356 guidelines related to modeling parameter and acceptance criteria is adopted. The document put forth two sections to determine the yielding of frame member during the pushover analysis as shown in figure 2 viz. deformation-controlled (ductile action) or force-controlled (brittle action) of plastic hinge. Figure 2(a) represents the idealized inelastic force-deformation relationship for displacement-controlled action under flexure. Points labeled A, B, C, D, E represents various performance levels expressed directly in terms of strain, curvature, rotation, or elongation. Figure 2(b) represents force-deformation relationship for plastic hinge under force-control (shear failure) [7]

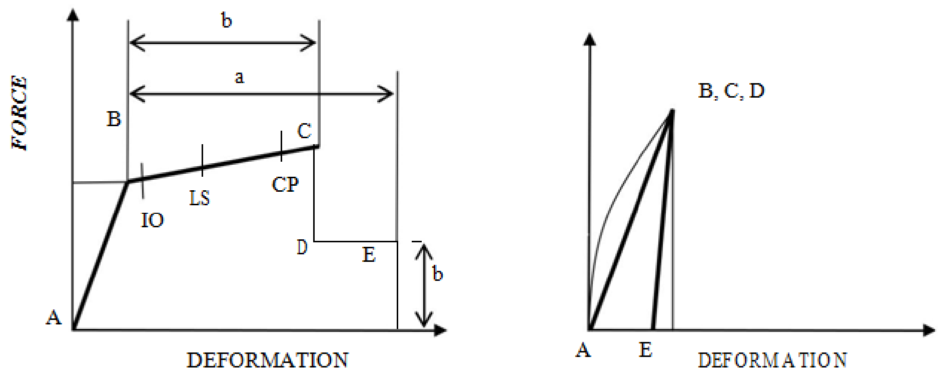
Table 6: Modeling parameters and numerical acceptance criteria for nonlinear procedures (reinforced concrete columns) [7]

Conditions			Modeling Parameters			Acceptance Criteria				
			Plastic rotation angle (radians)		Residual strength ratio	Plastic rotation angle (radians) Performance level				
						IO	Component type			
							Primary		Secondary	
$\frac{P}{A_g f_c'}$	Trans. Reinf.	$\frac{V}{b_w d\sqrt{f_c'}}$	a	b	c		LS	CP	LS	CP
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.0050.0150.02			0.02	0.03

The parameters (a, b) represents the portion after plastic deformation (yield). Parameter (c) represents reduced resistance after sudden reduction from C to D. Numerical values of a, b and c adopted for present example is presented in table 6 and table 7. These parameters depend on the sectional properties such as, percentage of steel (tension and compression), balanced percentage of steel, design shear strength, design axial load, characteristic strength and cross-sectional dimensions.

Table 7: Modeling parameters and numerical acceptance criteria for nonlinear procedures (reinforced concrete beams) [7]

Conditions			Modeling Parameters			Acceptance Criteria				
			Plastic rotation angle (radians)		Residual strength ratio	Plastic rotation angle (radians) Performance level				
						IO	Component type			
							Primary		Secondary	
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.	$\frac{V}{b_w d\sqrt{f_c'}}$	a	b	c		LS	CP	LS	CP
≥0.5	C	≤3	0.02	0.03	0.2	0.0050.010		0.02	0.02	0.03



(a) Deformation –controlled(flexure) (b) Force-controlled (shear failure)

Figure 2: Idealized inelastic force-deformation relationship

Acceptance criteria or performance levels for the plastic hinge formed near the ends of columns and beams are represented by IO (Immediate Occupancy), LS (Life Safety), and CP (Collapse Prevention). Default values given in software SAP 2000 V 14.0 [16] is used to represent P-M interaction curve (ACI code 2000), stress-strain curve for concrete (Kent and Park), M-θ relationship which represents acceptance criteria corresponding to performance levels.

