Analytical Investigation of Moment Resisting Frame Structure—A Case Study on Performance-Based Capacity Spectrum Method

Ajay Singh Thakur and Tanmay Gupta

Abstract This paper has only two primary concerns (a) to properly adapt the performance-based methods with the Indian Seismic code; (b) to analyse the structure for its ductile behaviours. This case study addresses these concern by analysing the moment resisting frame having a six-storey with three bays model for elastic and nonlinear static procedures. The results support the code-based design but only for service-level earthquake and design-based earthquake, but for maximum considered, an earthquake, the performance-based design seems to be a perfect fit rather than a code-based approach. Results seem to be fit for prescribed target displacement, but to satisfy the global needs, the model needs strengthening which can be sorted with the hinge results of beams and columns. Ductile behaviour can be noticed through the pushover curve after the first yield. Change in size in frame members after certain floor needs to be designed carefully as a result the hinges can be seen extending beyond collapse prevention for the displacement of 159 mm in Step 27 in this paper. Pushover, hinge result, performance point and coefficient displacement results are presented and discussed.

Keywords Moment resisting frame · Performance level · Design-based earthquake · Maximum considered earthquake

1 Introduction

India is classified into IV zones under the category of the high seismic zone, moderate seismic zone, and low seismic zone. The zone factor for each zone corresponds to the maximum considered earthquake of that zone. The value of the zone factor is damped to get the design-based earthquake by dividing the value of zone factor by

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denominator 2 given in Indian Seismic code IS1893:2016 [1]. The seismic ground motion is characterised by intensity, duration which is a function of magnitude, the distance between focus and epicentre, and characteristics of the propagation of waves; the waves show impulsive and complex behaviour due to which the structure responds by introducing the inertia forces as an opposing force to horizontal seismic force which is a type of surface wave. The fundamental natural period also plays an important role in calculating the average response acceleration coefficients for different soil conditions. The short period of vibration generates large inertia forces within a structure which may cause large stiffness degradation in members so to damp the seismic forces it is feasible to elongate the period which is done by using advanced techniques like base isolation, but these techniques are not cost effective for everyone. The partial factor of safety, redundancy and ductility help in extrapolating the design base shear to ultimate elastic force; thus, in Indian code, the nonlinearity is incorporated by response reduction factor denoted by *R*. By following the ductile detailing guidelines, we ignore to analyse the ductility part as the response beyond elastic is not a simple extrapolation. For analysing the response of structure beyond the elastic stage, the nonlinear method or dynamic method of time history is followed by analysing the structure on finite element software which is time consuming and needs an experienced designer to analyse the results of nonlinear time-history analyses [2]. Pushover analysis is used as an alternative for time-history analysis which saves computational time and can analyse the structure as per various international guidelines. Performance-based design is a design approach that eases out the process of designing for the owner to engineer by giving the flexibility to the owner to discuss the performance goals with the designer and engineer which helps to set up multiple performance objectives which is a function of performance level which expresses the extent of damage in structure due to hazard event [3, 4]. It is necessary to introduce PBD to meet the performance requirement in emerging society [5, 6]. In this case study, we will analyse the structure by cooperating the performance design approach with user-defined functions to reflect the values based on Indian Seismic codes.

2 State of Development

The performance-based design approach had been in development since the end of the '60s, and due to the unexpected economic loss in Loma Prieta Earthquake (1989) and followed by Northridge Earthquake (1994), the recommendation for the performance based was developed by the SEAOC committee under Vision 2000 [7]. The capacity spectrum method was one of the earliest methods to be applied in PBD for assessing the seismic vulnerability of structures. It is a graphical method that plots the relation between base shear and displacement. This method was used in the early '70s as a pilot project at the Puget Sound Naval Shipyard [8] supporting its correlation between ground motion and building performance by stating the hinge states from IO to CP and beyond due to which it serve as a verification design document in Tri-services [9]. The pushover curve which is a plot of base shear and displacement gives the capacity

of the structure. With some modification over time, the pushover curve was converted to an acceleration displacement response spectrum which merges the base shear and displacement with (a) spectral acceleration versus spectral displacement representation, (b) spectral acceleration versus time and (c) spectral displacement versus time. N2 method and coefficient displacement method are the two advancements after the capacity spectrum method under performance-based design methodology [10]. The method for converting the base shear and displacement into spectral acceleration was covered using the relation given in ATC40 [11]. Spectral acceleration and displacement can be calculated through Eqs. 1 and 2.

$$S_a = \frac{\frac{V_B}{W}}{\left(\frac{M_K}{M}\right)} \tag{1}$$

$$S_a = \frac{\Delta \text{rooftop}}{P_K \Phi_{K\text{rooftop}}} \tag{2}$$

V_B represents the base shear, W is the seismic weight of the structure, Δ is the displacement of the rooftop, P_K is the modal participation factor and $\Phi_{k\text{ rooftop}}$ is the modal amplitude at the rooftop.

