Wall Density and Seismic Performance of Confined Masonry Buildings

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Confined masonry represents one of the most widely used construction systems in Latin America in dwellings built under government social programs. The important investments that those constructions represent makes it necessary to have a seismic behavior qualification index in order to quantify their vulnerability to seismic action. This information is also required in any study of seismic risk and to evaluate provisions of the seismic design code that involves this type of buildings.

Confined masonry refers to the construction system where reinforced concrete vertical tie-columns, located at regular intervals and connected together with reinforced concrete horizontal tie-beams, confine unreinforced structural masonry walls.

Miranda (1996) has presented a simplified displacement-based method of analysis based on the comparison of maximum lateral displacement and ductility demands with their corresponding capacities in order to assess the seismic vulnerability of the structure during severe earthquake ground motions. This comparison of demands and capacities is made both at the global and at the local level of reinforced concrete structures.

In this paper a minimum wall density per unit weight per floor (*d*) for confined masonry buildings is determined, given the displacement capacity of the buildings and the required displacement when these structures are submitted to the action of real earthquakes. How this minimum wall density relates with strength requirements included in the Chilean seismic design code and with observed building damage in past earthquakes is also addressed.

The wall density, d, is defined according to *Meli* (1991) as the ratio between the total shear wall area in one direction, A_t, and the floor area A_p:

$$d = \frac{\sum F_i \; A_{mi}}{A_p} \tag{1}$$

where F_i is a factor that reduces the contribution of slender walls; it equals 1 if $H_i / L_i \leq 1.33$ and equals $(1.33\, L_i/H_i)^2$ if $H_i / L_i > 1.33$, with H_i = wall height and L_i = wall length.

The wall density per unit weight per floor, *d*, represents the ratio of wall to floor area, d, divided by the number of floors, N, and by the average floor weight, w, ($d = d / N \times w$). In others words, it is computed as the ratio of the wall cross-sectional area in the first floor to the total weight of the structure.

The displacement capacity of confined masonry buildings is determined using the story mechanism model (*Tomazevic*,(1982)) and push-over analysis. Displacement demands imposed by real earthquakes are obtained through non-linear dynamic analysis.

The objective of this paper is to determine a minimum wall density per unit weight per floor that ensures an appropriate seismic performance of confined masonry buildings. The work is based on damage observation, resistance prescribed by *NCh433.Of96* (1996) and displacement capacity and displacement demand determination.

DETERMINATION OF THE DISPLACEMENT CAPACITY

Two methods have been used to determine the displacement capacity of each building: the story mechanism model applied at the first floor and push over analysis. In both, the hysteresis envelope of the building's first story, which represents the relationship between the acting lateral seismic loading and the lateral deformations of the first story, is obtained. In that determination the following assumptions are made:

- the walls are connected together with horizontal tie-beams and floors, which act as rigid horizontal diaphragms;
- the walls of composite cross-sections are considered as a sum of parts of the walls, without compatibility in the vertical joints;
- the contribution of individual walls to the lateral resistance of the story depends on the attained lateral deformation of the walls;
- the walls can carry their part of the lateral loading until their deformations exceed the ultimate value;
- the story shear is distributed into the individual walls according to their stiffness. Two types of boundary conditions are considered: fixed-ended if masonry

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parapets couple the walls or cantilever if only reinforced concrete beams or slab are present;

- the hysteresis envelope of each wall is represented by a trilinear curve.

The elastic and shear modulus of the masonry have been reduced to reflect the fact that the stiffness of the primary curve is about one third of the tangent stiffness obtained for small strains. The geometric properties are evaluated using the composite section to include the fact that the cross section is heterogeneous.

The displacement capacity corresponds to the actual displacement when the strength has been reduced by about 20%.

An inverted triangular load pattern was used with pushover analysis. In this case the walls' non-linear behavior included shear and flexural stiffness degradation.

