MODEL FOR THE NONLINEAR ANALYSIS OF CONFINED MASONRY BUILDINGS

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ABSTRACT :

This paper presents a practical displacement-based evaluation procedure for the seismic assessment of low-height confined masonry buildings. First, the *Coefficient Method* is adapted to obtain rapid estimates of inelastic roof displacement demands for regular confined masonry buildings. Second, a nonlinear simplified model is introduced to perform pushover analysis of confined masonry buildings whose global and local behavior is dominated by shear deformations in the masonry walls. The model, which can be applied through the use of commercial software, can be used to estimate the capacity curve of a confined masonry building.

KEYWORDS: confined masonry, displacement-based evaluation, pushover analysis, capacity curve

1. INTRODUCTION

Nowadays, more practicing engineers use displacement-based procedures for the seismic evaluation of existing structures and for the preliminary design of new structures. The practical objective of a displacement-based procedure is to predict the expected performance of a structure in future earthquake shaking. For this purpose, performance-based formats characterize performance in terms of damage to structural and non-structural components. Since structural damage implies inelastic behavior, evaluation procedures require nonlinear analysis techniques to estimate the magnitude of inelastic deformations demands. Subsequently these demands are used to determine performance based on previously established acceptance criteria. Thus, the application of the concept of performance-based design and evaluation can only be successful in reducing seismic risk if nonlinear analysis techniques are applied in an extensive manner to existing and new construction.

Extensive experimental evidence has shown that structural damage is a consequence of lateral displacement demands imposed on structures during earthquake ground shaking. Thus, modern performance-based seismic assessment procedures for existing structures are based on: a) the evaluation of the structure-specific lateral deformation capacity, and b) the earthquake-induced displacement demand. However, most seismic assessment procedures for masonry structures are still based on the evaluation of their lateral load-resisting capacity, and there are few proposals to move from force-based to displacement-based assessment methodologies (e.g. Calvi 1999, Glaister and Pinho 2003, Rodriguez 2005). This paper presents a practical displacement-based evaluation procedure for the seismic assessment of low-height confined masonry buildings. Although the presentation is limited to confined clay brick masonry walls (CM), typically used in Latin-America and other regions of the world to build low-height housing units, the evaluation procedure can be applied to any type of masonry construction.

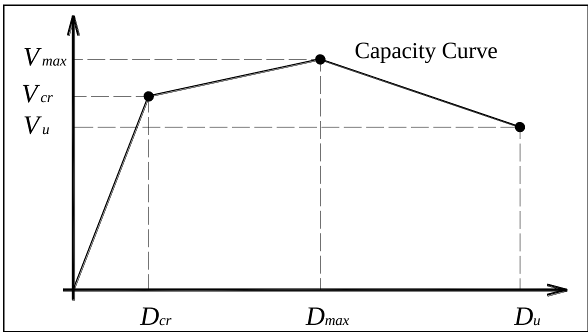
2. BASIS FOR A DISPLACEMENT-BASED APPROACH OF CM BUILDINGS

Based on ample experimental evidence derived from CM walls tested under in-plane lateral cyclic loading, Ruiz et al. (1998) established a relationship between an increase in lateral drift, the evolution of the crack pattern and the degradation of the structural properties (i.e. shear lateral strength and stiffness) of low-height CM walls. This relationship is summarized in Table 2.1. K_o and K represent the initial lateral elastic stiffness and the lateral stiffness associated to a particular value of inter-story drift (D), respectively; and V_{max} and V the maximum shear and the shear associated to a particular value of D, respectively. By using the information included in Table 2.1, it is possible to formulate displacement-based evaluation procedures for low-height CM buildings.

