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Influence of Opening Size and Internal Tie-columns Around Openings on the Seismic Response of Confined Masonry Walls: A Numerical Investigation

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ABSTRACT

Confined masonry (CM) is one of the popular structural systems embraced by many countries as an earthquake-resistant and affordable solution for building construction. Despite this, there are still gaps in the knowledge about seismic response. This paper presents a comprehensive numerical investigation into the seismic response of CM walls with openings, utilizing a micro-modelling approach in Abaqus software. A finite element (FE) model, validated on two cyclic shear tests of full-scale CM wall specimens, is employed for a parametric study exploring the effects of varying wall aspect ratios and opening configurations (no opening, door opening, window opening). The primary objective is to examine the influence of opening size and on the lateral resistance of CM walls, focusing on in-plane behavior, stiffness, strength, and ductility.

The results from the numerical simulations demonstrate that the presence of tie-columns around openings and shape of the openings significantly affect the seismic performance of CM walls. As the opening size increases, the strength reduction becomes more evident, with walls having AR = 0.71 showing a substantial decrease in load-bearing capacity due to the amplified effect of larger openings. Furthermore, the size of tie-columns around openings is recognized as crucial in mitigating strength losses. These findings underscore the necessity of further experimental tests to fully validate these conclusions.

KEYWORDS

confined masonry, numerical simulation, micro-modelling, Abaqus, window opening, door opening

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INTRODUCTION

Recent advancements in the European masonry industry have introduced innovative materials and construction techniques aimed at enhancing thermal efficiency in buildings. One prominent development is the use of hollow masonry blocks with larger cavities, specifically designed to improve thermal insulation and reduce energy consumption for heating and cooling. These blocks, when bonded with polyurethane (PU) adhesive instead of traditional thin-bed mortar, form bed-joints with practically zero thickness, significantly reducing thermal losses through them.

Confined masonry (CM) system, which consists of load-bearing masonry walls enclosed with cast-in-place reinforced concrete (RC) tie-columns and tie-beams, offers an effective solution for enhancing the seismic response. While the fundamental concept of CM remains consistent globally, notable regional variations exist, especially regarding wall thickness and tie-column dimensions. For instance, in Latin America, where thermal insulation is not necessary, CM walls are generally thinner, with examples from Mexico showing thicknesses of 12 cm [1]. Conversely, in Europe, Eurocode 8 [2] mandates a minimum wall thickness of 24 cm. However, recent developments in the masonry industry have resulted in some masonry blocks reaching a thickness of 38 cm, as seen in this study, or even more. These significant differences in wall thickness present challenges for achieving effective confinement, particularly when tie-columns are thinner than the walls, which is common in thicker walls.

To enhance thermal performance and minimize material consumption, tie-columns and masonry walls are often designed with different thicknesses, because reducing tie-column dimensions can significantly decrease concrete consumption. Experimental tests [3, 4] have shown that using tie-columns smaller than wall thickness is not problematic, but recent shear-compression tests on full-scale CM walls [5] revealed a new type of damage at the tie-column-masonry interface.

For modelling of seismic response of CM, the Finite Element Method (FEM) is commonly used, with DIANA [6] and Abaqus [7] being the most widely used FE software for the analysis. Although 2D models are generally preferred for their accuracy and efficiency in simulating in-plane responses, this study employs a fully 3D FE model due to the unique shear damage observed at the interface between the tie-columns and masonry wall.

In recent years, greater attention has been given to experimental campaigns investigating the seismic behavior of CM walls with openings, particularly focusing on the effects of opening size, position, and the role of RC confining elements around these openings [8]. These studies have highlighted that larger, unconfined openings lead to significant reductions in strength and stiffness, while the addition of RC tie-columns around openings make up for the lost capacity and improves seismic performance. Numerical modeling remains an area requiring further development to augment experimental research, as full-scale experiments are costly and challenging to conduct.

The main objective of this study is to investigate the seismic performance of CM walls with openings built from the new type of masonry, focusing on parameters such as wall aspect ratio (AR), opening size, shape, and the presence and dimensions of tie-columns around them. Following the research gap identified by Pérez Gavilán et al. [9], future research should focus on the effect of RC tie-columns size on the seismic performance of CM walls with openings. In this parametric study, this effect is investigated by considering two different dimensions for the internal tie-columns.