DETERMINATION OF REQUIRED DISPLACEMENT

Most damage observed in confined masonry buildings during earthquakes, as well as in wall testing, has been due to shear failure without plastification of the columns. Based on experimental results, a simple analytical model to predict the inelastic response of confined masonry walls has been proposed by *Moroni et al.* (1994). This model, which can be used with the frame equivalent method, consists of a flexible bar coupled with a shear spring and has been incorporated into program DRAIN-2D. Nonlinear behavior is restricted to the shear spring, which is characterized by a trilinear primary curve and degrading stiffness hysteresis loops.

Several time history analysis were performed considering non-linear behavior for different acceleration records, and assuming 5% critical damping.

Table 1. Damage Categories

Category	Damage Extension	Action
0 No damage	No damage	No action is needed
1 Light non-structural damage	Fine cracks on plaster, falling of plaster on limited zones	It is not necessary to evacuate the building. Only arquitectural repairs are needed
2 Moderate structural damage	Small cracks on masonry walls, falling of plaster block in extended zones. Damage in non-structural members, such as chimneys, tanks, pediment, cornice. The structure resistance capacity has not been reduced noticeablement. Generalized failures in non-structural elements.	It is not necessary to evacuate the building. Only arquitectural repairs are needed in order to ensure conservation
3 Severe structural damage	Large and deep cracks in masonry walls, widely spread cracking in reinforced concrete walls, columns and buttress. Inclination or falling of chimneys, tanks, stair platforms. The structure resistance capacity is partially reduced.	The building must be evacuated and raised. It can be reoccupied after retrofitting. Before arquitectural treatment is undertaken structural restoration is needed.
4 Heavy structural damage	Wall pieces fall down, interior and exterior walls break and lean out of plumb. Failure in elements that join buildings portions. Approximately 40% of essential structural elements fail. The building is in a dangerous condition.	The building must be evacuated and raised. It must be demolished or major retrofitting work is needed before being reoccupied.
5 Collapse	Collapse of part or complete building.	Clear the site and rebuild.

Some of the buildings were subjected to the action of normalized records to an Arias Intensity of 5 m/sec (16.4 ft/sec). With this normalization the records elastic spectra are representatives of *NCh433.Of96* seismic code elastic spectra for seismic zone 3 and soil type II. The same buildings were subjected to the action of scaled records in order to produce varying degrees of inelastic response. The scale factor is the ratio between the elastic base shear ratio, C_e, and the yield shear strength, C_y, of the structure. C_y is defined as the sum of maximum shear wall capacities divided by the total reactive weight ($S = F_2/W$) in each direction. The maximum shear wall capacity depends on the masonry shear strength t_m and the normal stress due to vertical applied loads, s_o , according to Equation 2:

$$F_2 = \left(0.45\tau_m + 0.30\sigma_o\right) A_{mi} \leq 1.50\tau_m \ A_{mi} \qquad \textbf{(2)}$$

The records, that have been selected considering its capacities to induce inelastic behavior, correspond to those registered during the March 3, 1985 earthquake in Chile (M_s = 7.8) at Llolleo, Melipilla, Viña del Mar and San Fernando stations. They correspond to soil type II and seismic zone 2 and 3. According to its destructiveness potential, the first three represent a collapse condition while the latter a serviceability condition (*Saragoni et al*, (1989)). However, this ordering is lost when the records are normalized, because the destructiveness potential changes.

DAMAGE OBSERVATION

After the March 3, 1985 earthquake, damage that occurred in masonry buildings located in Santiago (Mercalli Modified Intensity equal or greater than 7) was surveyed. The damage level was classified in five categories according to Table 1. Category 4 corresponds to a state of damage that can not be repaired, but it represents an accepted level of damage by the seismic design code when only collapse is avoided.

Table 2. Relation Between the Level of Damage and the Wall Density Per Unit Floor

Level of Damage	Damage Category N_d	Wall density d/N (%)
light	0 - 1	³ 1.15
moderate	2	0.85 - 1.15
severe	3	0.5 - 0.85
heavy	4 - 5	£0.5

A relation has been established empirically between the level of damage N_d and the wall density per unit floor, *d/N*, which is shown in Table 2 (*Astroza et al*, (1993)). In this determination, Mexican data has also been used. Assuming an average weight of 6.50 kPa (0.94 psi) this same relation gives the results shown in Figure 1, as a function of the wall density per unit weight per floor. The high level of damage of building J is due to poor quality of the mortar and lack of reinforcement in the reinforced concrete elements.

