NON LINEAR BEHAVIOR OF STEEL FRAMES AND SPECTRUM REDUCTION FACTOR

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SUMMARY

This paper presents a study oriented to obtain a better understanding of the Spectrum Reduction Factor used in some of the current seismic design codes. The chilean NCh433 code makes no difference between steel frames and concrete frames, as well as between steel moment resistant frames and steel braced frames. It means the code uses the same spectrum reduction factor for all of them, i.e., the same seimic design loading.

In order to study the differences between the above mentioned structural systems a set of buildings, from 8 to 32 stories were designed according to the chilean code and practice. The earthquake requirements on the buildings were obtained from non-linear analyses, using some representative earthquake records, such as Chile 1985, Mexico 1985, Northridge 1994, and Kobe 1995. The building capacities were obtained by two methods: push over analyses and dynamic nonlinear analysis under scaled records.

Results showed several differences in the behavior of the structural systems, as well as differences between the code and the calculated reduction factors.

INTRODUCTION

The present trend of some seismic codes for defining the earthquake forces is to use a design spectrum, which is obtained from an elastic spectrum reduced by the so called Response Modification Factor (R). It is assumed that it takes into account the energy dissipation capacity of the structure, redundancy, changes in damping and period of the structure, and observed performance of different structural types. The chilean approach was to define this factor as a function of the fundamental period instead of the constant factor used in some other codes, such as the american UBC, SEAOC and ATC. The theorethical bases for this choice was the non linear analysis of a single degree of freedom system under a set of several ground motion records [Ridell, Hidalgo and Cruz 1989]. The final result was equation 1.

$$R^* = 1 + \frac{T^*}{0.1T_0 + \frac{T^*}{R_0}} \tag{1}$$

where: T*: Fundamental Period of the Structure
R_o: Structural Quality Factor
T_o: Soil Parameter

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This factor varies between 1 and $1+R_o$. The code makes no difference between the structural quality factor of steel and concrete frames, and there is no difference between steel moment resistant frames and steel braced frames. That is, both of them are designed to withstand the same seismic forces.

The previous reasoning suggest the following questions: Do steel and concrete frames have the same behavior during an earthquake ?. Would different type of structures, such as Moment Resistant and Braced Frames be designed for the same forces ?. Should the R factor be the same either for concrete or different types of steel structures ?

In order obtain an answer to some of these questions, it was decided to study the structural response of a set of different types of steel buildings under the same earthquake records, using a representative sample in terms of ground motions records and buildings.

STRUCTURAL TYPES AND GROUND MOTIONS

A set of buildings from 6 to 32 stories with the same plan were designed according to the chilean standards. A typical plan and elevation for eccentrically braced frames is shown on figure 1.

