

Review

Analysis and Design of Confined Masonry Structures: Review and Future Research Directions

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Abstract: This article discusses the significance of confined masonry (CM) structures in terms of their remarkable seismic performance in past earthquake events. However, the variability of CM structures with differing materials, detailing, and construction practices across different regions poses challenges in developing standardized design guidelines. To address the challenges, the state-of-the-art developments in CM are comprehensively reviewed in the present article. This review encompasses experimental campaigns studying CM walls and buildings to evaluate the effect of important parameters on their performance, a discussion of various numerical and analytical models with their respective benefits and limitations, and an examination of design procedures for CM in nine country codes and their local guidelines. This review identifies gaps in the current knowledge, including the need for more studies on the performance of CM structures under earthquake loads and the use of new materials and construction techniques. The article concludes by formulating future research directions to address the identified gaps, including the need for more experimental studies and the development of sophisticated numerical models that can capture the complexities of these structures under the action of different loads. Overall, the article serves as a valuable resource for researchers and practitioners working on the analysis, design, and construction of CM.

Keywords: confined masonry; seismic behavior; experimental studies; numerical modelling; structural design



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1. Introduction

Throughout history, masonry has been a widely used building material consisting of masonry units and mortar. Its abundance, affordability, and desirable structural and architectural properties have made it a popular choice for constructing various structures, from ancient pyramids to heritage buildings. Combining this versatile material with innovation and technology has resulted in numerous notable masonry structures. In the past, load-bearing unreinforced masonry (URM) constructions were quite commonly used as housing methods even in seismically active areas. However, since the mid-twentieth century, reinforced concrete (RC) frame structures have become prevalent, replacing the thick and heavy load-bearing walls of URM with RC load-bearing frames. As a result, masonry is now primarily used as infill.

The safety of a building is affected by various factors, such as material quality, workmanship, and whether or not the construction and design guidelines were followed. However, the cost of construction is primarily influenced by the availability of local materials and labor. The connection between safety and affordability in housing construction becomes complicated in seismically active regions, more so in under-developed and developing countries, as cost significantly influences the building process. The emphasis on minimizing construction costs frequently leads to substandard-quality construction, even for structures that have accessible design and construction guidelines, such as URM and

RC frame buildings. The endeavor to prioritize affordability over safety has heightened seismic vulnerability in certain seismically active areas, as evidenced by several previous earthquakes [1–6]. Various factors account for the unsatisfactory performance of some of the prevalent housing typologies, including the adoption of cost-cutting measures that overlook seismic features and the utilization of inferior-quality materials and workmanship. It is essential to prioritize safety over affordability and to ensure that construction practices adhere to appropriate guidelines to reduce the risk of damage and loss of life during seismic events.

To achieve safe and affordable housing while preserving the authenticity of masonry construction, various methods of reinforcing masonry walls have emerged over the years. Confined masonry (CM) is one such housing typology that has been proven to be safe and affordable. Initially developed for rebuilding the buildings damaged during the 1908 Messina (Italy) earthquake that had a 7.2 magnitude, the CM technique has since spread to high-seismic-risk countries such as Mexico, Colombia, Chile, and various other countries [7–9]. CM is utilized in non-engineered and engineered residential construction, ranging from single- to six-storey buildings. The CM system involves constructing masonry walls, followed by the cast-in-place of nominally reinforced RC columns and beams at the periphery of each wall and around the openings. The key components of a typical CM building are depicted in Figure 1.

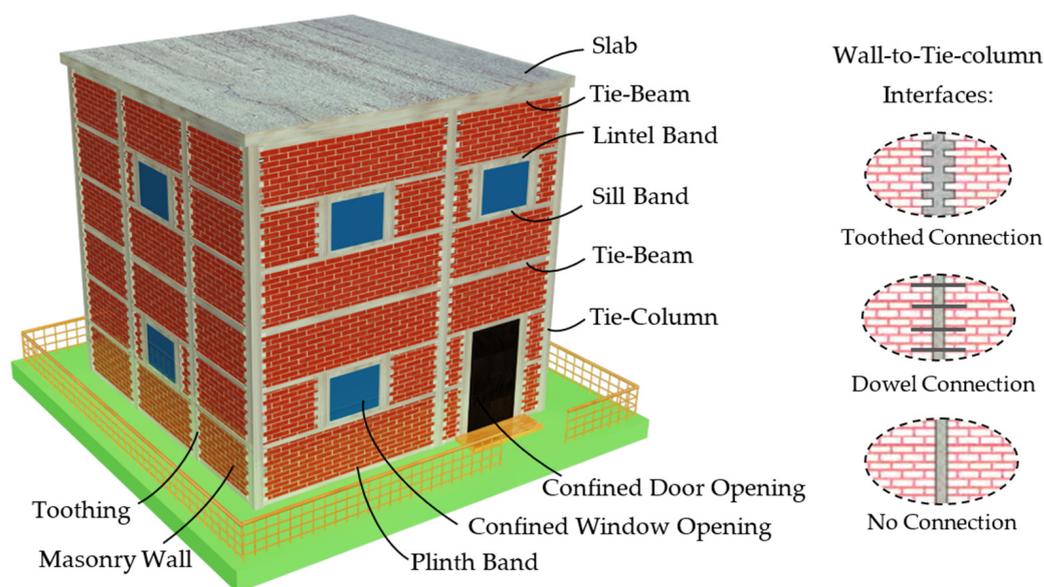


Figure 1. Features of confined masonry building.

Gravity and lateral loads are transmitted from the floor and roof slabs to the walls. In CM construction, the masonry wall is responsible for transmitting the gravity load from the slab down to the foundation and for resisting seismic forces. The RC tie-columns and tie-beams provide support to the masonry walls, improving their lateral stability and preventing complete disintegration even in severe earthquakes. To enhance the bond between the masonry walls and the RC tie-elements, toothed edges, i.e., masonry units that are staggered at the tie-column locations, are often used. Good bonding can also be achieved by providing dowels anchored into the RC tie-columns. Continuous horizontal bands (lintel/sill bands) are provided in CM buildings to secure the wall against the possibility of being knocked out during an earthquake. These horizontal bands with vertical confining members extend only to the height of the opening and also serve as a confining scheme for the wall openings. The plinth band transmits the load from the walls to the foundation and protects the ground-floor walls from excessive settlement in soft soil conditions. Lastly, the foundation transmits the loads from the structure to the ground.

Figure 2 shows a simple comparison between the three building construction types—traditional masonry, infilled RC, and CM—to provide a basic understanding of their behavior under loading. In traditional masonry, URM walls are designed to withstand vertical as well as lateral seismic loads, and these walls primarily fail under shear from horizontal bed-joint cracking. In contrast, the other two building construction types additionally use RC elements other than masonry elements. While infilled RC frames and CM structures may appear similar, their behavior under gravity and lateral loading differs significantly. In masonry infilled RC frame buildings, the RC members are first cast and designed to withstand all anticipated loads on the structure; subsequently, an infill panel is constructed. A small gap usually exists between the beam soffit and the masonry panel due to the fact that the gravity load transferred to the infill panel is almost negligible. Under lateral loading, the RC frame attempts to deform in a flexural mode, whereas infill primary deforms under shear stress, causing separation between the RC members and the infill walls along the interface. However, in CM structures, concrete is cast in place after the masonry wall, resulting in an integral composite action of the RC and masonry elements. The masonry wall is the main load-bearing element in CM; the primary purpose of the small-sized RC tie-elements is to improve the lateral stability of the masonry wall, enhancing their deformation capacity and connectivity with other walls and floor diaphragms. CM offers economic advantages, as it utilizes the full masonry strength in the main load-bearing element instead of the RC confining frame [10,11]. In summary, CM is a safe, affordable, and effective method for constructing masonry buildings that can withstand seismic activity, and it has been successfully implemented in various countries worldwide.

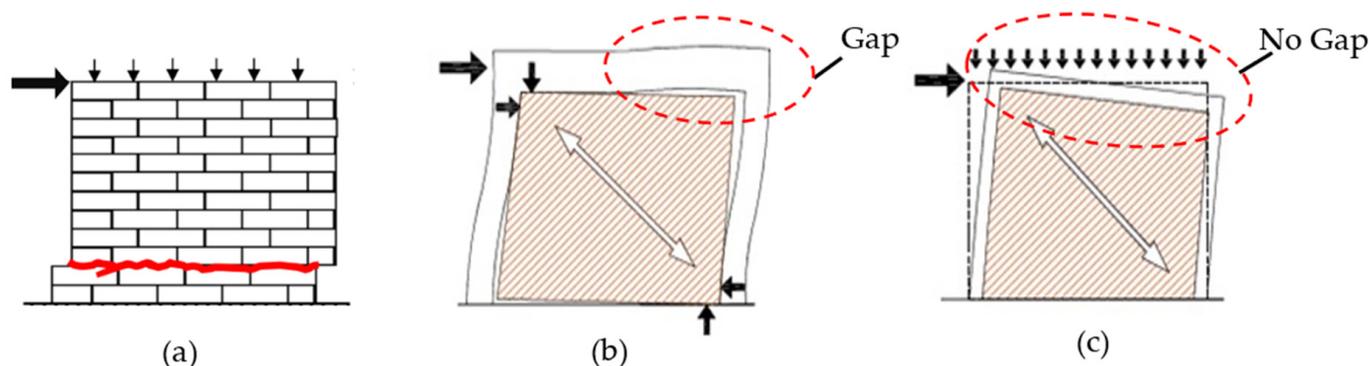


Figure 2. Comparison of (a) traditional masonry, (b) infilled RC frame, and (c) CM wall.

2. Research Significance

While the fundamental construction approach for homes built under the CM system remains consistent, there are significant differences in various regions of the world regarding the characteristics of the construction materials used, tie-element detailing, construction techniques, and other factors. A large number of studies have been carried out in the past to understand the behavior of these types of structures and to establish some analysis and design principles for the basic applications of this building type. In this article, different studies have been reviewed to provide an overall understanding of this system under gravity and seismic loadings.

The initial focus of the article is on the performance of CM buildings during previous earthquakes, as documented by multiple research teams. The aim is to identify potential failure modes and gain a better understanding of the overall behavior of CM buildings when subjected to loading. Following that, a review of various past experimental investigations conducted on CM structures is presented to determine the impact of significant parameters on their lateral load response. The capabilities and limitations of the numerical and analytical models for simulating the performance of CM structures are then reviewed. Finally, the design codes of different countries are reviewed and compared in terms of the recommendations on the design forces (axial, shear and flexural forces) of CM wall.

Thus, the present article aims to enhance our knowledge of the behavior, analysis, and design of CM buildings, benefiting code developers, design engineers, and researchers in the development of systematic analyses and safe and affordable design rules for their practical application in India as well as other countries.

