



## USE OF NONLINEAR STATIC ANALYSIS FOR THE DISPLACEMENT-BASED ASSESSMENT OF CONFINED MASONRY BUILDINGS

Amador Teran-Gilmore<sup>1</sup>, Oscar Zuñiga-Cuevas<sup>2</sup> and Jorge Ruiz-Garcia<sup>3</sup>

### ABSTRACT

The Coefficient Method established in several FEMA documents is adapted to obtain rapid estimates of inelastic roof displacement demands for regular confined masonry buildings. The parameters that should be considered during the nonlinear analysis of masonry buildings are identified, and a simple analytical model for confined masonry buildings that go beyond their elastic limit of behavior is formulated. Comparison of analytical and experimental results suggest that the proposed analytical model yields a reasonable estimate of the local and global behavior of single and multi-story masonry structures subjected to a state of monotonically increasing lateral deformation. The structural performance of a three-story confined masonry specimen that was tested in a shaking table is evaluated using the coefficient method and the proposed nonlinear model. The lateral displacement demands measured in the specimen are favorably compared to those predicted analytically. Although the proposed procedure was successful in mapping the walls that were damaged during the experimental testing, their predicted level of damage is conservative with respect to that actually observed.

### Introduction

Nowadays, some 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.

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<sup>1</sup> Professor, Universidad Autonoma Metropolitana, San Pablo 180, Col. Reynosa Tamaulipas, Mexico 02200, D.F.

<sup>2</sup> Graduate Student, Universidad Autonoma Metropolitana, San Pablo 180, Col. Reynosa Tamaulipas, Mexico 02200, D.F.

<sup>3</sup> Professor, Universidad Michoacana de San Nicolás de Hidalgo, Facultad de Ingeniería Civil, Edificio C, Cd. Universitaria, 58040 Morelia, México

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. 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), the evaluation procedure can be calibrated and applied to any type of masonry construction.

### Basis for a Displacement-based Approach for CM Buildings

Based on ample experimental evidence derived from CM walls tested under in-plane lateral cyclic loading, Ruiz (1998) established a relationship between an increase in lateral drift, and the evolution of the crack pattern and the degradation of the structural properties of low-height CM walls. This relationship is summarized in Table 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 the table, it is possible to formulate displacement-based evaluation procedures for low-height CM buildings.

Table 1. Damage and degradation of confined masonry walls (Ruiz 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 tension 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. Flores (1996) proposed a trilinear curve to characterize the backbone curve of CM walls (Figure 1). While  $V_{cr}$  corresponds to the design shear strength of the wall established according to the Masonry Technical Requirements of the Mexico City Building Code (2004),  $h$  is the height of the wall.

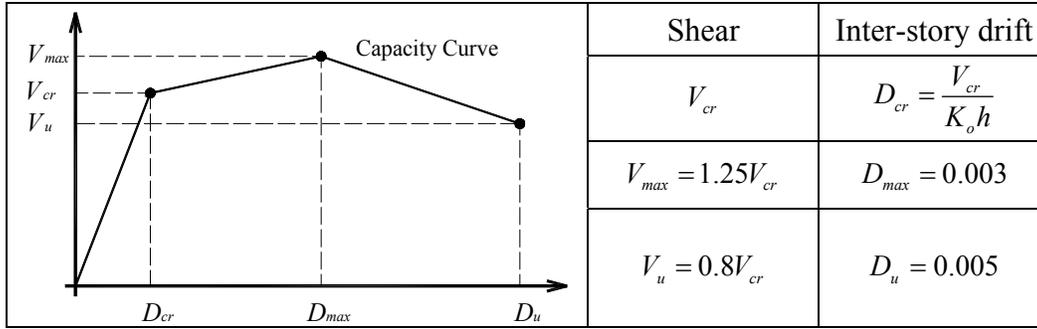


Figure 1. Idealized backbone curve for confined masonry walls (after Flores 1996)

### Wide-Column Model

According to this model, each wall of a CM building is modeled as an equivalent column that concentrates its flexural and shear properties on its centerline. In addition, equivalent beams 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 (Teran 2009).