Seismic Performance

The present study aims to assess seismic response of ten storey building frame for preliminary data considered in table 4. The static nonlinear analysis (pushover analysis) of lateral seismic forces is preferably applied after initial pushover analysis for dead load plus live load combination. Figure 3 shows the capacity responses of deformation-control actions of application of forces in reference to FEMA 356 (2000) guidelines. The maximum value of base shear, ultimate roof displacement and the ratio of base shear value obtained from pushover analysis to IS 1893:2002 codes are presented in table 8.

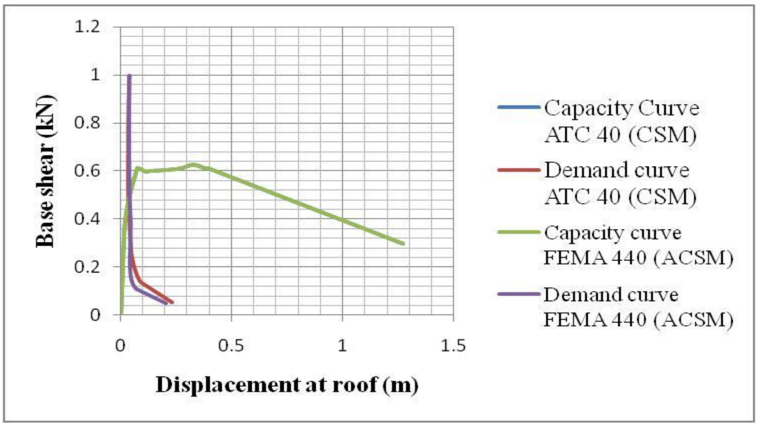
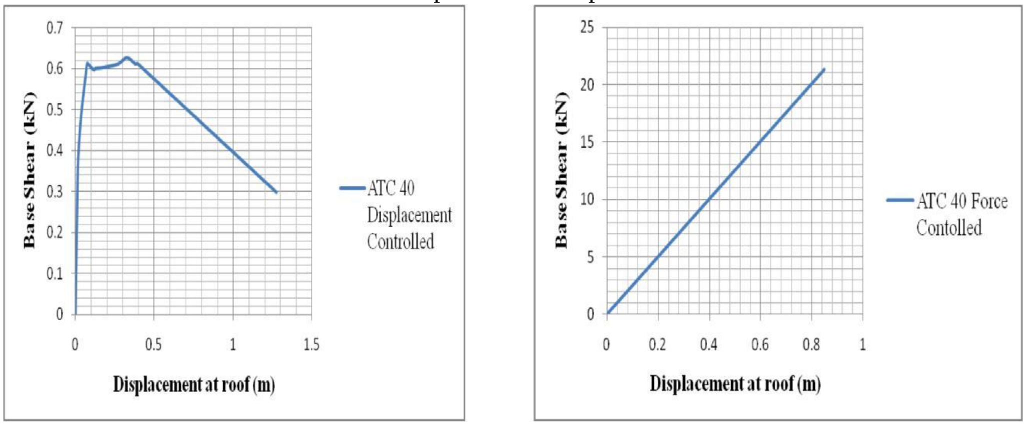


Figure 3: Capacity curve of building

Table 8: Characteristics of performance point according to performance based evaluation procedure presented in first, second and next generation procedures

Performance evaluation procedure	Maximum base shear (kN)	Ultimate displacement of roof (m)	Ratio of maximum base shear to base shear obtained by IS 1893: 2002 (part 1)
ATC 40 (CSM)	371.264	0.048	1.06
FEMA 356 (DCM)	404.247	0.068	1.154
FEMA 440 (ACSM)	382.553	0.054	1.092
FEMA 440 (MDCM)	420.168	0.078	1.20

Figure 4 represents the capacity response of two actions of the plastic hinges up to failure, once when the hinge is subjected to the shear failure and another one to flexural failure. The maximum base shear of the structure and the ultimate roof displacement are presented in table 9.



