The Effect of high-strength Steel Bars on the Out-Of-Plane Flexural Response of Slender Masonry Walls

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ABSTRACT

Slender masonry walls with a slenderness ratio exceeding 30 are commonly used in single-story buildings in Canada. However, the design of these walls is subject to strict limits and requirements under the Canadian masonry standard (CSA S304-24), which can affect their capacity. One way to increase wall capacity and reduce the amount of reinforcement and grout used in conventional reinforcement masonry walls is by using high-strength reinforcement (HSR). High-strength reinforcement (HSR) are widely used in concrete standards such as (ACI 318-24) and (CSA A23.3:24). However, masonry standards such as (CSA S304-24) and (TMS 402/602-22) prohibit the use of high-strength reinforcement in all structural elements including beams, walls, and columns. Due to the limited information on this topic, this research aims to study the effects of detailing slender masonry walls with high-strength reinforcement. A numerical simulation was used to predict the expected out-of-plane performance of slender masonry walls with high-strength reinforcement and conventional steel. The same height-to-thickness ratio, loads, and boundary conditions were used to compare their performance. A parametric analysis was conducted to examine the effects of key parameters, including bar yield strength, block thickness, and bar diameter, on the flexural capacity of slender masonry walls. The results indicate that an increase in bar yield strength leads to a corresponding enhancement in the flexural capacity of the walls.

KEYWORDS

slender masonry walls, high strength reinforcement, analysis model, flexural capacity

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INTRODUCTION

Masonry design has evolved from basic principles, emphasizing larger cross-sections for safety, to more sophisticated methods accommodating modern structural demands. Early designs relied on allowable stress approaches suitable for unreinforced masonry under gravity loads. With the rise of reinforced masonry, the need for advanced techniques emerged to support slender structures subject to out-of-plane flexural loads. These walls, governed by flexural behavior, often require substantial reinforcement to address second-order effects like P-δ moments. However, standards restrict reinforcement ratios to ensure ductile failure, necessitating innovative solutions.

One approach is using wider masonry blocks to increase the moment arm, but this compromises cost and efficiency, reducing masonry's competitiveness. Alternative methods, such as reinforcing with glass fiber-reinforced polymer (GFRP), have shown promise. GFRP enhances flexural strength and durability while mitigating issues like steel corrosion. Studies have demonstrated that partially grouted (PG) walls reinforced with GFRP achieve comparable performance to fully grouted walls, providing opportunities for material optimization [1]. High-strength masonry blocks are another solution. Experimental results show significant improvements in flexural strength, especially in grouted prisms, which benefit from confinement effects [2].

Textile-reinforced mortar (TRM) has also been explored, leveraging fabric-reinforced cementitious matrices (FRCM) for masonry repair and strengthening. Comparative testing revealed disparities in tensile performance based on boundary conditions, emphasizing the need for standardized testing protocols to bridge laboratory results and field applications [3]. Near-surface-mounted (NSM) reinforcement has enhanced wall stiffness and reduced cracking while maintaining flexural strength. However, challenges such as reduced shear resistance in ungrouted NSM walls highlight areas for improvement [4]. Taking into account the stiffness at the base of the wall, rather than assuming zero stiffness as in cases with rotational base stiffness (RBS), has demonstrated significant improvements in the stability of slender walls. This consideration reduces deflections and increases load capacity, offering substantial enhancements in structural performance.[5].

While current masonry design codes restrict over-reinforcement due to brittle failure concerns, concrete standards allow it under controlled conditions such that it is not a seismic area. Fiber-reinforced materials, such as highly ductile concrete (HDC) and hybrid fiber systems, have been shown to improve ductility and mitigate brittle behavior in over-reinforced systems [6], [7]. These advancements suggest potential for broader adoption in masonry design.

HIGH STRENGTH REINFORCEMENT IN MASONRY STRUCTURES

High-strength reinforcement typically has a yield strength exceeding 400 MPa. The usage of HSR in masonry structures has evolved significantly over the past several decades, reflecting advancements in both material science and engineering standards. Beginning in the 1950s, Grade 40 (276 MPa) and 50 (345 MPa) steel were prevalent, and subsequent standards introduced higher grades in response to growing demands for structural integrity.

In 1971, ACI 318-71 raised the upper yield strength limit to 80 ksi (552 MPa), while capping the maximum at 60 ksi (400 MPA) for seismic applications. Over time, further enhancements were introduced, including ASTM's development of Grades 60, 75, 80, 100, and 120. By 2019, ACI 318-19 had allowed Grade 80 reinforcement in specific seismic systems.

HSR in masonry structures offer multiple benefits, as well as some drawbacks. Chief among the advantages is enhanced member strength and reduced steel congestion, which directly translates into simplified construction and increased productivity, with a potential reduction in overall construction costs. Also, using HSR can lower carbon footprint of a building, thus aligning with more sustainable construction practices.

