

Numerical parametric investigation on the compression and shear-compression response of masonry panels strengthened by CRM

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ABSTRACT

In recent years, strengthening techniques based on externally bonded composite materials have been increasingly adopted to improve the structural performance of masonry structures. Among these, Composite Reinforced Mortar (CRM) systems have proven effective for both compression and shear-compression retrofitting. Despite their growing use in engineering practice, research on CRM-strengthened masonry remains limited, particularly about in-plane flexural capacity, drift performance, and the contribution of CRM layers to out-of-plane stability. Moreover, comprehensive analytical models capable of explicitly quantifying the mechanical contribution of CRM reinforcement are still lacking, with current design guidelines mainly relying on simplified amplification factors applied to masonry mechanical properties. This study presents the development of a finite-element model capable of reproducing both the shear-compression response and the out-of-plane buckling behavior of masonry walls under compression strengthened with CRM systems. A parametric numerical investigation is carried out to analyze the in-plane shear response and out-of-plane buckling behavior of CRM-retrofitted panels. The numerical models are calibrated using experimental results from previous tests conducted by the authors and validated against additional data available in the literature. The results identify the key parameters governing the CRM-masonry interaction and enable the proposal of an analytical approach for estimating the effective mechanical properties of retrofitted masonry panels.

1. Introduction

The Italian territory is characterized by a vast heritage of historical masonry buildings, which are generally highly vulnerable to seismic actions due to the high masses and low tensile strength of masonry. Although masonry exhibits substantial compressive capacity, walls, especially slender ones, remain susceptible to out-of-plane instability when vertical loads act even with small eccentricities. To improve the mechanical performance of existing structures, several strengthening systems have been developed, among which inorganic composite materials have gained increasing prominence in recent decades. Composite-based strengthening systems represent promising solutions for enhancing the seismic behaviour of masonry structures, with respect to both in-plane [1–3] and out-of-plane failure mechanisms [4–6]. Among these, experimental evidence consistently shows that CRM (Composite Reinforced Mortar) can enhance both vertical load and seismic capacity [3,7,8], with its contribution under compression becoming particularly significant when the wall is prone to out-of-plane buckling [9–12]. Several experimental studies have investigated the seismic response of

masonry panels strengthened with CRM. Most of the available research is based on diagonal compression tests [1,2,13–15], although more recent works have employed shear-compression testing on piers and shear tests on spandrels [16–19], which provides a more direct assessment of the effectiveness of the strengthening systems under horizontal loading. Furthermore, the effectiveness of CRM strongly depends on the continuity of reinforcement along the wall height and the presence of strengthened spandrels, as interruptions at floor levels can reduce its contribution to flexural and compressive behaviour [9]. Given the diversity of geometries and boundary conditions encountered in practice, numerical modelling is essential to generalize experimental observations. Existing approaches range from detailed micro-models to homogenized macro-models and limit-analysis schemes [20,21]. For CRM-strengthened walls, macro-modelling is typically preferred, with composite layers represented through equivalent material parameters and interfaces capturing potential slippage [16,22,23]. Despite the growing experimental and numerical research, neither Italian nor European standards currently provide analytical procedures specifically tailored to CRM-reinforced masonry, and available code amplification

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factors do not explicitly incorporate reinforcement properties or failure mechanisms [24]. In this study, a finite element model was developed and carefully calibrated on the results of an extensive experimental program conducted by the authors [12,17]. After validating its ability to reproduce the response of both unreinforced (URM) and CRM-strengthened (RM) walls, the model was employed in a parametric investigation aimed at assessing the influence of CRM on in-plane shear-compression behaviour and out-of-plane buckling under compression. The results enabled the identification of governing parameters and supported the formulation of predictive analytical relationships.

2. Numerical modelling

The compression and shear-compression behaviour of CRM-strengthened masonry panels was investigated using a three-dimensional finite-element model developed in DIANA FEA v. 10.5 [25]. A macro-modelling strategy was adopted, employing eight-node solid elements and the Total Strain Crack (TSC) model [26]. Linear tensile softening and parabolic compressive hardening/softening were used to represent the masonry response. Constitutive parameters included the Young modulus (E_m), Poisson ratio (ν_m), compressive and tensile strengths ($f_{c,m}$ and $f_{t,m}$), fracture energies (G_c and G_t), and the crack bandwidth (h), computed according to the Rots element-based criterion ($h = \sqrt[3]{V}$). The compressive strength reduction under large transverse tensile strains was accounted for by using the formulation by Vecchio and Collins [27]. For CRM-reinforced panels, a detailed macro-modelling approach was adopted to explicitly represent the interface between the mortar coating and the masonry substrate to reproduce possible debonding at the masonry–mortar interface. The numerical model was calibrated using the experimental results reported in Vienni et al. [17]. The cited study provided mechanical characterization of the solid-brick masonry and of each component of the CRM system, consisting of a 30-mm-thick NHL 3.5 hydraulic lime mortar layer and a glass-fibre mesh with 80 mm $\times$ 80 mm openings. Mechanical tests included compression and flexural tests on the coating mortar, direct tensile tests on the glass-fibre wires, tensile tests on CRM coupons, and bond tests at the brick–CRM interface. In particular, clamping-grip tests on CRM coupons allowed the mechanical properties of the strengthening system to be defined: the ultimate tensile strength of the CRM, equal to $\sigma_u = 503$ MPa for yarns in the weft direction and $\sigma_u = 485$ MPa for yarns in the warp direction, was derived. Furthermore, the tensile strength of the mortar was also evaluated as the ratio between the load at first cracking and the net plaster area of the coupons, obtaining an average value of $f_{t,pl} = 0.90$ MPa (CoV = 17%). Fig. 1 shows

the experimental tensile behavior of the CRM, as obtained from the coupon tests. Moreover, single-lap bond tests were carried out to evaluate the interface shear strength between the plaster and solid brick masonry. Six specimens with dimensions 230x120x390 mm³ were tested, each reinforced with CRM strips applied on the short side of specimens and characterized by a bonded length of 300 mm. The specimens were cleaned and wet before applying the 30 mm thick CRM, reinforced with a grid comprising two longitudinal yarns. To allow tensile loading, the upper 320 mm of the grid was left unbonded. Push–pull tests were performed by restraining the masonry against vertical movement, while the free ends of the fibres were clamped to the testing machine. The tests were displacement-controlled at a constant rate of 0.0033 mm/s. Four LVDTs measured the axial displacements of the fibres and the mortar. Two LVDTs (s_1 and s_2) recorded the displacement of the fibres just outside the bonded length, while two additional LVDTs (s_3 and s_4), attached to L-shaped profiles glued to the plaster, measured the displacement of the upper mortar layer. The LVDTs were installed symmetrically on both sides of the specimens to detect possible asymmetric response. The fibre displacement relative to the substrate, s_f , and the mortar displacement, s_{pl} , were obtained as the average values of the corresponding LVDT pairs. A schematic view of the test setup is shown in Fig. 2 together with the results in applied tensile load to yarns' slip s_f . All specimens showed the same failure, due to the debonding at the matrix-to-masonry interface without significant damage of the wall, usually preceded by the cracking of the upper portion of the mortar. The interface strength was calculated as the ratio between the ultimate debonding load and the bonded area (300 $\times$ 120 mm²), yielding an average value of 0.24 MPa (CoV = 13%). Finally, uniaxial compression and shear-compression tests were performed on both unreinforced and CRM-strengthened piers to evaluate the effect of reinforced plaster on the structural response. After calibration, the reliability of the proposed modelling approach was assessed by comparing the numerical predictions with further experimental results from the literature.

2.1. Calibration of the numerical model

The numerical model was calibrated using the experimental data reported in Vienni et al. [17]. Material characterization tests were first employed to define the mechanical properties of the masonry substrate, the CRM strengthening system, and their interface. The model was then used to simulate the experimentally conducted shear–compression tests, to assess its capability to reproduce the in-plane response of masonry panels. In a final step, the numerical model was used to simulate the direct compression tests performed with and without load eccentricity.

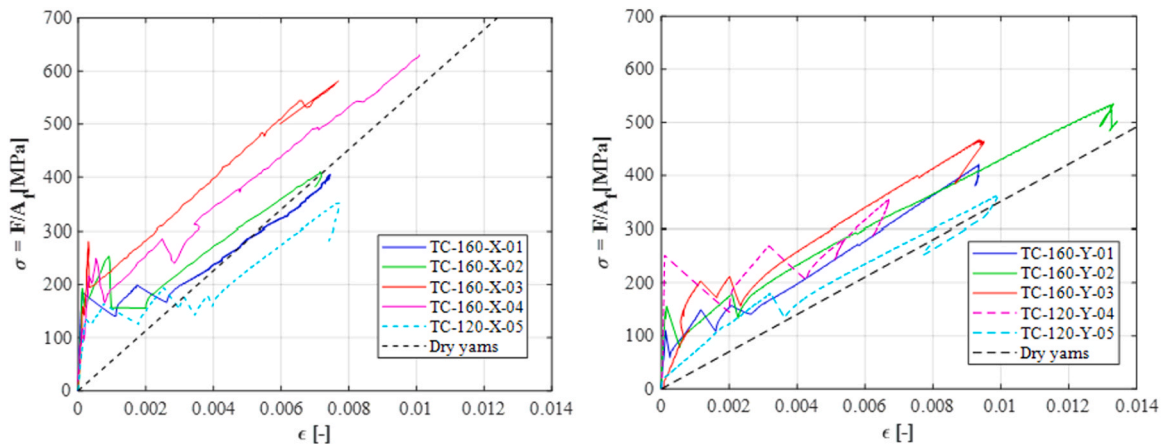


Fig. 1. Trilinear behaviour of CRM coupons derived from experimental tests [17]: (a) longitudinal yarns in the weft direction; (b) longitudinal yarns in the warp direction.

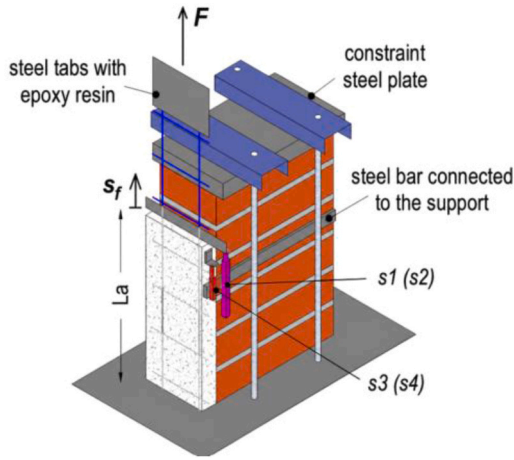
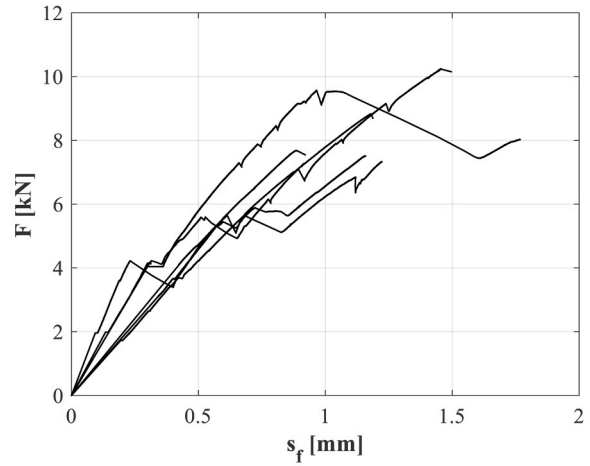


Fig. 2. Schematic view of the bond test setup and tensile load-fibre slip response [17].



This phase enabled further refinement of the calibration, with particular emphasis on the constitutive modelling of the plaster–substrate interface. The calibration procedure is described in detail in the following sections.