A wall density per unit weight per floor greater than 0.013 m²/ton (0.0091 in.²/1b) would be necessary in order to ensure the occurrence of only moderate damage, and greater than 0.008 m²/ton (0.0056 in.²/1b) if only heavily damage is to be avoided.

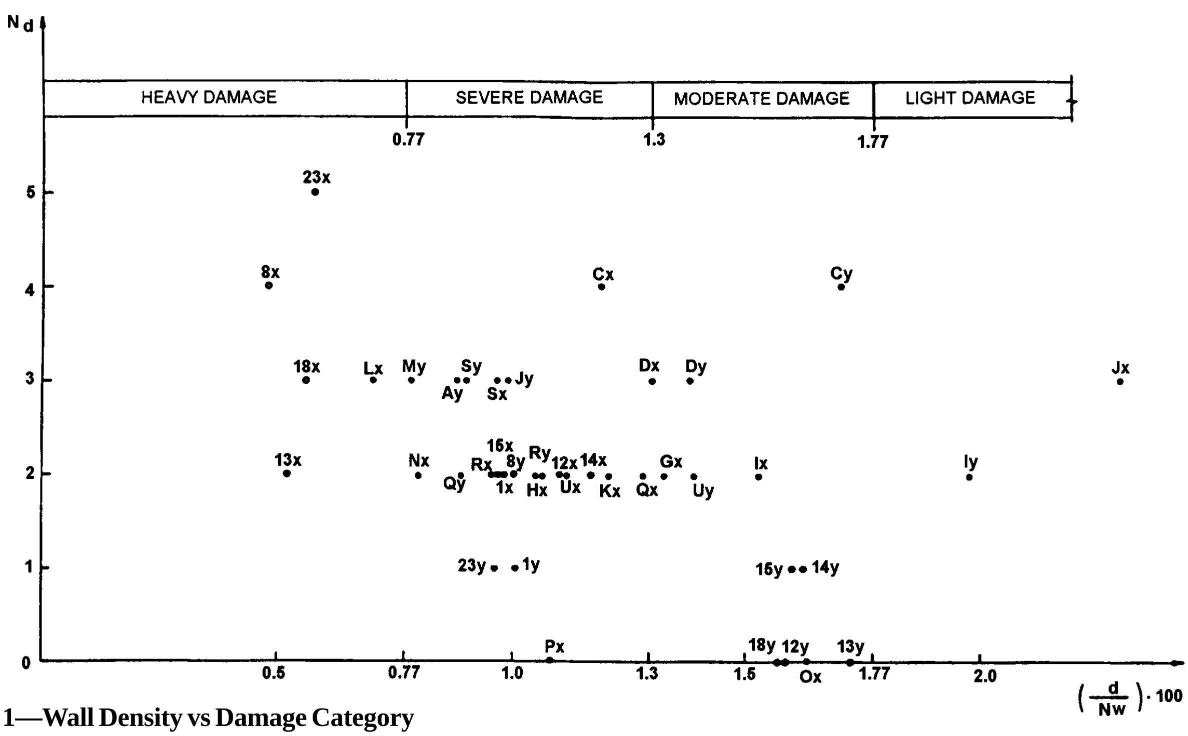

Figure 1—Wall Density vs Damage Category

Table 3. Buildings Characteristics (1 ton = 2,250 lbs, 1 m^2/ton = 0.7 in.^{2}/1b)

