THREE-DIMENSIONAL BUILDINGS SUBJECTED TO BI-DIRECTIONAL EARTHQUAKES. VALIDITY OF ANALYSIS CONSIDERING UNI-DIRECTIONAL EARTHQUAKES

Ernesto F CRUZ[1] And Silvana COMINETTI[2]

SUMMARY

The elastic and inelastic maximum responses of five story concrete buildings of different structural configurations of plant stiffness eccentricity, lateral stiffness, coupling and slenderness of the corner columns are studied. The buildings are subjected to gravitational and seismic loads. Earthquake action is considered with only the larger peak acceleration component acting in one direction (uni-directional earthquake) and with the two components acting in the lateral and in the transverse directions (bi-directional earthquake). The maximum response resulting from a time history analysis of the buildings is studied. The non-linear model considers the formation of a plastic hinge in each node of the elements. The interaction between the bending moments and the axial force in the yield surfaces is considered. The differences between the responses obtained from the uni- and the bi-directional earthquakes are evaluated. The design strengths determined by the Chilean design practice in which the seismic design spectra is considered acting independently in the two mean directions of the building are compared with those resulting from considering the two horizontal components of the earthquake acting at the same time. It is observed that there are no great differences between them. The differences between the responses of buildings subjected to uni- and to bi-directional horizontal loads combined with gravitational loads are smaller than the differences of responses of those subjected to just seismic uni- and bi-directional loads. The columns seismic axial strength is smaller in buildings subjected to uni-directional earthquakes in regard to those of buildings subjected to bi-directional earthquakes. These differences vary if the building behavior goes into the non-linear range. The maximum lateral displacements of the diaphragm depend mainly on the horizontal ground movement acting in that direction. The elastic torsional behavior of the building is not well estimated by an uni-directional earthquake, and the estimate is worse when estimates the inelastic torsional behavior of the building.

INTRODUCTION

A common practice in structural building design is to use the seismic response originated by ground movement acting separately in the two orthogonal directions of the building. Particularly in Chile the complete structure subjected to uni-directional earthquakes is analyzed. In this analysis a significant contribution to the building torsional stiffness due to the traverse frames appears. An additional contribution to the lateral building stiffness arises when the three-dimensional stiffness of the elements is considered. However, the fact that the earthquake has indeed an arbitrary direction, represented by a bi-directional ground movement, could reduce the contribution of the traverse frames to the building torsional and lateral stiffness. This is particularly notorious if the earthquake causes non-linear behavior in the transverse frames, producing a significant modification of the inelastic torsional behavior of the building. Another effect of particular interest is related to the magnitude of the axial force acting in the corner columns, which is different in structures subjected to bi-directional ground movements compared to the forces in corner columns of structures subjected to uni-directional earthquake. It is of interest to evaluate these differences in the elastic and inelastic behavior. Thus this evaluation allows to validate the adequacy of the seismic analysis considering the earthquake acting in one direction for design

[1] *Dept. Ingeniería Estructural y Geotécnica, Pontificia Universidad Católica de Chile. Santiago, Chile. ecruz@ ing.puc.cl*
[2] *Dept. Ingeniería en Obras Civiles, Universidad de Santiago de Chile. Av. Ecuador Santiago, Chile*

purposes, moreover the knowledge of either the global building seismic responses and the local seismic responses of particular elements can be increased. Several investigations show few agreements on the conclusions concerning the influence of the transverse resistant elements to the earthquake direction. According to [Goel and Chopra, 1970] the incorporation of the transverse elements in the building model subjected to an uni-directional earthquake affects significantly the response. [Tso and Zhu, 1992] outline that when a structure is subjected to two simultaneous horizontal earthquake components, the transverse element behavior can be non-linear, therefore their contribution to the real torsional stiffness is smaller. [Correnza and Hutchinson, 1994] analyzed one-story models with and without transverse elements subjected to uni- and to bi-directional earthquakes. They concluded that the inclusion of the transverse elements in the model affects significantly the response of the border elements when the structure is subjected to the two earthquake components. [Hisada et al., 1988] studied the orthogonal effects on elastic structures varying the characteristics of the soil foundation. They observed maximum response values 1.3 to 1.7 times larger than the values observed in buildings subjected to uni-directional earthquakes. In the axial corner column, force values got up to 2 times higher. Generally, studies in which the gravitational loads, the seismic loads and the bi-axial effects have been included in the analysis refer to a particular element and not to a complete structure. However there are studies in which the experimental response of concrete frames with rectangular columns subjected to horizontal bi-directional ground movements that make the structure behave non-linearly is analyzed [Oliva and Clough, 1987]. Several three-dimensional five stories buildings subjected to earthquakes acting in one and in two directions are analyzed in this work. The differences between the responses of systems subjected to uni-directional and to bi-directional earthquakes are evaluated. One of the objectives of this work is to determine the validity of the design practice based on an uni-directional seismic analysis.