2.1.1. Shear-compression response

The lateral response of URM piers is influenced by several parameters, most notably the compression ratio $p = \sigma_0/f_{c,m}$, being σ_0 the average compression stress, the panel height-to-width ratio H/L , the static scheme, and the masonry typology. Among these, the compression ratio plays a dominant role in governing the failure mode, distinguishing rocking for low p , shear failure for intermediate p , and toe-crushing for high p . For the calibration of the numerical model, the response of URM panels was first analysed using the results of shear-compression tests available in two online experimental databases, compiled by Vanin et al. [28] and Morandi et al. [29]. For brevity, only two representative tests are presented in Fig. 3. These specimens exhibit different masonry typologies, geometries, boundary conditions, and axial load ratios, and consequently show distinct failure mechanisms. The calibration process enabled the definition of consistent relationships between the numerical input parameters and the mechanical properties reported in the experimental studies, to minimize discrepancies between experimental and numerical capacity curves. As a result, for all types of masonry, the compressive strength and elastic modulus are typically derived from experimental data obtained from the reference testing, while the tensile strength of masonry was set to $f_{t,m} = 1/100 f_{c,m}$. The calibrated lower-bound factor in the Vecchio and Collins (1993) formulation for

compressive strength degradation due to lateral cracking was set to 0.50. Literature values for the masonry compressive fracture energy typically range between 0.010 J/mm^2 and 0.020 J/mm^2 and are approximately two orders of magnitude larger than the corresponding tensile fracture energy. The calibration yielded compressive and tensile fracture energies of 0.012 J/mm^2 and $G_c/120$, respectively, consistent with these ranges. Focusing on the CRM-strengthened panels, the numerical model was calibrated using the results of the experimental campaign presented in [17]. The mechanical properties of the masonry and CRM constituents were derived directly from the corresponding material characterization tests. The mortar coating forming the CRM was modelled using the same smeared-crack constitutive law adopted for the masonry. The compressive strength $f_{c,pl}$ was obtained from prism compression tests and set to 13.80 MPa , while the elastic modulus E_{pl} and tensile strength $f_{t,pl}$ were derived from clamping-grip tensile tests and set to 10 GPa and 0.90 MPa , respectively. The compressive and tensile fracture energies were estimated using the Model Code 90 formulations [30], yielding $G_{c,pl} = 15 + 0.43 \bullet f_{c,pl} - 0.0036 \bullet f_{c,pl}^2 = 0.020 \text{ J/mm}^2$ and $G_{t,pl} = 0.025 \bullet (f_{c,pl}/10)^{0.7} = 0.000033 \text{ J/mm}^2$.

The reinforcement mesh was modelled using embedded reinforcement elements. A multi-linear elastic constitutive law was assigned to the fibres, assuming linear-elastic brittle behaviour in tension and no resistance in compression. The tensile strength was defined as the minimum between that obtained from bare-yarn tensile tests and from clamping-grip tests on CRM coupons; the elastic modulus was derived from bare-yarn tests. Since in the experimental campaign conducted by the authors [17], the CRM-strengthened walls exhibited debonding at

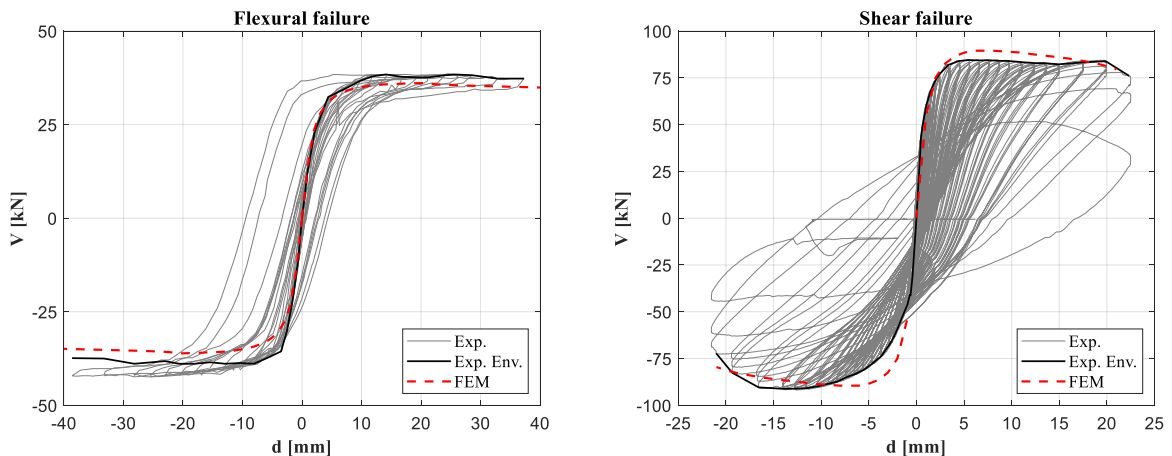


Fig. 3. Comparison between experimental and numerical shear compression results (from Vanin et al., 2017).

the masonry-mortar interface, reducing the efficacy of the strengthening, a two-dimensional interface based on the combined cracking-shearing-crushing model [31] was introduced between the plaster and the masonry to capture the limited shear stiffness and strength associated with possible detachment. The interface formulation is based on multi-surface plasticity, combining a Coulomb friction law with a tension cut-off and an elliptical compression cap. The shear yield stress is computed as

$$f_2(\sigma, \kappa_2) = \tau + \sigma \tan \Phi - \hat{c}(\kappa_2) \quad (1)$$

where the yield value $\hat{c}(\kappa_2)$ can be expressed as

$$\hat{c}(\kappa_2) = c e^{-c/G_f^H \kappa_2} \quad (2)$$

where c is the cohesion, Φ the friction angle, G_f^H the mode-II fracture energy, and κ_2 a softening parameter. Mode-I behaviour follows an exponential softening law, while compression follows a parabolic hardening-softening response. Mode-II parameters (G , G_f^H , and c) were calibrated from experimental bond-test results, while the interface behaviour perpendicular to the plane was not explicitly modelled. Out-of-plane effects, such as parasitic bending, observed during bond tests, were indirectly accounted for through the calibration of shear parameters. Consequently, the same strength, stiffness, and fracture energy as the coating mortar were assigned to the normal tensile and compressive behaviour of the interface. Frictional effects were neglected by assigning a very small friction angle ($\Phi = 10^{-4}$). The interface shear strength $\tau_{\max}$ was set equal to the average value obtained from experimental bond tests, namely 0.24 MPa. The mode-II fracture energy $G_f^H = 0.00036 \text{ J/mm}^2$ and the stiffness value were calibrated to reproduce the response observed in shear-compression and uniaxial compression experimental tests on CRM-reinforced panels conducted in [17]. Concerning transverse connectors, experimental evidence indicated that their presence, in the typical number of 4–5/m², has a negligible influence on interface strength, which is mainly governed by the mortar-substrate adhesion. Their role is instead to retain the plaster after debonding, slightly improving the post-peak ductility by delaying spalling, as observed also in other studies [14]. For this reason, and to reduce computational cost, transverse anchors were not explicitly modelled. Table 1 shows the input parameters of the model, and Fig. 4 provides an extruded view of the adopted configuration.

With the input parameters defined as above, the numerical model was calibrated by reproducing the shear-compression tests carried out in [17]. The experimental setup used in the testing campaign and shown in Fig. 5 was simulated. The setup consisted of a self-balanced steel frame specifically designed for shear-compression testing of masonry

panels with dimensions $1000 \times 1250 \times 240 \text{ mm}^3$. A constant vertical load of 350 kN, equal to the maximum capacity of the vertical actuator, was applied and maintained constant during the test, while horizontal loading was imposed in displacement control via a horizontal actuator. A HE280B beam was positioned at the top of the panel to ensure uniform distribution of the vertical load and to transfer the horizontal action. Out-of-plane displacements were restrained by an adjustable system composed of two UPN 160 beams and steel plates reacting against the top beam; Teflon sheets were interposed to minimize friction and allow unrestrained in-plane movement. To reduce the free height of the wall preventing premature rocking failure, an intermediate restraint was introduced at approximately mid-height. This consisted of two steel plates ($300 \times 600 \times 50 \text{ mm}^3$) clamped against the panel through pre-loaded M24 bolts connected to two horizontal HE320B beams. This configuration effectively reduced the panel's free height, avoiding rocking mechanisms that would otherwise be induced by the combined effect of the limited vertical load and cantilever boundary conditions for the full panel height (1250 mm). Preventing rocking was essential to enable a reliable assessment of the CRM strengthening system, whose grid was not anchored at the base. Three tests were performed: one on the unreinforced panel and two on panels strengthened with CRM on one or both faces. The response of the unreinforced wall (SC_URM_02) and of the panels reinforced on two sides (SC_RM_2_02) and on one side (SC_RM_1_02) was numerically simulated. The numerical model reproduced the actual experimental boundary conditions. For the strengthened specimens, the CRM layers were modelled using 30-mm-thick solid elements placed on the relevant wall surfaces. A schematic view of the three-dimensional model is provided in Fig. 6. A vertical load of 350 kN was applied on the upper face of the top beam, corresponding to the vertical action imposed in the experimental tests. The horizontal action was imposed in displacement control on the upper portion of the reaction beam, monotonically increasing the displacement in the x-direction. Vertical displacements of the top section were left free to permit rotation at the top. At the base, the wall was restrained against translations, and the lateral steel beams were supported at their ends by hinges. Fig. 7 compares the capacity curves obtained from the finite element simulations (dotted red lines) with the experimental envelope responses (black lines), showing good agreement between numerical and experimental results. Although the overall behavior is well captured, a difference is observed in the post-peak response of the reinforced walls, where the FE models exhibit a slight increase in resistance after the experimental peak load. This discrepancy is attributed to different damage evolution mechanisms: experimentally, failure is governed by brittle shear, particularly for specimen SC_CRM_2, with rapid formation of a dominant diagonal crack and a sudden strength drop, also related to the plaster detachment, whereas the numerical model predicts a more gradual and distributed cracking process with delayed localization. Nevertheless, the global response is well reproduced, and the predicted failure mode is fully consistent with the experimental observations. After calibration, a sensitivity analysis was conducted on the two-sided reinforced configuration to evaluate the influence of the main input parameters. Nonlinear shear-compression simulations were performed by varying the interface shear properties, mortar mechanical parameters, and fiber tensile strength. Each calibrated parameter was independently increased or decreased by a factor of 1.5 while keeping all the other parameters constant. The results confirmed the robustness of the numerical model, as moderate parameter variations produced only limited changes in the global response. Regarding the masonry-to-plaster interface properties, the peak shear strength exhibited negligible influence, whereas the mode II fracture energy G_f^H is the parameter which the results were most sensitive to, modifying the ultimate displacement by +6% and –15% when increased or reduced, respectively. Variations in plaster compressive strength were minimal, while tensile strength showed a direct proportionality with panel shear capacity. Increasing the plaster elastic modulus led to slight reductions in strength and ductility, attributed to

Table 1
Input parameters of the numerical model.