Building	N	W (ton)	Direction	d (%)	$d \times 100$ (m^2/ton)	T	Type of Coupling	C_y
A	4	358.1	X	1.9	0.63	0.154	M	0.33
			Y	2.5	0.66	0.163	C_p	0.50
C	3	233.0	Y	3.2	1.69	0.101	U	0.76
G	3	187.8	Y	3.3	1.58	0.103	C_p	0.80
H	4	741.3	X	2.8	1.06	0.195	U	0.54
I	3	163.8	Y	3.5	1.99	0.117	C_p	0.99
J	3	142.9	X	5.1	2.31	0.081	U	0.98
K	4	144.5	X	2.7	1.20	0.262	U	0.52
N	4	376.6	X	2.9	0.71	0.16	U	0.43
O	3	235.3	X	3.5	1.62	0.09	U	0.71
P	4	387.5	X	3.4	1.08	0.177	U	0.56
Q	4	321.0	X	3.8	1.28	0.123	U	0.64
1.A	3	209.4	X	2.4	1.40	0.089	C_p	0.64
			Y	3.2	1.90	0.099	U	0.85
1.B	4	287.1	X	2.4	1.00	0.122	C_p	0.50
			Y	3.2	1.40	0.147	U	0.66
2.A	3	115.5	X	2.1	0.90	0.12	M	0.44
			Y	3.4	1.50	0.096	U	0.70
2.B	4	158.7	X	2.1	0.70	0.169	M	0.35
			Y	3.4	1.10	0.137	U	0.55
3.A	3	222.9	Y	2.8	1.20	0.109	U	0.60
3.B	4	307.7	Y	2.8	0.90	0.16	U	0.47
4.A	3	271.3	X	2.6	1.80	0.064	M	0.76
			Y	2.4	1.70	0.087	U	0.72
4.B	4	391.5	X	2.6	1.20	0.093	M	0.57
			Y	2.4	1.20	0.135	U	0.54

BUILDING LAYOUT AND MODELING

The buildings are constructed mainly by confined masonry shear walls coupled by reinforced concrete lintels or masonry parapets and reinforced concrete slabs. The buildings correspond to actual three to four story dwellings that have been built in Chile in recent decades, in accordance with the recommendations of Chilean Code *NCh433.Of72*. They are regular, symmetrical in most of the cases, so that two independent directions are considered for the analysis, neglecting any torsional effects. Non-structural partitions are not considered as load bearing elements.

Table 3 shows some characteristics of the buildings analyzed, such as: the number of floors, N; the total reactive weight of the structure, W; the wall density , d; the wall density per unit weight per floor, d; the fundamental period T, the type of coupling between walls: U indicates the existence of only slab or concrete tie-beam, C_p indicates a masonry parapet that couple the walls and M a mixed condition; and C_y denotes shear strength by unit weight. In general, the shear strength by unit weight is rather high (> 0.5).

The Chilean Seismic Design of Buildings Codes *NCh433.Of96* allows static analysis for buildings up to 5 storys high. The design base shear, Q_o, is:

$$Q_o = CIW \tag{3}$$

where I, the importance factor equals 1 and the seismic coefficient is given by:

$$\frac{A_o}{6g} \le C = 2.75\,\frac{A_o}{gR}\left(\frac{T'}{T^*}\right)^n \le 0.55\,S\,\frac{A_o}{g} \tag{4}$$

where A_o is the effective ground acceleration, R is the structural modification factor and is equal to 4 for confined masonry, $T¢ S$ and n are parameters dependent on the soil conditions and T^*, is the structural period corresponding to the largest participation mass.

In this type of building, rather rigid, the maximum conditions for the base shear will dominate, so for soil type II (dense gravel) and seismic zone 3, the base shear coefficient is 0.22.

According to *NCh2123.Of97* (1997), the allowable shear stress for confined masonry wall, t_a, is given by:

$$\tau_a = \left(0.23\tau_m + 0.12\sigma_o\right) \leq 0.35\,\tau_m \qquad \textbf{(5)}$$

This value may be increased by 33% when seismic loads are considered, provided that one wall does not take more than 45% of the total shear load in each level.

Considering t_m equal 0.5 Mpa (73 psi), all the buildings analyzed with a wall density per unit weight per floor greater than 0.009 m^2/ton (0.0063 in.^{2}/lb) fulfill the *NCh433.Of96* requirements in seismic zone 3. It must be pointed out, that the Code aims to avoid collapse rather than to avoid cracking in the elements.

RESULTS

The displacement capacities of several confined masonry buildings have been determined by performing nonlinear static analyses. Moreover, the required displacements demanded by the action of several acceleration records have been evaluated by performing non-linear dynamic analyses.

