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# Experimental and analytical investigation of the lateral load response of confined masonry walls

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**Abstract** This paper investigates the behavior of confined masonry walls subjected to lateral loads. Six full-scale wall assembles, consisting of a clay masonry panel, two confining columns and a tie beam, were tested under a combination of vertical load and monotonic pushover up to failure. Wall panels had various configurations, namely, solid and perforated walls with window and door openings, variable longitudinal and transverse reinforcement ratios for the confining elements and different brick types, namely, cored clay and solid concrete masonry units. Key experimental results showed that the walls in general experienced a shear failure at the end of the lightly reinforced confining elements after the failure of the diagonal struts formed in the brick wall due to transversal diagonal tension. Stepped bed joint cracks formed in the masonry panel either diagonally or around the perforations. A numerical model was built using the finite element method and was validated in light of the experimental results. The model showed acceptable correlation and was used to conduct a thorough parametric study on various design configurations. The conducted parametric study involved the assessment of the load/displacement response for walls with different aspect ratios, axial load ratios, number of confining elements as well as the size and orientation of perforations. It was found that the strength of the bricks and the number of confining elements play a significant role in increasing the walls' ultimate resistance and displacement ductility.



## Introduction

Confined masonry (CM) is considered one of the popular forms of low-cost, low-rise constructions throughout the world; including the Middle East, South and Central America, Mexico, South-East Asia, and South-Eastern Europe [1]. The system relies on a load-bearing wall encased by small cast-in-place reinforced concrete tie columns and tie beams [2,3]. The distinguishing feature of confined masonry construction is that the masonry wall is constructed prior to the casting of

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the confining elements, tie columns and tie beams, thus both elements respond integrally when subjected to lateral loads in addition to the cost reduction of the formwork. In general, tie columns have a rectangular section whose dimensions typically correspond to the wall thickness. For tie beams, both wall thickness and floor type influence the choice of the dimensions. The confining elements are intended to confine the masonry panel preventing disintegration, to enhance wall deformation capacity, and connectivity with other walls and floor diaphragms. The recent European codes state that the contribution of vertical confinement to vertical and lateral resistance should be ignored [4]. The amount of reinforcement is determined empirically on the basis of experience, and depends on the height and size of the building.

In a way, the behavior is similar to that of infilled reinforced concrete frames. However, in the case of confined masonry, tie-columns do not represent the load-bearing part of a structure. The in-plane response of a confined masonry wall is distinctly different from that of reinforced concrete infilled frames, where the frame is constructed prior to the masonry infill. Although a confined masonry wall experiences both flexural and shearing deformations, the masonry infill deforms in a shear mode within a frame that attempts to deform in flexural mode, resulting in separation of the frame and infill wall along the interface.

If properly constructed, confined masonry construction is expected to show satisfactory performance in earthquakes. The bad experience with this form of construction in past earthquakes involved structures that were built without tie columns and/or tie beams, with inadequate roof-to-wall connection, or with poor-quality materials and construction. The main observed damage patterns can be summarized as: (1) shear cracks in walls that propagate into the tie-columns; most cracks passed through mortar joints [5,6], (2) crushing of masonry units has been observed in the middle portion of the walls subjected to maximum stresses, (3) horizontal cracks at the joints between masonry walls and reinforced concrete floors or foundations [7,8], (4) cracks in window piers and walls due to out-of-plane action in inadequately braced walls, (5) crushing of concrete at the joints between vertical tie-columns and horizontal tie beams when the reinforcement was not properly anchored [9,10].

Since 2010, an extensive research program, aiming at developing structurally and economically efficient hybrid building system for developing countries in general and for Egypt in particular, is being undertaken at the Department of Structural Engineering of Ain Shams University. This paper presents the findings of the experimental and analytical phases of this research program on wall assemblies designed and built using locally available materials and with common workmanship and construction practices.

## Experimental program

### *Description of the tested walls*

A total of six wall panels were tested in this experimental program. All the panels had an aspect ratio of 1.00 and built with near full-scale dimensions. Fig. 1 shows the dimensions and reinforcement details of typical wall panels, i.e., solid and perforated walls. Table 1 summarizes the various design parameters of the tested panels. Variations include the type of panel (solid, window and door openings), type of used brick (cored clay and solid concrete masonry units), and longitudinal and transverse reinforcement ratios in the confining elements. Single Wythe masonry walls were built directly over reinforced concrete footings using bricks with nominal dimensions of $250 \times 120 \times 60$ mm for both the clay and concrete masonry units. The units were laid in running bond using 10-mm mortar joints and a half brick in alternating courses was left intentionally vacant to form a toothed interlocking connection with the confining columns. Reinforced concrete columns and beams, having rectangular cross-sections of dimensions $120 \times 250$ mm, were cast against the brick wall and side timber formwork. Fig. 2a through 2d summarizes the construction sequence of the walls.

For control purposes, standard concrete cubes were cast alongside the walls and were tested at the same day as the tie beams, in order to provide values of the 28-day concrete characteristic compressive strength, ($f_{cu}$), which was on average 25 MPa. Standard five-brick masonry prisms were built next to the walls and tested at the same day of the wall testing. The mean compressive strength of the clay masonry prisms ($f'_m$) was 4.5 MPa. The main reinforcement of all confining elements was made of deformed steel bars (Grade 36/52) of nominal yield stress ($f_y$) of 360 MPa, and ultimate tensile strength ($f_u$) of 520 MPa. The transverse reinforcement was made of mild steel smooth bars (Grade 24/35) of nominal yield stress ($f_y$) of 240 MPa, and ultimate tensile strength ($f_u$) of 350 MPa. All the previous reinforcements had a modulus of elasticity ($E_s$) of 200 GPa. The walls were left to cure for 28 days before testing and were white washed with non-latex paint to ease the visualization of the developed cracks during testing.

### *Test setup, boundary conditions and loading scheme*

The walls were monotonically tested up to failure under a combination of vertical and monotonically increasing lateral loads. Fig. 3 shows the test setup of the walls. In this respect, a single concentrated load of 150 kN was firstly distributed by a stiff steel distributor I-beam laid on top of secondary steel beams and separated by four rolling steel cylinders as shown in Fig. 3. The secondary beams were laid on top of the concrete tie beam of the wall assembly using gypsum bedding to avoid stress concentration. The purpose of the rolling cylinders is to allow the wall to displace laterally while maintaining the distributed vertical load. The load was chosen to simulate that of a typical module in a two-story residential building with commonly used module dimensions. The lateral load was applied to the tie beam using a 500 kN hydraulic jack. A thick steel plate was placed between the jack and the beam to avoid stress concentration at the loading point. The footing was held in place using two sets of steel struts, the first (Strut A) being horizontal and reacting against the loading frame column to prevent the wall sliding and the second (Strut B) being inclined and reacting against the opposite column to restrain the footing uplift at the loading side. The loading procedure comprised of one loading cycle, during which the load was incrementally increased by 20 kN up to failure. At the end of each load step, the load was held constant for a period of two minutes, during which measurements and marking of cracks took place.