Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Seismic force reduction factor for masonry building

M.O. Moroni, M. Astroza & J. Gómez
Universidad de Chile, Chile

ABSTRACT: In the new Chilean seismic code, presently under revision, an elastic spectrum has been proposed to determine seismic forces, which must be reduced by a factor R in order to account for ductility and overstrength of the structure. This factor depends on the basic structural system, the soil conditions and the natural period of the structure. In this paper a methodology to estimate the R value is described, and those corresponding to confined masonry buildings are calculated in order to verify the expression given in the code for that factor.

1 INTRODUCTION

Most of modern seismic codes, including the Chilean code NCh433.cR89, prescribe elastic response spectra to determine seismic forces, together with a structural response modification factor R to scale it down to account for the ductility and overstrength of the structures. However, there is not a common basis to formulate explicit expressions for that force reduction factor.

In the case of NCh433.cR89, R is given by equation (1) (Arias, 1989), which depends on the soil conditions T_o, the period of the structure T and the parameter R_o which is related to the structure ductility. This equation is based on both results from analysis of systems of one degree of freedom, and observation of damage in structures during past earthquakes. A value of $R_o = 4$ is suggested for confined masonry buildings.

$$R = 1 + \frac{T}{0.10T_o + \frac{T}{R_o - 1}} \tag{1}$$

The objective of this paper is to verify that expression and calibrate the R_o value for that type of buildings. This is done by analyzing several confined masonry buildings, subjected to the action of different records of a strong earthquake, and comparing the base shear that develops in the structure when a linear and elastic behavior is supposed versus the base shear that develops when a nonlinear behavior is adopted.

2 METHODOLOGY

The force reduction factor may be defined as the ratio between the base shear V_{eu} that develops in the structure if it were to remain in the elastic range, and the minimum required design base shear V_d. The latter may referred either to a level V_s, beyond which the global structural response starts to deviate significantly from the elastic response, which is consistent with material codes that use a strength design approach, or to the service load level V_w which is consistent with material codes that use the allowable stress design method. The Chilean confined masonry code corresponds to the latter case.

Figure 1 represents a general structural response. From that curve and according to Uang (1991), the total reduction factor corresponding to the allowable-stress design format can be derived in terms of the ductility reduction factor R_μ, the overstrength factor Ω, and the allowable stress factor Y, as follows:

$$R_w = \frac{C_{eu}}{C_w} = \frac{C_{eu} C_y C_s}{C_y C_s C_w} = R_\mu \Omega Y \tag{2}$$

In this paper, several 3-D time history analysis for different buildings were performed considering linear elastic and nonlinear behavior for different acceleration records, and a 5% of damping. From these analysis a curve, similar to figure 1, which relates the base shear with the roof deflection is obtained. In this curve is also included the result from a linear response spectrum analysis using the linear response spectra proposed in the NCh433.cR89 code, V_{eun}.

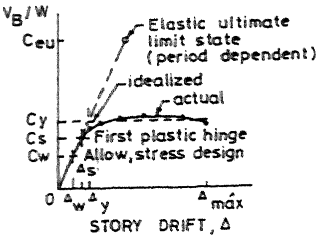

Figure 1. General structural response.

From that curve the following parameters can be calculated for each record: V_{eu}, the maximum base shear that develops in the structure if it were to remain in the elastic range; V_y, the base shear corresponding to the maximum strength of the building; and V_s, the base shear corresponding to the formation of the "first plastic hinge", which is associated with the formation of shear cracks in the wall with the heaviest load. According to Nch433.cr89, $V_w = 0.5*1.33\ V_s$, i.e. the average allowable shear stress of 50% of the nominal cracking stress may be increased by one-third for seismic loads.

The records correspond to those registered during the March 3, 1985 earthquake in Chile and represent different soil conditions, intensities I and destructiveness potential factors P_D. Table 1 contains some characteristics of the records.

Table 1. Characteristics of the records.

