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# EXPERIMENTAL ASSESSMENT OF SHEAR STRENGH AND DISPLACEMENT CAPACITY OF PARTIALLY GROUTED REINFORCED CONCRETE MASONRY SHEAR WALLS

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**ABSTRACT:** Unexpected damages observed in reinforced masonry buildings in the most recent Chilean earthquakes have demonstrated, once again, the seismic vulnerability of this kind of construction and have pointed out the need to review the buildings' seismic performance. For this purpose, ten full-scale partially grouted reinforced concrete masonry walls were tested under cyclic lateral loading. All specimens were constructed using hollow concrete masonry units and tested with a cantilever-type boundary condition. The design parameters under study were pre-compression axial level, wall aspect ratio and horizontal reinforcement ratio. In this paper, the effect of these parameters on the shear strength and lateral displacement capacity is analyzed and discussed. Experimental results indicate that parameters under study present a joint influence when the shear strength is evaluated. On the other hand, lateral displacement shows little variation when the horizontal reinforcement ratio increases from 0.04% to 0.09%.

## 1. Introduction

In Chile, Reinforced Masonry (RM) buildings are commonly used for residential dwellings up to four stories high. In the north region of Chile, constructions based on RM walls of hollow concrete blocks are a very popular typology; in the central and southern regions of Chile, RM walls are generally built using multi-perforated clay bricks. In addition, a large part of this building stock has been constructed with partially grouted walls.

In general, buildings with reinforced masonry walls have not performed well in recent seismic events. In particular, the earthquakes of Tarapacá (2005, Mw=7.8), Maule (2010, Mw=8.8) and Iquique (2014, Mw=8.2) showed that partially grouted reinforced masonry constructions continue to be highly vulnerable. Post-earthquake observations conducted by several researchers (Santander, 2007; Almazán, 2010; Nuñez, 2010; D'Ayala & Benzoni, 2012; Astroza et al., 2012; Valdebenito et al., 2015) have indicated that these masonries continue to present constructive, structural and design deficiencies. The majority of observed damages are associated with diagonal cracks (x-shaped) reflecting a shear failure mode. In addition, it has been noted that the reinforced masonry built with hollow concrete blocks and partial filling is more vulnerable than the partially grouted masonry built with hollow ceramic units (Astroza et al., 2012).

In the last few decades, the in-plane experimental behavior of partially grouted reinforced concrete masonry shear walls (PG-RCMSW) has received increasing attention (Lüders et al., 1985; Ingham et al., 2001; Voon & Ingham, 2006; Elmapruk, 2010; Minaie et al., 2010; Nolph & ElGawady, 2011; among others). These experimental works have indicated that parameters such as wall aspect ratio, level of axial compression stress, amount and distribution of shear reinforcement, and material properties of the components used in wall construction have an important role in the shear response of PG-RCMSW. However, and despite the large research effort undertaken in recent years, further experimental results are required to achieve better insight on the main variables influencing their in-plane shear behavior.

Few studies have been carried out in Chile to investigate the in-plane shear behavior of PG-RCMSW. With the purpose of enlarging the available experimental evidence, ten partially grouted reinforced concrete masonry shear walls were constructed and tested under cyclic lateral loading simulating earthquake effects. Based on preliminary results, this paper presents preliminary conclusions about the influence of three design parameters such as aspect ratio, horizontal reinforcement ratio and axial load level on shear strength and lateral displacement capacity of PG-RCMSW.

## 2. Seismic performance of reinforced masonry constructions in Chile

### 2.1. General characteristics

In Chile, about 32% of housing stock are built with partially grouted reinforced masonry (Astroza et al., 2012). Its use started in the 1970s. Basically, this construction system is characterized by the use of one leaf walls with a thickness of 140 mm. Steel bars, with diameters of 8mm to 16mm, are used as vertical reinforcements. These bars are located in some hollow cores and are filled with cement grout. Prefabricated reinforcement meshes (ladder type) with 4.2mm diameter are typically used as horizontal reinforcement. This reinforcement is embedded inside of horizontal mortar bed joints. Fig. 1 shows a scheme of a wall constructed with the system described above. It should be noted that this construction system does not use bond beams, differentiating it from North American reinforced masonry construction.

