

Estimating the In-plane Load-bearing Capacity of Reinforced Masonry Walls Using a Simple Numerical Approach

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Abstract

Accurately assessing the structural behavior of existing masonry structures is challenging. The characteristic properties of these structures, particularly the materials used in their construction, often exhibit significant variability. As a result, various assumptions must be made in numerical analyses to represent the structure, along with data obtained from experimental studies. Moreover, reinforcement applications introduce additional complexity into the numerical analysis process. In this study, a simplified method known as the truss model was tested to address this issue. This method, developed by the authors and validated in a previous article, was used to numerically evaluate various reinforcement applications in masonry walls selected from the literature. The investigation focused particularly on the in-plane load-bearing capacity of reinforced walls. In addition, the failure mechanisms and damage patterns of the walls were assessed. The results showed that the in-plane behavior of reinforced masonry walls, including load-bearing capacity and damage development, can be approximately determined using the proposed approach.

Keywords

reinforced masonry walls, simplified numerical model, in-plane structural behavior, capacity curves, truss model

1 Introduction

With the advancement of computational technologies, the analysis of masonry structures has become more efficient and accurate. A wide range of numerical methods, ranging from highly complex to simplified approaches, has been proposed in the literature [1–4]. Each method presents distinct advantages and limitations, depending on the specific application and analysis objectives. The accuracy of the results can vary significantly based on the selected method and the input parameters used.

Simplified methods reduce the workload compared to more complex approaches, offering a more straightforward design process and requiring less computational effort. As a result, the sensitivity or accuracy of the results may be limited. However, complex models do not always meet expectations due to uncertainties in material properties and/or numerical modeling.

Various simplified methods, such as the equivalent frame method, the truss method, and the strut-and-tie method, have been proposed in the literature. Truss members act only along their longitudinal axis and have a straightforward theoretical basis. The idealization of masonry structures using

truss members is a relatively recent topic, and the number of related studies remains limited [5–7]. Hybrid calculation models have also been proposed by researchers [8–9]. In addition, the idealization and evaluation of masonry infilled frames using the Multi Pier Method were presented by Ahmadvand et al. [10]. Each method offers alternative design standards and analysis procedures.

The evaluation of the structural performance of reinforced masonry structures is also a significant challenge. The reinforcement of masonry structures is a comprehensive topic and can be categorized in various ways [11–12]. To begin with, the characteristic properties of a masonry structure and its materials must be identified. Subsequently, potential damage zones and failure mechanisms should be assessed. Based on these findings, an appropriate reinforcement method can be proposed, with all related calculations being scientifically justified.

Fiber reinforced polymer (FRP) materials may be preferred for the strengthening of masonry structures due to their high tensile strength and ease of installation [13–15]. The effectiveness of reinforcement in masonry structures has

also been investigated through numerical studies, and corresponding design criteria have been established [16–18]. Moreover, numerous experimental and numerical studies evaluating alternative reinforcement strategies have been reported in the literature [19–22].

The evaluation of reinforced masonry structures using truss members has been addressed in several studies. FRP delamination on masonry substrates was investigated, and the proposed method was able to predict the debonding behavior of the composite systems with acceptable accuracy, both in terms of the global load-slip curve and local actions [23]. In another study, a masonry arch reinforced with steel-reinforced grout was analyzed, and the proposed procedure yielded robust results regarding global force-displacement curves and failure mechanisms [24]. This method was also applied to predict the failure of historical masonry vaults and to evaluate the effects of possible retrofitting interventions [25].

An alternative study investigated reinforced masonry walls (RWs) with fully grouted sections, that is, sections where all the voids in the hollow concrete units were filled with grout. The numerical results were reported to be in good agreement with the experimental findings. Moreover, the proposed method successfully captured the hysteretic behavior and damage patterns of shear-dominated walls [26]. Another study focused on the out-of-plane resistance of masonry walls reinforced with FRP. The method yielded satisfactory results when the reinforcement was applied uniformly in direction and aligned parallel to the span of the structure [7].

The truss method used in this study shows similarities to the Discrete Macro-Element Model developed and verified for assessing the in-plane behavior of masonry structures [27], considering the theoretical background of the latter. Masonry piers and spandrels can be idealized as articulated quadrilaterals constituted by four rigid edges connected by four hinges and two diagonal nonlinear springs. Each side of the panel can interact with other panels, elements, or supports through a discrete distribution of nonlinear springs. This proposed approach was later adapted for the evaluation of masonry arches [28] and for the investigation of strengthening of externally bonded composite masonry structures [29].

In this study, the truss method was employed to assess the in-plane behavior of reinforced masonry walls incorporating alternative materials and configurations under lateral loading. Four reinforced masonry walls identified from the literature were idealized and analyzed numerically.

The study specifically focused on strengthening with carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP). Additionally, a case was included to investigate the influence of steel bars placed in both horizontal and vertical directions.

The numerical approach adopted in the analyses was previously presented by Güner and Nuhoglu [30]. That study focused on developing the theoretical framework and validating the proposed numerical method using various masonry walls. In the present study, the model was further evaluated for reinforced masonry walls, with particular emphasis on the idealization of the strengthening techniques and the agreement between the numerical predictions and the experimental results. Accordingly, this study proposes a simplified numerical approach for evaluating the effects of strengthening on masonry walls.

In this study, the key parameters of the proposed analysis model, along with the idealization of masonry walls and strengthening techniques, are described step by step in Section 2. Section 3 presents the numerical validation, compares the predicted damage patterns with the experimental observations, and discusses the results in detail. Finally, the main findings are summarized in the Conclusions section.

2 Description of the analysis model

The methodology of the analysis model was presented in detail by the authors in the previous manuscript [30]. In this section, the modeling approach and its general working principles are introduced. In the proposed model, masonry walls are idealized solely using nonlinear truss elements oriented in the vertical, horizontal, and diagonal directions (Fig. 1). These elements act only along their longitudinal axes, which facilitates the formulation of the theoretical basis of the numerical model.