3. Performance

3.1. General Performance

The ability of CM structures to resist structural loading is achieved through the combined action of the masonry walls and the adjacent RC confining elements, including tie-columns and tie-beams as well as a combination of plinth bands, sill bands, lintel bands, and roof slabs [7,8,12–16]. In CM, failure due to only vertical loading is not considered a critical issue since masonry walls are always designed such that they are subjected to relatively less axial compressive stress. Despite being designed to withstand vertical loads, masonry structures may still experience critical failure modes when subjected to gravity and lateral seismic loading, as the low tensile strength of masonry can become a limiting factor. As reported in past literature, CM buildings have performed excellently during earthquakes [8,17–31]. Observations made during past earthquakes have revealed damage to CM buildings that had some design and construction flaws. These flaws included the use of poor-quality materials, insufficient tie-columns at wall intersections and around openings, a lack of detailing in the RC tie-columns, insufficient anchorage of reinforcement in the tie-elements, poor diaphragm connections, and torsional issues due to irregularities, among others. Past earthquake damage reports and research studies have identified several potential failure modes of CM buildings, which include in-plane failure, overturning or out-of-plane (OOP) failure, diaphragm failure, connection failure, and non-structural failure [8,18,32].

The walls of a building are generally classified as either in-plane or out-of-plane walls (Figure 3), and it is necessary for the walls to have sufficient strength to withstand both in-plane and out-of-plane loading. In-plane seismic effects have been found to be critical in ground-story walls [8,33]. On the other hand, OOP effects are prominent in the upper stories of CM buildings [34].

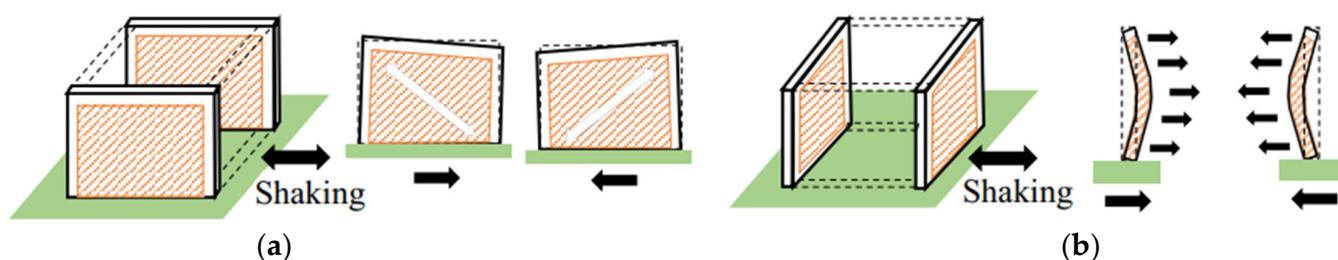


Figure 3. Description of CM behavior in (a) in-plane direction, and (b) out-of-plane direction.

The in-plane failure mode is most critical for CM walls as it occurs along the primary lateral load transfer path (Figure 3a). When subjected to lateral loading in an OOP direction (as shown in Figure 3b), masonry walls experience bending and shear stresses that can cause cracks due to the limited tensile strength of the masonry, increasing the risk of collapse due to overturning. OOP failure in a wall can be vertical or horizontal depending on the relative distances between the vertical and horizontal lateral supports. Factors such as the wall's geometry, diaphragm flexibility, and connection with adjacent confining elements influence the OOP displacement response of a wall [32]. In earthquakes, the OOP failure of URM walls is one of the most commonly observed failure modes of URM buildings. In addition, loose-fitting masonry beneath the concrete beam in infilled RC frames was found to be quite common, resulting in the OOP collapse of these panels. However, CM walls have superior integration between the masonry and the adjacent RC tie-elements due to their unique construction sequences. The effective bond between the confining frame and the

masonry wall in CM buildings creates a thrust on the beams and columns, forming an arching mechanism. This leads to better resistance against OOP failure for CM structures compared to URM and infilled RC frame structures [8,35–37].

The failure modes of CM can vary based on a number of factors, including geometric parameters such as aspect ratio and slenderness ratio as well as the type and magnitude of loading as observed in several previous studies (Table 1). As shown in the table, shear and flexural failure are the two primary failure modes of CM walls when subjected to in-plane lateral loading [7,8,38]. Cracks develop in the masonry panel of a CM wall when the tensile strength of the masonry is insufficient to withstand the stress demand. The pattern of these cracks depends on the relative strength of the mortar joints, the brick mortar interface, and the brick units; they can either follow the mortar joints (stepped) or pass through the bricks. The application of in-plane lateral loading can lead to a compressive failure mode characterized by the crushing of the masonry, particularly when there is a large vertical load on the wall or when low-compressive-strength masonry is used in construction (Figure 4a). In contrast, seismic loads can cause the bed-joint sliding shear failure of the wall (Figure 4b) when there is a low vertical load, weak horizontal mortar joint, and low concrete shear strength. In most cases, the masonry wall develops a compressive diagonal strut perpendicular to the tensile stresses when subjected to lateral loading (Figure 4c). In a few cases (especially for slender CM walls), a portion of the wall may experience tensile stresses, which results in horizontal flexural cracks in the lower courses of the wall (Figure 4d).

Table 1. Basic failure modes of CM walls in past experimental studies [12,13,16,35,36,39–56].

Load Direction	Failure Mode	References
In-plane	Compressive failure	Though crushing of the masonry is observed in almost all studies, compressive failure is not regarded as a major failure mode.
	Sliding shear failure	Sliding shear failure has been observed in limited studies and mostly in CM walls that have a very low aspect ratio and low gravity loads, as reported by Yoshimura et al. [39,40], Kuroki et al. [41], Wijaya et al. [42], Gavilán et al. [43], etc.
	Diagonal tension failure	Formation of diagonal shear cracks resulting in the diagonal tension failure of the CM is the most common type of failure observed in past studies by Kato et al. [44], Aguilar et al. [45], Iiba et al. [46], Yoshimura et al. [39], Tomažević and Klemenc [16], Yoshimura et al. [40], Kumazawa and Ohkubo [47], Yoshimura et al. [48], Yáñez et al. [49], Marinilli and Castilla [50], Zabala et al. [51], Gouveia and Lourenço [52], Bourzam et al. [12,13], Kuroki et al. [41], Wijaya et al. [42], Matošević et al. [53], Singhal et al. [35,36], Gavilán et al. [43], Gavilán [54], etc.
	Flexural cracks	Flexural failure is generally not observed in CM walls, but flexural cracks have been reported by Kato et al. [44], Iiba et al. [46], Yoshimura et al. [40,48], Zabala et al. [51], Gouveia and Lourenço [52], Matošević et al. [53], Varela-Rivera et al. [55], etc.
Out-of-plane	Vertical/horizontal cracks	Out-of-plane failure is not generally observed in CM walls due to the confinement effects of RC tie-members. Some studies by Varela-Rivera et al. [56], Singhal et al. [35,36], etc., have reported vertical/horizontal cracks due to the out-of-plane response of CM walls.

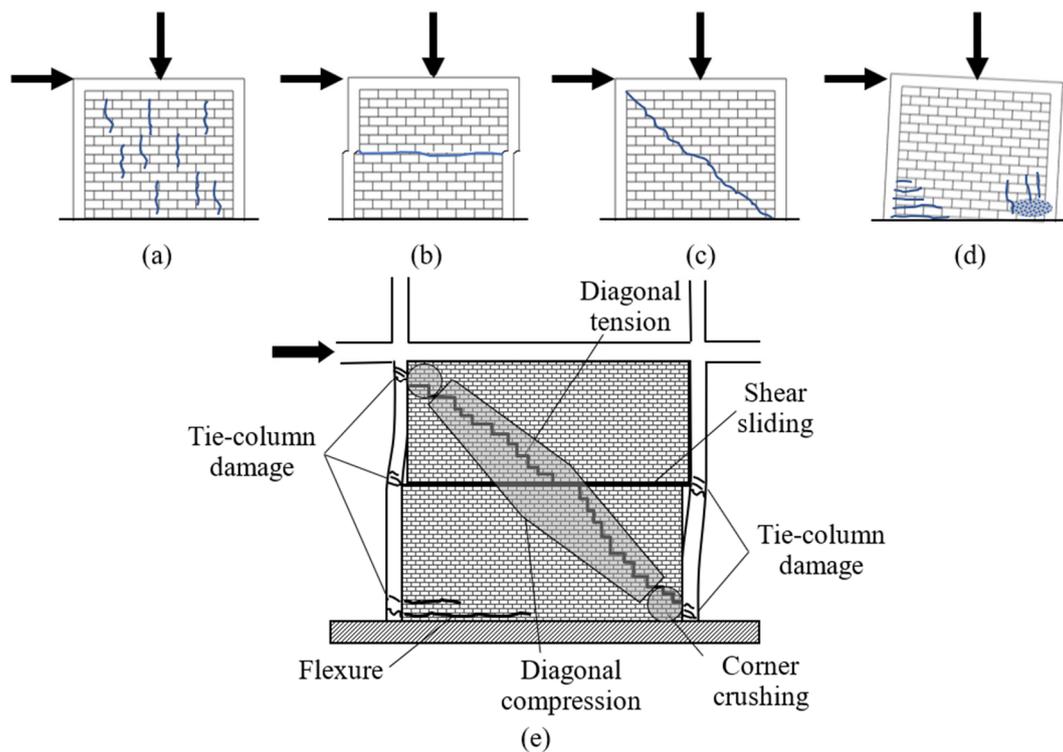


Figure 4. Possible in-plane failure modes in CM walls: (a) compressive failure of masonry wall, (b) bed joint shear sliding in masonry wall, (c) diagonal tension in masonry wall, (d) flexural failure in masonry wall, and (e) combination of different in-plane failure modes in CM.

Although individual failure mechanisms can theoretically happen, observed damages in CM structures usually involve a combination of different modes; e.g., Figure 4e. It is common to observe in CM structures shear-induced in-plane damages, which can take different forms, such as bed joint sliding, diagonal compression or strut action, and diagonal tension in which shear cracks start from the masonry wall and propagate towards the tie-columns, as reported by [1,7,38,43,57,58]. Typically, well-developed diagonal cracks are seen in CM buildings with light frames, i.e., those with small cross-sections of tie-columns and a lower percentage of steel, which is often the case. However, for strong frames where the relative stiffness of the RC frame is significant compared to the masonry wall, or where the tie-elements have larger sections, the behavior of the CM wall may be similar to masonry-infilled RC frames [7]. In such cases, as observed by [58], masonry walls fail through different mechanisms such as bed joint sliding, diagonal cracking, and toe-crushing.

Flexure-induced in-plane damages, as shown in Figure 4e, are relatively rare and include horizontal tensile cracks in the lower courses of the masonry and tie-columns at the tension end of the wall, and they also include the crushing of bricks and concrete in the compression zone and reinforcement yielding in tie-columns [51,55]. In-plane flexural damages are more likely to occur in walls with a higher aspect ratio (i.e., slender walls) or a higher moment-to-shear ratio. CM walls with a high aspect ratio (i.e., narrow width in comparison to height) are more prone to in-plane flexural failure under seismic loading. This is especially common in piers between openings or in walls with low overburden load, as well as in walls with insufficient flexural strength due to a lower percentage of reinforcement in the tie-columns. One can expect a mixed shear-flexural failure to occur more frequently in CM structures, although pure flexural failure is not commonly observed [38].

