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SEISMIC NORMALIZATION OF STRUCTURAL ANALYSIS METHODS

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ABSTRACT

Seismic standards generally establish that the mechanical actions on the structures be obtained from linear elastic models with fixed basis. The Venezuelan Code COVENIN 1756 (1987) and other codes prescribe the analysis methods of structures in accordance with the height (H) of the building and the eccentricity over width (e/B) ratio. It is proposed to include the performance level and the use of the static and 3-dimensional dynamic analysis, according to the degree of irregularity. This paper evaluates, clarifies and normalizes the application of 2D and 3D analysis for different slenderness (H/B) and irregularity (e/B) ratios. With this purpose a three dimensional complete elastic program (3DCE) was prepared for the solution of frames without soil-structure interaction, with infinitely rigid nodes, rigid diaphragms and spectral forces, calculated according to the Venezuelan Code COVENIN 1756 (1987). The results were compared with a comercial 3D dynamic analysis program and besides it studies the application of the 3DCE methods to distribute the forces of the 3D dynamic analysis.

KEYWORDS

Analysis methods, normalization methods, horizontal irregularity, slenderness building, seismic normalization, structural analysis.

INTRODUCTION

One of aspects with greatest uncertainly is the selection of the mathematical model and the method of analysis that will consider adequately the superstructure, the non-structural elements, the foundation system and the conditions of the underlying soil. These days, codes continue to recommend the application of elastic analysis methods and the inelastic methods are being included very slowly. The elastic methods have been a part of the formation of structural engineers, are easier to interpret and in all the world there are sufficient programs available. However, their usage has not been normalized, when to use 2D or 3D methods nor effects to be included as shear deformations, axial effects, nodal stiffness, base settlements or P-Delta effects. Thus, each designer obtains as a result of the analysis a series of different actions that determine different designs. In this paper and in others which should be developed, different cases of buildings are analized to know and compare the safety of the methods from the most elementary to the most complex according to the irregularity and structural slenderness, with a view towards normalization. The seismic analysis determines the maximum actions on the global building and the structural analysis transfers these actions to the structural elements.

Generally the codes normalize the design methods and the determination of the seismic actions, but do not say anything about the structural analysis methods. It is clear that the seismic analysis must be differentiated from the structural analysis.

ANALYSIS METHODS

According to Dowrick (1977) methods of analysis of the seismic forces depend on the structural complexity and the system of applied forces. He considers that a non-linear analysis with an inadequate input is less realistic than another with a desirable response spectra. According to COVENIN 1756 (1987) the methods of seismic analysis are adopted according to height and to the irregularity of the building as shown in Table 1, where ESM is the Equivalent Static Method, EST is Equivalent Static Torsion, AD1 is Dynamic Analysis with one degree of freedom and AD3 is Dynamic Analysis with three degree of freedom.

Table 1. Seismic analysis methods according to COVENIN 1756 (1987)

Buildings	**Regular**	**Irregular**		
		$e/B \leq 0.08$	$0.08 < e/B \leq 0.12$	$e/B > 0.12$
Heights no greater than 20 levels or 60 meters	ESM + EST	AD1 + EST or AD3	AD3 or AD1 + EST	AD3
Heights greater than 20 levels or 60 meters	AD1 + EST			

According to Lobo-Quintero (1993) the seismic analysis methods can be used according to the level of performance and the degree of irregularity as shown in Table 2, where AE3 is Static Analysis with three degrees of freedom per level.

Table 2. Selection of the seismic analysis method

Performance level	**Regular**	**Irregular**		
		$e/B \leq 0.08$	$0.08 < e/B \leq 0.12$	$e/B > 0.12$
PL1	ESM + EST	ESM + EST	AD1 + EST	AE3
PL2	ESM + EST	AD1 + EST	AE3	AE3 or AD3
PL3	AD1 + EST	AE3	AE3 or AD3	AD3

When the actions on the members are required, after the seismic analysis the structural analysis methods are applied. The distribution of the forces, except in some particular cases, is done based on the infinitely rigid diaphragm hypothesis. On this basis, in this paper a table is proposed to select the structural analysis method according to the seismic analysis used, according to the irregularity of the building and the level of performance according to Grisolía (1995a).