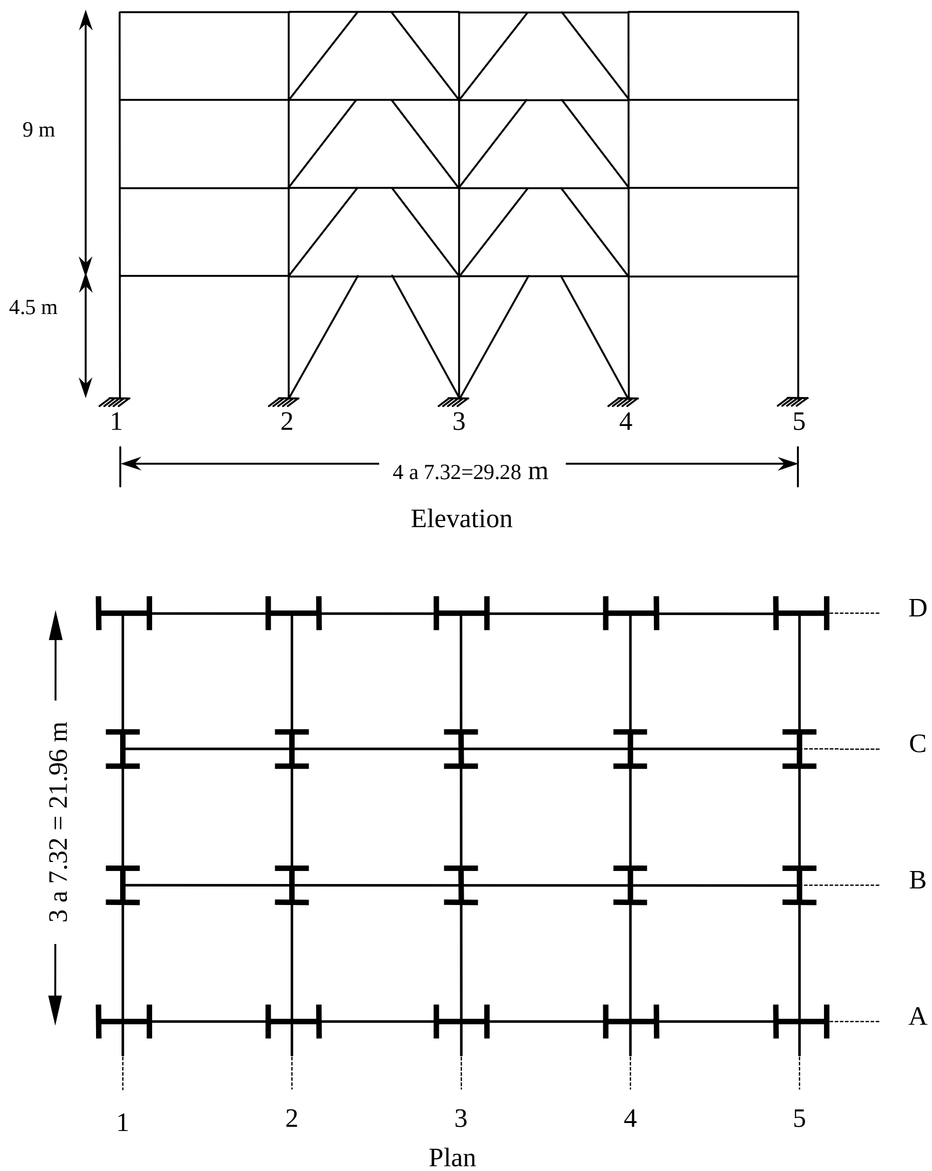

Fig 1.-Typical Plan and Elevation of the Buildings Selected

The study focused in the following two aspects:

1. **Earthquake Requirements.** It was studied the nonlinear response of the buildings under ground motions records ocurred since 1985 in Chile, Mexico, United States, and Japan. The records were selected because they represent some of the most important earthquake events of the last two decades. Observed damages as well as other information has been extensively reported. Table 1 shows some of the main records properties.

Table Error! Unknown switch argument.**.- Records Properties.**

EARTHQUAKE	RECORD	COMP.	SOIL	EP. DIST. (Km)	DURAT. (sec)	RICHTER MAGNIT.	PEAK (g)
CHILE - 3/3/85	Viña del M.	S20W	Alluvium	84	116	7.8	0.36
Chile - 3/3/85	Llolleo	N10E	Sand	45	116	7.8	0.67
México – 19/9/85	SCT	E00W	Soft Clay	400	62	8.1	0.17
Northridge - 17/1/94	Sylmar	N00E	Stiff Soil	15	60	6.8	0.84
Japón - 17/1/95	Kobe	N00E	Alluvium	40	40	6.9	0.84

2. **Seismic Building Capacity.** Because every structural typology is more or less sensitive to different lateral forces, there is not a unique approach to study the building capacity,. Two methods were applied:
   - **Pushover Analysis:** by using a lateral uniform loading and a lateral triangular loading, which represent a lower and upper bound to the maxima dynamic forces [Bertero, 1984].
   - **Non Linear Dynamic Analysis under Scaled Ground Motions:** this second approach was followed to obtain a dynamic estimation of the building capacity. Considering that the ground motion condition is unknown, this approach seems to be reasonable.

OUTLINE OF THE BUILDING ANALYSIS

All the buildings were designed according to the chilean practice, based in the chilean code NCh 427, "Structural Design of Steel Buildings". The only limit to the bracing members was the maximum slenderness allowed by the code. That means there was no a special provision to avoid buckling. The sample includes 4, 8, 16, 24 and 32 story buildings.

The non linear analysis of the building's frame was performed with Ruaumoko program [Carr, 1996] in the case of braced frames, and Drain-2D Program [Kanahn and Powell, 1973] for moment resistant frames. The Remennikov's model [Remennikov and Walpole, 1995] included in Ruaumoko program was used for the bracing elements and the Ricles' model [Ricles and Popov, 1989] for the links. The ultimate limit conditions were imposed according to the behavior of the elements, such as the maximum joint rotation for moment resistant frames and links, and the maximum axial deformation in the case of braces. The maximum ultimate rotation adopted was 0.07 rad and the ultimate axial deformation is given in Figure 2.