In CM buildings, tie-columns play an important role in providing additional lateral support to the masonry walls. In the event of severe damage to the masonry walls, the tie-columns take over the load-carrying function and prevent the building from collapsing [7].

This is why it is important to ensure that the tie-columns are properly designed and constructed to resist the expected loads and forces. According to the literature, under lateral loading, RC tie-columns and tie-beams of CM walls can act during both tension and compression, depending on the direction and magnitude of the earthquake and gravity loads [7,8,59]. Additionally, when subjected to a combined axial load and bending moment, the wall cross-section develops tensile and compression stresses. However, masonry and concrete have low tensile strength, so the longitudinal reinforcement of tie-columns transfers the tensile stresses, while a combination of concrete, masonry, and longitudinal reinforcement resists the compression stresses [7,38]. In the case of tie-columns, past observations by [55,60] have shown that shear cracks can occur at the ends, flexural cracks can occur along the height, concrete crushing can occur at the ends due to compression, and rebars can yield at the end due to tension.

3.2. Experimental Performance

In recent decades, there has been an increased interest in the further study of CM buildings because of their good performance during previous earthquakes. To characterize their seismic behavior, various experimental efforts have been undertaken. The primary objective of quasi-static cyclic tests and shake table tests carried out on CM walls has been to characterize the performance of a CM system under gravity as well as under lateral loadings. The controlled tests provide valuable insight into the performance of structures subjected to varying levels of lateral drift. In contrast, shake table tests provide details related to the dynamic behavior of structures required for an earthquake-resistant design. While past experimental studies have investigated individual CM walls under the action of different loads, including gravity, in-plane, and out-of-plane, the majority of the previous studies have concentrated on testing CM walls subjected to lateral in-plane loading.

3.2.1. Influence of Some Important Parameters on Behavior of CM Walls

Meli et al. [7] provides a summary of the experimental studies conducted over the past 30 years on the in-plane behavior of CM walls. The study states that during in-plane lateral loads, a masonry wall initially resists the forces. As the masonry panel starts to crack and lose strength, the vertical reinforcement in the tie-columns engage to resist the tensile and compressive stresses. However, increasing lateral deformations cause further damage to the masonry wall and tie-columns, ultimately leading to failure in many cases, wherein the tie-columns fail under shear stress due to the extension of the diagonal shear failure of the masonry wall. The OOP behavior of CM walls has also been studied experimentally by various research teams. The CM walls were either subjected to monotonically increasing uniform static pressures using airbags or subjected to out-of-plane dynamic loads. For instance, Refs. [35–37] conducted shaking table tests to investigate the OOP behavior of CM walls. Several experimental studies have been conducted in the past to understand the behavior of CM structures under various parameters. These parameters include material properties, overburden pressure, geometric characteristics, number and spacing of tie-columns, reinforcement detailing of the tie-columns, openings, and number of stories, among others. The range and frequency of occurrence of some important parameters in previous experimental studies were shown in Borah et al. [61]. Understanding the parameters that affect the behavior of CM structures can aid in developing effective seismic retrofitting techniques to enhance their performance during earthquakes.

(i) Type of masonry

Iiba et al. [46] conducted shake table tests on CM walls and found that walls constructed with Japanese bricks (with a compressive strength of 36 MPa) had 1.5 times higher lateral strength compared to those constructed with Mexican bricks (with a compressive strength of 5 MPa) due to the bricks' higher compressive strength. Meanwhile, Meli et al. [7] observed that walls made of low-strength hollow concrete blocks had more brittle failures than those made of solid concrete or clay units. Decanini et al. [62] found that CM walls made of solid clay bricks exhibited 50% more strength against ultimate cracking than

against initial cracking. On the other hand, walls made of hollow clay bricks showed only a 20% increase in strength against ultimate cracking. Furthermore, Yáñez et al. [49] discovered that walls constructed with hollow clay brick units had significantly higher lateral strength and energy dissipation capacity than those made with concrete masonry units, but the former specimens degraded in strength and stiffness more quickly. In summary, the strength, deformation, and energy dissipation behavior of CM walls, as well as the degradation of their strength and stiffness, varied greatly depending on the type of brick units used in construction. Walls built with high-strength units demonstrated better lateral strength and performance under lateral loads, while low-strength hollow units were prone to brittle failure.

(ii) Overburden load

The capacity of a CM wall to resist sliding is mainly dependent on the friction and adhesion between the bricks and the mortar. Studies by Yoshimura et al. [40] and Varela-Rivera et al. [55] showed that an increase in normal stress applied to the wall enhances the frictional resistance, resulting in improved lateral capacity and energy dissipation characteristics. The test results by Yoshimura et al. [40] on half-scaled CM walls made of hollow concrete blocks and with an aspect ratio of about 0.84 revealed that the ultimate lateral shear strength increased in proportion to the applied vertical axial load. Again, Varela-Rivera et al. [55] conducted full-scale tests on CM walls with an AR of more than 1 that were made of hollow clay bricks with reduced longitudinal reinforcement in the tie-columns to induce flexural failure. Results from the three CM wall specimens of [55]—M4, M5, and M6—for three different overburden loads ranging from 0.24 MPa to 0.71 MPa (detail specifications can be obtained from the original paper) were compared in Figure 5a. The findings indicated that an increase in the axial compressive stress on the wall led to an increase in flexural strength, while the drift ratios and displacement ductility decreased.

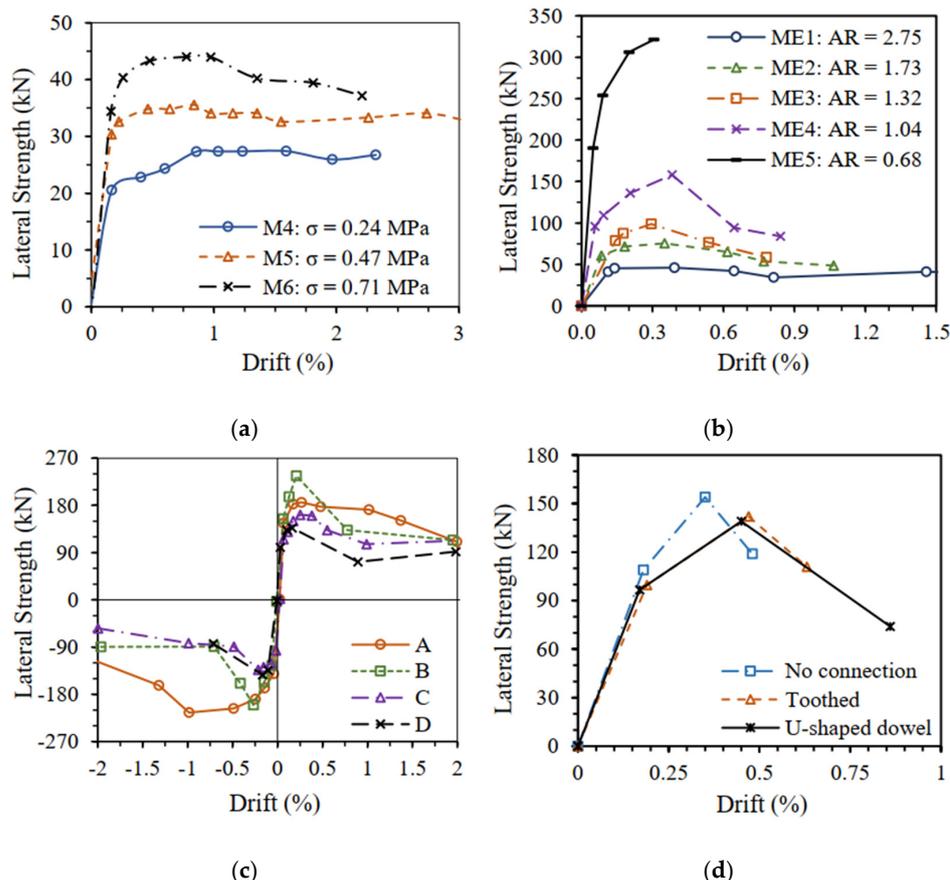


Figure 5. Cont.

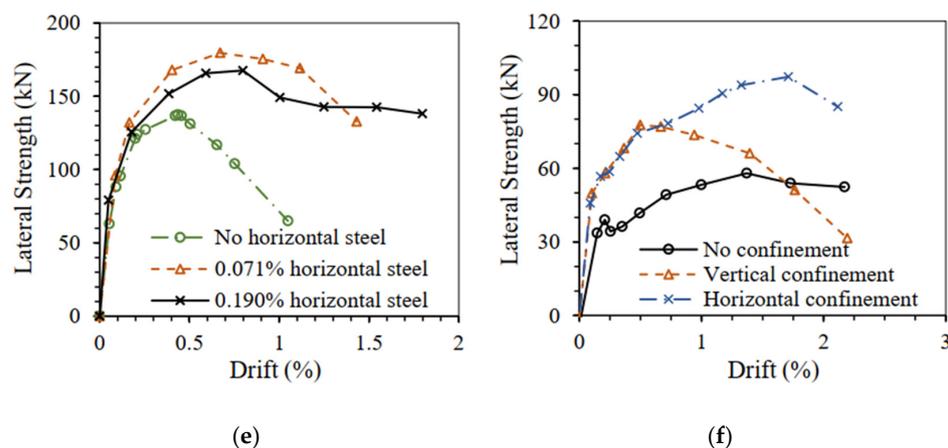


Figure 5. Differences in CM wall performance due to variation in: (a) overburden load [55], (b) aspect ratio [43], (c) tie-column reinforcement [44], (d) Connection between wall and tie-column [53], (e) reinforcement in wall [45], and (f) opening confinement [36].

(iii) Aspect ratio

Aspect ratio is a significant geometric parameter of CM walls, defined as the ratio of wall height (excluding tie-beam) to wall length (excluding end tie-columns). It predominantly determines the failure mode of a CM wall. Previous experimental investigations have typically focused on single-bay CM wall specimens with an aspect ratio range of approximately 0.6 to 2.75, with the majority of studies examining squat walls or a range of 0.6 to 1.5 [61]. Gavilán et al. [43] tested seven solid CM walls with different aspect ratios ranging from slender to squat and observed that all walls failed under shear stress. As shown in Figure 5b, when the aspect ratio decreased (slender wall ME1 with AR = 2.75 to squat wall ME5 with AR = 0.68 were considered here), the lateral strength increased, and the drift corresponding to the ultimate load decreased. The panels of squat walls with intermediate tie-columns behaved as a single structure rather than behaving like panels of isolated walls with the same aspect ratio.

Varela-Rivera et al. [55] also studied slender CM walls, as mentioned earlier. They observed that wall behavior was characterized by the yielding of the longitudinal reinforcement, followed by vertical and diagonal cracks in the masonry panel. The failure of the walls was considered a pure flexural failure, and included the crushing of the concrete in the tie-columns. The test results demonstrated that the flexural strength of the CM wall increased, while the drift ratio decreased as the aspect ratio decreased. Again, the quasi-static cyclic lateral load testing of CM walls with three different aspect ratios (from slender to squat) was conducted by Borah et al. [57], and it was found that the relatively squat wall experienced a significant degradation in post-peak strength due to early damage in its tie-columns. The study suggests that the current design methodologies for CM walls, particularly for tie-columns, need to be improved to account for the impact of aspect ratio.