A notable disadvantage is the potential reduction in ductility, which can affect the deformation capacity of reinforced structures under extreme loading conditions—an especially important consideration in seismic regions.

Although current North American Masonry standards (e.g., CSA S304-24 and TMS 402/602-22) do not permit the use of HSR, standards for reinforced concrete (e.g., ACI 318-19 and CSA A23.3:24) do permit it.

The use of HSR allows for a reduction in the quantity of steel required Compared to mild reinforcement, this leads to lower reinforcement ratios in structural members, which may translate into potential increases in deformation and curvature. As using High Strength Reinforcement (HSR) causes the bars to experience greater stress, which leads to increased cracking in the masonry. ACI 318-19 provides revised approaches for calculating deflection and crack widths. Also, it includes changes to minimum dimensions and reinforcement spacing that account for the properties of HSR. For example, it offers detailed provisions for determining development lengths of tensioned bars, highlighting the need to account for the higher tensile stresses experienced by such bars and their implications for anchorage and overall structural performance. Additionally, the ACI 318-19 introduces certain restrictions on using HSR in seismic zones, as it may reduce ductility. To mitigate this, precautions such as increasing the number of stirrups are recommended. This enhancement in confinement can lead to improved ductility.

Similarly, CSA A23.3-24 adjusts requirements for minimum member thickness in beams and slabs to prevent excessive deflections, which might become more pronounced due to the higher strength of HSR. The CSA A23.3-24 standard has updated the equation for calculating the development length by incorporating a new parameter to account for the use of HSR. For beams and columns where HSR is used, enhanced confinement reinforcement is required to reduce the likelihood of buckling under load, particularly in seismic zones. This involves more stringent requirements for tie spacing and diameter around longitudinal reinforcement to ensure structural elements can sustain increased loads without premature failure.

Likewise reinforced concrete, the integration of HSR in masonry structures necessitates careful consideration of flexural capacity, shear capacity, and ductility considerations.

Flexural capacity is influenced by the development length required to fully transfer stress from the bar to the masonry. Because HSR can carry significantly larger stresses than conventional steel, ensuring an adequate development length becomes crucial. The effective stiffness in structural elements is also an important item of concern. The higher stresses in HSR tend to increase cracking in the masonry, which can reduce member stiffness more than expected. Consequently, the standard equations for estimating effective stiffness may need modification to account for the stiffer—and simultaneously more brittle—behavior introduced by HSR. Additionally, as the HSR increases cracking in masonry elements, it is also necessary to consider the deflection, as the use of HSR contributes to an increase in deflection.

The shear capacity of masonry elements with HSR requires special attention. While HSR can enhance the shear capacity provided by stirrups or external reinforcement, the masonry itself may experience reduced shear strength due to increased crack propagation under high stress.

Lastly, the ductility capacity in masonry structures with HSR is a critical concern. In earthquake-prone regions, ductility is paramount for energy dissipation and overall structural resilience. Using HSR can decrease ductility, as the higher strength bars tend to exhibit less inelastic elongation. To compensate, additional measures—such as reducing stirrup spacing to increase confinement—can be implemented. This enhanced confinement helps restore some of the lost ductility, ensuring that the structure can better withstand cyclic loading without catastrophic failure. Addressing these three areas—flexural behavior, shear resistance, and ductility— it is a possible roadmap to integrate HSR into masonry structures while maintaining reliable performance under both everyday loads and seismic events.

Khalid and Kalliontzis [8], [9], [10] conducted the only studies that address the integration of HSR in masonry structures, underscoring the importance of their work. Their research is part of a broader, ongoing effort to incorporate provisions for HSR into the TMS 402/602-22 standard. They began by examining development length, which serves as the foundational step in understanding HSR behaviour within masonry. Inadequate embedment can result in premature bond loss, leading to failure modes that do not accurately reflect the true structural performance of HSR. Establishing a proper development length thus lays the groundwork for subsequent investigations into the flexural capacity of various masonry elements. So, for the first study [8] Khalid & Kalliontzis presented a detailed series of lap-splice length tests on concrete masonry wall panels, aimed at quantifying bond strength when HSR are used. Key parameters included the reinforcement bar size and lap-splice length. The results demonstrated that Grade 80 steel bars, when properly detailed, not only comply with but exceed the requirements of TMS 402. The same authors [9] offer an extensive analysis of the bond interface between HSR bars and concrete masonry through finite element (FE) simulations validated against experimental data. The study focuses on predicting the behavior of Grade 60 and Grade 80 bars embedded in masonry under direct tension**.** The FE model validation via lap-splice tests indicated good compatibility with TMS 402 specifications, suggesting potential for broader application to more complex masonry assemblies.

