Performance-based Seismic Assessment of MRF using Multi-Mode Pushover Analysis

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ABSTRACT

Ten storied 3-bays reinforced concrete bare frame designed for gravity loads following the guidelines of IS 456 and IS 13920 for ductility is subjected to lateral loads. The seismic demands of this building were calculated by following IS 1893 for response spectra of 5% damping (for hard soil type). Plastic hinges were assigned to the beam and column at both ends to represent the failure mode when member yields. Multi-mode pushover analysis was performed to evaluate the performance of the building in reference to first (ATC 40), second (FEMA 356) and next-generation (FEMA 440) performance-based seismic design procedures. Base shear versus top displacement curve of structure, known as pushover curve. Lateral deformation corresponding to performance point proves the building capability to sustain a certain level of seismic loads. The failure is represented by a sequence of formation of plastic hinges. The results of pushover analysis are compared with results obtain from time-history performed on the structure. The study aims to understand the effects of multi-mode lateral load pattern on the performance of the structure.

Keywords–*Performance-based seismic design, lateral load pattern, example MRF, Nonlinear time-history, nonlinear response parameters*

I. INTRODUCTION

In India, the reinforced concrete (RC) structure is designed in accordance to IS 456:2000 [1] and IS 1893: 2002 [2]. When these structures are subjected to the moderate earthquake they have buck minor damages. In case of the strong earthquake, they have to fetch large damages and led loss of life. A summary of such damages and seismic events are given in Table 1[3]. IS 456:2000 and IS 1893:2002 are force-based. The structural components are designed for forces within the elastic limit. The inelastic behavior of structural members is taken care by applying the response modification and ductile detailing. The ductile detailing of reinforcement is done as per the provision of IS 13920:2016 [4]. Such an indirect approach results in the improper judgment of the non-linear behavior of RC structures.

Table 1: Fatalities and damages due to earthquakes in India [3]

Sr. No.	Earth-quake	Year	Int.	Fata-lities
1	Latur (khillari)	30,Sept. 1993	M 6.2	9748
2	Chamdi	29, Mar. 1999	M 6.8	103
3	Gujrat (Bhuj)	26, Jan. 2001	M 7.7	20000
4	Off-west coast (Northern Sumatra)	26, Dec. 2004	M 9.1	15000
5	Kashmir	08, Oct. 2005	M 6.9	1350
6	Gangtok (Sikkim)	18, Oct. 2005	M 6.9	118
7	North India	25,April 2015	M 7.8	8900
8	Bihar	25,April 2015	M 7.3	4
9	North India	12, May 2015	M 5.6	3 injured
10	Dibrugarh (Assam)	28, June 2015	M 7.8	57
11	Kashmir	26, Oct. 2015	M 7.7	4
12	Manipur	04, Jan 2016	M 6.7	6
13	Tripura	03, Jan. 2017	M 5.7	1

Performance-based seismic design (PBSD) has emerged as the best alternative to the force-based procedure. PBSD provides the reliable and realistic understanding of the probable structural performance in future earthquakes. PBSD involves following key elements; defining the performance objectives, estimating the seismic demands, and evaluating the capacity of the structure [5-7].

In PBSD the structure's capacity is evaluated by performing pushover analysis (POA). In POA a predefined lateral load pattern is applied to the structure up to a target displacement. There are several methods defined in codes to estimate the lateral load pattern to be applied to the RC structure, but generally the elastic first mode- lateral load pattern is used in practice [8]. Elastic first mode lateral load pattern provides reasonably good results for medium storey structures, but for higher storey structures it underestimates the displacements at the storey and inter-storey level. Also, significant effects on the other response parameters like base shear, storey shear, and stiffness are reported in the past literature [9-11].

In this study, we have subjected example MRF to multi-mode lateral load patterns using the higher mode contributions. The storey displacements, inter-storey displacements, base shear, and stiffness were estimated for a example MRF. The obtained results are storey drift were compared with storey drift obtained from time-history response of the same example MRF

II. PERFORMANCE-BASED SEISMIC DESIGN

In ATC 40 [12], FEMA 440 [13] and FEMA 445 [14] PBSD is documented as generalized design philosophy in which design criteria are expressed in terms of achieving the stated performance objectives for a stated set of seismic hazard [5]. PBSD is an iterative process, which begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment of whether the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved [5,14]. Fig. 1 displays the flowchart representing key steps in the PBSD procedure.

