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Seismic Evaluation of Multi-Storey RC Frame Using Modal Pushover Analysis

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Abstract. The recently developed pushover analysis procedure has led a new dimension to performance-based design in structural engineering practices. With the increase in the magnitude of monotonic loading, weak links and failure modes in the multi-storey RC frames are usually formed. The force distribution and storey displacements are evaluated using static pushover analysis based on the assumption that the response is controlled by fundamental mode and no mode shift takes place. Himalayan-Nagalushai region, Indo-Gangetic plain, Western India, Kutch and Kathiawar regions are geologically unstable parts of India and some devastating earthquakes of remarkable intensity have occurred here. In view of the intensive construction activity in India, where even a medium intensity tremor can cause a calamity, the authors feel that a completely up-to-date, versatile method of aseismic analysis and design of structures are essential. A detailed dynamic analysis of a 10-storey RC frame building is therefore performed using response spectrum method based on Indian Standard Codal Provisions and base shear, storey shear and storey drifts are computed. A modal pushover analysis (MPA) is also carried out to determine the structural response of the same model for the same acceleration spectra used in the earlier case. The major focus of study is to bring out the superiority of pushover analysis method over the conventional dynamic analysis method recommended by the code. The results obtained from the numerical studies show that the response spectrum method underestimates the response of the model in comparison with modal pushover analysis. It is also seen that modal participation of higher modes contributes to better results of the response distribution along the height of the building. Also pushover curves are plotted to illustrate the displacement as a function of base shear.

Key words: dynamic analysis, higher mode contribution, modal pushover analysis, pushover curves, response spectrum analysis (RSA)

1. Introduction

Design of earthquake-resistant structures is possible using the current state-of-the-art methods in structural engineering but it is not possible to predict the magnitude (intensity) and direction that would be expected on a structure due to earthquake-induced motion. No algorithm has been developed on this front due to the high degree of complexities involved in the process. Earthquake ground motions create inertia or imaginative lateral forces to act at different storey levels along the height of the buildings in a particular direction. Vertical forces are also created in a similar manner but are of less interest. Himalayan-Nagalushai region, Indo-Gangetic plain, Western India, Kutch and Kathiawar regions are geologically unstable parts of India. Some devastating earthquakes of remarkable intensity have occurred in this region, which has stimulated the importance of seismic strength assessment of buildings proposed to be constructed in India. Therefore it is essential to understand the seismic analysis of buildings by a much simpler procedure that could lead to accurate and reliable results. Although seismic effect on structures are quite complex, the response of many type of structures can be predicted with a greater degree of accuracy by subjecting them to a single set of static forces applied at all floor levels.