The idealization of masonry walls begins with the determination of material properties and mesh size. Based on the selected mesh size, the cross-sectional areas of the truss members are calculated. External loads are then applied to the nodes connecting the truss members, and the appropriate boundary conditions, including support constraints, are

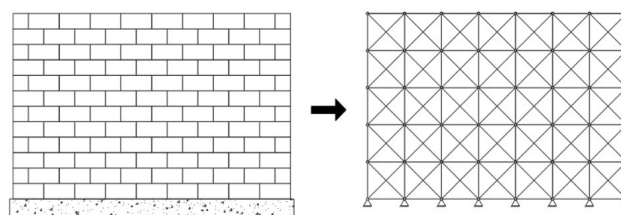


Fig. 1 Idealization of the masonry wall using truss members

assigned. Subsequently, an iterative analysis is performed. First, the vertical and dead loads are applied, followed by horizontal loading. At the end of each iteration, the analysis results are recorded, and the maximum horizontal displacement is evaluated against a predefined error threshold. Once the analysis is complete, the main outputs, including the capacity curve, stress distribution, support reactions, and a visual representation of the damage pattern, are obtained. Each stage of this analysis procedure is described in detail in the following subsections.

2.1 Material behavior

The simplified method reduces the number of required material parameters. For vertical and diagonal truss members, material behavior under compressive or tensile stress is described using a bilinear stress-strain relationship (Fig. 2(a)). However, for horizontally oriented truss members, an alternative tensile behavior is adopted, as illustrated in Fig. 2(b). These members are not responsible for transferring the applied loads to the supports; instead, they provide horizontal stability. Accordingly, the maximum stress capacity ($\sigma_{\max-h}$) and axial strain ($\epsilon_{\max-h}$) are defined to represent the shear behavior of the masonry wall. Specifically, these parameters enable the horizontal truss members to simulate sliding along the horizontal bed joints. The compressive behavior is identical for all truss members, as illustrated in Fig. 2(a).

Depending on the type of loading (compression or tension) acting on a truss member, the appropriate parameter values for the stress-strain relationship are assigned automatically. ϵ_{cr} and ϵ_{cr-h} define the strain at the end of the linear elastic region. When the maximum axial strain ($\epsilon_{\max}$ or $\epsilon_{\max-h}$) is exceeded, the truss member is considered to have lost its load-bearing capacity.

In published studies, the elastic modulus and compressive strength are generally reported. However, not all material parameters are provided. Therefore, the maximum strain values are assumed to be constant for each sample based on the available literature. In Fig. 2(a), $\epsilon_{\max}$ is assumed to

be 0.001 under tensile stress and $2.75\epsilon_{cr}$ under compressive stress [6]. $\sigma_{\max}$ is taken as the compressive or tensile strength reported in the literature. In Fig. 2(b), $\epsilon_{\max-h}$ is taken as 0.008 [31], and $\sigma_{\max-h}$ can be calculated using the Mohr-Coulomb formula (Eq. (1)) if it is not determined experimentally. However, this value may not always be appropriate for numerical analysis; therefore, it can be adjusted to improve the sensitivity and reliability of the results. In Eq. (1), c , μ , and σ_v represent cohesion, friction coefficient, and vertical stress, respectively. The friction coefficient can be assumed to be 0.4 [32]. The vertical stress is obtained by dividing the axial load acting on the masonry wall by its cross-sectional area.

$$\sigma_{\max-h} = c + \mu\sigma_v \quad (1)$$

2.2 Softening behavior

In numerical analysis, the total external load is evenly divided among all iterations. At each iteration, a small external force is applied to the structure, and the axial strain of each member is calculated using the Finite Element Displacement Method. When the cumulative axial strain in any truss member exceeds its corresponding strain limit (ϵ_{cr} or ϵ_{cr-h}), a new negative elastic modulus is assigned to that member based on the slope of the softening curve. In this way, any reduction in the load-bearing capacity of the truss member is reflected in the overall structural stiffness. The same approach has been proposed in other studies in the literature [33–34].

If the axial strain exceeds the maximum allowable strain, the load-bearing capacity of the truss member is assumed to be negligible; in other words, the elastic modulus is assigned an extremely small value.

2.3 Mesh size and cross-sectional areas of truss members

In the idealization of masonry walls using truss members, the mesh size (M) plays a critical role. The masonry wall is idealized using a macro-modeling approach, allowing different mesh sizes to be adopted. However, when a small mesh size is selected, the system's load-bearing capacity is often predicted to be higher, particularly in the plastic strain region, due to the increased number of elements. A smaller mesh size also leads to an increase in the total analysis time. Conversely, when a larger mesh size is used, the structural behavior and damage formation may not be captured effectively. In some cases, the mesh size can affect the predicted capacity curve and the convergence of the numerical analysis. In such cases, different mesh sizes can be adopted to assess the influence of

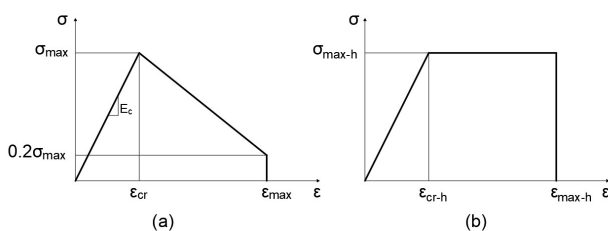


Fig. 2 Material behavior of truss members: (a) vertical and diagonal members; (b) horizontal members

mesh discretization on the numerical results and to verify the robustness of the proposed model.

A relationship between mesh size and total wall height was established, taking into account the aspect ratio. It was recommended that the mesh size-to-total height ratio should be maintained within the range of 5–11% [30]. In addition, geometric constraints, such as wall dimensions, play a determining role. In this study, the representation of the reinforcement application in the numerical model must also be considered when determining the mesh size.