MATERIALS, WALLS AND RESULTS FROM SHEAR-COMPRESSION TESTS

The masonry blocks used in the study are vertically perforated, commercially available units (length/height/thickness = 250/249/380 mm) designed for constructing thermally efficient walls. The mechanical properties of these blocks have been extensively tested in prior research by Gams et. [5].

Two identical CM wall specimens, labelled W7 and W8, 175 cm in length, 195 cm tall, and 38 cm thick (Fig. 1a) were tested in the laboratory by Gams et al. [5]. The walls were built on a RC foundation beam for fixation to the strong floor during the test. The tie-columns measured 25x25 cm (Fig. 1a) and were reinforced with four Ø 14 mm ribbed bars, resulting in a longitudinal reinforcement ratio of 1%, as required by Eurocode 8 [2]. Transverse reinforcement consisted of Ø 8 mm stirrups spaced approximately 20 cm apart (see Fig. 1b). The tie-columns were flush with one side of the wall, while on the opposite side, they were depressed 13 cm from the surface, with the gap filled with thermal insulation. Finally, a RC spreader bond beam was cast together with the tie-columns.

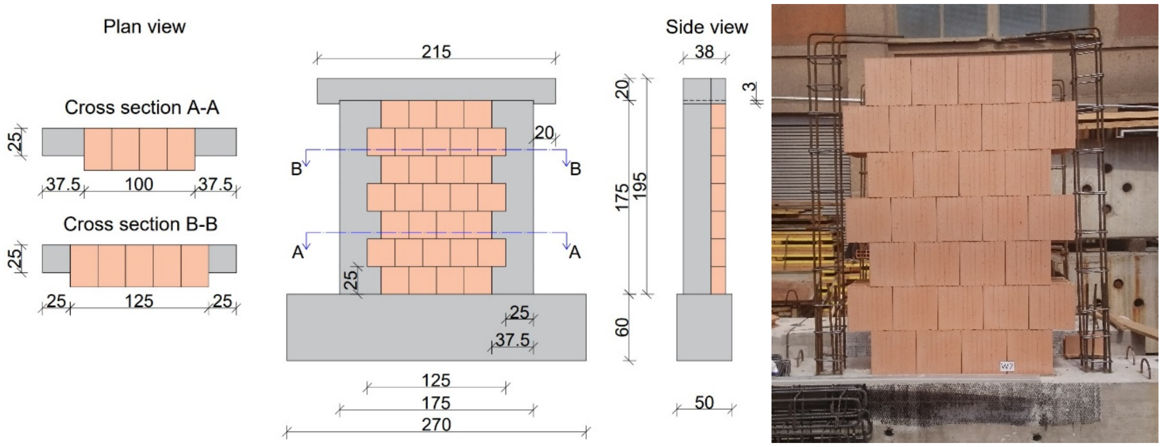

Figure 1: a) CM wall test specimen (dimensions in cm); b) tie-column reinforcement [5].

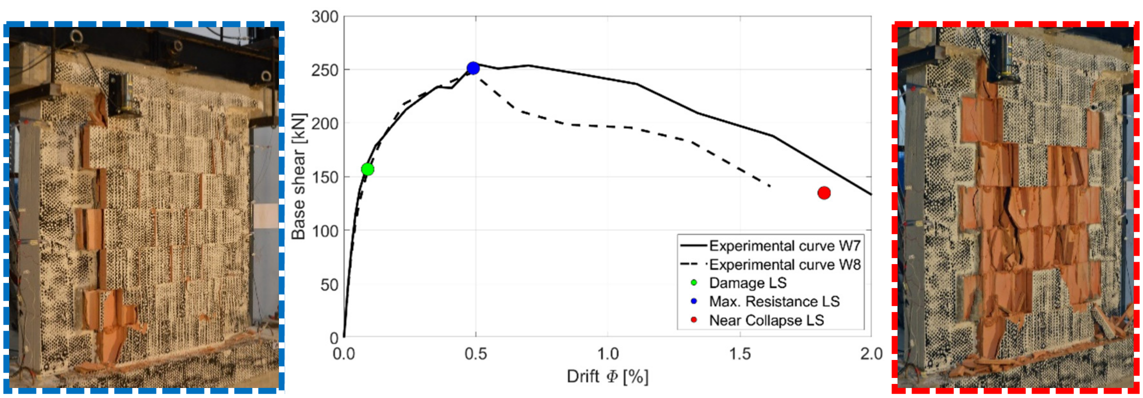

Figure 2: Experimental envelope curves of the tested CM walls, highlighting different LSs and damage patterns at 0.5% (left) and 1.3% drift (right).

