# PERFORMANCE-BASED EARTHQUAKE-RESISTANT DESIGN OF CONFINED MASONRY WALLS

**Mario RODRIGUEZ[1] And Victor RODRIGUEZ[2]**

## SUMMARY

Using results from the literature, lateral strength and deformation capacity of confined masonry units subjected to reversed cyclic lateral loading was analysed in this paper. These units were built with fired clay solid bricks and are representative of typical confined masonry in Latin America. The analysis showed a significant higher variability on strength prediction as compared to deformation capacity prediction given by a simple procedure proposed in this paper, which suggests that a Performance-Based Design (PBD) based on lateral deformations of confined masonry construction is a promising approach for carrying out a seismic design. Based on the above finding, guidelines for implementing a PBD approach for confined masonry construction in seismic zones is given in this paper. The procedure proposed here is targeted only to a collapse prevention performance level.

## INTRODUCTION

Current seismic design of structures generally follows a strength-based approach. The observed damage and collapse of structures during several past earthquakes has raised concerns about the convenience of using this approach. As a result of these concerns, alternative seismic design approaches have been proposed in recent years. For example, recent seismic design recommendations for seismic zones in the USA (FEMA 273, 1997) suggest several levels of seismic performance for a structure, which are related to specific levels of damage. This alternative approach, which is also known as performance-based design (PBD), is not at a code level yet, but it is likely to be incorporated in future seismic design codes. Since in PBD the control of seismic damage is explicitly introduced in the design process, it is hoped that the use of this approach will lead to important reduction of the vulnerability of structures. In the case of confined masonry (masonry with vertical tie-columns and bond-beams along walls at floor levels) the use of PBD is promising. Confined masonry construction is widely used in the world, and achieving a relevant reduction of the seismic vulnerability of this type of construction by using PBD may have an important impact in reducing the cost of masonry construction. This is especially relevant in less affluent countries, such as for example many in Latin America.

This paper gives some guidelines for implementing PBD for confined masonry. The guidelines are based on the lateral deformation capacity of confined masonry walls observed in laboratory testing conducted in Mexico and some countries in South America.

[1] *Instituto de Ingenieria, National University of Mexico, CP 04510, Mexico City, mrod@servidor.unam.mx*
[2] *Universidad Michoacana de San Nicolas de Hidalgo, Morelia, Mexico*

## BASIS OF THE PERFORMANCE-BASED DESIGN APPROACH

PBD is based on defining several levels of seismic performance, as well as several earthquake hazard levels. For example, the FEMA 273 guidelines for the seismic rehabilitation of buildings define three performance levels for a structure: a) immediate occupancy, b) life safety, and c) collapse prevention.

The immediate occupancy performance level corresponds to low damage in a structure and small reduction on lateral stiffness and strength. The life safety performance level corresponds to important damage in a structure and a likely loss of initial stiffness; however, after this performance level, the structure has some lateral deformation capacity before reaching the collapse stage. The collapse prevention performance level is associated to the onset of total or partial collapse, and at this level the corresponding structural damage is important, but with enough resistance to gravity loads.

Figure1 shows the performance and deformation demand for a non-ductile structure. This figure shows three points corresponding to the above performance levels as well as two performance ranges: a) damage control and b) limited safety. In the later performance range, the lateral deformations are larger than those corresponding to the life safety performance level, and they could reach those corresponding to the collapse level. If a designer chooses the limit safety range of behaviour for a structure, he or she should be aware of the high seismic risk level for that structure, as well as of the likely high cost of the seismic damage in the structure.

It should be mentioned that the several performance levels considered in PBD are related to different earthquake hazard levels. For example, according to FEMA 273, important buildings (such as schools and fire stations) should be designed for an immediate occupancy level associated to a 475 year-recurrence interval. For the same buildings, the life safety performance level suggested by FEMA 273 is associated to a 2475 year-recurrence interval.

## PERFORMANCE-BASED EARTHQUAKE-RESISTANT DESIGN OF CONFINED MASONRY WALLS

According to FEMA 273, masonry construction should be designed for three performance levels. However, to simplify a design procedure for masonry construction, and considering the difficulties in assessing the life safety performance level for this type of construction, the authors suggest only the first and last performance levels suggested by FEMA 273 be considered. This leads only to the immediate occupancy and collapse prevention performance levels.