2. Background

Vamvatsikos and Cornell [1] discussed the Incremental Dynamic Analysis (IDA) as a parametric analysis method which has recently emerged in different forms for evaluating structural response under seismic loads. This method essentially involves subjecting a structural model to one or more typical ground record, which is scaled to multiple levels of intensity. They analyzed the properties of IDA curves for single degree of freedom (SDOF) and multi-degree of freedom (MDOF) models. They concluded that IDA is a valuable tool of seismic engineering because it addresses both ductility demand and capacity of structures. They proposed some definitions and examples on IDA properties and collapse definitions. Wilson and Habibullah [2] discussed static and dynamic analysis of multi-storey buildings including P-Delta effect. The lateral movement of a strong mass to a deformed position generates second-order overturning moments. This second-order behavior is addressed as P-Delta effect. By implementing the P-Delta effect in the basic analytical formulation, the mode shapes and frequency obtained for dynamic analysis indicate the structural softening and the member forces therefore satisfy both static and dynamic equilibrium. They showed that the additional P-Delta moments are consistent with the calculated displacements directly. The dynamic analysis of a multi-storey RC frame is essentially to know the measure of contribution of each mode to the response quantity. The modal analysis procedure gives the force distribution in the form of modal components based on the vibration properties of the structural system; namely, natural frequencies (ω) and mode-shape (Φ). However it becomes very interesting to know the relative contributions of various modes to the final storey shear computation. It is equally important to know the number of modes that should be included in such dynamic analysis with reference to any specific problem, i.e., the modal contribution factor which depends on the spatial distribution of forces to be investigated. The number of modes to be included in dynamic analysis also essentially depends on minimizing the error in static response below a pre-selected value. Therefore the number of modes required also depends on the response quantity of interest. For example, to compute the storey shear, one may consider first three modes and for computing storey drift, one may consider first two modes. Chopra [3] presented the nonlinear response analysis through rigorous dynamic analysis with rules for combination of different modes. The SRSS rule, as commented by the author is appreciably accurate for estimating peak storey shear of buildings with widely spaced natural frequencies. Luco et al. [4] presented the evaluation of predictors of nonlinear seismic demands using "fishbone" models of SMRF buildings. They evaluated precision of predictors that make use of (i) elastic modal vibration properties of the structure, (ii) the result of nonlinear static pushover analysis, and (iii) elastic and inelastic SDOF time history analysis for a specified ground motion record. The relatively small number of degrees of freedom for each "fishbone" model enabled to consider several short-to-long period buildings and modes of near and far field earthquake ground motion. They concluded that the predictor takes care of the effects on inelasticity in addition to the elastic contributions of both first and second modes. They highlighted the accuracy in the precision of predictors through "fishbone" models of the buildings which is nearly same as those obtained using full frame models, however with less CPU time. Chandrasekaran et al. [5] presented the plan irregularity effects on seismic vulnerability of Moment Resisting Frame Structures (MRFS). In the study conducted, the authors highlighted the effect of re-entrant corner of MRFS on its seismic vulnerability for A/L ranging from 0.15 to 0.2. The study was conducted on different shapes of buildings and base shear computations were presented for different soil conditions. The shear force per column in an asymmetric building was found to be more along the considered direction of earthquake. It was also stated that the base shear obtained using NEHRP provisions showed lesser values in comparison to IS-1893 [6] under similar site conditions.

Stratta [7] discussed the seismic design procedure of buildings with respect to its degree of simplicity. He mentioned the accidental torsion may be caused even in symmetrically braced buildings. In case of asymmetrical buildings, the relative rigidity of each load carrying element and its position from the centre of gravity plays a vital role for their torsion assessment. He pointed out that the ductility in concrete members subjected to seismic loading is mandatory. On the basis of the report ATC–11, he suggested that the decision to use ductile concrete shear wall should be based on aesthetics, flexibility of floor space and economics of construction. Shome and Cornell [8] presented seismic demand analysis with the consideration of collapse. The authors pointed out that nonlinear time history analysis is one of the most accurate procedures for seismic demand calculations. The authors presented a new methodology for computing the seismic demand by considering a three-parameter probability distribution model. The additional advantage of this method is that it simplifies the demand calculation procedure through seismic demand curve for multi-level performance evaluation. Ghosh and Fanella [9] presented a step by step design using structural dynamics. They discussed in detail the static and dynamic analyses procedures with a special focus towards Equivalent Lateral Force (ELF) Method. They pointed out that three modes are usually sufficient for many type of structures including structures of moderate height. The authors concluded that modal analysis procedure which is one of the methods of dynamic analysis is fairly accurate and the method is mandated.