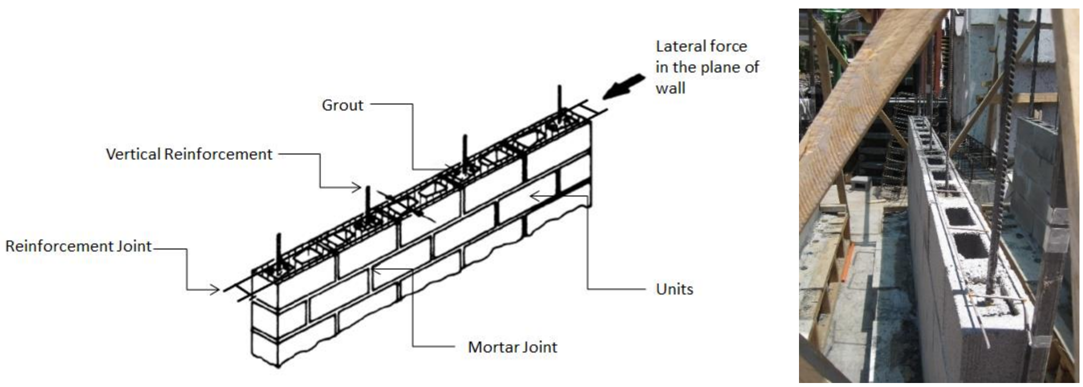



Fig. 1 – Typical reinforced masonry used in Chile

### 2.2. Damage observations

Unexpected severe damages were observed in a significant number of RM buildings in the recent earthquakes (D'Ayala & Benzoni, 2012; Astroza et al., 2012; Valdebenito et al., 2015). A failure mode characterized by a diagonal cracking pattern is prevalent in the majority of the damaged buildings. Causes of this behavior include a low density of walls together with a deficient quality of constituent materials; these could be factors that led to brittle failure mechanisms due to shear stresses, rather than ductile failure mechanisms due to flexural stresses. The failure due to shear stress is known to be a brittle failure mechanism characterized by a limited capacity of energy dissipation, quick degradation of stiffness and quick deterioration of shear capacity after reaching maximum lateral resistance. Fig.2 shows some damaged RM constructions during the most recent Chilean earthquakes.

a) b) c)

Fig. 2 – Damaged constructions: a) two-story social housing in 2005 Tarapacá earthquake; b) four-story building in 2010 Maule earthquake; c) two-story social housing in 2014 Iquique earthquake.

## 3. Experimental program

### 3.1. Test specimens

A total of 10 partially grouted reinforced concrete masonry shear walls (PG-RCMSW) were designed and constructed to full scale according to the NCh1928Ofc.93 standard (INN, 1993). The main variables were the aspect ratio of wall specimens, axial compression level and horizontal reinforcement ratio. The ranges of these design variables were selected in order to complement the Chilean experimental database. With the objective of ensuring the occurrence of the shear failure mode observed in real walls after earthquakes, a high vertical reinforcement ratio was used in all specimens. The vertical reinforcement bars and their spacing were kept constant for each aspect ratio considered. Fig. 3 shows the typical configuration of walls tested.

The wall height-to-length ($h_{ef}/d$) ratios under study were $h_{ef}/d$ =0.44 (short walls), $h_{ef}/d$ =0.97 (regular walls) and $h_{ef}/d$ =1.95 (slender walls). The horizontal reinforcement ratios of 0.04%, 0.08% and 0.09% were considered. Except walls MBH-7 and MBH-10, all specimens were axially loaded by means of a vertical actuator. The axial load level applied corresponded to 10% of the compressive strength of masonry. Table 1 summarizes the main characteristics of the wall specimens tested in this experimental program.