(a) Displacement-Controlled action (b) Force-Controlled action

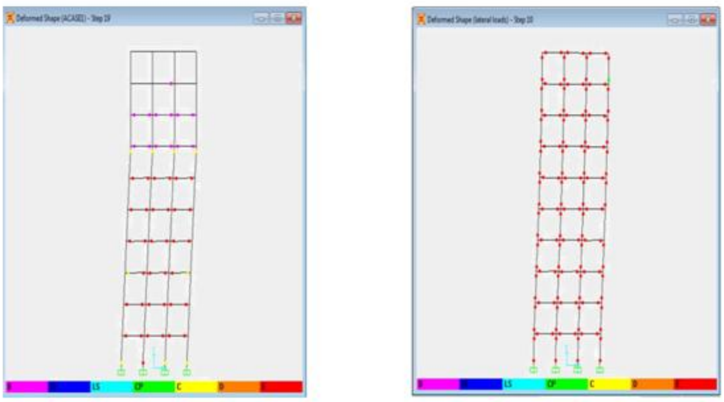
Figure 4: Capacity curve of building for different control actions for plastic hinge

Table 9: Characteristics of performance point for (a) displacement-controlled and (b) force-controlled options for plastic hinges according to performance based evaluation procedure presented in first, second and next generation procedures

Performance evaluation procedure	Maximum base shear (kN)		Ultimate displacement of roof (m)	
	Displacement-controlled	Force-Controlled	Displacement-controlled	Force-Controlled
ATC 40 (CSM)	371.264	733.870	0.048	0.057
FEMA 356 (DCM)	404.247	674.184	0.068	0.052
FEMA 440 (ACSM)	382.553	733.871	0.054	0.057
FEMA 440 (MDCM)	420.168	653.555	0.078	0.051

Figure 5 shows the plastic hinge patterns at final load step of loading and control options which govern the behavior of plastic hinge during analysis. Performance levels are illustrated by appropriate color codes based on acceptance criteria.

Figure 6 shows the capacity curve of the frame structure according to FEMA 440 (MDCM) approach. Displacement ductility represents a simple quantitative indication of severity of the peak displacement to the displacement necessary to initiate yielding for the present case it works out to be 3.795. Ductility ratio directly affects hysteretic behavior in reinforced concrete structures. Lateral deformations at the performance point are checked against the deformation limits as illustrated in table 10. Maximum total drift is defined as storey drift corresponding to performance point. Maximum inelastic drift is the portion total drift beyond the effective yield point.



(a) Displacement-controlled option (b) Force-controlled option

Figure 5: Plastic hinge patterns at final load step - displacement and force controlled actions of plastic hinge during analysis

Table10: Story drift ratio of the present analysis and deformation limits

Storey drift	ATC 40 (CSM)	FEMA 440 (ACSM)	Performance level			
			Immediate occupancy	Damage control	Life safety	Structural stability
Maximum total drift	0.048	0.054	0.01	0.01-0.02	0.02	0.33□$S_i W_i$□ / (0.01853) at roof
Maximum inelastic drift	0.0288	0.0348	0.005	0.005-0.015	No limit	No limit