The walls were tested under in-plane shear compression conditions at a constant vertical stress of 0.63 MPa. Rotations were restricted by using double-fixed boundary conditions. Horizontal displacements were applied cyclically at the level of the bond beam, with three repetitions per displacement amplitude in both directions, simulating seismic loading in a standard manner [10]. Detailed information regarding the test setup, instrumentation, loading protocol, and comprehensive test results can be found in [5].

A Digital Image Correlation (DIC) system was used to measure the displacement and strain fields on the wall surface. Further details on the damage progression at different limit states (LSs) can be found in study by Krtinić et al. [11]. The experimentally obtained averaged envelope curves for both tested CM walls in the push and pull directions, are shown in Fig. 2. The LSs are indicated by differently colored circles: green for the damage LS, blue for the max. resistance LS, and red for the near-collapse LS. At a drift of 0.5% (Fig. 2, left), major damage occurred due to the shear-off effect at the interface between the tie-columns and the masonry, leading to the break-off of protruding masonry pieces. Further, the protruding masonry parts completely sheared off and fell down at a drift of 1.3% (Fig. 2, right).

NUMERICAL SIMULATION AND MODELING

This section presents a description of the FE model developed for numerical simulations and validation of the in-plane global seismic response of the tested CM walls. The model was built in Abaqus/Explicit software [7], enabling the simulation of both global behavior and local effects in the CM walls. After validating the FE model against experiments, the model was used for parametric study to examine the effects of ARs and openings on the seismic performance of CM walls.

Description of the Numerical Model

A fully 3D model was developed to accurately capture and simulate the shear-off effect in the protruding masonry, as seen in Fig. 3. The masonry units were modeled as solid elements, while the effect of holes is taken into account by adjusting the material properties (modulus of elasticity and compressive strength) according to Stavridis and Shing [12]. The test specimen's geometry was modeled based on actual dimensions (Fig. 1a), excluding the foundation beam, which was represented by a rigid plate to reduce computational effort and time. RC tie-elements (tie-columns and bond beam) and masonry units were modeled using C3D8R elements, as recommended for explicit dynamic analysis [7], while the longitudinal bars and stirrups were modeled using T3D2 elements (Fig. 3a). Interactions between concrete and reinforcement were modeled using the “Embedded Constraint” feature in Abaqus. Boundary conditions were applied to match the experimental setup, with the rigid plate assumed fixed using an "Encastre" boundary condition, which effectively simulated the constraint provided by the foundation beam in the experiments.

The load application in the FE model consisted of three steps: (1) gravity load, (2) vertical stress on the wall, and (3) horizontal displacement. The vertical loading was aligned with the experimental setup, while the lateral in-plane loading was applied monotonically in the push direction, as opposed to the cyclic loading used in the experiments. Gravity load, accounting for the weight of the concrete, masonry, and reinforcement, was applied first. Next, a vertical pressure of 0.63 MPa was uniformly distributed across the bond beam's upper surface (Fig. 3b). Prior to the application of the horizontal loading, a constant compressive stress was achieved in the CM wall, and the vertical forces (compression) remained constant throughout the numerical simulation. Finally, an in-plane horizontal displacement was applied at the bond beam level through a reference point as a boundary condition (U1 displacement, Fig. 3b).

Geometric nonlinearities were included in all loading steps, with their durations carefully optimized through multiple simulations to ensure convergence and eliminate dynamic effects. The first two steps were set to 1.0 s, while the main loading step was 12.5 s. At the beginning, several numerical simulations were conducted to identify an optimal mesh size of 32.5 mm, achieving a balance between accuracy and stability and providing nearly four elements across the protruding masonry (Fig. 3a).

The general contact available in Abaqus [7] was used with specific interaction properties to define the contact interfaces between masonry units and the joints with surrounding RC tie elements, as shown schematically in Fig. 3c. A "Tie" constraint was applied between masonry units and RC tie-elements to represent the chemical bond formed during concreting, ensuring a rigid connection at all interfaces (green dashed line), except between the masonry units in the first row and the RC foundation beam.