METHODOLOGY

Analized Models

A three-dimensional frame structure subjected to two horizontal components of seismic ground movements acting in the principal directions of the building is analyzed. The elements of the frames provide lateral stiffness and resistance in the plane in which they are located, as well as in their transverse direction (figure 2). The beams are considered as elements that are axially rigid. The structural model considers masses concentrated in each story. The diaphragms are modeled as rigid elements. The buildings are studied considering the monolithic behavior of the structure, including the vertical deformations of the corner column elements. The ends of the elements are subjected to bi-axial bending moments, M_u and M_v, acting simultaneously with the axial load N and the torsion T. The parameters that define the behavior of the analyzed buildings are detailed later. Symmetric models were selected with either medium or large stiffness eccentricity, and this was achieved by modifying the ratio between the inertia of the elements of frame 1 and the inertia of the elements of frame 3, I_1/I_3 (figure 1).

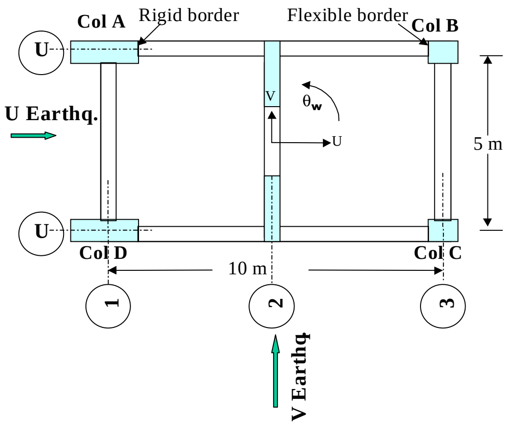


Figure 1: Basic model analyzed

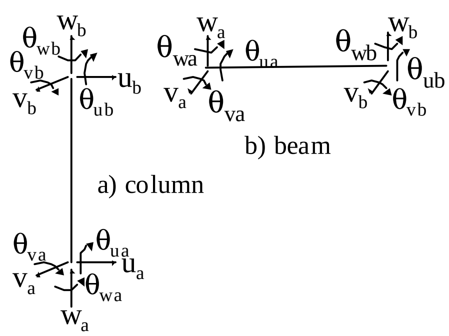
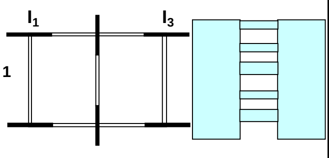
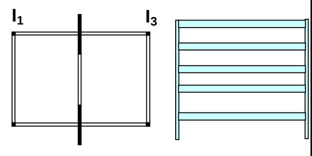
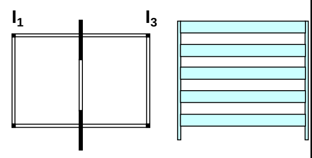
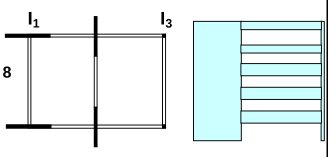
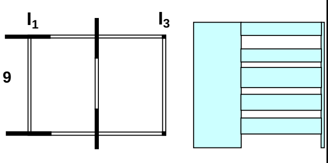
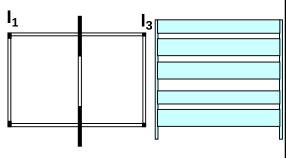
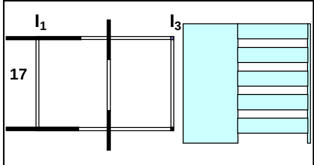
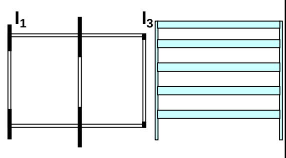
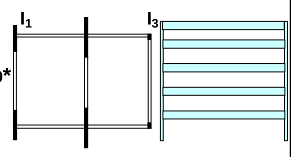


