EVALUATION OF A SIMPLIFIED METHOD FOR THE DETERMINATION OF THE NON LINEAR SEISMIC RESPONSE OF RC FRAMES

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SUMMARY

In this paper a simplified method is developed for the evaluation of the seismic behavior of building structures, in which the contribution to the response of higher modes of vibration is important. The steps are given for the application of the method. To validate results of the proposed method a 17-story reinforced concrete building is studied. The results obtained are compared with those corresponding to a nonlinear step by step dynamic analysis and show a good approximation.

INTRODUCTION

Current seismic design codes for structures are based on providing enough capacity so that they resist without collapse the seismic forces to which they will be subjected during their useful life. In the definition of these forces it is generally accepted that structures can suffer damage when subjected to the action of intense earthquakes, however, the actions for which the structures are designed are defined from elastic analysis, which only considers in an approximate way the damage that they may experience. In general, it is not possible, through a linear analysis, to estimate the behavior of the structures when they undergo seismic forces that make them behave nonlinearly, since the forces to which they are subjected not only depend on the characteristics of the excitation but also on the dynamic properties of the structures in the linear and nonlinear range of behavior. Thus, in to know the real behavior of the structures, it is necessary to carry out nonlinear step by step dynamic analyses which have the inconvenience of being complex and expensive.

Recently, research has been carried out to approximate using simplified methods of evaluation the nonlinear behavior of building structures subjected to earthquakes, e.g., Camilo (1995). With these methods it has been possible to evaluate the behavior of regular buildings, with the restriction that they do not include in their formulations the contribution to the response of higher modes of vibration, which is of importance for structures of considerable height. Another aspect that these methods ignore is that when subjected to intense earthquakes, the lateral stiffness of the structure diminishes with increasing levels of base shear, which in turn causes changes in the distribution of the equivalent lateral forces to which the structure is subjected.

The main objective of this work is to develop and verify an approximate method for the evaluation of the seismic behavior of frames, based on the Capacity Spectrum Method, CSM, as originally proposed by Freeman *et al* (1975). This method considers explicitly in its formulation the contribution of higher modes of vibration and the variation of the stiffness of the structure when inelastic effects occur during its earthquake response.

This paper presents the different existing approaches to calculate the distributions of lateral static forces equivalent to the seismic ones. It describes in detail the evaluation method and an application example, which illustrates its practical implications. The results obtained from this example are compared with those corresponding to a nonlinear step by step analysis. Finally, the conclusions derived from this work are presented and recommendations are given stressing the fact that there are some important aspects that deserve further research.

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EQUIVALENT LATERAL STATIC LOADS FOR THE SEISMIC ANALYSIS OF BUILDINGS

Most existent design regulations accept as a substitute to the dynamic seismic analysis of a structure a linear static analysis with lateral loads equivalent to the seismic ones. Recently, use of these equivalent loads has been extended to the evaluation of the nonlinear seismic behavior of buildings through a nonlinear static analysis in which the magnitude of the loads is monotonically increased until collapse. In this type of analysis, known as pushover, the magnitude of these lateral loads is not important, but their distribution with height is, for which different investigators have proposed different ways to calculate their distribution.

MODELS OF EQUIVALENT LATERAL LOADS

The most common distribution of equivalent lateral loads specified in seismic design codes is in the form of an inverted triangle, which approximately corresponds to the distribution if only the contribution in the fundamental mode were considered. However, when structures of considerable height are analyzed, the contribution to the response of higher modes of vibration can be important, making it necessary to carry out a dynamic or approximate static analysis with distributions that explicitly consider this contribution.

Load distributions with the contributions of only the fundamental mode (approach 1).

In buildings that respond dominantly in their fundamental mode, the distribution of equivalent static lateral loads is calculated using the equation:

$$F_i = \frac{m_i \phi_i}{\sum_j^N m_j \phi_j} V \qquad (1)$$

where Fi , m_i and ϕ_i are the respective lateral force, mass and floor ordinate of the fundamental mode corresponding to the level i , V is the base shear force and N is the number of levels of the structure. This load distribution is similar to that proposed in the Construction Code for the Federal District, DDF (1993), among other regulations to define the equivalent static seismic forces in buildings.

Load distributions with the contributions of higher modes (approach 2)

. A generalization of the previous pattern of loads, which includes the contribution of higher modes of vibration in the calculation of the distribution of the equivalent lateral loads, involves the use of a probabilistic approach of superposition of modal contributions. In this work two formulations are investigated, one used by Freeman *et al* (1998), approach 2-A, and the other by Valles *et al* (1996), approach 2-B.