Each truss member is connected to another through a node that moves along the X and Y axes. Thus, each node has two degrees of freedom. Changes in mesh size directly affect the number of nodes, and consequently, the computational burden may increase or decrease. In the samples considered in this study, the number of nodes ranged from 265 to 377. For low-slenderness walls such as those considered in this study, the mesh size can be selected close to the upper limit of the recommended mesh size-to-total height ratio. However, the representation of the strengthen- ing method in the truss model limits this option.

The overall dimensions of the numerical model are matched as closely as possible to those of the experimen- tal structure. Minor geometric discrepancies are com- pensated for by adjusting the cross-sectional areas of the outer truss members.

In determining the effective cross-sectional areas, the approach proposed in the literature for the idealization of reinforced concrete members using a truss-based anal- ogy was considered [35]. This approach was subsequently significantly revised and adapted for the application to masonry walls. Thus, the cross-sectional areas of the truss members are determined by multiplying the effective sur- face width associated with the interior and outer members by the wall thickness. For the diagonal members, an equiv- alent cross-sectional area is defined to account for their inclined geometry. The inclination angle (θ_d) is taken as 45° , based on the literature review [23, 26] and the authors' previous investigation [36]. The effective surface widths associated with the interior, outer, and diagonal members are illustrated as shaded regions in Fig. 3.

2.4 Loading

External loads in both the vertical and horizontal directions are applied through a steel beam idealized using truss mem- bers (Fig. 4). This beam exhibits linear behavior and has a higher elastic modulus compared to the truss members rep- resenting the masonry wall. Additionally, the nodes on the beam

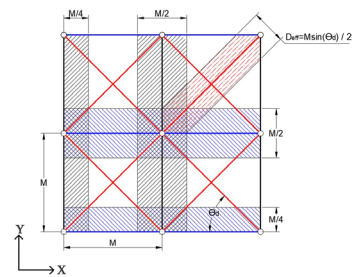


Fig. 3 Effective surface widths of the truss members

are connected to a single master node using the Master-Slave Method, thereby restricting the beam's in-plane rotation.

The wall is first subjected to an axial load, followed by a lateral load. The axial load is distributed among the nodes shown in Fig. 4. In the upper row, new nodes are added at the intersections of the diagonal members, as this arrange- ment enhances the load-bearing capacity by improving the distribution of axial stress, particularly in the diag- onal members. Then, the horizontal load, which is distrib- uted equally among the nodes in the upper row, is applied. Finally, if the self-weight of the wall is considered in the analysis, the total weight is equally distributed among all nodes in the model before the analysis begins.

Since the loads are applied incrementally, the analysis is conducted using a pushover procedure. Pushover analy- sis is commonly used in the assessment of masonry struc- tures [37–38]. The number of iterations can influence the results; therefore, an optimal value should be determined through several preliminary analyses. If a low number of iter- ations is selected, the accuracy of the results may decrease. Conversely, selecting a high number of iterations increases the total analysis time.

The loading is applied in a load-controlled manner; therefore, the analysis stops when the maximum load is reached. To identify this maximum load, displacement control is performed at the nodes. Specifically, the hori- zontal displacement at the node moving in the direction opposite to the applied load is measured and compared

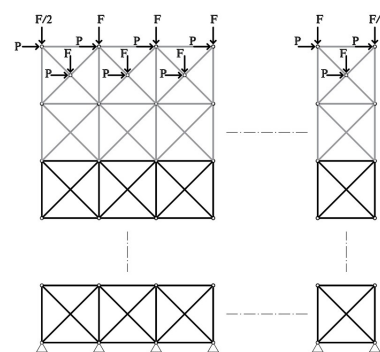


Fig. 4 Idealization of the steel beam (gray) and the masonry wall (black)

with the error threshold defined at the beginning of the analysis. If the deviation exceeds this threshold, the analysis is terminated. In addition, the capacity curve of the model can be examined to determine the maximum load. However, due to the load-controlled approach, the drop in the capacity curve cannot be represented.

The error threshold is initially set to a very small negative value (assuming the horizontal loading direction is along the positive X-axis) before starting the analysis. After several preliminary analyses and evaluation of the load-displacement curves, an appropriate threshold value can be determined. In the previous study, this value ranged between 0.25 mm and 0.35 mm [30]. In the present study, it was set to 0.05 mm, except for Sample 1, for which a value of 0.25 mm was used.

2.5 Analysis process

The software was developed by the authors using the Python programming language [39] within the PyCharm environment [40]. The analysis process begins with defining the required input data, such as node coordinates and material parameters. Subsequently, calculations are performed, and the results obtained at each iteration, such as strains, member forces, and nodal displacements, are recorded. In the next step, the cumulative displacements are compared with the error threshold to determine whether the analysis should proceed. If so, the process returns to the calculation step with updated geometric and material parameters.

In the final stage, the software outputs the capacity curve for a selected node, the stress distribution in the truss members, the support reactions, and a visual representation of wall damage. Further details of the analysis process are provided in the previous study [30].

2.6 Damage

Truss members deform only along their longitudinal axes and exhibit either linear or nonlinear behavior. At the end of each iteration, the cumulative axial strain in each truss member is calculated and compared with the corresponding stress-strain curve, taking into account the member orientation (horizontal or vertical/diagonal) and the stress state (compression or tension). This approach allows simultaneous observation of damage development in the numerical model.

To facilitate the interpretation of the results, the stress-strain relationships shown in Fig. 2 are divided into three regions based on the axial strain: linear, softening, and failure. The failure region indicates that a truss member has exceeded its maximum allowable axial strain and has lost its load-bearing capacity. The colors representing these regions,

depending on the truss member orientation and the stress state, are summarized in Table 1.

