ELASTIC AND INELASTIC RESPONSE OF THREE-DIMENSIONAL BUILDINGS MODELS, A COMPARISON OF DIFFERENT MODELS FOR ANALYSIS

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SUMMARY

The maximum response of five story concrete buildings characterized by their stiffness eccentricity, fundamental elastic lateral period, coupling level, and slenderness of the corner columns are studied. The buildings are subjected to gravitational forces originated by the dead and live loads and bi-directional ground movements. The maximum responses of structures considering elastic and inelastic behavior of the materials are looked at. The responses of a pseudo-three-dimensional model and of a three-dimensional model of the same buildings are evaluated. The differences between the elastic lateral, transverse, and torsional period and between the elastic energies of both models are analyzed. The differences between the elastic and inelastic diaphragm displacements when using both models are studied. Finally, the difference between the elastic and inelastic combined forces and the difference between the elastic and inelastic seismic forces are evaluated. The buildings studied present more transversal and torsional stiffness if the analysis considers a three-dimensional model. The responses under seismic load only are significantly more sensitive to the analysis model than those corresponding to the combined loads. The elastic torsional behavior of eccentric buildings with large transversal elements is more sensitive to the analysis model. The inelastic torsional behavior of structures with these characteristics and which also are laterally flexible presents even more accentuated sensibility to the model used. The evaluation of the seismic axial forces in the corner columns of buildings is very sensitive to the model used, especially if they are slender.

INTRODUCTION

Building analysis usually is done considering a plane behavior of the resisting frames connected by rigid diaphragms to represent the complete structure. The vertical deformations of the corner elements are not considered and the contribution of transverse stiffness of the frame elements is not considered. To consider these effects probably modifies the behavior of the system periods and consequently their responses. The inelastic response, since the columns are considered to have different levels of axial loads and bi-axial bending moments could also be altered. These columns are simultaneously subjected to the action of the axial load, two bending moments, two shear forces and torsion. However, and looking at it from a professional practice point of view, the use of complex models which include all the effects is not always justified. There are not many systematic studies to evaluate the difference between the elastic and inelastic responses of buildings modeled as 3D and pseudo-3D systems. [Huria, Raghavendrachar, and Aktan, 1991] studied the behavior of unsymmetrical walls under the combined effect of bi-axial bending moments and axial force, concluding that the interaction of the bi-axial bending moments could carry important changes in the uni-axial behavior of the walls.

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METHODOLOGY

Basic Model

A 3D model of a five story concrete building with lateral and transverse frames subjected to seismic ground movements represented by two horizontal components of actual earthquakes, which coincide with the principal directions of the building, is analyzed. The frame elements provide stiffness and strength in their lateral plane, as well as in their transverse directions. The vertical deformations of the columns are considered. The structural model considers a system of concentrated masses in each story. The ends of the elements are considered subjected to bi-axial bending moments, M_u and M_v, and to axial load N. A schematic representation of the model is shown in figure 1a. There are two symmetric frames in the U direction and three different frames in the V direction. Two of these are located in the borders of the plan (frames 1and 3), and the other (frame 2) is in the center of mass (figure 1b). The inelastic 3D response is obtained with the PC-ANSR program [Maison, 1992; Rihai et al., 1979]. The inelastic behavior of each element is concentrated in the element end, and the force-deformation curve for the element behavior is of the bi-linear type with a 90% loss of stiffness in the second branch (figure 1c). Systems with corner elements of big dimensions in the U direction (similar lateral and transverse stiffness), with corner elements of small dimensions in both directions (low transverse and torsional stiffness) and with corner elements of big dimensions in the V direction only (low transverse stiffness) are studied (figure 2). In each case symmetric and asymmetric structures are analyzed. The eccentricity of the structures comes from the variation of the dimensions of frames 1 and 3. The inertia ratio, I_1/I_3, take values $I_1/I_3=1$ (symmetric), $I_1/I_3=10$ (medium stiffness eccentricity), and $I_1/I_3=100$ (high stiffness eccentricity). The level of coupling of the frame columns is represented by the "beam-column stiffness ratio" ρ, defined by [Blume, 1968] as the ratio between the beam stiffness and the column stiffness for an intermediate story. Structures with ρ values varying from ρ=0.001 to 100 are analyzed. In all the cases, systems with lateral elastic fundamental periods, T_v=0.25, 0.75, and 2.0 seconds are studied, modifying the dimensions of the frame 2. In the pseudo 3D model two orthogonal columns represent the corner columns by assigning inertia only in the direction of the frame which they belong to. The axial degrees of freedom of the two columns are z_1 and z_2, as shown in figure 3(a). The individual elements have three degrees of freedom in each node. In the 3D model columns are considered to have six degrees of freedom at each node (figure 3b). The bending inertias about the two principal axes of the elements and the torsional stiffness are considered. As in the pseudo 3D model, the columns are