	Brick masonry	CRM Mortar	GFRP grid	Interface
Compr. strength f_c [MPa]	11.2	13.8	0	13.8
Tensile strength f_t [MPa]	0.11	0.90	503 (x) - 485 (y)	0.90
Young modulus E [MPa]	4848	10000	56500 (x) - 35100 (y)	-
Fracture energy G_c [J/mm ²]	0.012	0.020	-	0.020
Fracture energy G_t [J/mm ²]	0.0001	0.00004	-	0.000033
$\tau_{\max} = c$ [MPa]	-	-	-	0.24
G_f^H [J/mm ²]	-	-	-	0.00036
$G_{\text{interface}}$ [N/mm/ mm ²]	-	-	-	0.80

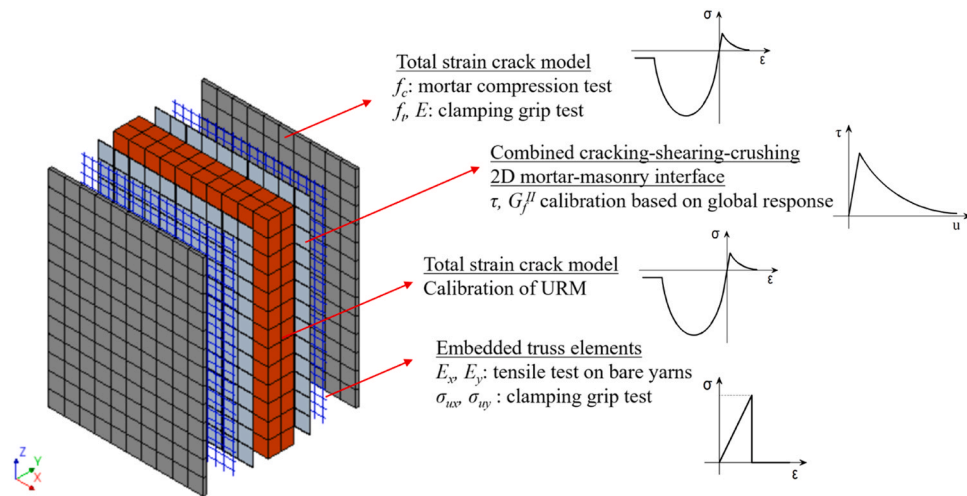


Fig. 4. Extruded view of the numerical model of CRM reinforced wall and constitutive laws of materials.

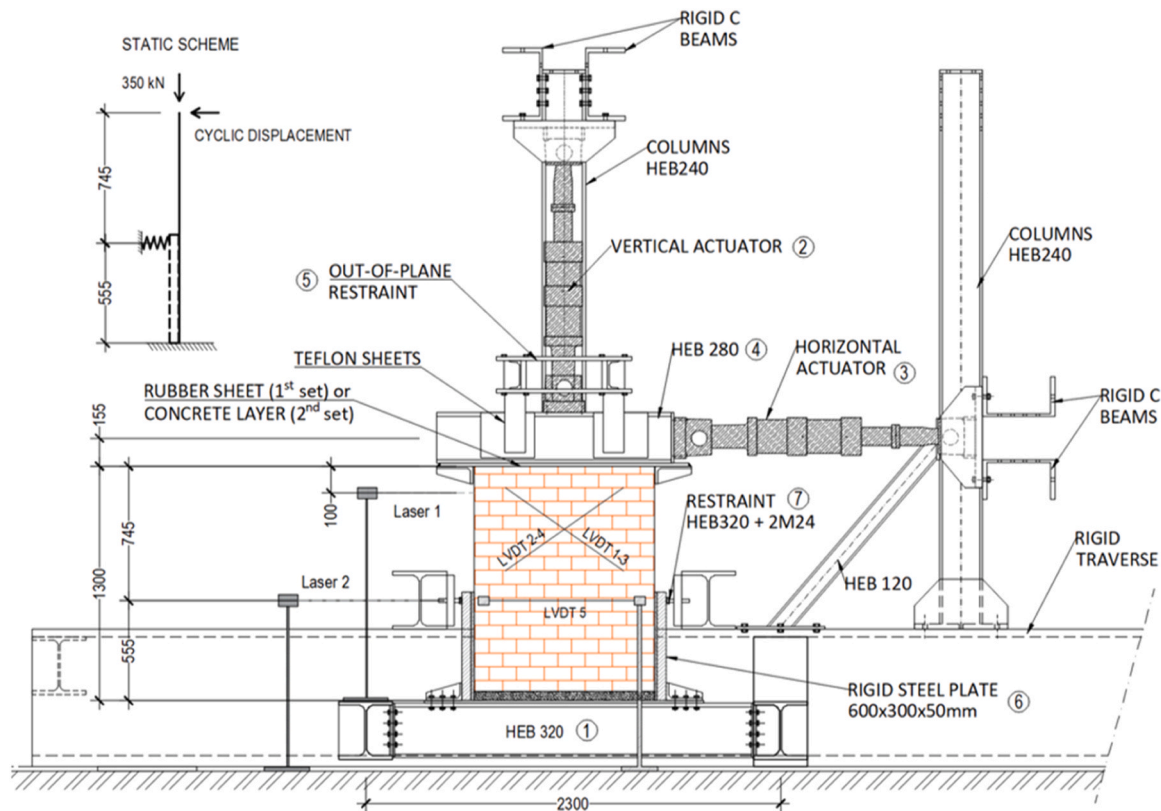


Fig. 5. Longitudinal view of the experimental setup used for the shear-compression tests in Vienni et al., 2024 [17].

premature debonding, consistent with previous experimental observations. Finally, modifying the tensile strength of the GFRP yarns had a limited impact, as their contribution becomes significant only after matrix spalling [12,14,17].

2.1.2. Uniaxial compression response

The numerical model was further calibrated to reproduce not only the shear-compression behavior but also the compression response of CRM-reinforced panels. To this end, centered and eccentric compression tests on slender columns experimentally tested in [17] were simulated. The experimental tests were conducted on small-scale masonry specimens with size 230 mm $\times$ 115 mm $\times$ 1150 mm to allow their placement

within the testing machines available at the University of Florence laboratory. Although these dimensions are not representative of real masonry structures, the results were used for an initial calibration of the numerical model, which was subsequently validated based on additional experimental tests from the literature carried out on larger-scale panels [9]. Therefore, five FE models were developed, reproducing the experimental setup. Columns were analyzed either unreinforced or strengthened with one or two CRM layers and subjected to concentric or eccentric axial loading. Models COMP_2_c, COMP_1_c, and COMP_2_ecc replicated the available experimental tests (Fig. 8), enabling direct comparison with measured responses. Models COMP_UR_c and COMP_UR_ecc were also introduced to numerically assess the contribution of

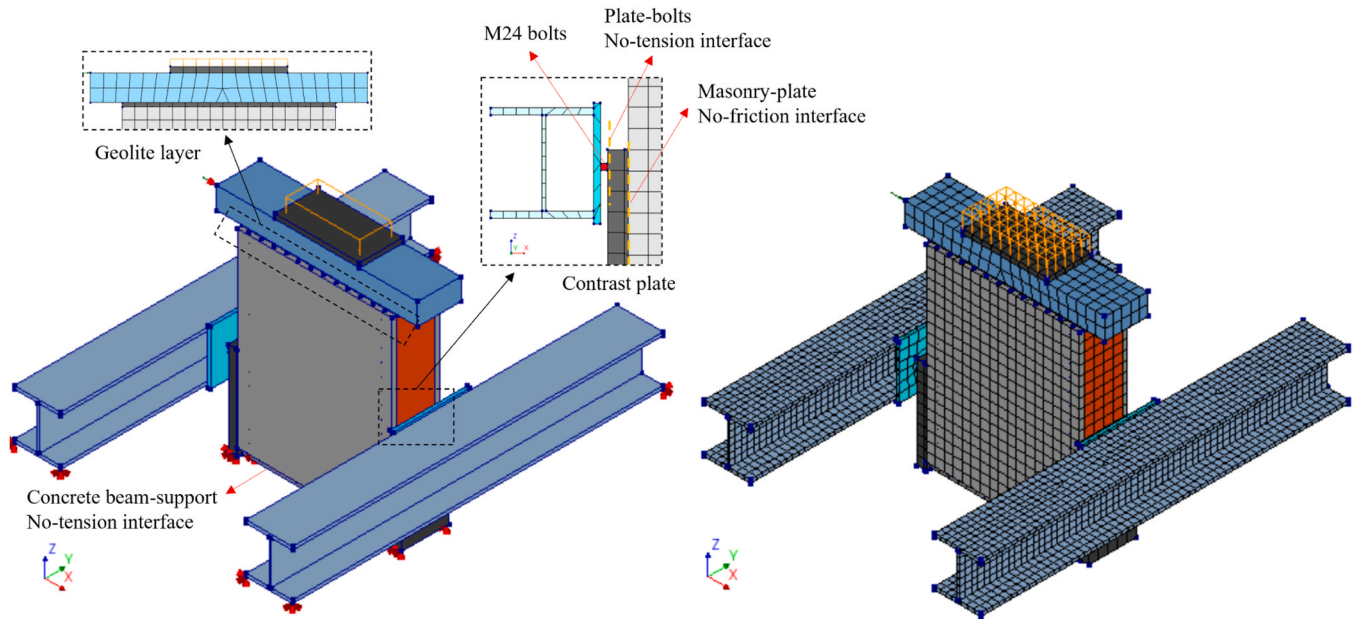


Fig. 6. 3D-model developed to reproduce shear-compression tests. Extrude view on the left, mesh on the right.

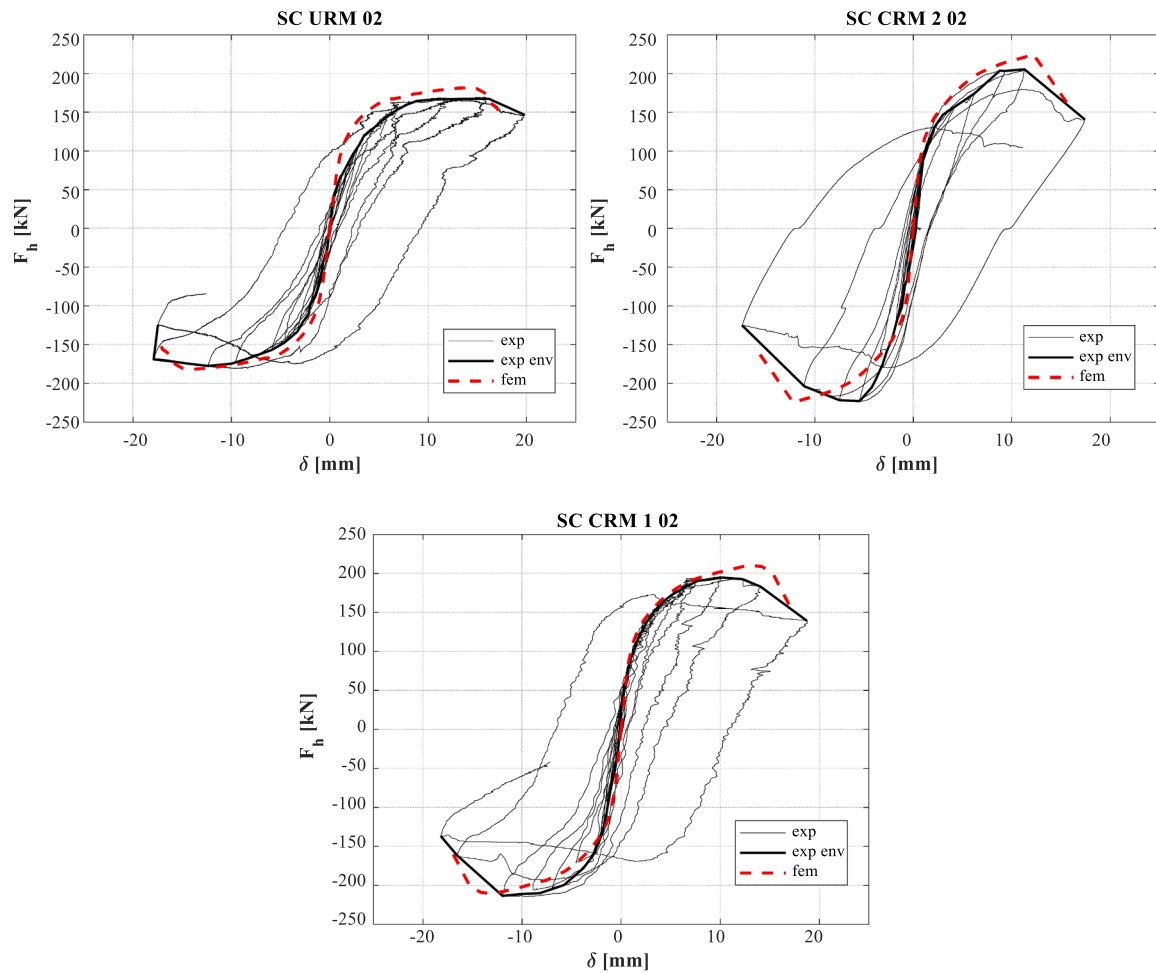


Fig. 7. Comparison between experimental (black lines) and numerical capacity curves (dashed red lines).