Record	Peak accel (g)	P_D (10^{-4}) gseg^3	Soil cond	I (M.M.)
Llolleo N10E	0.669	201.6	sand	8.5
LLolleo S80E	0.426	80.3	sand	8.5
Melipilla NS	0.68	43.3	gravel	8.0
Melipilla EW	0.648	37.1	gravel	8.0
Iloca EW	0.281	32.0	sand	7.0
San Fdo EW	0.335	14.8	gravel	7.5
Quintay EW	0.243	7.2	rock	6-7

3 ANALYTICAL MODEL

The buildings are structured mainly by confined masonry walls. Most of the damage observed in this type of buildings during earthquakes and in wall testings has been shear type failure without any plastification of the columns. Based on these facts, the walls are represented in the analysis by a flexible bar coupled with a shear spring as proposed by Astroza, Moroni and Navarrete (1991). That element can be used in the frame equivalent method and has been incorporated into programs DRAIN 2D and DRAIN-TABS. Non linear behavior is restricted to the shear spring, which is characterized by a trilinear primary curve and degrading stiffness hysteresis loops as shown in figure 2.

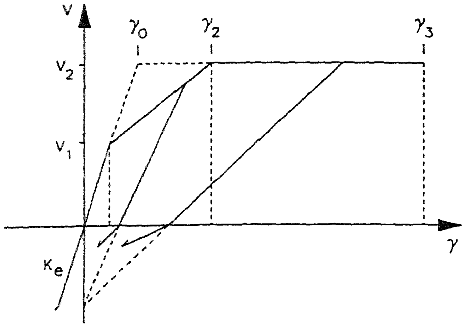

Figure 2. Hysteresis model for shear spring.

The parameters V_1, V_2, K_e, γ_o, γ_2, γ_3 are defined as follows:

V_1 : lateral load related to the "first significant yield level", i.e. the level beyond which the response starts to deviate significantly from the elastic response. This corresponds to shear strains about 1/2400.

V_2 : maximum lateral load.

K_e : effective stiffness of the shear spring.

γ_o : yield deformation of the equivalent elasto plastic system with effective stiffness K_e and lateral load V_2.

γ_2 : deformation obtained for the maximum lateral load V_2.

γ_3 : maximum available (ultimate) deformation, post-peak deformation when the load carrying capacity has undergone a reduction of about 20%.

The parameter K_e is given by equation (3), which reflects the fact that the stiffness of the primary curve is about one third of the tangent stiffness obtained for small strains.

$$K_e=\frac{G_m \cdot A_e}{3h} \tag{3}$$

$$A_e=A_m+2\eta_g A_c,\ \eta_g=\frac{G_c}{G_m} \tag{4}$$

Variables G_m and G_c refer to the masonry and reinforced concrete shear modulus respectively, A_m and A_c to the masonry and columns areas and h to the height of the wall.

The lateral loads V_1 and V_2 depend on the vertically applied loads σ_o and the masonry shear strength τ_m; they can be estimated using equations (5) and (6):

$$V_1=(0.23\tau_m+0.12\sigma_o)A_m \le 0.35A_m\tau_m \tag{5}$$

$$V_2=(0.45\tau_m+0.30\sigma_o)A_m \tag{6}$$

For γ_2 and γ_3, Astroza et al. (1991) suggested the values 3 γ_o and 10 γ_o, which best fitted the experimental values.