The definition of the immediate occupancy performance level depends on the amount of structural and non-structural damage that is considered acceptable by the owner. The selected level of damage in this case should allow the immediate occupancy of the structure. This performance level is not addressed in this paper. Only the collapse prevention performance level is dealt with in this paper.

## EVALUATION OF THE COLLAPSE PREVENTION PERFORMANCE LEVEL IN CONFINED MASONRY WALLS BUILT WITH SOLID CLAY UNITS

Fired clay solid units are widely used in confined masonry construction in Latin America. In order to evaluate the collapse prevention performance level in this type of construction, experimental results obtained in nine specimens built with solid clay units were analysed. Six of these specimens were tested in Mexico and the remaining in Colombia and Chile. The geometry and reinforcement of these specimens are shown in Table 1. The data base of tests shown in Table 1 comprises walls with different aspect ratios and different amounts of longitudinal reinforcement in both the vertical tie-columns and the bond-beams. All the specimens were subjected only to in-plane reversed cyclic lateral loading.

Envelopes of the measured lateral load-deformation hysteresis cycles for the specimens were obtained to define their lateral deformation capacities. The maximum experimental lateral load, $V_{max}$, was evaluated from these envelopes, and collapse in a specimen was defined as the point at which the lateral strength decreased to $0.8V_{max}$ The interstorey drift at the collapse level is defined as $d_2$, see Figure 2. Figure 3 shows results of applying these definitions to the specimens whose details are shown in Table 1. As can be seen in Figure 3, $d_2$ ranges between

0.0045 and 0.056. These results suggest that a reasonable conservative estimation of $d_2$ in confined masonry walls built with solid clay units would be 0.004. It is of interest that similar results have been found for brick masonry walls constructed and tested in Italy (Magenes and Calvi, 1997).

To study the variation in the prediction of the lateral strength at the walls, the provisions of the Mexico City Building Code (MCBC, 1989) were used incorporating a strength reduction factor equal to one. This lateral strength, $V_{MBC}$, was evaluated as:

$$V_{MBC} = (0.5 v^* A_T + 0.3P) \leq 1.5 v^* A_T \tag{1}$$

where v* is the shear stress resistance specified by the MCBC for confined masonry, $A_T$ is the transversal wall area and P the applied axial load. Figure 4 shows results obtained using this procedure, expressed in terms of the ratio $V_{MAX}/V_{MBC}$. As it can be seen, this ratio varies between 1.2 and 2.7. In contrast, Figure 3 indicates that little variation exists in the lateral deformation capacity of the specimens.

The above finding suggests that PBD based on lateral deformations of this type of construction is a promising approach for carrying out a seismic design.

## PROCEDURE FOR IMPLEMENTING A PERFORMANCE-BASED DESIGN IN CONFINED MASONRY CONSTRUCTION

A straightforward procedure for implementing a PBD in confined masonry construction would be to perform a static non-linear analysis of a structure, from which the lateral deformation could be compared with the several levels of deformation capacities corresponding to the performance levels. This procedure might not be practical en most cases, since in general non-linear analysis is a time-consuming process and might not be appealing to most designers, although this situation might change in a near future, because of recent developments on hardware and software. Considering the current state-of-art and practice in seismic design, it seems convenient that in addition to strength spectra, future seismic codes should also provide displacement spectra associated to required performance levels for confined masonry construction. A brief description on the basis for evaluating these spectra is given in the following paragraph.

The displacement spectra, $S_d$, can be easily evaluated for several levels of damping ratios. However, its use is limited to one-degree-of-freedom (SDOF) systems, and $S_d$ spectra cannot be considered a direct measure of displacements of a multi-storey building. In this case, a measure of the global response of a building would be a useful tool for evaluating its global response. This measure could be given by the roof displacement $\delta$. Particularly, it is of interest the maximum roof drift ratio, $D_{rm}$, defined as:

$$D_{rm} = \frac{\delta_m}{H} \tag{2}$$

where $\delta_m$ is the maximum $\delta$ value and H is the height of the building, see Figure 5. $\delta_m$ can be evaluated as:

$$\delta_m = \gamma S_d \tag{3}$$

where $\gamma$ is equal to unity for SDOF systems, and for regular buildings up to five levels a value of 1.3 would be in most cases a conservative estimation.