The formulation used by Freeman *et al* (1998) is based on the static method of seismic analysis, which includes the contribution of higher modes of vibration in the distribution of the lateral loads by means of the combination of the contributions of each considered mode with the rule of modal superposition of the square root of the sum of squares, SRSS, Chopra (1985). In this formulation the distribution of lateral loads depends on the pseudo spectral acceleration, Sa, corresponding to each mode, which in turn depends on the state of damage of the structure. The equation to calculate this distribution of the lateral loads is:

$$F_i = \sqrt{\sum_{j=1}^{N} \left(\left[\frac{\sum_{k=1}^{N} m_k \phi_{kj}}{\sum_{k=1}^{N} m_k \phi_{kj}^2} \right] \phi_{ij} Sa_j m_i \right)^2} \qquad (2)$$

where ϕ_{kj} is the modal shape of floor k and mode j and Sa_j is the pseudo spectral acceleration of mode j.

An alternative to the distribution of eq 2, is that proposed by Valles *et al* (1996) in which, the contribution of higher modes of vibration is included by accepting the existence of an "equivalent" fundamental mode, $\bar{\phi}_i$, which is determined through a combination of vibration modes using the SRSS rule, i.e.:

$$\bar{\phi}_i = \sqrt{\sum_{j=1}^{N} \left(\phi_{ij}\Gamma_j\right)^2} \tag{3}$$

where Γ_j is the participation factor of the mode j defined as:

$$\Gamma_j = \frac{\sum_{k=1}^{N} m_k \phi_{kj}}{\sum_{k=1}^{N} m_k \phi_{kj}^2} \tag{4}$$

The equation that defines the distribution of equivalent static loads is:

$$F_i = \frac{m_i \bar{\phi}_i}{\sum_{k=1}^{N} m_k \bar{\phi}_k} V \tag{5}$$

Using a distribution of lateral loads in a pushover analysis, the capacity of the structure when subjected to incremental lateral forces may be determined. During this analysis in each load step, the base shear and the corresponding roof displacement are recorded, and with these the curve called the capacity or pushover curve of the structure may be plotted. It is possible to consider that when the stiffness of the structure changes during pushover, i.e. structural elements reach maximum strength and plastic hinges appear, a new distribution of lateral loads may be calculated with which it is possible to consider the state of deformation of the structure in the calculation of the lateral loads with which the pushover analysis is carried out.

DETERMINATION OF THE NONLINEAR SEISMIC BEHAVIOR OF PLANE FRAMES

To determine the nonlinear behavior of plane frames, in this paper a simplified procedure is proposed for the calculation of the distribution of story displacements, interstory drifts and distribution and magnitude of plastic hinges as the result of subjecting the structure to a given seismic demand. The proposed method is a variation of the CSM, but differs in that to determine the performance point, the capacity curve of the structure is not compared with the response spectrum of the excitation. Instead, to determine the behavior of a frame subjected to one or several earthquakes, the original structure represented by a multiple degrees of freedom system, MDFS, is transformed into one equivalent single degree of freedom system (SDFS) using concepts of structural dynamics. The equivalent SDFS is then subjected to one or several seismic records and its maximum displacement is obtained. This displacement corresponding to the spectral displacement (Sd) of the fundamental mode is then transformed to the corresponding roof displacement of the MDFS with which the performance point of the structure to a given seismic demand is determined.

To include the contribution of higher modes of vibration to the distribution of equivalent static loads required by this method, the two formulations discussed above are evaluated. In approach 2–A, from the capacity curve resulting from the pushover only the contribution of the fundamental mode is extracted, making it possible to determine the properties of the equivalent SDFS. In approach 2 B, the behavior curve of SDFS corresponding to the "equivalent" mode, which includes the contribution of higher modes is directly obtained from the capacity curve derived from the pushover analysis. The steps involved in the procedure proposed in this investigation are as follows:

1. A static nonlinear pushover analysis of the structure represented by a MDFS is carried out and with the results of base shear and roof displacement the capacity or pushover curve of the structure is constructed.