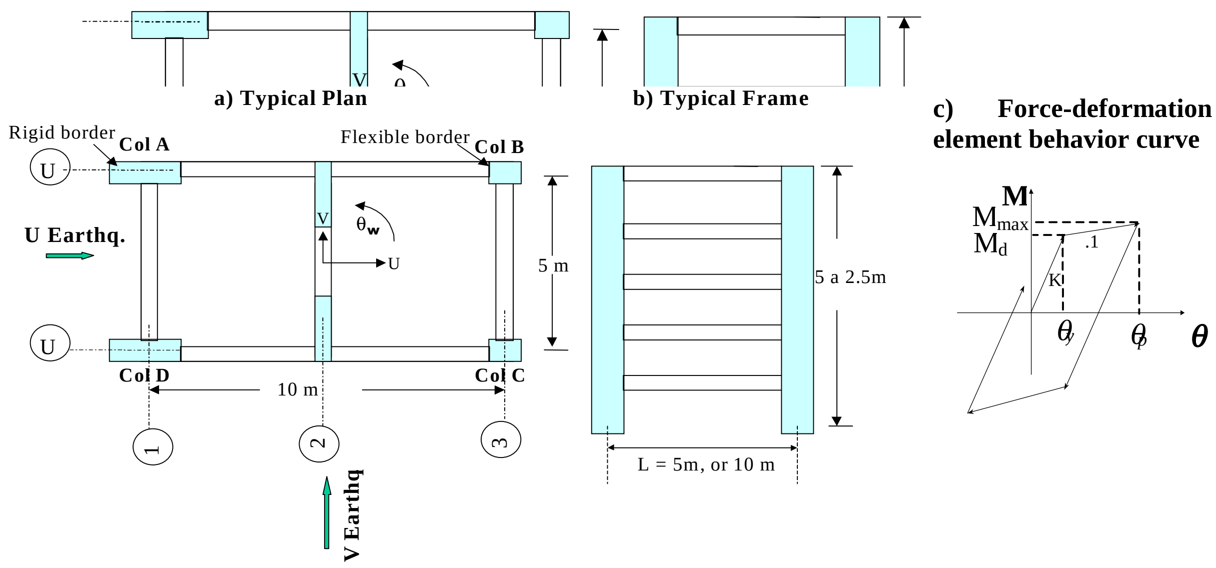
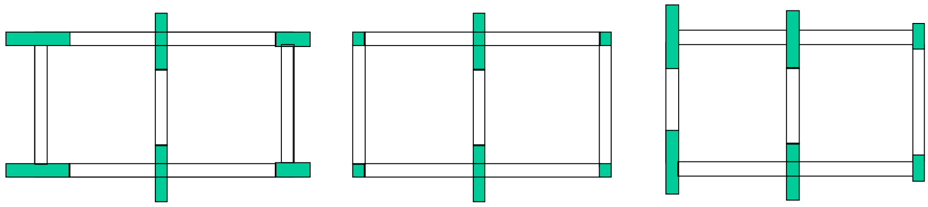