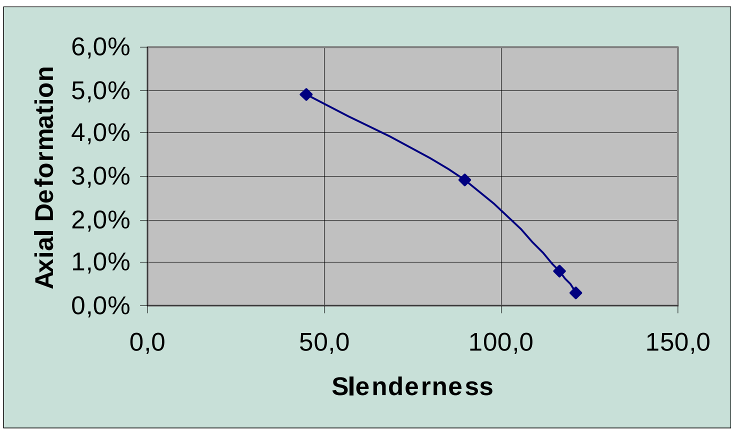

Fig 2.-Ultimate Axial Deformation
[Remennikov, 1995; Balendra, 1995 and Martinelli 1998].

RESULTS FROM ANALYSIS

Earthquake Requirements.

From the ground motions selected, the March 1985 Chilean Earthquake is a main concern for the study of the chilean earthquake effects. Two ground motion components of this earthquake were included. The well known Northridge and Kobe Earthquakes are important because of the damage in steel structures reported, so ground motion records from those earthquakes were also included. The September 1985 Mexican Earthquake was included because it shows great amplitudes for low frequencies, and it was thought that long period buildings were going to be more sensitive to this earthquake record. Figure 3 shows the response spectra of the ground motion records selected.

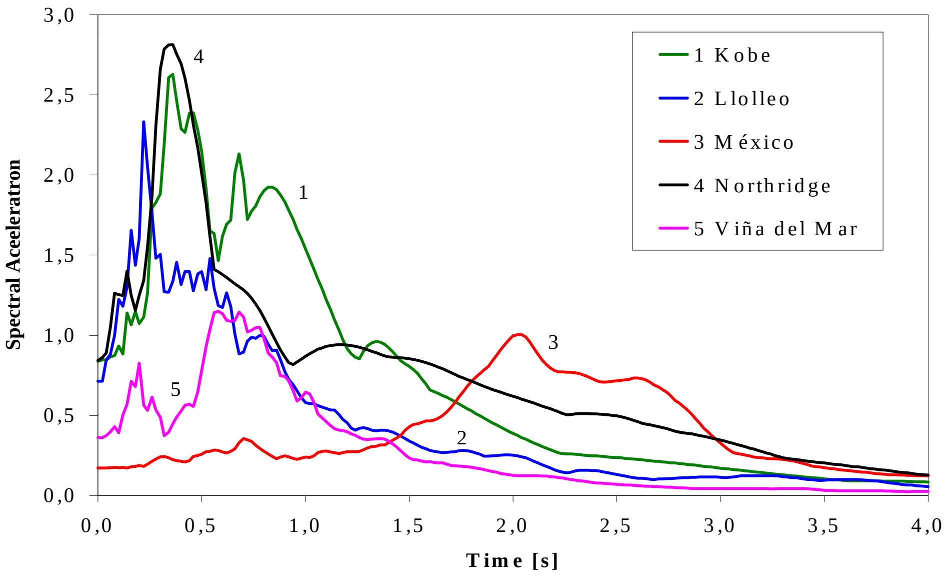

Fig 3.- Response Spectra of the Selected Ground Motios

Roof displacement for two different eccentric frames are shown in figures 4 and 5.