Table 1 – Reinforced masonry specimens

| Wall | Wall Dimensions | | | Aspect Ratio | Vertical Reinforcement | | | Horizontal Reinforcement | | | | Pre-Compression Axial |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | d (mm) | $h_w$ (mm) | $h_{ef}$ (mm) | $h_{ef}/d$ | Vertical Reinforcement | $A_v$ ($mm^2$) | $\rho_v$ (%) | Joint Reinforcement | $S_v$ (mm) | $A_h$ ($mm^2$) | $\rho_h$ (%) | $\sigma_n$* (Mpa) |
| MBH-1 | 1990 | 2000 | 1930 | 0.97 | $2\Phi22+2\Phi10$ | 917 | 0.33 | $4-2\Phi4.2$ | 400 | 111 | 0.04 | 0.56 |
| MBH-2 | 1990 | 2000 | 1930 | 0.97 | $2\Phi22+2\Phi10$ | 917 | 0.33 | $4-2\Phi4.2$ | 400 | 111 | 0.04 | 0.56 |
| MBH-3 | 1990 | 2000 | 1930 | 0.97 | $2\Phi22+2\Phi10$ | 917 | 0.33 | $9-2\Phi4.2$ | 200 | 249 | 0.09 | 0.56 |
| MBH-4 | 1990 | 2000 | 1930 | 0.97 | $2\Phi22+2\Phi10$ | 917 | 0.33 | $9-2\Phi4.2$ | 200 | 249 | 0.09 | 0.56 |
| MBH-5 | 2590 | 1400 | 1130 | 0.44 | $2\Phi16+3\Phi10$ | 638 | 0.18 | $3-2\Phi4.2$ | 400 | 83 | 0.04 | 0.56 |
| MBH-6 | 2590 | 1400 | 1130 | 0.44 | $2\Phi16+3\Phi10$ | 638 | 0.18 | $6-2\Phi4.2$ | 200 | 166 | 0.08 | 0.56 |
| MBH-7 | 2590 | 1400 | 1130 | 0.44 | $2\Phi16+3\Phi10$ | 638 | 0.18 | $3-2\Phi4.2$ | 400 | 83 | 0.04 | 0.00 |
| MBH-8 | 990 | 2000 | 1930 | 1.95 | $2\Phi22+1\Phi10$ | 839 | 0.61 | $4-2\Phi4.2$ | 400 | 111 | 0.04 | 0.56 |
| MBH-9 | 990 | 2000 | 1930 | 1.95 | $2\Phi22+1\Phi10$ | 839 | 0.61 | $9-2\Phi4.2$ | 200 | 249 | 0.09 | 0.56 |
| MBH-10 | 990 | 2000 | 1930 | 1.95 | $2\Phi22+1\Phi10$ | 839 | 0.61 | $4-2\Phi4.2$ | 400 | 111 | 0.04 | 0.00 |

### 3.2. Material properties

Materials commonly used in the Chilean RM constructions were selected for this campaign. Concrete block masonry units with nominal dimensions of 390 mm long, 190 mm high and 140 mm wide, as well as pre-mixed commercial mortar, were used in the construction of the test specimens. All walls were constructed according to Chilean standards by an experienced mason under supervision, and were air-cured in the laboratory environment for a minimum of 28 days prior to testing. Table 2 presents a summary of mechanical properties of the materials used as well as the composite material.