2. The capacity curve is approximated with a bilineal form and this, in turn, is transformed to the space of Sa versus Sd by means of the following equations:

$$S_a = \frac{V}{\alpha_j W} \tag{6}$$

$$S_d = \frac{\Delta}{PF_{ij}} \tag{7}$$

where Δ is the roof displacement of the building, W is total weight of the structure and PF_{ij} is the modal participation factor for level i and mode j, i.e.:

$$PF_{ij} = \left[\frac{\sum_{k=1}^{N} m_k \phi_{kj}}{\sum_{k=1}^{N} m_k \phi_{kj}^2} \right] \phi_{ij} \tag{8}$$

and α_j is the base shear participation factor for mode j, defined as:

$$\alpha_j = \frac{\left[\sum_{k=1}^{N} m_k \phi_{kj} \right]^2}{\left[\sum_{k=1}^{N} m_k \right] \sum_{k=1}^{N} m_k \phi_{kj}^2} \tag{9}$$

3. The mass corresponding to the fundamental mode, m^1, is obtained by transforming the total mass of the MDFS with the chosen approach for distribution of loads. The equation required by approaches 1 and 2-A is:

$$m^1 = \sum_{i=1}^{N} m_i \phi_i^2 \tag{10}$$

and, by approach 2-B:

$$m^1 = \sum_{i=1}^{N} m_i \bar{\phi}_i^2 \tag{11}$$

4. The modal base shear (Vm) versus Sd curve is determined by multiplying the modal mass by Sa obtained in step 2. This curve is the behavior curve of the SDFS from which one can calculate the elastic and inelastic stiffness of the system, as well as its yielding force.

   In the case of approach 2–A, from the original capacity curve considering the contribution of higher modes, the contribution of only the fundamental mode required to calculate the Vm versus Sd curve needs to be extracted. To do this the original curve is approximated with straight line segments, so that the points corresponding to first yielding and to other intermediate states of interest in the behavior of the structure may be defined, fig 1. To determine the line segments of the reduced capacity curve corresponding only to the contribution of the first mode, curve O-A'-B'-C'-D ' in fig 1, it is necessary to transform each of the line segments of the original curve O-A-B-C-D, with the following equations:

$$\Delta^{(\text{mode 1})} = \alpha_1 \Delta^{(\text{N modes})} \tag{12}$$

$$V^{(\text{mode 1})} = \alpha_1 V^{(\text{N modes})} \tag{13}$$

   where $\Delta^{(\text{mode 1})}$ and $V^{(\text{mode 1})}$ are the roof displacement and base shear corresponding to the contribution of the fundamental mode and $\Delta^{(\text{N modes})}$ and $V^{(\text{N modes})}$ are the corresponding roof displacement and base shear including the contribution of all modes, .

5. A nonlinear analysis of the SDFS subjected to a given seismic demand is carried out, obtaining the maximum Sd, which represents the performance point of the SDFS.

6. The maximum Sd, obtained in step 5, is transformed from the SDFS to the MDFS considering the evaluation approach used

   Approach 1

$$\Delta = S_d \; PF_{\mathrm{N1}} \tag{14}$$

Approach 2-A

$$\Delta^{(\text{mode 1})} = S_d \; PF_{\mathrm{N1}} \tag{15}$$

$$\Delta^{(\text{ N modes })} = \beta \Delta^{(\text{mode 1 })} \tag{16}$$

Approach 2-B

$$\Delta^{(\text{equi. mode })} = S_d \; PF_{\mathrm{N1}} \tag{17}$$

where Δ is the roof displacement neglecting the contribution of higher modes, β is the ratio of the roof displacement with the contribution of N modes to that corresponding to the fundamental mode, and $\Delta^{(\text{equi. mode })}$ is the roof displacement considering the contribution of the "equivalent" fundamental mode

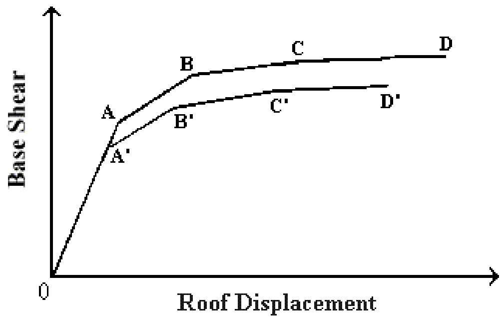


Figure 1. Idealized capacity curves.

7. A new pushover analysis of the MDFS is carried out to the maximum roof displacement obtained in step 6, from which the lateral displacements of the floors, the interstory drifts and the distribution and intensity of the plastic hinges of the structure can be determined.