Table 2.1. Damage and degradation of confined masonry walls (Ruiz-García et al. 1998)

Observed damage	D (%)	K/K_o	V/V_{max}	Level of Damage
Flexural hairline horizontal cracking. Hairline vertical cracking near the tie-end RC columns.	0.04	0.8	0.5	Light
First diagonal cracking due to diagonal tensión in the masonry wall surface	0.13	0.35	0.85	Moderate
Beginning of the inclined diagonal cracking at the ends of the tie-end columns.	0.20	0.27	0.90	Heavy
Fully formed "X-shape" cracking on the masonry wall surface.	0.23	0.24	0.98	Heavy
Concrete crushing; horizontal cracking spread over the tie-end column height.	0.32	0.18	1.0	Heavy
Concentrated diagonal cracking at the end of tie-end columns. Concrete spalling in the tie-end columns.	0.42	0.13	0.99	Severe
Progression of diagonal cracking into the tie-end columns leading to rebar kicking of the longitudinal steel	0.50	0.10	0.80	Severe

The backbone curve of CM walls provides information that is fundamental for their structural assessment. As discussed in FEMA 440 (Federal Emergency Management Agency, 2005), this curve corresponds to the envelope of the hysteresis loops obtained experimentally in walls subjected to in-plane cyclic loading. In the case of low-height CM walls, their behavior tends to be dominated by shear deformations in such manner that their hysteretic behavior is characterized by significant cyclic and in-cycle strength degradation. Based on ample experimental evidence, Flores and Alcocer (1996) proposed a trilinear curve to characterize the backbone curve of CM walls. Figure 1 depicts the Flores and Alcocer model. While V_{cr} corresponds to the design shear strength of the wall established according to the Masonry Technical Requirements of the 2004 edition of the Mexico City Building Code (2004), h is the height of the wall.



Shear	Inter-story drift
V_{cr}	$D_{cr} = \frac{V_{cr}}{K_o h}$
$V_{max} = 1.25 V_{cr}$	$D_{max} = 0.003$
$V_u = 0.8 V_{cr}$	$D_u = 0.005$

Figure 1. Idealized backbone curve for confined masonry walls (after Flores and Alcocer 1996)

3. WIDE-COLUMN MODEL FOR THE ELASTIC ANALYSIS OF CM BUILDINGS

In recent years, Mexican practicing engineers have widely used the *wide-column* model for the analysis and design of CM buildings. According to this model a multi-story CM building can be idealized as bare frames. Each wall is modeled as an equivalent column that concentrates its flexural and shear properties on its centerline. In addition, equivalent beams having a width estimated according to the Masonry Technical Requirements of the Mexico City Building Code (2004), are used to model the coupling effect that the slab provides to the masonry walls.

The *wide-column* model has the potential to model the contributions of the masonry panel and of the confining tie-end reinforced concrete columns during the estimation of the mechanical properties of the CM wall. According to the wide-column model, the elastic lateral stiffness of the wall can be estimated as follows:

$$K = \left[\frac{h^3}{\beta EI} + \frac{h}{GA_V} \right]^{-1} \tag{3.1}$$

where h is the height of the wall; A_V and I the shear area and moment of inertia of its cross section, respectively; E and G the masonry's modulus of elasticity and shear modulus, respectively; and β a factor that accounts for the end-fixity of the wall.

Through Eqn. 3.1, it is possible to take into account the wall's aspect ratio and its end-support conditions to establish the relative contribution of the flexural and shear lateral deformations to the lateral response of CM walls. In general, shear deformations become more important than flexural deformations in walls having low aspect ratios, while flexural deformations governs the behavior of slender walls with high aspect ratios. Particularly, the lateral response of CM walls having both fix-end conditions and aspect ratios smaller than one is usually dominated by shear deformations. Independently of its aspect ratio, the effects of shear deformation tend to significantly increase relative to those associated to flexure as the level of damage increases in a CM wall (Zuñiga and Teran-Gilmore 2008).

Table 3.1 presents a comparison between the lateral stiffness obtained from the experimental response of three full-scale confined masonry specimens (designated as *W-W*, *WBW*, *WWW*) and a full-scale two-story tri-dimensional specimen (designated as 3D) tested under cyclic loading during a experimental program carried out in Mexico (Alcocer and Meli 1993, Ruiz 1995), and the corresponding stiffnesses computed from their *wide-column* models. The wide-column model is able to capture with reasonable approximation the elastic lateral stiffness of CM walls with different aspect-ratio and end-support conditions.