By combining the damage states of all truss members, typical failure modes can be identified, as shown in Fig. 5. When nonlinear behavior occurs in both vertical and diagonal members under compressive stress, toe crushing due to flexural failure can be simulated. Conversely, if truss members are subjected to tensile stress, sliding or diagonal shear failure modes can be simulated. Horizontal members primarily contribute to the overall structural stability. However, once significant horizontal displacement occurs, they can simulate sliding damage along the horizontal bed joints.

2.7 Idealization of reinforcement application

Reinforcement applications are also simplified and represented by truss members. Depending on the position and cross-section of the reinforcement elements, equivalent truss members are defined. In Fig. 6, the first validation wall and its idealized truss model with reinforcement are presented. The wall was reinforced with CFRP in both directions [41]. The positions and widths of the FRP strips were preserved

Table 1 Color-coded scheme for visualizing axial strain levels in truss members

Orientation	Vertical / Diagonal		Horizontal	
Stress state	Compression	Tension	Compression	Tension
Linear	Black	Black	Black	Black
Softening	Green	Yellow	Green	Orange
Failure	Magenta	Red	Magenta	Blue

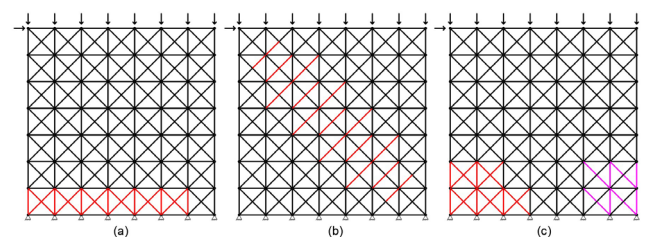


Fig. 5 Representation of typical in-plane failure mechanisms in masonry walls using the truss model: (a) sliding shear failure; (b) diagonal shear failure; (c) flexural failure

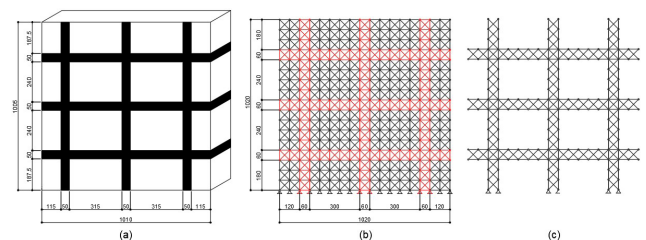


Fig. 6 Reinforcement configuration and its representation in the truss model: (a) RW; (b) truss model; (c) idealization of reinforcement (unit: mm)

in the truss model as accurately as possible. In the numerical model, truss members representing the URW and the reinforcement were connected via nodes. This connection was assumed to exhibit perfect bond behavior and to prevent relative movement at the connected nodes. Rupture and independent movement of the FRP were not considered in the model.

While the vertical and diagonal truss members were used to represent the vertical FRP strips, the horizontal and diagonal truss members were used to represent the lateral FRP strips. The cross-sectional areas of the truss members representing the reinforcement were kept constant in the corresponding directions. For example, the effective area in the vertical direction was calculated as the sum of the cross-sectional areas of two vertical and two diagonal truss members. However, the contribution of the diagonal members was determined by multiplying their cross-sectional areas by $\sin\theta_d$. The sum of these effective areas was set equal to the cross-sectional area of a single FRP strip, from which the cross-sectional area of each truss member was determined.

The software was upgraded to accommodate the requirements of the validation walls. In particular, a new material model was added to represent the behavior of FRP, which acts only under axial tension and exhibits linear elastic behavior. For Sample 4, an alternative material model was also defined to simulate the behavior of steel bars, which exhibit linear elastic behavior under both axial tension and compression. Additional details of the numerical models are provided in the following subsections.

3 Numerical validation

Four different walls reinforced using alternative methods were examined in this study. The dimensions of the unreinforced walls (URWs) and the applied axial stress levels are presented in Table 2. In Samples 3 and 4, the self-weight of the walls was also considered in the numerical analyses due to low or zero axial stress levels. All walls, except for Sample 2, were tested under in-plane monotonic lateral loading, while Sample 2 was subjected to quasi-static cyclic loading. For the cyclic loading case, the corresponding capacity curves are also provided in the relevant study.

Table 2 Basic information on the validation walls.

Sample	Height (mm)	Length (mm)	Depth (mm)	Axial stress (MPa)
Sample 1 [41]	1005	1010	280	1.06
Sample 2 [42]	1225	1000	250	1.40
Sample 3 [43]	1260	1030	75	0.20
Sample 4 [44]	1280	995	220	-

The analysis process begins with modeling the URW using the proposed truss model, considering the geometric, loading, and support conditions. Next, the material parameters provided in the relevant study are adapted for the numerical model. The reinforcement is then modeled. Finally, the RW is analyzed, and its load-displacement curve is compared with that presented in the literature.

Determining the material parameters required for numerical analysis is not straightforward. In the proposed model, five main parameters are needed: the elastic modulus (E), shear modulus (G), and the maximum compressive, tensile, and shear strengths. The shear strength refers to the tensile capacity of the horizontal truss members ($\sigma_{\max-h}$).

The elastic modulus is one of the key material parameters influencing the results, particularly in assessing structural stiffness. The agreement between the load-displacement curves obtained from numerical and experimental studies is closely related to the elastic modulus [45]. However, accurately determining this parameter for masonry structures is challenging due to several factors, such as the type of brick or stone bonds, material quality, mortar mix ratio, and workmanship. Furthermore, the selected method, whether analytical or experimental, may also affect the results [46–47]. Therefore, an elastic modulus reported in the literature was initially adopted for the numerical analyses and subsequently updated based on the agreement between the linear portions of the numerical and experimental load-displacement curves.

