Shaikh Zeeshaan Md. N., et. al. International Journal of Engineering Research and Applications
www.ijera.com
ISSN: 2248-9622, Vol. 16, Issue 8, August 2026, pp 11-18

RESEARCH ARTICLE OPEN ACCESS

Assessment of Seismic Response of High-Rise Moment Resisting Frames through Pushover Analysis

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ABSTRACT
Seismic force introduces inelastic incursions in a structure, which in turn demands adequate strength and ductility. A gravity-based design approach assures that the capability of the structure meets the stated demands, but in an indirect way through the application of a response reduction factor. In the present study an attempt has been made to assess the seismic capabilities of gravity-based design G+12 Moment Resisting Frame [MRF]. For this, the example MRF was subjected to seismic loads using pushover analysis [PoA]. In PoA, the structure was subjected to incremental lateral loads until a target displacement is reached. The capacity of the structure is illustrated through a capacity curve, which shows the relationship between lateral forces and lateral displacements. Parametric studies, which includes base shear, lateral displacement, inter-storey drift, ductility, stiffness and strength were carried out. Non-linear modelling was done following the guidelines of FEMA 440.

Keywords: Base shear, lateral displacement, inter-storey drift, ductility, stiffness, strength

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Date of Submission: 25-07-2026 Date of acceptance: 06-08-2026

I. Introduction

High-rise building has become a characteristic feature of an urban development. The high-rise building has demanded life safety concern especially when subjected to seismic events. Structural designers have shifted their focus on attaining adequate strength and ductility in a structure. In India the structural design is done as per the guidelines of IS 456:200; IS 1893:2016 and IS 13920: 2016. All these codes provide the gravity-based design procedure. However, they address ductility through response reduction factor, but damage assessment is still unanswered.

Performance-based seismic design (PBSD) has emerged as the best alternative toward these seismic code design approaches [1-3]. PBSD has provided four structural analysis procedures, which include (a) Linear Static, (b) Linear Dynamics, (c) Nonlinear Static and (d) Nonlinear Dynamic. A nonlinear dynamic procedure uses real-time seismic history to evaluate the response of a structure but involves a tiresome iterative process. Nonlinear static analysis is a rational approach, but obtained results are in close proximity to nonlinear dynamic processes. Due to its simplicity, it has become common in practice [4-5]. Nonlinear static analysis has two different approaches: (a) Capacity Spectrum Method (CSM) and (b) Displacement Coefficient Method (DCM). In CSM, the inelastic strength and displacement spectra are used for the determination of seismic demands by performing nonlinear analysis of an inelastic single degree of freedom system. The maximum structural response is estimated to be the point where the capacity curve crosses the demand spectrum. For this, 5% damped elastic spectrum of a given seismic demand is lowered in agreement with the structure's response. This intersection point is called the "point of intersection" and a nonlinear response at this point evaluates the structure's performance. Figure 1, illustrates the CSM procedure.

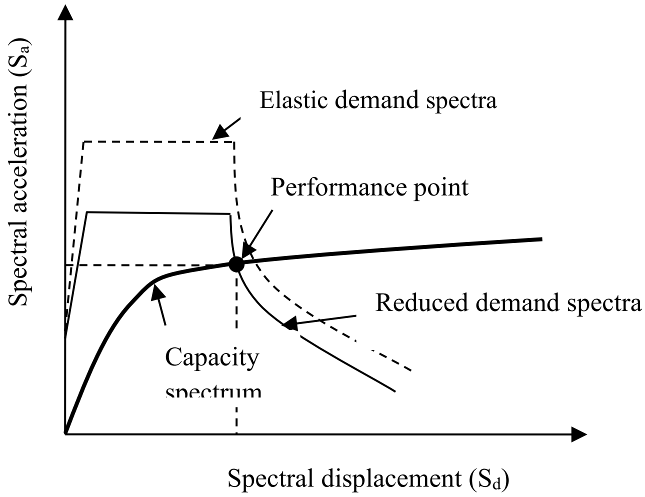


Fig. 1: Capacity Spectrum Method

DCM estimates the maximum displacement by using ductility. The empirical relationship given in equation 1 is used to calculate the displacement demand.

