A case for use of dynamic analysis in designing for earthquake forces

Reinforced concrete (RC) frame buildings are the most common type of constructions in urban India, which are subjected to several types of forces during their lifetime, such as static forces due to dead and live loads and dynamic forces due to wind and earthquakes. Unlike static forces, amplitude, direction and location of dynamic forces, especially due to earthquakes, vary significantly with time, causing considerable inertia effects on buildings. Behaviour of buildings under dynamic forces depends upon the dynamic characteristics of buildings which are controlled by both their mass and stiffness properties, whereas the static behaviour is solely dependent upon the stiffness characteristics.

Performance of buildings largely depends on the strength and deformability of constituent members, which is further linked to the internal design forces for the members. The internal design forces in turn depend upon the accuracy of the method employed in their analytical determination. Analysing and designing buildings for static forces is a routine affair these days because of availability of affordable computers and specialized programs which can be used for the analysis. On the other hand, dynamic analysis is a time-consuming process and requires additional input related to mass of structure, and an understanding of structural dynamics for interpretation of analytical results.

Nearly a century ago, after the 28 December 1908 Messina earthquake in Italy, the reconnaissance committee appointed by the government of Italy made specific recommendations to design structures for some earthquake resistance[1]. For the first time, it was reported that the effects of earthquake forces on structures are dynamic in nature and should be dealt with as such. It was recognized that structural dynamics was not sufficiently developed by that time; therefore a simplified method was recommended, which takes care of the dynamics involved in the problem to a certain extent. The method was referred to as the Equivalent Static Method of Analysis (ESMA). The committee recommended that the first storey be designed for horizontal forces equal to 1/12 the building weight above and the second and third storeys to be designed for 1/8 of the building weight above.

With the development of earthquake engineering and computers, the Seismological Field Survey of the US Coast and Geodetic Survey, USA, installed the first strong-motion accelerograph in late 1932, which recorded the ground motions of 10 March 1933 Long Beach earthquake[1]. Recording of ground motions greatly improved the understanding of earthquake forces and an improved ESMA was introduced in the building code of Los Angeles on 1 January 1943. In the improved method, the design horizontal forces varied over the height of buildings and were also a function of the total height, which is approximately related to the fundamental period of vibration of buildings (T_1)[1]. ESMA that is being used presently in the seismic codes of most of countries worldwide, including India, is the result of several modifications in the original method proposed in 1908.

Computers made possible the speedy analysis of accelerograms (accelerograph recordings) and development of response spectrum of earthquake motions and design spectrum. Later, with digital computers, structural dynamics was developed and detailed linear dynamic analysis (DA) method was evolved, which considers the effects of all the dynamic characteristics of buildings. It was only after the 1971 San Fernando earthquake in USA that linear DA was made mandatory in Los Angeles for structures over about 50 m in height[1].

In reality, during earthquakes, buildings are generally subjected to large inertia forces, which cause members of buildings to behave in a nonlinear manner, i.e. stress does not remain proportional to strain (material nonlinearity) in addition to nonlinearity associated with large deformations. Earthquake shaking of structure is a nonlinear dynamic problem and structural analysis should be able to incorporate the nonlinear behaviour of members for evaluating the actual response of structures. Nonlinear analysis requires a lot of input data related to material and section properties and loads, which are generally difficult to obtain accurately. Experimental data are not available in sufficient quantity to develop accurate analytical models for analysis procedures to characterize nonlinear dynamic force-deformation behaviour of members. Further, the interpretation of analysis results requires a great deal of expertise and in-depth understanding of the nonlinear behaviour of structures.

Therefore, the national codes of a few countries[2] recommend nonlinear analysis only for highly irregular and important structures. In comparison, linear DA is simpler and adequately captures dynamic behaviour in elastic range and therefore is a better indicator of structural performance than ESMA. However, it fails to capture the capacity-related information of structural members, which is only possible with nonlinear dynamic or static procedures. A simple nonlinear static (Pushover) analysis is being used nowadays for certain projects, especially those related to seismic strengthening and rehabilitation[3].