(iv) Number of tie-columns and their spacing

As the lateral load increases following the initial diagonal crack in the masonry of a CM wall, cracks propagate towards the tie-columns, which leads to shear concentration at the ends of the tie-column. Hence, the number of tie-columns and their spacing are important parameters for the analysis and design of CM walls. Marinilli and Castilla [50] conducted experiments to evaluate the effect of the number and spacing of confining columns on the seismic behavior of four full-scaled CM walls made of hollow concrete blocks. The findings suggest that the presence of more confining columns with smaller spacing appears to distribute the cracking along the masonry panels more evenly, thereby improving the distribution of damage. Moreover, the addition of confining columns was found to increase the strength of the walls.

(v) Tie-column reinforcement

The role of confining elements in masonry walls is to prevent premature failure, but minimum reinforcement is necessary to avoid sudden failure. However, there is no consensus in the literature regarding the influence of longitudinal and transverse reinforcement in tie-columns on the lateral strength of CM walls, despite consistent improvement in lateral deformability and ductility with increasing reinforcement content. Shake table testing of CM walls with varying axial reinforcements in the tie-columns showed that the four-fold increase in longitudinal reinforcement percentage (ρ_l) and two-fold increase in transverse reinforcement percentage (ρ_t) did not result in significant increases in lateral strength, but they did lead to a two-fold increase in drift at peak lateral strength (δ_m) and a slight increase in drift at ultimate load (δ_u) [46]. However, Kato et al. [44] studied half-scaled CM walls with four different sets of reinforcement detailing in the tie-columns—A: had a high longitudinal reinforcement ratio (3.8%) and a high shear reinforcement ratio (1.28%), B: consisted of a high axial (3.8%) and poor shear (0.3%) reinforcement ratio, C: had a poor axial (0.99%) and high shear (1.28%) reinforcement ratio, and D: had poor axial (0.99%) and poor shear (0.3%) reinforcement ratio combinations. It was observed that a 3.8-fold increase in ρ_l resulted in a 1.5-fold increase in lateral strength and a 1.4-fold increase in δ_m , as shown in Figure 5c.

In a study by Zabala et al. [51] on the quasi-static cyclic lateral load testing of CM walls with varying overburden loads and tie-column reinforcements, a 1.5-fold increase in ρ_l , along with a two-fold increase in overburden pressure (σ), led to around a two-fold increase in lateral load capacity, with δ_m increasing by around three-fold and a slight increase observed in δ_u . Additionally, Quiroz et al. [63,64] demonstrated from the quasi-static cyclic lateral load testing of four full-scaled CM walls with different tie-element reinforcements that a 1.3-fold increase in ρ_l results in a 1.4-fold increase in lateral capacity, and δ_m and δ_u increases by around four times. These studies indicate that the influence of longitudinal and transverse reinforcement in tie-columns on the lateral strength of CM walls is complex and may vary depending on several factors, including the type of testing and the specific details of the wall construction. However, increasing reinforcement content consistently improves the lateral deformability and ductility of CM walls, which can enhance the wall's ability to withstand seismic forces. Therefore, careful consideration must be given to reinforcement the content of tie-columns when designing CM walls for seismic regions.

(vi) Connection between wall and tie-column

To ensure good seismic performance and to delay the occurrence of undesirable cracking and separation between masonry walls and RC tie-elements, sufficient bonding between them (as shown in Figure 1) is necessary. A study by Wijaya et al. [42] found that continuous anchorage provided the highest lateral load capacity for CM walls compared to short anchorage or zigzag toothed connections. However, the wall with zigzag toothed connections had the lowest capacity, while the wall with short anchorage had intermediate capacity. It was also observed that the lateral drift capacity at different limit states was significantly higher in the wall with zigzag toothed connections, followed by those with continuous anchorage and short anchorage. Another study by Singhal and Rai [35] tested three half-scale two-bay CM wall specimens with different densities of toothed connections (no tothing specimen and toothed specimens with the height of the toothed edges (a) equal to the thickness of two brick course-coarse, (b) equal to the thickness of one brick course-fine, and the length of tothing being one-half of the brick unit length) under successive applications of in-plane (quasi-static cyclic) and out-of-plane (dynamic) loading. They concluded that the increased density of tothing did not have a significant effect on OOP behavior at the initial stages, but it did cause a significant improvement in post-peak behavior under in-plane loads. The specimen with a high density of tothing showed larger ductility and reduced strength degradation compared to the other schemes. Additionally, the tothing density had a significant effect on the ability to control the OOP displacement

at a higher in-plane drift level. Moreover, Matošević et al. [53] found that connection details such as toothed connection and U-shaped dowel connection improved nonlinear wall behavior and hysteretic energy dissipation (especially after 0.2% drift level, up to which the structures remained practically elastic). Walls with toothed or U-shaped dowel connectors exhibited more ductile behavior compared to walls with no connection (Figure 5d). Overall, these studies demonstrate the importance of proper bonding between masonry walls and RC tie-elements for good seismic performance; they also demonstrate that connection details such as toothed or U-shaped dowel connections can improve nonlinear wall behavior and can increase ductility.

(vii) Reinforcement in wall

The effect of wall reinforcement on the lateral load response of CM walls has been investigated in various experimental studies in the past. While the placement of a horizontal wall reinforcement within the mortar joints is not a commonly used technique in many regions, it has been found to improve the seismic response of CM walls. The inclusion of reinforcement delays the process of crack initiation and propagation by resisting shear-induced tensile stresses. This results in an improved lateral load and deformation capacity, as well as a more uniform distribution of inclined cracking in walls under lateral loading. The experimental studies carried out by [39,40,45,47,48,51,52,65,66] have investigated the influence of wall reinforcement on the seismic response of CM walls. Figure 5e illustrates the improvement in deformation capacity with the provision of horizontal wall reinforcement.

(viii) Opening confinement

When a wall has an unconfined opening, this tends to have a negative effect on its capacity under seismic loads. During lateral loading, the corners of the openings experience stress concentration, which leads to shear cracks that destabilize the panel and cause failure. However, this problem can be addressed by incorporating confining elements around the openings in a CM wall. The location and arrangement of these elements can vary based on different studies, such as in [36,41,45,49,67,68]. Based on various experimental studies, it has been determined that the use of continuous sill and lintel beams can improve the behavior of CM walls with openings (as shown in Figure 5f).

In the study by Singhal and Rai [68], the lateral load response of a double-panel perforated (window opening) infill RC frame with only lintel beams, which was named SI-O_{2WA} (no confinement), was compared with two similar CM specimens with two different confining schemes around the window openings. The first scheme included continuous tie-columns along the entire wall height, with discontinuous tie-beams at the top and bottom of the window opening, and was named SC-O_{2WB} (vertical confinement), while the second scheme consisted of continuous sill and lintel beams along the entire wall length, with discontinuous tie-columns on both sides of the opening, and was named SC-O_{2WC} (horizontal confinement). SC-O_{2WB} and SC-O_{2WC} showed about a 44% and 80% higher in-plane capacity, respectively, and a higher energy dissipation capacity (>40%) than the perforated infill wall SI-O_{2WA}. Additionally, the in-plane as well as the out-of-plane performance of SC-O_{2WC} with continuous sill and lintel bands was better due to the beams dividing the wall into smaller panels with a lower aspect ratio, resulting in well-distributed diagonal shear cracks throughout the wall. Conversely, the piers in SC-O_{2WB} were not confined from the top and bottom, and they experienced more damage due to their large slenderness. In summary, confining elements around openings in a CM wall are essential for ensuring satisfactory earthquake performance and delaying undesirable cracking and separation at the wall-to-tie-column interface.

(ix) Number of stories

There is a lack of literature on the impact of the number of stories on the behavior of CM buildings. A limited number of studies have been conducted to investigate this. San Bartolomé et al. [69] carried out a shake table test on a 1:2.5 scaled three-story CM structure consisting of two parallel perimeter walls connected by RC slabs. Similarly,

Scaletti et al. [70] conducted pseudo-dynamic tests on one full-scale and one half-scale CM building consisting of two parallel walls connected by stiff horizontal slabs. Alcocer and Casas [71], Flores et al. [72], Alcocer et al. [73], and Tomažević and Klemence [33] also studied scaled specimens of one- to three-story CM buildings with different configurations. The results of these tests indicate that damage or collapse usually occurs in the lowest story, which has also been a common observation during previous seismic events.

3.2.2. Comparison of CM with Other Similar Building Typologies

Researchers have studied the in-plane performance of CM walls in comparison to URM or infilled RC frame walls. Yoshimura and Kikuchi [74] conducted tests on nine specimens to compare the behavior of CM walls with URM walls and RC ductile moment-resisting frames that had the same cross-sectional details as the CM specimen. The CM wall specimen exhibited higher strength and ductility than the URM and infilled RC frame specimens, leading to the conclusion that CM construction is an excellent structural system. The results of the study suggest that CM walls have the potential to be a viable alternative to URM or infilled RC frame walls in terms of in-plane performance. Tomažević and Klemenc [16] conducted a study on three specimens of both plain and confined masonry walls that had an aspect ratio of 1.5 and were made at a 1:5 scale. The specimens were subjected to a constant vertical load that was approximately 22% of the masonry compression strength, and cyclic horizontal displacements. The study concluded that confining the wall with RC tie-columns improves the lateral resistance of a URM wall by more than 1.5 times and enhances the deformation and energy dissipation capacity by almost five and six to seven times, respectively, as shown in Figure 6a. Although both types of walls exhibited similar stiffness, the confined walls exhibited significant improvement in ductility and energy dissipation capacity.

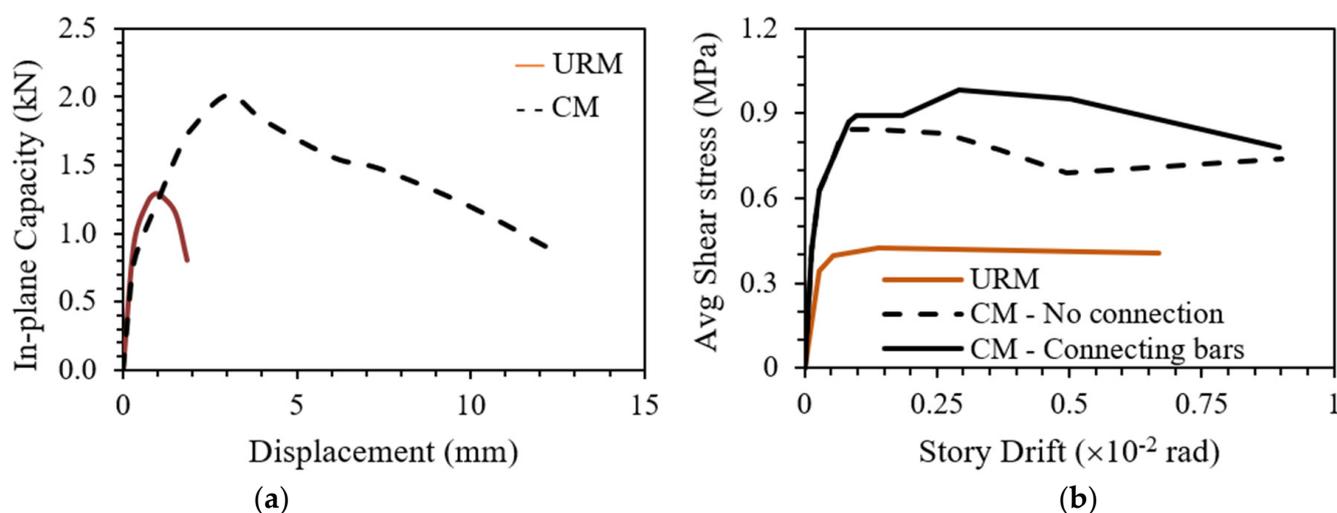


Figure 6. Comparison of CM and URM under in-plane loading: (a) Tomažević and Klemenc [16], and (b) Yoshimura et al. [75].