MODEL STUDIED

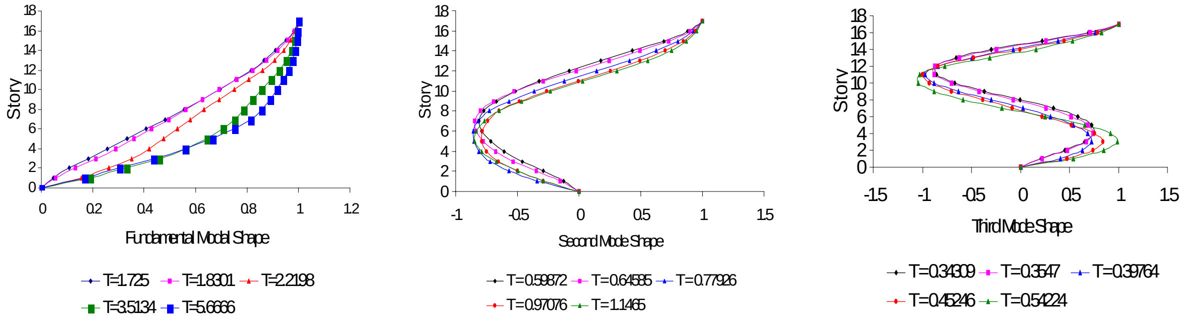
The structure studied is a regular square shape plan office building, located in area III of the seismic zoning of the valley of Mexico. The structure consists of four reinforced concrete frames in each direction with three bays each 8 m long. The building has 17 floors with a story height of 3.20 m, except the first one which is 4 m. Beam sections are 35x90 cm in all floors. Columns are of 4 types with sections, 110x110 cm from story 0 to 7, 90x90 cm from story 8 to 11, 75x75 cm from story 12 to 14 and 60x60 cm for the last three floors. Slab thickness in all floors is 10 cm. The evaluation method was applied to an interior frame.

COMPARISON AND ANALYSIS OF THE RESULTS

To validate the evaluation method, the changes in the modal shapes when inelastic effects in the frame occurred under increasing lateral loads during pushover were determined. Fig 2a shows the changes in the fundamental mode as the damage increases in the structure. Likewise, figs 2b and 2c illustrate the changes in the second and third modes correspondingly.

Taking into account the two considered approaches, the distributions of lateral loads in the frame with increasing values of base shear were determined, fig 3a. In fig 3b the variation of the distribution of the lateral loads of approach 1 is presented as damage in the structure occurs.

Using the evaluation method with the considered approaches of distribution of equivalent lateral loads, the seismic performances of the investigated frame were determined. For approach 1 the pushover analysis of the structure was carried out and the corresponding capacity curve determined, fig 4. This capacity curve was approximated with a bilineal form and then transformed, into the Sa versus Sd, curve, i.e., the behavior curve of the SDFS, fig 5. For the equivalent SDFS the modal mass was calculated with eq 10 and a fraction of critical damping of 0.05 was assigned. Using as seismic demand the SCT-EW record of the 1985 Michoacán earthquake, the maximum response of the equivalent SDFS was determined and, in an inverse process, the corresponding maximum roof displacement. This displacement was 0.443 m. The same procedure was carried out using approaches 2-A and 2-B. For approach 2-A the capacity curve corresponding to the fundamental mode base shear was used, fig 6, and the maximum roof displacement was 0.438 m. For approach 2-B the capacity curve of the frame and its bilineal idealization is presented in fig 7. The corresponding roof displacement was 0.422 m.



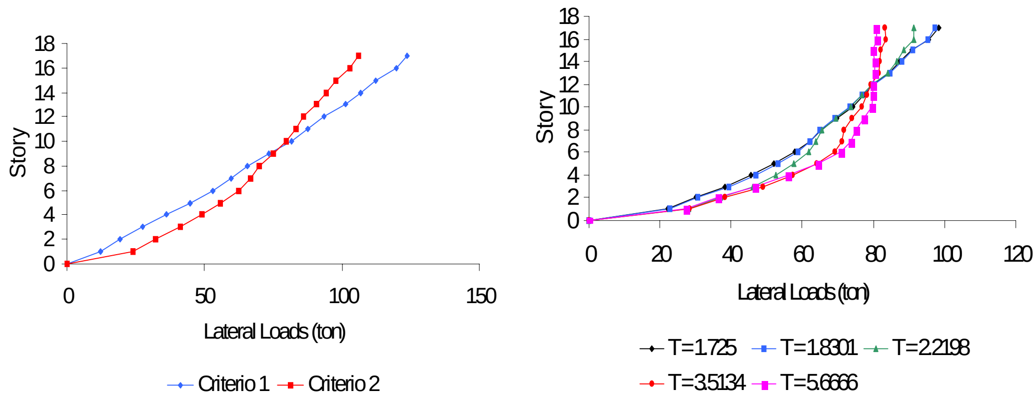
(a) (b) (c)

Figure 2. Variation modal shapes with change of stiffness.