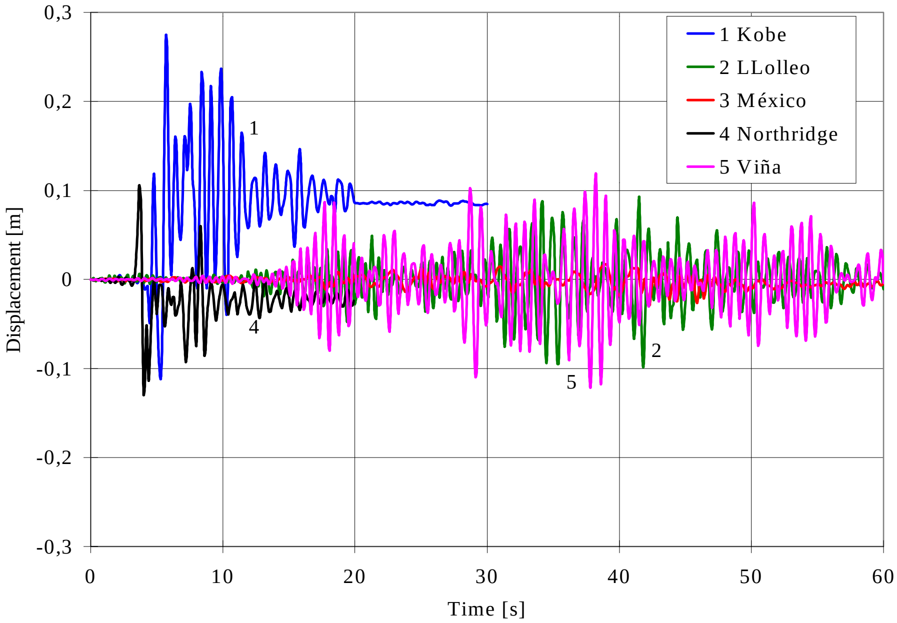

Fig. 4.- Roof Displacement for the eight stories eccentric building.

The effects of the Mexican Earthquake on the eigth story building, and in general for short to medium period buildings, is not significat. Displacement requirements are low and the structure remains in the elastic range. Also, it can be seen that Llolleo and Viña del Mar ground motion records produce non linear roof displacements. The non linear behavior concentrates on the braces, the maximum displacement is, in both cases around 10 cm. The Northridge ground motion produces a 15 cm displacement at the roof and the nonlinear behavior took place in the first story members, including braces, some beams, and columns. In this case, after the earthquake, the structure ends with a permanent 2.4 cm roof displacement. Kobe ground motion produces the largest roof displacement, twenty seven cm after 5.4 seconds from the record starting. Altough the inelastic seismic activity was important under Kobe record, it was not enough to reach some limit condition in the frame, either in terms of member displacements or a collapse mechanism of the frame.

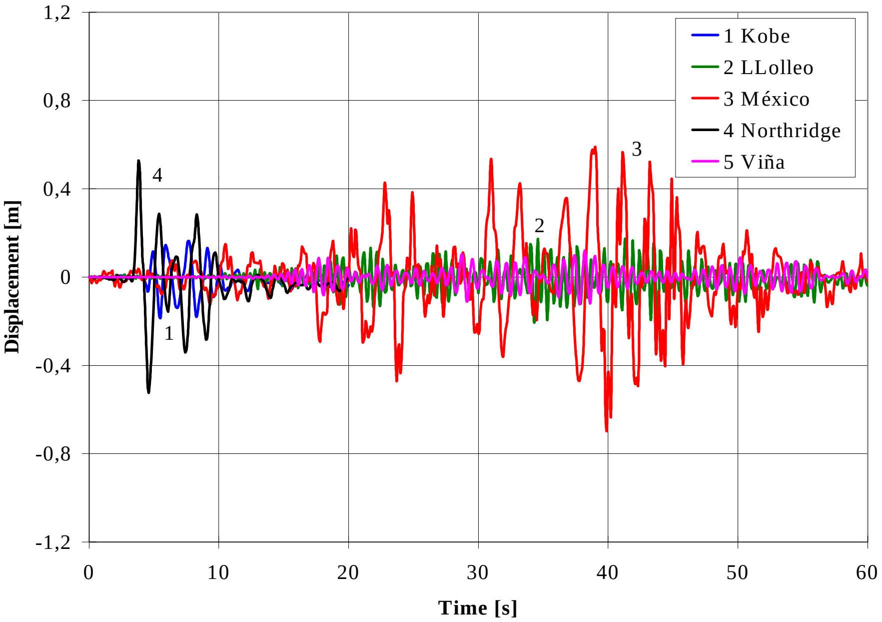

Fig. 5.- Roof Displacement for the thirty two stories eccentric building

Figure 5 shows the influence of the long period waves of the Mexican Earthquake on one of the tallest building of the sample. The largest displacement was 89 cm. It ocurred after fourty seconds from the record starting. A summary of the maxima roof displacement requirements for the eccentric buildings set is shown on Table 2.