Yoshimura et al. [75] conducted a study to investigate effective seismic strengthening methods for masonry walls in developing countries. They constructed and tested 28 URM and CM walls that had an aspect ratio of 0.77 and that consisted of 2D and 3D walls with or without wall reinforcing bars or U-shaped connecting bars. The test results showed that the CM wall system with connecting bars at the vertical wall-to-wall connections, as well as the horizontal wall reinforcing bars, developed reasonably higher ultimate lateral strength with the increase in vertical axial load, and they showed better ductility compared to conventional URM specimens (Figure 6b). In a study conducted by Goveia and Lourenço [52], sixteen walls were tested to investigate the effects of confining the masonry walls with RC tie-elements and providing bed joint reinforcement. The walls

were constructed at a 1:2 scale and consisted of URM and CM walls made of lightweight concrete blocks with an aspect ratio of 1. The results showed that confining the masonry walls with RC tie-elements improved the lateral capacity of the standard URM walls by 1.17 times and deformation capacity by 1.43 times compared to the CM walls without bed joint reinforcement. Additionally, providing bed joint reinforcement increased the lateral capacity of the CM walls by about 1.22 times and deformation capacity by 1.43 times compared to the similar URM walls.

Some researchers have studied the behavior of CM walls under OOP loading conditions by applying dynamic loads such as shake-table-generated ground motions or uniformly distributed surface pressure using airbags. Tu et al. [37] conducted shake table tests on a full-scale single-story CM building to study the OOP behavior of CM walls with different thicknesses compared to masonry-infilled RC frames. Singhal and Rai [35,36] studied the seismic performance of CM walls and compared it with typical masonry-infilled RC frames. They subjected the specimens to successive applications of quasi-static cyclic in-plane loading and OOP dynamic loading on a shake table. Their study showed that CM walls with or without tothing exhibited better in-plane and out-of-plane responses compared to infilled RC frames (Figure 7). The test results showed that the CM walls were able to withstand significant OOP loads without major damage. The composite action between the wall and the confining frames prevented the masonry panels from falling out of the frame and helped to reduce the influence of inertia forces caused by their own weight. Conversely, the infill panels in the masonry-infilled RC frames separated from the boundary frame and collapsed due to the OOP inertia forces. The findings suggest that CM walls may be more resistant to OOP loads than masonry-infilled RC frames.

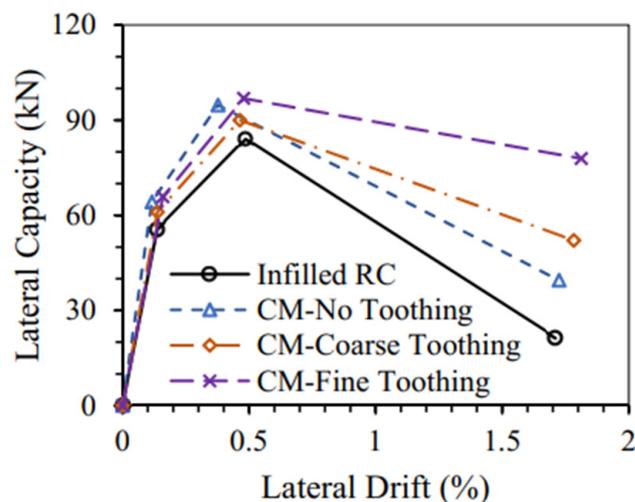


Figure 7. Influence of tothing on in-plane and OOP response of CM walls [35].

4. Analysis Methods

Despite their longstanding history, the engineering behavior of CM buildings has been established at a slow rate. This is due to the significant variation in materials and construction methodologies, which has resulted in a limited understanding of the complex composite behavior of CM walls under different loading conditions. Although limited studies have attempted numerical modeling strategies for the analysis of CM buildings, the modeling process is complex, and reliable 3D finite element (FE) models (Figure 8a) are not easy to obtain due to the computationally intensive nature of the process and the requirement for a large number of input parameters. To address these issues, different analytical modeling techniques have been developed for the analysis of CM walls over the years. Since the behavior of CM walls is similar to that of other building types, such as masonry-infilled RC frame buildings and URM buildings, various analytical modeling techniques have been utilized to simulate and analyze CM systems [76]. These modeling

strategies, as summarized in Figure 8, have varying degrees of refinement and precision for capturing failure modes and predicting the lateral load behavior of CM walls. Different commercial software programs such as ABAQUS, ANSYS, LS-DYNA, ATENA, SAP2000, and ETABS have been utilized to predict the response of structures with these models. In simplified 2D line element models, the building elements are modeled using either two-noded beam-column elements or four-noded shell elements. The use of simplified models became popular in the literature due to their practical applicability.

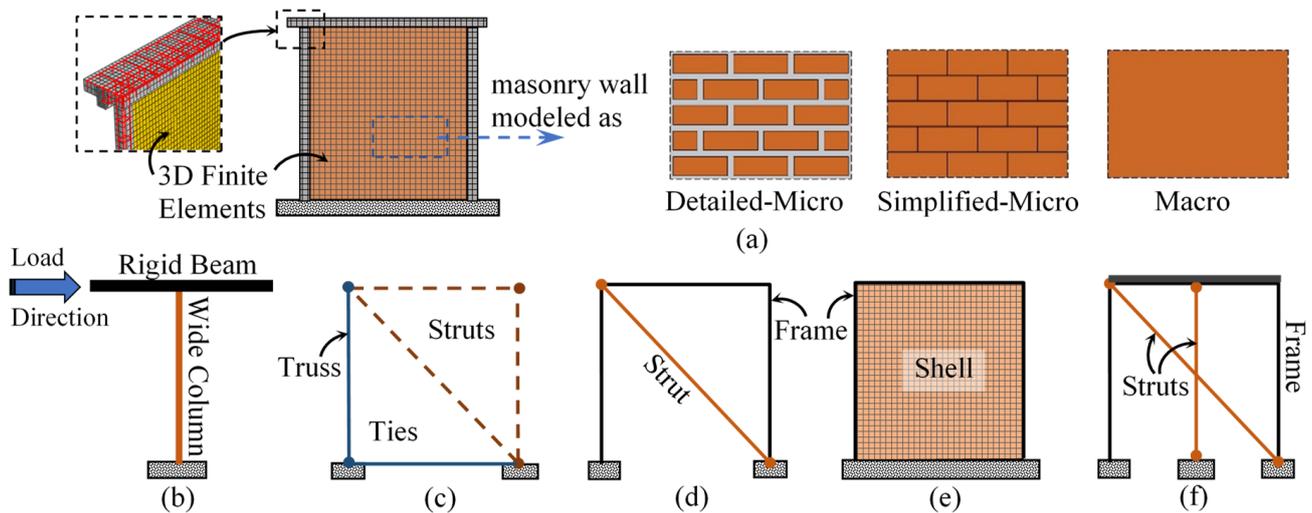


Figure 8. Different modeling strategies for confined masonry: (a) finite element methods, (b) wide column method, (c) strut and tie method, (d) equivalent strut method, (e) equivalent shell method, and (f) vertical-diagonal strut method.

Various simplified models have been developed, such as WCM (wide-column model), STM (strut-and-tie model), ESM (equivalent strut/shell model), ETM (equivalent truss model), and VDSM (vertical-diagonal strut model). The applicability of different simplified models was assessed in Borah et al. [77] by analyzing single-story CM walls and a three-story CM building under linear analysis as well as nonlinear static pushover analysis. Overall, while analytical modeling techniques have advanced over the years, reliable and practical FE models are still challenging to obtain, particularly for large structures. This section reviews the past analytical and numerical research carried out to improve our understanding of the behavior of confined masonry structures under different loading conditions.

Figure 9 presents a flowchart summarizing the various simplified analysis methods for CM structures, which depend on their nature of application and the requirement of input data for nonlinearity definitions. This information has been gathered from various research studies [8,12,13,16,36,38,43,50,60,65,78–116]. With the exception of the conventional STM, all models can be developed in any structural analysis program. The nonlinear analysis methods are divided into two groups based on the input hinge definition types required. Many researchers have developed prediction equations to estimate the lateral load and lateral drift of CM walls at three critical stages. The first group, WCM and ETM, use these equations for nonlinearity definitions in the wide-column and diagonal strut, respectively. In contrast, the second group, ESM-strut and VDSM, use the input definition of masonry prism properties for nonlinearity definitions in the diagonal struts.