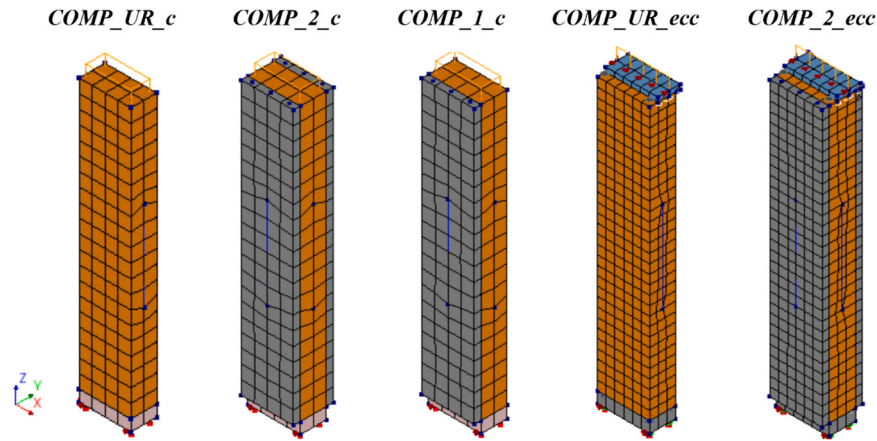


Fig. 8. Numerical simulations for the calibration of uniaxial compression tests.

CRM under centered and eccentric compression, since corresponding experimental results are not available. For centrally loaded specimens, a uniformly distributed vertical load was applied to the masonry top surface, excluding the plaster layer, while the concrete base beam was fully restrained in translation. This modelling choice was made, consistently with the experimental setup, considering that in real applications CRM systems are typically installed on masonry already subjected to compressive stresses due to self-weight and permanent actions and that additional vertical loads are generally transferred directly through the floors to the masonry, while the strengthening layer mainly develops shear stresses at the masonry–mortar interface. For eccentrically loaded columns, the axial force was applied as a line load with an eccentricity of $t_m/4$, being t_m the wall thickness, and laterally constrained to reproduce the experimental setup. Nonlinear static analyses were performed in load control. Table 2 compares numerical and experimental results. The control parameters were the peak load, P_u , and the elastic modulus, E_m , obtained from the stress-strain response. Axial stresses were calculated by dividing the applied load by the masonry cross-section, and E_m was evaluated at one-third of the ultimate load.

The comparison between numerical and experimental results confirmed the reliability of the model in reproducing both strength and stiffness under uniaxial loading. The close agreement in elastic moduli further validated the calibration of the mortar-masonry interface stiffness. Under eccentric loading, the simulated deformation pattern and failure mode closely matched the experimental observations, including plaster detachment and toe crushing. No significant out-of-plane effects were detected for one-sided reinforcement, while the CRM layers increased axial stiffness up to the onset of debonding. Two additional unreinforced models were analyzed to quantify the effectiveness of CRM reinforcement. Under centred compression the presence of one or two CRM layers produced negligible strength enhancement, primarily due to mortar-masonry debonding associated with the application of compressive load only to the masonry cross-section. Conversely, in the case of eccentric loading, the presence of the CRM layer on the tension side significantly restrains the out-of-plane buckling of the masonry, providing a strength amplification factor of 1.22.

2.2. Validation of the numerical model

The FE model was calibrated using experimental data to minimize discrepancies between numerical predictions and measured responses. The model was here validated against independent tests from the literature. In particular, the shear-compression experiments by Gattesco et al. [16] were used to evaluate the model's ability to reproduce the in-plane behavior of CRM-strengthened panels, while the uniaxial compression tests by Donnini et al. [9] were used to verify its accuracy under pure compression. To this end, the experimental tests in [9,16] were numerically simulated by reproducing the geometric configuration and the boundary conditions, adopting the material mechanical properties defined within the respective experimental campaigns. In [16], tests were performed on rubble limestone panels sized 1500x2000x350 mm³ strengthened with a 30 mm-thick CRM layer composed by a mortar with a compressive strength of $f_{c,pl} = 19$ MPa and a Young modulus equal to $E_{pl} = 18$ GPa. Masonry average compressive strength and elastic modulus were derived through compression tests and were equal to $f_{c,m} = 4.50$ MPa and $E_m = 2430$ MPa. Two specimens were tested in shear-compression: an unreinforced panel (MSR1) and a two-sided reinforced panel (MSR2). Double-bending static scheme was used, with a compressive ratio $p = 0.20$. The unreinforced panel failed by diagonal cracking, while the reinforced one displayed plaster cracking followed by masonry-mortar detachment. The FE model was developed using the same procedure adopted for the calibrated configuration, with input parameters for masonry and CRM taken from values provided in the experimental work. The paper did not give information about the tensile strength of the plaster, therefore the same ratio $f_{c,pl}/f_{t,pl} \approx 15$ obtained in [17] has been used for the input, leading to a value $f_{t,pl} = 1.27$ MPa. Fracture energy in compression and tension of masonry were set equal to 0.012 J/mm² and 0.0001 J/mm² consistent with the model described for the calibration procedure. The fracture energy of mortar coating was derived using the Model Code 90 formulations. A shear interface was inserted between the mortar and the masonry to reproduce the observed failure mechanism, using for the interface definition the same values of stiffness, fracture energy, and shear

Table 2

Numerical models developed to simulate uniaxial compression tests on masonry columns. Comparison between experimental and numerical peak loads and Young moduli.

Model	Eccentricity [mm]	Exp. results	$P_{u,fem}$ [kN]	$P_{u,exp}$ [kN]	$P_{u,fem}/P_{u,exp}$ [-]	$E_{m,fem}$ [MPa]	$E_{m,exp}$ [MPa]	E_{fem}/E_{exp} [-]
COMP_UR_c	0	no	295,2	-	-	4848	-	-
COMP_2_c	0	yes	298,6	296,1	1,01	6241	5915	1,06
COMP_1_c	0	yes	295,1	287,1	1,03	5603	5980	0,94
COMP_UR_ecc	28,75	no	157,9	-	-	4844	-	-
COMP_2_ecc	28,75	yes	192,8	190,9	1,01	7480	7677	0,97

strength adopted in the calibrated model (Table 1). The experimental setup was reproduced by modelling the concrete beams located above and below the wall, and the steel beam used to apply the horizontal and vertical loads was represented by a rigid steel prism. Linear elastic behaviour was assumed for both concrete and steel, with elastic moduli of 30 GPa and 210 GPa, respectively. In the strengthened model, the CRM layer was left unconnected to the upper and lower concrete beams, reproducing the reinforcement discontinuity at the specimen ends of the experimental tests. This modelling choice allowed the stress concentrations developing at the masonry-CRM interface near the reinforcement ends, which promoted the experimentally observed debonding mechanism. Fig. 9 compares the numerical and experimental capacity curves. The good agreement confirms the model ability to reproduce both stiffness and peak strength under combined shear-compression, although a longer quasi-elastic phase and a more abrupt stiffness degradation are observed in the FEM results. This difference can be attributed to the use of average values of compressive strength and elastic modulus of the masonry experimentally derived in [16]. In contrast, the experimental response in shear-compression corresponds to a specific specimen, for which the actual mechanical properties are not available. The simulated failure mechanisms also matched the experimental observations: diagonal cracking and toe crushing in MSR1, and diagonal cracking of the plaster followed by interface debonding in MSR2. Focusing on compression response, in Donnini et al. [9], experimental compression tests were carried out on brick masonry panels with dimension $1000 \times 250 \times 1200 \text{ mm}^3$ characterized by a strength and elastic moduli equal to $f_{c,m} = 7.89 \text{ MPa}$, $E_m = 9670 \text{ MPa}$. Strengthening consisted of two 30 mm TRM layers made of glass fabric in lime mortar, connected by stainless steel helical bars with a nominal diameter of 8 mm passing through the masonry and folded over the glass fabric for a length of 100 mm. The mortar coating was characterized by a compressive strength equal to 8.35 MPa and an elastic modulus equal to 12320 MPa. The tensile strength of mortar was derived from the axial stress reached at the formation of the first crack during the tensile test on CRM coupons obtaining a value equal to $f_{t,pl} = 1.66 \text{ MPa}$. Compression was applied only to the masonry core by using neoprene sheets, leaving the TRM layers unloaded. The numerical stress-strain curves showed good agreement with the tests. Both experimental and numerical results confirmed that TRM provided negligible improvements in compressive strength and stiffness, as failure was governed by masonry crushing and mortar-masonry detachment. Moreover, it should be noted that the elastic modulus of the tested bare masonry is significantly higher than the values typically reported for solid brick masonry. Consequently, the increase in stiffness provided by the CRM system, relative to the

unstrengthened specimens, may have been less pronounced. Table 3 reports the experimental ultimate load, compressive strength, and elastic modulus compared with the corresponding FE predictions.

3. Parametric numerical simulations

In this section, the previously described FE model is employed to conduct a parametric study aimed at investigating the response of masonry walls under compression and shear-compression loading by varying the reinforcement configuration, the mechanical and geometrical properties of both the substrate and the CRM system.

3.1. Preliminary consideration: CRM continuity

The effectiveness of CRM in improving the load-bearing capacity of masonry walls is strongly dependent on boundary conditions and on the continuity of the plaster layer at wall edges [9,32].

In real applications, this continuity is generally ensured at the top of masonry piers adjacent to openings, due to the presence of spandrels and rigid nodes that allow the reinforcement to remain continuous across structural elements. Conversely, continuity at the base of the walls may not be always guaranteed, as CRM may be interrupted at the inter-storey level by floor slabs or at the foundation level. When the strengthening system is properly detailed, extending through floors and into the foundation with adequate anchorage, full continuity can also be achieved in these regions. In this work, three possible end-section conditions were identified, as schematically illustrated in Fig. 10: (i) discontinuity at both tensile and compressed corners (isolated piers or weak spandrels with no connection across the floor), (ii) continuity in compression but discontinuity in tension (piers connected at the base to stiff floors such as RC slabs, where the CRM layer bears against the slab without effective tensile continuity), and (iii) continuity at both corners (piers well integrated with strong spandrels or connected at the base foundation). Discontinuous CRM can generate large tangential stresses at pier corners and reduces the tensile contribution of vertical fibers, whereas continuous CRM enables full composite action under both tension and compression. Intermediate configurations also arise in real structures, for example, walls with openings where CRM continuity may be interrupted at the base for the presence of the floor but preserved at the top thanks to the presence of lateral spandrels. To evaluate the influence of CRM continuity on seismic response, a two-story wall with three piers and two openings per level was modelled (Fig. 11). In this modeling approach, the spandrel elements were modeled using the same constitutive laws calibrated for the masonry piers. Three configurations were analyzed: unreinforced, CRM-reinforced with continuous end sections, and CRM-reinforced with discontinuous end sections. CRM layers were 30 mm thick on both faces and incorporated an $80 \text{ mm} \times 80 \text{ mm}$ GFRP grid. Mechanical properties of masonry, mortar, and reinforcement matched those used in the experimental calibration and Italian Building Code [24] specifications for existing brick masonry ($f_{c,m} = 4.3 \text{ MPa}$, $E_m = 2000 \text{ MPa}$). Wall geometry was: $B_p = 1500 \text{ mm}$, $B_o = 1000 \text{ mm}$, $H_f = 3200 \text{ mm}$, $t_m = 250 \text{ mm}$, with symbols referenced in Fig. 11. The axial load was chosen to obtain a compressive ratio $p \approx 0.30$ for each pier. Fig. 12 presents the distribution of principal cracks at the end of the analysis for the three configurations, while Fig. 13 shows the normal stresses in the GFRP grid. The results show that the unreinforced wall failed through diagonal cracking of the piers at the ground floor and combined shear-bending mechanisms in the ground-floor spandrels (Fig. 12a), with an ultimate drift $\theta_u \approx 0.30\%$, typical of shear-dominated unreinforced masonry. CRM reinforcement significantly increased performance: shear strength rose by 95% with continuous CRM and by 56% with discontinuous CRM, while ultimate displacement increased by 75% and 40%, respectively. Moreover, the crack width at the end of the analysis is significantly reduced thanks to the presence of the strengthening system (Fig. 12). The reduced effectiveness of discontinuous CRM was mainly governed by the absence of anchorage at the base, which

