SEISMIC BEHAVIOR QUALIFICATION METHODOLOGY FOR CONFINED MASONRY BUILDINGS

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SUMMARY

In order to evaluate seismic performance of confined masonry buildings a methodology that compares displacement demands imposed by real earthquakes and their displacement capacities is proposed. The wall density per unit floor and unit weight turns up to be the more appropriate qualification index. The way in which this wall density index relates with strength requirements included in the Chilean Seismic Code and with observed building damage in past earthquake, like the earthquakes occurred in Chile and Mexico in 1985, is also addressed.

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. In order to ensure the occurrence of only moderate damage, when the materials fulfill the Chilean Code requirements, the wall density per unit weight per floor must be around 0,012 m^2/ton.

INTRODUCTION

Confined masonry buildings consist of load-bearing unreinforced masonry walls, commonly made of clay units and cement mortar, confined by cast-in-place reinforced concrete vertical tie-columns. These tie-columns are located at regular intervals and connected together with reinforced concrete horizontal tie-beams. The system has been widely used for dwellings and apartment buildings up to four stories high in Chile. Tie-columns have a rectangular section whose dimensions typically correspond to the wall thickness (150 to 200 mm) and a depth usually equal to 200 mm . Similarly, tie-beams width is equal to the wall thickness and the depth is usually equal to 200 mm. Both tie-columns and tie-beams have minimum four 10 mm diameter longitudinal reinforcement and stirrups of 6 mm diameter spacing at 100 to 200 mm.

The system is such that walls must resist both vertical and lateral loads. The tie-columns have the longitudinal reinforcement necessary to resist overturning moments and the confinement effect, due to tie-column and tie-beam, improves the wall displacement capacity and the seismic behavior under cyclic lateral loads as compared with unreinforced masonry panels, and ensures the connections between walls and between walls and floor diaphragms. Floor systems generally consist of cast-in-place reinforced concrete slabs with a thickness between 100 and 120 mm.

Confined masonry walls have limited shear strength and ductility as compared to common reinforced concrete walls; nevertheless, typical housing buildings have good earthquake resistance, because they have high wall densities and wall layouts are symmetric and regular, both in plan and in elevation. Their seismic behavior has been satisfactory, particularly in one or two stories high buildings during strong earthquake [Monge, 1969].

In this paper a minimum wall density per unit weight per floor (δ) for three and four stories high confined masonry buildings is determined taking in account the displacement capacity of the buildings and the required displacement when these structures are submitted to the action of real earthquakes.The wall density per unit

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

The wall density, d, is defined according to Meli [Meli, 1991] as the ratio between the total shear wall area in one direction, A_m, and the floor area A_p:

$$d = \frac{\sum F_i A_{mi}}{A_p} \qquad (1)$$

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 displacement capacity of confined masonry buildings is determined using the story mechanism model [Tomazevic, 1987] and push-over analysis. Displacement demands imposed by real earthquakes are obtained through non-linear dynamic analysis.

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 [Mesias, 1996] and push over analysis [Mosqueira, 1998]. In both, the hysteresis envelope of the building's first story, which represents the relationship between the acting lateral seismic load and the lateral displacement of the first story, is obtained. In that determination the following assumptions are made [Moroni et al., 1996]:

- 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-end if masonry 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 modules 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 shear strength of the building has been reduced by about 20%. Different criteria were used to identify the conditions that generate strength reduction in the building: (i) maximum shear deformation has been attained by all the walls, (ii) maximum shear deformation has been attained by any wall.

An inverted triangular load pattern was used with push-over analysis. In this case the non-linear behavior of the walls included shear and flexural stiffness degradation. Increasing the shear wall capacity using prefabricated wire joint reinforcement, like ladder type, the shear capacity of the building can be increased up to 60%, therefore its displacement capacity will also increase.

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. For this type of failure and based on experimental results, a simple analytical model to predict the inelastic response of confined masonry walls has been proposed [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; the most important parameters of the primary curve are: the initial shear stiffness, the shear strength and the shear deformation capacity.

Tomazevic [Tomazevic, 1997] has tested two scale models of a three story typical confined masonry building on a shaking table and their results has been used to validate the computer model. Both scale models have been tested by subjecting them to a sequence with gradually increased intensity of motions in each successive test run up, named R5 to R200, until the final collapse of them. For the tests the original Petrovac record of April 15 - Montenegro earthquake with a peak acceleration equal to 0,43 g and Arias intensity of 5,0 m/sec, was used. The original building's wall density per unit weight per floor in the principal direction of the plan are 0,0156 m^2/ton and 0,0149 m^2/ton, for the scale models are 0,0708 m^2/ton (Model M1) and 0,065 m^2/ton (Model M2). The scale models were analyzed with DRAIN-2D using different structural modelation´s criteria to consider the coupling between the structural elements and two differents primary curves, one based on Chilean experimental results (Nav-1) and the other proposed by Tomazevic (Tom-1), that includes the wall slenderness. Comparisons between experimental and calculated envelopes are presented in figure 1 [Salinas, 1999], in this figure a reasonable correlation is observed, in spite of the buildings have been subjected to the action of quite severe records and the theoretical model uses a constant damping and does not include strength degradation. In this figure, the model M1 represents a coupled wall condition and the model M2 represents an uncoupled wall condition.