For the solution of framed structural systems the following methods were applied:
(1) Simplified 2D analysis (A_1): solution of simple plane frames with flexural deformations in all members and also axial deformations in columns.
(2) Complete 2D analysis (A_2): solution of plane frames considering additionally shear deformations, infinite stiffness of joints and P-Delta effects.
(3) Complete 3D analysis (A_3): solution of structures as 3D frames with rigid diaphragms and three degrees of freedom per level, considering shear deformations, infinite stiffness of joints and P-Delta effects..
(4) Complete 3D analysis with forces obtained from a dynamic analysis with three degrees of freedom per level (A_4), taken as a basis for comparation (AD3 with A_3).

(5) Complete 2D analysis with forces obtained from a dynamic analysis with three degrees of freedom per level, taking only one component of the earthquake in the direction analyzed (A_5).
(6) Complete 2D analysis with forces obtained from a dynamic analysis with three degrees of freedom per level, taking account the components of the earthquake in the two directions analyzed (A_6).
The methods A_1, A_2, A_3 and A_4 were compared in the paper by Grisolía (1995a) and later a complementary paper was prepared to compare methods A_4, A_5 and A_6 according to Grisolía (1995b). The results obtained conform the body of this paper.

MODEL USED

The different methods were applied to examples of regular and irregular buildings of different heights, to establish comparisons between 2D and 3D structural analysis. 16 buildings were selected: 4 of 4 levels, 4 of 8 levels, 4 of 16 levels and 4 of 20 levels, with slenderness ratios (H/B) of 2/3, 4/3, 8/3 and 10/3 respectively, and with the same square plan form of 3 bays of 6 meters each in both directions, as shown in Fig. 1.

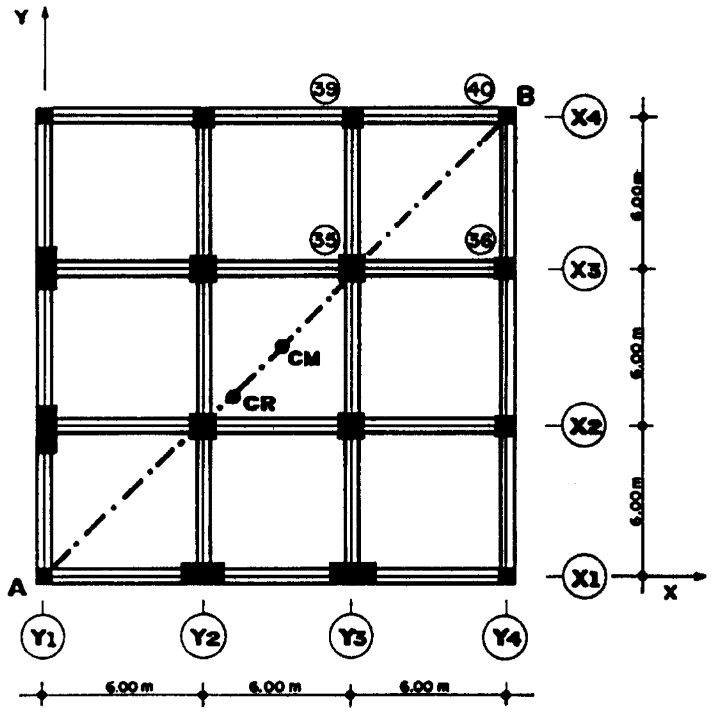

Fig. 1. Plan of the buildings

The dimensions of the columns along the X_1 and Y_1 axis were modified to vary the static eccentricity according to the classification of Table 1. The following cases were studied: $e/B = 0$; $e/B \leq 0.08$; $0.08 < e/B \leq 0.12$; and $e/B > 0.12$. As the eccentricity changes with height, the ground floor eccentricity was taken for this classification. The plan form in all buildings is symmetric with respect to the diagonal that passes through points A y B. This symmetric means that the analysis for the earthquake in the X direction is the same as for the Y direction.