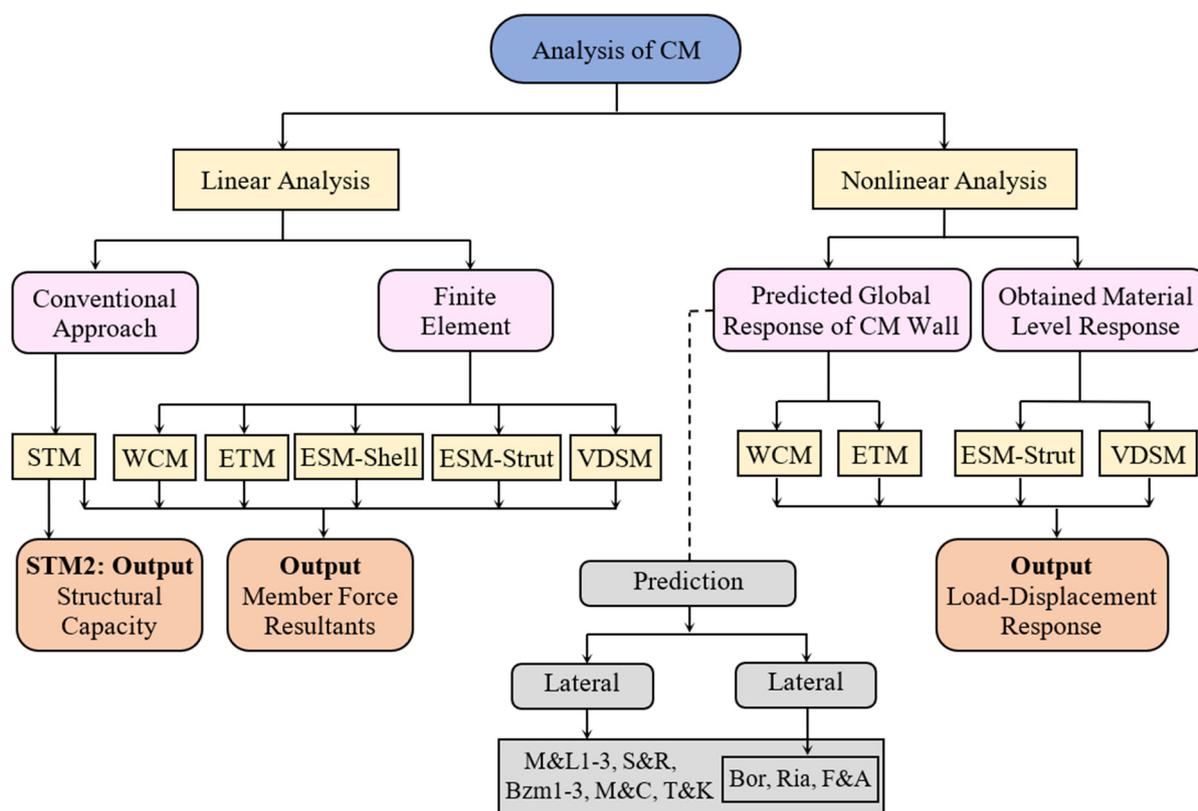


Figure 9. Flowchart showing different simplified analysis methodology for CM structures. Note: Bor [113]; M&L3 and M&L2 [38]; S&R [36]; M&L1 [114]; Ria [115]; Bzm1, Bzm2, Bzm3 [12]; M&C [50]; T&K [16]; F&A [116].

4.1. Finite Element Method (FEM)

The FE method is widely recognized as a reliable technique for structural analysis, offering a comprehensive approach to studying the structure in question (Figure 8a). Masonry structures can be analyzed at the micro or macro level using FE analysis, according to several past studies [78–80]. Micro-modeling approaches are typically applied to small structural elements in order to accurately represent the heterogeneity of masonry through the properties of each constituent and interface. Simplified micro modeling has also been used, in which bricks are expanded to up to half of the mortar thickness in both the vertical and horizontal directions, while the mortar is clamped into the mortar interface. Macro-modeling approaches are commonly adopted to represent the overall structural behavior, wherein no distinction is made between the individual units and joints, and the material parameters are obtained from masonry tests under homogeneous states of stress. Researchers have widely employed both modeling approaches to predict the behavior of masonry walls. Smoljanović et al. [81] employed an extremely detailed micro-modeling approach to replicate the behavior of CM walls. They used linear elastic triangular elements to discretize the RC and masonry components. Material non-linearity, including the fracturing and fragmentation of the discrete elements, was taken into account using smeared contact elements between them. The joint interface between the masonry elements was modeled by taking into account tension cracking and sliding along the bed joints of the masonry using the Coulomb dry friction model. Amouzadeh Tabrizi and Soltani [82] also utilized a micro-modeling approach to simulate masonry walls, where masonry blocks were modeled using a continuum model and potential cracks were smeared into the developed model. The masonry joint interfaces were modeled by considering shear sliding and the opening of joints. Using this approach, the analysis precisely predicted crack propagation and reinforcement bar yielding.

Borah et al. [83], Yacila et al. [84], Tripathy and Singhal [85], and Okail et al. [86] followed a slightly different approach to simulate the behavior of CM walls using a FE macro model. They used continuum elements to simulate the response of RC elements and masonry panels. The modeling of the interfaces utilized the traction separation law to account for the contact between materials, while hard contact was adopted to simulate the interaction in the normal direction. The frictional behavior in the tangential direction was described by the application of the Mohr–Coulomb failure criteria. Medeiros et al. [65] employed a comparable technique, whereby the masonry and RC elements were characterized using the smeared crack model. Eshghi and Pourazin [87] made a modification to the FE simulation approach by introducing a 2D macro model for CM walls. In this model, plasticity-based material models were implemented along with a Coulomb friction model featuring a tension cut-off mode for the interface zone between the confining elements and the masonry panel. Janaraj and Dhanasekar [88] discussed a macro-modeling approach to calibrate unconfined and confined masonry panels tested under diagonal compression. This approach used smeared properties to define both the hollow and the grouted masonry, which effectively predicted the deformation characteristics, shear strength, and failure modes of the CM walls.

4.2. Wide Column Method (WCM)

The Wide Column Model (WCM) for CM walls was developed based on the Equivalent Frame model of URM buildings [89,90]. In this model, the CM wall is represented by a one-dimensional two-noded centerline beam-column element (wide column) as depicted in Figure 8b. The section properties of the element are transformed to account for the composite action of the masonry and the RC tie-columns. The width of the tie-columns is therefore transformed to equivalent width of the masonry such that the equivalent area of the wide column section (A_{wc}) becomes equal to

$$A_{wc} = A_w + 2mA_c \quad (1)$$

A_w is the cross-sectional area of the masonry wall, A_c is the cross-sectional area of the RC tie-column, and m is the ratio of the modulus of elasticity of the concrete (E_c) to the modulus of elasticity of the masonry (E_m). The tie-beam in the CM walls is represented by a two-noded beam-column element, and the axial rigidity provided by the masonry wall below the tie-beams is simulated by modeling the tie-beam as rigid with infinite stiffness. For walls with openings, two types of beam sections are used, namely rigid and flexible [91]. The WCM has been recognized as a viable model by NTC-M: 2004 [92] for the structural analysis of CM structures with commercial computer programs. Terán-Gilmore et al. [93] demonstrated the application of WCM for analyzing a three-story building, where nonlinearity (axial lumped hinge) was defined near the bottom of each wide column using the shear force–deformation relationship from a past backbone curve model of a CM wall. The applicability of the WCM in the analysis of CM structures has also been demonstrated by several past studies [91,94,95]. Choosing a trustworthy backbone model for defining shear hinges in the WCM for CM walls can be a difficult undertaking. Moreover, the assumption of monolithic behavior in WCM does not permit the investigation of individual behavior of tie-columns and intricate force transmission in various elements. The assumption of the monolithic behavior of tie-columns and masonry walls is primarily applicable when representing the initial elastic response.

4.3. Strut-and-Tie Method (STM)

The Strut-and-Tie Model (STM) was initially developed as a manual method for analyzing and designing shear-critical structures and disturbed regions in concrete structures. STM employs trusses to depict internal load paths within the structure, with compressive stress fields (struts) connected by tensile stress fields (ties). Design member force resultants are determined using static equilibrium as long as the STM is statically determinate [96,97]. These load paths are based on experience and intuition. Previous studies have used the

Strut-and-Tie Model (STM) to analyze wall-type structures, considering them as analogous to deep beams, where the entire wall serves as the D region in accordance with St. Venant's Principle. Past literature [8,85,92,98–100] suggests using the STM analysis for CM walls subjected to lateral loading, where the wall is modeled as a pin-jointed structural truss with both tension and compression members (Figure 8c). No specific guidelines have been provided on how to develop an STM for walls with openings. The shear capacity of the masonry wall is determined by the horizontal component of the diagonal strut force, while the required amount of reinforcement in the tie-members is found using the calculated axial forces in the ties.

Brzev and Gavilán [99] and Brzev [100] demonstrated the use of STM in a four-story two-bay CM wall, where one bay contained a wall with an opening on each story. However, the study disregarded the strut action of the walls with openings and only analyzed the structure for the given story shear forces. While Ghaisas et al. [98] proposed configurations for the orientation of strut elements influenced by the openings and panel configurations, the ability of these configurations to predict lateral capacity and load distribution in different members was not validated further. On the other hand, Tripathy and Singhal [85] and Singhal [101] used the strut-and-tie analysis of a CM wall based on principal stress resultants obtained from an FE analysis and then estimated its lateral load-carrying capacity. The axial forces in the ties ($f_{yl}A_{sl}$) are calculated by assuming yielding in the longitudinal reinforcement of the tie-columns. A_{sl} is the total area of the longitudinal reinforcement in a tie-column, and f_{yl} is the yield strength of the reinforcement. An empirical relationship was recommended by [85] to estimate the limiting axial capacity (F_{ss}) of a diagonal strut in the STM:

$$F_{ss} = F_1 \left(\sqrt{f'_m A_w} \right) \quad (2)$$

where,

$$F_1 = C_1 \left(\frac{1}{\lambda} \right) \left(\frac{H}{L} \right) + C_2 \quad (3)$$

$$\lambda = H \left(\frac{E_m t \sin 2\theta}{4E_c I_c H} \right)^{0.25} \quad (4)$$

The constants $C_1 = 2.23$ and $C_2 = 0.1$ for $H/L > 1$, and $C_1 = 2.65$ and $C_2 = 0.08$ for $H/L \leq 1$. The variable f'_m is the masonry prism compressive strength; H is the height of the CM wall, including the tie-beam depth; L is the length of the CM wall, including the tie-column width; t is the thickness of the wall; I_c is the moment of inertia of the tie-column section; θ is the angle between the horizontal axis and the centerline of the strut. The axial force in the diagonal strut is determined by employing the method of joints, which considers the yielding of the longitudinal reinforcement in the tie-columns, i.e., using $f_{yl}A_{sl}/\sin\theta$. When the force exceeds the limiting capacity (F_{ss}), the axial force in the strut is set to F_{ss} , and the lateral capacity of the wall is determined by the horizontal component of the force.

4.4. Equivalent Truss Method

Rankawat et al. [102] proposed a modified version of the STM for CM walls such that it can be implemented in commercial software, and the method was renamed the Equivalent Truss Model. Utilizing the relationship between the axial stiffness of the diagonal strut and the lateral stiffness of the wall, the diagonal strut width was suggested as

$$w_{ds} = \frac{K_i L_d^3}{t E_m L_c^2} \quad (5)$$

L_d is the diagonal strut length, L_c is the centerline length of the CM wall in the STM, and K_i is the initial stiffness of the wall, which can be estimated using past studies. The model's ability to perform nonlinear analysis was demonstrated on a three-story building, where the nonlinear behavior was limited to the diagonal strut and defined in terms of stress-strain using the available CM wall backbone model. Therefore, it is crucial to choose

a dependable backbone model to define the axial hinge in the diagonal strut. Although the method can accurately estimate the lateral strength of CM walls, it does not account for tie-column failure as it does not incorporate any nonlinearity in the tie-members.