Not all the required material parameters can be obtained from the literature. Therefore, simplifications were made for the remaining parameters. The shear modulus and maximum tensile strength were assumed to be $0.4E$ and 0.10 MPa, respectively. The maximum compressive and shear strengths were determined based on values reported in the literature, observed damage patterns, and numerical analysis results. The material parameters of the masonry walls are presented in Table 3.

3.1 Sample 1

Doran et al. [41] constructed and tested double-wythe stone masonry walls at Yildiz Technical University. Table 2

Table 3 Material parameters used in the numerical model (MPa)

Sample	E	f_{cm}	$\sigma_{\max-h}$
Sample 1 [41]	2500	8.00	0.72
Sample 2 [42]	1310	13.84	0.66; 0.60; 0.55
Sample 3 [43]	2000	6.50	0.25
Sample 4 [44]	10000	6.40	0.14

summarizes the wall geometries and the constant axial stress level. Two walls were strengthened using different CFRP configurations: double-sided diagonal strips (LP_K , Fig. 7) and grid strips (LP_T , Fig. 6). The widths of the fiber strips were constant at 50 mm and 125 mm for the grid and diagonal layouts, respectively. The CFRP thickness was 0.17 mm, and its elastic modulus was 230 GPa. The walls were initially pre-damaged, subsequently strengthened, and then re-tested.

The URW was modeled using 1,190 truss members without the steel beam in the proposed approach. The reinforcement applications were then modeled using 408 and 260 truss members for LP_T and LP_K , respectively. The cross-sectional areas of the truss members were 4.98 mm² and 5.43 mm² for LP_T and LP_K , respectively. The optimum mesh size was determined to be 60 mm.

The load–displacement curves of the URW and RWs are presented in Fig. 8. The truss model predictions mostly overlap with the experimental results. Since the loading is load-controlled, the analysis terminates after reaching the peak load; therefore, the softening behavior is not captured. The reinforcement applications significantly contribute to the load-bearing capacity in the plastic range. The similarity in the initial stiffness of the numerical models is also reflected by the natural period values, which are 0.102 s, 0.101 s, and 0.101 s for URW, LP_T , and LP_K , respectively.

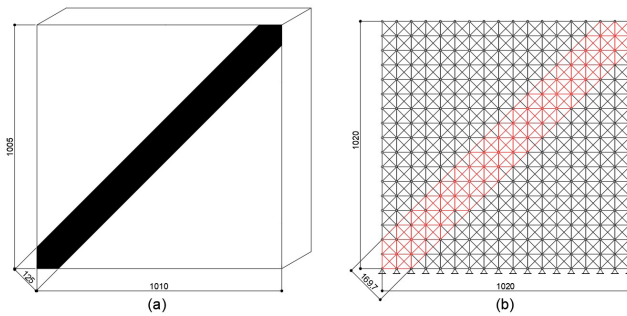


Fig. 7 Reinforcement configuration and its representation in the truss model (LP_K): (a) RW; (b) truss model (unit: mm)

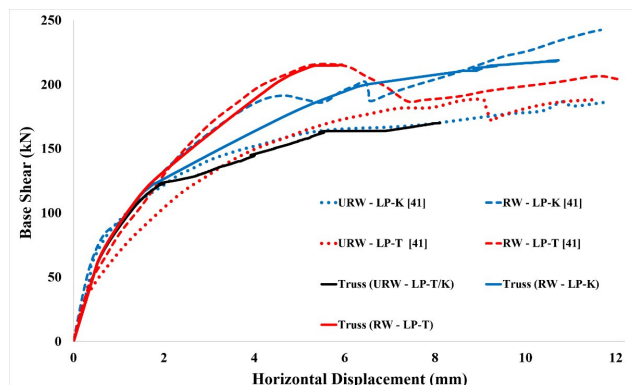


Fig. 8 Load-displacement curves

3.2 Sample 2

An experimental study conducted by Vienni et al. [42] evaluated the effect of composite reinforced mortar (CRM) on the seismic response of brick masonry walls. The CRM system, composed of a glass fiber grid embedded in a 30 mm-thick lime mortar matrix, was applied to walls on either one side or both sides. In addition, an URW was used as a reference. The geometric properties and axial stress level are presented in Table 2. However, the free height of the wall was 720 mm, as opposed to the total wall height.

The glass fiber grid had a square mesh with a pitch of 80 mm between yarns. The cross-sectional area of a single yarn was 11.9 mm² in the longitudinal direction and 9.9 mm² in the transverse direction. The elastic modulus of the glass fiber grid was 35.1 GPa and 56.5 GPa in the longitudinal and transverse directions, respectively. The average compressive strengths of the mortar matrix and small masonry panels were 14.38 MPa and 13.80 MPa, respectively.

In the proposed model, the URW was modeled with 808 truss members without the steel beam (Fig. 9). To ensure compatibility with the glass fiber grid, a mesh size of 80 mm was adopted in both the longitudinal and transverse directions. Accordingly, the glass fiber grid was modeled by incorporating an additional 418 truss members. Furthermore, the presence of a 30 mm-thick mortar layer on either one or both sides was accounted for by updating the cross-sectional areas of the truss members representing the masonry wall. To ensure a clear height, the horizontal movement of the corner nodes was restricted from the ground level up to 480 mm.

The maximum shear strength capacity ($\sigma_{\max-h}$) was calculated separately for each wall using the Mohr-Coulomb formula due to variations in wall thickness. In the calculations, the cohesion value was assumed to be equal to the tensile strength (0.10 MPa). As a result, shear strengths of 0.66 MPa, 0.60 MPa, and 0.55 MPa were determined for the URW, RW one-sided, and RW two-sided walls, respectively.