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

Eq. (1)

Where; C_0 is modification factor to relate spectral displacement of an equivalent single degree of freedom system to the roof displacement of multi degree of freedom system. C_1 is modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. C_2 Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. C_3 Modification factor to represent increased displacements due to dynamic *P*-Δ effects.S_a represents spectral acceleration. Figure 2, explain DCM procedure.

In PBSD, identifying and assessing a structure's performance capability is a critical phase in the design process and affects several design choices. Fig. 3 shows the key phases of PBSD. The first step in PBSD is choosing performance objectives, which are defined in terms of the damage caused to both structural and non-structural components. These damages help in the classification of building performance levels. Damage states can be expressed in the form of casualties, direct economic costs, and downtime (time out of service). Methods for estimating losses and communicating these losses to stakeholders are at the heart of the evaluation of PBSD but needs efforts to scale them in numeric value [7-8].

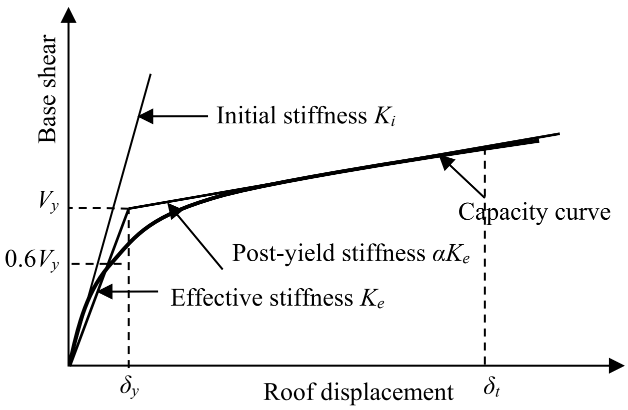


Fig 2: Displacement Coefficient Method

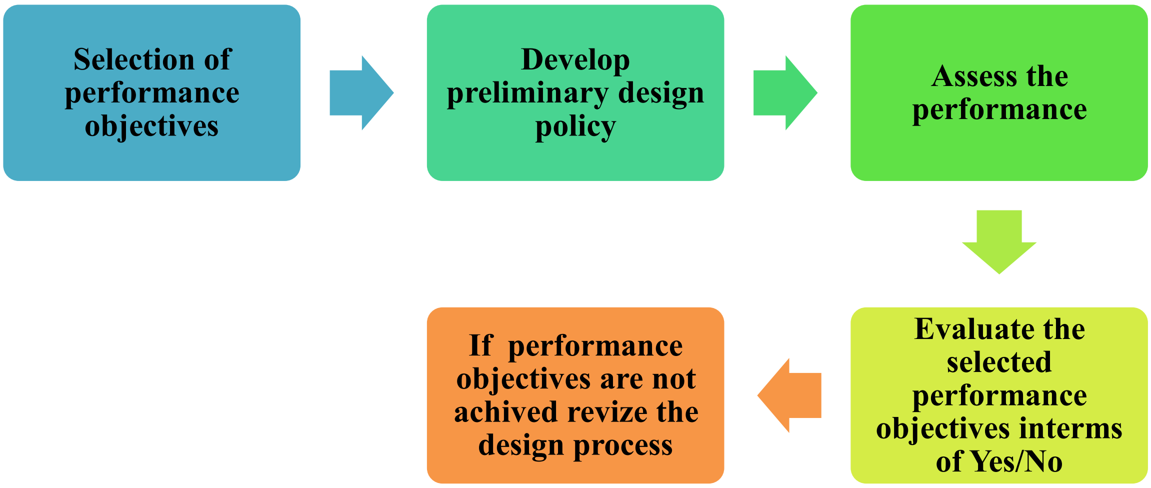


Fig. 3: PBSD Procedure

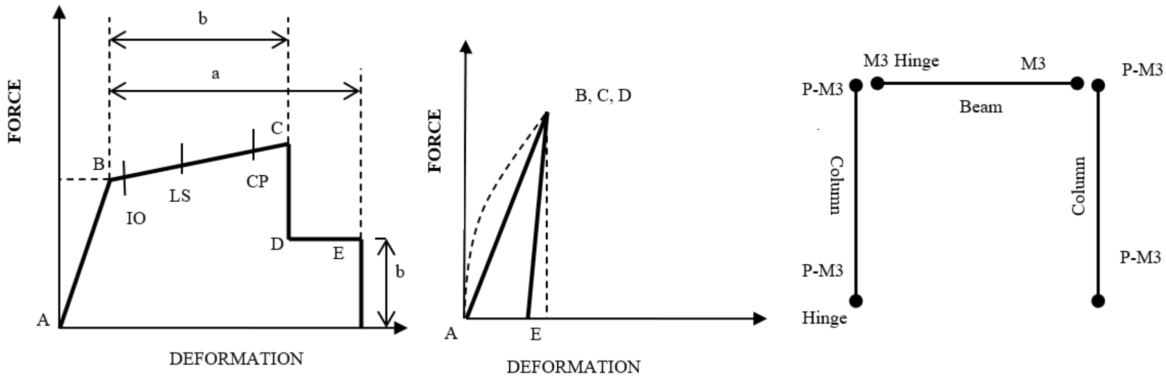
Correctness of evaluation procedure depends on efforts in modelling of structure with plastic hinges. There are two different approaches of actions of hinges (a) force-controlled (shear action) and (b) displacement-controlled (Flexure action). Table 1-2 gives the acceptable limits of inelastic incursions.

Table 6: Modeling parameters and numerical acceptance criteria for nonlinear procedures (reinforced concrete columns) [9]

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
$\leq$ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03