Table 3.1. Experimental and analytical lateral stiffness of confined masonry specimens

Specimen	Experimental (*Ton/cm*)			Analytical (*Ton/cm*)
	K_0 (+)	K_0 (-)	K_0 (Average)	
WW	113.51	104.48	109.00	104.77
WBW	88.07	88.07	88.07	95.12
WWW	128.09	144.19	136.14	101.56
3D	113.87	165.12	139.47	130.57

4. EVALUATION OF LATERAL DISPLACEMENT DEMANDS IN CM BUILDINGS

Nonlinear analysis procedures have been widely used by American practicing engineers since the publication of the ATC40 (Applied Technology Council 1996) and FEMA 273/274 guidelines (Federal Emergency Management Agency 1997). Particularly, the nonlinear static procedures (NSP) have become popular due to their simplicity and ability to provide useful insight regarding the expected performance of earthquake-resistant structures. Among the options available to estimate target displacement demands of existing structures is the *Coefficient Method*.

Field reconnaissance after earthquake events and experimental results have consistently shown that severe structural damage tends to concentrate in the first story of CM structures. Under these circumstances, the value of the coefficient C_0 involved in the Coefficient Method should be close to one for structures exhibiting heavy to severe damage. In addition, if as expected, low-height CM buildings do not exhibit significant $P-\Delta$ effects, coefficient C_3 could be neglected during the computation of the target displacement. Thus, taking into account that the inelastic displacement ratio C_R (Ruiz-Garcia and Miranda, 2003) can contemplate simultaneously the effects accounted for by parameters C_1 and C_2, the target roof displacement for a CM building can be estimated as:

$$\delta_T = C_O C_R S_a \frac{T_e^2}{4\pi^2} g \tag{4.1}$$

where C_0 tends to one as the level of damage in the structure increases, S_a is the pseudo-acceleration evaluated at T_e, and T_e is the effective fundamental period of the single-degree-of-freedom model of the structure.

4.1 Inelastic displacement ratio for CM buildings

The inelastic displacement ratio is defined as the maximum lateral inelastic displacement demand, Δ_i, normalized by the maximum lateral elastic displacement demand, Δ_e, of single-degree-of-freedom systems having the same mass, same first-mode period of vibration (i.e. initial stiffness), and same damping ratio when subjected to a given earthquake acceleration time-history. This ratio can be expressed as follows:

$$C_R = \frac{\Delta_i}{\Delta_e} \tag{4.2}$$

In the above equation, Δ_i is computed in systems with constant yielding strength relative to the strength required to maintain the system elastic (i.e. constant relative strength). The relative lateral strength is usually measured through the lateral strength ratio, R , defined as:

$$R = \frac{m\,S_a}{V_y} \tag{4.3}$$

Where m is the mass of the system, and V_y the lateral yield strength of the system. The numerator in Eqn. 4.3 represents the lateral strength required to maintain the system elastic, which sometimes is also referred to as the elastic strength demand.

Negrete (2006) estimated the central tendency and dispersion of C_R for 54 ground motions recorded in the Mexican Pacific Coast. In general, for the typical period range of CM buildings, the central tendency of C_R strongly depends on the fundamental period of vibration, T_1, and R (i.e. C_R increases at a nonlinear rate as T_1 decreases and R increases). In addition, high levels of dispersion can be observed in the estimation of C_R for very short period systems due to the sensitivity of the structural degrading model used to some earthquake ground motions.

4.2 Simplified equation to estimate C_R for CM buildings

The estimation of maximum inelastic roof drift demands for CM buildings requires a simplified equation to estimate C_R. The following equation, which follows the same format as that incorporated into the FEMA 440 document (Federal Emergency Management Agency 2005), is proposed to estimate the central tendency of C_R:

$$C_R = 1 + \left(\frac{1}{a \cdot T_1^{\,b}}\right)(R-1) \tag{4.4}$$

where a and b are coefficients that can be obtained from regression analysis. A nonlinear regression analysis was conducted to compute coefficient estimates $\hat{a} = 260$ and $\hat{b} = 3$ from the results obtained by Negrete (2006). Eqn. 4.4 provides good estimates of the geometric mean of C_R and it is suitable to be used to obtain rapid estimates of target roof displacement with Eqn. 4.1.

5. SIMPLIFIED NONLINEAR ANALYSIS TECHNIQUE FOR CM BUILDINGS

As discussed in the FEMA 440 document (Federal Emergency Management Agency 2005), the implementation of a practical displacement-based evaluation procedure requires the development of nonlinear seismic analysis techniques that apply to the structure to be assessed. Thus, a nonlinear model capable of reflecting the inter-story and local response of CM buildings as a function of their lateral displacement demands needs to be developed. Because of its ability to model the elastic properties of walls, the *wide-column* model was considered a good starting point to develop such nonlinear model.