Figure 1: Basic model analyzed

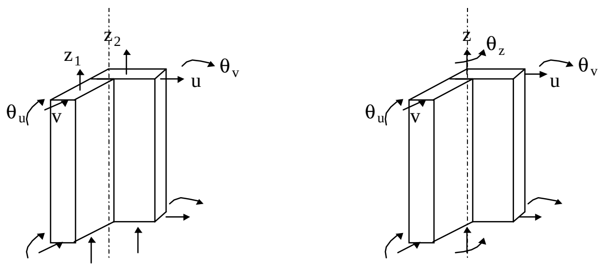


a)Similar transverse and lateral stiffness
Mod. 1,2,3,5,8,9,10,11,12,13,14,17,18

b)Low transverse and torsional stiffness
Mod.4,6,7,16

c)Low transverse stiffness
Mod. 15,19,20,21,22

Figure 2: Scheme of the analyzed plan



a) column in pseudo three dimensional model

b) column in three dimensional model

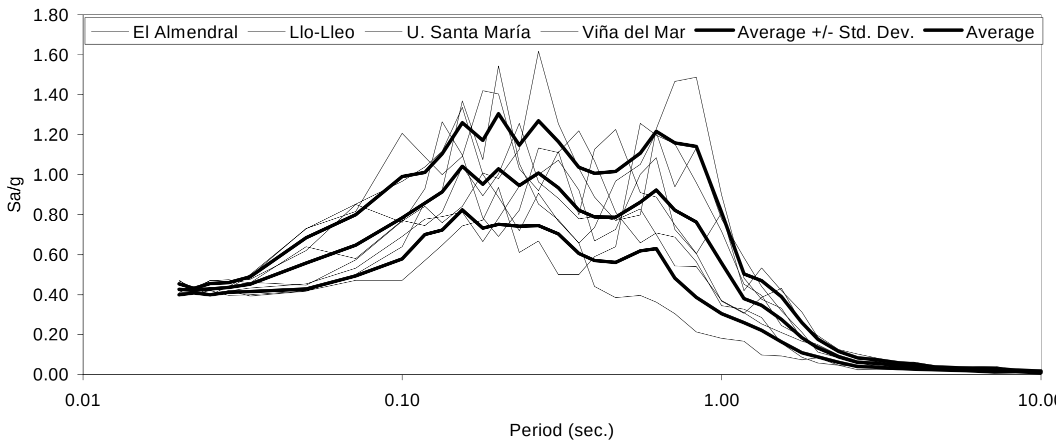
Figure 3: Corner column elements in pseudo-three-dimensional and in three-dimensional models

separated in two orthogonal elements, but, in the 3D model, they have one common axial degree of freedom, z.

EXTERNAL LOADS

When studying structures with elements that can behave non-linearly it is not possible to separate the gravitational loads effects from the seismic effects. In this work all the external loads that will be present during the occurrence of an important seismic ground movement which may result in a non-linear response have been defined. The effects of the dead and live loads of the structure are considered as an uniformly distributed load of w_g =1.0 T/m^2. The two horizontal components of four seismic events recorded in the earthquake occurred in the central region of Chile, March of 1985, have been selected. The elastic spectra of pseudo-acceleration of the used earthquakes are shown in figure 4. The records applied in the V direction have been normalized to a maximum acceleration of 0.4g.

Figure 4: Elastic pseudo-acceleration spectra of the Chile central earthquake, 1985. Peak acceleration normalized to 0.4 g. Damping coefficient = 5%.



In order to ensure that the comparative intensities of the two components are maintained as in the original records, the U components have been normalized with the same scale factor applied to the V component. The design criterion based on the recommendations of the Chilean Code for earthquake design [INN, 1996] is used; that is, the design strengths come from applying the earthquake loads independently in the two principal direction of the structure.

Interaction Between Bending Moments and Axial Force in the Non-linear Behavior

The interaction between the moments M_u and M_v is considered in the PC-ANSR program by the definition of two uni-axial interaction curves, M_u-N and M_v-N, in which the axial force N is constant for both curves (figure 5). Each interaction curve is generated by lines which intersect each other at the points of coordinates $\{M_{umax};\alpha N\}$ and $\{M_{vmax};\alpha N\}$ in the forces space, and the curves slopes are defined by the values of α and β. β is a fraction of M_{umax} and M_{vmax}. In the pseudo 3D model, each column, modeled as two plane elements, introduces M-N interaction curves with independent axial forces for each of the two elements (figure 6).

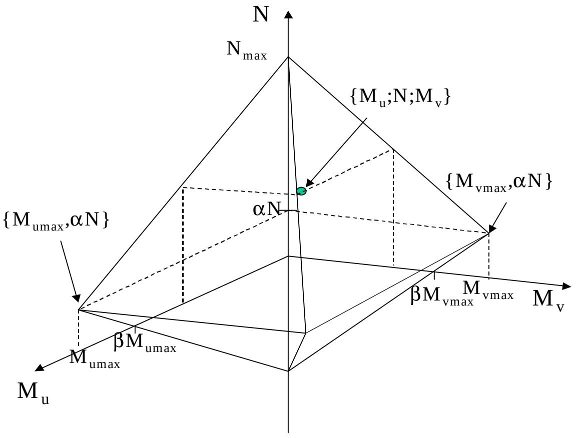


Figure 5: Bending moments-axial force interaction surfaces in three dimensional columns used in PC-ANSR program

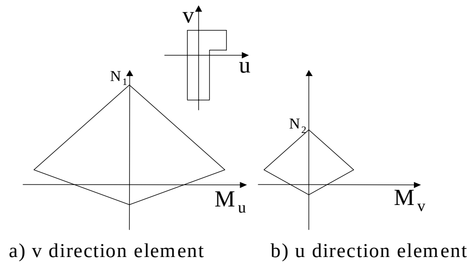


Figure 6: Bending moment-axial force interaction diagrams in corner column modeled as orthogonal plan elements