Table 7: Modeling parameters and numerical acceptance criteria for nonlinear procedures (reinforced concrete beams) [9]

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
$\geq$ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.010	0.02	0.02	0.03



(a) Deformation –controlled (flexure) (b) Force-controlled (shear failure) (c) Location of Plastic hinges
Figure 4: Idealized inelastic force-deformation relationship and location of plastic hinges [9]

Location of plastic hinges significantly impacts on nonlinear responses of structure [10]. In POA structure is subjected to incremental lateral loads till the targeted displacement is reached. It has been recommended to selected a set of lateral load patterns which result in upper bound and lower bound values. Literature studies reveal that uniform load pattern provides lower bound values and first-mode lateral load distribution results in upper bound values. The IS-code based trivial load pattern results in average values [7-9]. In present study IS-code based trivial load pattern is used for analysis and all prescribed guideline of FEMA 440 has been used for modelling the structure. ETABS V 22 is used for modelling the example structure [11].

Modelling of example MRF:

Example MRF is a bare frame design for gravity loads and ductility provision given in IS 456; IS 875 (Part 1 & 2), IS 13920 and IS 1893[12-17]. The MRF has Ground plus 12 stories (G+2). A live load of 9 kN/m and imposed dead load of 45 kN/m was applied on the beams (includes loads from partition walls and slabs). For seismic loads MRF is considered to be located in seismic zone V (Zone factor (Z) = 0.36) on medium soil with commercial use, Response factor (R) = 5. The modal time period of the structure was found to be 0.613 secs and modal frequency is 10.251 rad/sec. The seismic weight of structure is 8156.40 kN and a base shear 651.55 kN. The lateral storey drift is shown in figure 5. Figure 6 displays storey-wise lateral load distribution.