Although [8], [9], [10] represent the only existing studies on the use of HSR in masonry, neither addresses the flexural behavior of out-of-plane masonry walls. As experimental investigation is time-consuming and resource-demanding, the present study adopts an efficient, simplified numerical modeling approach to examine the feasibility and effectiveness of HSR in out-of-plane masonry walls with height-to-thickness ratios exceeding 30. The focus of this study is on the flexural behavior of slender masonry walls, with shear effects being beyond the scope of this paper. A parametric analysis was conducted by varying key parameters, including bar yield strength, block thickness, and bar diameter, to evaluate their influence on structural performance.

ANALYSIS MODEL

The numerical model was developed utilizing the Open System for Earthquake Engineering Simulation (OpenSees) open-source software[11].This finite-element platform has been used to successfully capture the behaviour of slender masonry walls [12][13][14].A 2D nonlinear finite-element (FE) model was built using a macro-modeling approach, as illustrated in Figure 1. This model features a masonry wall divided into 30 nonlinear beam-column elements, which employ a fiber section with material nonlinearity. The top of the wall is constrained along the X direction in the global axis, while it remains free in the Y direction and rotational movement. At the base, the wall is restrained in both the X and Y global axes, with free rotation. The material (Concrete02) nonlinearity was captured using uniaxial stress-strain models available in the OpenSees library.

A pushover analysis was conducted using the macro model, where an eccentric axial load was applied at the top of the wall. Once the axial load was fully applied and maintained, a lateral load was incrementally applied along the height of the wall until the crushing strain in masonry at midspan was reached. Second-order effects were accounted for using the geometric transformation (corotational transformation) available in the OpenSees library.

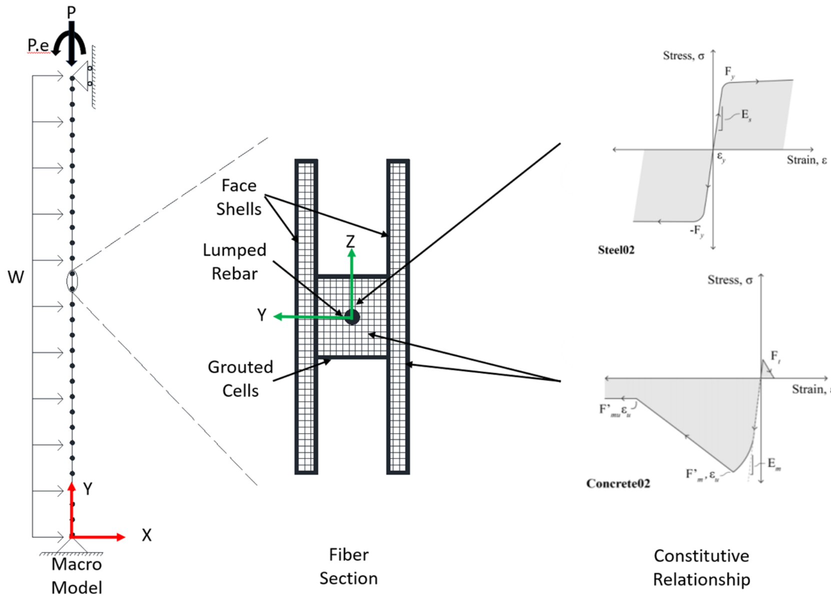

Figure 1: Masonry Wall Macro-Model (Global Axes in Red; Local Axes in Green)

Specimen configuration

The numerical specimens consist of partially grouted (PG) concrete masonry unit (CMU) walls built with standard blocks of thicknesses 190 mm, 240 mm, and 290 mm arranged in a running bond pattern. Each wall is 1.19 meters wide, with heights of 5.7 m, 7.2 m, or 8.7 m and varying thicknesses to achieve a slenderness ratio of approximately 30.

To replicate typical loading scenarios and evaluate the slenderness effect under service conditions, each specimen will be subjected to a vertical compressive load of 15 kN, applied at the top of the wall with an eccentricity of 170 mm from the centerline. The reinforcement scheme reflects typical non-seismic masonry wall designs. Vertically, two steel bars (either 15M or 20M) are placed at 600 mm on center within the reinforced cores of the CMU, providing the primary resistance against out-of-plane bending and ensuring adequate ductility under service loads. Figure 2 illustrates the geometric dimensions of the specimen.