DESIGN STRENGTHS

To simplify the study and to avoid new parameters that could obscure the discussion of the results, a common level of design for all the analyzed models is used, so the value of the strength reduction factor R^* is considered as a constant. This value has to be adequately representative of the values used in Chile. The average of R^* for the models of different stiffness is considered, founded in soils of medium compacity (soil type II and III), resulting in $R^*=7$. The beams design moment, M_d, is the resulting from the analysis of the structure subjected to gravitational loads, M_g, added to the maximum elastic moment resulting from the analysis of the structure subjected to the seismic loads, M_{sel}, reduced by R^*, so, $M_d=M_g+M_{sel}/R^*$. This process is realized for the bending moments in the two directions, U and V. The maximum seismic elastic time history responses of axial force N_{sel} and bending moments M_{suel} and M_{svel} are evaluated to build the M_u-M_v-N interaction surfaces for the design of the columns. But, these maximum effects do not occur at the same time. To evaluate the most unfavorable combination of N, M_u and M_v in each column, the design strengths are defined based on the probable strengths that could have occurred at any instant of time. To illustrate the procedure in an uni-axial case, to build the N-M curve is necessary to evaluate the maximum and the minimum values of the N and M combinations, that is:

$$M_1 = M_g + M_{sel}/R^*; \qquad M_2 = M_g - M_{sel}/R^*; \qquad N_1 = N_g + N_{sel}/R^*; \qquad N_2 = N_g - N_{sel}/R^* \qquad (1)$$

and to choose the most unfavorable combination of the N-M pairs. The interaction surface is amplified by 25% in order to apply the "strong column-weak beam" concept. The slopes of the interaction curves are defined based on curves commonly used in the concrete column design practice.

ANALYSIS OF THE RESULTS

Average responses of maximum diaphragm displacements, forces and local rotation ductilities in each element end are analyzed. The average maximum combined (from gravitational and seismic loads) responses and maximum seismic responses are analyzed. In table 2 the periods, the transverse to torsional and the transverse to lateral frequencies ratio of the 3D systems and the differences between the periods of the structures modeled as pseudo-3D and as 3D systems are shown. It can be observed that the greater differences between the transverse and/or torsional periods are generated in buildings with very small transverse and/or torsional stiffness (ω_u/ω_v and/or $\omega_\theta/\omega_v < 0.85$). In general, the pseudo-3D model is transversally and torsionally more flexible compared to the 3D model. Only in eccentric structures with low transverse stiffness (models 15, 16, and 22) the lateral period is different when the building is analyzed by both models. Certain relationships between the differences in the evaluation of the potential and the input energy and of the periods are observed. Generally the elastic energies evaluated in pseudo-3D models are less than those evaluated in 3D models (table 3).

Table 2: Periods and vibration modes of the 3D model, transverse to lateral and torsional to lateral frequency ratios and differences between the periods evaluated in the pseudo-3D and in the 3D models (in percentage).

Model	Three-dimensional					Differences (%)		
	T_u	T_v	T_θ	ω_u/ω_v	ω_θ/ω_v	ΔT_u	ΔT_v	ΔT_θ
1	0.15	0.25	0.19	1.66	1.33	0.1	0.0	2.9
	0.15	0.74	0.19	4.87	3.87	0.0	0.0	2.2
	0.15	2.03	0.19	13.36	10.63	0.0	0.1	1.8
2	0.13	0.25	0.16	1.94	1.61	0.1	0.0	2.3
	0.13	0.76	0.16	5.82	4.78	0.0	0.1	1.8
3	0.12	0.25	0.15	2.17	1.73	0.0	0.0	1.8
	0.12	0.74	0.15	6.37	5.03	0.0	0.0	1.5
	0.12	2.03	0.15	17.50	13.81	0.0	0.1	1.2
4	3.40	0.25	1.24	**0.07**	**0.20**	**181.0**	0.0	**166.0**
	4.30	0.69	1.62	**0.16**	**0.43**	**124.0**	0.0	**103.0**
	5.46	1.99	2.14	**0.36**	**0.93**	**75.1**	0.1	**53.4**
5	0.19	0.25	0.23	1.35	1.10	0.3	0.0	3.0
	0.19	0.75	0.23	4.03	3.27	0.1	0.0	2.1
6	2.03	0.26	1.04	**0.13**	**0.25**	**43.4**	0.0	**64.7**
	2.26	0.74	1.27	**0.33**	**0.58**	**28.8**	0.0	**35.5**
	2.56	2.00	1.52	**0.78**	1.32	**13.9**	0.0	**13.4**
7	1.52	0.25	0.55	**0.16**	**0.45**	**25.8**	0.0	11.7
	1.71	0.76	0.59	**0.44**	1.28	**11.8**	0.0	4.3
8	0.22	0.26	0.27	1.19	0.97	0.1	0.0	4.6
	0.22	0.74	0.27	3.39	2.75	0.1	0.0	3.3
	0.22	2.08	0.27	9.54	7.68	0.0	0.1	2.5
9	0.18	0.25	0.22	1.37	1.15	0.1	0.2	3.6
	0.18	0.75	0.22	4.14	3.46	0.0	0.1	3.0
10	0.25	0.27	0.23	1.09	1.15	0.3	-10.0	**22.0**