It is of interest that a general expression for H can be obtained in terms of the interstorey height, h; the number of floors, n; and the parameter $\lambda$ (Rodriguez and Aristizabal, 1999). This parameter allows an approximate evaluation of the fundamental building period, T, by using the following expression:

$$T = \frac{n}{\lambda} \tag{4}$$

The above expression for H, and Equations (2) and (3) leads to:

$$D_{rm} = \frac{\gamma S_d}{\lambda h T} \qquad (5)$$

By using Equation (5) and appropriate values for the parameters involved there, roof drift ratio demands for specific earthquake records and typical regular structures can be evaluated.

It should be understood that when using $D_{rm}$ as a basic parameter for an approximate evaluation of the global response of a regular building, it would be necessary to relate it to the interstorey drift, $d_r$. The later parameter is related to performance levels such as those recommended by FEMA 273. In the case of masonry construction, a simple and useful relationship between $D_{rm}$ and $d_r$ can be obtained by assuming that most of the seismic response is concentrated in the first level; that is, assuming a soft-storey behaviour. This type of behaviour has been observed in laboratory after reversal lateral loading in a full-scale model of a two-story confined masonry structure (Ruiz, 1995), as well as in shaking table tests on two 1:5 scale models of a three-storey confined masonry structure (Tomazevic and Klemenc, 1997). A schematic representation of this type of behaviour is shown in Figure 5. From the lateral displacement profile shown there, a relationship between the maximum interstorey drift, $d_{rm}$, and $D_{rm}$ can be expressed as:

$$d_{rm} = \frac{H}{h} D_{rm} \qquad (6)$$

As an example of the use of the above suggested PBD procedure, for a three-storey regular confined masonry structure with a soft-story mechanism, $d_{rm}$ would be equal to 3 $D_{rm}$. After evaluating $d_{rm}$, the following step would be comparing $d_{rm}$ with the corresponding $d_r$ values associated to a specific target performance level. For example, for a collapse prevention performance level, the $d_{rm}$ value calculated according to Equation (6) should be compared with $d_2$.

## CONCLUSIONS

An analysis of the variability of results from predictions of lateral strength and deformation capacity of typical confined masonry units subjected to reversed cyclic lateral loading was described in this paper. The analysis showed a significant higher variability on strength prediction as compared to deformation capacity prediction given by a simple procedure proposed in this paper. This suggests the convenience of implementing a PBD following a displacement-based approach for seismic design of confined masonry construction.

This paper also outlines a simple procedure for implementing a PBD approach for confined masonry construction in seismic zones. The procedure proposed here is targeted only to a collapse prevention performance level and is based on conservative hypotheses for analysing the lateral deformation demand and capacity of a confined masonry construction. One of these hypotheses is that a typical failure mode in this type of construction is of the soft-story type. The procedure uses a displacement spectra which are related to target performance levels.

## REFERENCES

Aguilar, G. (1994). “Efecto del refuerzo horizontal en el comportamiento de muros de mampostería confinada ante cargas laterales”. BSc Thesis. Engineering School, National University of Mexico, México.

Alcocer, S., Meli, R, and Sanchez, T. (1994), “Comportamiento ante cargas laterales de muros de mamposteria confinada con diferentes grados de acoplamiento a flexion”. CENAPRED, Cuaderno de investigación No. 17.Mexico City.

FEMA 273 (1997), “NEHRP Guidelines for the Seismic Rehabilitation of Buildings”, Federal Emergency Management Agency, Washington DC, USA

Garcia, L. and Yamin L., “Comportamiento sismico de muros de mampostería confinada “ School of Engineering,Universidad de los Andes , Bogota, Colombia .