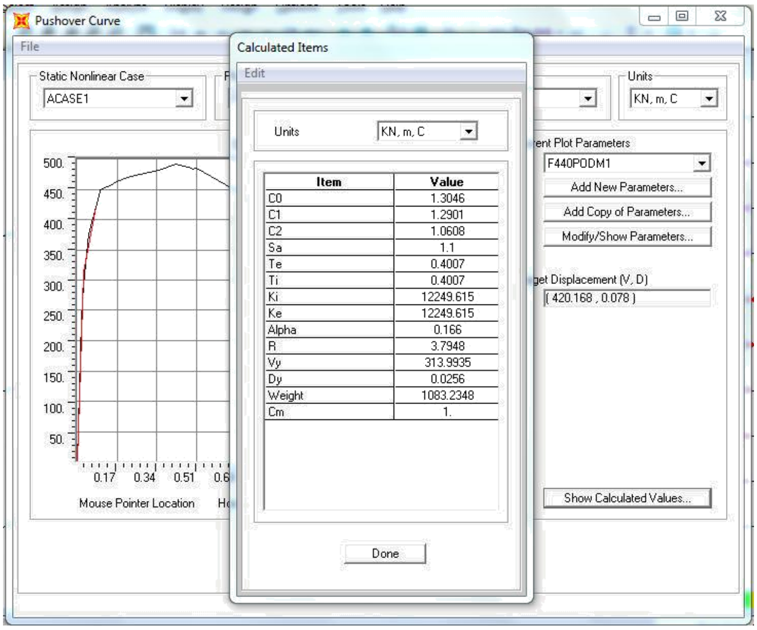


Figure 6: Capacity curve of the frame according to FEMA 440 (MDCM)

For the structural stability, the maximum total drift in story i at the performance point should not exceed the quantity of 0.33□S_i/W_i□, where$S_i$is the total calculated shear in story i and$W_i$is the gravity load at story i [7].

II. Conclusions

Performance evaluation procedure recommended in first, second and next generation procedure utilizes pushover analysis to estimate the capacity of RC structures towards given seismic supply. The documents provide acceptance criteria in terms of performance levels based on transient and permanent drift. In order to represent inelastic behavior structural elements are modeled with elastic hinges at the ends for both beams and columns. The formation of collapse mechanism by yielding of plastic hinges inelastic behavior of components can be concluded as;

- The maximum base shear of the structure and ultimate roof displacement obtained for first, second and next generation procedure are in close tolerance to values of base shear deduced from I.S 1893: 2002 (part 1).
- The document describes two actions to determine yielding of frame members during inelastic analysis viz.;

(a) Displacement-controlled (flexure) and, (b) force-controlled (brittle) actions. When the frame was modelled for deformation-controlled action shows sequence of formation of plastic hinges in the beams only. This shows building clearly behaves like strong column-weak beam mechanism.

- When the frame was modelled for force-controlled (brittle) shows the formation of hinges in columns because of inadequate shear reinforcement.
- Lateral deformations of example building at the performance points for ATC 40 (CSM) and FEMA 440 (ACSM) when checked against maximum total drift, maximum inelastic drift and structural stability that it’s safe for life against seismic loads.
- The ductility ratio of the frame structure according to displacement coefficient method FEMA 356 (DCM) and FEMA 440 (MDCM) gives a simple quantitative indication of the severity of the peak displacement relative to the displacement necessary to initiate yielding.

- Comparing both the action i.e. displacement-controlled and force-controlled actions, force-controlled actions place the structure in the severity level.
- Present analysis shows that structural behaviour inelastic zone depends on design and ductility provision present in design codes any missing design data will lead towards the misjudging of performance.
- Though the various performance evaluation procedures are unique in their methods but results in conflicting results rising question that, when and where they should be used.
- The procedure results in formation of yield mechanism but fails to quantify actual damage occurred to building
- There is a need of defining a proper damage indicator to estimate actual damage for ductility and structural stability.

III. Abbreviations

PBSD	Performance Based Seismic Design
PCA	Portland Cement Association
SEAOC	Structural Engineers Association of California
ATC	Applied Technological Council
FEMA	Federal Emergency Management Agency
NRHA	Nonlinear Response History Analysis
NLSP	Nonlinear Static Pushover Analysis
ESDOF	Equivalent Single Degree of Freedom
CSM	Capacity Spectrum Method
DCM	Displacement Coefficient Method
MCSM	Modified Capacity Spectrum Method
MDCM	Modified Displacement Coefficient Method

Conflict of interest

I (we) declares that there is no conflict of interest with any financial organization regarding the material discussed in the manuscript

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