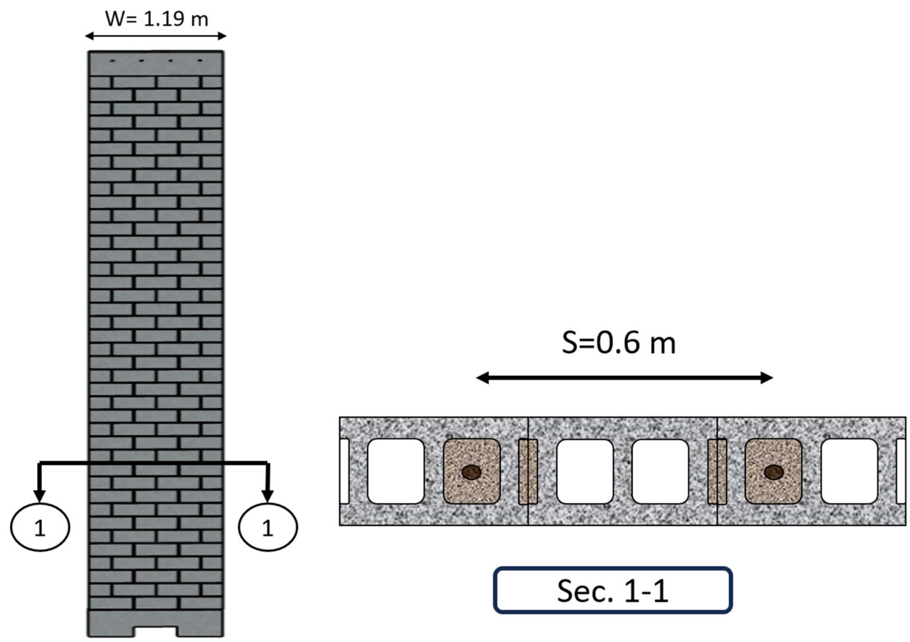

Figure 2: Specimens geometry

Material models

The longitudinal steel reinforcement was modeled using the Steel02 material model, which includes isotropic strain hardening based on the Guiffre-Menegotto-Pinto model [15]. The conventional reinforcing steel was assigned a yield strength of 400 MPa, while the HSR was likewise defined with the Steel02 model but with higher yield strengths of 500 MPa and 600 MPa with a ultimate strain equal to 0.13, 0.12 and 0.1 respectively based on ASTM A706. In the model, the steel component is represented as a single, continuous element without any lap splices. To represent the masonry assembly as a homogeneous material, the Concrete02 material model was used, based on the Kent-Scott-Park model [16]. The peak compressive strength occurs at a strain of 0.002[14]. The maximum tensile strength of the masonry was set at 0.65 MPa (CSA S304-24), with the material behaving linearly elastic up to the point of cracking and then exhibiting linear tension softening behavior.

Model validation

The model predictions were validated using the Test Report on Slender Walls (1982)[17] document prepared by ACI-SEASC. Nine fully grouted (FG) reinforced-masonry tall walls were tested under a combination of eccentric axial load and uniform out-of-plane lateral pressure applied with an airbag.

Figure 3 presents a comparison between the numerical models and experimental data. These models successfully replicate the overall behavior of the walls, encompassing both the elastic and post-cracking phases. Consequently, the principles underlying these models have been integrated to formulate models for the construction methods proposed in this study.