4.5. Equivalent Strut Method or Equivalent Shell Method (ESM)

The equivalent strut method is a widely used and accepted method for modeling masonry-infilled reinforced concrete (RC) frame structures. However, the gravity load and lateral load behaviors of these structures are significantly different from those of CM wall buildings [103]. The equivalent strut model for masonry infills in RC frames typically uses a diagonal strut with released end moments to represent the lateral resistance of the infill panel. Polyakov [104] proposed this method to estimate the lateral stiffness of masonry infilled RC frames, and it has since been used as the basis for several other models developed in subsequent years. The single strut model with the width of the strut equal to one-third of the diagonal length of the panel was proposed by [105], but subsequent studies showed that this model overestimates the stiffness of the infilled frames. Thus, various empirical formulations have been proposed to define the effective width based on different experimental and analytical observations. Paulay and Priestley [106] suggested that the equivalent diagonal strut width be one-fourth of the diagonal length of the infill wall; this is widely used and generally an effective width for masonry struts. Some studies in the literature argue that a single strut cannot accurately simulate the complex interaction between the masonry panels and the RC frames. As a result, multi-strut models have been used, which can provide a limited simulation of the contact between the masonry panel and the RC frame [103].

Previous studies have tried to apply the equivalent strut model to CM walls (as shown in Figure 8d), but their effectiveness is restricted since they assume the failure modes in CM walls to be similar to those in masonry-infilled frames. Kaushik and Sanganee [107] proposed a strut width reduction factor for the equivalent strut model (ESM) to be applied to CM walls. However, their investigation only focused on calibrating the lateral stiffness of the CM walls to match past experimental studies. Torrisi and Crisafulli [108] and Torrisi et al. [109] developed a 12-node masonry panel element model for the nonlinear analysis of infilled RC frames and CM walls. The model includes six diagonal struts to simulate the behavior of both infilled RC frames and CM walls. Similarly, some researchers have employed four-noded shell elements to model masonry walls situated between the centerline beam-column elements of the RC frame, as depicted in Figure 8e. According to [110], the equivalent shell model was found to be better than the equivalent strut model for estimating initial stiffness; however, it was also suggested that the strut model could be useful for conducting parametric studies. Some other research has employed linear shell elements to model masonry walls in order to perform a linear analysis of CM walls [98,107,111,112]. However, this approach is limited, as shell models cannot accurately simulate nonlinear responses.

4.6. Vertical-Diagonal Strut Method (VDSM)

The Vertical-Diagonal (V-D) strut model is a modified version of the Equivalent Static Method (ESM), developed for the purpose of analyzing CM structures under the combined influence of gravity and seismic loading [112]. It was observed that the ESM, a reliable modeling technique for infilled RC structures, requires significant modifications before it can be effectively applied to CM structures. The VDSM approach models the tie-elements of a CM wall as frame elements, while the masonry wall is represented as a combination of pin-jointed vertical and diagonal strut elements (Figure 8f). To replicate realistic tie-beam deflection and tie-column axial forces under gravity loading, the VDSM method assumes that the width of the vertical strut is 75% of the panel length and enhances the flexural stiffness of the tie-beam by a factor of 20 to 25 times its original stiffness. Both struts are modeled with a thickness equal to the actual wall thickness. To achieve a similar initial

lateral stiffness value as observed in experimental studies, the width of the diagonal strut is set to one-third of its length.

A lumped plasticity approach can be employed at designated hinge locations to account for nonlinearity in both the diagonal struts and the tie-columns. To capture the nonlinearity present in masonry walls, axial hinges are defined in the diagonal strut, while flexure and shear hinges are utilized to simulate the nonlinear behavior of tie-columns. The application of masonry prism strength to define nonlinearity in the diagonal strut, as is done in the ESM for infilled RC frame structures, was found to overestimate lateral strength due to the distinct failure modes exhibited by CM walls. Therefore, to more accurately reflect the failure of CM walls, the effective shear strength of the masonry (f_{ss}), which represents the strength corresponding to the weakest failure mode in such walls, was utilized in place of the masonry prism strength as the axial strength of the diagonal strut. Empirical relationships were suggested for the estimation of f_{ss} using six independent parameters: masonry prism strength (f'_m), concrete compressive strength (f'_c), wall aspect ratio (AR), overburden pressure on tie-beams (σ), % steel reinforcement in tie-columns (ρ_l), and thickness of wall (t):

$$f_{ss} = 0.096t^{0.217}f'_m{}^{0.827}f'_c{}^{0.081}\rho_l{}^{0.018}(1+\sigma)^{0.235}AR^{0.101} \quad \text{for } AR \leq 1 \quad (6)$$

$$f_{ss} = \frac{0.134t^{0.133}f'_m{}^{0.886}f'_c{}^{0.107}\rho_l{}^{0.002}(1+\sigma)^{0.004}}{AR^{0.248}} \quad \text{for } AR > 1 \quad (7)$$

Borah et al. [112] verified the accuracy of the V-D strut model and the empirical equations for estimating the lateral load behavior of CM walls by using data from 35 single-bay and multi-bay specimens across 14 different studies. The model was found to adequately replicate the realistic linear and nonlinear behavior of CM walls with satisfactory accuracy.

4.7. Using Backbone Curves

Previous experimental studies have also led to the development of analytical backbone curve models for performance-based designs and for analyzing the lateral load behavior of CM walls, including their stiffness, strength, and deformability [12,13,16,36,38,43,50,60,113–115]. These backbone curve models for the lateral load analysis of CM walls have either been developed using methods intended for unreinforced and reinforced masonry walls or based on a limited number of laboratory tests. These models assume different parameter sets that can vary regionally, making it difficult to generalize due to their particular calibration for specific CM wall systems. Significant variations in critical parameters in the experimental database, such as masonry compressive strength (ranging from 1.7 MPa to 61 MPa) and shear strength (ranging from 0.24 to 2.74 MPa), concrete compressive strength (10 to 44 MPa), wall aspect ratio (0.3 to 2.75), longitudinal reinforcement in tie-columns (0.5% to 6.6% of cross-sectional area), tie-column cross-sectional area relative to the total area (0.07 to 0.32), and vertical axial load on walls (−0.1 to 1.8 MPa), introduce various challenges for creating idealized load–deformation envelope curves for CM wall analysis and design. One example of a backbone curve for CM walls is presented in Figure 10, where the equations are suggested by [113] for estimating the lateral load and lateral drift capacity of CM walls at three critical stages: initial cracking, maximum load resistance, and ultimate stage for 20% strength degradation (represented by points A, B, and C in Figure 10). The suggested equations were developed for two types of masonry using regression analysis and data obtained from past experimental studies.

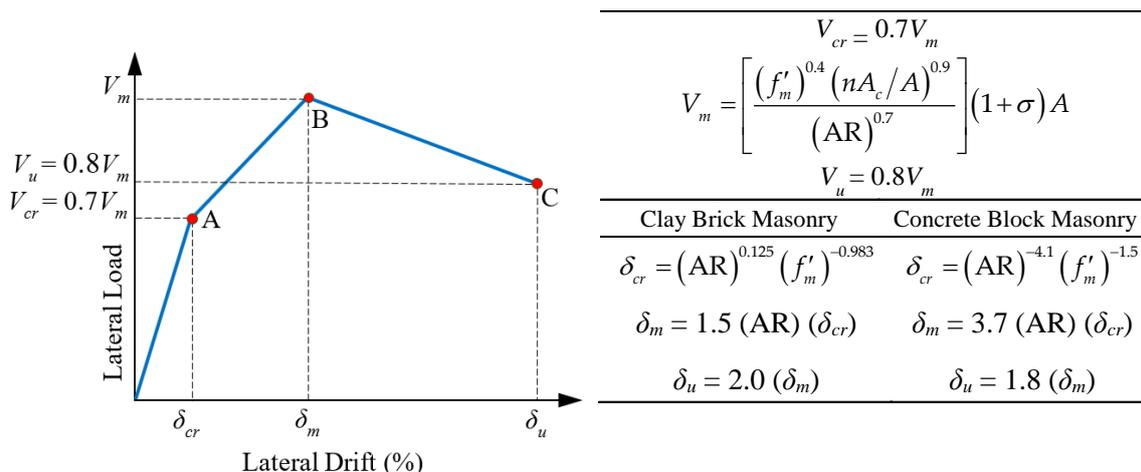


Figure 10. A generalized backbone curve model developed for CM [113].

The accuracy of the non-linear analysis results obtained from WCM and ETM depends on the force–deformation backbone curve model chosen for CM walls, which is represented by prediction equations. Past studies have focused on predicting lateral loads, with only a few offering methods for lateral drift estimation at different damage stages. Borah et al. [113] conducted a quantitative study on the suitability and limitations of existing prediction models and reported that while some prediction models can confidently predict the maximum lateral load resistance of CM walls (e.g., [113,116]), predicting lateral drift capacity at different damage stages remains challenging due to the high level of uncertainty in deformation prediction in the nonlinear range.

5. Design Methodologies

5.1. Basic Design

Standard codes and guidelines play a crucial role in ensuring public safety and welfare by ensuring that buildings are designed and constructed to perform adequately in terms of safety and serviceability. Design codes for CM buildings have been developed in some countries. Further, several guidelines have been established over the years to provide fundamental information on CM and encourage its use in construction, as shown in Table 2 [7,59,117–130]. In many other countries, CM buildings are constructed using basic guidelines without the development of any formal building codes. Common fundamental principles for the seismic design of CM buildings are shared among all available codes and guidelines. The way seismic forces are distributed within a structure is determined by the building configuration, including its size and shape, as commonly noted in various guidelines and codes. Irregular configurations are generally avoided due to their tendency to cause stress concentration and torsion. To manage torsion within a manageable range, it is necessary to have a symmetrical arrangement of mass and balanced stiffness in both directions.

Table 2. Design codes and guidelines for CM [7,59,117–132].

Design Codes	Guidelines
Mexico: NTC-M: 2017 [117]	Meli et al. [7]
Peru: NT E.070: 2019 [118]	Schacher and Hart [59]
Chile: NCh2123: 2003 [119]	Brzev et al. [126]
Argentina: INPRES-CIRSOC 103: 2018 [120]	Carlevaro and Roux-Fouillet [127]
Colombia: NSR-98: 2010 [121]	Arya et al. [128]
Costa Rica: CSCR: 2000 [122]	Totten [129]
Europe: CEN: 2005 [123]	Boen and associates [130]
China: GB 50003: 2011 [124]	Schacher [131]
India: BIS: 2022 [125]	Blondet [132]

The design of a CM building involves meeting several requirements outlined in various guidelines, which include maximum slenderness ratio, minimum wall aspect ratio, maximum wall height, minimum wall thickness, tie-element details, and foundation details, among others. Although the fundamental principles of CM building design remain the same, the design codes and guidelines differ significantly, especially with regards to the dimensions of masonry walls and tie-columns, as well as the detailing of tie elements, materials, and other safety factors [7,38]. To achieve satisfactory seismic performance, different country codes propose the use of tie-columns measuring between 100 mm and 200 mm with four longitudinal reinforcement bars and ties 6 mm in diameter. However, the bar diameter for longitudinal reinforcement varies among different country codes, with some codes such as those of Colombia and Chile recommending 10 mm, while those of other countries such as Argentina, Eurocode, and China suggest different bar diameters ranging from 5 to 12 mm.