The main purpose of linear DA is to evaluate the time variation of stresses and deformations in structures caused by arbitrary dynamic loads. As in any dynamical system, vibrational properties of buildings can be estimated by solving Eigen value problem given by:

$$[k - \omega_n^2 m]\phi_n = 0, \quad \text{where } \omega_n = \frac{2\pi}{T_n}, \qquad (1)$$

where k and m are the stiffness and mass matrices of buildings respectively, and ω_n, ϕ_n and T_n are the natural frequency, mode shape and natural period of buildings respectively, for the nth mode. Given k and m, the eigenvalue problem is to find positive ω_ns and corresponding ϕ_ns.

Buildings can vibrate in different mode shapes, as shown for a typical ten-storey RC building in Figure 1. There can be as many mode shapes possible as number of dynamic degrees of freedom in the building. Dynamic degrees of freedom in a structure are the number of independent coordinates in which the structure can undergo motion under dynamic forces. Depending upon the building type, only the first few mode shapes may govern the response of the building. Lateral displacement, u at any point on buildings during earthquakes can be expressed as a linear combination of all the mode shapes of buildings as given below:

$$u = \sum_{n=1}^{N} \phi_n q_n, \qquad (2)$$

where q_n are the nth modal coordinates and N is the total number of modes. Shear forces on buildings can be estimated as stiffness times the lateral displacement.

Therefore, mode shapes of buildings play an important role in estimating the design base shear for buildings.

Along each principal axis, contribution of first mode is highest among all possible mode shapes in regular buildings while contribution of higher modes reduces depending upon the natural characteristics of buildings. Regular buildings have simple geometry without any abrupt change in dimension, and uniform and symmetric distribution of mass and stiffness along height as well as in plan. Depending upon its contribution, each mode imposes some force demand on buildings, and adding the forces imposed by all the modes gives the total force on the building for which it is needed to be designed.

In some short buildings, the first vibration mode may be the only governing mode with more than 90–95% participation factor. With increasing number of floors, flexibility of buildings increases bringing higher mode effects into the picture. Therefore, for taller buildings (generally ten or more storeys), even if they are regular, the first mode may not be the only governing mode; participation from higher modes may also be significant (Figure 1). In reality, several types of irregularities are introduced into buildings because of unsymmetrical and non-uniform distribution of mass and stiffness, which are often responsible for the predominance of higher modes in the seismic force demand on buildings. Thus, the effects of higher modes are important in analysis, design and ultimately in the performance of buildings. ESMA cannot take into account such behavioural changes, which are highly dynamic in nature.

In addition, masonry infill walls are generally responsible for introduction of several types of irregularities in RC frames, e.g. torsional irregularity in plan and soft storeys in elevation. The mode shapes and the corresponding contribution of different modes depend upon the amount and location of infills in the frame because of their high initial stiffness, as shown in Figure 2, where a single frame of the ten-storey building is shown. In the case of a fully infilled frame, lateral displacements are uniformly distributed throughout the height as shown in Figure 2 *a* and *b*. On the other hand, in the case of open first-storey buildings, most of the lateral displacement is accumulated at the first-storey level itself because the first storey is the most flexible due to absence of infills (Figure 2 *c*). Similarly, the seismic storey shear forces and subsequently the bending moments concentrate in the open first storey, instead of gradually varying as in fully infilled frame (Figure 2 *c* and *b*).