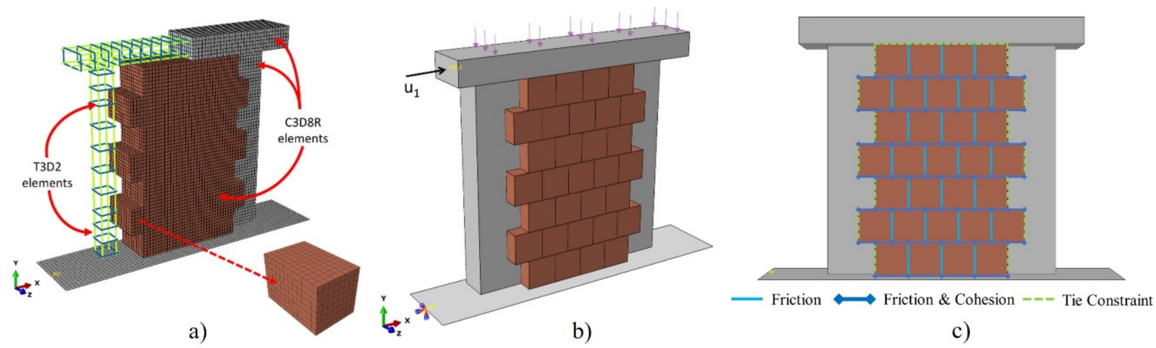

Figure 3: A 3D FE model built in Abaqus: a) mesh, b) loads and c) interaction.

The interface properties for the unfilled head joints between masonry units (light blue solid line) were defined to minimize surface penetration, with compressive resistance, zero tensile resistance, and a friction coefficient of 0.57, derived from experimental tests. A surface-based cohesive interaction was used to model the contact for PU-filled bed joints (dark blue solid line). The behavior of these joints was defined by individual contact properties, combining this cohesive interaction with the previously established global interaction properties, such as "hard" contact and penalty friction.

Surface-based cohesive interaction is especially relevant when the interface thickness is negligibly small, as in cases where PU glue replaces traditional mortar. Due to the negligible thickness of the PU-filled joints, the interaction was defined directly between the individual masonry units. Initially, the contact behavior is assumed to be linear elastic with uncoupled stiffness in all three directions, exhibiting linear shear behavior until maximum tensile or shear stress is reached, after which degradation occurs. The stiffnesses in the normal (K_{nn} =23.13 MPa/mm) and shear directions (K_{ss}=K_{tt}=9.64 MPa/mm) were calculated using the equations proposed by Nazir and Dhanasekar [13]. During the final calibration of the FE model, the elastic stiffnesses values were reduced by 20% to enhance the match with the experimental results.

The maximum tensile stress (t_n=0.12 MPa) was taken from the literature [14] as the threshold for damage initiation in the contact surface (quadratic traction criterion). The maximum shear stresses (t_s=t_t=0.097 MPa) were derived from triplet tests on masonry samples, tested with and without precompression (see Section 2 in [5] for details). Once the damage initiation threshold was reached, a damage evolution law was applied to model the degradation of cohesive stiffness. To model in a proper way the damage evolution, fracture energies for Mode I (G_n=0.001 N/mm) and Mode II (G_s=G_t=0.5 N/mm) were defined through calibration and values from the literature [15], respectively. The damage evolution model used was an energy-based exponential mixed-mode law, with the cohesive parameter η taken from [14].

The Concrete Damaged Plasticity (CDP) model, widely used in Abaqus [7], was adopted to simulate the inelastic behavior of both concrete and masonry units. Detailed calculations for the non-linear behavior of concrete and masonry under compression and tension, the corresponding damage variables (d_c and d_t), and the model used for steel reinforcement are beyond the scope of this paper and are thoroughly described in the study by Krtinić et al. [11].

Validation of the Numerical Model

The numerical model was validated by comparing its results with experimental data from two test full-scale specimens (labelled W7 and W8). The comparison primarily focused on the averaged hysteretic response envelopes (over both push and pull directions) and the numerically obtained force–drift curve (blue curve), as shown in Fig. 4. The results from the numerical model closely matched the experimental results, especially regarding initial stiffness. The predicted peak strength capacity aligned well with the experiments, with only minor discrepancies of 7.4% for specimen W7 and 4.4% for specimen W8. The drift at peak capacity was almost identical, with the model predicting 0.44% compared to 0.5% observed in the experiments. Additionally, the model accurately predicted the post-peak response and damage distribution, including shear-off effect and shear failure in tie-column, as presented in [11].