5.2. Estimation of Design Forces for CM Wall

A structure's seismic design is heavily reliant on its characteristic force–deformation relationship, which is determined by three crucial design parameters: stiffness, strength, and deformability/ductility. In order to ensure adequate seismic performance, the strength and deformation capacities must exceed the demand imposed by a design earthquake. While most of the codes specify the design lateral loads in terms of the design shear strength of a CM wall, a few codes provide empirical relationships to estimate the design flexural strength too (as shown in Figure 11).

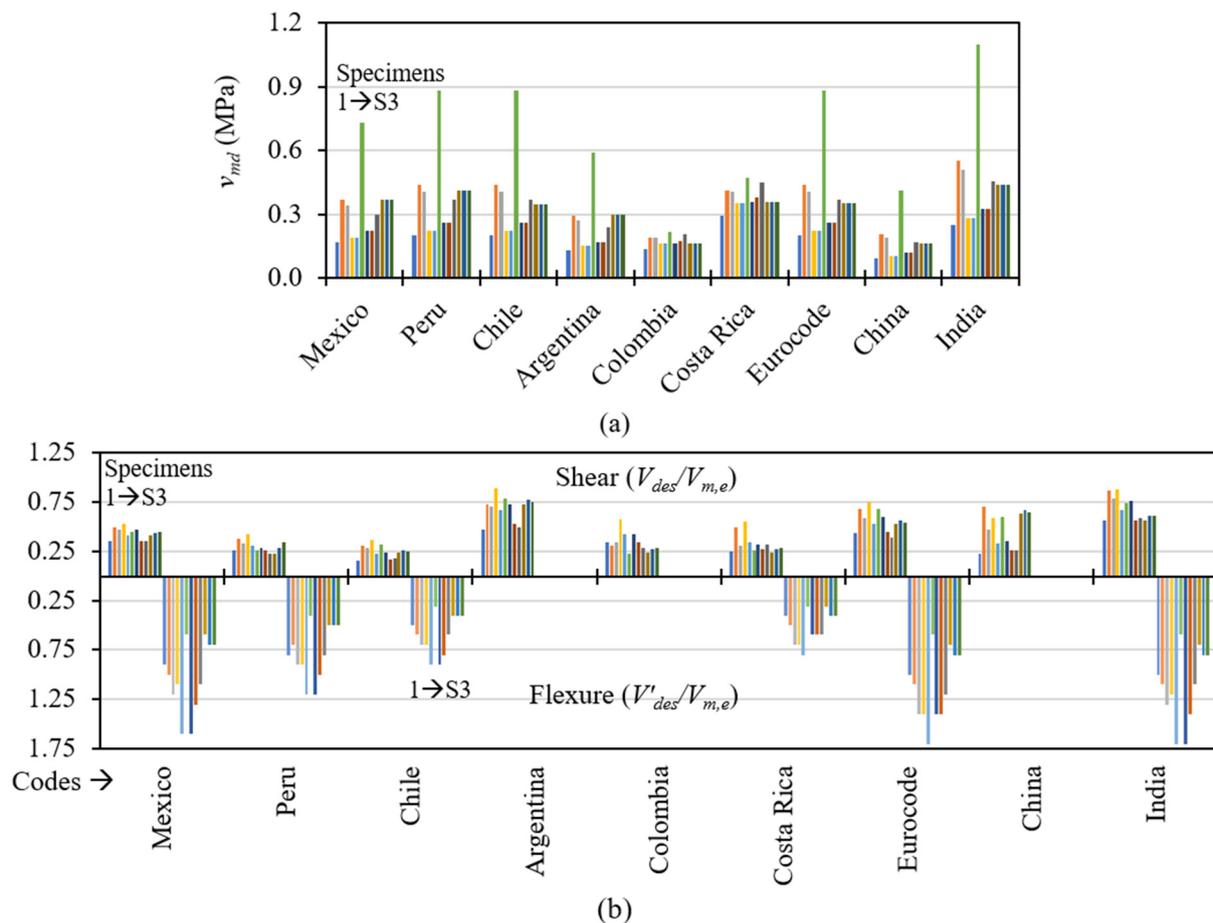


Figure 11. Estimation of (a) design shear strength of masonry (v_{md}) and (b) design-to-experimental strength against shear and flexural failure mode, using the recommendations of different codes (Each bar represent a specimen).

Borah et al. [57] conducted a study comparing the design-to-experimental strength ratios of twelve solid, single-story, single-bay CM walls with different specifications (variation mainly in aspect ratio, masonry strength, percentage reinforcement in ties, etc.) previously tested by different researchers (MO of Aguilar et al. [45], Pattern 1-Clay Brick-2 of Yáñez et al. [49], M1 of Marinilli and Castilla [50], 2 and 4 of Zabala et al. [51], JCM of Bourzam et al. [12], ME2, ME4, and ME5 of Gavilán et al. [43], S1, S2, and S3 of Borah et al. [57]) to examine the safety margins used in various codes against flexural and shear failure. These tests were conducted under the action of gravity loads and lateral quasi-static cyclic loads. The masonry design shear strength (v_{md}) estimated using different codes is shown in Figure 11a. Figure 11b demonstrates that there is a significant difference in estimated safety margins (ratio of design to experimental) in various codes against both shear failure (0.21 to 1.31) and flexural failure (0.3 to 1.7). The blank spaces in Figure 11b indicate that some codes (Argentina, Colombia, and China) do not have design flexural strength provisions. The inconsistent estimation of design strength by the design codes and the limitations of some of the design codes for the safe and economical design of CM walls against shear and flexure failure is highlighted in the study. These inconsistencies may be due to the limited availability of experimental data and the highly uncertain material properties of masonry. To enhance the construction practices of CM buildings worldwide, it is essential to conduct comprehensive experimental and finite element studies that consider all critical parameters, thereby improving the CM building design codes.

5.3. Design Force Distribution in Different CM Elements

The existing design codes and previous research studies lack clear methods for estimating individual design forces for the tie-columns and masonry walls of CM walls. Insufficient research and inconsistent design standards have led to the nominal design of tie-elements that does not consider the impact of significant parameters. Limited recommendations for the seismic design of tie-members are given by only a few design codes. The utilization of the method recommended in Peru [118] involves designing the confining columns to bear a shear force equivalent to a certain fraction of the design shear strength of the masonry wall, depending on the number of tie-columns in the CM wall and the length of the wall. According to this method, the shear force in tie-column for one bay CM walls comes out to be 50% of the design shear strength of the masonry wall. The Argentina code [120] provides some guidelines for calculating the axial strength of tie-columns in its design as a fraction of the shear strength of the wall. The Mexican code [117] proposes a basic approach to calculate the minimum reinforcement area required for tie-columns. Borah et al. [133] conducted a parametric FE study to investigate the forces in the members of a CM wall under lateral loading. Based on this, a method was developed to distribute the shear forces to the tie-columns and masonry walls of CM walls. The proposed approach suggests designing masonry walls for the total shear force equivalent to the lateral strength of CM walls. Tie-columns, on the other hand, should be designed for 15% to 50%, depending on factors such as wall thickness, masonry strength, and aspect ratio. To ensure the safe and economical design, it is important to consider the size, configuration, and reinforcement detailing. It is recommended that tie-columns should not be required to resist more than 50% of the total lateral shear force resisted by walls. In cases where tie-columns are required to resist more than 50% of the total lateral shear force, the CM walls should be reconfigured and redesigned to avoid such situations.

6. Research Challenges and Future Directions

Numerous studies have been conducted worldwide, such as in Latin American, European, and Asian countries, particularly in regions with high seismic activities, to study the behavior of CM structures. Researchers have studied the structural behavior of CM walls and identified their potential failure modes through experimental investigations and previous earthquake experiences by exploring the influence of different parameters such as material, geometry, and loading. Various numerical and analytical modeling techniques

have been proposed for the analysis of CM structures. Moreover, some construction and design rules have been established to ensure safe and adequate performance. However, despite the progress made in understanding CM structures, there are still significant gaps in our current knowledge, especially in experimental testing, analysis methods, and seismic design. The existing analytical models and design codes are mainly based on empirical observations, and more generalized analytical modeling techniques and design methodologies are needed. Additional experimental and analytical research is necessary to develop a comprehensive understanding of the behavior of CM structures and to advance our knowledge of seismic design for a desired performance level. In summary, while past studies have contributed significantly to the understanding of CM structures, further research is essential to address the remaining gaps and to develop effective seismic design guidelines and methods for CM structures that can be applied globally.

Additional research studies are needed in several areas to fill the gaps in the current knowledge:

- Aspect ratio has a significant influence on the behavior of CM walls. The available experimental studies are limited and do not provide a sufficient understanding of the influence of AR on CM behavior, especially considering the significant variations in material and construction methodologies throughout the world. Further research is needed to develop a comprehensive understanding of the influence of AR on the behavior of CM structures. Limited studies have been carried out on slender CM walls; therefore, additional studies are required to be carried out for walls with ARs greater than one.
- The current modeling and simulation techniques available for analyzing CM buildings have limited applications. A suitable interaction model is needed to simulate the interface between wall panel and confining elements. A comprehensive numerical model that can accurately simulate the response of CM walls to both gravity and lateral loads is yet to be fully developed. The development of modeling and simulation techniques is specifically required for multi-storey CM buildings and CM walls with openings.
- The literature reveals significant variations in the size and detailing of tie-elements used in CM constructions. Additional studies are needed to evaluate the behavior of tie-columns, particularly in terms of reinforcement yielding and damage patterns with variations in different parameters such as relative strength and stiffness of wall and confining elements and aspect ratio. It is important to evaluate the limiting size and detailing of tie-columns such that the system does not behave like an RC frame building.
- The current literature does not provide a clear discussion of the distribution of design lateral forces to masonry walls and tie-columns. Further research is necessary to understand the behavior of tie-elements under different loading conditions and to determine the appropriate distribution of design forces to different members of confined masonry buildings. Thickness of wall and confining elements and aspect ratio affect the wall-to-tie-column and should be compatible with existing HPU. A minimum design force for which the tie-columns should be designed needs to be established for their improved design.
- The openings in CM walls have a negative influence on their strength and deformation capacity; however, only limited studies have been performed to understand the effect of various configurations of openings. Comprehensive experimental and numerical studies are needed to quantify the effects of openings and the contribution of confining elements around them. Studies need to be conducted that consider various sizes and locations of openings in CM walls.
- To adopt performance-based seismic design methodology for CM structures, a complete backbone profile, i.e., relationship between the lateral load and the corresponding displacement, must be known. However, the existing models for predicting lateral stiffness, strength, and deformability at different performance levels of CM walls need to be assessed. This assessment will help to identify any gaps in the existing

models and improve performance-based design methodology. Extensive experimental studies need to be conducted that consider the different parameters discussed earlier for this purpose.

- Over the years, various guidelines and design codes have been established in different countries to promote the use of CM and provide basic details on its construction. However, these guidelines and codes exhibit significant differences and gaps. It is therefore necessary to evaluate their effectiveness in ensuring the seismic safety of CM buildings, particularly in countries where the available masonry is weak and soft. This assessment is essential to ensure that the guidelines and provisions of different codes can adequately ensure public safety and welfare related to the adequate performance of CM buildings.

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