Model	Three-dimensional					Differences (%)		
	T_u	T_v	T_θ	ω_u/ω_v	ω_θ/ω_v	ΔT_u	ΔT_v	ΔT_θ
11	0.28	0.24	0.30	0.88	0.82	0.5	0.1	7.7
	0.28	0.75	0.30	2.70	2.50	0.2	0.9	5.9
12	0.37	0.25	0.43	0.69	0.58	0.8	0.0	8.9
	0.39	0.74	0.44	1.91	1.67	0.3	0.2	5.7
	0.39	1.98	0.45	5.09	4.37	0.1	0.1	3.5
13	0.27	0.24	0.28	0.86	0.85	0.4	1.1	7.1
	0.28	0.76	0.27	2.75	2.84	0.1	2.0	6.0
14	0.23	0.26	0.23	1.15	1.12	0.6	-8.3	15.9
	0.23	0.75	0.25	3.28	3.03	0.2	0.5	4.8
15	0.68	0.27	0.16	0.40	1.70	1.4	11.4	3.8
	0.69	0.51	0.01	0.74	72.57	0.1	**37.2**	0.0
16	1.31	0.25	0.61	0.19	0.41	**20.4**	3.0	**14.9**
	1.41	0.86	0.56	0.61	1.55	11.9	**-85.0**	**63.7**
	1.58	2.02	0.68	1.28	2.98	0.0	1.6	-8.1
17	0.07	0.25	0.09	3.44	2.79	0.0	0.0	1.2
18	0.17	0.27	0.14	1.57	1.90	0.1	4.5	6.1
19	1.61	0.25	0.12	0.16	2.20	**20.6**	1.7	0.5
	1.83	0.75	0.12	0.41	6.27	5.8	4.2	0.4
20	1.61	0.25	0.11	0.16	2.22	**20.6**	2.5	0.6
	1.85	0.75	0.12	0.41	6.36	4.8	8.4	0.6
21	1.01	0.25	0.11	0.25	2.20	**12.2**	1.8	0.6
	1.10	0.75	0.12	0.68	6.23	3.1	4.5	0.6
22	0.81	0.26	0.14	0.33	1.94	10.1	**-86.0**	**98.7**
	0.86	0.75	0.21	0.87	3.54	3.8	4.6	**-30.0**

Table 3: Differences between elastic energies evaluated in the pseudo-3D and in the 3D model

Model	T_v	Kin.E.	Damp.E.	Pot. E.	Input E.
	sec	Δ (%)	Δ (%)	Δ (%)	Δ (%)
1	0.25	-0.1	0.0	0.0	-0.1
	0.74	0.0	0.0	0.0	0.0
	2.00	-0.4	-0.2	-0.4	-0.2
2	0.25	0.0	0.0	0.1	0.0
	0.76	0.5	-0.1	0.1	0.0
3	0.25	0.0	0.0	0.0	0.0
	0.74	0.0	0.0	0.0	0.0
	2.00	-0.4	-0.2	-0.5	-0.2
4	0.25	**-33.7**	**15.3**	**-200.0**	**-52.9**
	0.69	-0.1	10.7	**-119.7**	**-15.7**
	2.00	-4.2	3.8	**-59.6**	**-25.8**
5	0.25	-0.8	-0.1	0.0	-0.1
	0.74	0.1	-0.1	0.0	0.0
6	0.25	-0.8	2.1	**-34.9**	-3.7
	0.73	1.1	3.7	0.2	1.9
	2.00	0.5	11.0	-8.8	9.2
7	0.25	**20.1**	5.8	2.7	3.8
	0.76	2.2	1.3	0.7	0.9
8	0.25	-0.4	-0.2	-0.3	-0.2
	0.74	0.0	0.0	0.0	0.0
	2.10	-0.6	-0.1	-0.6	-0.1
9	0.25	0.1	-0.3	0.6	-0.3
	0.75	-0.7	0.1	0.0	0.1
10	0.27	-4.5	1.4	-8.2	1.5
	0.77	1.7	2.5	-3.0	2.5

Model	T_v	Kin.E.	Damp.E.	Pot. E.	Input E.
	sec	Δ (%)	Δ (%)	Δ (%)	Δ (%)
11	0.25	-0.5	0.1	-1.8	0.1
	0.75	-1.8	1.3	0.5	1.3
12	0.25	-1.5	-1.0	0.7	-1.0
	0.74	-0.1	0.0	0.6	0.0
	2.00	0.5	-0.2	-0.6	-0.2
13	0.24	-5.0	1.0	-3.0	1.0
	0.76	-3.8	2.0	-4.2	2.0
14	0.26	-4.1	1.1	-5.7	1.2
	0.75	-2.7	0.3	-0.1	0.4
15	0.27	**-15.1**	-7.3	**-11.5**	-7.2
	0.51	**-48.8**	-27.0	**-45.5**	**-27.0**
16	0.25	7.2	2.8	-4.7	1.6
	0.86	**13.4**	4.6	5.2	4.2
	2.00	4.4	0.5	5.0	0.4
17	0.25	0.1	0.0	0.0	0.0
18	0.27	**-24.3**	-1.0	**-17.2**	-1.0
19	0.25	**27.9**	8.4	**18.4**	6.5
	0.75	-9.2	3.3	-10.4	3.1
20	0.25	**27.2**	8.0	**19.1**	6.1
	0.75	-8.8	**12.3**	-3.0	**12.0**
21	0.25	5.2	**20.9**	-4.7	**20.3**
	0.75	-7.3	5.8	-5.8	5.8
22	0.26	7.6	**13.6**	**11.0**	**13.3**
	0.75	-2.7	10.6	2.2	10.6