Several time history analysis were performed considering non-linear behavior for different acceleration records, and assuming 5% critical damping. Fixed base condition has been considered for all the buildings.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 [INN, 1996]. According to its destructiveness potential, the first three represent a collapse condition while the latter a serviceability condition [Saragoni et al, 1989].

DAMAGE OBSERVATION

After the March 3, 1985 earthquake, damage that occurred in masonry buildings located in Santiago (Mercalli Modified Intensity between 7 and 8 degree) was surveyed. The damage level was classified in five categories according to Table 1. Category 4 corresponds to a state of damage that can't be repaired, but it represents an accepted level of damage by the seismic design code when only collapse is avoided.

An empirical relation has been established 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 have also been used [Meli, 1991].

Assuming an average floor weight per unit of floor area of 6.50 kPa, the preceding relation may be expressed in terms of the wall density per unit weight per floor. In this case a value greater than 0.013 m^2/ton would be necessary in order to ensure the occurrence of only moderate damage, and greater than 0.008 m^2/ton if only heavy damage is to be avoided.

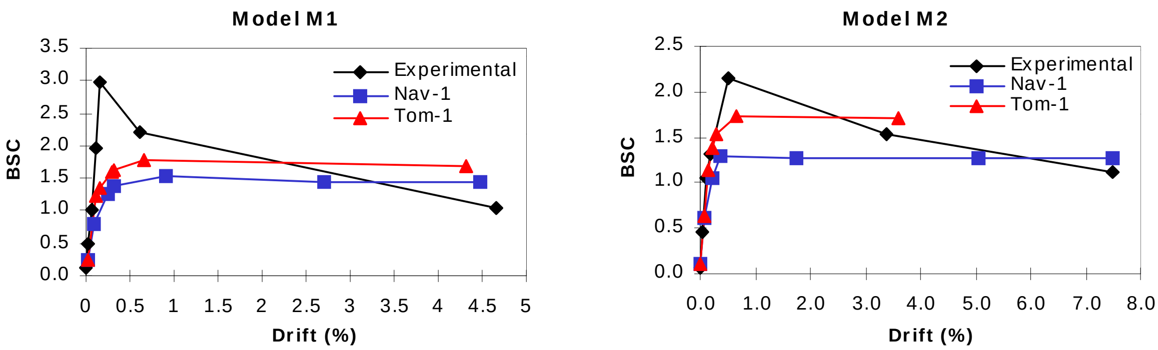

Fig.1: Base Shear Coeficient v/s First Story Drift

BUILDINGS LAYOUT

The buildings are constructed mainly by confined masonry shear walls coupled by reinforced concrete lintels or masonry parapets and reinforced concrete slabs. They correspond to actual three to four story high buildings that have been built in Chile in recent decades, in accordance with the recommendations of Chilean Code NCh433 [INN, 1996]. They are regular, symmetrical in most of the cases, so that two independent directions are considered for the analysis, neglecting any torsional effects.

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, δ; the fundamental period T, the type of coupling between walls: U indicates the existence of only slab or concrete tie-beam, C indicates a masonry parapets 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).

Considering the nominal masonry shear strength ,τ_m , equal 0.5 MPa, all the buildings analyzed with a wall density per unit weight per floor greater than 0.009 m^2/ton 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.

TABLE Nº 1 Damage Categories

Category	Damage extension	Action to take
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 architectural 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 notably. Generalized failures in non-structural elements.	It is not necessary to evacuate the building. Only architectural 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 architectural 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.