S_a versus S_d will give the capacity curve, and for the demand curve, the response spectrum graph having S_a and T is also converted by relation given in Eq. 3 and [12].

$$S_d = \frac{T^2}{4\pi^2} S_a \tag{3}$$

T was represented by radial lines in the transformed plot. The response spectrum curve gives the demand curve. The intersection of the capacity curve and demand curve gives the performance point. The damping of spectra can vary from 5 to 20% or more. The pushover analysis was not introduced in Indian Seismic Code, but ATC 40 procedure can be adapted by using the value of C_a and C_v parameters concerning response spectra used in the Indian Seismic code for a different type of soil [13]. A case study model S6B3 was used from an original paper [14]. The CSM graphs will be discussed later in the paper.

3 Case Study

The model S6B3 consist of six storey and three bay with a spacing of 4 m in each direction. The building is an office building with an importance factor of 1.5 on medium soil having a floor height of 3 m with a mean live load of 4kN/m². Lateral design load will be calculated from the equivalent static method [1, 11]. ATC40 guidelines will be followed and adapted in the Indian code for finding the performance point. The 4% of height was selected as target displacement [1, 11]. M25 grade and Fe 415 grade steel were used. Beams of size 300 × 400 mm were used. The column

Table 1 Column size detail for S6B3

Floor	Exterior column	Internal column
1–3	450 × 450	530 × 530
4–6	350 × 350	450 × 450

Fig. 1 Plan of S6

12m

12m
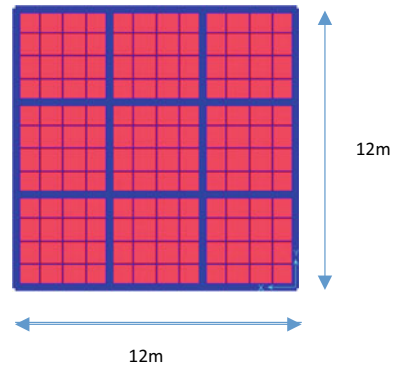
was designed separately for 1–3 floors and 4–6 floors with external and internal columns as given in Table 1. In this case study, we will use the modified value of C_a and C_v as per medium soil and Indian response spectra for 5% damping and check the structure for 0.004H permissible limit for Indian code and verify the structure hinges for collapse prevention parameter 0.04H. Property modifiers for beam and column as per Indian seismic codes were applied. It is very important after gravity load to add a new load case as gravity with nonlinear behaviours which will be force controlled, and after that push load cases can be used which will start from unstressed gravity load case. The plan of six storeys three-bay (S6B3) is shown in Fig. 1.

In Sap2000, pre-target displacement of 0.04H can be changed with the user-calculated target displacement. The push cases will be deformation controlled. Hinges can be user defined or predefined based on the designer. In this study, hinges were defined at 5 and 95% at both i and j node of the beam and column while for beams M3 hinge were assigned and for column P-M2-M3, hinges were used as a relationship to describe the post-yield backbone behavior of beam and column [15].