In preceding sections, two facts were established: a) the *wide-column* model represents a feasible alternative for modeling the elastic behavior of CM walls; and b) the lateral response of CM walls is governed by shear deformations, particularly as the walls experience increasing inelastic behavior. Based on these facts, a modified version of the *wide-column* model, which exclusively relates the lateral stiffness degradation of the CM wall to its shear properties, can be formulated. This assumption implies that after the diagonal cracking in the wall's surface occurs, the flexural stiffness component remains constant while the shear stiffness component is modified as a function of the drift demand.

5.1 Nonlinear static analysis procedure for CM buildings

The nonlinear analysis of moment-resisting frames usually considers that inelastic behavior concentrates in plastic hinges located at the ends of beams and columns. Usually, shear deformations on these structural elements are neglected, in such manner that only flexural properties need to be modeled. Contrasting with this situation, the shear effects on masonry walls are important and should be explicitly considered. Nonlinear analysis of earthquake-resistant structures needs to account for two types of nonlinearity; the first related to the material's behavior and the second to the deformed configuration of the structure. In the case of low-height CM buildings, the displacement threshold associated to ultimate is usually low, in such manner that the second type of nonlinearity can be neglected.

The model proposed herein to make possible a pushover analysis of a CM building implies modeling each wall through a modified *wide-column*. While the flexural stiffness of the walls is kept constant during the analysis, their shear properties are modified according to the Flores and Alcocer (1996) backbone curve (see Figure 1). Figure 2 illustrates the modified *wide-column* model technique: while a constant flexural stiffness is assigned to the column, the shear behavior of the wall (including its inelastic range of behavior) is modeled through a rotational spring located at its base. The spring is located at the base with the purpose of relating the inelastic shear behavior of the walls with the inter-story drift due to shear deformation. As an example, Figure 3a shows specimen 3D modeled through the modified *wide-column* modeling technique using the commercial software *SAP2000* (Computers and Structures Inc. 2004). In addition, Figure 3b shows the deformed shape and potential plastic hinges in specimen 3D under increased lateral displacement.