Table 2.- Roof Displacement Requirements.

	Chilean Code	Displacements [cm]				
	Max. Separat. [cm]	Kobe	Llolleo	México	Northridge	Viña
4 Stories	0.91	13.3	8.0	0.4	25.1	0.7
8 Stories	3.31	27.4	9.8	2.1	14.9	12.2
16 Stories	9.95	26.6	13.9	10.5	30.3	12.3
24 Stories	23.30	40.1	18.3	44.6	42.1	15.5
32 Stories	58.58	18.5	23.4	67.7	52.7	12.3

The Northridge record produces the sudden collapse of the four stories building after 4.8 seconds shaking, close to an important peak of the ground motion record. This ground motion produces buckling of the first story braces and plastic rotation in some joints.

The relationship between base shear and roof displacement as well as other parameters, were also obtained. The curves were not included in the paper. A summary of maxima base shear is shown in table 3. More detailed results can be found in the original report [Uribe, 1999].

Building Capacities.

All the buildings were analyzed by two approaches, as it was previously mentionned: Pushover Analysis and Dynamic Seismic Non Linear Analysis by using Scaled Ground Motions.

Figure 6 shows the relationship between roof displacement and base shear for a thirty two story building. It was obtained by pushover analysis. Triangular and rectangular static lateral loading results are shown, as well as bilinear equivalent curves. First yielding and ultimate lateral deflection are shown for both cases.

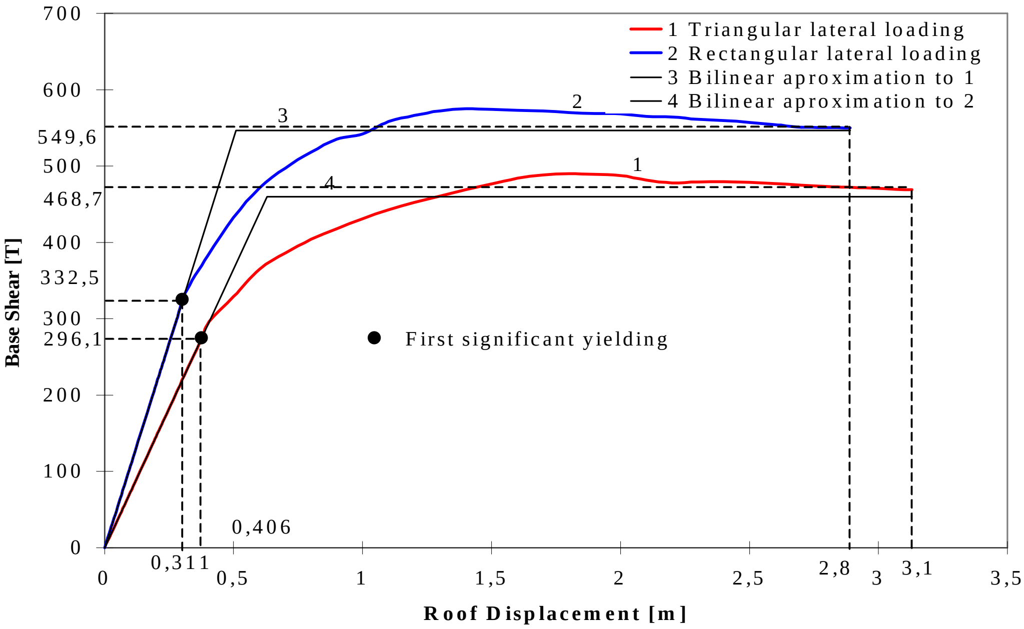

Fig. 6.- Roof Displacement vs. Base Shear for the thirty two stories eccentric building

It was easy to foresee a better lateral deflection capacity of the frame when the uniform lateral load is applied. Table 3 shows all the maxima base shear obtained, for the buildings and loading system applied.