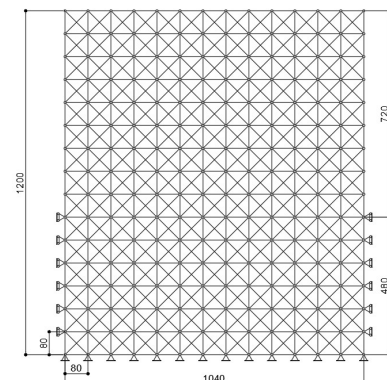


Fig. 9 Truss model (unit: mm)

The capacity curves are shown in Fig. 10. The elastic behavior and the onset of plastic behavior are accurately captured. Moreover, the horizontal displacements corresponding to the maximum load-bearing capacities are accurately predicted for all cases except the URW. However, the numerical analyses predict higher load-bearing capacities. This discrepancy is largely attributed to the computational model implemented in the analysis software, particularly the type of loading applied.

Changes in the initial stiffness of the walls can be inferred from their natural periods. The natural period decreases by approximately 12.6%, from 0.150 s (URW) to 0.133 s (RW two-sided) in the numerical model. Thus, the wall reinforced on both sides exhibits higher initial stiffness, which is consistent with the capacity curve.

3.3 Sample 3

The behavior of hollow concrete brick masonry walls reinforced with composite materials was experimentally investigated by Bui et al. [43]. In this study, an unreinforced wall and a reinforced wall strengthened with carbon sheets applied to both faces were examined. Each sheet was 60 mm wide and 0.48 mm thick, with an elastic modulus of 105 GPa. The vertical reinforcement ratio was 0.3.

The geometric configuration of the sheets and the idealized truss model are presented in Fig. 11. Additionally, to prevent wall overturning under lateral loading and to maximize the effectiveness of the reinforcement application, CFRP anchors, shown in magenta in Fig. 11, were employed.

A total of 1,113 truss members were used to idealize the URW without the steel beam. The optimum mesh size was determined to be 70 mm, considering both geometric constraints and the total number of members. To represent the reinforcement, 288 and 24 truss members were used for the carbon sheets and anchors, respectively. The cross-sectional areas of the truss members were 1,696.95 mm² for the sheets,

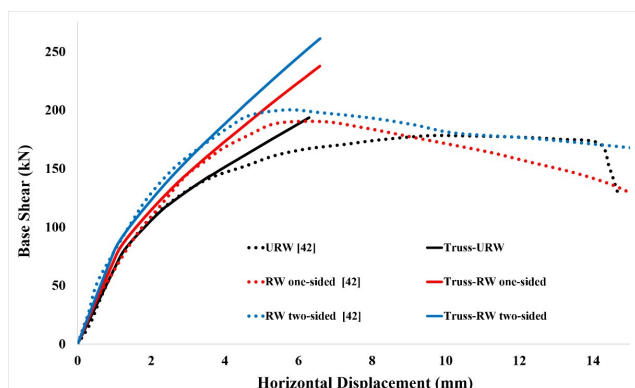


Fig. 10 Load-displacement curves

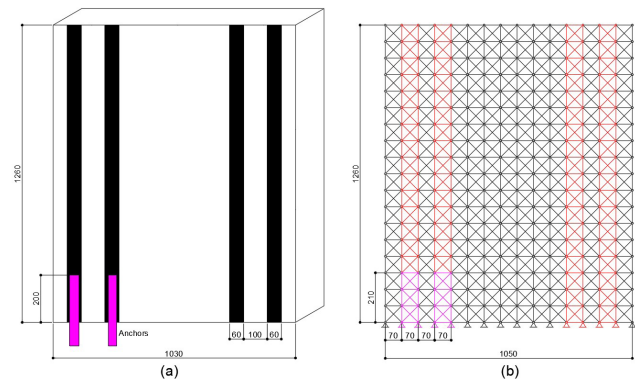


Fig. 11 Reinforcement configuration and its representation in the truss model: (a) RW; (b) truss model (unit: mm)

based on the vertical reinforcement ratio, and 100 mm² for the anchors. The maximum compressive strength of the wall was assumed to be equal to that of the individual masonry blocks, at 6.5 MPa.

The capacity curves are presented in Fig. 12. Although the curves do not overlap exactly, the load-bearing capacities of the URW and the RW are captured consistently by the truss model. The effect of the loading type is particularly noticeable in the URW, and the numerical analyses terminate once the maximum load-bearing capacity is reached. The separation in the elastic region between the two numerical models can be explained by their natural period values, which reflect differences in initial stiffness. With reinforcement, the natural period decreases from 0.062 s to 0.039 s.

3.4 Sample 4

The shear strength and stiffness of masonry walls were evaluated by Madhavi et al. [44]. Walls with various configurations were constructed and tested. One specimen, built in English bond using grooved concrete blocks, was idealized using the truss model approach. Unlike the other samples, this specimen was not subjected to any axial load apart from its self-weight, and reinforcement was provided by steel bars instead

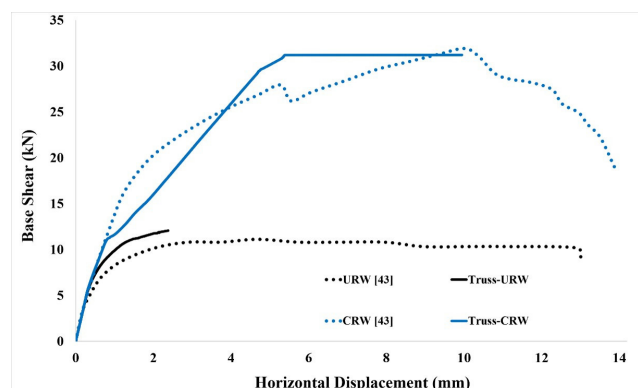


Fig. 12 Load-displacement curves

of carbon or glass fibers. The positions of the 8 mm-diameter steel bars and the corresponding truss model are shown in Fig. 13. The elastic modulus of the steel bars was 210 GPa.

The absence of axial load facilitates the overturning of the wall under lateral loading. To prevent this, a vertical restraint was applied at the loaded end of the wall using the tie-down bars shown in magenta in Fig. 13. Thus, shear failure can occur. In the truss model, the restraint was represented as a bar with a higher elastic modulus of 2100 GPa to minimize axial deformation. This bar acted only in tension and connected the top of the wall directly to the ground.