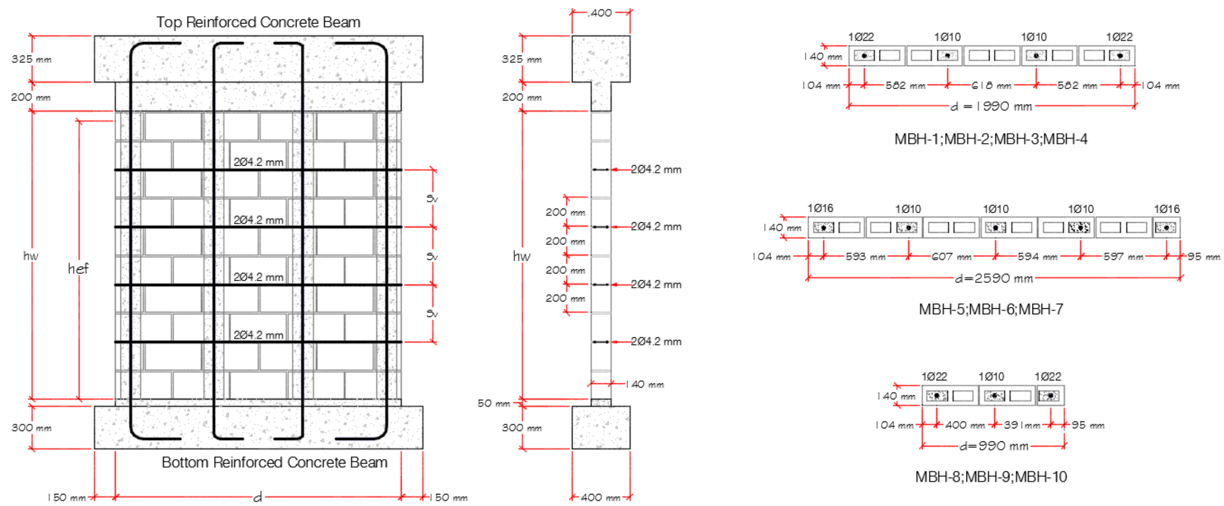



Fig. 3 – Typical design details of test walls

Table 2 – Material properties

| Property | Notation | Average (Mpa) | COV (%) | Test Standard |
|---|---|---|---|---|
| Concrete block compression strength | $f'_{cu}$ | 6.36 | 16 | NCh182.Of 55 |
| Cement mortar compression strength | $f'_{cm}$ | 18.03 | 2 | NCh158.Of 67 |
| Cement mortar flexural strength | $f'_{bm}$ | 4.73 | 6 | NCh158.Of 67 |
| Grout cylinder compression strength | $f'_{cr}$ | 31.72 | 2 | NCh1037.Of 77 |
| Yield strength shear reinforcement | $f_{yh}$ | 610 | 3 | NCh200.Of 72 |
| Ultimate strength shear reinforcement | $f_{uh}$ | 660 | 3 | NCh200.Of 72 |
| Young's modulus of shear reinforcement | $E_{sh}$ | 189598 | 8 | NCh200.Of 72 |
| Yield strength vertical reinforcement | $f_{yv}$ | 474 | 3 | NCh200.Of 72 |
| Ultimate strength vertical reinforcement | $f_{uv}$ | 765 | 3 | NCh200.Of 72 |
| Young's modulus of vertical reinforcement | $E_{sv}$ | 209145 | 9 | NCh200.Of 72 |
| Masonry compression strength | $f'_m{}^*$ | 5.54 | 15 | NCh2123.Of 97 Mod.2003. Anexo B |
| | $f'_m$ | 8.65 | | |
| Young's modulus of masonry | $E_m{}^*$ | 6465 | 17 | NCh2123.Of 97 Mod.2003. Anexo B |
| | $E_m$ | 10115 | | |
| Masonry shear strength | $\tau_m{}^*$ | 0.61 | 11 | NCh1928.Of 93 Mod.2009. Anexo A |
| | $\tau_m$ | 0.95 | | |
| Shear's modulus of masonry | $G_m{}^*$ | 1976 | 24 | NCh1928.Of 93 Mod.2009. Anexo A |
| | $G_m$ | 3083 | | |

Note: The properties $f'_m$, $E_m$, $\tau_m$ and $G_m$ are calculated on net area while the properties $f'_m{}^*$, $E_m{}^*$, $\tau_m{}^*$ and $G_m{}^*$ are calculated on gross area.

## 3.3. Test setup and instrumentation

Typical test setup used during the tests is shown in Fig. 4. All walls were placed in a steel loading frame and then were fixed to the floor and to the top steel transfer beam. To simulate a cantilever condition, the pantograph, located at the top of the frame load, was disconnected for all tests. Thus, specimens were free to laterally displace and rotate at the top. The lateral load was applied by means of a horizontal 500 kN actuator at a height $h_{ef}$, resulting in a moment diagram as shown in Fig. 4.

The specimens were instrumented with 2 load cells, 16 displacement transducers (LVDT's) and 2 strain-gauges. Fig. 5 shows the general scheme of instrumentation used in this experimental program. Load cells were connected to both horizontal and vertical actuators in order to measure the applied forces. The displacement transducers were installed in the specimens to measure displacements and to obtain curvatures. Strain gauges were glued on to the base of vertical bars at both ends in order to evaluate their contribution to the response of the walls.