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 2008).

The experimental and analytical lateral stiffnesses obtained for several full-scale confined masonry specimens have been compared successfully by Teran (2009). It has been concluded that 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.

### Evaluation of Lateral Displacement

Nonlinear analysis procedures have been widely used by American practicing engineers since the publication of the ATC40 (Applied Technology Council 1996) and FEMA 273 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*. Taking into account the particularities of the response of CM buildings, their target roof displacement can be estimated as:

$$\delta_T = C_o C_R S_a \frac{T_e^2}{4\pi^2} g \quad (1)$$

where  $S_a$  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.  $C_0$  takes into account multi-degree-of-freedom effects and can be estimated according to Table 2 provided the building develops a soft ground story (as CM buildings usually do).

Table 2.  $C_0$  values for CM buildings that develop a soft story

Number of Stories	Performance Level		
	Immediate Operation	Life Safety	Collapse
1	1.0	1.0	1.0
2+	1.2	1.0	1.0

The estimation of maximum inelastic roof drift demands for CM buildings requires a simplified equation to estimate  $C_R$ :

$$C_R = 1 + \left( \frac{1}{a \cdot T_1^b} \right) (R - 1) \quad (2)$$

where  $a$  and  $b$  are coefficients that can be obtained from regression analysis, and  $R$  is the lateral strength ratio, defined as:

$$R = \frac{m S_a}{V_y} \quad (3)$$

where  $m$  is the mass of the system, and  $V_y$  the lateral yield strength of the system. The numerator in Equation 3 represents the lateral strength required to maintain the system elastic, which sometimes is also referred to as the elastic strength demand.

A nonlinear regression yielded  $\hat{a} = 260$  and  $\hat{b} = 3$  for the central tendency of a set of 54 motions recorded in firm soil. None of these motions exhibited pulse-like characteristics or significant directional effects, in such manner that the calibration is only applicable to masonry buildings located in the Mexican Pacific (site specific calibrations are required for sites exhibiting different dynamic characteristics).

### **Simplified Nonlinear Analysis Technique for CM Buildings**

As discussed in the FEMA 440 document, 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.

The model proposed herein implies modeling each wall through a modified *wide-column*. While the flexural stiffness of the wide-column is kept constant during the analysis, its elastic shear properties are set equal to zero. The shear behavior of the wall is entirely modeled through a nonlinear spring (either translational or rotational), and is modified according to the Flores and Alcocer backbone curve (see Figure 1). If a rotational spring is used, it should be located at the base of the wide-column with the purpose of relating the inelastic shear behavior of the walls

with the inter-story drift due to shear deformation. Backbone curves obtained experimentally from several full scale confined masonry specimens compare well with their capacity curves derived from nonlinear model proposed herein (Teran 2009).

### Illustrative Example

The structural performance of a three-story CM building was evaluated using the proposed displacement-based procedure. Figure 2 shows the structural layout of a 1:2 scale model (of the building under consideration) tested at the shaking table located at the Universidad Nacional Autonoma de Mexico (Barragan et al. 2006).