The general data for the 16 buildings are the following: reinforces concrete of f'c = 350 kgf/cm2, weight 2500 kgf/m3; interstorey height: 3 meters; length of bays: 6 meters; location: Zona 4 (maximum seismicity) COVENIN 1756 (1987); S1 soil; structure Type I; use: offices; design level 3; ductility $\mu = 6$; accidental eccentricity factor: 0.10; limit on elastic interstorey drift: 0.003; limit on inelastic interstorey drift: 0.018. The permanent and the variable service loads were taken respectively as 1.0 and 0.5 tf/m for the border beams and 2.0 and 1.0 tf/m for the central beams.

The dimmensioning was done according to the stiffnesses of Tso (1990) and with a dynamic amplification factor τ obtained according to COVENIN 1756 (1987) as shown by Grisolía (1995a).

METHODOLOGY

For the A_1, A_2 and A_3 analysis the seismic forces were obtained by the Equivalent static Method and for A_4, A_5 y A_6 analysis by modal superposition with 3 degrees of fredom per level. To determine the maximum actions on the members and the maximum lateral displacements, five ultimate load combinations were considered in each of the analysis.

RESULTS OBTAINED

In each of the analysis done, the maximum value of lateral drift, shear force and flexural moment was obtained amongst the five load cases considered. To analyze and compare results, columns 35, 36, 39 y 40 were selected as shown in Fig. 1 because they are the farthest from the center of stiffnesses and correspond to the flexible side of the plan. The most representative graphs are shown in Fig. 2 to 17.