To compare, a nonlinear dynamic step by step analysis was carried out using the same seismic demand giving a maximum roof displacement of 0.437 m. Fig 8 shows the comparison of the lateral displacements and fig 9 that of the interstory drifts. Finally, figs 10a to 10c show the distributions and sizes of plastic rotations of hinges as obtained with approaches 1, 2-A and the nonlinear dynamic analysis, respectively.

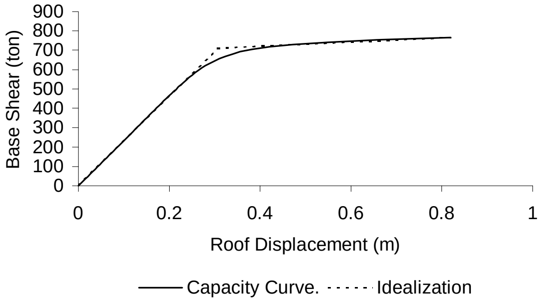
CONCLUSIONS

In this paper an approximate method of seismic evaluation of tall building frames was presented and tested. The results obtained considering only the contribution of the fundamental mode and its variation with the state of deformation of the frame approximate well those obtained from the nonlinear step to step dynamic analysis. Also, the analyzed frame, being regular, is deformed basically in its first mode, even when the frame presents considerable damage when plastic hinges occur in the structural elements. The lateral displacements, the distortions and the distribution and intensity of the plastic hinges obtained taking into account the contribution of higher modes approximate very well those from the nonlinear step by step dynamic analysis.



(a) (b)

Figure 3. Comparison of the distribution of lateral loads



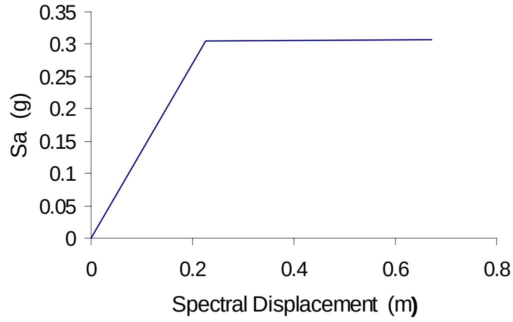


Figure 4. Capacity curves corresponding to approach 1 of evaluation.

Figure 5. Behavior curve Sa vs Sd (approach 1 of evaluation).

From the results obtained when applying the simplified method of evaluation to the 17-story frame one may conclude: 1) The stiffness degradation of the structure influences more the fundamental mode shape than those of higher modes. 2) The distribution of lateral loads varies as plastic hinges are formed and/or increased in the structural elements. 3). The effect of higher modes is important when the frame behaves in the nonlinear range. 4). In the determination of the seismic behavior, it is very important to consider the changes in the modes of vibration.

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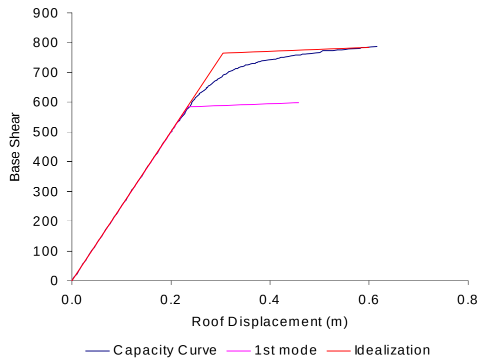

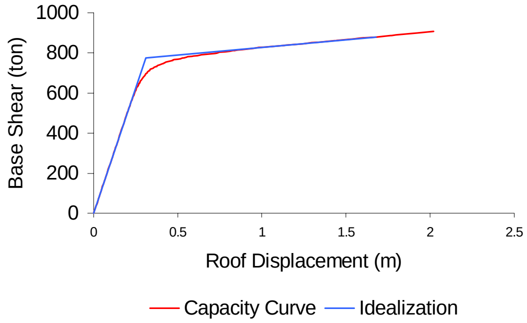

Figure 6. Capacity curves corresponding to approach 2-A of evaluation.