Figure 2 shows the relationship between the buildings' maximum displacement capacity with the wall density per unit weight per floor when the story mechanism model is used. The wall with less displacement capacity controls the displacement capacity of the buildings, which depends mainly on the boundary conditions and the vertical load that contributes to it. Generally the walls in the facade will fail first.

In buildings where all the walls have the same boundary conditions, the displacement capacity diminishes with increasing wall density. Buildings with uncoupled walls have larger displacement capacity. On the other hand, more flexible buildings have larger displacement capacity but less overstrength.

A plan with walls with different displacement capacities allows the structure to reduce its resistance gradually. Therefore it is not wise to have a very stiff wall with small displacement capacity because it reduces the displacement capacity of the building. Unfortunately this happens very often in buildings that have a very stiff median wall.

The same results are obtained from push-over analysis; when the structure response is controlled by shear behavior, the upper floors do not have any influence in the displacement capacity. Flexural behavior affects only buildings with slender walls; in this case the displacement capacity may be increased, but for that to occur some horizontal reinforcement must be placed in the masonry in order to increase the shear resistance of the walls.

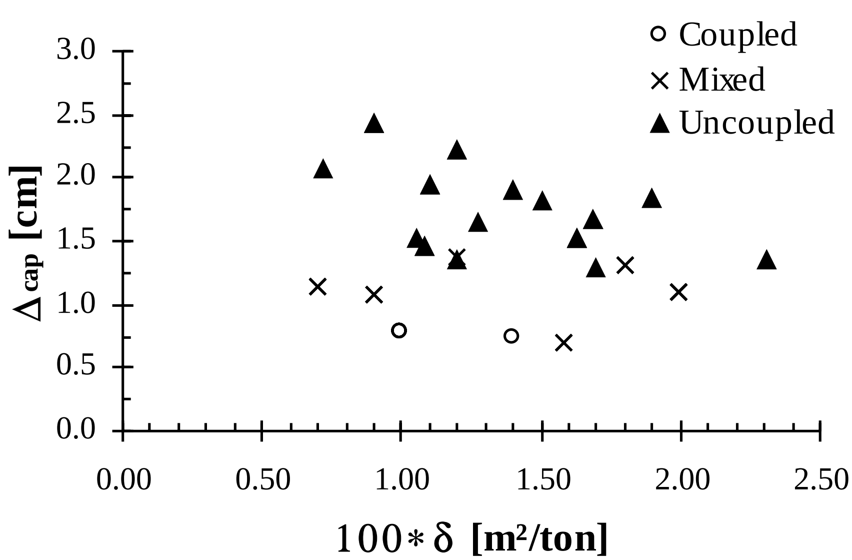

Figure 2 —Displacement capacity vs Wall Density d (1 cm = 0.39 in., 1 m^2/ton = 0.7 in.^{2}/lb)

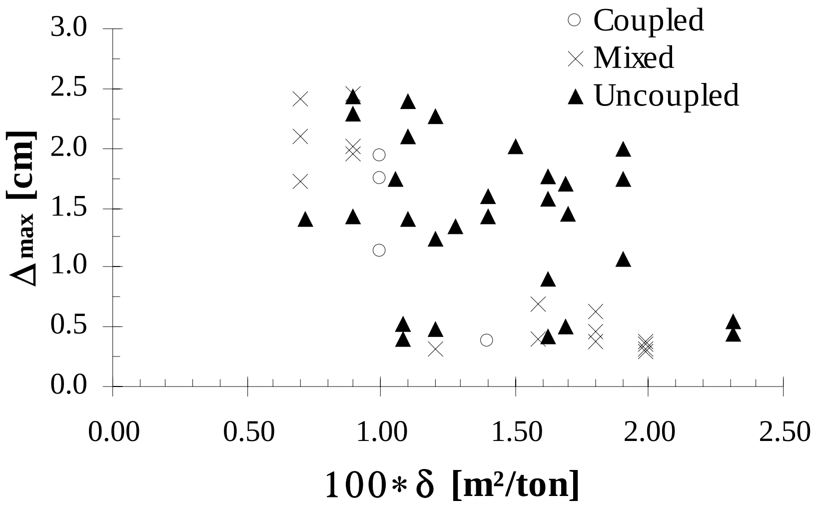

Figure 3—Displacement Demand vs Wall Density d(1 cm = 0.39 in., 1 m²/ton = 0.7 in.²/lb)

