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DISPLACEMENT CAPACITY AND REQUIRED STORY DRIFT IN CONFINED MASONRY BUILDINGS

MARIA MORONI, MAXIMILIANO ASTROZA, PAULA MESIAS

Dept. of Civil Engineering, University of Chile
Casilla 228/3, Santiago, Chile

ABSTRACT

In this paper the displacement capacity of confined masonry buildings is determined using the storey mechanism model and the displacement capacity of each wall. In addition, required displacements obtained through non-linear dynamic analysis of buildings subjected to the action of different records of strong earthquakes are related to those obtained for a one degree system subjected to the same earthquakes.

The displacement capacity of the buildings depends on the wall density and the coupling between shear walls. The required displacement Δ_{max} can be estimated as 1.8 S_d, the elastic displacement spectra, relationship that represents an upper boundary.

KEYWORDS

Confined Masonry; Displacement Capacity; Required Displacement; Seismic Design

INTRODUCTION

Seismic design provisions are developed aim to ensure serviceability requirements during frequent moderate earthquakes and life safety during major earthquakes. Therefore, in the latter case extensive damage to the structure may be acceptable as long as collapse is prevented.
According to that, the following steps could be performed as a design procedure:
i.- Linear elastic analysis with a reduced response spectra. Conventional design always includes this step.
ii.- Non linear static analysis to evaluate the ultimate strength and the displacement capacity of the structure and to verify that the failure mechanism is appropriate. Failure mechanism and seismic resistance can be determined by a non linear static analysis with monotonic increasing lateral loads, distributed uniformly or triangular in height. In the case of masonry structures the so called storey mechanism model has been proposed to calculate storey hysteresis envelope (Tomazevic, 1987).
iii.- To compare the displacement capacity of the structure with those required by major earthquakes. Elastic displacement spectra and expected maximum inelastic displacements for one degree system with different strength level have been computed for several earthquakes (Saez and Holmberg, 1990, Naeim, 1993). This information must be the base to estimate the requirements on real structures.

In this paper the displacement capacity of confined masonry buildings is determined using the storey mechanism model and the displacement capacity of each wall. The storey mechanism model is applied to the lower floor of the building which is the most critical. The redistribution of seismic loading from one wall to another is carried out by assuming that the walls as a whole, and not their critical sections, behave as ductile structural elements.

In addition, required displacements obtained through non-linear dynamic analysis of buildings subjected to the action of different records of strong earthquakes are related to those obtained for a one degree system subjected to the same earthquakes considering linear and non-linear behavior.

DETERMINATION OF THE DISPLACEMENT CAPACITY

The displacement capacity of each building is obtained through the hysteresis envelope (Tomazevic, 1991) of the building's first storey, which represents the relationship between the acting lateral seismic loading and the lateral deformations of the first story. 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 storey 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 storey shear is distributed into the individual walls according to their stiffness. Two type of boundary conditions are considered: fixed-ended 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, as is shown in Fig. 1.

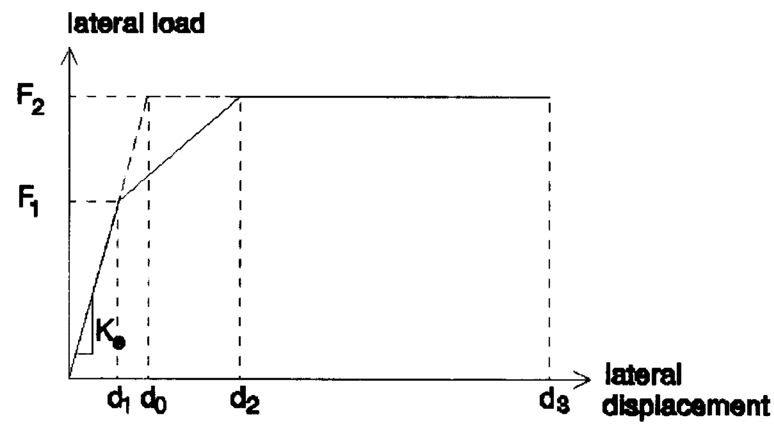

Fig. 1. Hysteresis envelope of each wall

Load level F_1 and F_2 represent the allowable and maximum shear capacities of the walls; depend on the masonry shear strength τ_m and the vertical applied loads σ_o; are given by