In PBSD the building performance levels are defined in terms of damage state of structural and nonstructural components. The various performance levels defined in PBSD documents are; Operational Level (OP), Immediate Occupancy (IO), Damage Control Range (DCR), Life Safety (LS), Limited Safety Range (LSR), Collapse Prevention Level (CP), and Collapse (C). These ranges are assessed by attainment of drift and rotation by a member beyond its elastic limit [15].

Step 1
Select Performance Objectives
(These are the statements of acceptable level of damages to structural and non-structural components)

↓

Step 2
Preliminary Design
(These are based on selected response parameters such as displacement, stiffness, dissipated energy, etc)

↓

Step 3
Assess performance capability
(This involves the comparison of demand imposed and structures capacity)

↓

Step 4
Redesign/Reassessment
(This is done when stated performance objectives do not achieved)

Fig. 1 PBSD Flowchart

In order to evaluate the inelastic behavior of RC structures the nonlinear static or dynamic analysis is needed. The nonlinear dynamic analysis is time consuming and involves more computational efforts hence not preferred in common practice.

Nonlinear Static Analysis (NSP) is simple, the results are closer to dynamic analysis became common in practice. PBSD has provided various performance evaluation procedures using NSPs. They are Capacity Spectrum Method (CSM)[13], Displacement Coefficient Method (DCM)[17], Improved Capacity Spectrum

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Method (ACSM) [13], and Improved Displacement Coefficient Method (ADCM) [14].

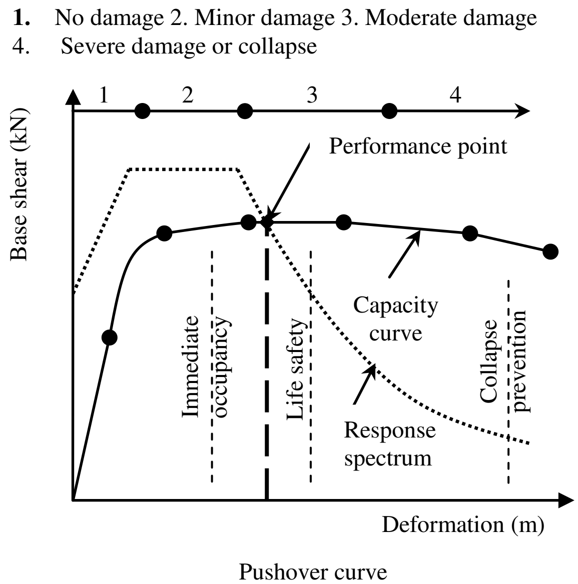

Fig. 2 Typical NSP Procedure

NSP is carried out to examine the deformation of and damage pattern of the structure. A typical lateral load-roof displacement performance relationship for a structure obtained from the NSP is shown in Fig. 2. The internal forces and deformations computed at the target displacement levels are estimates of strength and deformation capacities which are then compared with the expected performance objectives and demands. The sequence of component cracking, yielding, and failure, as well as the history of deformation of the structure, can be traced as the lateral loads (or displacements) are monotonically increased [8].

The NSP identifies the sequence of component damage or failure and if the behavior is modeled properly, the ultimate load and the drift at the failure of the structure can be determined. It is a simple and promising approach for determining the lateral load resistance of RC structures [8].

III. EXAMPLE MRF

The structural system considered for this study is Reinforced Concrete (RC) frame of 3 bays, 10 storey representing medium rise structure, located in seismic zone IV as per IS 1893 [2]. The width of each bay is 3m and height of each storey is 3m. Fig. 3 describes the geometry of the building and member designation. The structural system supports gravity loads of magnitude 4.75 kN/m^2 (dead load) and 3kN/m^2(live load). The RC design of the building was based on IS 456 [1]. The gravity design result is presented in Table 2. The seismic demands on building are calculated following IS 1893 [2]. Further details such as total height, weight, storey shear and modal parameters are tabulated in Table 3. The frame is designed with M 25 grade of concrete (having 28 days compressive strength of 25 MPa)and Fe 415 grade reinforcement (having characteristic yield strength of 415 MPa) [1]. The gravity design result is presented in Table 3