4 Results and Discussion

Pushover curve—Graph of base shear and displacement for the X- and Y-axis was discussed in Fig. 3. The target displacement was revised from 0.004H to 0.04H to consider more displacement effect as a result we can get the results for larger displacement and larger rotations in hinges. Figure 2 states the hinge result of the bottom node of frame object 214 hinges 2 (214H2).
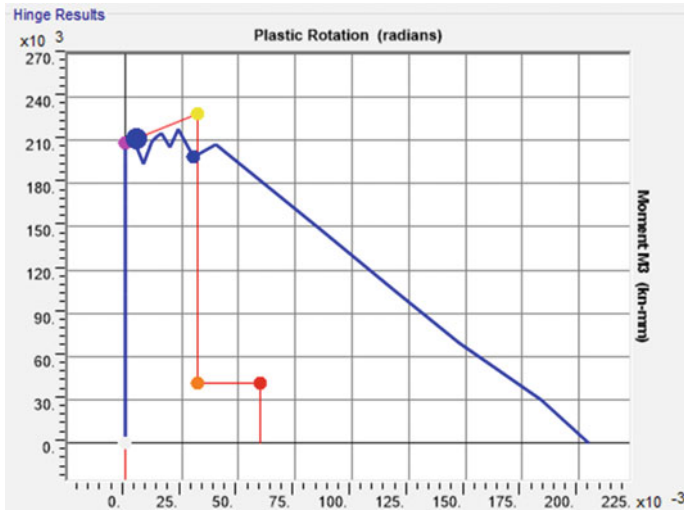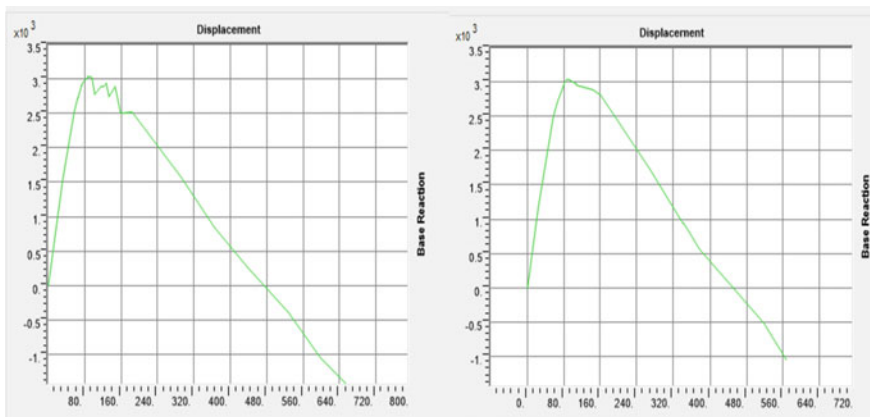
Fig. 2 214H2 P-M2-M3 hinge result

Fig. 3 Pushover curve X-axis and Y-axis

P-M2-M3 hinges were assigned for a column with M3 as hinge degree of freedom resulting in plastic rotation of 0.0304 radians at Step 27 of Push X stating the degradation of fibres after CP point. Beam hinges were found to be between IO-LS state, change in a property of column in fourth storey result in loss of stiffness in columns.

Pushover results also consider the hinge state and results for various push steps which will be discussed after covering the capacity spectrum graph. The top node of the structure (Node81) was pushed up to target displacement which results in loss of stiffness in members while exposing the ductile capacity of the structure. The S6B3 model performs satisfactorily till the permissible limit given by IS codes and follows

the strong column weak beam methodology. At Step 11, base shear was 2949 kN and displacement was 76.46 mm with only four hinges which change their state from IO to LS as given in Table 2. According to IS code, the structure performance was elastic, but for global results, performance point must be checked thoroughly.

Capacity spectrum method shows the performance point by superpositioning the pushover curve given by green colour to the ADRS representation of damped responded spectrum by 5, 10, 15, 20% indicated by four red lines known as the demand curve as shown in Fig. 4. The S_a and S_d values correspond to the 5% damped response spectra for X and Y push [16]. The performance point for X (2694.003 kN, 63.779 mm) and Y (2677.617 kN, 64.827 mm) was given in the form of base shear and displacement (V, D point). C_a value was taken as 2.5(Z/2) and C_v value 1.39/T(Z/2) for medium soil for Indian Code [13]. T is the modal fundamental period of building corresponding to mode 1. Z is the zone factor 0.36. The 1.36/T value was referred from the value of the average response spectrum for medium soil when the period was between 0.55 and 4 s.

FEMA 356 Coefficient method [17] in Fig. 5 shows the bilinearisation of the pushover curve which represents the maximum displacement and force as a performance point for the structure. For Push X and Push Y, the value of performance point was (2297.31 kN, 210.587 mm) and (2533 kN, 158 mm). These values are discussed below in hinge results as shown in Fig. 6.