$$F_1=(0.23\tau_m+0.12\sigma_o)A_a\leq 0.35\tau_m A_a \tag{1}$$

$$F_2=(0.45\tau_m+0.30\sigma_o)A_a\leq 1.50\tau_m A_a \tag{2}$$

There are several possible definitions for the elastic deformation limit, d_o, and the ultimate deformation limit, d_3. In this case the elastic limit is defined as the displacement of the equivalent elasto-plastic system with initial stiffness, K_e, and maximum load resistance, F_2. The ultimate displacement corresponds to the displacement when the strength has been reduced in about 20%; the same criteria is used to determine the displacement capacity of the building. For each individual wall, $d_2 = 3\ d_o$, and $d_3 = 10\ d_o$, values that provide a best fit to experimental data.

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.

DETERMINATION OF REQUIRED DISPLACEMENT

Several pseudo 3-D time history analysis were performed considering non-linear behavior for different acceleration records, and assuming 5% critical damping. The walls are represented by a flexible bar coupled with a shear spring that has been incorporated into program DRAIN-TABS (1977). Rigid diaphragm in all floors has been considered. Non-linear behavior is restricted to the shear spring, which is characterized by a trilinear primary curve and hysteresis loop that includes stiffness degradation (Moroni et al., 1994). This model represents seismic behavior observed in confined masonry building during earthquakes as well as in wall testing when most of the damage has been due to shear failure.

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 ($M_s = 7.8$), and represent different soil conditions, intensities, I, and destructiveness potential, P_D. Table 1 contains some characteristics of those records.

Table 1. Seismic records characteristics

Record	Peak accel (g)	P_D (10^{-4} gseg^3)	Soil Cond.	I (MM)
Llolleo N10E	0.669	201.57	sand	8-9
Llolleo S80E	0.426	80.29	sand	8-9
Melipilla NS	0.68	43.33	gravel	8
Melipilla EW	0.648	37.06	gravel	8

It is interesting to compare the maximum displacement obtain at the first floor with the elastic displacement spectra of every record and with maximum displacement of single degree of freedom considering non-linear behavior. The data available were obtained for elasto-plastic models with yield levels related to the design base shear proposed in the SEAOC (1968) and to the design base shear proposed in the NCH433.Of93. Neither correspond exactly to the strength level of the buildings analyzed in this work.

BUILDINGS LAYOUT

The buildings correspond to typical 3 to 4 story height dwellings that have been built in Chile in the past years. They are structured mainly by confined masonry shear walls coupled by reinforced concrete lintels or masonry parapets and reinforced concrete slabs. Figure 2 shows the plan layout of some of the buildings; all dimensions are in mm. Table 2 shows some characteristics of the buildings analyzed, such as the number of floors N, the story height, h, the weight of the reactive mass, W, the ratio of walls to floor area in both directions, $\Sigma\ A_w/A_p$, and the wall densities δ_x and δ_y, computed as the ratio of the wall cross-sectional area in the first floor to the total weight of the structure. The two last columns may be related to the strength of the buildings.