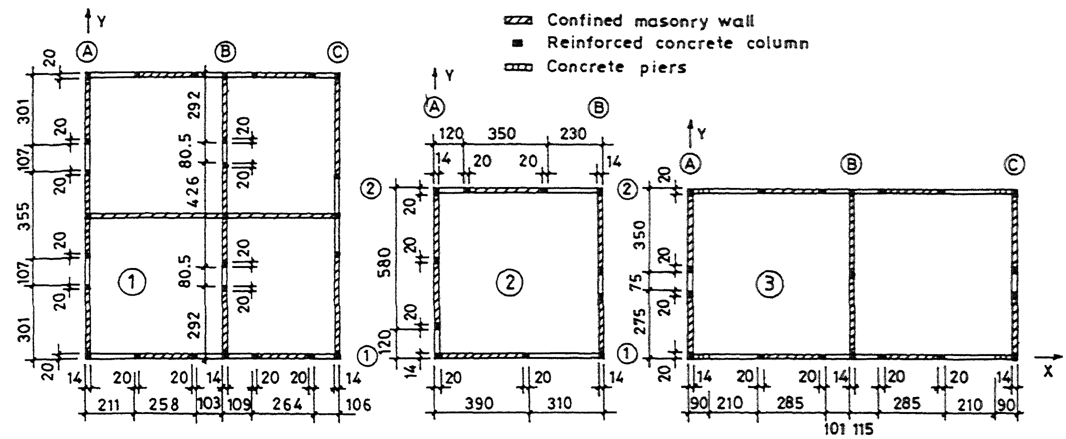

Figure 3. Plan layout of buildings.

Stiffness degradation is introduced by setting a common point at $V_1/2$ on the y-axis and assuming that unloading lines target that point until they reach the x-axis, after which they aim to the previous maximum or minimum points.

4 BUILDINGS LAYOUT

The buildings correspond to typical 3 to 4 story height dwellings that have been built in Chile in the past years, in accordance with NCh433 of72. An elastic modulus = 5690 MPa, shear modulus = 1260 MPa and τ_m= 0.8 MPa have been considered. Figure 3 shows the plan layout of some of them.

Table 2 shows some characteristics of the buildings analyzed, such as the number of floors N, the period T, the ratio of wall to floor area δ in both directions, and the weigth of the reactive masses W.

Table 2. Buildings analyzed

Id	N	T_x (sec)	T_y (sec)	W (ton)	δ_x %	δ_y %
1.A	3	0.089	0.099	209.4	2.38	3.32
1.B	4	0.122	0.147	287.1	2.38	3.23
2.A	3	0.12	0.096	115.5	2.11	3.44
2.B	4	0.169	0.137	158.7	2.11	3.44
3.A	3	0.092	0.109	222.9	2.16	2.81
3.B	4	0.125	0.16	307.7	2.16	2.81

5 RESULTS

Figure 4 shows the structural response of building 3.B when the Llolleo S80E acceleration record is applied in the y-direction. From that curve a factor R_w = 3.24 is obtained.

Table 3 contains the R_w factor calculated for the different buildings and the maximum Δ_m/Δ_c : the ratio between the maximum story drift reached by any wall in the first floor and a "collapse displacement", defined as the story drift corresponding to a shear spring deformation of γ_3, for that wall.

Figure 5 shows the plot of eq. (1) and the average value of R_w for each building, only including the cases when Δ_m/Δ_c is less or equal to 1.5; beyond this limit the wall cannot stand further deformation without a severe damage and strength degradation.

It can be observed that the expression given by equation 1 is a lower bound for R_w representing a conservative criteria. The buildings that are closer to the curve are those with the biggest ratio of wall to floor area and according to the present code they are somewhat overdesigned. On the contrary, those that are farther correspond to buildings that just satisfy the requirements of the code. The dependency on the period is clear, but more analysis are needed to propose a new relation. It must bear in mind that the response level obtained from the elastic response spectra proposed in the code for this type of buildings is smaller than the elastic response obtained from any of the records used in this paper, with the exception of Llolleo S80E; therefore a bigger R does not necessarily imply a smaller strength level.