In Step 27, the hinges in the column of four storeys change their state to IO-LS at one end, and the far end column reached beyond C point in which the column degrades its strength abruptly with very minute rotation and displacement. The maximum performance point values approximately correspond to Step 27 to 29 value. The hinge state for the column is shown in Fig. 1. Step 35 consists of the response due to ultimate displacement which results in the collapse of the column at four storey; hence, the moment redistribution due to storey 4 columns was already at its peak from Step 27. The base shear can be checked for displacement and its hinge state from Table 2.

5 Conclusion

For designing an earthquake-resistant important structure, performance-based design is a perfect fit, as in this case the structure was design only for elastic base shear with the use of Indian seismic code and later on verified for a global response with nonlinear static analysis which shows the structure ductility performance, with the response of hinges generated at the nodes of beam and column in the relative distance of 0.5 and 0.95. These methods with the help of hinges show the potential beams, the column for retrofitting in the existing structure and redesigning in case of a new building to achieve the proper performance. Only selected frame members to be properly modelled as in our case label 195, 214, 217, and 199 need to be retrofitted to impart additional strength and ductility. The hinge results give a good indication of

Table 2 Pushover capacity curve

Pushover capacity curve

Load case	Step	Displacement	Base force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	E	Total
	Unitless	mm	kN									
P X	0	0.025704	0	480	0	0	0	0	0	0	0	480
P X	1	31.812695	1520.154	479	1	0	0	0	0	0	0	480
P X	2	56.653549	2511.808	396	84	0	0	0	0	0	0	480
P X	3	64.814425	2720.484	377	103	0	0	0	0	0	0	480
P X	4	64.814429	2718.852	377	103	0	0	0	0	0	0	480
P X	5	64.814436	2718.85	377	103	0	0	0	0	0	0	480
P X	6	72.03646	2881.369	351	129	0	0	0	0	0	0	480
P X	7	72.038725	2881.462	351	129	0	0	0	0	0	0	480
P X	8	73.261773	2903.57	344	136	0	0	0	0	0	0	480
P X	9	73.261809	2903.571	344	136	0	0	0	0	0	0	480
P X	10	73.269009	2896.84	344	136	0	0	0	0	0	0	480
P X	11	76.36627	2949.383	338	138	4	0	0	0	0	0	480
P X	12	76.374821	2949.762	338	138	4	0	0	0	0	0	480
P X	13	76.382021	2948.005	338	138	4	0	0	0	0	0	480
P X	14	85.379513	3018.694	322	142	16	0	0	0	0	0	480
P X	15	85.386713	3018.843	322	142	16	0	0	0	0	0	480
P X	16	86.182507	3027.189	322	142	16	0	0	0	0	0	480
P X	17	87.774095	3037.745	318	146	16	0	0	0	0	0	480
P X	18	89.365683	3041.413	316	148	16	0	0	0	0	0	480

(continued)

Table 2 (continued)