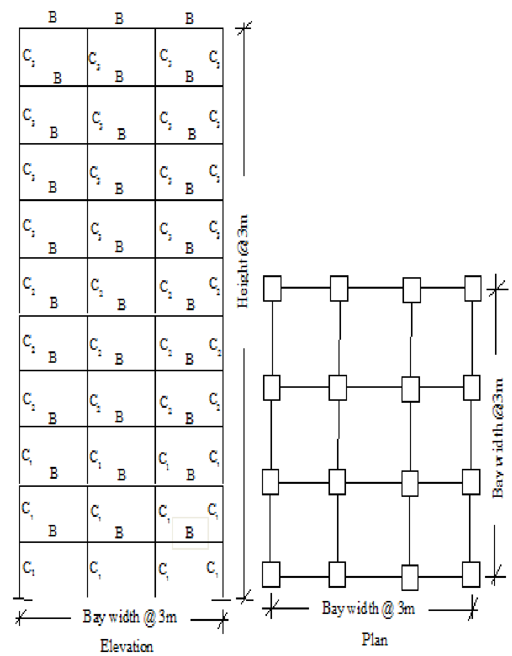

Fig 3 Geometry of the building and member designation

Table 2 Structural parameters of example MRF

Storey Level	Weight (kN)	Vertical shear distribution (kN)	Time period (secs)	Frequ -ency rad/s
Roof	158.75	41.93	0.048	130.5
9	206.15	42.19	0.055	112.4
8	206.15	33.33	0.056	110.8
7	206.15	25.52	0.067	93.45
6	217.63	19.87	0.067	92.64
5	229.10	14.57	0.084	74.35
4	229.10	9.32	0.116	54.04
3	250.95	5.78	0.165	37.88
2	273.15	2.80	0.296	21.20
1	273.15	0.702	0.827	7.60

Table 3: Gravity design results for RC sections

Fl. No	Mem-bers	Size		Bars	Stirrups
		Width (mm)	Depth (mm)		
1-3	Column (C_1)	650	650	16 Nos-12mm	10mm@110mm c/c
4-6	Column (C_2)	500	500	10 Nos-12mm	8mm @100mm c/c
7-10	Column (C_3)	400	400	8 Nos-12mm	8mm @100mm c/c
1-3	Beam (B)	300	450	4Nos-12mm	8mm @110mm c/c
4-6					8mm @100mm c/c
7-10				3 Nos-12mm	8mm @100mm c/c

IV. LATERAL LOAD PATTERN

In Earlier versions of pushover methods documented in PBSD the lateral load distribution applied represents the dynamic inertia load by an equivalent static load. For structures with mainly first mode motion, the lateral load is normally taken as being uniformly distributed over the stories or an inverted triangular code type seismic load. There are two major drawbacks of conventional POA: (1) higher mode effects are neglected, and (2) neglect the changes in the dynamic properties of the structures that lead to a continuously altered loading pattern [18, 19].

The need for accounting for higher-modes led towards the generation of Multi-Mode Pushover Analysis (MMPOA) [18]. The MMPOA follows the procedures of a typical POA except that load patterns are based not only on the first mode but higher modes. The building is modeled and mode shapes and periods are determined. Load pattern, based on the modes of interest, is determined. For each load pattern, forces are applied to the building with increasing magnitude until a collapse mechanism is formed. PBSE procedures available in PBSD are used to evaluate various response parameters.

The load patterns for the MMPOA are based on elastic mode shapes. Any number of modes can be used; because the mass participation factors become increasingly smaller at the higher modes. The load patterns are determined by multiplying the weight (or mass) at each level by mode shape that is the load patterns for a particular mode are proportional to $W\phi_i$.

Table 4-7AND Fig. 4 shows the calculations for the load patterns for the first three modes and combined mode of example MRF. The seismic weight at each level is multiplied by modal amplitude at that level for a particular mode. In this case, the patterns were all arbitrarily normalized so that the base shear equaled 196.05kN.Once load patterns for each mode are determined; POA is performed for each load pattern.

Table 4: First mode load pattern for example MRF

Floor Level	Weight (kN)	Mode 1 ($Ø_i$)	$W_iØ_i$	F_x (kN)
Roof	158.75	0.1028	16.32	26.83
9	206.15	0.0980	20.20	33.22
8	206.15	0.0900	18.55	30.51
7	206.15	0.0791	16.31	26.81
6	217.63	0.0655	14.25	23.44
5	229.10	0.0535	12.26	20.15
4	229.10	0.0403	9.23	15.18
3	250.95	0.0265	6.65	10.93
2	273.15	0.0150	4.10	6.74
1	273.15	0.0050	1.37	2.25
Total	**2250.8**			**196.05**

Table 5: Second mode load pattern for example MRF

Floor Level	Weight (kN)	Mode 2 ($Ø_i$)	$W_iØ_i$	F_x (kN)
Roof	158.75	-0.0998	-15.84	-59.14
9	206.15	-0.0731	-15.07	-56.25
8	206.15	-0.0294	-6.06	-22.62
7	206.15	0.0212	4.37	16.31
6	217.63	0.0645	14.04	52.39
5	229.10	0.0832	19.06	71.15
4	229.10	0.0846	19.38	72.34
3	250.95	0.0674	16.91	63.13
2	273.15	0.0423	11.55	43.13
1	273.15	0.0153	4.18	15.60
Total	**2250.8**			**196.05**