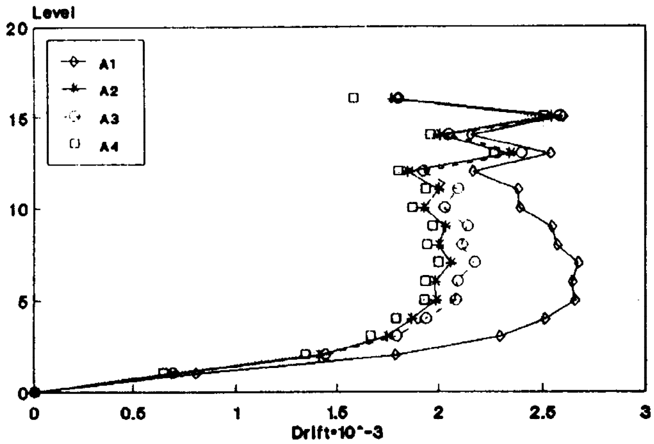

Fig. 2. Lateral drift. Columns 35 and 36
e/B=0

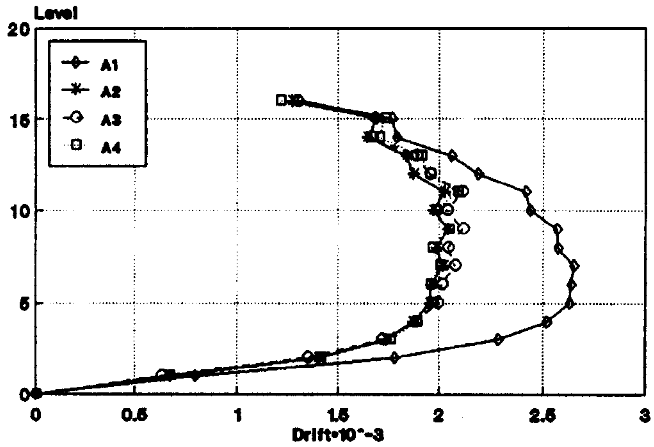

Fig. 3. Lateral drift. Columns 35 and 36
e/B>0.12

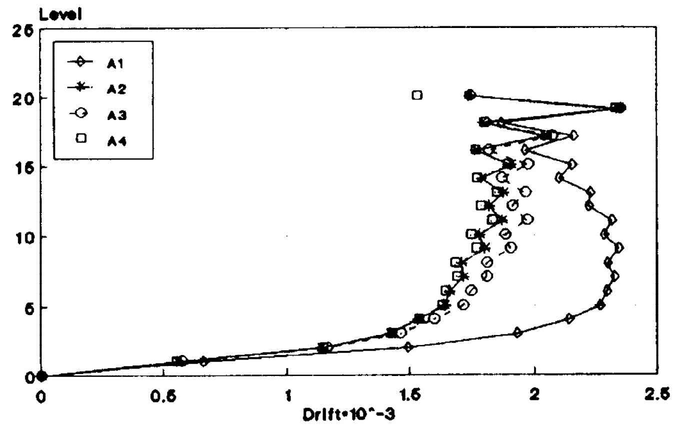

Fig. 4. Lateral drift. Columns 35 and 36
e/B=0

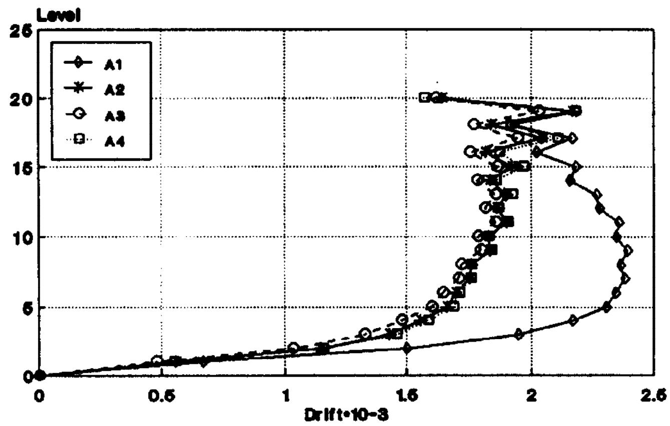

Fig. 5. Lateral drift. Columns 35 and 36
e/B>0.12

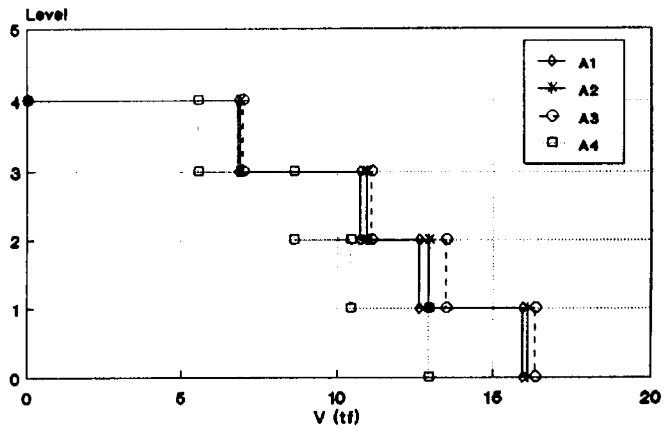

Fig. 6. Shear force. Column 35
e/B<0.08

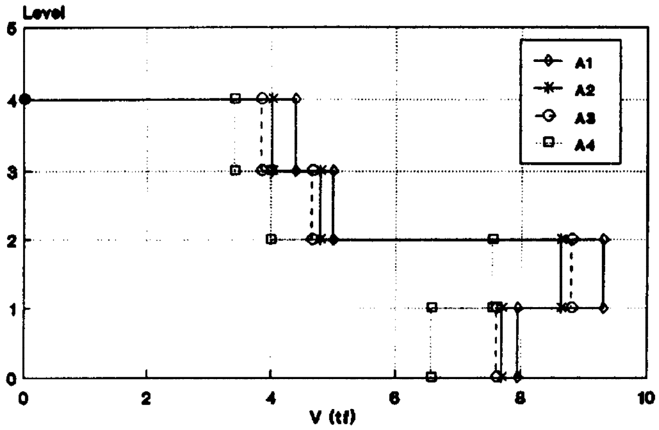

Fig. 7. Shear force. Column 36
0.08<e/B<0.12

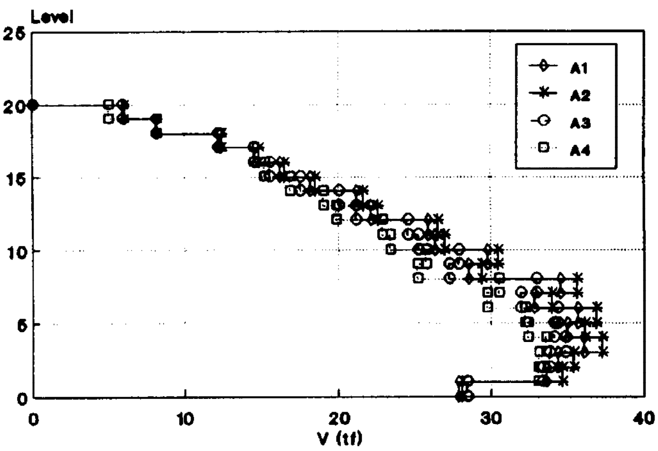

Fig. 8. Shear force. Column 39
e/B=0

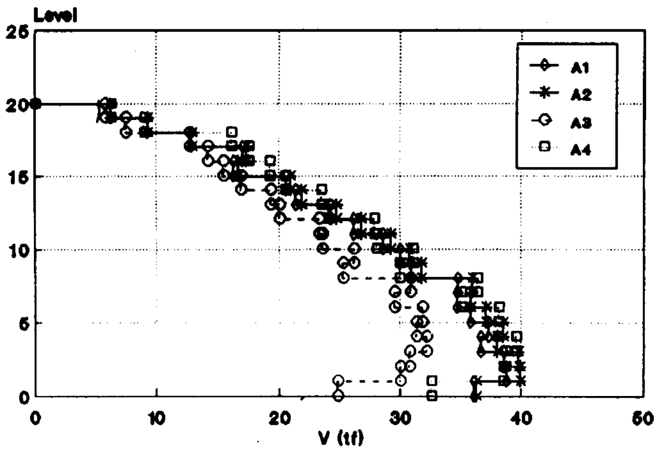

Fig. 9. Shear force. Column 39
e/B>0.12

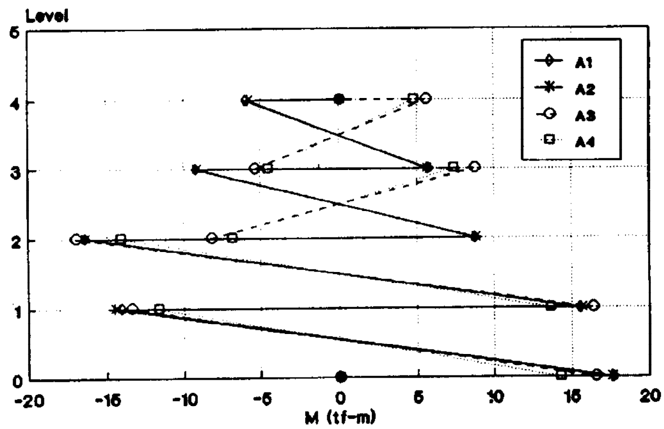