Figure 7. Capacity curve corresponding to approach 2-B of evaluation.

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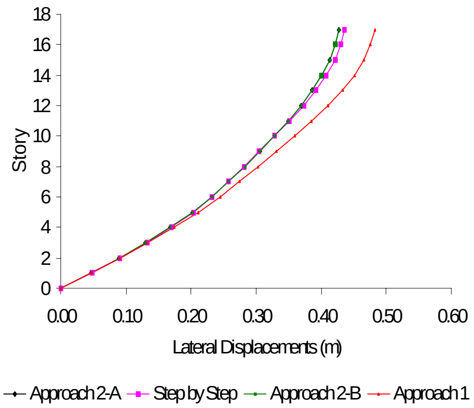


Figure 8. Comparison of floor displacements.

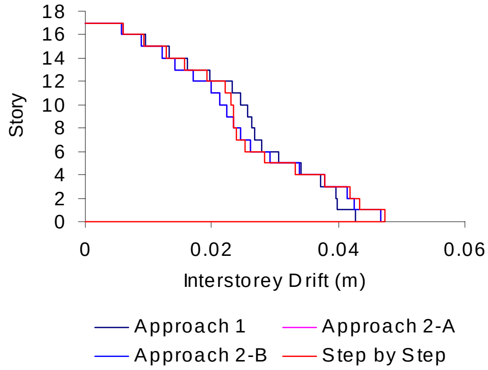
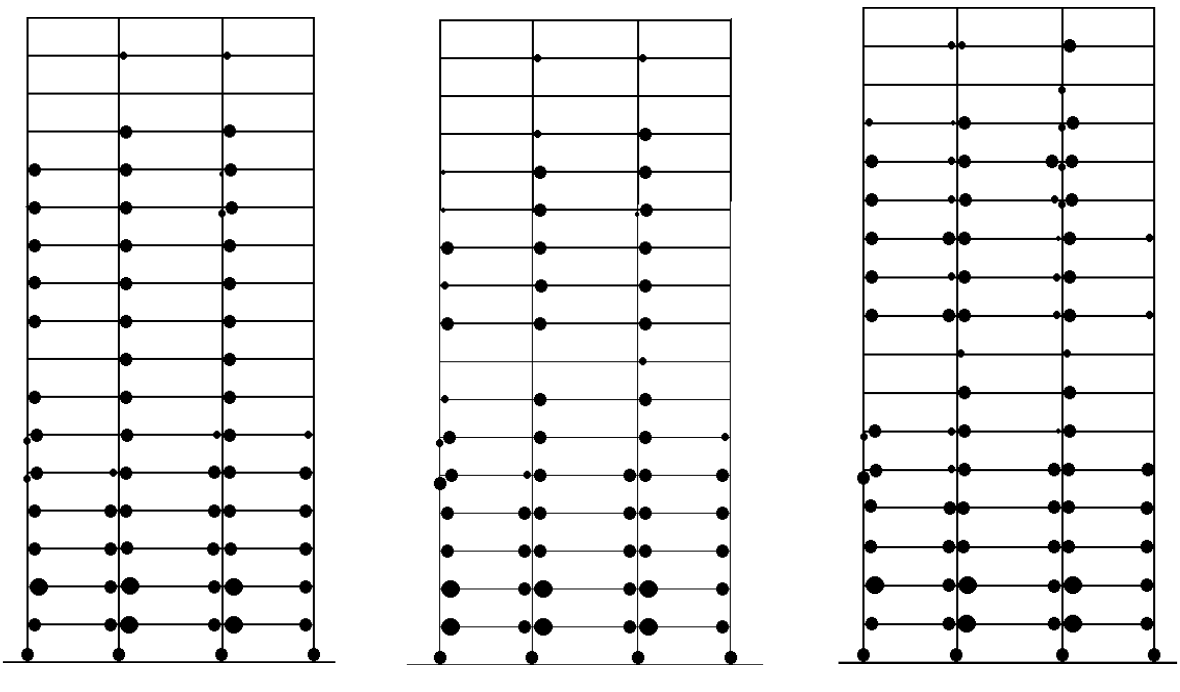


Figure 9. Comparison of interstory drifts.



(a) (b) (c)

Figure 10. Distribution and intensity of plastic hinges, a) approach 1, b) approach 2 and c) dynamic analysis.