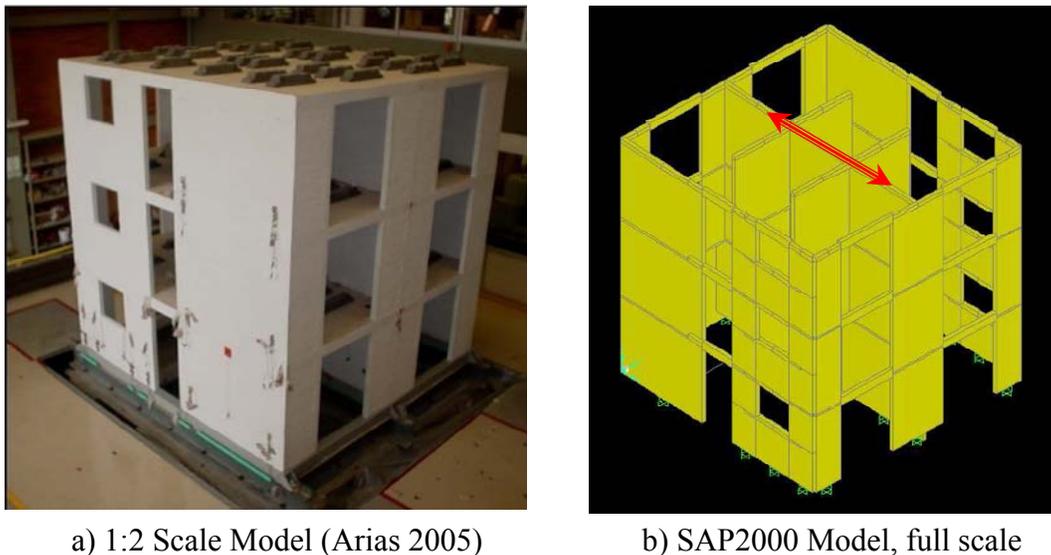


Figure 2. Structural layout of sample confined masonry building

The design strength for the concrete used to build the tie-end elements and slab was 20 MPa; that of the mortar was established at 12.5 MPa. While #3 steel bars with a nominal strength of 420 MPa were used for the longitudinal reinforcement used in the slab and tie-end elements, the transverse reinforcement consisted in #2 stirrups built of steel having 250 MPa nominal strength. The building was designed according to the 2004 edition of the Masonry Technical Requirements of the MCBC, and has a total weight of 1060 KN. While 30.6% of this weight is located at the roof, each one of the first two stories contributes 34.7% to the total weight. A detailed description of the physical, geometrical and mechanical characteristics of the building and its 1:2 scale model (denoted specimen from here on) can be found in Barragan et al. (2006). The mechanical and geometrical properties of the specimen were established following rules of *simple similarity* (Arias 2005). A simple similarity model is built with materials having the same strength and weight as those used in the building it models. In the next sections, the deformation demands measured experimentally will be compared to those estimated analytically. It should be noted that this comparison implies the scaling of the experimental demands according to the rules of simple similarity.

An earthquake ground motion recorded in the Mexican Pacific coast was used as a basis for the testing program. The seed motion, recorded in the city of Acapulco, on the 25<sup>th</sup> of April

of 1989, has a peak ground acceleration of 0.34g and was recorded during a seismic event with magnitude ( $M_s$ ) of 6.9. The specimen was subjected to a sequence of eight synthetic seismic excitations by increasing gradually the intensity of motion at each test run up until the maximum lateral strength of the specimen was attained. While Figure 2b indicates the direction in which the motions were applied to the specimen, Table 3 summarizes some of their characteristics and outlines the level of damage observed after the third and eighth motions.

Table 3. Motions used during experimental testing of 1:2 scale model

Motion	Magnitude	PGA (g)	Duration (sec)	Observations
1	7.6	1.49	15.9	
2	7.8	1.66	24.2	
3	8.0 (60%)	1.54	29.3	First significant cracking. Level of damage is associated to elastic limit (immediate operation)
4	8.3 (60%)	1.89	39.1	
5	8.0 (90%)	1.69	29.3	
6	8.3(90%)	1.97	39.3	
7	8.3(100%)	2.07	29.6	
8	8.3(125%)	2.00	39.4	The building was considered to reach its maximum lateral strength. Level of damage is associated to life safety

### Properties of the Building

Figure 2b shows the SAP2000 (Computers and Structures 2004) model used to carry out the pushover analysis of the building. The reinforced concrete slabs were considered to be infinitely rigid in-plane. The analytical model estimates a fundamental period of vibration of 0.14 seconds. Ambient tests carried on the specimen yielded a fundamental period of vibration of 0.075 seconds, which according to the rules of simple similarity corresponds to an un-cracked period of 0.15 seconds for the building.