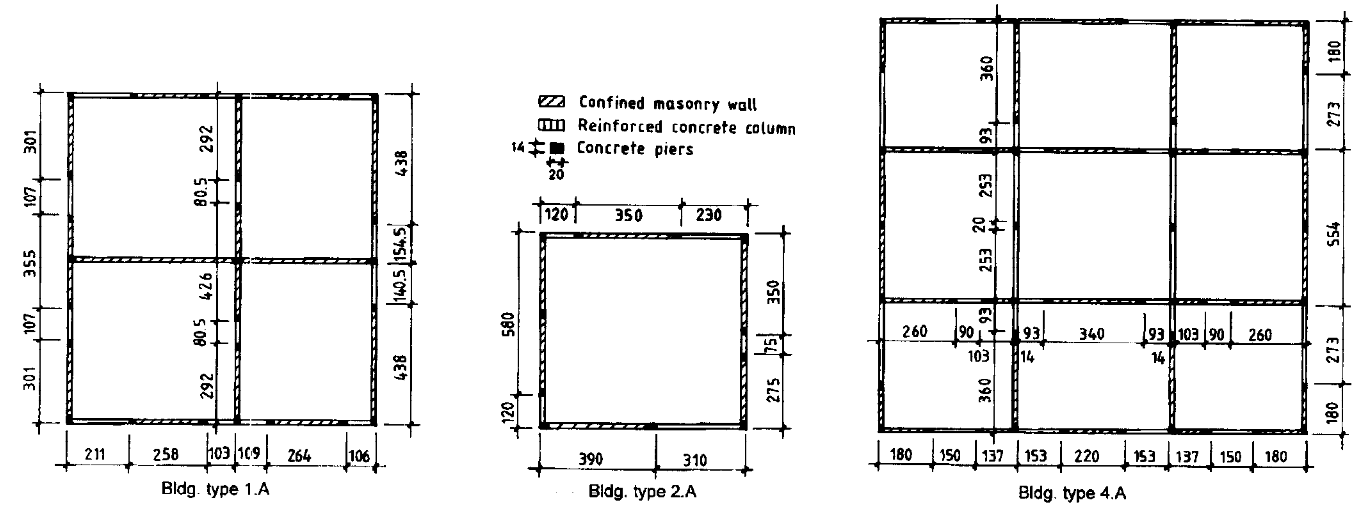

Fig. 2. Plan layout of buildings

Table 2. Buildings characteristics

Building	N	h (m)	W (ton)	$\Sigma A_w/A_p$ (%) X Y	δ_x - δ_y (m^2/ton)
A	4	2.40	358.1	1.9 - 2.5	0.63 - 0.66
B	4	2.60	567.8	1.3 - 2.3	0.30 - 0.58
C	3	2.47	233.0	2.2 - 3.2	1.18 - 1.69
D	3	2.30	352.1	2.5 - 2.9	1.30 - 1.38
E	4	2.52	1013.3	2.1 - 1.3	0.60 - 0.34
F	4	2.52	293.7	1.1 - 3.9	0.48 - 1.51
G	3	2.42	187.8	2.7 - 3.3	1.32 - 1.58
H	4	2.47	741.3	2.8 - 3.9	1.06 - 0.58
I	3	2.42	163.8	2.4 - 3.5	1.53 - 1.99
J	3	2.48	142.9	5.1 - 2.4	2.31 - 0.99
K	4	2.40	144.5	2.7 - 1.8	1.20 - 0.63
L	3	2.50	467.1	1.3 - 1.3	0.70 - 0.59
M	3	2.38	179.7	4.8 - 1.8	2.62 - 0.77
N	4	2.45	376.6	2.9 - 1.4	0.71 - 0.43
O	3	2.42	235.3	3.5 - 1.4	1.62 - 0.71
P	4	2.42	387.5	3.4 - 2.2	1.08 - 0.71
Q	4	2.46	321.0	3.8 - 2.4	1.28 - 0.89
S	4	2.45	286.1	2.4 - 2.2	0.97 - 0.91
T	4	2.45	1097.6	2.8 - 2.2	0.88 - 0.68
U	4	2.45	431.0	3.1 - 2.4	1.38 - 1.11
V	4	2.40	131.1	3.8 - 2.7	1.80 - 1.17
1.A	3	2.47	209.4	2.4 - 3.2	1.40 - 1.90
1.B	4	2.47	287.1	2.4 - 3.2	1.00 - 1.40
2.A	3	2.45	115.5	2.1 - 3.4	0.90 - 1.50
2.B	4	2.45	158.7	2.1 - 3.4	0.70 - 1.10
3.A	3	2.45	222.9	1.1 - 2.8	0.80 - 1.20
3.B	4	2.45	307.7	1.1 - 2.8	0.60 - 0.90
4.A	3	2.30	271.3	2.6 - 2.4	1.80 - 1.70
4.B	4	2.30	391.5	2.6 - 2.4	1.20 - 1.20

Table 3 shows the mechanical properties of the materials: Young modulus E; shear modulus, G; concrete compressive strength, f'_c; masonry compressive strength, f'_m; and masonry shear strength, τ_m. Buildings A to V used masonry type a and 1.A to 4.B used type b.