Figure 2: Used elements

The ratio between the beam stiffness and the column stiffness of the frames in the U direction, defined as "beam-column stiffness ratio ρ_u", that measures the level of coupling between the U frames columns, was an additional parameter selected to describe the structures. Models with different lateral stiffness, measured by the fundamental elastic lateral period, T_v, and different torsional and transverse stiffness, the one which was measured through the torsional-lateral frequencies ratio $\Omega_{\theta v}$, and the transversal-lateral frequencies ratio Ω_{uv} were analyzed. The models analyzed and their parameters are summarized in table 1.

Table 1: Summary of the parameters of the analyzed models

Model	Plan and U Frames Elevation	I_1/I_3	ρ_u $\Sigma K_v/\Sigma K_c$	T_v sec	$\Omega_{\theta v}$ ω_θ/ω_v	Ω_{uv} ω_u/ω_v
1		1	0.001	0.25 0.75 2	1.33 3.88 10.6	1.66 4.87 13.36
4		1	0.25	0.25 0.75 2	0.21 0.43 0.93	0.07 0.16 0.36
6		1	25	0.25 0.75 2	0.24 0.58 1.32	0.12 0.32 0.78
8		10	0.0025	0.25 0.75 2	0.95 2.75 7.68	1.16 3.4 9.56
9		10	0.025	0.25 0.75	1.16 3.45	1.37 4.14
16		10	100	0.25	0.41	0.19
17		100	0.025	0.25	2.78	3.44
19		100	0.25	0.25 0.75	2.21 6.25	0.16 0.41
20*		100	0.25	0.25 0.75	2.22 6.36	0.16 0.41

Note: * In models 19 and 20 only the beams width are different

External Loads

Buildings subjected to gravitational and seismic loads are studied. Four sets of horizontal acceleration records obtained at the earthquake that occurred in the central region of Chile in March of 1985 have been selected [Cominetti and Cruz, 2000]. The components with the maximum acceleration peak, normalized to 0.4g, are applied in the V direction. In the analysis of buildings subjected to a bi-directional earthquake, the two horizontal components of each acceleration record set are used. To ensure that the relative intensities of both components of the original record are maintained, the U acceleration values are normalized with the same scale factor applied to the corresponding V component. The response of buildings subjected to uni-directional earthquakes is obtained by applying with maximum acceleration of the component in the V direction.

Non-linear Model

The non-linear model analysis allows the response in each time step to be known considering the stiffness variations experienced when some critical sections yield. The structure is solved with the non-linear dynamic analysis of three-dimensional structures program, PC-ANSR [Mondkar and Powell, 1979]. The force-deformation behavior curve is bi-linear with a loss of 90% of stiffness in the second branch. Interaction surface of bending moments M_u and M_v and axial force N is considered.

Design Strengths

The design strengths are determined combining the strength resulting from gravitational loads, M_g and N_g, with the average of the maximum elastic strengths resulting from seismic excitations, M_{es} and N_{es} reduced by a reduction strength factor R^* =7 (refer to [Cominetti and Cruz, 2000] for details). This R^* value is considered as representative in the Chilean design practice for ductile concrete structures of medium period and founded on a medium compacity soil.

Design Criteria

Two basic design criteria of the structure are used and these are shown schematically in figure 3. The first is related to the considerations that usually are carried out in the Chilean design practice. The elements are designed with the maximum strengths resulting from the application of the larger earthquake component, S_1, in the two orthogonal directions, independently. The second criteria, consists in designing the elements with the maximum elastic strengths resulting from the application of the seism with both components S_1 and S_2 acting simultaneously, and interchanging the components. Both criteria require to solve the structure excited by two seismic load systems.