Figure 3a shows with continuous line the capacity curve for the first story of the building. The curve was established by applying to the wide-column model, through a pushover analysis, a lateral load distribution proportional to that derived from a modal spectral analysis. The circles in the figure represent the largest lateral displacements demands and their corresponding normalized base shears derived from the experimental study of the specimen. The number associated to each circle corresponds to each one of the eight motions under consideration. The “analytical” and “experimental” curves exhibit a good correspondence. The vertical discontinuous lines included in the figure establish the displacement thresholds that according to the analytical model can be associated to first significant cracking (immediate operation) and maximum lateral strength (life safety). A fair correspondence can be observed between the experimental and analytical thresholds for first significant cracking. While an excellent correspondence for the initial stiffness predicted by both curves is observed, the main difference involves a larger post-cracking stiffness for the experimental derived curve.

### Roof Displacement

To establish estimates of roof displacements demands ( $\delta_r$ ) for the analytical model, first it

is necessary to establish within its capacity curve the base shear associated to first yielding ( $V_y$ ). Although a CM building does not strictly “yield”, it is reasonable to consider first cracking as the yielding point for the building. By using Figure 3a, it is possible to establish that  $V_y/W$  equals 1.5. The effective fundamental period of the building is equal to its elastic fundamental period ( $T_e$ ); that is,  $T_e$  is equal to 0.14 seconds. With the values of  $V_y/W$  and  $T_e$  under consideration, roof displacement demands for the building can be estimated with Equations 1 to 3.

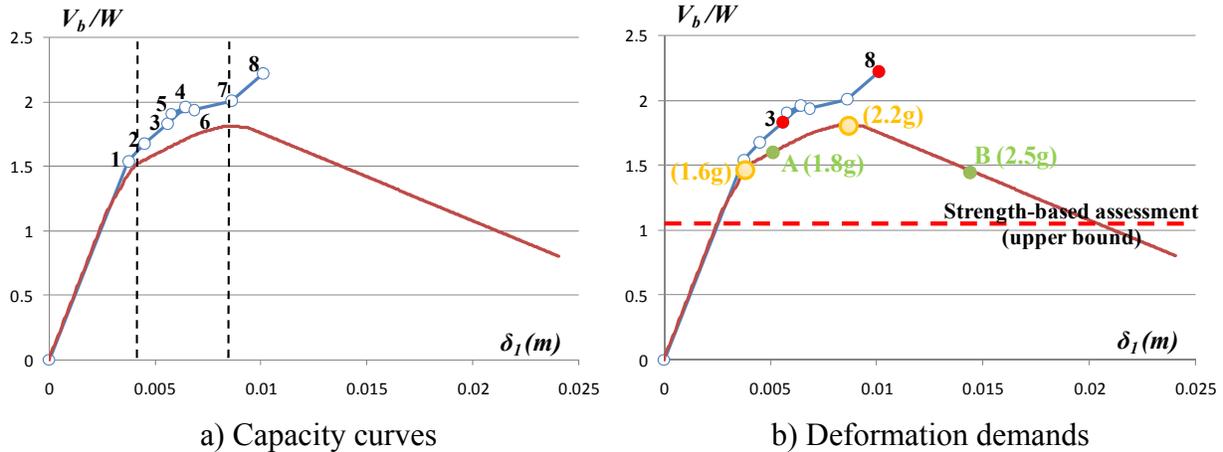


Figure 3. Comparison of experimental and analytical results.

Figure 4 shows elastic strength spectra corresponding to the third and eighth motions. The spectra correspond to 5% of critical damping. For a period of 0.14 seconds,  $S_a/g$  demands of 1.8 and 2.5 can be read for the third and eighth motions, respectively. According to the levels of damage observed in the specimen, values of 1.2 and 1.0 were assigned to  $C_\theta$ . The estimation of  $\delta_t$  results in roof displacement demands of 0.014 and 0.024 meters, respectively. Experimental roof displacements of 0.017 and 0.024 meters, respectively, are obtained. Figure 3b compares the deformation demands estimated experimentally (red circles) and analytically (green circles) in the ground story of the sample building. The yellow circles indicate the  $S_a/g$  ordinates that according to the analytical model are required to reach the immediate operation and life safety limit states.