Pushover analysis is a static nonlinear procedure in which the magnitude of structural loading is incrementally increased in accordance with a certain predefined pattern. The increase in the magnitude of loading causes weak links and develops failure modes. This method of analysis is basically an attempt to evaluate real strength of the structure and it is effective for performance-based design. Nonlinear pushover analysis provides an insight into the structural aspects which controls performance during severe earthquakes. This analysis provides data on the strength and ductility of the structure which otherwise cannot be predicted. Base shear *versus* tip displacement curve of the structure, called pushover curves, are essential outcomes of pushover analysis. These curves are useful in ascertaining whether a structure is capable of sustaining certain level of seismic load. However this comparison can be made on the basis of force or displacement. In pushover analysis, both the force distribution and target displacement are based on a time independent phenomena and therefore this analysis may not be very significant for structures where higher modes are significant. Habibullah and Pyle [10] have presented three dimensional nonlinear static pushover analysis explaining the recent advancements against static pushover analysis. They discussed in detail the steps for nonlinear static pushover analysis as prescribed by ATC-40 and FEMA-273. They recommended the procedure for strengthening the existing building and also suggested changes in the detailing so as to modify the hinge formation criteria. Qian and Zhou [11] presented full range pushover analysis of an RC frame. They proposed composite structure method for pushover analysis of RC frame to obtain its full range capacity curve with descending branch. They have shown that with the descending branch of the capacity curve, the ultimate deformation capacity as well as the ductility ratio could be obtained. Ballard et al. [12] discussed the use of pushover analysis in the seismic performance of steel bridges. Normal engineering analysis assumes linear elastic behavior for structural members, which fail to reliably account for redistribution of forces due to member nonlinear behavior and dissipation of energy due to material yielding. They discussed nonlinear pushover analysis as a key component for the nonlinear dynamic analysis and overall retrofit design of the bridge. Jain et al. [13] discussed in detail the damages of reinforced concrete structures during Bhuj earthquake with preliminary results of pushover analysis of two damaged buildings. They demonstrated the serious inadequacies in the current building design practice. The failure pattern indicated that the ductile detailing is essential to enable storey seismic effects. They also commented that irregular structural configurations results in poor seismic performance. Chopra and Goel [14] presented Modal Pushover

Analysis (MPA) to estimate seismic demand for buildings. They presented a detailed procedure based on the structural dynamics theory, which retains the conceptual simplicity and computational attractiveness with invariant force distribution. The equations of motion for a symmetric-plan multi-storey building subjected to earthquake ground acceleration $\ddot{u}_g(t)$ are same as those for external forces, known as the effective earthquake forces (Chopra [3]).

$$\mathrm{P}_{\mathrm{eff}}(t) = -\mathbf{m}\mathbf{1}\ddot{u}_g(t) \tag{1}$$

where $\mathbf{m}$ is the mass matrix and $\mathbf{1}$ is a vector with all elements equal to unity. Defined by $\mathbf{s} \equiv \mathbf{m1}$, the spatial (heightwise) distribution of forces can be expanded into its modal components $\mathbf{s}_n$

$$\mathbf{s} = \sum \mathbf{s}_n, \quad \mathbf{s}_n \equiv \Gamma_n \mathbf{m}\boldsymbol{\Phi}_n \tag{2}$$

where $\boldsymbol{\Phi}_n$ is the nth-mode and

$$\boldsymbol{\Gamma}_n = \frac{\boldsymbol{\Phi}_n^{\mathrm{T}}\mathbf{m1}}{\boldsymbol{\Phi}_n^{\mathrm{T}}\mathbf{m}\boldsymbol{\Phi}_n} \tag{3}$$

In the MPA procedure, the peak response of the building to $\mathrm{P}_{\mathbf{eff}}(t) = -\mathbf{s}_n\ddot{u}_g(t)$, the nth-mode component of effective forces, is determined by a nonlinear static pushover analysis. The peak demands due to these modal components of forces are then combined by an appropriate modal combination rule. They demonstrated MPA procedure to provide much superior results compared to force distributions of FEMA-273 [15] and FEMA-356 guidelines.

3. Objectives and Scope

In this study one of the principal aims is to bring out the contribution of modal participation factor in the storey shear computation for different class of buildings *vis-à-vis* symmetrical, square and rectangle. On the basis of the literature reviewed, the primary objective of the study is set to bring out the superiority of pushover analysis over the dynamic analysis method recommended by the code. The following objectives are also focused.

(i) To evaluate the contribution of higher modes in storey shear.
(ii) To plot pushover curves to illustrate storey drift as function of base shear.

4. Numerical Studies and Discussion

Ten-storey RC-framed buildings considered for the analysis are shown in Figures 1–3, respectively. The storey height is 4 m. The response spectra used in the study as obtained from IS 1893 (part 1): 2002 is given in Figure 4. The study is conducted by taking the soil base as soft soil. The structural properties of the elements are given in Table 1. Table 2 shows the time-period and mode-shape coefficients and Table 3 gives the modal participation factor of all the modes for the 10-storey RC-framed building of type A.

Tables 4 and 5 show mode-shape coefficients and modal participation factors for type B buildings. Tables 6 and 7 show the mode-shape coefficients and modal participation factors for type C buildings.