Fig. 10. Flexural moment. Column 39
0.08<e/B<0.12

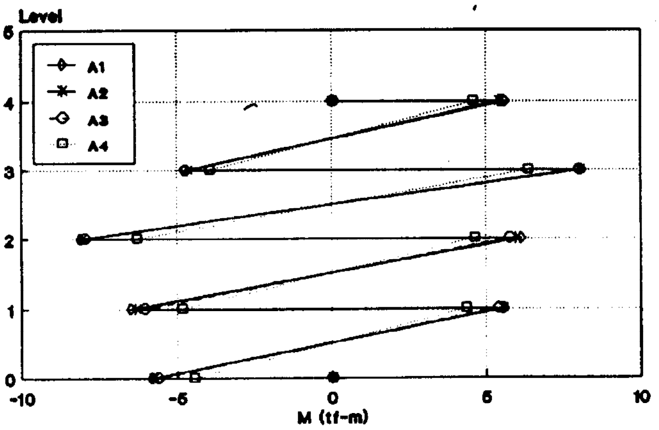

Fig. 11. Flexural moment. Column 40
e/B=0

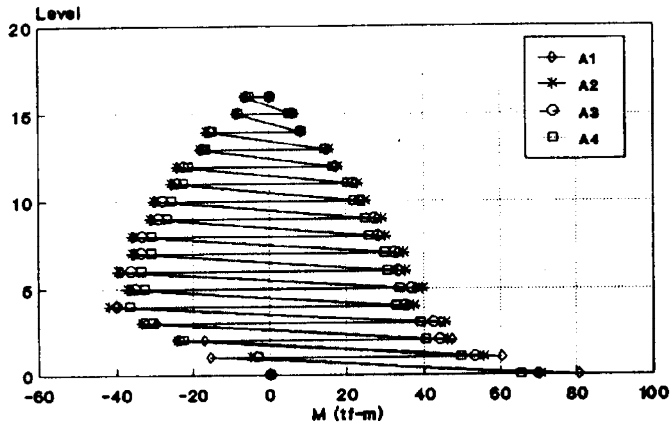

Fig. 12. Flexural moment. Column 39
e/B=0

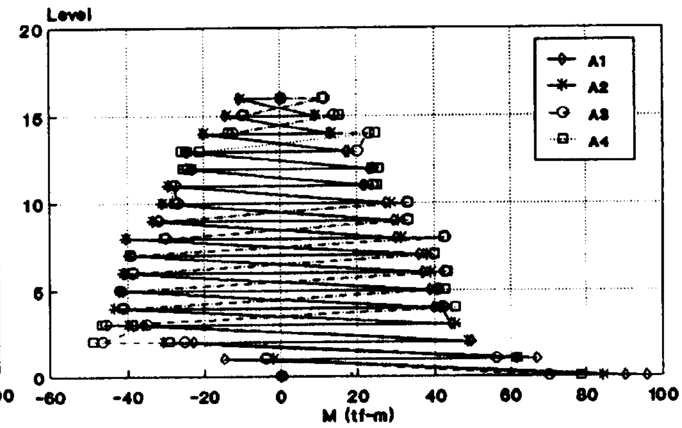

Fig. 13. Flexural moment. Column 39
e/B>0.12

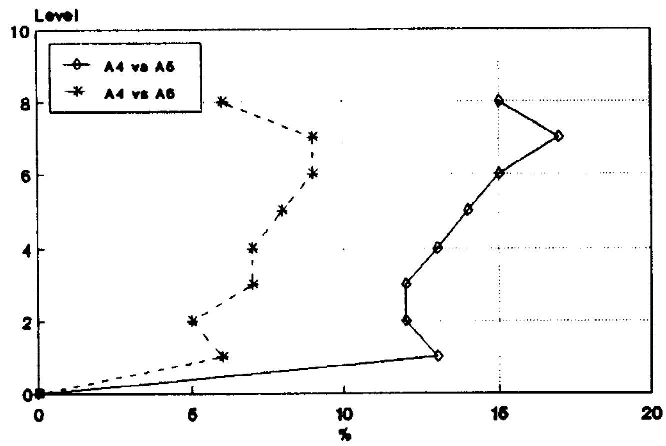

Fig. 14. Differences in lateral drift
Columns 35 and 36 e/B>0.12

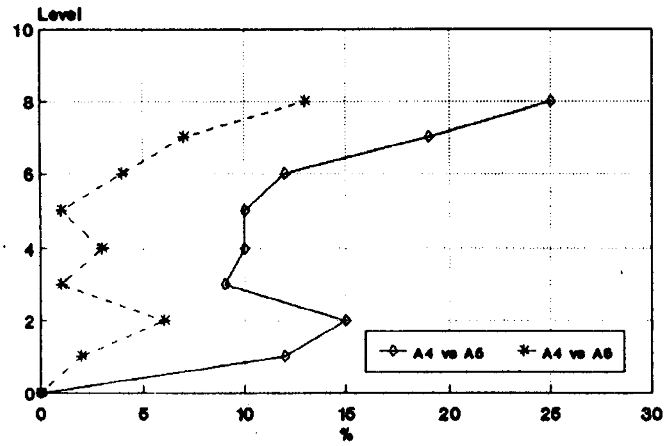

Fig. 15. Differences in shear force.
Column 40 e/B>0.12

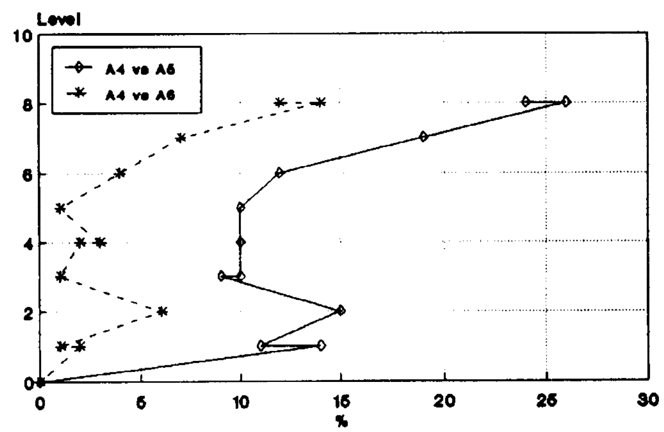

Fig. 16.Differences in flexural moment.
Column 40 e/B>0.12

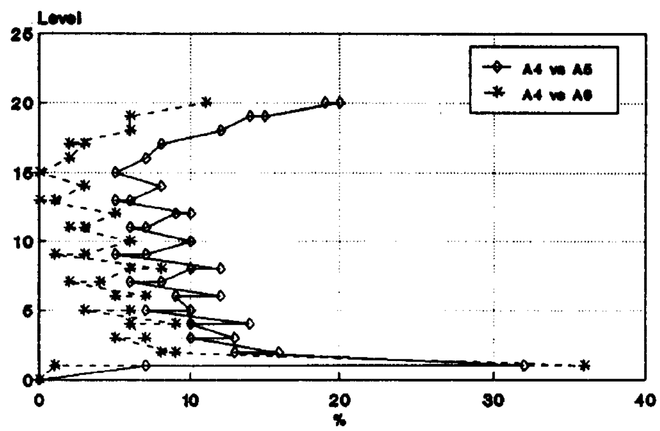

Fig. 17. Differences in flexural moment.
Column 40 e/B>0.12

CONCLUSIONS