Table 3.- Base Shears Requirements

Building	4 Stories		8 Stories		16 Stories		24 Stories		32 Stories	
Loading	$Q_{máx}$ [T]	% P	$Q_{máx}$ [T]	% P	$Q_{máx}$ [T]	% P	$Q_{máx}$ [T]	% P	$Q_{máx}$ [T]	% P
Kobe	159.0	40.1	241.8	32.6	258.0	15.9	234.1	11.2	139.5	4.07
Llolleo	147.4	37.2	221.1	29.8	137.0	8.43	162.0	7.50	147.4	4.30
México	18.00	4.53	86.3	11.6	133.0	8.18	223.5	10.3	399.2	11.7
Northridge	149.0	37.5	236.9	31.9	267.9	16.5	269.2	12.5	354.4	10.3
Viña	36.40	9.17	224.8	30.3	130.0	8.00	96.6	4.47	87.8	2.57
D. Uniforme	157.8	39.7	264.6	35.7	291.8	17.9	302.0	13.9	549.6	16.1
D. Triangular	150.1	37.8	238.6	32.2	244.0	15.0	211.1	9.77	468.7	13.7

Similar results were obtained by using scale ground motions and dynamic non linear analysis. The records were scaled until the failure ocurrence. Lateral displacement capacities obtained are a little smaller than those obtained by using pushover analysis, except the case of the four story building, where they became higher with the scaled analysis approach. On the other side, base shear obtained with this second approach is a little higher than the first one.

RESPONSE MODIFICATION FACTORS

The comparison of the earthquake requirements and the building capacities allows to estimate the response modification factors . Equation (2), (Uang, 1991), defines the approach:

$$R = \frac{Q_{oe}}{Q_y} * \frac{Q_y}{Q_s} = R_\mu * R_{os} \tag{2}$$

where: Q_{oe}:Shear Base from Elastic Analysis by using Elastic Spectrum
Q_y :Shear Base at Yield Frame Level
Q_s :Shear Base at First Significative Yielding Occurency
R_μ : Reduction Factor due to Ductility
R_{os}: Reduction Factor due to Overstrength

Figure 7 shows R factors curves for the different buildings using pushover analysis. It has been included equation (1) of the chilean code.

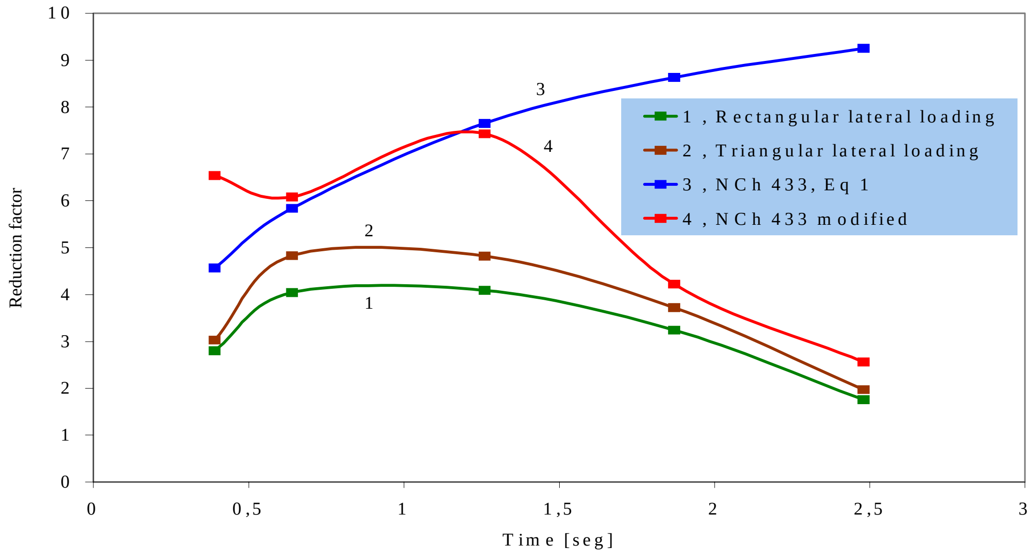

Fig. 7.- Modification Factor obtained from comparison of Earthquake. Requirements and Building Capacities and Chilean Code Eq.

There is a clear difference in magnitude and trend between the R factor of the chilean code and the R factor obtained from the previous analysis. The code equation, based on a ductile behavior of a SDFS and several non linear analyses under differents ground motions, cannot include redundancy, overstrength or higher mode effects, because those aspects require a different model and some additional hypotheses. Same differences have been pointed out before [Seneviratna and Krawinkler, 1996, Miranda, 1997]. The figure also shows a modified R factor, including the upper and a lower bounds impose to the base shear in the chilean code. That curve is a better approach to the MDFS behavior when the periods is higher than 1.8 sec, but it seems to be high for buildings periods between 1sec and 1.5sec.