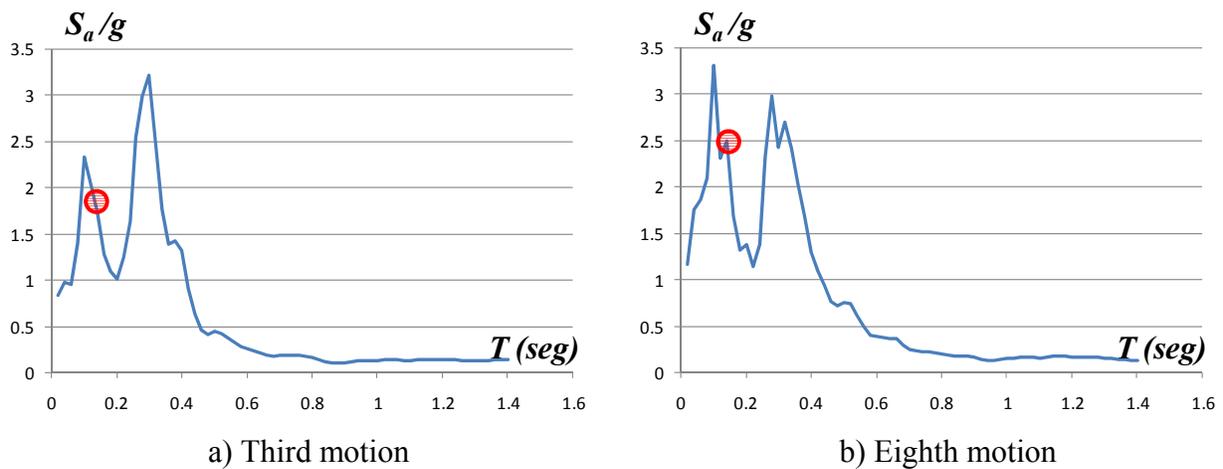


Figure 4. Elastic strength spectra corresponding to motions three and eight.

## Damage Assessment

To assess the level of damage in the critical story, the methodology requires the estimation of inter-story drift ratios. According to the pushover analysis, roof displacements of 0.014 and 0.024 meters correspond to first floor inter-story drift ratios of 0.0021 and 0.0060, respectively. Experimental inter-story drift ratios for the first story of the specimen are 0.0023 and 0.0042. Figure 3b illustrates analytical and experimental displacement demands in the first story (points A and B correspond to analytical demands for the third and eighth motions, respectively). Using the recommendations included in Table 1, the proposed methodology establishes a damage level *Heavy-IV* for the third motion, and damage level *Severe-Not classified* (close to ultimate) for the eighth motion. According to the descriptions provided by Arias (2005), damage in the specimen's walls for these motions correspond to *Moderate-III* and *Heavy-V*, respectively. As shown in Figure 5, both the experimental and analytical models indicate that damage tends to concentrate in the first story, and that a few walls in the second story end up slightly damaged.

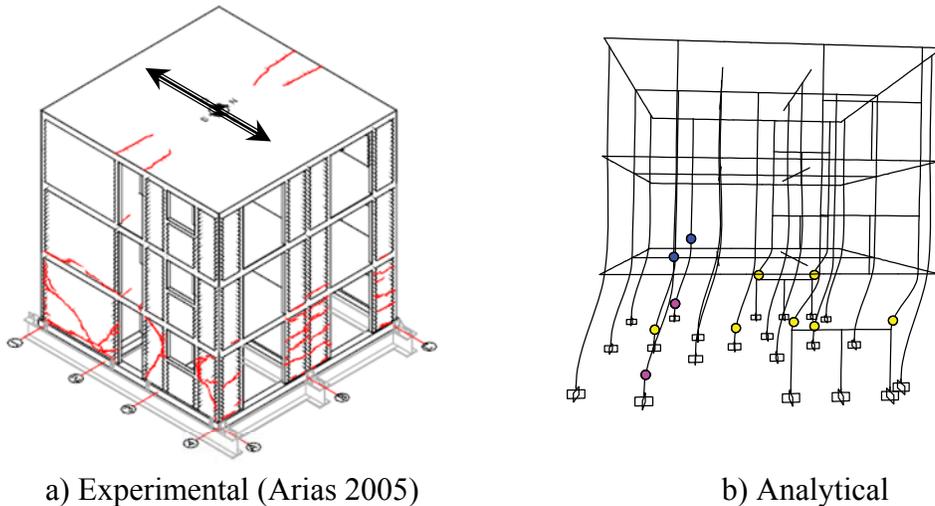


Figure 5. Damage distribution on sample building after eighth motion.