Table 3. Mechanical properties (Mpa)

Material	E	G	f'_c	f'_m	τ_m
concrete	28500.	11400.	22.5		
masonry (a)	6000.	1800.		6.	0.8
masonry (b)	5690.	1260.		12.2	0.8

RESULTS

The displacement capacity of several confined masonry buildings have been determined by performing non-linear static analysis. Moreover, the required displacement demanded by the action of several acceleration records have been evaluated by performing non-linear dynamic analysis. These records were assumed to act non-concurrently on the structure in the direction of each principal axis.

Figure 3 shows the relationship between the buildings' maximum displacement capacity with the wall density and Fig. 4 the relationship with the period of the buildings; coupled walls refers to the presence of masonry parapets, while uncoupled walls refers to the presence of slab or reinforced concrete beam. 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 tributes on it. Generally the walls in the facade will fail at first.

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

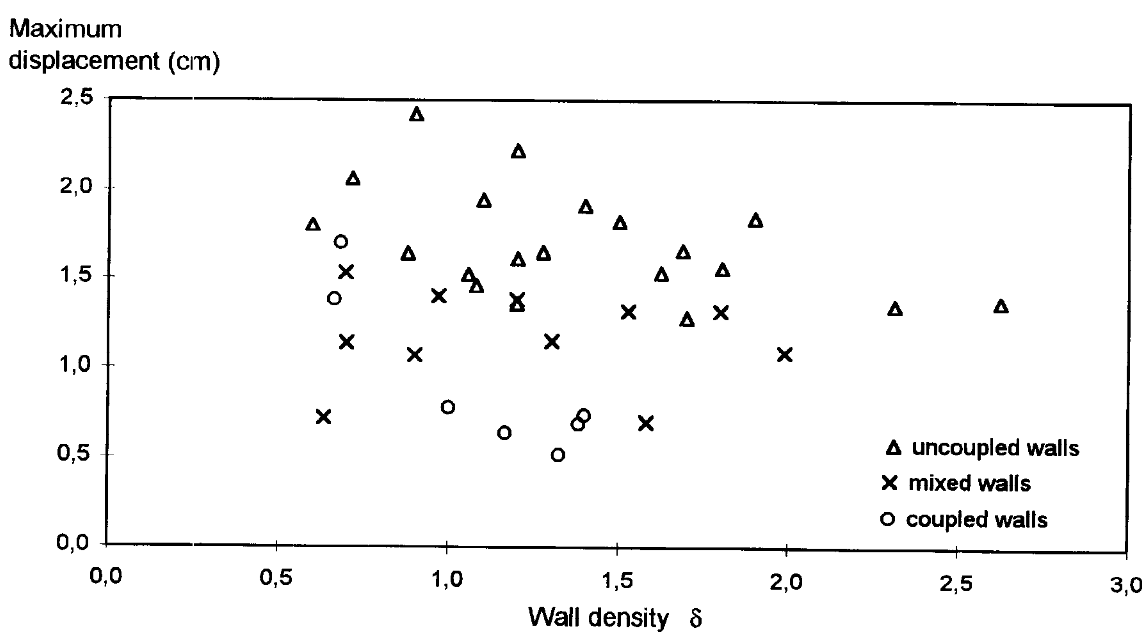

Fig. 3. Maximum displacement vs Wall density.