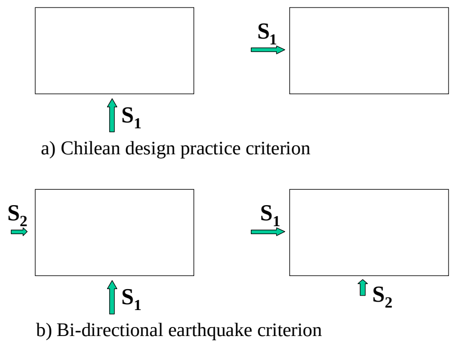


Figure 3: Design criteria

ANALYSIS OF THE RESULTS

Maximum Diaphragm Displacements

The observed values of the lateral displacements of buildings with elastic behavior, δ_v, generally do not vary according to the seismic loads model. The maximum difference between lateral diaphragm displacements resulting from uni- and bi-directional earthquake was 18%. Notorious differences in the torsional elastic behavior under the two types of excitation are not observed. This indicates that the uni-directional excitation evaluates adequately the elastic torsional behavior of the eccentric models studied. Regarding the analyses of buildings with non-linear behavior, the maximum observed displacements of the models 4 and 6 exceed the maximum expected displacements before finishing the applied ground acceleration records, which might would indicate a collapse of the structure. This occurs in the one and in the two direction earthquakes, with exception of the most flexible building with characteristics corresponding to model 6. It can be observed that these models have very small values of torsional-lateral and transverse-lateral frequency ratios, indicating that they are very flexible buildings both torsional and transversely. A very low redundancy level is observed, and the complete seismic shear is practically taken by the central frame. In the laterally flexible building corresponding to model 6, a more balanced distribution of the seismic shear forces is produced because of the small dimensions of the central frame, similar to those of the extreme frames. Aside from these elastic parameters, no other parameters seem to indicate the probability of developing such big displacements under important seismic ground movement. As observed in table 2 the average base shear values resulting from the excitations applied on the rigid and on the semi-rigid building models are higher than the minimum base shear force recommended by the Chilean seismic Code, [Norma Chilena Oficial, 1996]. Observing models 4 and 6, it can be said that they are symmetric and regular in height and in plan, and that the maximum drift conditions recommended by the Chilean code are fulfilled. These observations indicate that this type of building would be feasible of being constructed. It is then important to limit also the torsional flexibility of the buildings and their redundancy level. It is interesting to observe that in model 16, with frequency ratios similar to those of models 4 and 6, the building does not collapse when subjected to seismic events that carry it to non-linear behavior. The structures corresponding to model 16 present a high level of coupling the columns in the U direction (ρ_u=100). This makes the columns to experience smaller maximum displacements. We can conclude that buildings that have small torsional-lateral and transverse-lateral frequency ratios, low redundancy level, and low coupling level of the columns at the same time, have high risk of failure in important seismic events. Important non-linear rotations in symmetric buildings appear, and in eccentric buildings these generally diminish with respect to those with elastic behavior. This decrease is more significant in the uni-directional excitation case than in the bi-directional case. This is the reason for the substantial increase when observing the differences in the non-linear torsional behavior of buildings subjected to uni- and to bi-directional earthquakes (table 3). It can be concluded that the elastic torsional behavior is similar when buildings are subjected to uni- and to bi-directional seismic movements. However the inelastic torsional behavior turns out to be very different when the structure is excited by uni- or by bi-directional earthquakes, indicating the possible incursion of the transverse frames elements in the inelastic behavior range. This effect in addition to diminishing the transverse stiffness causes the torsional stiffness contributed by the transverse elements to decrease considerably. The elastic and the inelastic lateral behavior of the buildings is quite similar when subjected to either uni- or bi-directional earthquakes. However, great

Table 2: Average of base shear force applied (Chilean Code V_{0max} = 17.6 Ton)

Model	1			4			6			8		
Period [sec]	0.25	0.75	2.0	0.25	0.75	2.0	0.25	0.75	2.0	0.25	0.75	2.0
Bi-dir. V-U	28.9	23.8	11.9	24.5	23.5	11.7	24.3	24.0	11.3	24.5	20.7	11.9
Uni-dir. V	24.6	23.8	11.9	24.5	23.5	11.6	24.3	24.0	11.2	24.5	23.7	11.9