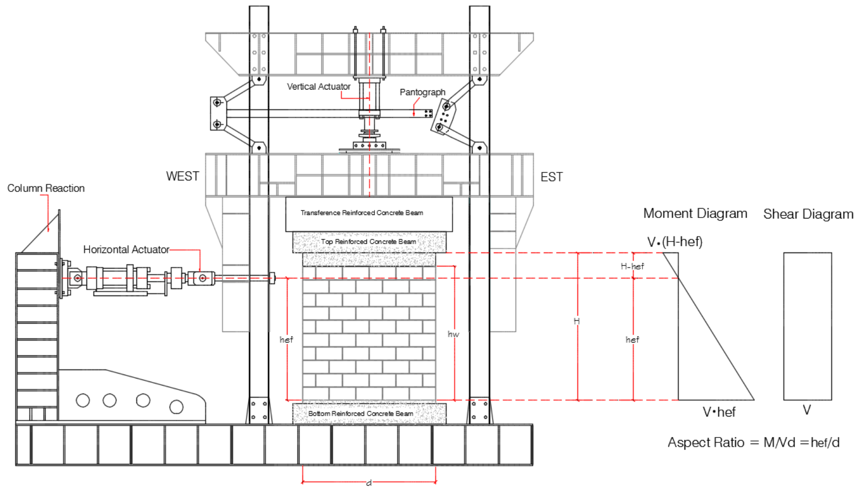



Fig. 4 – Test setup for cyclic loading tests

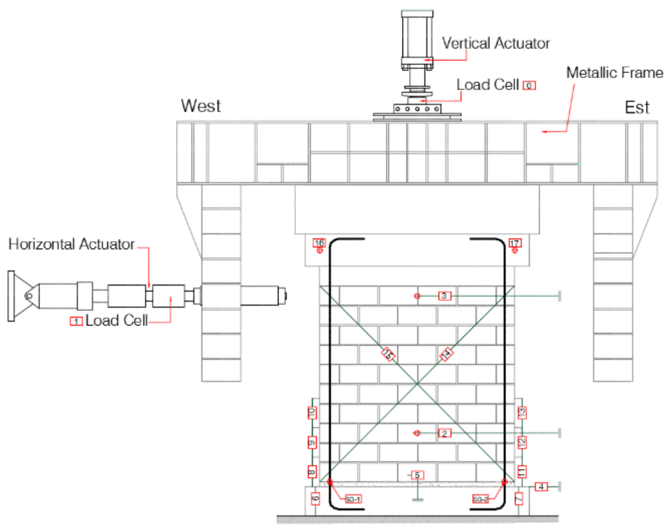



Fig. 5 – General scheme of instrumentation

## 3.4. Testing procedure

Constant axial load and increasing lateral displacements were applied to the walls. The displacement protocol used in the tests was the same used in previous Chilean investigations (Sepúlveda, 2003; Alcaíno & Santa María, 2008). The cyclic loading sequence is shown in Fig. 6. As was adopted in Voon & Ingham (2006), this experimental program also defined the wall failure as the point on the loading curve where the lateral force was reduced to 80% of the maximum lateral force recorded during the test. The displacement rate was applied at a constant velocity of 7 mm/min. To capture any sign of stiffness and strength degradation, two load cycles with the same lateral displacement were applied for both load directions.

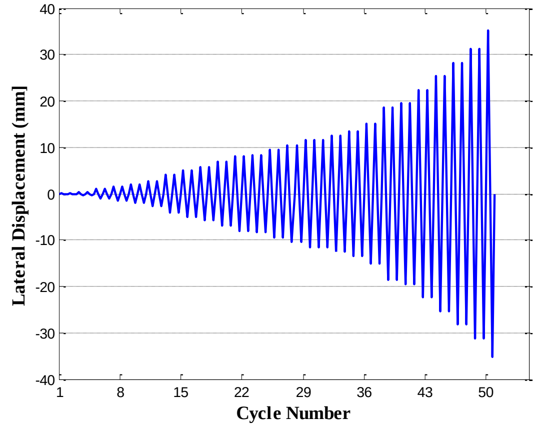



Fig. 6 – Imposed displacement history

# 4. Experimental results and discussion

## 4.1. General observations

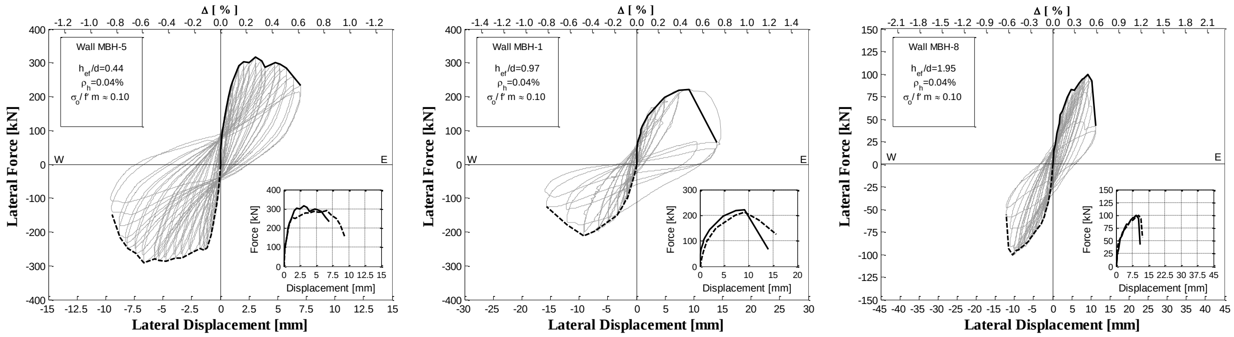
As expected, all walls exhibited a shear failure mode. The damage associated with this failure mode was characterized by diagonal cracking (stepped and sloped), penetration of cracks in the compression zones, spalling of external walls of units, grout cracking, crushing of the bottom corners and lateral deformation of vertical reinforcement (dowel action effect). As a result, the experimentally obtained force-displacement curves for walls MBH-1, MBH-5 and MBH-8 are shown in Fig.7. Meanwhile, Fig. 8 illustrates the cracking pattern at the end of testing for these same walls. These walls have different aspect ratios, but they have a similar horizontal reinforcement ratio and the same axial load level.