Herrera E. (1992), ”Efectos de la carga vertical en el comportamiento de muros de albañilería reforzada sometida a carga lateral alternada”, BSc Thesis, Facultad de Ciencias Fisicas y Matematicas. Universidad de Chile, Chile.

Magenes, G. and Calvi, G.M., (1997), “In-Plane Seismic Response of Brick Masonry Walls”, Earthquake Engineering & Structural Dynamics Journal, 26, pp 1091-1112.

Meli, R. (1975) “Comportamiento sismico de muros de mamposteria”. Report 352. Instituto de Ingeniería National , National University of Mexico, Mexico.

Meli, R. and Salgado G., (1969) “Comportamiento de muros de mampostería sujetos a carga lateral” . Report 237. Instituto de Ingeniería, National University of Mexico, Mexico.

MCBC (1989), Mexico City Building Code, Masonry Structures, Gaceta Oficial del Departamento del Distrito Federal, Mexico City.

Rodriguez, M. and Aristizabal JC, (1999), “An Evaluation of a Seismic Damage Parameter”,Earthquake Engineering & Structural Dynamics Journal, 28, pp 463-477.

Ruiz J. (1995), “ Reparación y refuerzos de una estructura tridimensional de mampostería confinada de dos niveles a escala natural”, Master Thesis, Engineering School, National University of Mexico, Mexico.

Tomazevic, M., and Klemenc I., (1997), “Verification of Seismic Resistance of Confined Masonry Buildings”, Earthquake Engineering & Structural Dynamics Journal, 26, pp 1073-1088.

**Table 1: Characteristics of Confined Masonry Units**

| Specimen | Reference | Dimensions | | Vertical-tie Column | | Bond-beam | |
|---|---|---|---|---|---|---|---|
| | | L x h (m) | Thickness (m) | Dimensions a1 x a2 (m) | Long. Reinf (mm) | Dimensions a1 x a2 (m) | Long. Reinf (mm) |
| S/I | Meli (1975) | 1.80 x 1.80 | 0.14 | -------- | -------- | -------- | ------- |
| 902 | Meli & Salgado (1969) | 2.40 x 2.40 | 0.12 | 0.12 x 0.15 | 4 $\phi$12.7 | 0.12 X 0.20 | ------- |
| 3D | Ruiz (1995) | 2.00 z 2.50* | 0.125 | 0.125 x 0.15 | 4$\phi$9.5 | 0.125 X 0.25 | 4$\phi$12.7 |
| WBW | Alcocer et al (1994) | 2.00 x 2.50* | 0.125 | 0.125 x 0.15 | 4$\phi$9.5 | 0.125 X 0.25 | 4$\phi$12.7 |
| WWW | Alcocer et al (1994) | 2.00 x 2.50* | 0.125 | 0.125 x 0.15 | 4$\phi$9.5 | 0.125 X 0.25 | 4$\phi$12.7 |
| MURO 6 | Garcia & Yamin (1994) | 3.15 x 2.15 | 0.12 | 0.12 x 0.20 | 3$\phi$12 | 0.12 X 0.20 | 4$\phi$9.5 |
| B12 | Herrera (1992) | 2.40 x 2.40 | 0.14 | 0.14 x 0.20 | 4$\phi$10 | 0.25 X 0.20 | 4$\phi$10 |
| B2 | Herrera (1992) | 2.40 x 2.40 | 0.14 | 0.14 x 0.20 | 4$\phi$10 | 0.25 X 0.20 | 4$\phi$10 |
| M2 | Aguilar (1994) | 2.40 x 2.40 | 0.12 | 0.12 x 0.15 | 4$\phi$10 | 0.12 X 0.25 | 4$\phi$12.7 |

Notes:
* Average Length
l Wall length
h Wall height
a1 RC element width
a2 RC element height