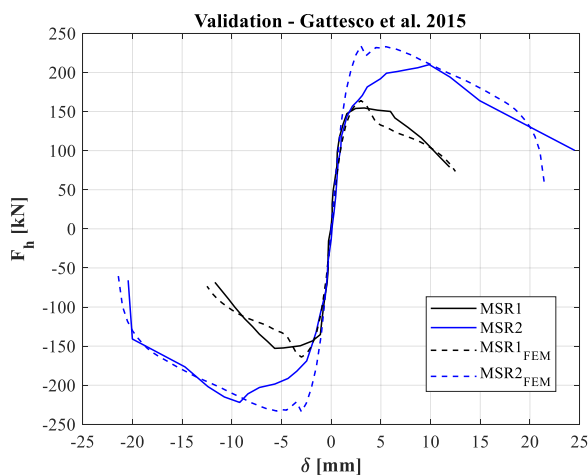
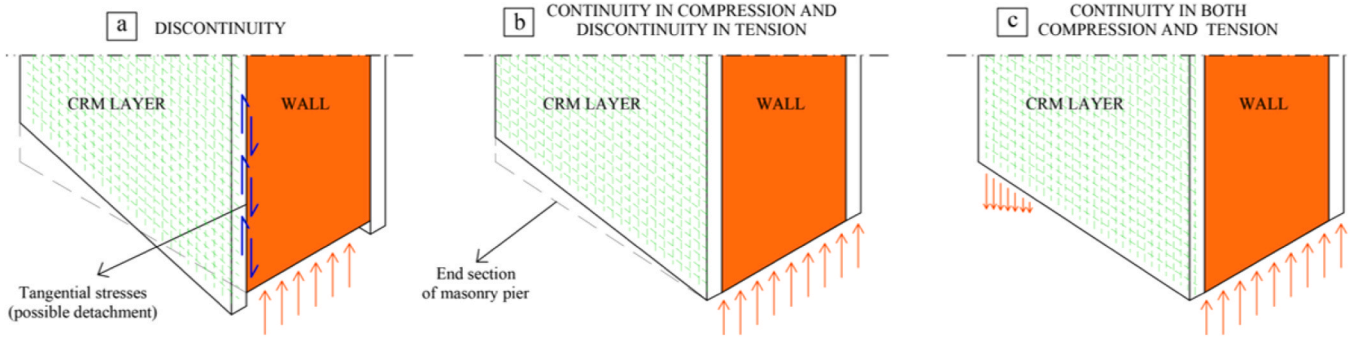
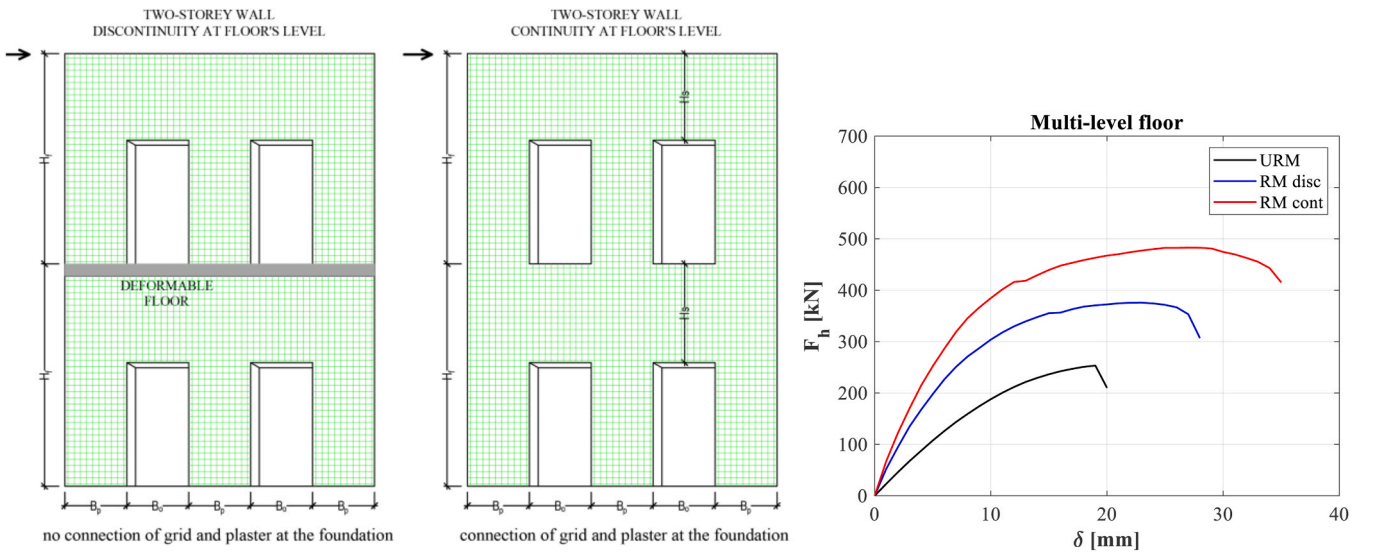
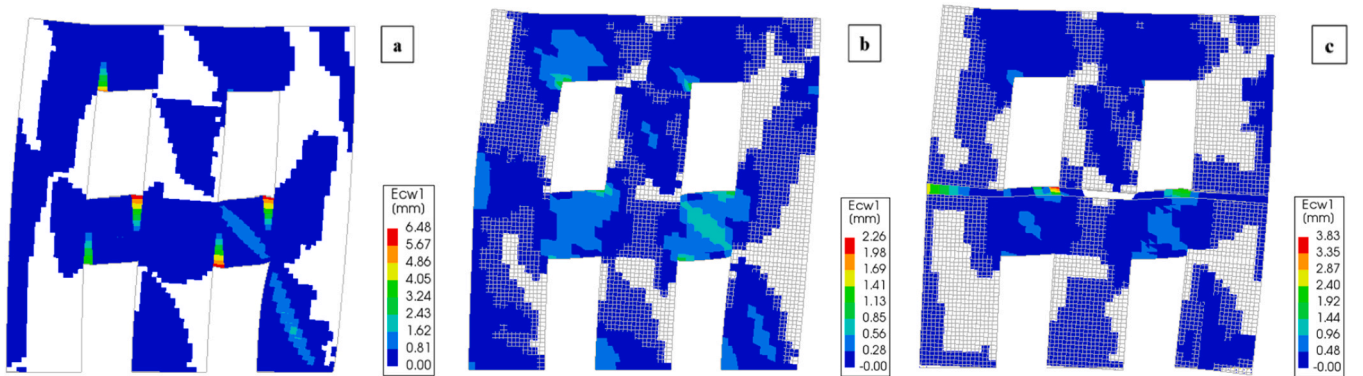


Fig. 9. Comparison between capacity curves of MSR1 (unreinforced) and MSR2 (reinforced) walls obtained experimentally (Gattesco et al. 2015) and numerically (present simulations).

Table 3

Validation of FE model by comparing numerical and experimental (Donnini et al. 2021) compression tests.

	$P_{max,exp}$ [kN]	$P_{max,AVG,exp}$ [kN]	$P_{max,num}$ [kN]	$f_{m,AVG,exp}$ [MPa]	$f_{m,num}$ [MPa]	$f_{m,AVG,exp}/f_{m,num}$ [-]	$E_{m,exp}$ [MPa]	$E_{m,AVG,exp}$ [MPa]	$E_{m,num}$ [MPa]	$E_{m,AVG,exp}/E_{m,num}$ [-]
MC1	1781	1972	1998	7.89	7.99	0.98	9630	9670	9500	1.02
MC2	2163						9710			
MC1_R	1676	2018	2080	8.07	8.32	0.97	8290	9095	10050	0.91
MC2_R	2360						9900			

**Fig. 10.** Boundary conditions of a CRM reinforced panel: a. discontinuity at the corners; b. continuity in the compressed corner thanks to the contrast offered by the floor; c. continuity in both tensile and compressed sides.**Fig. 11.** Shear-compression response of a multi-story wall by varying the boundary conditions.**Fig. 12.** Distribution of principal crack widths (E_{cw1}) in unreinforced (a), continuous CRM (b) and discontinuous CRM (c) configurations.

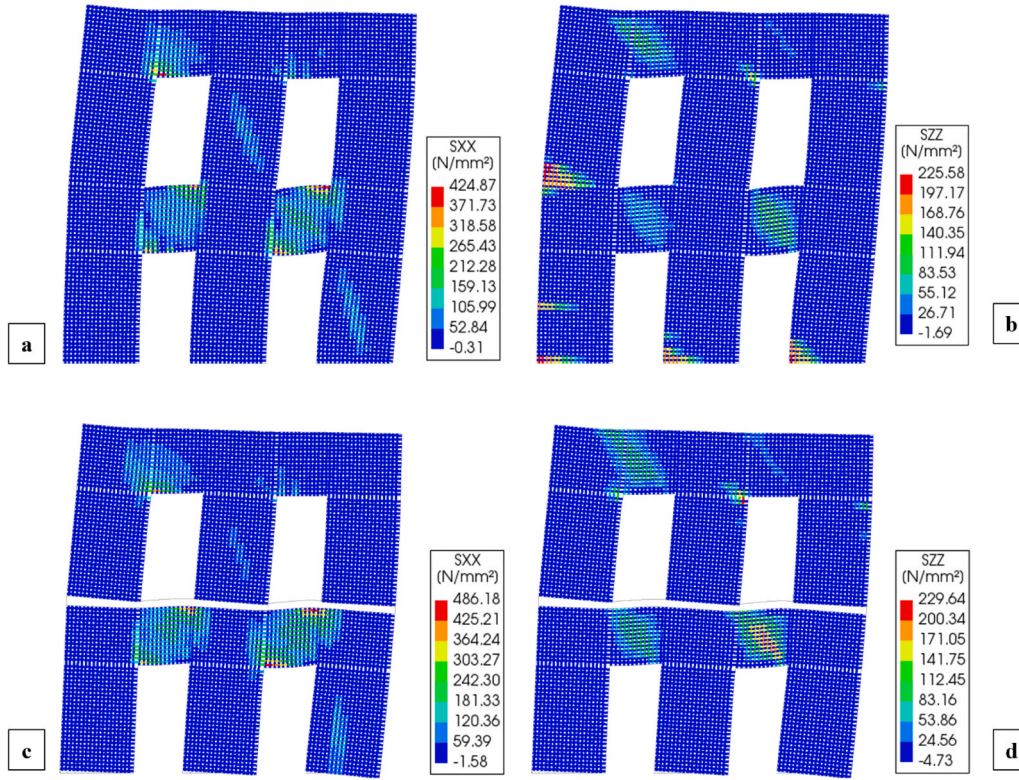


Fig. 13. Horizontal (S_{xx}) and vertical (S_{zz}) normal stresses in GFRP grid in case of continuous (a, b) and discontinuous (c, d) configurations.

limited the contribution of vertical fibers and the flexural capacity of piers, as can be seen in Fig. 13 b-d, where it can be observed that the vertical stresses in the reinforcement mesh at the base of masonry piers are negligible in the discontinuous case, whereas they are significant in the continuous one. Conversely, the shear behaviour of the piers and the flexural response of the spandrels were only marginally affected by CRM continuity along the height, as the continuity of the strengthening system is in any case ensured at the center of the panels and at the spandrel-pier joints. The full-scale simulations demonstrate that CRM anchorage has a marked influence on the global seismic capacity of masonry walls.