The proposed methodology yields conservative assessment of damage for the sample building. On one hand, the recommendations given in Table 1 (derived from static and pseudo-static testing of CM walls) are conservative when applied to walls subjected to dynamic loading. For instance, while the table indicates that a statically applied drift of 0.0013 results in first diagonal cracking in the wall's surface, the measurements derived from the shaking table indicate first cracking occurs at drifts larger than 0.0020. On the other hand and for similar reasons, the Flores and Alcocer model used to formulate the wide-column nonlinear model predicts that cracking occurs at smaller drifts than those observed experimentally in the shaking table. Because of this, the nonlinear model predicts the formation of the soft first story earlier than it should; and as a consequence, the first story inter-story drifts predicted by the proposed procedure for life safety are larger than those measured experimentally.

## Observations

Before concluding this section, the authors would like to discuss three issues:

- 1) Strictly speaking, the methodology introduced herein should not have been used to predict the displacement demands corresponding to the eighth motion. While the methodology assumes the building is undamaged before the motion, the specimen was moderately damaged before the eighth motion. As discussed in detail in FEMA-273, previous damage of moderate nature usually is reflected in large differences in displacement demands for low intensity motions. Displacement demands in the damaged and undamaged states of a building would tend to be similar for high intensity motions.
- 2) From a strength perspective, an upper bound base shear for the building can be estimated by adding up the shear strength, estimated according to the Masonry Technical Requirements of the MCBC, of all the walls located in the first story. As Figure 3b shows, an “optimistic” force-based assessment of the sample building indicates that a spectral ordinate  $S_a/g$  greater than 1.05 requires the seismic rehabilitation of the building. Nevertheless, the displacement-based assessment shows that the building can accommodate values of  $S_a/g$  close to 2.2 before reaching its ultimate capacity. In spite of its conservative nature, the displacement-based assessment predicts that the building exhibits 100% more seismic capacity with respect to the most optimistic force-based assessment.
- 3) The results associated to the analytical assessment of the sample building are slightly different than those presented for the same building in Teran (2009). It should be mentioned that the results reported in Teran (2009) were derived from an analytical model that had minor modeling errors.

## **Conclusions**

Displacement-based assessment methodologies can and should be formulated for confined masonry buildings. In the case of low-height confined masonry buildings, their dynamic behavior is dominated by their fundamental mode of vibration, in such manner that the *coefficient method* can be adapted to provide reasonable estimates of their local and global lateral deformation demands. Furthermore, the degradation of the structural properties of the walls is fundamentally associated to their shear behavior, in such a manner that a simplified nonlinear model derived from the widely used *wide-column* model is able to provide a reasonable estimate of the capacity curve of such buildings. The integration of the coefficient method and the simplified nonlinear model constitutes the basis from which a simple and reliable displacement-based assessment methodology for low-height confined masonry buildings can be formulated.

The application of the proposed displacement-based procedure to a three-story building yielded reliable estimates of its global and local deformation demands. The demands predicted by the methodology have a close correspondence to those estimated for the same building from the measurements taken on a 1:2 scale model tested on a shaking table. Regarding the estimation of the level of damage and structural degradation in the walls, the methodology yields conservative estimates for the damage observed during the experimental studies.

While force-based procedures tend to underestimate the seismic capacity of a structure

and do not provide for an understanding of how to achieve certain performance levels, displacement-based procedures are capable of better estimating the actual capacity of the building, and provide useful insights into what does an engineer need to do so that a particular building can achieve adequate damage control.

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