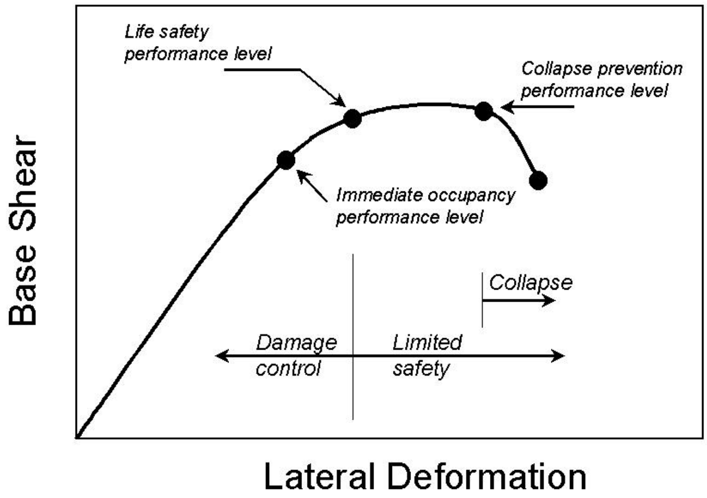


**Figure 1: Performance and deformation demand for nonductile structures**

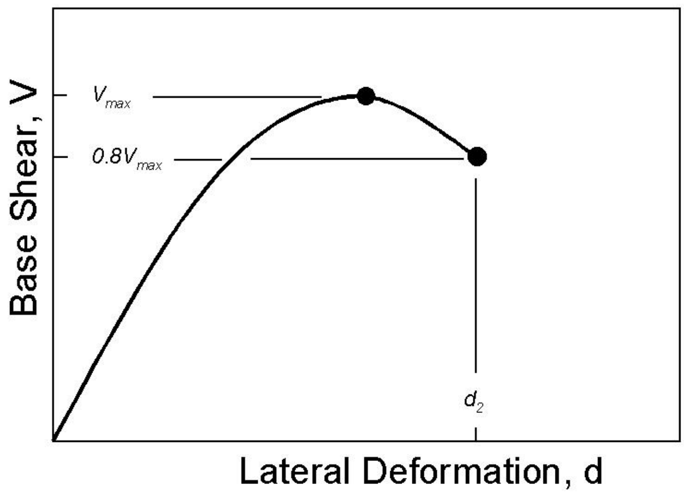


**Figure 2: Definition of interstorey drift at collapse prevention performance level**

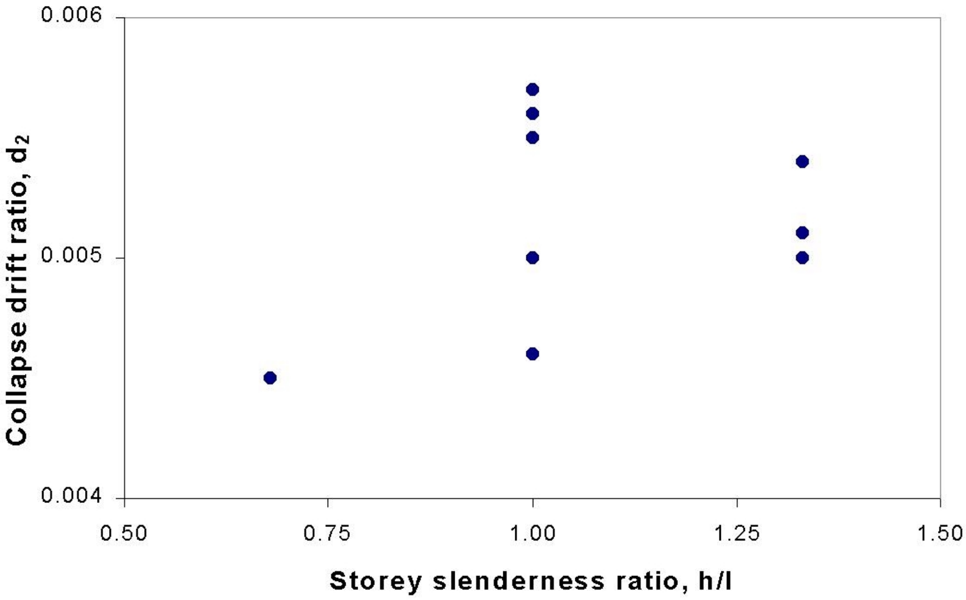