Model	9			16			17			19			20		
Period [sec]	0.25	0.75	2.0	0.25	0.75	2.0	0.25	0.75	2.0	0.25	0.75	2.0	0.25	0.75	2.0
Bi-dir. V-U	24.1	22.3	-	24.1	-	-	24.6	-	-	32.2	29.2	-	32.1	27.8	-
Uni-dir. V	24.1	22.3	-	25.7	-	-	24.6	-	-	32.2	29.1	-	32.1	27.8	-

Table 3: Maximum average differences between maximum seismic non-linear displacements of the diaphragm resulting from uni- and bi-directional earthquakes (in percentage)

	$\Delta\delta_V$ (%)							$\Delta\theta$ (%)						
Period \ Model	1	8	9	16	17	19	20	1	8	9	16	17	19	20
0.25	0.0	15.6	0.3	-71.6	0.1	41.5	0.2	-65.2	-14.9	-38.9	-79.2	-13.5	-99.7	0.2
0.75	0.0	-35.6	0.1	-	-	-20.9	1.3	-65.3	-54.3	-51.8	-	-	-99.8	-0.5
2.00	-0.3	-0.7	-	-	-	-	-	-60.8	-63.7	-	-	-	-	-

differences between the inelastic lateral behavior due to these types of seismic excitations are observed in buildings that are eccentric, rigid, and with small values of Ω_{uv} (models 16 and 19).

Axial Forces in Columns

In table 4 the maximum average differences between the maximum axial forces in corner columns, resulting from the application of gravitational and seismic loads (combined loads) and of seismic loads alone, regarding uni-directional versus bi-directional earthquake are shown. Observations in buildings with elastic and inelastic behavior have shown that usually the differences in elastic combined axial loads resulting from uni-directional and those resulting from bi-directional earthquakes are small. However, in eccentric buildings these differences increase and elastic combined axial forces of the columns located in the flexible border can be underestimated in up to 34% for the uni-directional earthquake. Tipically the lateral flexibility of the building does not affect these differences substantially. The evaluation of the elastic seismic axial forces in columns of buildings subjected to uni-directional and to bi-directional earthquakes shows, of course, more significant differences than the combined elastic axial forces, since the seismic axial force is only a fraction of the total force. It is important to mention that the evaluation of systems excited only by earthquakes could lead to conclusions that do not reflect the real situations that building normally meet. The differences in seismic axial forces get to maximum values of -95%. An exceptional case is observed for the column located at the rigid border of the rigid building in model 8. The big difference is produced for an uncoupled element in the U direction (ρ_u=0.0025), which behaves very differently under the action of the two types of ground movements exciting the structure. In both cases the seismic axial force is very low. There is no evidence of the influence of the building eccentricity on the difference between the seismic axial force of the building columns, subjected to different earthquake. It can be observed that when entering in the non-linear behavior range, the differences in certain cases decrease and in others increase. It indicates that the inelastic behavior is different from the elastic behavior when the building is subjected to the two types of earthquake. The increases in the maximum inelastic seismic axial forces are often extraordinarily big, and sometimes the axial forces resulting from an uni-directional analysis are bigger than those resulting from bi-directional analysis. It is possible that the bi-directional earthquake produces a compensation effect in the maximum responses of columns subjected to large positive forces when an uni-directional earthquake is applied. The transverse component of bi-directional earthquake generates compression forces, which decreases the column seismic axial force.

Design Strengths Comparison

Although the strengths in each section of the elements are different in structures subjected to bi-directional or to uni-directional earthquakes, the determination of the design strengths requires a search procedure of the maximum strength in each element to be conducted. In the Chilean practice this procedure is performed exciting the whole structure by an uni-directional design earthquake in each principal direction. If the design strengths obtained through this procedure are compared to the design strengths obtained from a bi-directional earthquake criterion analysis of the structure, generally the strength values for both procedures are very similar (table 5). The bigger difference detected in the element design moments is of –24.9% in the beams of the frame located in the flexible border of a semi rigid and very eccentric and with very small transverse elements model (model 20).