CONCLUDING REMARKS

1. The previous analysis shows only some aspects of this widespread approach of seismic forces definition by applying a reduction factor to the elastic response spectrum. It is clear this is not a simple matter. Many codes put in this bag all those things that are not well known, so the R factor becomes a kind of useful magic number eventhough it is not well understood what it is being taking into account.

2. Redundancy, overstrength, structure deterioration, higher mode effects and experience requires to be explicit included in the seismic codes. Only some of them can be obtained from theoretical analyses. Some others require laboratory tests, field measurements, and a valuable local knowledge called experience.
3. R factor definition, based on non linear SDFS analyses, allows to include the building period in the reduction factor. However, it is necessary to include the behavior of MDFS which is different to the SDFS, specially in conection to energy dissipation characteristics.
4. The reduction factors obtained with this analysis are smaller than the chilean code factors. It means, the buildings capacities are higher than the earthquake requirements and this is on the safe side. The analyses by using chilean earthquakes, in some cases show a sligth non linear behavior of the structures, but not significant damage. It would seem that NCh 433 code is safe due to the nature of the chilean earthquakes.
5. Northridge and Kobe earthquakes produces large displacements of the buildings, sometimes permanent, with extensive damage, The smaller is the building the highest is the damage. The Northridge record produces the sudden collapse of the four stories building after 4.8 sec shaking. The NCh 433 code is not appropiate for these type of earthquakes conditions.
6. The Modification Reduction Factor of the chilean code should be reviewed. The trend to increase the reduction factor for large period buildings disagrees the MDFS behavior. It seems reasonable to look for a factor that fits better to the MDFS energy dissipation characteristics, which is the typical building model..
7. The upper and lower boundary to the base shear of the chilean code are on the rigth way. They improve the R factor and put some control levels to the spectrum.
8. In order to provide a similar seismic protection for any type of building, the reduction factor should reflect the expected behavior of the structures.

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REFERENCES

Balendra, T.,Lim,L.,Liaw,Y., (1995), *Program to Reduce the Earthquake Hazard of Steel Moment Frame Structures*, Federal Emergency Management Agency, Seismic Safety of Buildings.

Bertero V. (1984), "State of the Art and Practice in Seismic Resistant of R/C Frame-Wall Structural Systems," *8th World Conference on Earthquake Engineering Proceedings*, Vol 5, pp 613, San Francisco.

Carr A., (1996), *Ruaumoko Inelastic Analysis Program*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Instituto Nacional de Normalización, (1996), S*eismic Design of Buildings*, Norma NCh433.Of 96, Santiago, Chile.

Kanahn A. E., Powell G.H., (1973), *General Purpose Program for Inelastic Dynamic Response of Plane Structures*, Report No. 73-6, Department of Civil Engineering, University of California, Berkeley California (in spanish).

Martinelli, L., Mulas, M., Perotti,F., (1998), "The Seismic Response of Concentrically Braced Moment-Resisting Steel Frames," *Earthquake Engineering and Structural Dynamics*, vol 12, 749-769.

Miranda, E., (1997) "Strength Reduction Factors in Performance Based Design", *The EERC-CUREe Symposium in Honor of Vitelmo V. Bertero*, EERC, University of California, Berkeley.

Remennikov, A. and Walpole, W. , (1995) , *Incremental Model for Predicting the Inelastic Hysteretic Behavior of Steel Bracing Members*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Ricles,J. and Popov, E., (1989) , "Inelastic Link Element for EBF seismic analysis," *Journal of Structural Engineering*, ASCE, 120(2), (441-463).

Ridell, R., Hidalgo, P., and Cruz, E., (1989), "Response Modification Factors for Earthquake Resistant Design of Short Period Buildings", *Earthquake Spectra*, Vol. 5, No 3.

Seneviratna, G., D., P., K. and Krawinkler, H., (1996), "Modification of Seismic Demands for MDFS," 11th *World Conference on Earthquake Engineering,* CD-ROM paper Nº 2129, Acapulco, Mexico.

Uang, C., M., (1991), "Establishing R and Cd factors for building seismic provisions", *Journal of Structural Engineering*, ASCE, vol. 117(1), (19-28).

Uribe, R., (1999), *Response Modification Factors on K-Braced Steel Frames*, Memoria para optar al título de Ingeniero Civil, UTFSM, Valparaíso, Chile (in spanish).