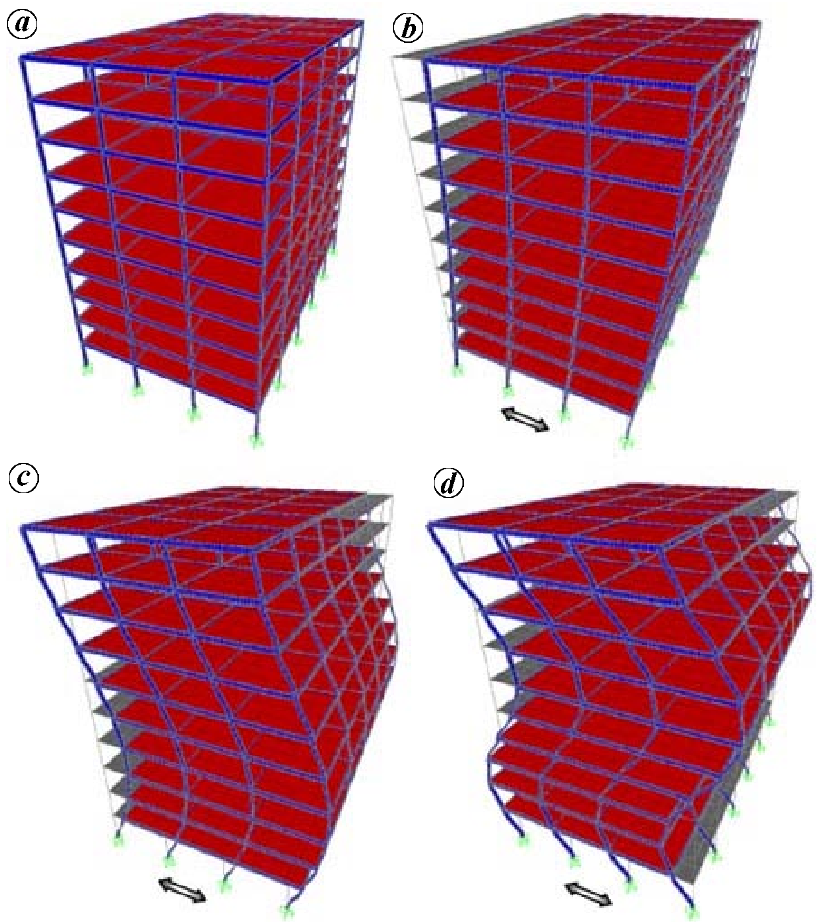

Figure 1. First few mode shapes of a typical ten-storey RC building along one direction. ***a***, Typical building; ***b***, I mode of vibration; ***c***, II mode of vibration; ***d***, III mode of vibration.

The basic assumption in ESMA is that only the first mode of vibration of buildings governs the dynamics and the effects of higher modes are not significant; therefore, higher modes are not considered in the analysis. Thus, irrespective of whether the building is regular or irregular, ESMA cannot adequately capture the true behaviour of multistorey buildings; the design forces for the members in buildings may be grossly underestimated. However, several uncertainties and approximations are involved in DA in describing the true dynamic loads, estimating the actual material and sectional properties, etc. Therefore, DA must be used with great caution.

IS:1893-2002 has divided India into four seismic zones depending upon the seismic hazard associated with different regions[4] and recommends different analysis methods depending upon height, location (zone) and configuration of buildings. ESMA is permitted for regular buildings of height up to 90 m (~ 30 storeys) in lower seismic zones (zones II and III), and of height up to 40 m (~ 13 storeys) in higher zones (zones IV and V). On the other hand, for irregular buildings, ESMA can be used up to height of 40 m (~ 13 storeys) and 12 m (~ 4 storeys) in lower and higher zones respectively. Linear DA is required for buildings not covered under the above restrictions.

Using IS:1893-2002, regular buildings can be designed as high as 30 storeys in all the metros of India, except New Delhi, without using linear DA. Also, for irregular buildings design can be done for up to 13 storeys. New Delhi is located in a higher seismic zone (zone IV), thus, ESMA can be used for up to 13 storey high regular

Table 1. Conditions on use of ESMA in various national codes

	Maximum building height[+] (m)				
Country	Regular	Irregular	Seismic zone	Soil profile	T_1 (s)
India[4]	40	12	Higher	–	–
	90	40	Lower	–	–
USA (IBC)[5]	2–3 storeys	–	Lower	–	$< 3.5T_c$*
Eurocode 8 (ref. 2)	–	–	–		< 2.0 or $< 4T_c$*
Columbia[2]	60	–	–	Not on soft clay	< 0.7
	–	18	–	–	–
Israel[2]	80	–	All	–	< 2.0
	–	80	Lower	–	< 2.0
	–	5 storey	Lower	For buildings with a soft storey	–
	–	20	All	For buildings with plan irregularities	–
The Philippines[2]	70	20	–	Soft clay <12 m thick	< 0.7
New Zealand[2]	15	–	–	–	< 2.0 (regular)
	–	15	–	–	< 0.45 (irregular)
Algeria[2]	65	–	Lower	–	–
	30	8–23	Higher	–	–
	–	All	Lowest	–	–
Costa Rica[2]	30	–	–	–	–
Iran[2]	50	18	–	–	–
Nepal[2]	40	–	Lower	–	–
Venezuela[2]	–	60	–	For buildings with plan irregularities	–

[+]Typical storey height in buildings is about 3.0 to 3.5 m.
*T_c is the natural period corresponding to the beginning of velocity-sensitive region on the response spectrum as shown in Figure 3.