The following observations regarding failure mode, stiffness degradation and energy dissipation can be made based on these results.

- All walls developed a brittle failure mode, but collapse of the short walls was more gradual than that observed in regular and slender walls. In the case of these last two wall types, collapse was sudden and explosive.
- Stiffness degradation exhibited by regular walls was more severe and quick than that showed by short and slender walls.
- For a same level of lateral deformation, short walls showed more hysteretic energy dissipated than regular and slender walls. The capacity of regular walls to dissipate energy was higher than in slender walls.

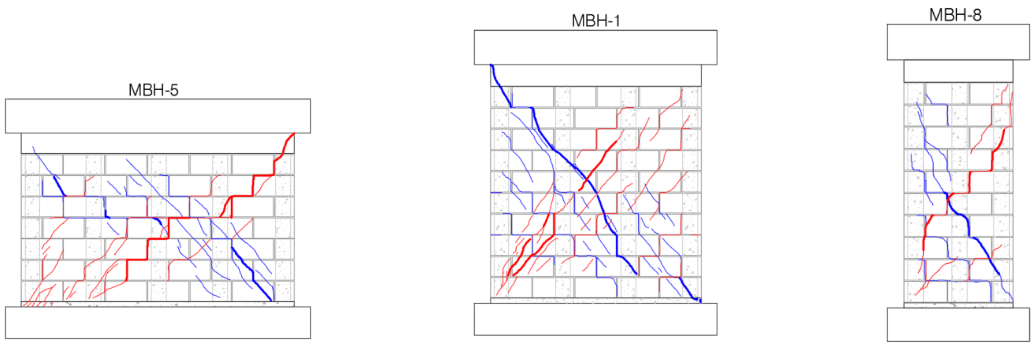
## 4.2. Shear Strength

For comparison purposes, the shear strength is treated in terms of the ratio between shear stress ($v_n$) and the square root of masonry compressive strength ($f'_m$). To assess the influence of each design parameter under study on the shear strength, non-dimensional term $v_n/\sqrt{f'_m}$ was plotted against the lateral displacement. Fig. 9a shows that when the aspect ratio is increased, the shear strength decreases, as has been reported by other authors (Matsumura, 1988; Voon KC & Ingham, 2006). However, two atypical cases were noticed. First, a short wall showed a shear strength lower than a regular wall (Fig. 9b), and second, a short wall showed a shear strength lower than a slender wall (Fig. 9c). On the other hand, it was observed that the shear strength increases when the horizontal reinforcement ratio is increased. This influence is more notable for regular (Fig. 9e) and slender walls (Fig. 9f) while that influence is not clear for short walls (Fig. 9d). Similarly, the axial pre-compression level has an important influence on the shear strength, with a greater influence on short walls (Figs. 9g and 9h). It should be noted that the shear strength values in terms of the parameter $v_n/\sqrt{f'_m}$ varied between 0.24 and 0.48 for all walls.




a) b) c)

Fig. 7 – Force-displacement diagrams for walls MBH-1, MBH-5 and MBH-8




a) b) c)

Fig. 8 – Cracking patterns for walls MBH-1, MBH-5 and MBH-8

## 4.3. Displacement capacity

The lateral displacement capacity of each wall tested was analyzed in terms of the drift (Δ), which was defined as the relation between horizontal displacement measured at the height of inflexion point and the effective height of the wall ($h_{ef}$). Horizontal displacement includes shear and flexural deformations, without displacements due to sliding and rocking movements which were not included in this analysis. Fig. 10 shows the envelope curves relating the horizontal force to drift. In these figures, the drift was considered up to the maximum lateral force. In Figs. 10b and 10c, it can be observed that the drift increases when the aspect ratio is also increased. However, this influence appears to be lower for walls with a low horizontal reinforcement ratio and presence of axial load (Fig. 10a). Similarly, from Figs. 10e and 10f, it is observed that the drift level increases when the horizontal reinforcement ratio is also increased. However, the increase of the horizontal reinforcement ratio does not show a clear influence on short walls (Fig. 10d). In the same way, it can be noted in Fig. 10h that the drift level tends to decrease for a wall axially loaded. However, this effect is more notable as the wall aspect ratio increases (Fig. 10g). Based on these results, it can be established that the variation of drift is less sensitive for walls with low aspect ratios ($h_{ef}$/d=0.44), independent of the horizontal reinforcement ratio and the axial load level applied.