(1) In all the buildings the pre-dimmensioning using method A_1 is conservative and results in the largest dimmensions of the members.
(2) In the smaller buildings as the pre-dimmensioning is according to code (minimum columns of 30x30 cmxcm), the structures results over-dimmensioned and has a stiff behavior, and therefore any of the analysis methods can be applied.
(3) In all of the building of 4 levels and in the 8, 16 and 20 levels ones which are regular or have eccentricities of up to 8%, if a conservative design is wanted it is convenient to use an A_2 or A_3 analysis, and if an economic design is wanted it is better to use an A_4 analysis.
(4) In the buildings of 8, 16 and 20 levels with eccentricities greater than 8%, it is convenient to use A_4 analysis, because the resulting actions on the members are greater.
(5) In many of the cases studied, the A_2 method gives results which are quite close to the 3D methods, therefore a 2D analysis which includes sufficient effects as to model the structure well can be considered acceptable.
(6) There are greater differences between the A_4 and A_5 analysis than between A_4 and A_6 analysis, which indicates that if a 2D analysis is to be used it should be an A_6.
(7) The indicated differences are greater at the lower and higher levels and increase with the eccentricity.
(8) In comparing the results obtained by the A_4, A_5 and A_6 analysis for $e/B > 0.12$, it was seen that the A_4 is greater than the others according to the maximum differences indicated in Table 3.