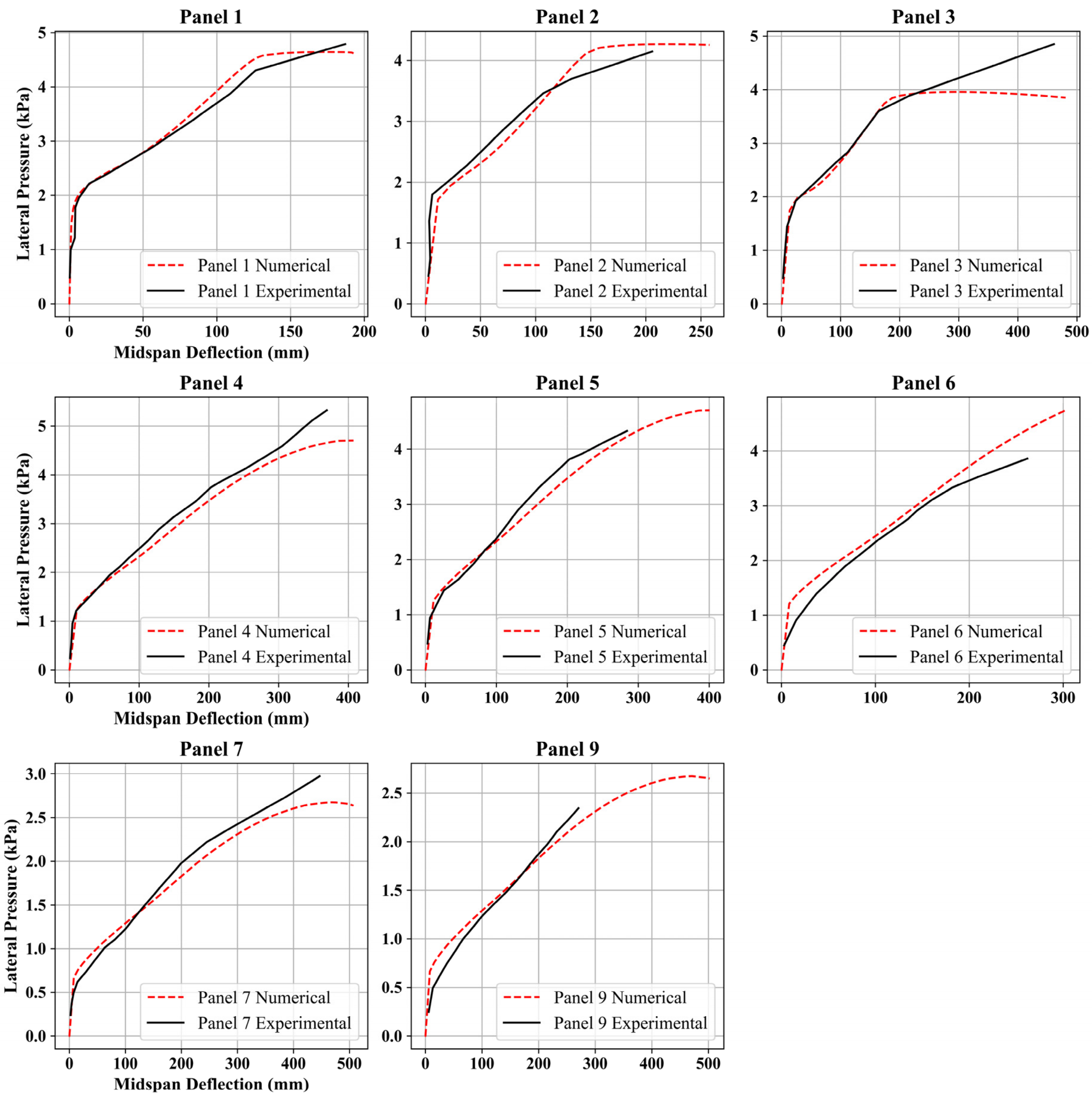

Figure 3: Model validation using the Test Report on Slender Walls (1982)

PARAMETRIC STUDY

A parametric study was conducted to evaluate the effect of the use of HSR in the out-of-plane flexural response of loadbearing-slender masonry walls

Fixed parameters

Table 1 shows the fixed parameters for a slender masonry wall with a 1000 mm effective width, a 15 kN axial load at 170 mm eccentricity, and a slenderness ratio of 30. The 15 MPa block compressive strength and 600 mm grout and bar spacing highlight key factors influencing the wall's out-of-plane performance

Table 1: Parametric study fixed parameters

Parameter	Value
Wall effective width	1000 mm
Axial load	15 kN
Load eccentricity	170 mm
Block compressive strength	15 MPa
Slenderness ratio	30
Grout & Bar spacing	600 mm

Independent parameters

Table 2 presents variable parameters for analyzing slender masonry walls. Block thickness ranges from 190 mm to 290 mm, allowing for variations in wall geometry. Bar diameters span 10 mm to 25 mm, accommodating different reinforcement configurations. Yield strength values of 400 MPa to 600 MPa reflect the use of HSR reinforcement, influencing the wall's load-carrying capacity and overall performance.

Table 2: Parametric study variable parameters

Parameter	Value
Block thickness	[190, 240, 290] mm
Bar Diameter	[10, 15, 20, 25] mm
Yield strength	[400, 500, 600] MPa

Dependent Parameters

The weight and height of the slender masonry wall were the dependent variables in this study. These parameters were derived from various cases involving block thickness, bar diameter, and yield strength.

RESULTS AND DISCUSSION

Maximum amount of reinforcement

According to the latest Canadian masonry standard (CSA S304-24), slender masonry walls with a slenderness ratio of 30 or greater must be designed as under-reinforced sections. To study the effect of the use high strength reinforcement on the concept of under-reinforced sections figures from 4 to 6 show the bar diameters that make the section under-reinforced referring to this bar diameter with the accepted bar diameters.

Figure 4 illustrates the permissible bar diameters for various yield strengths when using a 190-mm block thickness. Although multiple bar diameters and yield strengths were examined, the findings indicate that only a 15-mm diameter bar satisfies the Canadian masonry standard for slender masonry walls. Similarly, Figure 5 shows the acceptable bar diameters for different yield strengths when using a 240-mm block thickness, with results again indicating that only a 15-mm diameter bar meets the Canadian masonry standard. Finally, Figure 6 presents the acceptable bar diameters for a 290-mm block thickness, and in this case, only a 20-mm diameter bar is found to comply with the Canadian masonry standard.

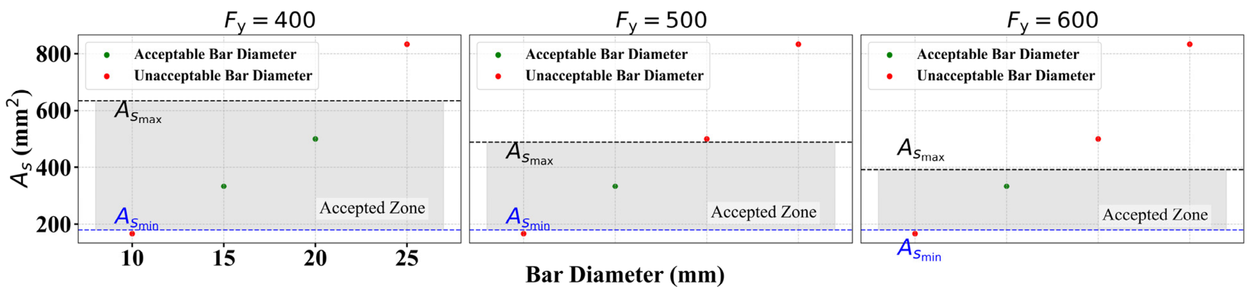

Figure 4: Acceptable bar diameters for various yield strengths in a 190 mm-thick block, with bar and grout spacing of 600 mm.