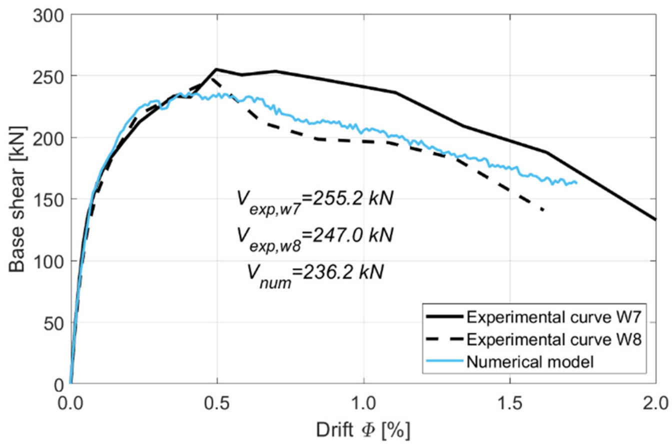

Figure 4: Numerical vs. experimental force–drift envelope curves.

PARAMETRIC STUDY

The objective of this parametric study is to analyze the influence of aspect ratio (AR), opening configurations, and RC tie-columns on the seismic performance. A total of 12 distinct models were analyzed, each featuring various opening sizes, shapes, as well as dimensions of tie-columns. A summary of the models is provided in Table 1, where each model is identified by a name that represents different combinations of aspect ratios, opening shape, and the inclusion of tie-columns around the openings, including their size when applicable. In the naming convention, "SW" represents a solid wall, "C" indicates a centrally located opening, while "Wop/Dop" refers to window or door openings. The numbers "1" and "2" refer to aspect ratios (1 corresponds to AR = 0.5, and 2 corresponds to AR = 0.71). "EC" represents a window opening larger than 1.5 m², and "conA" and "conB" denote the size of the internal tie-columns adjacent to the openings. For "conA", the tie-column size is 25x25 cm, while for "conB", it is 15x15 cm. All walls feature tie-columns measuring 25x25 cm along the sides.

Two aspect ratios (H_w/L_w=0.5 and H_w/L_w=0.71) were considered, with a fixed wall height of 2.5 m and wall lengths of 5 m and 3.5 m, ensuring compliance with Eurocode 8 [2], which stipulates that the horizontal spacing between vertical confining elements should not exceed 5 m. These geometrical configurations were chosen to ensure that the numerical models accurately reflect real CM walls, with dimensions and proportions which can be found in CM masonry structures.

For AR=0.5, the window dimensions were 1.0x1.25 m, and for AR=0.71, they were 1.0x0.75 m. The opening percentages (O_p) for these windows were 10% and 8.6%, respectively. Because both of these windows are small enough, codes and guidelines do not require to build tie-columns next to them. The limit for this is 10% of the CM wall area according to Brzev and Mitra [16] and Meli et al. [1], and 1.5 m² according to Eurocode 8 [2]. The windows were considered in central position. To analyze a case with a large enough window to require tie-columns, one case per AR was considered with window sized 1.25x1.25 m (area of 1.56 m²). In this case, the O_p were 12.5% and 17.9% for walls with AR of 0.5 and 0.71, respectively. These configurations were also considered with the placement of 15x15 cm and 25x25 cm tie-columns. The door opening dimensions were 1.0x2.25 m, resulting in O_p of 18% and 25.7% for walls with AR=0.5 and AR=0.71, respectively. The windows and doors were considered only in the central position.

Table 1: Overview of the parametric study.

Model No.	Model Name	Aspect Ratio	Opening Shape/Position	Opening Size	Internal Tie-Column Size
1	SW_1	0.5	Solid wall	-	-
2	CWop_1	0.5	Central window	1.0 x 1.25 m	-
3	CWopEC_1	0.5	Central window	1.25 x 1.25 m	-
4	CDop_1	0.5	Central door	1.0 x 2.25 m	-
5	CWopEC_1-conA	0.5	Central window	1.25 x 1.25 m	25 x 25 cm
6	CWopEC_1-conB	0.5	Central window	1.25 x 1.25 m	15 x 15 cm
7	SW_2	0.71	Solid wall	-	-
8	CWop_2	0.71	Central window	0.75 x 1.0 m	-
9	CWopEC_2	0.71	Central window	1.25 x 1.25 m	-
10	CDop_2	0.71	Central door	1.0 x 2.25 m	-
11	CWopEC_2-conA	0.71	Central window	1.25 x 1.25 m	25 x 25 cm
12	CWopEC_2-conB	0.71	Central window	1.25 x 1.25 m	15 x 15 cm