Pushover capacity curve

| Load case	Step	Displacement	Base force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	E	Total
	Unitless	mm	kN									
P X	19	92.548858	3031.362	316	148	16	0	0	0	0	0	480
P X	20	95.732034	3011.489	316	146	18	0	0	0	0	0	480
P X	21	102.098375	2782.71	316	116	48	0	0	0	0	0	480
P X	22	114.831405	2894.847	316	116	48	0	0	0	0	0	480
P X	23	121.197756	2895.837	316	116	48	0	0	0	0	0	480
P X	24	127.564108	2934.963	316	116	48	0	0	0	0	0	480
P X	25	133.924557	2748.697	316	116	48	0	0	0	0	0	480
P X	26	146.657271	2881.442	316	116	48	0	0	0	0	0	480
P X	27	159.398875	2507.931	316	116	42	0	0	6	0	0	480
P X	28	184.867597	2523.08	316	116	16	0	0	32	0	0	480
P X	29	292.82261	1575.692	316	116	16	0	0	32	0	0	480
P X	30	364.823046	849.084	316	116	16	0	0	32	0	0	480
P X	31	436.839233	270.955	316	116	16	0	0	32	0	0	480
P X	32	526.681203	−398.161	316	116	16	0	0	24	0	8	480
P X	33	598.681604	−1044.15	316	116	16	0	0	0	1	31	480
P X	34	652.545648	−1418.557	316	116	16	0	0	0	0	32	480
P X	35	657.053206	−1471.126	316	116	16	0	0	0	0	32	480
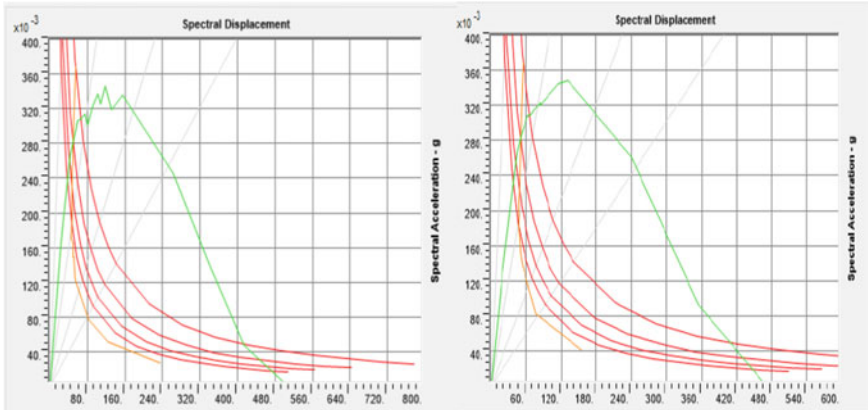
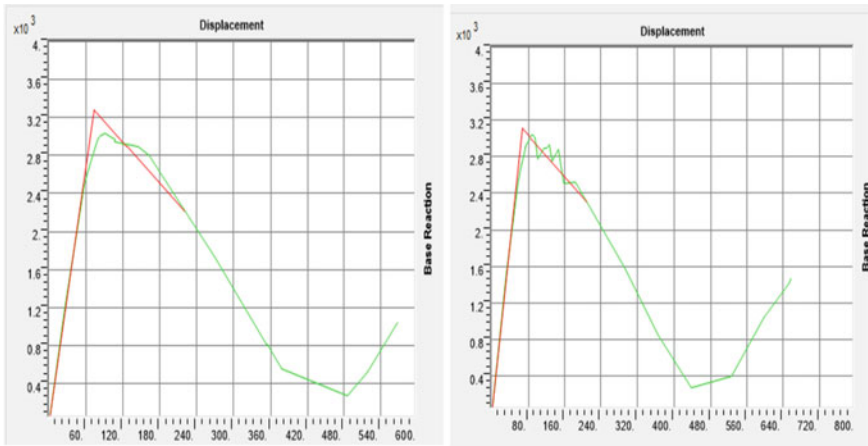**Fig. 4** ATC40 capacity spectrum for Push X and Push Y

Fig. 5 FEMA356 coefficient method for Push X and Push Y

moment and rotation with each increment of push. As the hinge behaviour is deformation controlled and up to Step 26 shows satisfactory ductile behaviour while with the adaptation of C_a and C_v values, the performance point shifts its self-according to the Indian response spectra. The structure was not analysed for the nonlinear time-history method, but in the case, it should be assigned with more than seven ground motions to get proper supporting results.
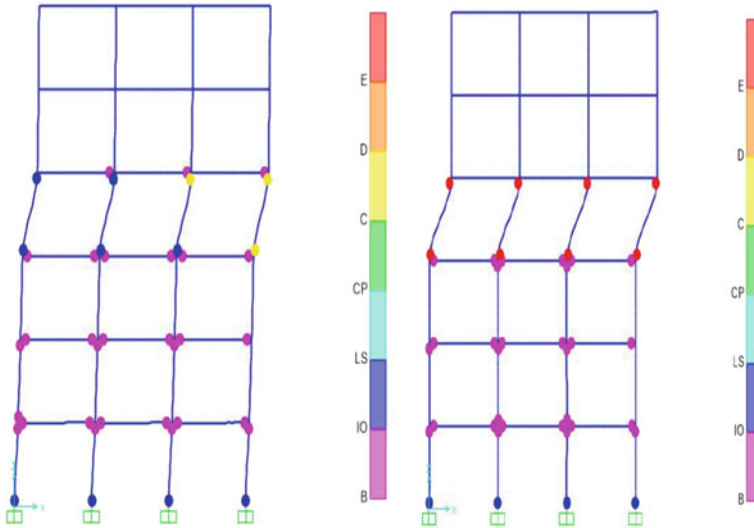
Fig. 6 Push X hinge formation at Step 27 and 35

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