**Figure 3: Measured interstorey drift at collapse prevention performance level**

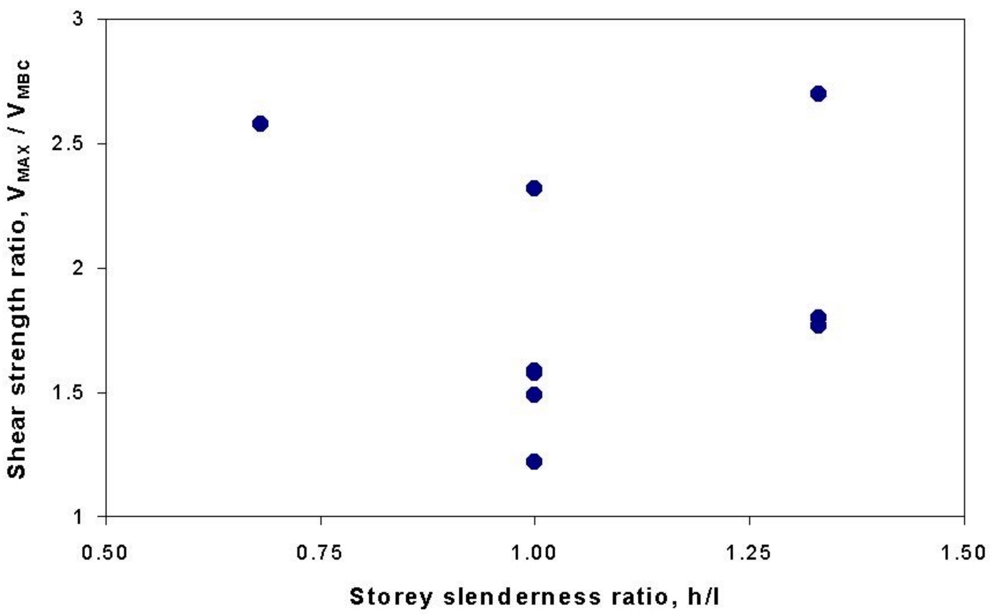


**Figure 4: Measured and predicted base shear strength**

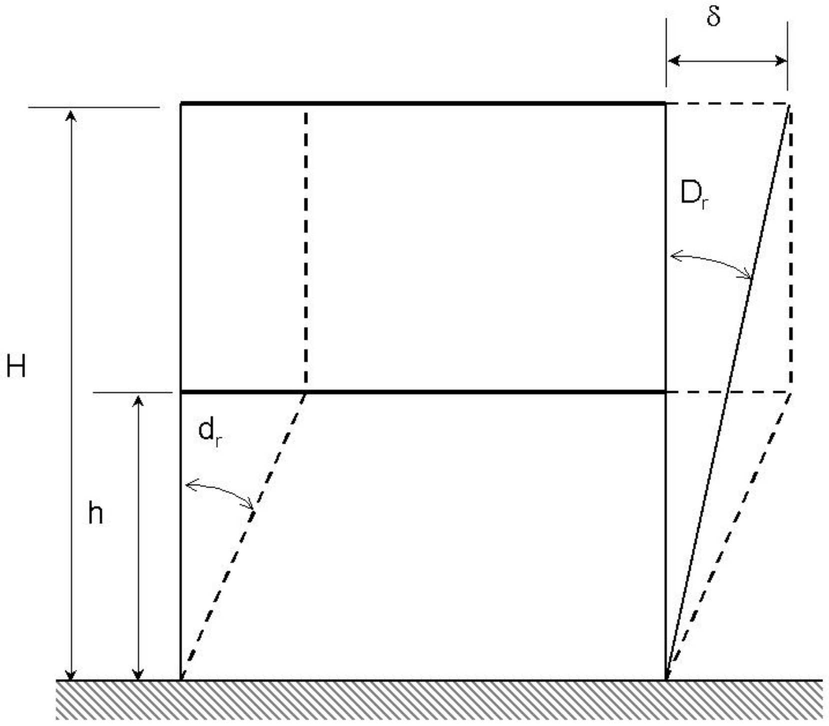


**Figure 5 : Lateral deformation profile for a soft-storey mechanism**