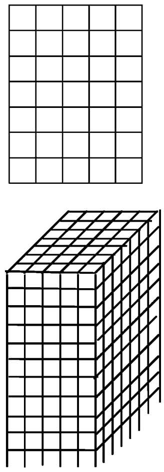

Figure 1. *Type A* – 10-storey RC building (15 m × 21 m).

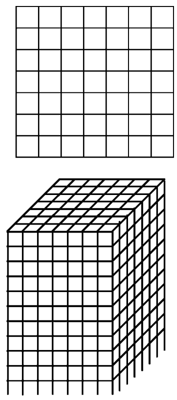

Figure 2. *Type B* – 10-storey RC square building (21 m × 21 m).

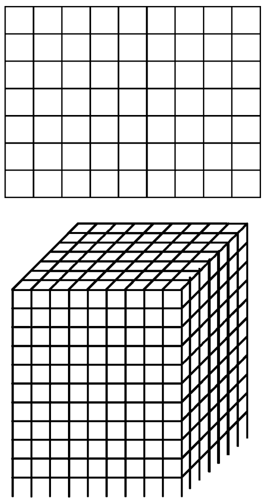

Figure 3. *Type C* – 10-storey RC building (27 m × 21 m).

Table 1. Structural properties of the building elements.

Column size	400 mm × 500 mm
Beam size	400 mm × 450 mm
Slab thickness	100 mm
Concrete mix	M_{25}
Type of steel	HYSD bars

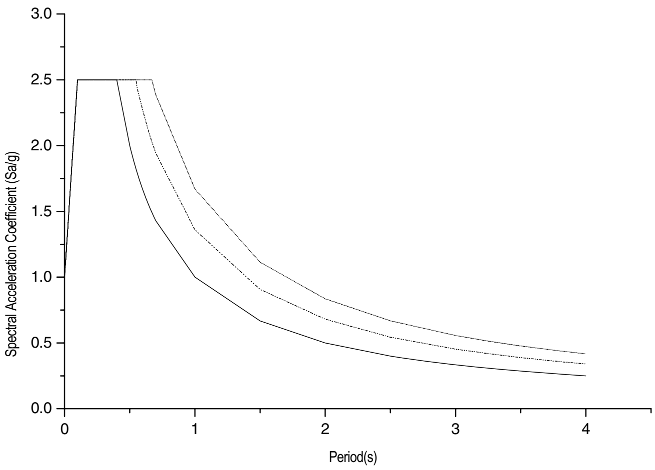

Figure 4. Response spectra for rock and soft soil sites for 5% damping.

While performing the dynamic analysis, the peak storey shear evaluated using SRSS rule by considering the cases such as: (i) only the first mode, (ii) modes with modal mass >90% of Total Seismic Mass (computed to be first and second modes), and (iii) modes with frequency up to 33 Hz (computed to be first, second and third modes). Then storey drift was calculated using the formula drift $= V_i/K_i$. In drift calculation, V_i is taken as that on account of contribution of all the modes.

Table 8 shows the peak storey shear and storey drift computed as above for type A buildings.

Figure 5 shows variation of mode-shape coefficients for the first three modes of type A buildings. Figure 6 shows the variation of lateral force with height for the first three modes.

Table 9 shows the peak storey shear and storey drift computed as above for type B buildings. Table 10 shows the peak storey shear and storey drift computed as above for type C buildings.

By comparing the peak storey shear given in Table 8 for type A buildings, it is seen that there is a marginal variation in the storey shear computed accounting for all the modes and using modes up to frequency of 33 Hz (first three modes). It is 1.2% more by considering the first two modes and 1.3% more by considering the first three modes in comparison to that obtained using first mode only. It is also seen that the peak storey shear varies by about 11.5% when computed using first three modes in comparison to that obtained using first mode only. A similar behavior is also noticed in the peak storey shear obtained for type B buildings as seen in Table 9 and type C buildings as seen in Table 10. Therefore it can be said certainly that for accurate evaluation for peak storey shears using SRSS rule (which is considered to be one of the most accurate rules), it is necessary to consider participation from higher modes and also to get the correct peak storey shear distribution along the height of such RC buildings.