The Effect of Opening Size on the Seismic Response of CM Walls

To examine the influence of opening size on the seismic response of CM walls, walls with central position of the opening and no internal tie-columns were compared with the reference solid walls. The pushover curves for the analyzed CM walls with AR=0.5 and AR=0.71, and with varying opening size (window or door type) are shown in Fig. 5a and 5b, respectively. It is evident that the presence of openings significantly reduces the strength of the walls compared to the solid ones (SW1, SW2). For example, comparing the solid wall SW_1 with the wall containing a 10% opening (CWop_1), the reduction in strength is 23%. Similarly, for walls with AR = 0.71, the wall CWop_2, which has an opening of 8.6%, exhibits the same 23% reduction in strength compared to the solid wall SW_2. When comparing walls with larger openings, the strength loss becomes even more pronounced. For walls with AR = 0.5, an increase in opening size from 10% (CWop_1) to 12.5% (CWopEC_1), leading to a strength reduction of 27% compared to the solid wall SW_1. On the other hand, for walls with AR = 0.71, the effect of larger openings is more severe. For instance, increasing the opening size from 8.6% to 17.9% (CWopEC_2) results in a 38% strength loss compared to the solid wall SW2.

These findings highlight a clear relationship between the size of the openings, the aspect ratio (AR), and the reduction in wall strength. As the opening size increases, the strength reduction becomes more significant, with walls having AR = 0.71 experiencing a more substantial loss in load-bearing capacity due to the amplified effect of larger openings.

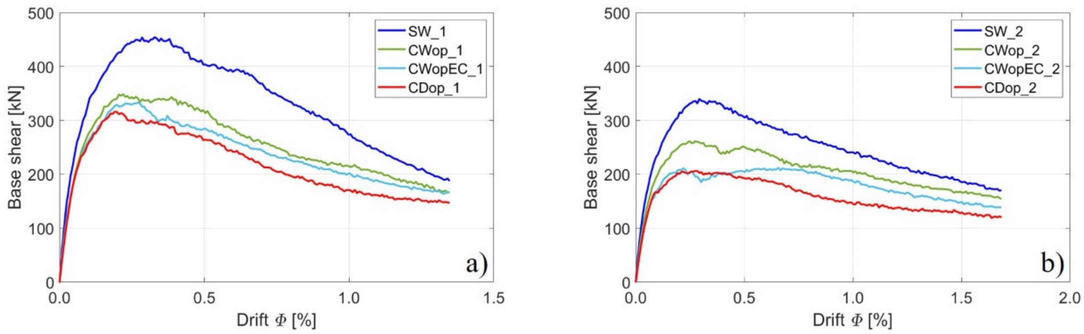

Figure 5: Pushover curves for the CM walls with different size of openings.

Fig. 6 illustrates the distribution of minimum principal stresses at the maximum resistance for only 2 configurations (because of limited space). The diagonal struts are clearly visible in the solid wall SW_2 (Fig. 6, left), indicating an efficient transfer of forces through the masonry. Interestingly, the inclination and width of the struts is not affected by the aspect ratio, and appear to be only a function of the unit overlap between rows. In CM walls with small openings, such as CWop_2 (Fig. 6, right), the stress flow is interrupted around the openings. Diagonal struts are observed to the left and right of the central window, but no stress bands form directly around the opening. Instead, stress concentrations occur at the corners of the window, particularly near the top and bottom edges.

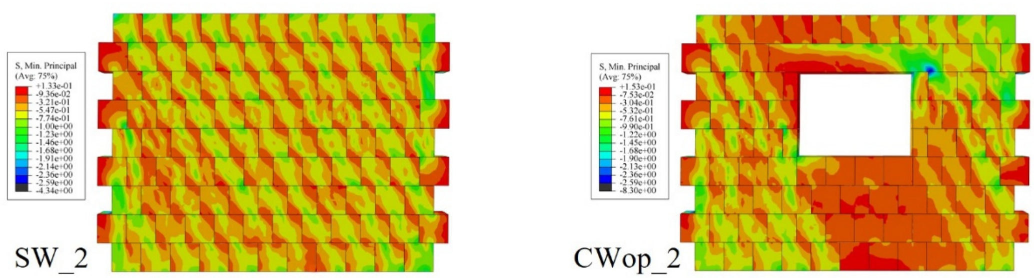

Figure 6: Distribution of min. principal stresses at drift corresponds to the max. resistance.