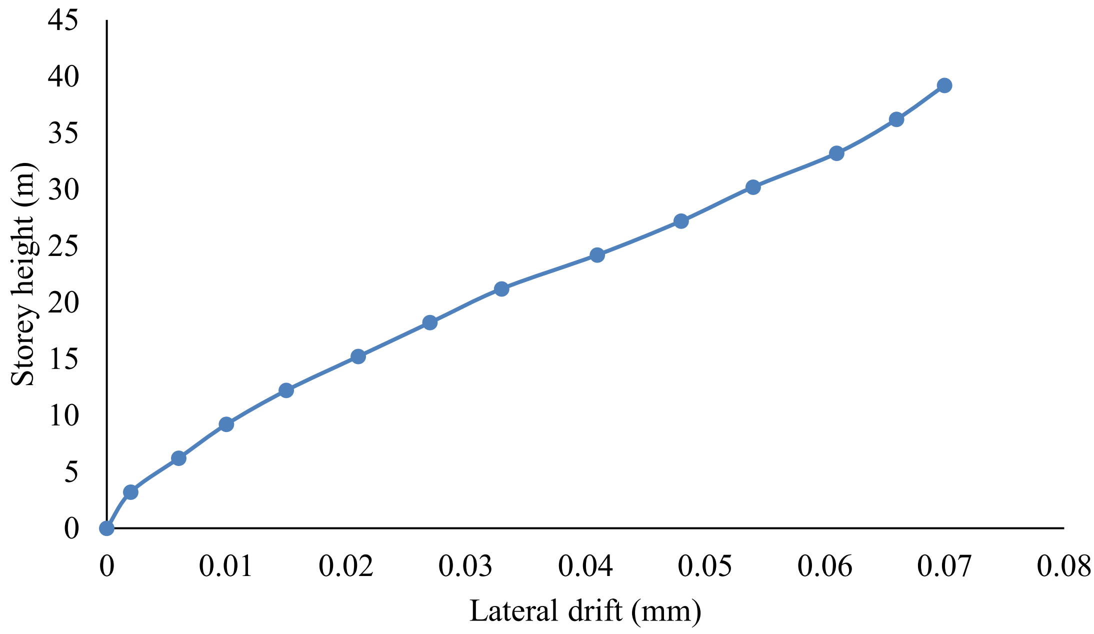


Fig. 5: Storey Displacement of Example MRF

To evaluate nonlinear response, MRF was subjected to PoA for lateral loads obtained from IS 1893: 2016 recommended distribution. The PoA is performed in two steps (a) Push Gravity and (b) Push Lateral. In Push Gravity the structure is subjected to dead load and live load within the force-controlled action and in Push Lateral the structure is subjected to lateral loads till a target displacement of 4% of storey height is reached (displacement-controlled action). The stiffness of Push Gravity was recalled in Push Lateral. Figure 7 provides the capacity curve of the MRF. The nonlinear responses of MRF were evaluated form the performance point obtained from CSM and DCM procedures. Table 7 represents the PoA result of example MRF. Table 8 shows various responses obtained at the performance point. The formation of plastic hinges illustrates the flow of inelastic incursions in MRF as; as step 2 first hinge is formed in B-C performance level which may be use to find yield stiffness. The displacement value is 28.76 mm and base shear value is 1808.66 hence yield stiffness is found to be 62.89 kN/mm.