3.2. In-plane shear-compression response

The effect of CRM on the seismic response of masonry panels is investigated by modeling walls with dimensions 1500 mm $\times$ 2000 mm

$\times t_m$, with thickness t_m set as a variable. Nonlinear static analyses under shear-compression were performed. CRM was applied on one or both faces, with initial simulations assuming discontinuity at panel edges and additional analyses considering fully anchored CRM to compare continuous and discontinuous layouts. The parametric study varied compression ratios $p = \sigma/f_{c,m}$, masonry types (B = brick, T = tuff, S = uncut stone, EB = experimental brick tested in [17]), wall thickness (120–400 mm), interface shear strength (0.12–0.48 MPa), and the presence of CRM (UR , $R1$, $R2$), resulting in 360 simulations. The fracture energy values in compression and tension for the masonry were assumed to be 0.012 J/mm² and $G_c/120$, respectively, consistent with the value obtained through the calibration and validation process. Horizontal force-displacement response was obtained for RM and URM walls. Capacity curves were schematized by two-piece linear curves, with secant stiffness K at 70% of peak shear V_{max} , ultimate displacement δ_u at 20% load reduction, and shear strength V_u at the peak load (see Fig. 14).

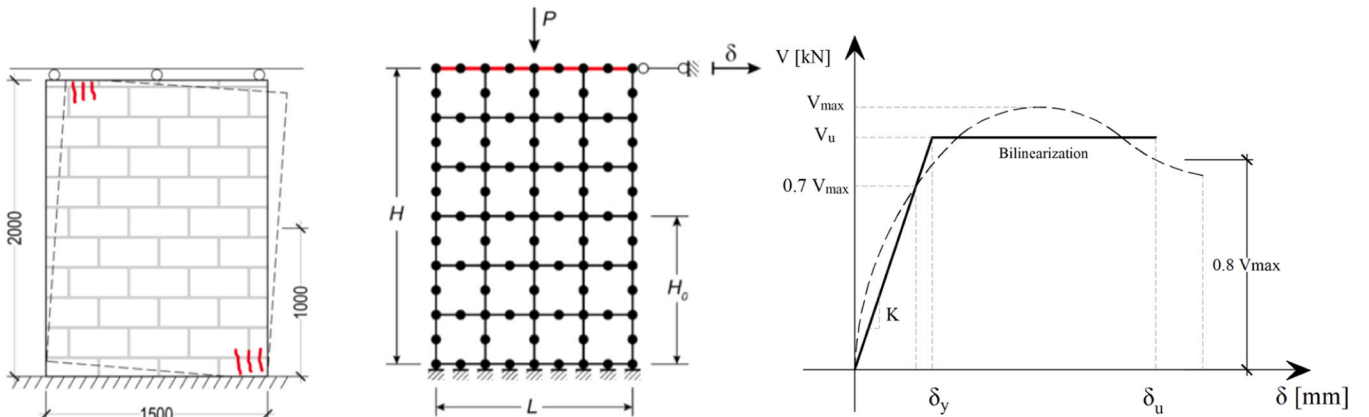


Fig. 14. Static-scheme, FE model and capacity curve of walls modelled for parametric simulations.

Table 4

Specimens used for the parametric analysis: geometrical characteristics, compression level, interface strength.

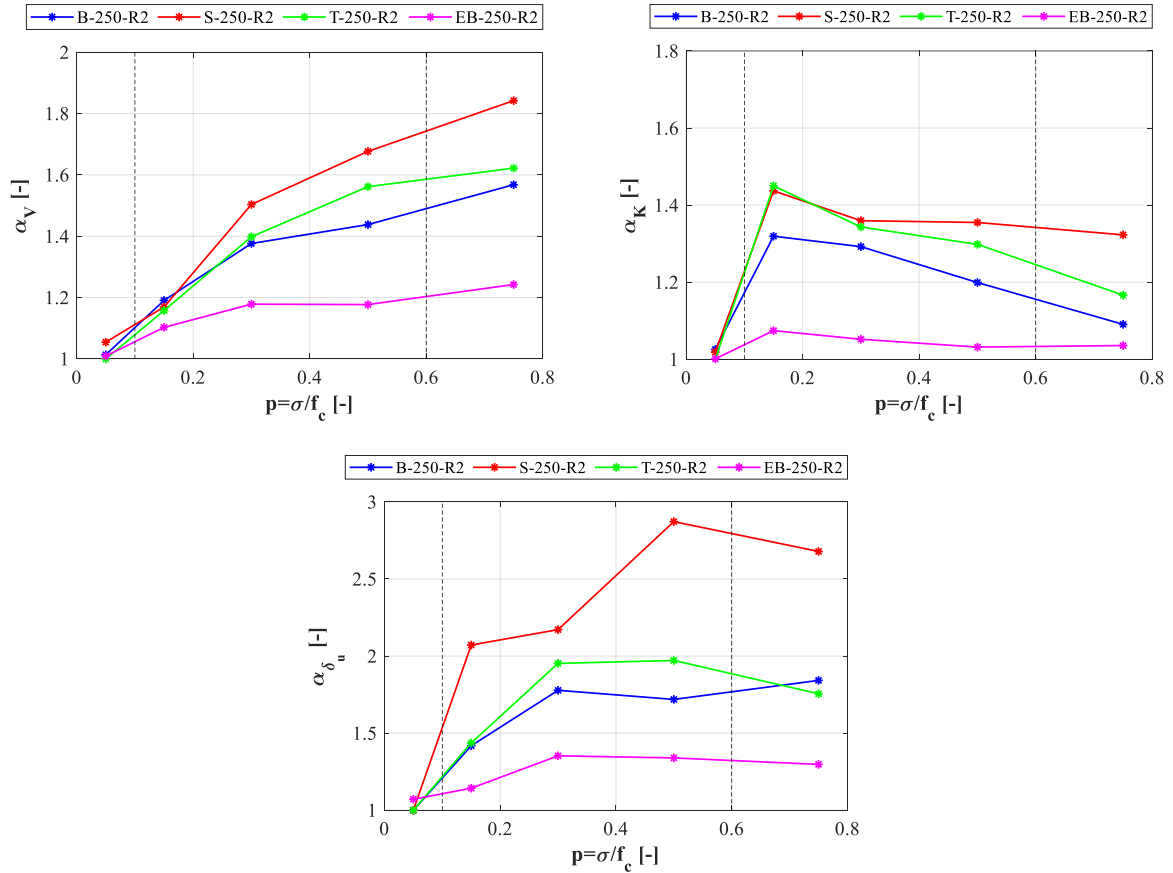
Masonry typology	Reinforcement	Compression level $p=\sigma/f_{c,m}$	Panel thickness t_m [mm]	Interface strength τ_{max} [MPa]
Existing brick (B)	URM	0.05–0.15–0.30–0.50–0.75	120–250	0.12–0.24–0.36–0.48
	CRM 1 side			
	CRM 2 sides			
Soft stone (T)	URM	0.05–0.15–0.30–0.50–0.75	250–400	0.12–0.24–0.36–0.48
	CRM 1 side			
	CRM 2 sides			
Uncut stone (S)	URM	0.05–0.15–0.30–0.50–0.75	250–400	0.12–0.24–0.36–0.48
	CRM 1 side			
	CRM 2 sides			
Experimental brick (EB)	URM	0.05–0.15–0.30–0.50–0.75	120–250	0.12–0.24–0.36–0.48
	CRM 1 side			
	CRM 2 sides			

Table 5

Specimens used for the parametric analysis: mechanical properties of materials implemented in the models.

	E_m [MPa]	$f_{c,m}$ [MPa]	$f_{t,m}$ [MPa]	τ_o [MPa]
Existing brick (B)	2000	4.3	0.04	0.14
Tuff stone (T)	1400	3.0	0.03	0.11
Uncut stone (S)	1440	2.0	0.02	0.065
Experimental brick (EB)	4000	14.0	0.14	0.42
Mortar coating CRM	10000	15.0	0.89	-

Amplification coefficients for stiffness $\alpha_k = k_R/k_{UR}$, strength $\alpha_V = V_{u,R}/V_{u,UR}$ and displacement $\alpha_\delta = \delta_{u,R}/\delta_{u,UR}$ were defined as the ratio between RM and URM walls, quantifying efficiency in performance. Table 4 summarizes the characteristics of the specimens adopted for the analyses, while Table 5 shows the mechanical properties of the materials used in the numerical modeling. It should be noted that the same masonry constitutive model is adopted in all analyses, namely the Total Strain Crack Model. These properties were either obtained from experimental results or derived from the mean strength and stiffness values for standard masonry defined by the Italian building code [24].

**Fig. 15.** Amplification coefficients for strength, stiffness, and ultimate displacement as a function of masonry typology and compression level in the case of discontinuous CRM.

To interpret the results of the parametric analysis, it is first necessary to recall that the seismic response of masonry walls is strongly governed by the compressive stress level. For the wall geometry and boundary conditions adopted in the simulations, three failure mechanisms are observed as a function of the compressive ratio p : rocking ($p < 0.10$), shear ($0.10 \leq p \leq 0.60$), and toe-crushing ($p > 0.60$). Regarding shear failure, since the Total Strain Crack Model adopted in the simulations does not explicitly require the parameter τ_o , its values for the different masonry typologies (reported in the last column of Table 5 and used in the subsequent evaluations) were indirectly calibrated based on the numerical results. Specifically, τ_o was determined by minimizing the difference between the shear strength of URM panels obtained numerically and that predicted by the analytical formulation for diagonal cracking usually adopted by European standards [24], namely $V_{u, \text{shear}} = \frac{L t_m - 1.5 \tau_o}{b} \sqrt{1 + \frac{\sigma}{1.5 \tau_o}}$, for panels governed by shear failure ($0.10 \leq p \leq 0.60$). This procedure resulted in a maximum discrepancy of approximately 5% between numerical and analytical values. It should also be noted that the present study addresses shear behaviour only with reference to the diagonal cracking mechanism, while sliding shear failure is not considered, as it would require different numerical and analytical approaches, based on micro-modelling simulations accounting for the presence of mortar joints [33] and warrants further investigations. Fig. 15 shows the amplification coefficients of strength, stiffness, and ultimate displacement as functions of the compressive ratio and masonry typology, assuming discontinuity of the CRM reinforcement at the edges, for panel with thickness of 250 mm and CRM applied on both sides. The vertical dashed lines indicate the failure-mode boundaries for unreinforced walls. The effectiveness of CRM in increasing strength and deformation capacity grows with p and

is more evident in weaker masonry. The α_δ parameter for stone masonry increases markedly at higher axial load ratios ($p = 0.50$ and 0.75) compared to other masonry typologies. This trend is attributable to the low compressive strength of such masonry, which significantly restricts displacement capacity under elevated axial loads due to crushing. Consequently, under these conditions, the strengthening intervention proves more effective in enhancing ultimate displacement. For rocking-dominated panels, the benefit is negligible due to the assumed lack of reinforcement connections at the edges. For shear and toe-crushing failure modes, CRM significantly enhances lateral stiffness at low compression levels, whereas its influence decreases with increasing p .

Fig. 16 shows the trend of α_i by varying the ratio between the URM panel's mechanical parameters multiplied by the thickness and the tensile strength or stiffness of the plaster multiplied by its thickness. In particular, the coefficient $\omega = \tau_o t_m / f_{t, pl} t_{pl}$ and the coefficient $\zeta = E_m t_m / E_{pl} t_{pl}$ have been defined. The trend lines plotted in the graphs were calculated excluding the data representing panels characterized by low compression level, namely $p = 0.05$ and $p = 0.15$, since in the hypothesis of discontinuity of CRM the influence is negligible. As expected, the CRM efficiency is greater the lower is the mechanical ratio.