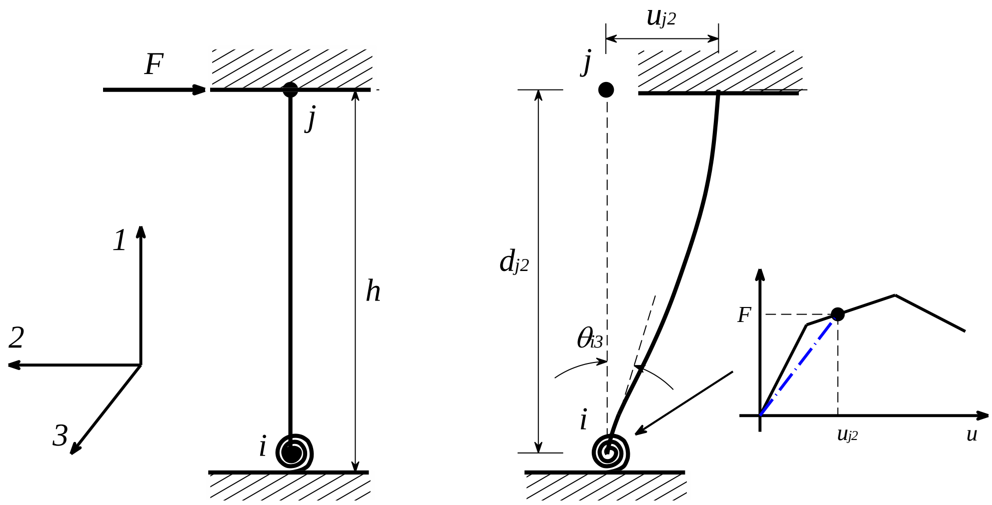
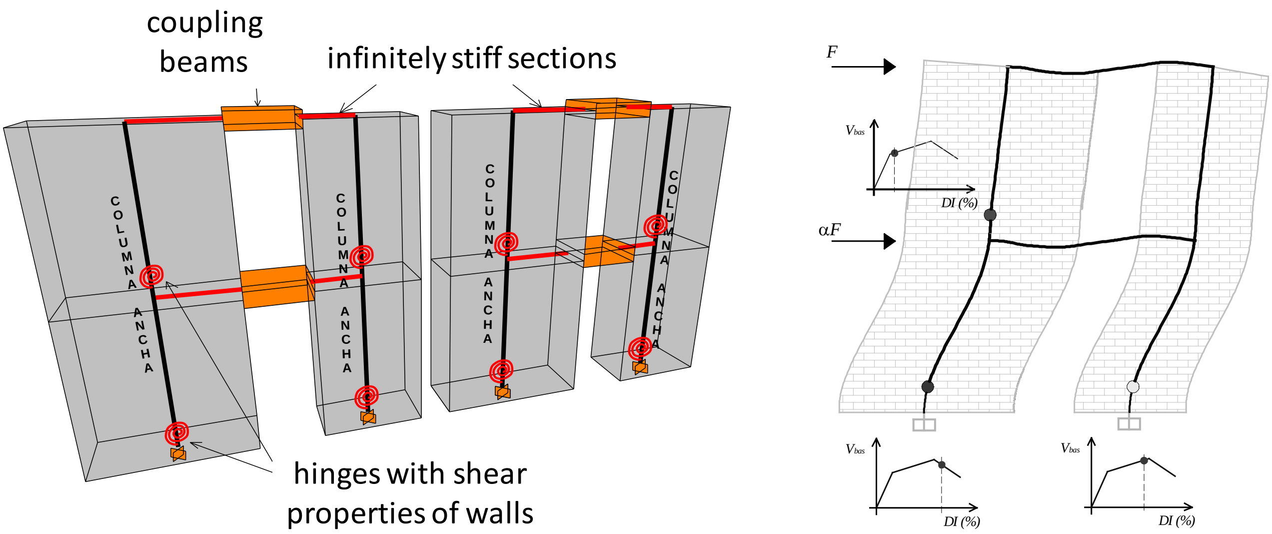


Figure 2. Modified wide-column model for static nonlinear analysis



(a) (b)

Figure 3. Modified wide-column model for Specimen *3D:* a) Nonlinear model, and b) Damage assessment

Figure 4 compares the backbone curves (positive in dark gray lines and negative in light gray lines) obtained experimentally from the reference confined masonry specimens (*W-W*, *WBW*, *WWW* and *3D*) with their capacity curves (in black lines) derived from nonlinear model proposed herein . In spite of the high variability observed in the experimental curves, the modeling technique proposed herein yields reasonable conservative estimates of the capacity curves. Besides providing a reasonable estimate of global behavior, the modeling technique allows for a reasonable estimation of the evolution of structural damage at the local level. This is illustrated in Figure 3b for specimen *3D*, which accumulated in the laboratory severe damage in the walls of the ground story and light damage in one of the walls of the upper story (Ruiz-Garcia 1995, Alcocer et al. 1996).

Due to space limitations, an example of use of the proposed methodology is not included in this paper. The example and a detailed discussion of the procedure presented herein will soon be published in an international journal.