Table 6: Third mode load pattern for example MRF

Floor Level	Weight (kN)	Mode 3 ($Ø_i$)	$W_iØ_i$	F_x (kN)
Roof	158.75	0.0975	-0.098	156.267
9	206.15	0.0311	-0.029	45.989
8	206.15	-0.0539	0.057	-91.182
7	206.15	-0.0975	0.100	-159.768
6	217.63	-0.0668	0.068	-108.050
5	229.10	-0.0072	0.007	-10.980
4	229.10	0.0539	-0.055	88.159
3	250.95	0.0798	-0.082	130.170
2	273.15	0.0632	-0.065	102.958
1	273.15	0.0260	-0.027	42.488
Total	**2250.8**			**196.05**

Table 7: Multi-mode load pattern for example MRF

Floor Level	Weight (kN)	Mode 1 F_x	Mode 3 F_x	F_x (kN)
Roof	158.75	26.83	156.27	183.10
9	206.15	33.22	45.99	79.21
8	206.15	30.51	-91.18	-60.68
7	206.15	26.81	-159.77	-132.96
6	217.63	23.44	-108.05	-84.61
5	229.10	20.15	-10.98	9.17
4	229.10	15.18	88.16	103.34
3	250.95	10.93	130.17	141.10
2	273.15	6.74	102.96	109.69
1	273.15	2.25	42.49	44.73
Total	**2250.8**			**392.10**

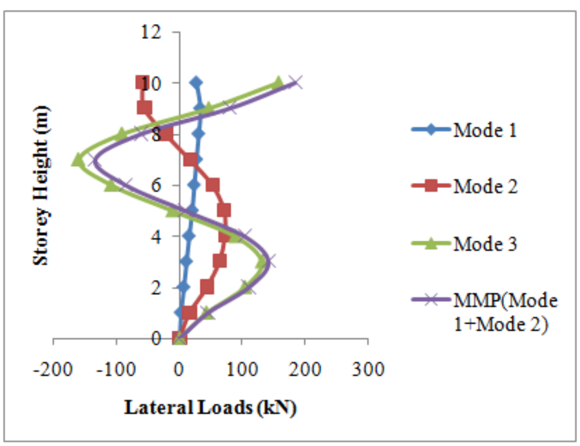

Fig. 4 Lateral Load Pattern applied on MRF

V. PUSHOVER ANALYSIS

Displacement controlled POA was performed to estimate the capacity of the structure. The equivalent lateral-force distributions adopted for pushover analysis are mode-1, mode-2, mode-3 and combination of mode-1 and mode 3. POA was performed using the SAP 2000 v 17.0 [16].

All beam and column are modeled using the plastic hinge. For beam the axial load effects are ignored considering the rigid floor diaphragm effect. For column, the effect of axial loads is considered. Performance based evaluation procedures given in first, second and next-generation documents have published modeling parameters, acceptance criteria, and procedures for POA. In present study second generation procedure FEMA 356 guidelines related to modeling parameter and acceptance criteria were adopted.

These documents put forth two actions of plastic hinges viz. deformation-controlled (ductile action) or force-controlled (brittle action) as shown in Fig. 5.Fig. 5 (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. Fig. 5 (b) represents the force - deformation relationship for plastic hinge under a force-control (shear failure) [17]

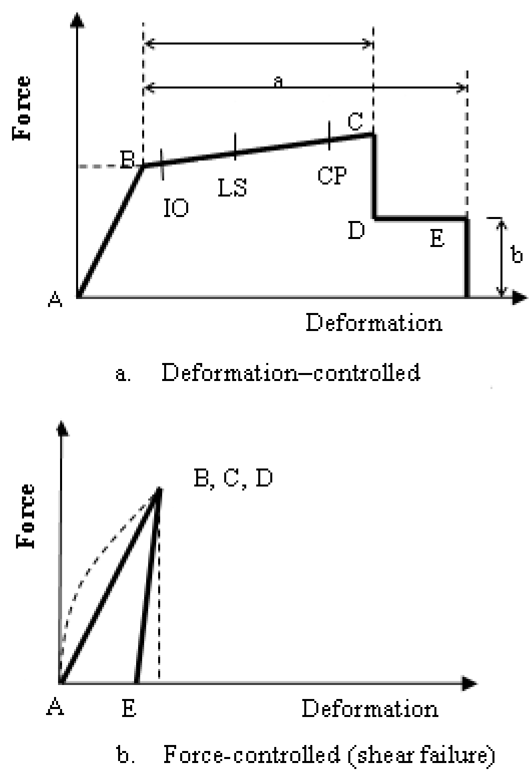

Fig. 5 Idealized force-deformation relationship of plastic hinges

The parameters (a, b) represent 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 MRF is presented in FEMA 356. 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).