Focusing on the application of the CRM on a single side of the walls, considering the whole set of simulations, one-sided CRM application resulted in an average strength and ultimate displacement increase approximately 40% lower than that obtained with double-sided reinforcement, while the stiffness increasing reduction was found to be on average 62%. The effect of the masonry-to-mortar interface shear strength was also investigated using four different values of $\tau_{\max}$ (0.12–0.48 MPa), taking the calibrated value of 0.24 MPa as a reference. The results show that variations in peak tangential strength have little

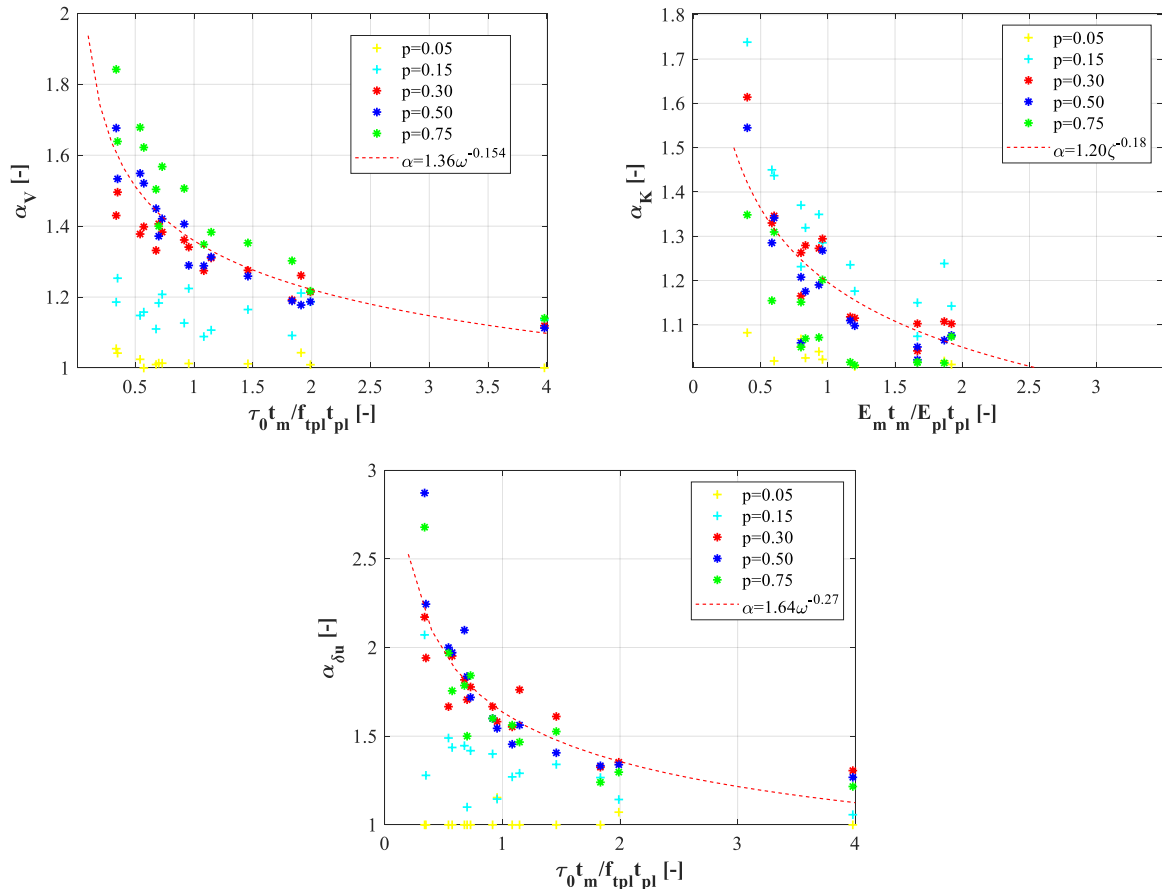


Fig. 16. Amplification coefficients for strength, stiffness, and ultimate displacement as a function of the masonry-to-mortar mechanical ratio in the case of discontinuous CRM.

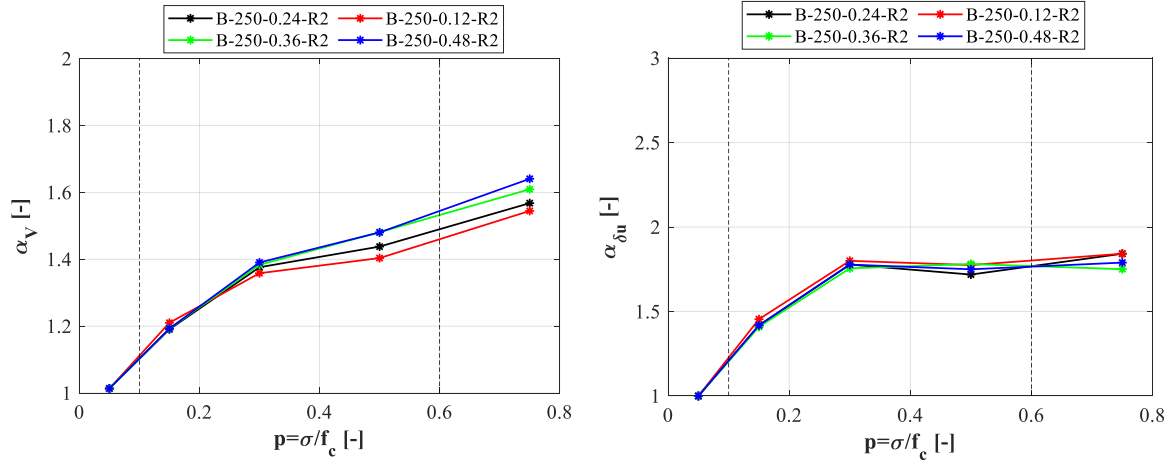


Fig. 17. Effect of interface shear strength τ_{max} on α_v and $\alpha_{\delta u}$ for brick masonry.

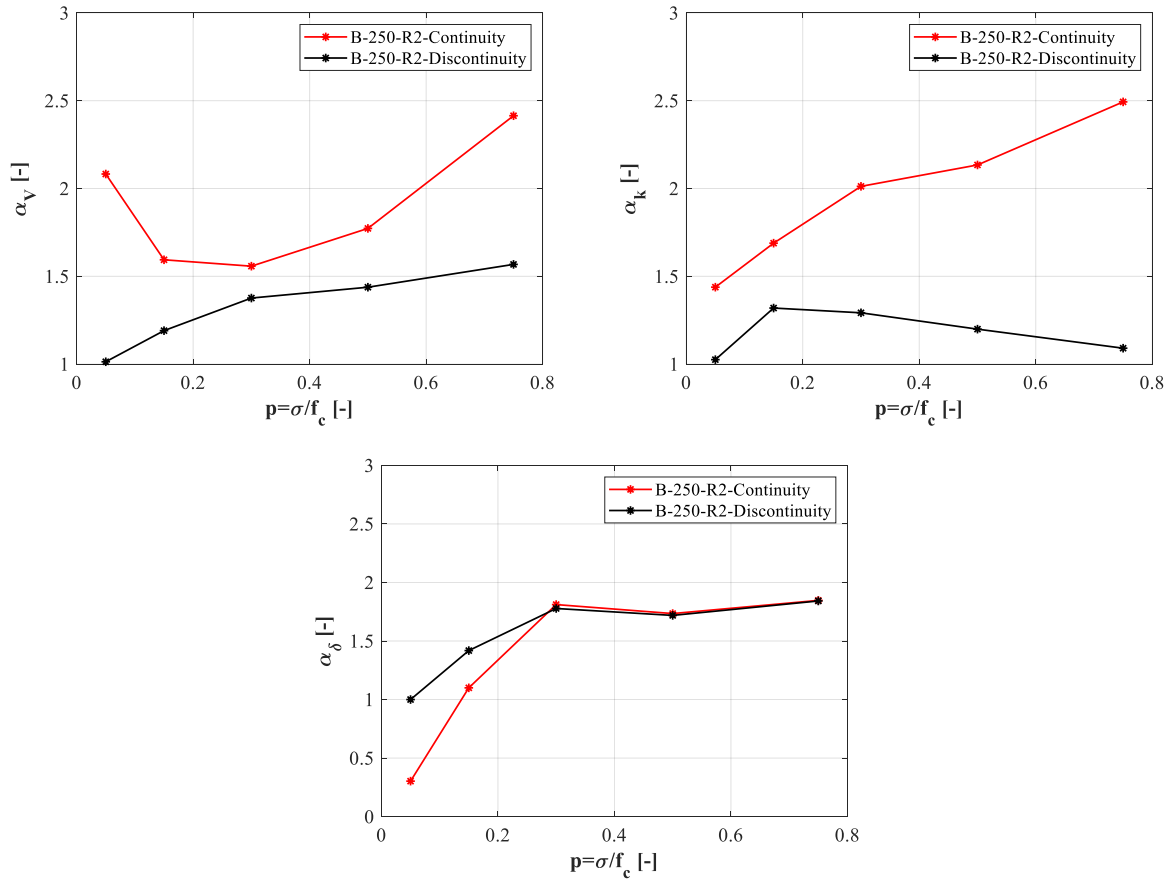


Fig. 18. Trend of strength, stiffness and ultimate displacement amplification factors in case of continuity (red lines) and discontinuity (black lines) of the CRM at the edges.

influence on the response of the panels, especially in terms of strength and displacement capacity. Fig. 17 shows the variation of the coefficients α_v and α_k for solid brick masonry as a function of interface shear strength.

The analysis was also extended to assess CRM effectiveness assuming its continuity at the wall edges, and the results were compared with the discontinuous configuration. In practice, the continuity of the CRM at sections where the mortar coating and the grid are interrupted, typically at floor levels or at the wall–foundation interface, can be ensured by

adopting connection systems between vertically aligned masonry panels. At these discontinuities, force transfer can be provided by discrete or distributed connectors that restore the vertical mechanical continuity of the strengthening system. More specifically, steel or fibre-reinforced polymer bars can be used, arranged either vertically or at an inclination, crossing the floor–wall joint or the wall–foundation interface and anchored by means of cementitious structural grouts or epoxy-based resins, or through mechanical end-anchorage devices. Examples of connection solutions at panel terminations are shown in [19,33,34].

Fig. 18 reports the corresponding strength, stiffness, and displacement amplification factors in the case of brick walls 250 mm thick, with black and red curves denoting discontinuous and continuous CRM layouts, respectively.

Results show that CRM is generally more effective when it is connected at the edges. Thanks to the proper anchorage of the reinforcement at the panel boundaries, the flexural failure mechanism involves the composite masonry–CRM section, with the vertical bars of the system acting as the main reinforcement. For this reason, both strength and stiffness are significantly enhanced in panels governed by flexural failure. At low levels of axial compression ($p = 0.05$), the presence of a properly anchored CRM system prevents rocking leading to a significant increase in flexural strength compared to the discontinuous configuration, whereas the rocking resistance remains unchanged in the absence of grid connections at the base of the panel. At high compressive stress ($p = 0.75$), the system exhibits hybrid shear–flexural behavior involving toe–crushing and diagonal cracking: in this range, both CRM configurations significantly enhance strength due to the mortar contribution in limiting crack opening and confining the end regions. For shear-dominated panels ($0.10 < p < 0.50$), edge continuity has a smaller influence on the strength, as shear resistance is mainly controlled by the mortar tensile capacity restraining diagonal cracking in the panel center. In terms of displacement capacity, CRM substantially increases ultimate drift for medium-to-high compression levels ($p > 0.30$). In this shear-governed regime, the effectiveness is nearly independent of p and of end connections, since drift enhancement stems from improved control of diagonal cracking at the center of the piers and the resulting redistribution of shear stresses. Conversely, for low compressive ratios, continuity markedly affects the response. Edge anchorage inhibits rocking, increasing strength but reducing ultimate drift. As a result, α_δ decreases for $p < 0.30$ and drops below unity for rocking-dominated panels. Regarding the stiffness, the effectiveness of the discontinuous CRM tends to decrease as the axial load increases, since higher compressive stresses reduce the extent of masonry cracking, thereby increasing the stiffness of the unreinforced masonry itself.