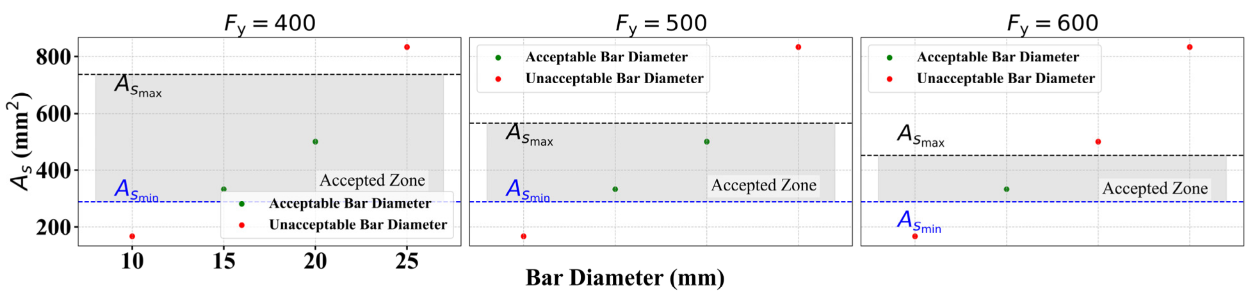

Figure 5: Acceptable bar diameters for various yield strengths in a 240 mm-thick block, with bar and grout spacing of 600 mm.

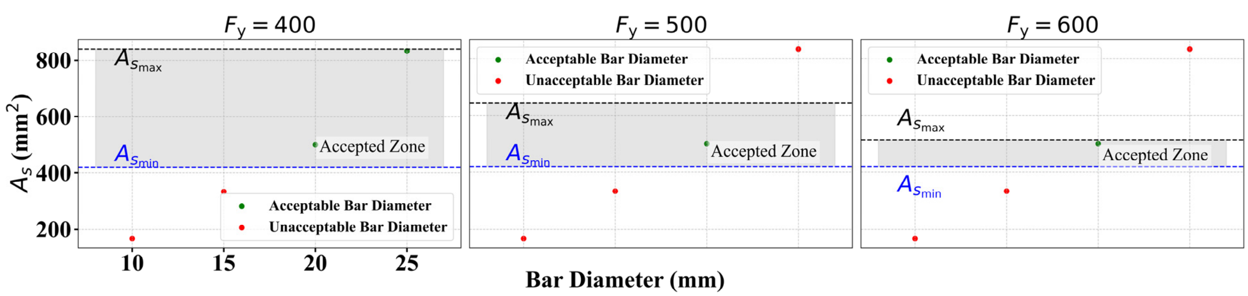

Figure 6: Acceptable bar diameters for various yield strengths in a 290 mm-thick block, with bar and grout spacing of 600 mm.

Effect of reinforcement yield strength

A push-over analysis was conducted based on the bar diameter laid in the accepted zone stated in the Maximum amount of reinforcement section to investigate the effect of increasing the yield strength of the bar on the under-reinforced section. Figure 7 presents capacity curves for slender masonry wall specimens with varying block thicknesses, reinforcement bar sizes, and steel yield strengths. In Figure 7(a), three walls with 15M bars @ 600 mm and a 190 mm block thickness show an initial steep slope, reflecting elastic behavior with minimal cracking and reinforcement below yield. As lateral pressure increases, steel yielding transitions the response into a nonlinear regime. The wall with 600 MPa steel achieves the highest peak load and deformation capacity.

Similarly, in Figure 7(b), walls with 15M bars @ 600 mm and a 240 mm block thickness demonstrate similar trends, with the 600 MPa specimen reaching a peak load approximately 35% higher than the 400

MPa specimen and 17% higher than the 500 MPa specimen, while the 500 MPa wall surpasses the 400 MPa by around 20%. These results confirm the capacity gains from higher yield-strength steel.

In Figure 7(c), walls with 20M @ 600 mm bars and a 290 mm block thickness exhibit comparable behavior, with the 600 MPa specimen achieving an ultimate load capacity 45% higher than the 400 MPa specimen and 15% higher than the 500 MPa specimen, while the 500 MPa wall exceeds the 400 MPa by 25%. Across all cases, softening beyond the peak load is observed due to extensive masonry cracking and plastic deformation in the steel.