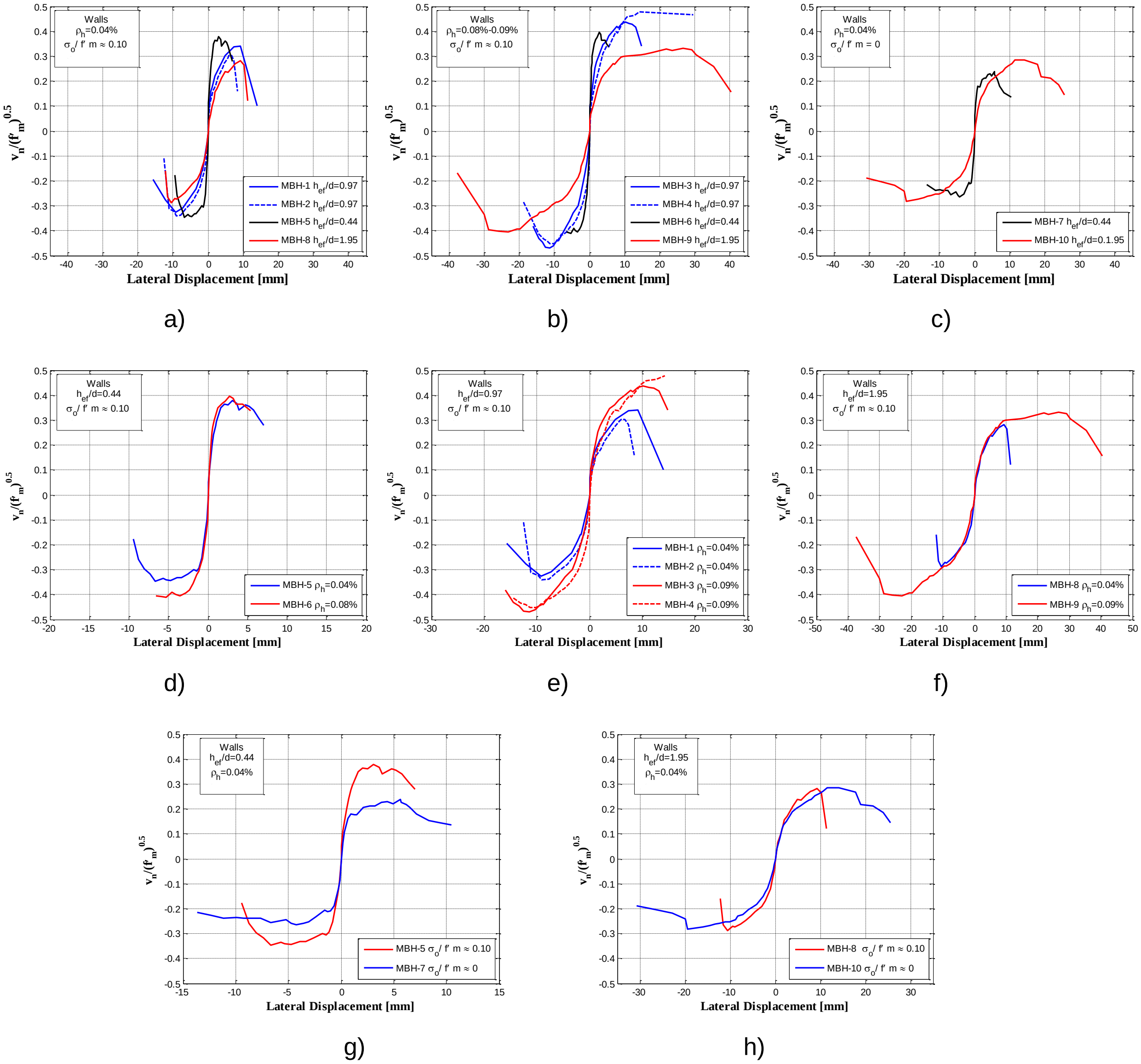



Fig. 9 – Influence of aspect ratio (a,b,c), horizontal reinforcement (d,e,f) and axial load (g,d) on the shear strength.

## 5. Conclusions

Preliminary experimental results have been presented about the influence of three relevant parameters on shear strength and lateral displacement of partially grouted reinforced masonry walls. It was noticed that when the wall aspect ratio is increased, the shear strength decreases. However, two atypical cases were observed. First, a short wall showed a shear strength lower than a regular wall, and second, a short wall exhibited lower shear strength than a slender wall. Although other studies have not reported similar atypical cases, more experimental research is required to clarify this behavior. On the other hand, the increase of the horizontal reinforcement ratio originated an increase in the shear strength, with a greater influence in regular and slender walls. Similarly, the presence of an axial load level equal to 10% of masonry compressive strength showed an increase in the shear strength, with a greater influence in short walls.