In-Plane Strength Reduction for CM Walls Without Internal Tie-columns

The strength reduction factor (R_s) is defined as the ratio of the strength of a wall with openings to that of a solid wall, and is one of the key parameters for design. The predicted strength reduction in CM walls with openings are taken from the FEM model. The obtained strength reduction factors were compared to the predictions of the model of Al-Chaar et al. [17] (Fig. 7a), and to the model of Basha et al. [18] (Fig. 7b). Both models are explicit functions ratio of opening area to the area of CM wall panel A_R. The accuracy of these equations was evaluated using the normalized strength reduction factor N_s, defined as the ratio of the predicted to the numerically (from FEM analyses) calculated strength reduction factor. Values of $N_s > 1$ indicate overprediction, which can lead to unsafe estimates of in-plane shear capacity, while $N_s < 1$ represents underprediction, resulting in conservative estimates. The results indicate that model of Basha et al. [18] aligns better with the results of the FEM analyses.

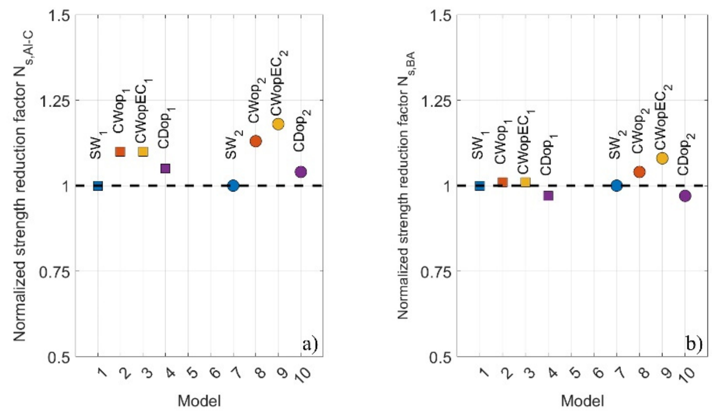

Figure 7: Effect of the openings through normalized strength reduction factor N_s.

Effect of Internal Tie-columns Around Openings

The considered wall configurations with two sizes of internal tie-columns (15x15 cm and 25x25 cm) are illustrated in Fig. 8.

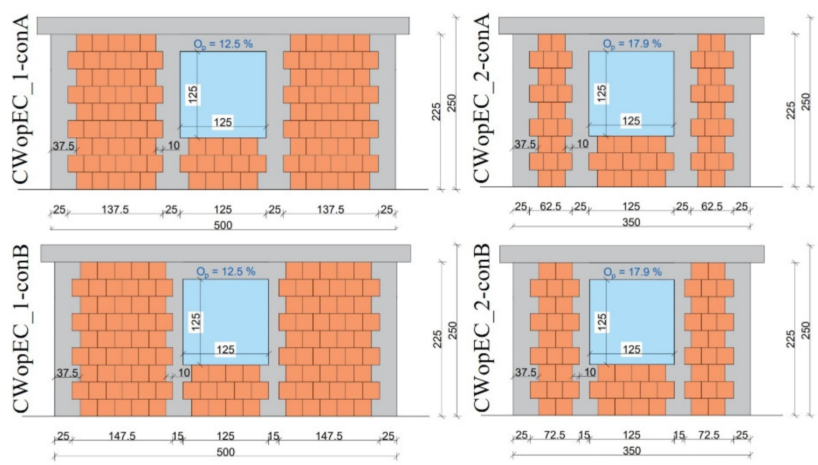

Figure 8: CM wall configurations with internal tie-columns.

The pushover curves presented in Fig. 9 show how much RC tie-columns around window openings improve peak strength and drift at peak strength, with larger tie-columns providing substantially larger improvements, as well as better post-peak behaviour.

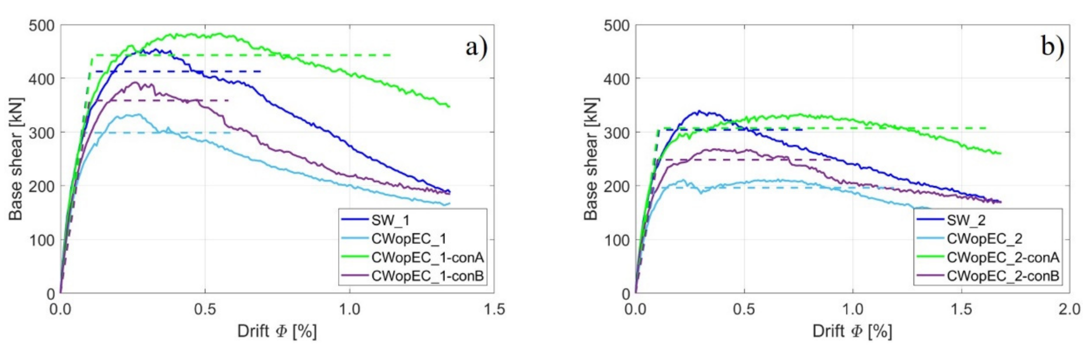

Figure 9: Pushover curves for the CM walls with and without internal tie-columns.