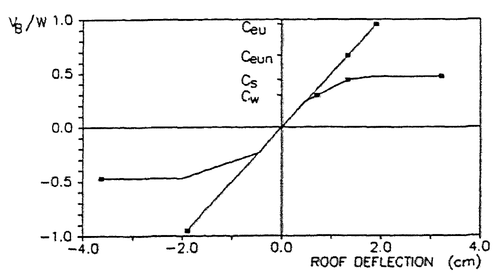

Figure 4. Base shear vs roof deflection, building 3.B

Table 3. Seismic force reduction factor

Id	Record	Direc	R_w	Δ_m/Δ_c
1.A	Llolleo N10E	x	3.76	1.93
		y	3.08	0.81
	LLolleo S80E	x	3.3	0.32
		y	*	0.22
	Melipilla NS	x	3.7	1.63
		y	2.46	1.03
	Melipilla EW	x	3.92	1.75
		y	3.16	0.72
1.B	Llolleo N10E	x	5.57	2.63
		y	5.08	1.39
	Llolleo S80E	x	2.24	0.91
		y	*	0.34
	Melipilla NS	x	7.05	1.44
		y	3.51	0.92
	Melipilla EW	x	5.67	1.5
		y	3.37	1.07
2.A	Llolleo N10E	x	5.48	2.85
		y	3.39	1.94
	Llolleo S80E	x	2.27	1.22
		y	*	0.25
	Melipilla NS	x	6.85	1.18
		y	3.55	1.73
	Melipilla EW	x	5.5	1.49
		y	3.52	1.27
2.B	Llolleo N10E	x	5.48	2.64
		y	5.18	3.37
	Llolleo S80E	x	4.37	0.94
		y	2.06	0.83
	Melipilla NS	x	7.3	1.31
		y	6.61	1.23
	Melipilla EW	x	5.86	1.16
		y	5.81	1.41
3.A	Llolleo N10E	y	5.49	3.21
	Llolleo S80E	y	*	0.32
	Melipilla NS	y	3.57	1.58
	Melipilla EW	y	5.7	1.44
3.B	Llolleo N10E	y	6.51	4.18
	Llolleo S80E	y	3.24	1.42
	Melipilla NS	y	5.6	0.84
	Melipilla EW	y	5.66	1.33

* elastic response

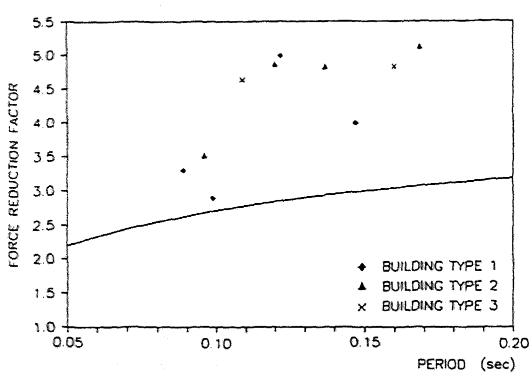

Figure 5. Seismic force reduction factor vs period of structure.

6 CONCLUSIONS

Seismic force reduction factors have been calculated by performing 3-D time history analysis of confined masonry buildings subjected to the action of severe earthquakes and comparing the base shear developed when linear and nonlinear behavior is assumed. In adition to this, limitations on story drifts were imposed in order to obtain feasible results.

Shear cracks in a wall occur at a load level which is very close to the ultimate carrying capacity load. Moreover, the damage is concentrated in the first floor and there is no load distribution, so the overstrength factor is almost negligible.

The R values proposed in the code NCh433. cR89 looks reasonable, however, it was found that to avoid severe damage in the buildings, it is necessary to limit the inelastic story drift in the walls of the first story.

ACKNOWLEDGMENTS

The author thank FONDECYT, Project 1096/92-T, for the financing of this work.

REFERENCES

Arias, A. 1989. Proposición de espectros de diseño para la nueva norma chilena de diseño sísmico. Proc. 5ª Jornadas Chilenas de Sismología e Ingeniería Estructural: 853-864.

Astroza, M., Moroni, M.O., Navarrete R. 1991. Análisis no lineal de edificios de albañilería confinada. Proc. XXV JSEE:IV-49-IV-60. Brasil.

Guendelman Israel, R., Powell, G. 1977. DRAIN TABS: a computer program for inelastic earthquake response of three dimensional buildings. EERC-08, University of California, Berkeley.

Instituto Nacional de Normalización, 1989 Diseño sísmico de edificios, Norma NCh433-.cR89, INN, Santiago.

Instituto Nacional de Normalización, 1972 Cálculo Antisísmico de edificios, Norma NCh433.of72, INN, Santiago.

Powell, G., Kanaan A. 1973. DRAIN-2D a general purpose computer program for dynamic analysis of inelastic plane structures. EERC-06, University of California, Berkeley.

Uang C. 1991. Establishing R (or R_w) and C_d factors for buildings seismic provisions. JSE, Vol 117, Nº1:19-28.