The wall was modeled using a total of 1,040 truss members, excluding the steel beam. The spacing between truss members in both directions was 70 mm. A total of 50 truss members were used to represent the steel bars. These truss members were capable of exhibiting elastic behavior under both tension and compression.

The compressive strength of the masonry wall was calculated using the mean value obtained from the brick units, and its shear strength was determined as the average of the values obtained from two tested samples in the study (Table 3).

The capacity curves are presented in Fig. 14. RW-1 and RW-2 represent two different test results for the same

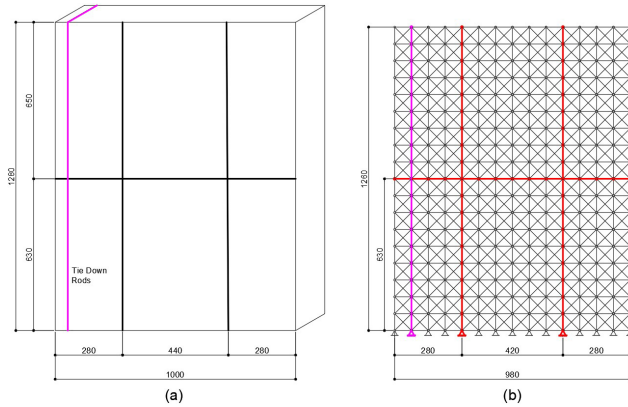


Fig. 13 Reinforcement configuration and its representation in the truss model: (a) RW; (b) truss model (unit: mm)

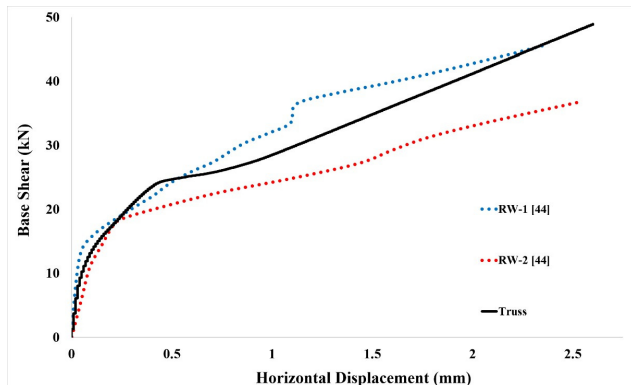


Fig. 14 Load-displacement curves

reinforcement configuration. The numerical analysis curve generally aligns with the first test result, particularly regarding the maximum load and elastic behavior.

3.5 Damage formation

For URWs, the proposed truss model approach allows for the detection of failure mechanisms and damage formation. However, reinforcement applications redistribute stress concentrations within the walls, making the evaluation and interpretation of damage more complex. Unlike detailed numerical models, the proposed approach typically does not allow for the direct identification of damage development. Instead, damage formation is inferred from changes in the strain of truss members and is represented by a limited number of elements.

In Sample 1, diagonal cracks were predominantly observed in both URWs and RWs during the experimental tests. Additionally, toe crushing was detected in the RWs [41]. Similar damage patterns are observed in the truss models (Fig. 15), where diagonal cracks are especially noticeable in the URW and RW with diagonal strips (LP_K). The direction of the cracks, shown by dashed black lines in Fig. 15, can be determined with the help of red and blue truss members. Many horizontal truss members exceed their maximum axial strain limit, highlighting the presence

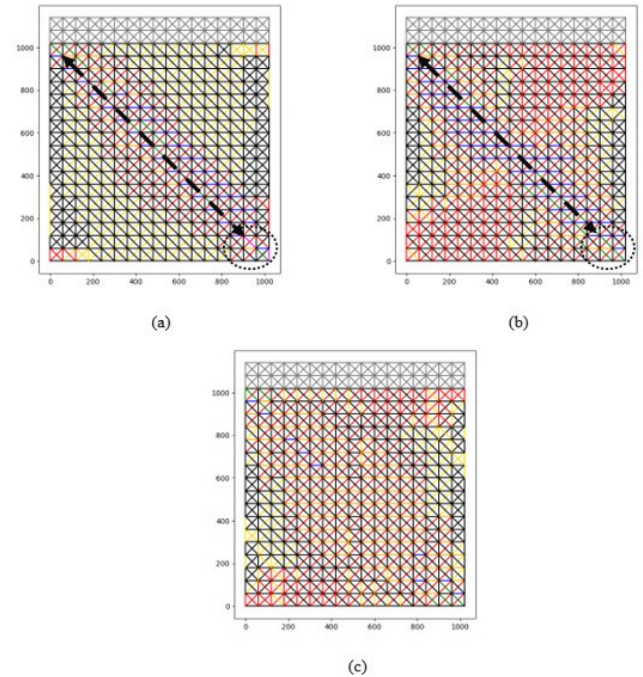


Fig. 15 Damage formation in Sample 1: (a) URW; (b) RW- LP_K ; (c) RW- LP_T (unit: mm). (Red: tensile failure in vertical and diagonal members; Blue: tensile failure in horizontal members; Magenta: compressive failure in all members. For more details, see Table 1)

of prominent diagonal cracking. Furthermore, toe crushing is represented by magenta- and green-colored truss members located at the bottom right. The meanings of these colors are explained in Table 1.

In Sample 3, a flexural failure mode was observed in the URW. Horizontal cracks appeared on the left side of the wall, while toe crushing occurred on the right side. The RW exhibited a combined shear-flexure failure mode. Shear cracks initiated in the middle of the wall and developed into a typical shear failure pattern. Crushing at the lower right corner, due to compression, ultimately led to the collapse of the wall [43]. In the truss model, flexural damage in the URW and diagonal cracking in the RW are clearly visible in Fig. 16. Simultaneous tension damage in the vertical and diagonal truss members indicates sliding shear behavior in the wall. Due to the limited horizontal displacement in the URW, crushing damage is not observed.