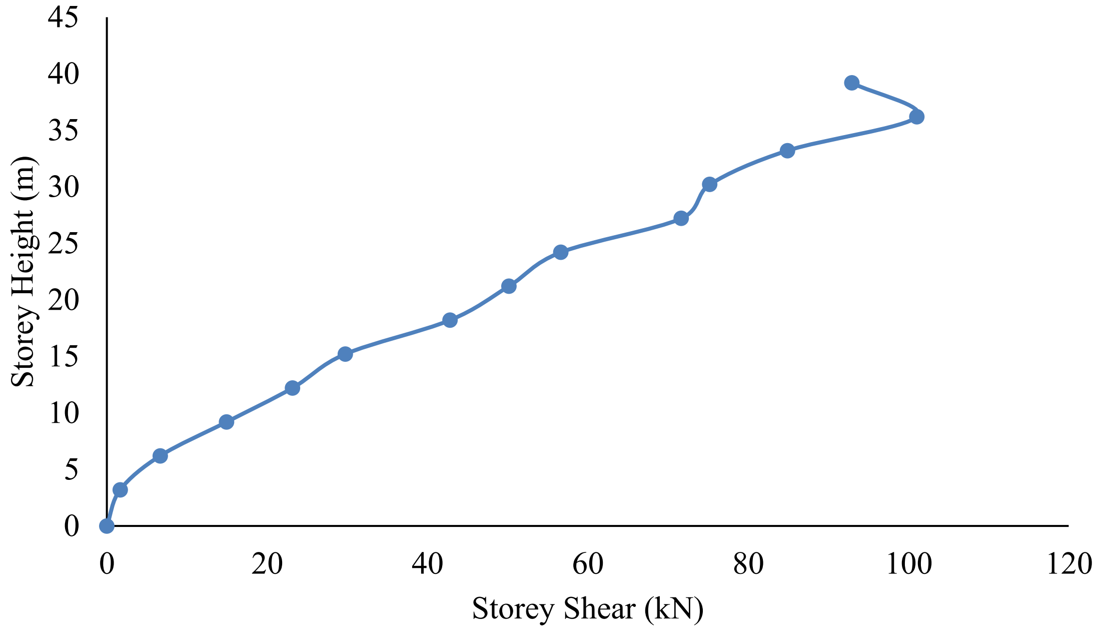
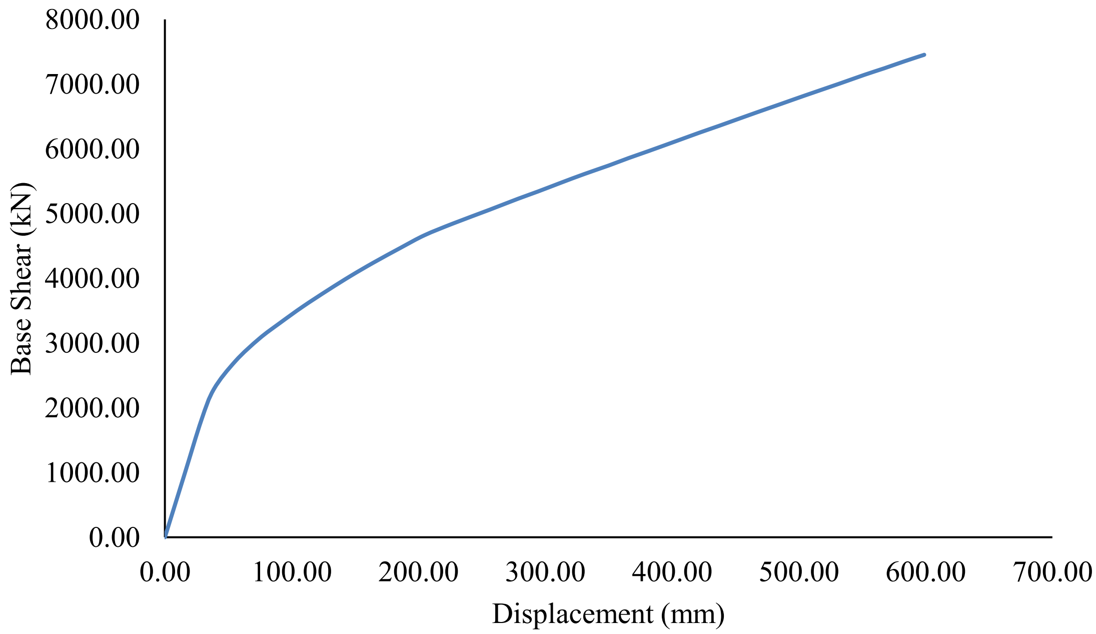


Fig. 6: Storey shear of example MRFs



At step 13 first plastic hinge appears in life safety range with values of displacement of 227.39 mm and base shear 4847.62 kN. The corresponding stiffness work out as 21.31 kN/mm. At step 14 first hinge surpasses the collapse range with displacement 256.15 and base shear of 5059.91 kN. The stiffness possessed at this stage is 19.75 kN/mm. It may conclude that there is loss in stiffness with incremental step till collapse. The maximum displacement observed is 598.93 mm which is less that 4% of total height of MRF (1568 mm).