Table 2. Time-period (T (s)) and mode-shape coefficients (Φ) for type a buildings.

	$T_1 = 0.939$	$T_2 = 0.316$	$T_3 = 0.192$	$T_4 = 0.141$	$T_5 = 0.113$	$T_6 = 0.097$	$T_7 = 0.086$	$T_8 = 0.079$	$T_9 = 0.075$	$T_{10} = 0.072$
Φ_1	0.066	0.191	0.298	−0.378	0.423	0.429	−0.397	0.331	−0.236	−0.123
Φ_2	0.129	0.343	0.432	−0.366	0.169	−0.085	0.308	−0.424	0.395	0.235
Φ_3	0.191	0.420	0.326	0.024	−0.355	−0.412	0.158	0.212	−0.423	−0.328
Φ_4	0.248	0.418	0.040	0.389	−0.312	0.167	−0.431	0.152	0.313	0.394
Φ_5	0.299	0.326	−0.268	0.352	0.229	0.379	0.176	−0.407	−0.099	−0.426
Φ_6	0.343	0.167	−0.428	−0.049	0.404	−0.242	0.294	0.369	−0.146	0.423
Φ_7	0.379	−0.026	−0.351	−0.399	−0.067	−0.332	−0.404	−0.066	0.344	−0.385
Φ_8	0.406	−0.214	−0.080	−0.338	−0.431	0.307	0.019	−0.284	−0.429	0.315
Φ_9	0.424	−0.358	0.235	0.073	−0.106	0.271	0.389	0.430	0.373	−0.218
Φ_{10}	0.432	−0.428	0.420	0.407	0.389	−0.361	−0.322	−0.267	−0.194	0.103

Table 3. Modal participation factor for type A buildings.

Mode (k)	Participation factor $\frac{\sum_{i=1}^{n} W_i \phi_{ik}}{\sum_{i=1}^{n} W_i \phi_{ik}^2}$
1	2.82785
2	0.928087
3	0.539273
4	−0.36610
5	0.264568
6	0.195307
7	−0.143103
8	0.100759
9	−0.064338
10	−0.031431

Note. $W_i = 249.1$ t.

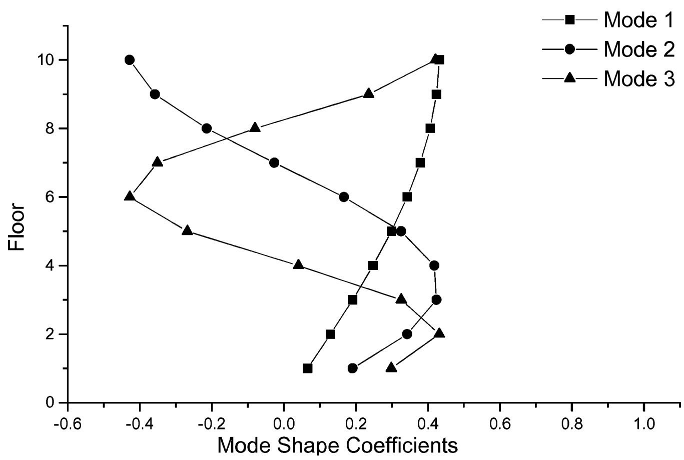

Figure 5. Variation of mode-shape coefficients.

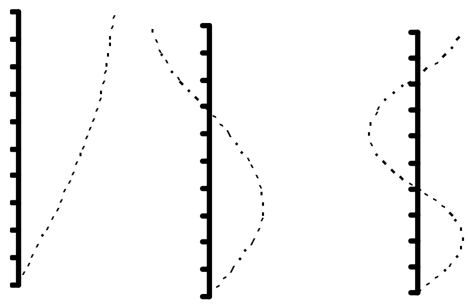

Figure 6. Lateral force distribution for first three modes in building of type A.

The positions of possible plastic hinges form one of the main criteria in the modal pushover analysis. FEMA-273 guidelines are followed in assuming the possible plastic hinges at different locations and the failure pattern are evaluated by trial and error process for all the three types of buildings taken for the study. The pushover curves showing the base shear *versus* displacement are plotted for the type A

Table 4. Time-period (T (s)) and mode-shape coefficients (Φ) for type B buildings.