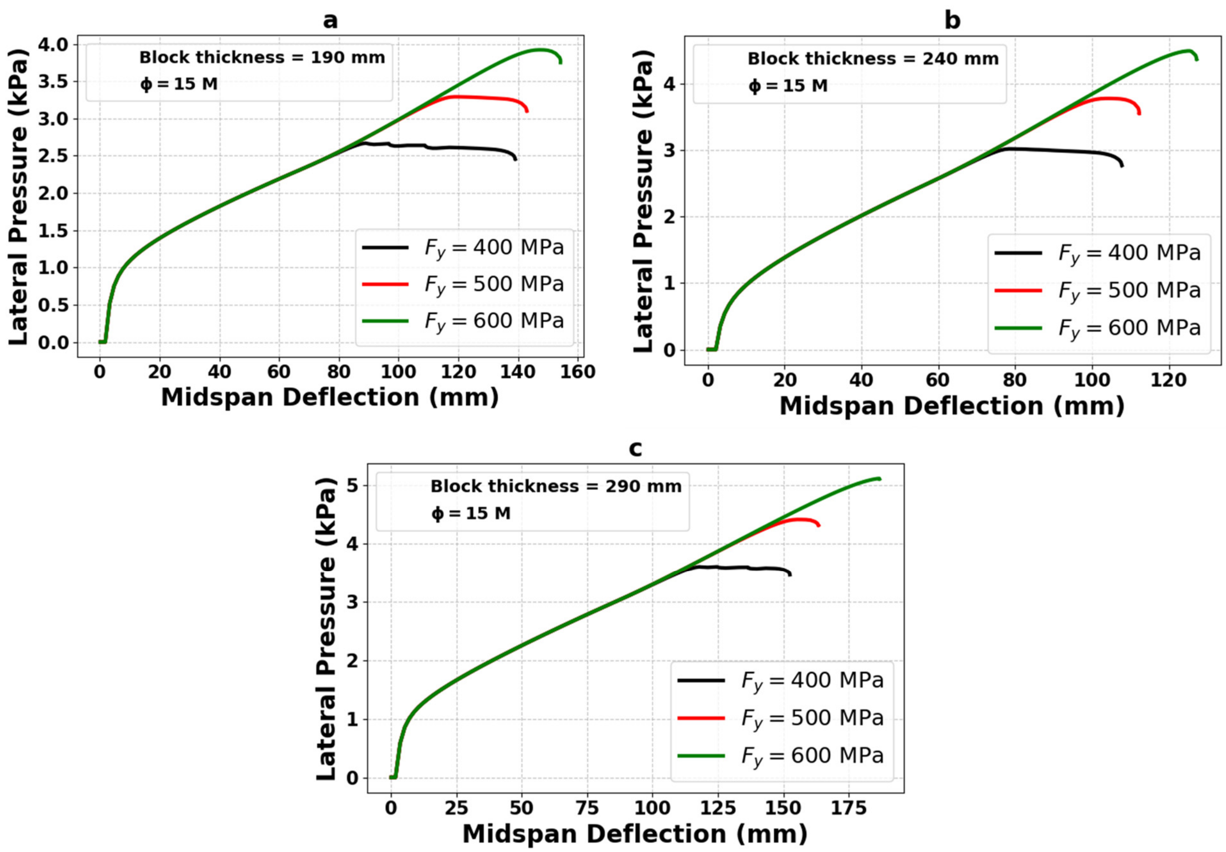

Figure 7: Capacity Curves for Varying Block Thicknesses and Yield Strengths

Effect of reinforcement ratio

A push-over analysis was carried out to show the difference between the under-reinforced section (15M) and the over-reinforced section (25M).

Figures 8 present the lateral pressure-deflection behavior of masonry walls with varying block thicknesses, reinforcement diameters, and steel yield strengths. For a 190 mm thick block with 400 MPa steel (Figure 8-a), the 25M reinforcement provides greater stiffness, evidenced by a steeper slope, and higher load-carrying capacity. 25M reinforcement achieves lateral pressure 75–80% higher than the 15M, with peak pressures of 6.5 kPa and 4 kPa, respectively. Similarly, for a 190 mm thick block with 500 MPa steel (Figure 8-b), the 25M reinforcement again outperforms the 15M, with approximately 50% higher lateral pressure at 25 mm deflection and a 40% increase in peak capacity (6.2 kPa versus 4.5 kPa). However, for a 190 mm thick block with 600 MPa steel (Figure 8-c), the 25M reinforcement surpasses the 15M across most of the deflection range. At 20 mm deflection, the 25M specimen achieves 30% higher lateral pressure, and near peak deflections (80–90 mm), it reaches 6.5 kPa compared to 4.5 kPa for the 15M, reflecting a 25–30% advantage.