In this model a difference of -20% in the axial design strength of the column located in the rigid border is also detected. In general, the differences are higher in eccentric than in symmetric buildings. It can be concluded that the criterion used in the practice for Chilean seismic design is adequate. This conclusion is subjected to the assumption that the most unfavorable earthquake direction coincides with the principal axes of the building plant. In the present analysis the effects of the seismic ground movement incidence angle have not been studied,

Table 4: Maximum average differences between maximum elastic and inelastic axial forces resulting from uni- and bi-directional earthquakes (in percentage)

	Elastic Forces																	
	Combined (Gravitational + Seismic)									Seismic								
	Δ (%) Columns in the Rigid Border																	
Model / Period	1	4	6	8	9	16	17	19	20	1	4	6	8	9	16	17	19	20
0.25	-13.8	-1.2	-1.3	-5.9	-15.1	-5.0	0.4	-0.2	-0.4	-91.6	-60.5	-30.8	210.2	-48.6	-19.4	-0.2	-0.2	-0.7
0.75	-18.5	-1.4	-5.4	-1.4	-8.7	-	-	0.3	1.5	-88.5	-18.2	-27.4	-8.4	-13.5	-	-	0.5	2.2
2.00	-16.7	-1.7	1.2	-6.7	-	-	-	-	-	-60.2	-14.4	4.1	-23.1	-	-	-	-	-
	Δ (%) Columns in the Flexible Border																	
Model / Period	1	4	6	8	9	16	17	19	20	1	4	6	8	9	16	17	19	20
0.25	-14.8	-1.2	-1.1	-8.8	-28.5	-8.9	-20.7	-1.7	-2.0	-94.5	-60.5	-28.0	-52.8	-70.9	-78.0	-59.2	-8.8	-10.6
0.75	-19.3	-0.7	-4.2	-13.4	-33.8	-	-	-3.0	-2.0	-91.4	-11.1	-21.8	-91.9	-75.3	-	-	-4.9	-1.9
2.00	-19.6	-1.1	-2.4	-18.0	-	-	-	-	-	-71.2	-11.8	-9.0	-92.0	-	-	-	-	-
	Non-linear Forces																	
	Combined (Gravitational + Seismic)									Seismic								
	Δ (%) Columns in the Rigid Border																	
Model / Period	1	4	6	8	9	16	17	19	20	1	4	6	8	9	16	17	19	20
0.25	-3.7	-	-	-3.9	24.6	-2.3	5.1	57.1	-0.6	-89.0	-	-	369.6	49.6	-27.4	61.1	232.9	-67.1
0.75	-4.6	-	-	3.1	32.7	-	-	74.0	-0.2	-84.3	-	-	276.2	26.0	-	-	188.7	-49.2
2.00	-3.8	-	-	-1.0	-	-	-	-	-	-29.1	-	-	-6.8	-	-	-	-	-
	Δ (%) Columns in the Flexible Border																	
Model / Period	1	4	6	8	9	16	17	19	20	1	4	6	8	9	16	17	19	20
0.25	-3.8	-	-	-11.7	-18.0	-4.1	-44.7	4.0	-3.2	-83.5	-	-	-61.3	-60.6	428.5	-89.0	-76.5	15.7
0.75	-4.5	-	-	-14.5	-18.7	-	-	29.8	-2.6	-80.8	-	-	-73.3	-67.8	-	-	-33.3	10.0
2.00	-3.5	-	-	-12.6	-	-	-	-	-	-25.8	-	-	-75.6	-	-	-	-	-

which could eventually modify the maximum element strengths.

Maximum Local Ductility of Rotation

Table 6 shows the maximum local ductilities of rotation required in the sections of beams and columns, and the differences when the structures are subjected to uni - and to bi - directional earthquakes. The output of the analysis program of the plastic deformations refers only to maximum and accumulated plastic rotations of the section. However the ductility in a column is related with the plastic rotations in conjunction with the axial plastic deformation of the section. In this work the local rotation ductilities in columns were evaluated only as a function of the plastic rotations, and thus these values should be taken only as tendency indicators of the inelastic behavior and should not be used as exact values. The maximum local ductilities required in beams of lateral frames are satisfactorily evaluated by uni-directional earthquake. In eccentric buildings big differences in the