In table 4 the differences between the elastic and inelastic maximum diaphragm displacements of some of the analyzed buildings, modeled as pseudo-3D and as 3D systems are shown. In structures with elastic behavior, the maximum lateral displacements, δ_v, vary according to the model utilized only in flexible systems with important transverse elements and are overestimated by the pseudo-3D model in at least 27% (model 1). The maximum transverse displacements δ_u, are generally underestimated by the pseudo-3D model especially in systems with large transverse elements with differences of up to 34%. The maximum elastic diaphragm rotations are estimated with differences of up to 47% by both models of analysis. None of the parameters shows noticeable influence in these differences. When analyzing the maximum displacements that occur in buildings with non-linear behavior, certain correspondence between the differences observed in the lateral and transverse elastic displacements of rigid and moderately flexible systems are observed. In flexible buildings the differences between the maximum lateral inelastic displacements grow up to 105%. The maximum inelastic rotations of the diaphragm of eccentric and flexible buildings, with important transverse elements, such as model 9 for instance, turns to be up to 244% different for both analysis models. It is important to note that models 4 and 6, characterized by having extraordinarily slender corner columns and fulfilling the plant and height regularity and the displacements requirements established in the Chilean Code, when entering in the inelastic behavior range, show very large lateral displacements, that could be associated with the "collapse" of the structure. The exception is the flexible building of model 6, the only one that reveals a torsional to lateral frequency ratio larger than 1. The differences between the elastic combined axial force in columns are generally quite low. The differences between the elastic seismic axial force in columns are greater than the differences between combined axial forces (table 5).

Table 4: Maximum average differences between maximum seismic elastic and inelastic displacements of the diaphragm resulting from pseudo-3D and from 3D models (in percentage).

	Model																	
	1						4						6					
I_1/I_3=1	Elastic			Inelastic			Elastic			Inelastic			Elastic			Inelastic		
T_v	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$
0.25	-22.8	-0.3	-37.0	-16.5	-0.7	-16.7	5.0	4.4	47.5				5.4	0.0	2.7			
0.75	-30.3	-0.8	-29.6	-9.1	1.4	-24.3	5.0	4.3	0.0		collapsed		-3.1	-0.2	-4.3		collapsed	
2.00	-34.3	27.3	45.0	-8.3	22.5	-15.7	5.0	-0.7	-0.5				-13.0	2.0	-4.3	-13.0	-80.4	-4.3

	Model																	
	8						9						16					
I_1/I_3=10	Elastic			Inelastic			Elastic			Inelastic			Elastic			Inelastic		
T_v	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$
0.25	-14.7	0.0	7.5	18.2	-4.0	-19.9	-24.5	-0.2	-22.3	-10.4	4.0	-19.3	1.9	-0.3	1.8	-8.1	-1.0	8.0
0.75	-20.5	16.5	-10.3	10.4	5.8	-21.5	-16.7	-0.7	-11.5	-10.4	-9.2	244						
2.00	-22.0	-1.0	-32.5	-31.3	105	-50.2												

	Model																	
	17						19						20					
I_1/I_3=100	Elastic			Inelastic			Elastic			Inelastic			Elastic			Inelastic		
T_v	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$	$\Delta\delta_U$	$\Delta\delta_V$	$\Delta\theta$
0.25	-30.6	0.1	-33.6	-21.0	1.0	-28.3	-4.6	4.0	-12.7	-20.3	-2.5	-3.3	21.9	3.0	-9.8	39.9	-2.6	-4.3
0.75							-1.0	-1.1	1.9	-6.4	-1.9	-3.2	0.5	-9.7	-4.4	-20.7	-20.6	-29.3