For walls with AR=0.5, the model with 25x25 cm tie-columns (green solid curve in Fig. 9a) shows an increase in peak strength of 45%, relative to the model without tie-columns (light blue curve, Fig. 9a). The drift at maximum strength increases by 212%, from 0.26% for the model CWopEC_1 to 0.55% for the model CWopEC_1-conA. In contrast, the model with smaller 15x15 cm tie-columns provides a more modest improvement, with 18% improvement in strength and no change in drift at peak strength.

Similarly, for walls with AR=0.71, the model with 25x25 cm tie-columns shows a 57% increase in peak strength (green and blue light solid curve in Fig. 9b) and a 330% increase in drift at maximum strength. This notable increase in drift highlights also the enhanced ductility of the CM wall. On the other hand, the model with 15x15 cm tie-columns provides a more limited improvement, with a 26% increase in the peak strength and a 61% increase in drift, from 0.23% to 0.37%.

It is interesting to note that the addition 25x25 cm tie-columns around window openings improves seismic performance of walls even compared to the solid wall. For walls with AR=0.5, the strength increases by 7% compared to SW_1. For walls with AR=0.71, 25x25 cm tie-columns give almost identical strength compared to the solid wall (SW_2).

To estimate the effect of tie-column sizes on the ductility of considered walls with window openings, the response curves were idealized into bilinear curves (depicted by dashed lines in the same colour), as shown in Fig. 9. The idealized curves have equal energy as the original one (areas both curves are the same). These results of the idealization are presented in Table 2 and show that 25x25 cm tie-columns substantially improve ductility, whereas the effect is much less for walls with 15x15 cm tie-columns. Higher ductility indicates a much more gradual post peak degradation and higher capacity for energy dissipation.

Table 2: Strength and ductility of CM walls with window openings and differently sized tie-columns around openings.

Model Name	Internal tie-column	Idealized resistance	Relative to SW_1	Ductility	Relative to SW_1
SW_1	-	412.6 kN	-	7.0	-
CWopEC_1	-	298.6 kN	0.72	7.0	1.00
CWopEC_1-conA	25 x 25 cm	443.1 kN	1.07	10.4	1.49
CWopEC_1-conB	15 x 15 cm	358.8 kN	0.87	5.8	0.83
SW_2	-	304.3 kN	-	7.2	-
CWopEC_2	-	196.5 kN	0.65	14.0	1.94
CWopEC_2-conA	25 x 25 cm	307.5 kN	1.01	15.5	2.15
CWopEC_2-conB	15 x 15 cm	248.6 kN	0.82	8.9	1.24

Overall, the inclusion of larger tie-columns (25x25 cm) significantly improved the seismic performance of the walls by reducing damage concentrations, better distributing stress, and limiting crack propagation compared to smaller tie-columns (15x15 cm) or reference models with unconfined window openings.

CONCLUSIONS

The paper studies the effect of the window/door openings and tie-columns on the response of CM walls using a pushover analysis and a FEM model. The results show the large effect that openings have on the seismic performance, regardless of their size. Even in cases, when the opening was less than 10% of the CM wall area, the resistance decreased by more than 15%. Furthermore, the post-peak response and ductility were affected as well. Additionally, the results show that adding tie-columns next to windows proves to be an effective measure to reduce the detrimental effect of openings. It was found that it was much better if tie-columns next to windows were 25x25 cm than 15x15 cm, because the effect of smaller tie-columns was much less. The strength reduction factor due to openings, as obtained from the present analysis, aligns quite well with the predictions of the model of Basha et al. [18], which confirms the suitability of this model for the considered type of masonry.

It should be noted that all of the conclusions presented here are relevant only for the considered type of masonry and are solely based on numerical simulations. Further experimental tests should be conducted to fully validate these conclusions.

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