Table 3. Maximum differences between A_4, A_5 and A_6

	Lateral drift		**Shear force**		**Flexural moment**	
	A_4 vs A_5	A_4 vs A_6	A_4 vs A_5	A_4 vs A_6	A_4 vs A_5	A_4 vs A_6
Max. diff. (%)	10.09	5.55	14.18	6.82	17.59	11.83

(9) The flexural moments for level 1 obtained with an A_4 analysis are greater by a very large percentage with respect to those obtained with A_5 and A_6, resulting a very large standard deviation, that increases the degree of unsafety or of uncertainty obtained using 2D analysis.
(10) The shear force that acts on the frames in the direction perpendicular to that being analyzed, obtained by the modal superposition method with 3 degrees of freedom per level, represent the percentages indicated in Table 4 that in some cases surpasses the 30% recommended by COVENIN 1756 (1987)

Table 4. Percentages of shear in the perpendicular direction

	8 levels		**16 levels**		**20 levels**	
	Frame X_3	Frame X_4	Frame X_3	Frame X_4	Frame X_3	Frame X_4
$0.08 < e/B \leq 0.12$	29	41	15	29	14	28
$e/B > 0.12$	37	48	21	34	20	34

(11) In the selection studied, the distribution of 3D dynamic actions, for an A_6 analysis, that is, with an earthquake applied along an inclined direction to produce maximum base shear, results in a structural response that is smaller than an A_4 analysis, but with non-significant differences, this would indicate that any of the two can be used indifferently.
(12) For buildings of more than 20 levels it is convenient to make a comparison of the results obtained by dynamic methods with 2D and 3D structural analysis.
(13) As in the model used the radius of giration is almost constant, more research should be done with other models with different sources of eccentricity
(14) As the diversity of structures is unlimited, the results only show trends, which shows the need for more research to know which is the most adequate method.

(15) For large eccentricities and great heights a greater range of uncertainty can be seen between the results of each method, which requires reliability studies using inelastic methods, to obtain more precise conclusions as to the method which is most reliable. The codes must consider this aspect.

RECCOMMENDATIONS

(1) Within the limitations of the model and the number of examples performed, the 2D analysis of frames is safe for buildings with $e/B = 0$ and $e/B \leq 0.08$.
(2) For eccentricities $e/B > 0.08$ and buildings slenderness ratio $H/B > 4/3$ the 3D static or dynamic methods must be used.
(3) When high levels of performance are required, with slender and regular structures it is recommendable to apply a static inelastic reliability analysis seeking to determine structural failure mechanisms, critical zones, ductility demands and real load factors.
(4) The methods of structural analysis should be established according to the level of performance, the structural irregularity and method of seismic analysis used, as shown in Table 5. The level of performance depends of the seismic zonification and importance of building.

Table 5. Selection of the structural analysis method.

Performance level	**Regular**	**Irregular**		
		$e/B \leq 0.08$	$0.08 < e/B \leq 0.12$	$e/B > 0.12$
PL1	ESM + EST with A_1	ESM + EST with A_1 or A_2	AD1 + EST with A_2	AD1 with A_3
PL2	ESM + EST with A_1 or A_2	ESM + EST with A_2	ESM with A_3	ESM with A_3*
PL3	AD1 + EST with A_2	AD1 + EST with A_2 or AD1 with A_3	ESM with A_3* or AD3 with A_3*	AD3 with A_3*

* A reliable static inelastic spectral analysis has to be used.

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