Table 5: Average Of Differences Between The Design Strengths Determined By Chilean Practice And By Bi-Directional Earthquake Criteria (In Percentage)

	Model								
Period [sec]	1	4	6	8	9	16	17	19	20
	Beam design bending moment								
0.25	0.0	0.0	0.5	3.2	-11.5	-5.5	2.3	-0.4	8.9
0.75	0.2	-0.4	-7.5	15.4	-13.5			-1.8	-24.9
2.0	-1.5	1.2	-1.0	-0.1	0.0				
	Column design bending moment								
0.25	0.6	0.0	3.2	12.2	-12.9	-7.6	4.9	-1.0	4.6
0.75	0.1	-0.3	-6.5	6.0	-17.1			-0.8	3.1
2.0	3.6	5.9	4.4	7.2	0.0				
	Column design axial strength								
0.25	0.6	0.0	3.2	12.2	-12.9	-6.4	4.6	-0.6	4.6
0.75	0.1	-0.2	-6.4	6.0	-17.1			-0.7	-20.2
2.0	3.6	5.7	4.4	5.2	0.0				

ductilities required by the beams of the V frames are observed, which can be underestimated in 28% when applying uni-directional earthquake (model 8, T_v= 0.75 sec.). Normally the uni-directional earthquake does not generate important ductilities in the transverse frame beams, that in general behave elastically. Nevertheless they behave inelastically when bi-directional earthquakes excite the structure. The observed differences when evaluating the maximum local ductility of rotation in columns of buildings subjected to uni- and to bi-directional ground movements are important. The local ductility of rotation required by the columns located in the flexible border is under estimated by the uni-directional earthquake, in up to 100%. In the columns located in the rigid border the ductility is under estimated in a 30% by the uni - directional earthquake.

Table 6: Average of maximum local ductility of rotation required in beams and in the first story columns. Differences between uni- and bi-directional earthquakes (in percentage)

	Beams																	
	Model 1									Model 8								
Period [sec]	0.25			0.75			2.0			0.25			0.75			2.0		
	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)
Rigid Fr.	10.2	10.2	0.0	8.8	8.9	0.08	19.5	19.5	-0.04	10.1	11.1	10.0	4.8	3.5	-26.7	20.7	20.7	-0.04
Flexible Fr.	5.2	5.2	0.0	6.0	6.0	-0.03	17.8	17.8	-0.07	5.4	6.0	10.0	4.0	2.9	-26.1	23.9	23.8	-0.1
U Frames	5.4	0.0	100	4.2	0.0	-100	10.9	8.4	-22.3	0.9	0.0	-100	0.6	0.4	-19.3	1.3	1.25	0.0
	Model 9						Model 16			Model 17			Model 19					
Period [sec]	0.25			0.75			0.25			0.25			0.25			0.75		
	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)
Rigid Fr.	10.0	10.3	2.8	0.3	0.3	-0.5	9.9	10.0	0.5	12.4	13.3	7.7	14.0	13.9	-0.1	10.6	10.7	1.1
Flexible Fr.	4.4	4.3	-1.9	4.4	4.4	0.2	4.4	4.5	1.0	10.6	11.4	7.2	13.3	13.3	0.3	7.1	7.0	-0.8
U Frames	3.7	0.0	100	3.3	0.0	-100	2.1	10.0	-55.3	1.8	1.1	-37.8	3.2	0.3	-91.9	3.6	0.7	-79.8
	Columns																	
	Model 1									Model 8								
Period [sec]	0.25			0.75			2.0			0.25			0.75			2.0		
	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)
Rigid Frame	3.9	2.8	-27.6	4.6	4.6	0.5	10.5	10.5	0.1	5.0	3.5	-30.5	4.3	2.8	-34.2	11.5	11.4	-0.1
Flexible Fr.	3.5	0.0	-100	2.6	0.4	-85.9	7.5	5.6	-25.4	2.5	0.0	-100	3.3	1.1	-65.6	6.7	3.6	-45.8
	Model 9						Model 16			Model 17			Model 19					
Period [sec]	0.25			0.75			0.25			0.25			0.25			0.75		
	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)	$\mu\theta_{bi}$	$\mu\theta_{uni}$	Δ (%)
Rigid Frame	5.3	4.0	-24.2	6.5	5.9	-10.2	9.1	9.0	-1.2	6.1	6.3	3.1	9.4	9.4	0.2	10.5	10.4	-0.4
Flexible Fr.	22.8	1.6	-93.0	19.4	1.6	-91.6	6.7	5.1	-24.2	7.0	25.7	268	5.1	4.9	-3.4	3.9	4.0	3.4