Figure 3 shows the variations of displacement demands in the first floor as a function of wall density per unit weight per floor. In general, demands are larger for buildings with uncoupled walls. Figure 4 shows the ratio between the required displacement, D_{max}, and the displacement capacity, D_{cap}, of each building, D_{max}/D_{cap}, as a function of wall density per unit weight per floor; in order to have values of D_{max}/D_{cap} less than one d must be greater than 0.012 m²/ton (0.0084 in.²/lb).

Requirements concerning displacement in the Chilean seismic code *NCh433.Of96* refer to the following types of condition: serviceability requirements in order to avoid damage in non-structural elements and torsional effects, and requirements to avoid pounding between two adjacent buildings. In the first case the story drift measured at the center of mass must be less than 0.002 times the story height; the story drift measured in any other point of the building must not exceed 0.001 times the story height the story drift of the center of mass. For the buildings analyzed in this research, this means maximum story drifts of 4.5-5 mm (0.177 - 0.197 in.), a condition that is fulfilled by all of them.

In the second case, in any level, the distance between a building and the median plane between two buildings must be larger than $R/3$ times the elastic displacement determined from prescribed lateral seismic forces, or 0.002 times the story height or 1.5 cm (0.59 in.), the latter being the condition that dominates.

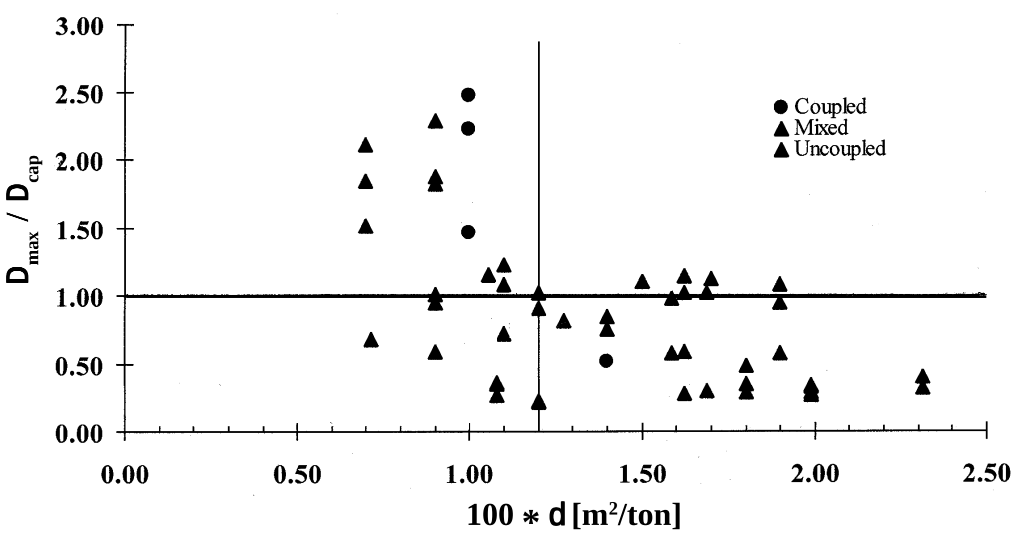

Figure 4—Ratio D_{max}/D_{cap} vs Wall Density d(1 m²/ton = 0.7 in.²/lb)

CONCLUSIONS

The displacement capacity of several confined masonry buildings has been determined by performing non-linear static analyses. The main conclusion obtained from these analyses is that the displacement capacity of the building depends on the wall density per unit weight per floor and on the coupling between shear walls. There is a trade-off between the resistance and the displacement capacity provided in the structure.

The required displacement D_{max} at the first floor has been evaluated by performing time-history analyses of several buildings subjected to the action of several acceleration records. Limitations on story drift were imposed in order to obtain realistic results.