Other samples can also be evaluated using a similar approach. In general, the damage patterns and failure mechanisms of masonry walls under in-plane loading can be identified. The approach used to determine damage development in URWs was discussed in detail during the development of the proposed model. However, the effect of reinforcement on damage formation complicates the evaluation process.

3.6 Evaluation of parameters and approaches

Capacity curves of various masonry walls reinforced with alternative procedures can generally be predicted with good consistency. Material properties and the selected calculation model directly influence these results. Matching the curves in the elastic region is relatively easier than matching those in the plastic region. Accurately capturing the nonlinear behavior requires the adopted nonlinear analysis model to be both

appropriate and effective throughout the analysis process. Additionally, factors such as the number of iterations per step, the choice of loading type (load-controlled or displacement-controlled), and the definition of boundary conditions are also critical parameters. The proposed simplified model significantly reduces the computational burden, enabling the evaluation of different parameter effects. This allows for the creation of confidence intervals to determine the upper and lower bounds of the capacity curves.

The mesh sizes selected for each analysis fall within the proposed range of 5–11% [30]. This range was determined based on the relationship between mesh size, aspect ratio, and total wall height. In these samples, the aspect ratio is close to one. The mesh size-to-total wall height ratios range from 5.47% to 6.53%.

A comparison of the maximum load and the corresponding displacement values for both the experimental and numerical results is given in Table 4. The results, particularly the maximum load capacities and the displacements of the reinforced walls, show good agreement. Considering the aforementioned factors, the consistency of the results could be further improved.

The compressive and/or tensile stress levels developed in the truss members representing the reinforcement are generally below the strength capacity values reported in the literature. Only a limited number of members exceed these values. For example, in Sample 2, just one truss member, representing the vertical glass fiber on the reinforced side of the masonry wall (RW one-sided), exceeded its strength capacity. In Samples 1 and 3, no members exceeded their strength capacities. As a result, the assumption of linear behavior for these truss members, which possess significantly higher strength than the masonry load-bearing units, does not cause errors in the analysis results.

4 Conclusions

In this study, a new simplified analysis method was adopted to evaluate the in-plane behavior of reinforced masonry walls. Various reinforcement applications were investigated, and the effectiveness of the proposed approach was demonstrated. Four masonry walls that were experimentally tested were analyzed using pushover analysis. The results, including load-displacement curves, failure mechanisms, and damage patterns, were evaluated in detail.

The proposed truss model can be used to determine the capacity curves of masonry walls. It significantly reduces the computational burden while providing reliable results for both URWs and RWs. Additionally, the model allows

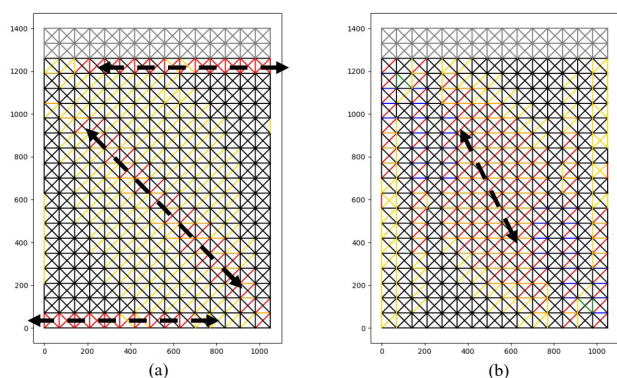


Fig. 16 Damage formation in Sample 3: (a) URW; (b) CRW (unit: mm). (Red: tensile failure in vertical and diagonal members; Blue: tensile failure in horizontal members; Magenta: compressive failure in all members. For more details, see Table 1)

Table 4 Comparison of the experimental and numerical results

	Wall	Maximum Load (kN)			Displacement (mm)		
		Num.	Exp.	Num./Exp.	Num.	Exp.	Num./Exp.
Sample 1	URW	170.02	186.05; 188.12	0.91; 0.90	8.11	11.76; 9.05	0.69; 0.90
	RW - LP_K	218.86	242.42	0.90	10.73	11.67	0.92
	RW - LP_T	215.46	215.37	1.00	4.76	5.30	0.90
Sample 2	URW	193.60	177.98	1.09	6.29	9.48	0.66
	RW one-sided	237.60	190.24	1.25	6.01	6.02	1.00
	RW two-sided	261.10	200.16	1.30	6.59	5.69	1.16
Sample 3	URW	12.07	11.11	1.09	2.38	4.69	0.51
	CRW	31.20	31.91	0.98	9.95	10.00	1.00
Sample 4	RW	48.92	45.66; 36.73	1.08; 1.33	2.60	2.36; 2.53	1.10; 1.03

for the detection of damage patterns in masonry walls under lateral loading. However, reinforcement complicates the interpretation of damage patterns.

Despite simplifications in material parameters and geometry, the success of the proposed approach in estimating the capacity of reinforced masonry walls indicates its potential for further development. Moreover, the calculation model can be improved and implemented by users through freely available software tools.

Mesh size, material parameters, loading type, and analysis procedure may influence the numerical results. To evaluate these effects, the numerical model can be generated with alternative mesh sizes and analyzed using various material parameters, loading types, or analysis

procedures. The resulting capacity curves can then be compared to assess the sensitivity of the model and quantify the variability of the numerical predictions.

The idealization of reinforcement methods also affects both the capacity curves and the damage patterns. Thus, appropriate design criteria can be developed by considering the structural requirements of masonry walls, as well as the reinforcement layout and application procedure.

In this study, a limited number of cases were considered, and the corresponding results were evaluated. In the future, a more comprehensive analysis procedure could be developed by extending the approach to multi-storey and multi-span masonry structures, as well as by incorporating their out-of-plane behavior.

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