An important increase is observed in eccentric and also in symmetric structures with slender corner columns (models 4 and 6). In eccentric models with big uncoupled transverse elements (model 8), the differences are extraordinarily large, but the values concerned are small. In buildings with inelastic behavior the differences in the axial seismic forces determined by the two models generally decrease, except in the columns of the rigid border of buildings that are eccentric and with important transverse stiffness. The design moments for each element determined by pseudo-3D or by 3D models are generally similar. Only in a few number of cases the maximum differences vary between -38% and +32%. It occurs in elements with small earthquake induced forces. Similarly the maximum differences in the determination of the design axial forces vary from –32% to +30%. Big differences are observed in the evaluation of the maximum local rotation ductility required by beams and columns depending on whether the building analysis considers the 3D behavior of the elements (table 6). In flexible structures the differences are important, especially in eccentric structures. The greater differences in the local ductility in columns are produced in those located in the flexible border. Generally there is no clear evidence regarding the reason why the increase of ductility in any model in particular is greater than in another. However, it could be said that in rigid buildings the pseudo-3D model overestimates the local rotation ductility, and in flexible buildings this model underestimates it. The differences vary between -82% and +98%. In the transverse elements important local rotation ductilities are generated when structures are subjected to bi-directional earthquakes, implicating a global non-linear behavior with the consequent decrease of the building stiffness in both directions. This effect modifies the inelastic torsional structure behavior.

Table 5: Average differences between the maximum combined and seismic elastic axial forces in columns resulting from pseudo-3D and 3D models (in percentage).

	Model											
	1				4				6			
I_1/I_3=1	Combined		Seismic		Combined		Seismic		Combined		Seismic	
T_v	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord
0.25	0.7	0.6	5.0	4.1	0.9	1.0	68.6	75.3	3.4	3.5	88.0	107.6
0.75	-0.3	-0.9	-1.8	-4.4	1.8	2.5	26.6	42.6	5.4	5.8	27.5	31.7
2.0	4.3	3.4	17.9	13.8	3.4	4.8	39.9	50.3	10.8	8.2	36.6	27.0

	Model											
	8				9				16			
I_1/I_3=10	Combined		Seismic		Combined		Seismic		Combined		Seismic	
T_v	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord
0.25	7.6	-6.1	654.7	-45.3	7.9	5.9	32.0	7.8	10.1	8	31.8	83.5
0.75	7.1	3.8	39.3	-21.8	3.4	31.1	6.9	61.9				
2.0	5.8	5.0	20.5	18.6								

	Model											
	17				19				20			
I_1/I_3=100	Combined		Seismic		Combined		Seismic		Combined		Seismic	
T_v	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord	Rig.Bord.	Flex.Bord
0.25	-5.6	53.7	12.8	11.4	-6.7	7	-11.5	38.1	-6.3	7.7	-10.4	37.6
0.75					-9.6	10.2	-14.0	18.7	-12.4	13.2	-16.8	26.7

Table 6: Maximum average local ductility of rotation required in the base of first story columns and in the beams. Differences between pseudo-3D and 3D models (in percentage).

	Model																	
	1									8								
Period [sec]	0.25			0.75			2			0.25			0.75			2		
Columns	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)
Rig. Border	3.9	4.7	21.6	4.9	4.4	-11.3	11	4.1	-61.9	4.9	4.7	-2.4	4.3	4.5	3.7	10	4.1	-60.7
Flex. Border	3.5	4.3	21.0	2.6	4.0	52.0	7.5	3.9	-48.4	2.5	4.2	69.3	3.3	4.2	27.5	6.7	4.1	-38.0

	Model																	
	9						16			17			19					
Period [sec]	0.25			0.75			0.25			0.25			0.25			0.75		
Columns	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)
Rig. Border	5.3	5.1	-4.3	6.5	5.8	-10.7	9.1	10.0	10.4	6.1	8.2	35.0	9.4	10.5	11.6	10.5	10.4	-1.0
Flex. Border	22.8	20.3	-11.1	19.4	20.0	2.9	6.7	9.1	35.9	7.0	6.9	-1.8	5.1	6.4	25.6	3.9	6.4	62.9

	Model																	
	1									8								
Period [sec]	0.25			0.75			2			0.25			0.75			2		
Beams	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)
Rig. Border	10.2	10.2	0.1	8.9	10.0	12.3	19.5	9.1	-53.3	10.1	9.6	-5.5	4.8	7.9	65.7	20.7	6.3	-69.6
Flex. Border	5.2	5.9	12.2	6.0	6.2	3.8	17.8	6.0	-66.1	5.4	5.4	-0.1	4.0	5.0	25.8	23.9	4.4	-81.6
U Frame	5.4	5.1	-6.4	4.2	4.7	12.2	10.9	4.4	-59.4	0.9	0.3	-72.2	0.6	0.3	-53.2	1.2	0.3	-79.1
Central	11.3	11.2	-0.6	9.3	10.8	15.9	19.7	9.8	-50.3	11.4	10.7	-5.7	4.7	8.7	87.0	21.3	6.8	-68.3