The drift level showed to be dependent on the three design variables considered in this study. Experimental results showed that the drift level increases when the aspect ratio is also increased or when the horizontal reinforcement ratio is increased. However, the drift level decreases when the axial load level is increased. Furthermore, it was observed that walls with low aspect ratios showed low variation when their horizontal reinforcement ratio varies from 0.04% to 0.09% or when their axial load level is lower than $0.10f_m'$.

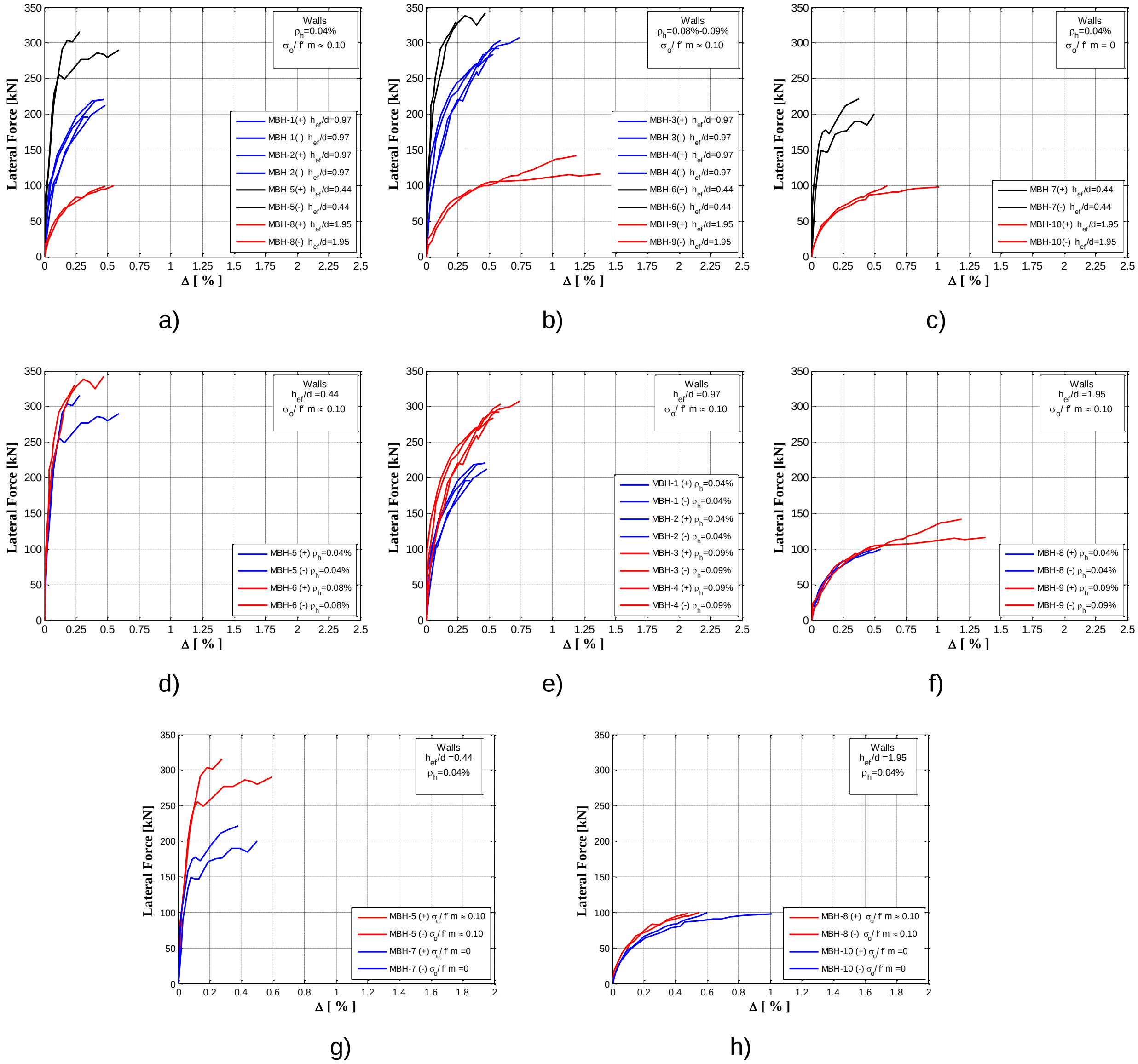



Fig. 10 – Influence of aspect ratio (a,b,c), horizontal reinforcement (d,e,f) and axial load (g,h) on drift

## 6. Acknowledgements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