	$T_1 = 0.951$	$T_2 = 0.319$	$T_3 = 0.195$	$T_4 = 0.143$	$T_5 = 0.115$	$T_6 = 0.098$	$T_7 = 0.087$	$T_8 = 0.079$	$T_9 = 0.076$	$T_{10} = 0.073$
Φ_1	0.066	0.191	0.298	−0.378	0.423	0.429	−0.397	0.331	−0.236	−0.123
Φ_2	0.129	0.343	0.432	−0.366	0.170	−0.085	0.308	−0.424	0.395	0.236
Φ_3	0.191	0.424	0.326	0.024	−0.354	−0.413	0.159	0.212	−0.424	−0.329
Φ_4	0.248	0.418	0.041	0.389	−0.313	0.166	−0.431	0.153	0.313	0.394
Φ_5	0.299	0.326	−0.268	0.353	0.229	0.379	0.175	−0.407	−0.099	−0.427
Φ_6	0.343	0.167	−0.428	−0.047	0.405	−0.241	0.295	0.369	−0.147	0.424
Φ_7	0.379	−0.026	−0.352	−0.399	−0.066	−0.332	−0.404	−0.064	0.345	−0.385
Φ_8	0.406	−0.214	−0.081	−0.338	−0.431	0.306	0.018	−0.285	−0.429	0.314
Φ_9	0.424	−0.358	0.235	0.071	−0.107	0.272	0.390	0.430	0.372	−0.217
Φ_{10}	0.432	−0.428	0.420	0.407	0.388	−0.360	−0.320	−0.266	−0.193	0.102

Table 5. Modal participation factor for type B buildings.

Mode (k)	Participation factor $\frac{\sum_{i=1}^{n} W_i \phi_{ik}}{\sum_{i=1}^{n} W_i \phi_{ik}^2}$
1	2.93926
2	0.96402
3	0.55938
4	−0.37893
5	0.27298
6	0.20066
7	−0.14623
8	0.10229
9	−0.06489
10	−0.03147

Note. $W_i = 340.14\,\text{t}$.

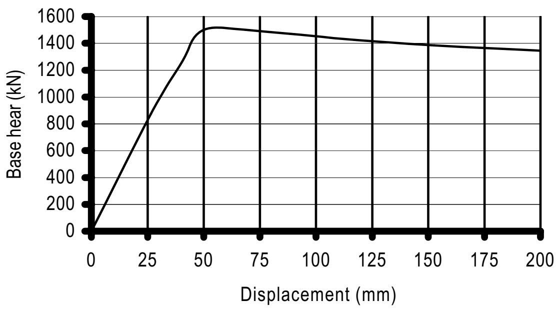

Figure 7. Modal pushover curve for type A buildings (mode 1).

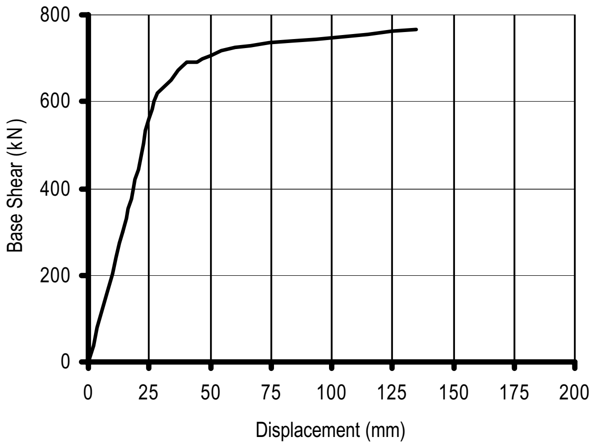

Figure 8. Modal pushover curve for type A buildings (mode 2).

buildings only. The plastic hinge rotation capacity is considered according to FEMA-273. The modal combinations are taken so as to represent the nonlinear response expected in any mode other than flexure.

Figures 7–9 show the pushover curves obtained for performing modal pushover analysis for type A buildings by taking the options as follows.