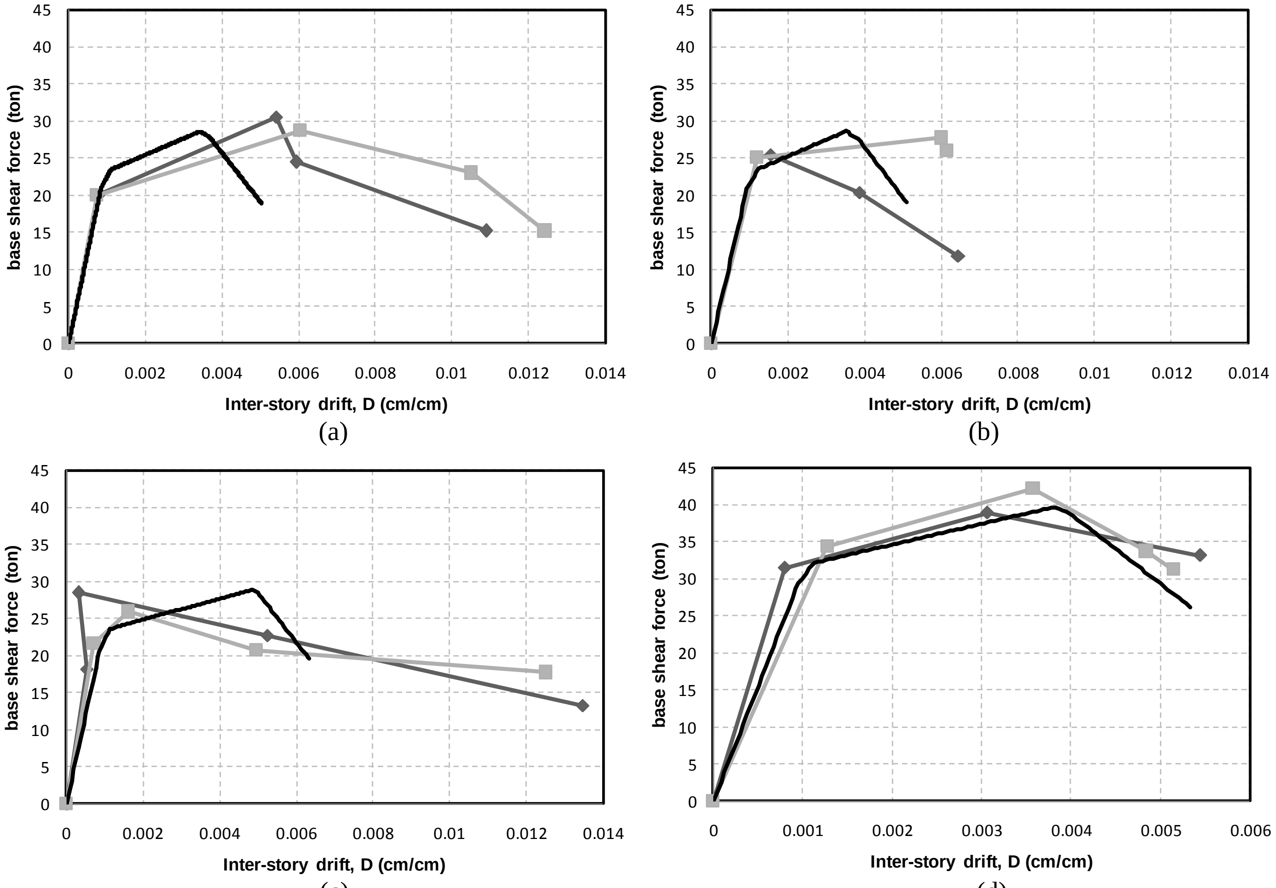


Figure 4. Comparison of capacity curves obtained experimentally and with the proposed technique for four full-scale specimens: a) *W-W*, b) *WBW*, c) *WWW*, d) *3D*

6. CONCLUSIONS

Several seismic events in Mexico and other Latin-American countries have shown the vulnerability of low-to-medium height confined masonry housing structures. Thus, practical procedures for evaluating the expected performance of confined masonry structures are highly valuable. Within this scope, a displacement-based procedure to be used during the seismic assessment of confined masonry structures was introduced in this paper.

First, the *Coefficient Method* for the seismic evaluation of existing structures was adapted to estimate roof inelastic displacement demands for regular CM buildings. In addition, a procedure to perform nonlinear static analysis of CM buildings was introduced. The nonlinear analysis model proposed herein assigns to each wall of a masonry building a modified wide-column composite element whose flexural properties are kept constant during the analysis while its shear properties are modified as a function of the inter-story drift due to shear deformation. While the model is capable of adequately establishing the capacity curve for confined masonry buildings, it is also capable of determining the relative level of structural damage exhibited by the walls as a function of the lateral displacement demand. The capacity curve can be used to perform simplified SDOF dynamic analysis to estimate inelastic roof displacement demands. However, because of the assumptions made during the formulation of the model, its application should be limited to buildings having five stories or less and that do not exhibit large slenderness ratio.

It is necessary to carry out further studies targeted at developing and integrating experimental, analytical and field data to allow for a better calibration of the seismic assessment procedure proposed herein. The formulation of rational criteria for the displacement-based evaluation and design of confined masonry buildings has the potential to reduce considerably the seismic risk in developing countries.

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