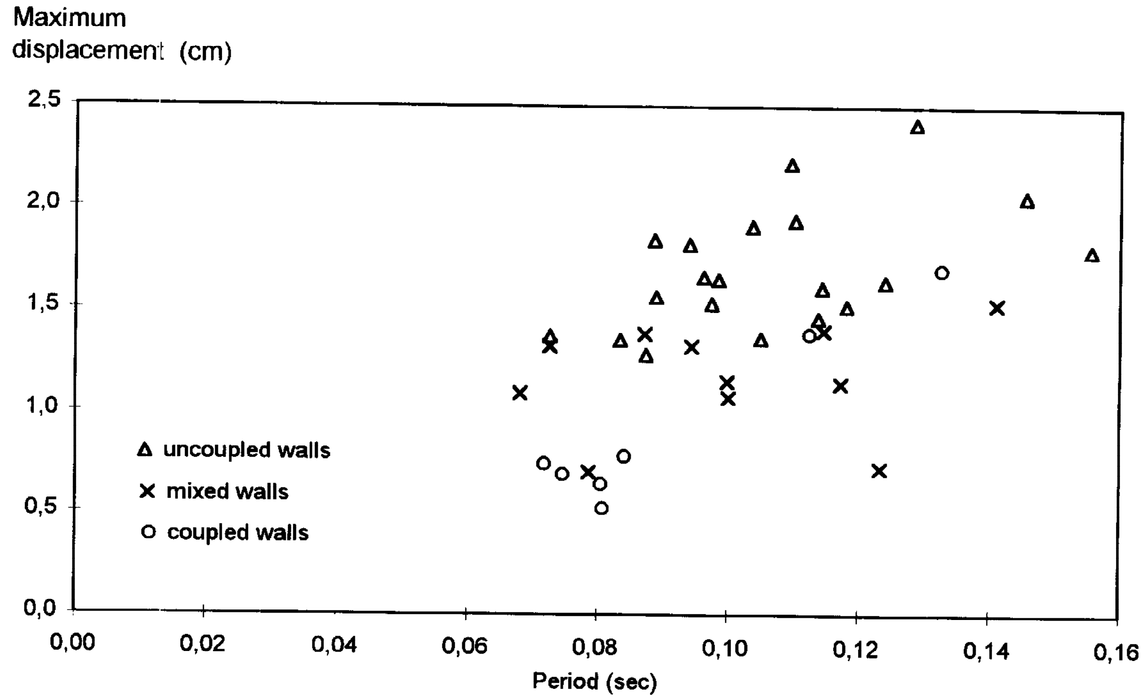

Fig. 4. Maximum displacement vs Period.

In Fig. 5 the maximum required displacements in the first floor, Δ_{max}, are compared with the elastic displacement spectra, S_d, obtained for each record and in Fig. 6 the average ratio of maximum displacement of SDOF evaluated considering non-linear behavior and two different yield strength to Δ_{max} is presented. The maximum shear capacity of the buildings varies between 0.33 to 1.17 W. Those cases where the ductility demands on individual walls were greater than their deformation capacities were not considered in these figures.