Table 6. Time-period (T (s)) and mode-shape coefficients (Φ) for type C buildings.

	$T_1 = 0.996$	$T_2 = 0.335$	$T_3 = 0.204$	$T_4 = 0.149$	$T_5 = 0.120$	$T_6 = 0.102$	$T_7 = 0.091$	$T_8 = 0.084$	$T_9 = 0.079$	$T_{10} = 0.077$
Φ_1	0.066	0.191	0.298	−0.378	0.423	0.429	−0.398	0.331	0.237	−0.123
Φ_2	0.129	0.343	0.432	−0.366	0.170	−0.084	0.308	−0.424	−0.395	0.236
Φ_3	0.191	0.424	0.327	0.024	−0.354	−0.413	0.159	0.211	0.424	−0.329
Φ_4	0.248	0.418	0.041	0.389	−0.313	0.166	−0.431	0.153	−0.313	0.394
Φ_5	0.299	0.326	−0.267	0.353	0.228	0.380	0.175	−0.407	0.098	−0.427
Φ_6	0.343	0.167	−0.428	−0.047	0.405	−0.240	0.296	0.368	0.148	0.424
Φ_7	0.379	−0.026	−0.352	−0.399	−0.065	−0.333	−0.404	−0.064	−0.346	−0.385
Φ_8	0.406	−0.214	−0.081	−0.339	−0.431	0.306	0.017	−0.286	0.429	0.314
Φ_9	0.424	−0.358	0.234	0.071	−0.108	0.274	0.391	0.430	−0.372	−0.217
Φ_{10}	0.432	−0.428	0.420	0.407	0.387	−0.359	−0.319	−0.265	0.192	0.101

Table 7. Modal participation factor for type C buildings.

Mode (k)	Participation factor $\frac{\sum_{i=1}^{n} W_i \phi_{ik}}{\sum_{i=1}^{n} W_i \phi_{ik}^2}$
1	2.93903
2	0.96276
3	0.56005
4	−0.37795
5	0.27369
6	0.19985
7	−0.14684
8	0.10174
9	−0.06527
10	−0.03126

Note. $W_i = 431.28$ t.

Table 8. Peak storey shear and storey drift for type A buildings.

				$V_{i,\mathrm{SRSS}}$ (kN)				
Floor	$V^{(1)}$ (kN)	$V_i^{(2)}$ (kN)	$V_i^{(3)}$ (kN)	Mode 1	Modes 1 and 2	Modes 1–3	All modes	Drift (mm)
1	1274.850	193.034	65.174	1274.850	1289.381	1291.028	1291.520	2.691
2	1245.240	153.324	29.117	1245.240	1254.644	1254.982	1255.100	2.615
3	1186.700	82.072	−23.048	1186.700	1189.535	1189.758	1190.350	2.480
4	1100.600	−6.063	−62.463	1100.600	1100.617	1102.388	1102.850	2.298
5	988.927	−92.951	−67.320	988.927	993.286	995.564	995.815	2.075
6	854.284	−160.717	−34.934	854.284	869.270	869.972	870.686	1.814
7	699.798	−195.422	16.779	699.798	726.572	726.766	727.567	1.516
8	529.056	−189.924	59.209	529.056	562.113	565.223	565.582	1.178
9	346.025	−145.356	68.883	346.025	375.315	381.584	383.234	0.798
10	154.956	−70.886	40.447	154.956	170.4	175.135	178.808	0.373

Table 9. Peak storey shear and storey drift for type B buildings.

				$\mathrm{V}_{i,\mathrm{SRSS}}$ (kN)				
Floor	$V^{(1)}$ (kN)	$V_i^{(2)}$ (kN)	$V_i^{(3)}$ (kN)	Mode 1	Modes 1 and 2	Modes 1–3	All modes	Drift (mm)
1	1787.570	274.014	92.389	1787.570	1808.449	1810.808	1811.490	2.831
2	1746.070	217.677	41.304	1746.070	1759.586	1760.071	1760.230	2.751
3	1664.030	116.585	−32.619	1664.030	1668.109	1668.428	1669.280	2.609
4	1543.370	−8.47568	−88.507	1543.370	1543.390	1545.929	1546.590	2.417
5	1386.870	−131.794	−95.456	1386.870	1393.118	1396.385	1396.740	2.183
6	1198.180	−228.016	−49.624	1198.18	1219.683	1220.692	1221.700	1.909
7	981.670	−277.358	23.647	981.670	1020.099	1020.374	1021.510	1.596
8	742.371	−269.675	83.843	742.371	789.835	794.273	794.777	1.242
9	485.838	−206.547	97.679	485.838	527.921	536.881	539.215	0.843
10	218.026	−100.953	57.504	218.026	240.264	247.0498	252.248	0.394