	Model																	
	9						16			17			19					
Period [sec]	0.25			0.75			0.25			0.25			0.25			0.75		
Beams	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)	$\mu_{\theta\,tri}$	$\mu_{\theta\,pse}$	Δ (%)
Rig. Border	10.0	11.1	10.5	0.3	9.3	2851	9.9	13.6	36.5	12.4	14.2	14.6	14.0	16.4	17.7	10.6	15.9	49.6
Flex. Border	4.4	4.7	7.1	4.4	4.7	6.7	4.4	6.5	47.0	10.6	3.3	-68.6	13.3	12.9	-2.7	7.1	10.6	50.1
U Frame	9.9	10.3	4.9	4.9	8.6	74.6	11.8	11.8	-0.6	10.5	10.7	1.6	13.5	12.8	-5.5	5.1	10.1	98.3
Central	3.7	4.5	19.5	3.3	4.4	34.1	2.1	3.2	49.8	1.8	1.8	-0.8	3.2	2.8	-11.1	3.6	3.0	-17.2

CONCLUSIONS

In a structural project the purpose of the evaluation of the structural system responses is often to define the design forces which ensure an adequate behavior of the building when subjected to all probable loading conditions which will occur during its service life. Although the available computing tools are becoming more powerful every day and this allows to improve the models of analysis, it is still frequent to use simplified models for the determination of the building responses. On the other hand, the use of simplified models diminishes the probability of error in the input data and in the solution process due to less uncertainties. There are some buildings that are sensitive to the analysis models. Some important conclusions regarding the comments above have been obtained, which are presented in the following paragraphs. In buildings with very small transverse and torsional stiffness the pseudo-3D analysis model estimates very different transverse and/or torsional periods compared to those evaluated by using the 3D analysis model. Generally the building represented by a pseudo-3D model is more flexible transversally and torsionally that the same building represented by a 3D model. In eccentric and transversally flexible structures the lateral period is different when evaluated by both models. There is certain relationship between the differences observed in the evaluation of the potential and the input elastic energies, and those detected in the evaluation of the periods by the two analysis models studied. The elastic and inelastic required strengths resulting from combined gravitational and earthquake loads normally are not different when analyzed by the pseudo-3D model or by the 3D model. This is relevant for practical purposes, because the building will be always subjected to these combined loads. The maximum elastic and inelastic seismic axial strengths required are very different depending on the analysis model used to describe the structure. This is especially notorious in buildings with slender corner columns, as well as in eccentric buildings with big transverse elements. The design strengths are quite similar in buildings represented by the two models. The maximum seismic lateral diaphragm displacements are generally similar when evaluated by the two models of analysis. The maximum seismic transverse displacements of the diaphragm show greater differences. The maximum seismic rotation of the diaphragm is different when evaluated by both models, and normally it is underestimated by the pseudo-3D model. Generally all the structures are sensitive to the model when evaluating the maximum local rotation ductility, being underestimated in the elements of flexible buildings and overestimated in the elements of rigid buildings by the pseudo-3D model.

As a general conclusion it could be said that to obtaining useful data to define the design strengths, the model of analysis is not very relevant, as well as for the estimation of the combined forces that the elements of a structure are subjected to. However, for analysis in which one is looking for only the seismic effects, and especially if the structure behaves non-linearly, the model of analysis is relevant to describe appropriately the structure in a large number of cases.

ACKNOWLEDGEMENTS

The research reported here has been funded by FONDECYT, the Chilean National Fund for Development of Science and Technology through their financial support of Project N° 1970711, which is gratefully acknowledged.

REFERENCES

Blume, J. A., (1968), "Dynamic Characteristics of Multistory Buildings", *Journal of Structural Engineering ASCE* Vol. 94, NST2, pp337-402.

García E., Riddell R., (1995), "Espectro de Energía Disipada por Histéresis en Sistemas Inelásticos Sometidos a Terremotos", *Publicación DIE Nº95-3,* Pontificia Universidad Católica de Chile, Escuela de Ingeniería, Departamento de Ingeniería Estructural y Geotécnica.

Huria V., Raghavendrachar M., Aktan A.E., (1991), "3-D Characteristics of RC Wall Response". *Journal of Structural Engineering, ASCE 117*, N° 10, pp3149-3167.

Maison B., (1992), "PC-ANSR A Computer Program for Nonlinear Structural Analysis", *Report Nº UCB/EERC***,** Earthquake Engineering Research Center, College of Engineering, University of California. Berkeley, California.

Norma Chilena Oficial NCh433.Of96, (1996), "'Diseño Sísmico de Edificios"**,** *Instituto Nacional de Normalización.*

Rihai Ali, Powell Graham H., Mondkar Digambar P., (1979), "3D Beam-Column Element (Type 2-Parallel Element Theory) For The ANSR -II Program", *Report Nº UCB/EERC-79/31* Earthquake Engineering Research Center, College of Engineering, University of California. Berkeley, California, December