Table 7: POA Result of example MRF

Step	Monitored Displacement (mm)	Base Force (kN)	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0.00	0.00	1040	0	0	0	0	1040	0	0	0	1040
1	15.60	980.98	1040	0	0	0	0	1040	0	0	0	1040
2	28.76	1808.66	1038	2	0	0	0	1040	0	0	0	1040
3	38.65	2288.30	944	96	0	0	0	1040	0	0	0	1040
4	55.62	2721.08	902	138	0	0	0	1040	0	0	0	1040
5	74.37	3067.22	870	170	0	0	0	1040	0	0	0	1040
6	90.78	3310.58	858	182	0	0	0	1040	0	0	0	1040
7	108.76	3562.80	846	194	0	0	0	1040	0	0	0	1040
8	128.63	3818.29	832	208	0	0	0	1040	0	0	0	1040
9	147.08	4043.62	820	220	0	0	0	1040	0	0	0	1040
10	164.66	4243.74	806	234	0	0	0	1040	0	0	0	1040
11	186.46	4477.19	802	238	0	0	0	1040	0	0	0	1040
12	204.85	4666.39	796	244	0	0	0	1040	0	0	0	1040
13	227.39	4847.62	778	262	0	0	0	1036	4	0	0	1040
14	256.15	5059.91	772	268	0	0	0	944	94	0	2	1040
15	278.45	5228.51	766	274	0	0	0	896	142	0	2	1040
16	294.05	5339.65	766	274	0	0	0	890	148	0	2	1040
17	312.41	5477.01	760	280	0	0	0	888	148	0	4	1040
18	330.88	5611.40	756	284	0	0	0	886	148	0	6	1040
19	350.27	5744.35	754	286	0	0	0	876	158	0	6	1040
20	365.87	5858.39	752	288	0	0	0	860	174	0	6	1040
21	381.47	5965.43	752	288	0	0	0	850	184	0	6	1040
22	405.90	6138.09	748	292	0	0	0	834	200	0	6	1040
23	421.50	6249.08	748	292	0	0	0	832	202	0	6	1040
24	437.10	6355.69	748	292	0	0	0	832	202	0	6	1040
25	465.94	6557.34	740	300	0	0	0	832	200	0	8	1040
26	485.44	6690.57	738	302	0	0	0	826	206	0	8	1040
27	504.94	6825.20	736	304	0	0	0	812	218	0	10	1040
28	532.24	7008.78	732	308	0	0	0	808	222	0	10	1040
29	551.74	7143.57	726	314	0	0	0	806	222	0	12	1040
30	567.34	7244.00	724	316	0	0	0	804	224	0	12	1040
31	582.94	7350.02	720	320	0	0	0	796	232	0	12	1040
32	598.54	7450.34	718	318	4	0	0	792	236	0	12	1040

33	598.73	7451.75	718	318	4	0	0	792	236	0	12	1040
34	598.73	7451.84	718	318	4	0	0	792	236	0	12	1040
35	598.93	7453.30	718	318	4	0	0	792	236	0	12	1040

Table 8: Nonlinear responses at performance from various PBSB methods

PBSD Method	Responses at Performance point	
	Base Shear (kN)	Displacement (mm)
FEMA 440 EL	3889.58	134.47
NTC 2008 TDM	2551.44	48.97
EC 8 -2008	2755.39	57.48
ASCE 41	3576.24	109.81

The performance levels described in FEMA 445 is discrete manner Viz.; Operational Level (OP), Immediate Occupancy Level (IO), Life safety Range (LS), Collapse Prevention Level (CP) and Collapse (C). Each performance levels define individual level of damages to structural and non-structural components. To understand a complete building performance levels for quick assessment of threat against seismic hazard, there is a need to integrate them line PL1 – minor or repairable damages to the structure that includes OP, IO and LS. PL2- downtime or major damages to the structure that includes CP, C and D (complete damage). The present study put forth the concept for future scope of the work.

II. Conclusions:

Seismic events are more destructive in nature they not only damage the structure but causes casualties and fatalities. Design a tall structure under seismic prone area has become a big challenge to structural designers. Present seismic codes intend to design building with minor or moderate damages. The society is demand of complete assessment of their life threat during seismic events. PBSD framework has provided various evaluation and assessment procedures to assess these loses and communicate them with stakeholder in terms of repairs, downtime and casualties. This domain demands more research and practice to make it common in practice. At present PoA has become a soft tool for such application. The definition of performance levels and their relevant assessment still a gray area for damage communication, the results of present study is an attempt to provide rational assessment of performance of a tall structure.

Abbreviations

PBSD Performance Based Seismic Design
FEMA Federal Emergency Management Agency
PoA Pushover Analysis
CSM Capacity Spectrum Method
DCM Displacement Coefficient Method

Acknowledgement

The authors kindly acknowledge the research contribution of all citation under reference and support from the Chairman and Director, MGM's College of engineering, Nanded (m.s).

Conflict of interest

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

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