Table 10. Peak storey shear and storey drift for type C buildings.

				$V_{i,\mathrm{SRSS}}$ (kN)				
Floor	$V^{(1)}$ (kN)	$V_i^{(2)}$ (kN)	$V_i^{(3)}$ (kN)	Mode 1	Modes 1 and 2	Modes 1–3	All modes	Drift (mm)
1	2165.510	346.656	117.467	2165.510	2193.081	2196.225	2197.130	2.747
2	2115.250	275.318	52.616	2115.250	2133.092	2133.741	2133.960	2.668
3	2015.910	147.309	−41.227	2015.910	2021.285	2021.705	2022.820	2.529
4	1869.770	−11.053	−112.173	1869.770	1869.802	1873.164	1874.050	2.343
5	1680.250	−167.207	−120.994	1680.250	1688.549	1692.879	1693.350	2.117
6	1451.730	−289.050	−62.812	1451.730	1480.226	1481.558	1482.890	1.854
7	1189.520	−351.531	30.202	1189.520	1240.376	1240.743	1242.240	1.553
8	899.717	−341.802	106.617	899.717	962.455	968.342	969.005	1.211
9	589.040	−261.865	124.181	589.040	644.625	656.477	659.546	0.825
10	264.704	−128.155	73.182	264.704	294.095	303.063	309.889	3.874

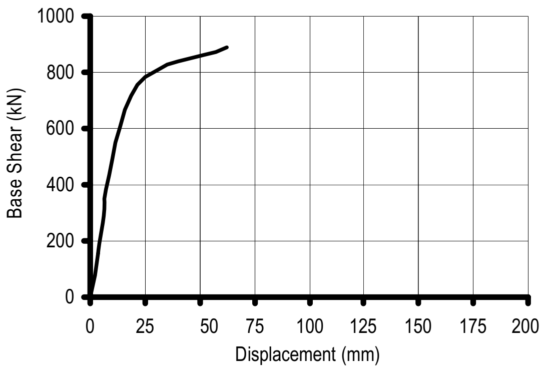

Figure 9. Modal pushover curve for type A buildings (mode 3).

(i) Only the first mode, termed as mode 1.
(ii) Only first and second mode, termed as mode 2.
(iii) Only first, second and third mode, termed as mode 3, respectively.

The modal pushover curves obtained for type A buildings using nonlinear pushover analysis followed by static analysis for gravity loading clearly figures out the nonlinearity in the behavior. It is also seen that the pushover curve obtained using three modes relatively gives closer accuracy in comparison with that obtained using first and second modes.

5. Conclusions

On the basis of the numerical studies conducted, the following conclusions can be drawn.

1. The modal participation of higher modes contributes to better results of response to 10-storey RC-framed building. It estimates peak storey shear with a greater accuracy which, otherwise would have not been included in the analysis if higher modes are neglected.
2. The pushover curves plotted display the nonlinear behavior of a 10-storey RC-frame building.
3. The response spectrum analysis (RSA) underestimates the response of the models of 10-storey RC-framed buildings in comparison with the modal pushover analysis.

4. MPA simplifies the nonlinear dynamic analysis of multi-storey RC frames in comparison with the conventional RSA. It is also capable of hierarchically indicating the formation of plastic hinges in beam and column sections in actual during the analysis. MPA is capable of performing the analysis for design later all forces accounted in different modes as per the choice of the analyst.
5. The modes whose frequency is at the cut-off of 33 Hz reasonably give accurate results of the response and therefore higher modes beyond this cut-off frequency shall not be considered for such analysis.

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