A comparison of Figure 8 reveals that for over-reinforced masonry sections, increasing the yield strength of the reinforcement does not significantly influence overall wall capacity, as failure is primarily governed by the compressive capacity of the masonry units or the wall thickness rather than the tensile capacity of the steel. However, the analysis highlights substantial capacity gains with larger bar diameters, such as 25M compared to 15M, which can be effectively utilized in non-seismic regions where concerns about brittle failure are less critical. This suggests that selecting larger bar diameters could enhance load-bearing capacity without introducing additional ductility-related risks in areas not subjected to high seismic demands.

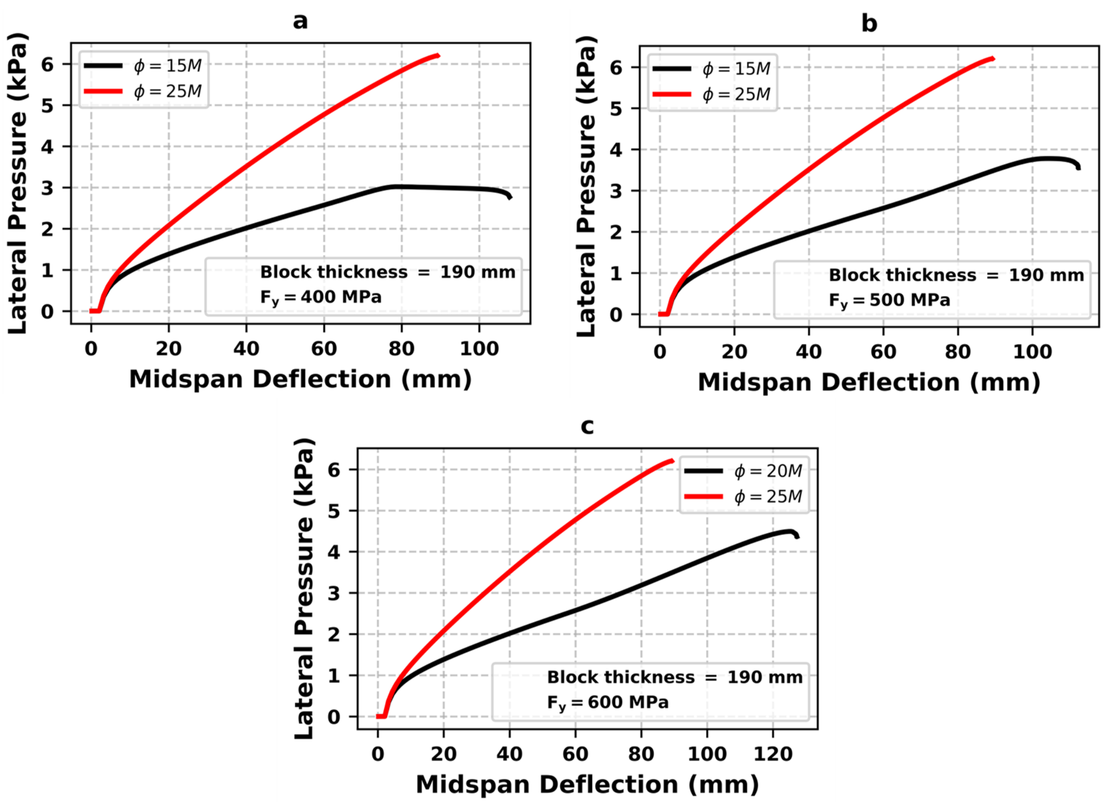

Figure 8: Capacity Curves Under Varying Yield Strengths and Bar Diameters

CONCLUSION

The study presents numerical analysis of the impact of HSR reinforcement on the out-of-plane flexural response of slender masonry walls, which have a height-to-thickness ratio exceeding 30. Through numerical modeling and parametric analyses, this research elucidates crucial aspects of structural performance, including capacity and ductility, across varying yield strengths of reinforcement bars.

The findings reveal that the use of HSR, particularly those with a 600 MPa yield strength, substantially increases the flexural capacity and enhances the load-bearing performance of slender masonry walls, while preserving ductile failure modes. It is important to note that these improvements are primarily observed in under-reinforced sections of the walls. Conversely, in over-reinforced sections, increasing the yield strength of the reinforcement bars does not influence the flexural capacity. However, using an over-reinforced section with the same yield strength leads to an increase in flexural capacity.

This investigation addresses a significant void in existing literature regarding the application of HSR in masonry structures under out-of-plane loads. The insights gained from this study suggest the potential for integrating higher yield strengths into masonry structures. Nonetheless, it is evident that further experimental research is essential to validate these findings and to facilitate the introduction of new yield strengths in masonry structural design. For future research, a comprehensive full-scale experimental study will be conducted to thoroughly investigate the flexural behavior of an out of plane wall associated with the proposed methods utilizing HSR. Numerical simulations will also be conducted to assess the shear capacity of beams while using HSR. Additionally, lap splice specimens will be subjected to experimental examination in the laboratory to substantiate our findings comprehensively.

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