The wall density per unit weight per floor is a good indicator of the expected seismic behavior for this type of building. Hence, minor nonlinear behavior will be required in buildings with high wall density, and as a consequence, the level of damage, if any, would be rather low.

To guarantee that the displacement capacity be greater than the displacement demand the wall density per unit weight per floor must be around 0.012 m^2/ton (0.0084 $in.^2$/lb).

On the other hand, observed structural performance of confined masonry buildings in past earthquakes suggests a wall density per unit weight per floor greater than 0.013 m^2/ton (0.0091$in.^2$/lb) in order to ensure the occurrence of only moderate damage, and greater than 0.008 m^2/ton (0.0056 $in.^2$/lb) if only heavily damage is tried to avoid.

Seismic forces prescribed by *NCh433.Of96* to confined masonry buildings requires wall density per unit of floor and weight greater than 0.009 m^2/ton (0.0063 $in.^2$/lb) in seismic zone 3, which means that the buildings may be exposed to moderate damage when they are subjected to severe earthquakes (*IMM* > 8).

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REFERENCES

Miranda, E., "Assessment of the Seismic Vulnerability of Existing Buildings", Proc. XI WCEE, Acapulco, Mexico, Paper Nº 513, 1996.

Meli, R., "Diseño sísmico de edificios de mampostería; la práctica actual y el comportamiento observado," Simposium Internacional Seguridad Sísmica en la Vivienda Económica, México D.F., México, 1991, pp. 398-423.

Tomazevic, M., "Dynamic modeling of masonry buildings: story mechanism model as a simple alternative," Earthquake Engineering & Structural Dynamics, 15, 1987, pp. 731-749.

Norma Chilena *NCh2123.Of97*, "Diseño Sísmico de edificios," Instituto Nacional de Normalización, Chile, 1997.

Moroni, M. O., Astroza, M. and Tavonatti, S., "Nonlinear Models for Shear Failure in Confined Masonry Walls," TMS Journal, 12, TMS, Boulder, 1994, pp. 72-78.

Saragoni, R., Celebi, M., Holmberg, A., and Saéz, A., "Análisis de los acelerogramas del terremoto de Chile de 1985, Segunda parte," Proc. 5th Jornadas Chilenas de Sismología e Ingeniería Antisísmica, ACHISINA, Santiago, Chile, Vol 1, 1989, pp. 357-368.

Astroza, M., Moroni, M., Kupfer, M., "Calificación Sísmica de edificios de albañilería de ladrillo confinadas con elementos de hormigón armado," Proc. XXVI Jornadas Sudamericanas de Ingeniería Estructural, Noviembre 1993, Uruguay, pp. I 327-338.

Norma Chilena *NCh2123.Of97*, "Albañilería Confinada-Requisitos para el diseño y cálculo," Instituto Nacional de Normalización, Chile, 1996

NOTATIONS

A_{mi} = shear wall area.
A_o = effective ground acceleration.
A_p = floor area.
A_t = total shear wall area in one direction.
C = seismic coefficient.
C_e = elastic base shear ratio.
C_p = indicates a masonry parapet that couple the walls.
C_y = yield shear strength of the structure defined as the sum of maximum shear wall capacities in each direction.
d = wall density.
F_i = factor that reduces the contribution of slender walls.
F_2 = shear wall capacity.
g = acceleration due to gravity.
H_i = wall height.
I = importance factor.
L_i = wall length.
M = indicates a mixed condition of coupling between walls.
M_s = magnitude.
n = prameter dependent on soil conditions.
N = number of floors.

N_d = level of damage.
Q_o = design base shear.
R = structural modification factor.
S = parameter dependent on soil conditions.
T = fundamental period.
T¢ = parameter dependent on soil conditions.
T^* = structural period corresponding to the largest participation mass.
U = indicates the existence of only slab or concrete tie-beam.
w = average floor weight.
W = total reactive weight of the structure.
d = wall density per unit weight per floor.
D_{cap} = displacement capacity.
D_{max} = required displacement.
s_o = normal stress due to verical loads.
t_a = allowable shear stress for confined masonry wall.
t_m = masonry shear strength.