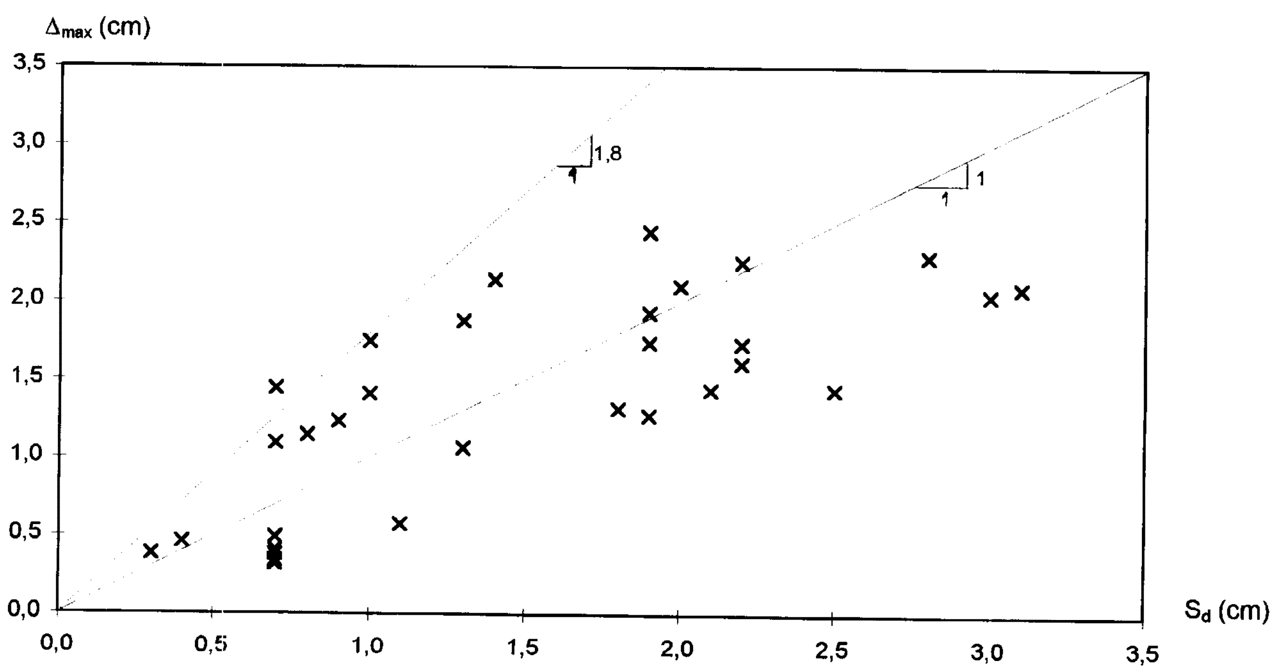

Fig. 5. Required displacement of building.

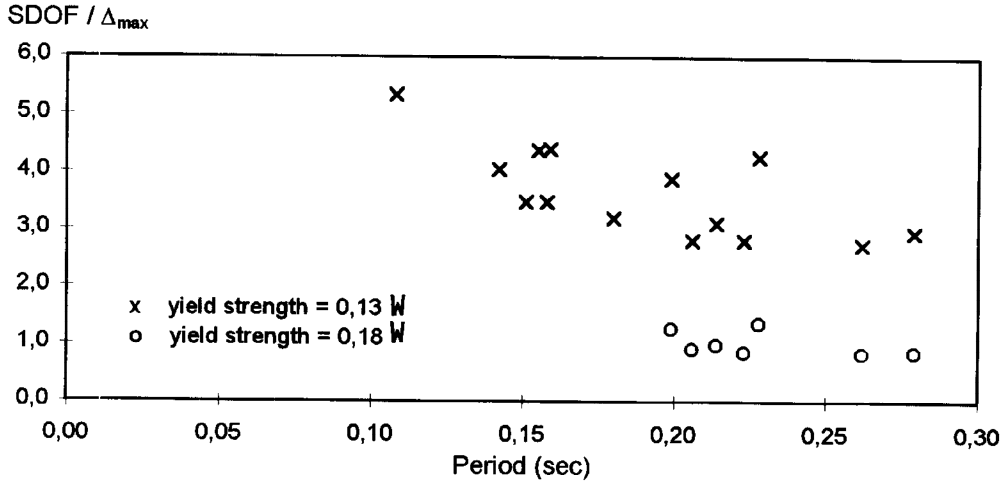

Fig. 6. Ratio of maximum displacement of SDOF over Δ_{max}.

CONCLUSIONS

The displacement capacity of several confined masonry buildings have been determined by performing non-linear static analysis. The main conclusion obtained from theses analysis is that the displacement capacity of the buildings depends on the wall density and the coupling between shear walls. There is a trade-off between the resistance and the displacement capacity provided to the structure.

The required displacement Δ_{max} at the first floor have been evaluated by performing pseudo 3-D time-history analysis of several buildings subjected to the action of several acceleration records. Limitations on story drift were imposed in order to obtain feasible results. The required displacement can be estimated as 1.8 S_d, the elastic displacement spectra, relationship that represents an upper boundary. Although, there is not complete information available on the non-linear behavior of SDOF system some relations are possible to establish depending on the yield level considered for the SDOF system.

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