TABLE Nº 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	$\geq$ 1.15
Moderate	2	0.85 - 1.15
Severe	3	0.5 - 0.85
Heavy	4-5	$\leq$ 0.5

TABLE 3. Buildings Characteristics

Building	n	W (ton)	Direction	d (%)	$\delta\times100$ (m^2/ton)	T	C.D	C_y
A	4	358.1	X Y	1.9 2.5	0.63 0.66	0.154 0.163	M C	0.33 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	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	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 Y	2.4 3.2	1.40 1.90	0.089 0.099	C U	0.64 0.85
1.B	4	287.1	X Y	2.4 3.2	1.00 1.40	0.122 0.147	C U	0.50 0.66
2.A	3	115.5	X Y	2.1 3.4	0.90 1.50	0.12 0.096	M U	0.44 0.70
2.B	4	158.7	X Y	2.1 3.4	0.70 1.10	0.169 0.137	M U	0.35 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 Y	2.6 2.4	1.80 1.70	0.064 0.087	M U	0.76 0.72
4.B	4	391.5	X Y	2.6 2.4	1.20 1.20	0.093 0.135	M U	0.57 0.54

RESULTS

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

The displacement capacity of the buildings is controlled by the wall with less displacement capacity, while the latter depends mainly on the boundary conditions and the vertical load that contributes to it. Generally the walls in the facade will fail first.

A plan with walls with different displacement capacities allows the structure to reduce its resistance gradually. Therefore it is not wise to have walls with equal displacement capacities or to have a very stiff wall with small displacement capacity because it reduces the displacement capacity of the building suddenly. 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 don't 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 shows the relationship between the buildings´maximun displacement capacity with the wall density per unit weight per floor when the story mechanism model is used. 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 and the displacement capacity of each building, $\Delta_{max}/\Delta_{cap}$, as a function of wall density per unit weight per floor; in order to have values of $\Delta_{max}/\Delta_{cap}$ less than one δ must be greater than 0.012 m^2/ton.

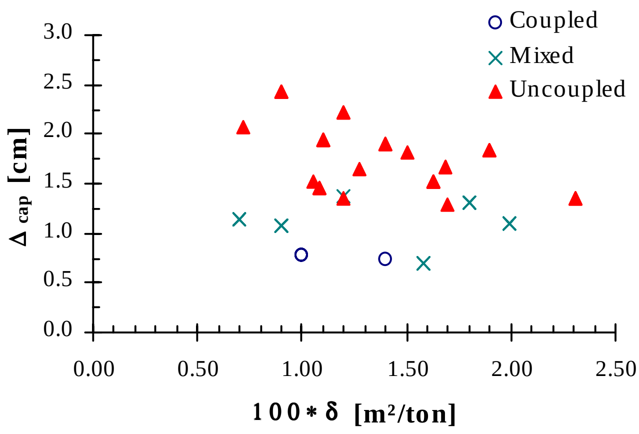

Fig.2: Displacement capacity v/s Wall density

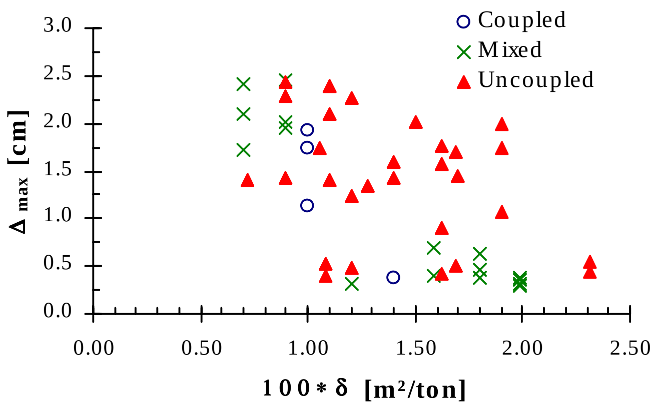

Fig.3: Displacement demand v/s Wall density

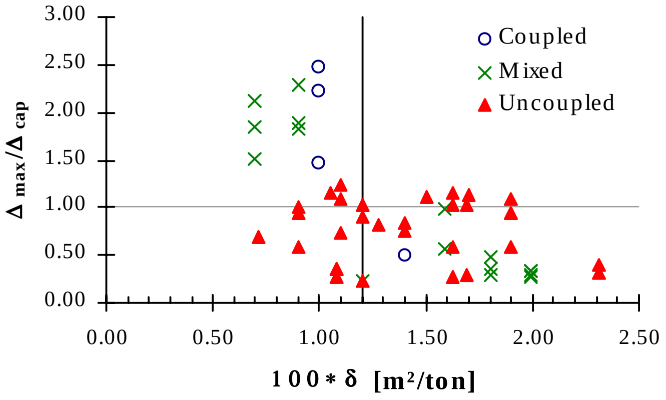

Fig.4: Ratio Δmax/Δcap v/s Density δ

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.

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.

Although the model used to estimate displacement demands may be conservative, a reasonable conclusion is that to guarantee 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.

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 in order to ensure the occurrence of only moderate damage, and greater than 0.008 m^2/ton if only heavy damage is tried to avoid.

Seismic forces prescribed by NCh433.Of96 to confined masonry buildings requires wall density per unit weight per floor greater than 0.009 m^2/ton 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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