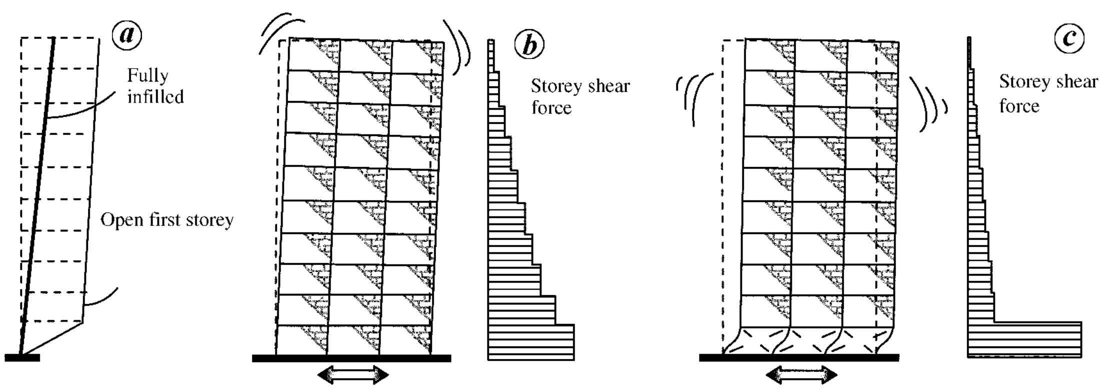

Figure 2. Effects of masonry infills on the first mode shape of a typical frame of a ten-storey RC building. ***a***, Displacement profile; ***b***, Fully infilled frame; ***c***, Open first storey frame.

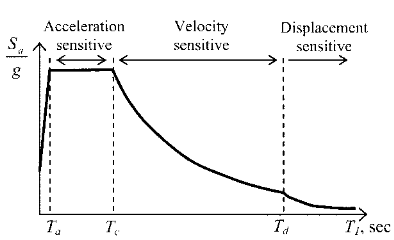

Figure 3. Acceleration response spectrum for average ground motion.

and up to 4 storey high irregular buildings. On the other hand, the International Building Code (IBC)[5] allows ESMA for regular and slightly irregular buildings consisting of only 2 to 3 storeys even in lower seismic zones; these being the most stringent requirements among the national codes worldwide[2].

Conditions imposed on use of ESMA by the national codes of twelve countries[2] from different parts of the world are summarized in Table 1. Codes of many developing countries follow height restrictions similar to the Indian code[4] for ESMA to be used. However, in many codes, soil type and T_1 also play an additional role in selecting the method of analysis.

According to several codes as shown in Table 1, ESMA is not allowed for flexible structures whose T_1 is more than a value varying from 0.45 to 2.0 s, or 3.5 to 4 times the natural period corresponding to the beginning of the velocity-sensitive region on the response spectrum (T_c, Figure 3), whichever is less. Velocity-sensitive structures have intermediate fundamental natural period of vibration on the response spectrum, and their response is governed by the ground velocity. The stiffer structures have lesser natural period (towards left of the velocity-sensitive region), and their response is governed by the ground acceleration; most buildings fall in this category. The flexible structures have larger natural period (towards the right of the velocity-sensitive region), and their response is governed by the ground displacement, for example, large span bridges.

The codes of Columbia and the Philippines do not allow ESMA for buildings on deep, soft clayey soils, which are likely to amplify the ground motion and cause buildings to experience larger-than-expected seismic forces. All the conditions specified in Table 1 for a particular code must be satisfied for the use of ESMA.

India lies in one of the most seismically active regions of the world. However, the provisions[4] in IS:1893-2002 governing the method of analysis to be used for seismic design of buildings are the most liberal. Use of ESMA could have been justified when computers were not available easily. Now, with high-speed digital computers available easily, several specialized software are available at an affordable price, which can be used for static and dynamic analysis considering linear and nonlinear behaviour.

The primary motive of this correspondence is to alert the readers that time has come to switch over to linear/nonlinear dynamic and pushover analysis methods, which not only provide better understanding of structural behaviour but also improved estimates of member design forces. The restrictions[4] in IS:1893-2002 governing the choice of methods of analysis need serious reviews and revision.

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