In this study, beams and column elements were modeled as nonlinear frame elements by assigning concentrated M3 and P-M3 plastic hinges, respectively, to both the ends [21].The responses of the example MRF were studied in terms base shear, roof displacement, obtained against the applied loads.

The result of POA is presented in the form of a capacity curve, which is typically a plot of rooftop displacement versus base shear. Figure 6-9 presents the base shear and the rooftop displacement of MRFs for different load patterns. The different lateral load pattern applied on example MRF are;

1. Push G – for full gravity load
2. Push 1- for IS 1893 load pattern
3. Push 2- for Elastic Mode 1
4. Push 3 –for Elastic Mode 3
5. Push 4- for combine Elastic Mode 1 and Mode 3

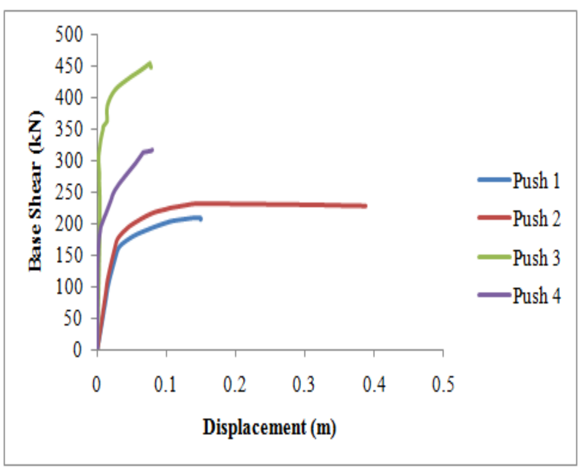

Fig. 6 Pushover curve of example MRF for different lateral load case

The nonlinear response of example MRF is approximated by the values of base shear and displacements obtained at performance point and at collapse as tabulated in Table 8-11.

Table 8 Comparison of POA results with first generation procedures (ATC 40-CSM)

Push case	ATC 40 CSM			
	At performance point		At collapse	
	Base shear (kN)	Displ. (m)	Base shear (kN)	Displ. (m)
Push 1	209.2	0.141	209.24	0.150
Push 2	228.89	0.132	230.51	0.496
Push 3	129.09	0.0023	453.69	0.079
Push 4	285.29	0.048	316.75	0.080

Table 9 Comparison of POA results with next-generation procedures (FEMA 440 –ACSM)

Push case	FEMA 440-ACSM			
	At performance point		At collapse	
	Base shear (kN)	Displ. (m)	Base shear (kN)	Displ. (m)
Push 1	206.27	0.141	209.24	0.150
Push 2	225.01	0.132	230.51	0.496
Push 3	129.09	0.0023	453.69	0.079
Push 4	255.04	0.027	316.75	0.080

Table 10 Comparison of POA results with second generation procedures (FEMA 356 –DCM)

Push case	FEMA 356-DCM			
	At performance point		At collapse	
	Base shear (kN)	Displ. (m)	Base shear (kN)	Displ. (m)
Push 1	207.63	0.179	209.24	0.150
Push 2	230.45	0.170	230.51	0.496
Push 3	NA	NA	453.69	0.079
Push 4	262.10	0.031	316.75	0.080

Table 11 Comparison of POA results with next-generation procedures (FEMA 440 –ADCM)

Push case	FEMA 440-ADCM			
	At performance point		At collapse	
	Base shear (kN)	Displ. (m)	Base shear (kN)	Displ. (m)
Push 1	207.63	0.218	209.24	0.150
Push 2	230.16	0.204	230.51	0.496
Push 3	NA	NA	453.69	0.079
Push 4	316.77	0.160	316.75	0.080

From the obtained values we conclude that in push 3 load case where higher mode contribution are considered there has been significant fall in seismic demand of structure which is not in equilibrium with capacity of structure, hence performance point was not obtained. When combination of higher modes are used results are very much promising as compared to code based lateral load pattern

VI. CONCLUSION

The multi-mode pushover procedure (MMP) was developed to account for the effects of higher modal response in POA. In this method the elastic response modes has been added to evaluate building response. Current POA procedures uses approximate inertia load pattern define by FEMA or IS 1893 which are not able to provide critical oversight over failure mode of building influenced by higher modes

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