CONCLUSIONS

Tipically the evaluation of the seismic response of a building in Chile is based on the analysis of the structure excited by uni-directional earthquakes represented by design spectra. In this work the responses of buildings with elastic and with non-linear behavior, subjected to uni - and to bi-directional earthquakes have been studied. It can be concluded that the evaluation of the element design strengths based on an uni-directional analysis of the whole structure is generally adequate. Only in structures with very different transverse stiffness respect to the lateral stiffness a bi-directional analysis would be necessary. The maximum seismic axial forces in columns are underestimated by an uni-directional earthquake. The seismic axial forces are much more sensitive to the type of excitation in buildings with non-linear behavior than in buildings with elastic behavior. The maximum combined (seismic and gravitational force) elastic and inelastic axial forces are less sensitive to the type of excitation. The maximum local ductility of rotation in the beams of the V frames is not extremely sensitive to the excitation; nevertheless, the maximum local ductility of rotation in the columns located in the flexible border is more sensitive to the seismic excitation. The beams of the U frames have elastic behavior when buildings are uni-directionally excited in the V direction, but the same frames show inelastic behavior when the buildings are excited by bi-directional earthquakes. The important inelastic torsional effect detected when the building is subjected to bi - directional earthquakes is related with an increase of the maximum local ductilities of rotation in the columns located in the flexible border, especially of rigid buildings. If the purpose of the analysis is to know the level of damage to which the columns of a structure are exposed, or the elastic or inelastic torsional behavior of building excited by seismic ground movements, this evaluation should be carried out considering the bi-directional earthquake. If the purpose is to define the design strengths, in most cases it would be adequate the use of uni-directional erathquake acting on the whole structure in the two principal directions independently. In this work has been observed that structures with low redundancy level and small torsional and transverse stiffness, experience excessive non-linear deformations that represent their collapse. Normally the analysis is carried out considering elastic behavior, and there would be no way to predict a possible collapse during an earthquake.

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

Cominetti S., Cruz E., (2000) "Elastic and Inelastic Response of Three-Dimensional Buildings Models, a Comparison of Different Models for Analysis", *Proceedings 12th World Conference on Earthquake Engineering*, Auckland, New Zealand, 372/6/A.

Correnza J.C, G.L. Hutchinson, (1994) "Effect of Transverse Load-Resisting Elements on Inelastic Earthquake Response of Eccentric-Plan Buildings", *Earthquake Engineering & Structural Dynamics*. Vol. Nº23, pp75-89.

Goel , A.K. Chopra, (1970) "Inelastic Seismic Response of One - Storey Asymmetric Plan Systems: Effects of Stiffness and Strenght Distribution", *Earthquake Engineering & Structural Dynamics*, Vol. Nº19, pp949-970.

Hisada, T., Miyamura, M., Kan, S. and Hirao, Y. (1988), "Studies on the Orthogonal Effects in Seismic Analyses" *Proceedings of Ninth World Conference on Earthquake Engineering* August 2-9, Japan, Vol. V, pp191-196.

Mondkar D. and Powell G. (1979), "ANSR-II Analysis of Non-linear Structural Response User's Manual", *Report Nº UCB/EERC 79/17* Earthquake Engineering Research Center, College of Engineering, University of California. Berkeley, California, July.

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

Oliva M. and Clough R. (1987), "Biaxial Seismic response of R/C Frames", *Journal of Structural Engineering*, Vol. 113 N°6, Jun., pp1264-1281.

Tso W.K., Zhu, (1992), "Design of Torsionally Unbalanced Structural Systems Based on Code Provisions I: Ductility Demands", *Earthquake Engineering & Structural Dynamics*, Vol. Nº21.