3.3. Compression response

The load-bearing capacity of masonry walls under compression is strongly affected by their slenderness and the eccentricity of the applied loads. For stocky walls, failure typically occurs by material crushing, whereas slender walls are more prone to lateral instability. Load eccentricity further promotes buckling and reduces strength. Due to the nonlinear response of masonry, mainly related to its low tensile strength and progressive cracking, the buckling behavior is complex. To address this, both the Italian Building Code [24] and Eurocode 6 [35] provide simplified empirical methods for estimating the ultimate load of walls through reduction coefficients accounting for slenderness and eccentricity effects. In this approach, the wall is modelled as a pinned–pinned column subjected to an axially applied eccentric load. The ultimate axial

load $N_{u,URM}$ is expressed as $N_{u,URM} = \phi \cdot t_m \cdot L \cdot f_{c,m}$ where ϕ is a reduction factor dependent on the slenderness ratio $\lambda = H/t_m$ and the eccentricity coefficient $m = 6e/t_m$. Experimental studies [36,37] have shown that ϕ decreases with increasing slenderness and eccentricity. Regarding CRM reinforced panels, it is essential to recognize that CRM is added to masonry already carrying compressive stresses from self-weight and permanent actions. Additional vertical loads are transferred directly through the floors to the masonry, while the reinforcement primarily develops shear stresses along the masonry–mortar interface. For this reason, the compression behaviour of reinforced panels was here investigated assuming that only the masonry cross-section carries the applied vertical load and the mortar coatings is not directly loaded. To investigate the influence of CRM on the lateral buckling induced by compressive loads, a parametric analysis on RM and URM walls with dimensions of $L \times H \times t_m \text{ mm}^3$ was conducted, where the thickness t_m and the height H of the panels were considered as a parametrical variable, and the length of the panel was set equal to $L = 1500 \text{ mm}$ for each model (see Table 6). Nonlinear static analyses were performed using a double-hinge model consistent with the scheme commonly adopted in technical standards [24,35]. Reinforced walls were strengthened with two CRM layers. A vertical load was applied at the top through a line load with prescribed eccentricity. Horizontal displacements at the load-application line were restrained, whereas rotation and vertical displacement were left free. The base section was hinged with the support applied at the same eccentricity as the top load. Steel plates

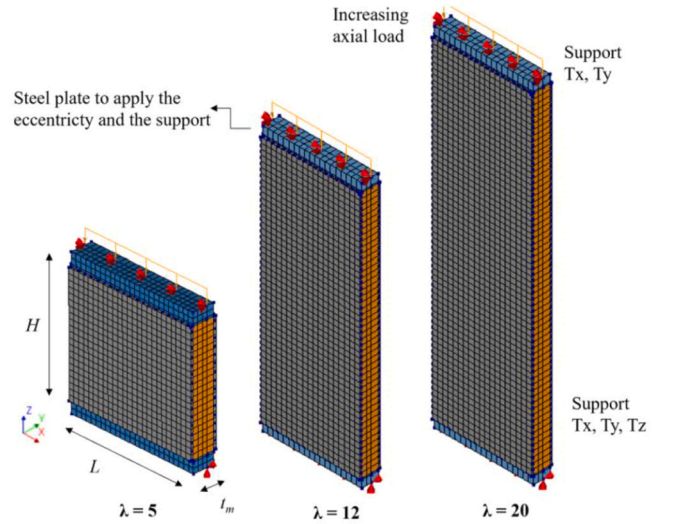


Fig. 19. Modelling of walls under compression: three different values of slenderness (5, 12, 20) and five different values of eccentricity $m = 6e/t_m$ (0, 0.5, 1, 1.5, 2) have been considered.

Table 6

Specimens used for the parametric analysis: wall's geometry and eccentricity of the load.

Masonry typology	Reinforcement	URM thickness t_m [mm]	URM height H [mm]	Slenderness $\lambda = H/t_m$	Eccentricity coeff. $m = 6e/t_m$
Existing brick (B)	- no	250	1250	5	0–0.5–1.0–1.5–2
	- CRM $t_{pl} = 30$	250	3000	12	
		250	5000	20	
Stone (S)	- no	250–400	1250–2000	5	0–0.5–1.0–1.5–2
	- CRM $t_{pl} = 30$	250–400	3000–4800	12	
		-	-	-	
Tuff (T)	- no	250	1250	5	0–0.5–1.0–1.5–2
	- CRM $t_{pl} = 30$	250	3000	12	
		250	5000	20	
Experimental brick (EB)	- no	250	1250	5	0–0.5–1.0–1.5–2
	- CRM $t_{pl} = 30$	250	3000	12	
		250	5000	20	

100 mm thick were applied at the base and top sections of the panels to simulate the eccentric support and to avoid stress concentration due to the application of the line loads. Analyses were conducted under load control using arc-length procedures to capture the post-peak response. The main output parameter was the ultimate axial capacity, N_u . From this, it was possible to evaluate the reduction coefficient of the compressive strength of unreinforced masonry as $\phi = N_{u,URM} / t_m \cdot L \cdot f_{c,m}$ and the amplification coefficient of the compression strength due to the presence of the CRM as $\alpha_N = N_{u,CRM} / N_{u,URM}$. The parametric study considered four masonry typologies, two wall thicknesses, three slenderness ratios, and five eccentricities. The mechanical properties of the unreinforced masonry are the same as those reported in the previous Section 3.2. Table 6 and Fig. 19 summarize the variable parameters, resulting in 145 simulations. The model assumes that the panels behave as double-hinged elements subjected to eccentric compression, with a monolithic response of the masonry that excludes disaggregation or separation between leaves. The CRM reinforcement is applied on both sides of the walls and is not directly loaded at the end cross-sections. Furthermore, the tangential interface between the mortar coating and the substrate is introduced to account for possible debonding, characterized by the calibrated values of tangential strength, fracture energy, and stiffness ($\tau_{max} = 0.24$ MPa, $G_f^I = 0.00036$ J/mm²) representative of the system adopted in the present work.

Analyses were first conducted on URM panels. For each configuration, the peak load $N_{u,URM}$ was derived, and the reduction factor $\phi = N_{u,URM} / t_m \cdot L \cdot f_{c,m}$ due to out-of-plane effects was computed. Fig. 20 compares the EC6 reduction factors (black dotted lines), selected experimental results reported by Sandoval et al. [38,39] and numerical simulations, showing the good agreement between experimental and numerical predictions. In Fig. 20a, the comparison refers to the uniaxial compression tests presented by Sandoval et al. [38]. The hinge-hinge static scheme of the panel was adopted in the experimental tests. The masonry used in the experimental campaign conducted in [38] was characterized by a ratio E_m/f_m of approximately 244; therefore, the experimental results were compared with the numerical simulations performed on experimental brick masonry, for which $E_m/f_m = 4000/14 \approx 285$ (see Table 5, showing the input parameters of numerical simulations). A similar comparison is shown in Fig. 20b with reference to the experimental results reported by Kirstchig et al. [39]. In

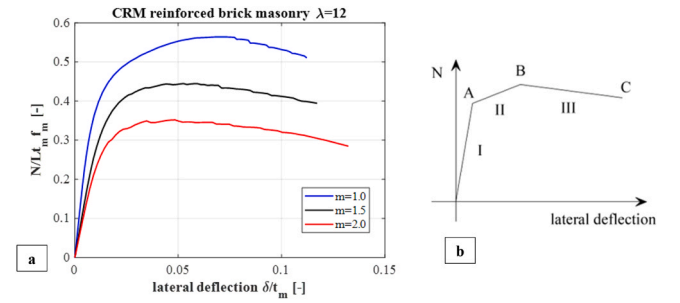


Fig. 21. a. Numerical response of the CRM reinforced brick masonry under compression for $\lambda = 12$ and three values of eccentricity; b. idealized behaviour of 2-sided CRM reinforced wall.

this case, the masonry exhibited an E_m/f_m ratio of about 700; accordingly, the comparison was carried out considering the numerical results obtained for stone masonry, characterized by a ratio $E_m/f_m = 1440/2 = 720$. Despite the slight differences in E_m/f_m ratios, a satisfactory agreement in the trend of the ϕ coefficient between experimental observations, EC6 predictions and numerical results is obtained in both cases.

Focusing on CRM reinforced walls, the study of the evolution of the compressive-buckling behaviour of a panel subject to an eccentric load was carried out by monitoring the step-by-step numerical response, by plotting the trend of the compression load as the horizontal displacement measured at the mid-height of the panel, namely the lateral deflection δ . In Fig. 21, the response of the brick masonry wall characterized by a slenderness equal to $\lambda = 12$ is plotted in terms of dimensionless deflection δ/t_m to dimensionless compressive load $N/(L \cdot t_m \cdot f_m)$, considering three values of eccentricity. Fig. 22 shows the evolution of failure mechanism during the FE analysis. As expected, the higher the eccentricity, the lower the load-bearing capacity and the greater the lateral deflections. A trilinear response can be recognized for CRM-reinforced panels (Fig. 21b and Fig. 22): (i) Phase I, Linear: eccentric loading produces high compression and interface shear stresses near the loaded edges. Debonding initiates when the interface stress reaches τ_{max} (point A), while masonry stresses remain below $f_{c,m}$. (ii) Phase II, Nonlinear: debonding of the compressed CRM layer propagates upward.

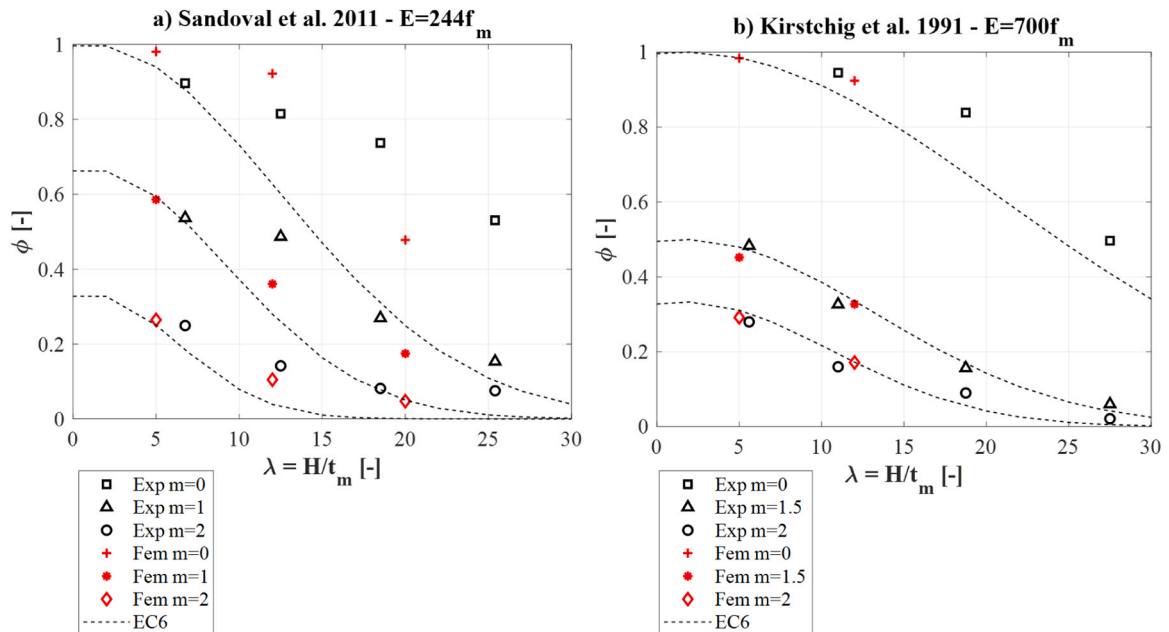


Fig. 20. Comparison between reduction coefficients ϕ derived from the numerical model in the present study with experimental results reported in Sandoval et al. 2011 (a) and Kirstchig et al. 1991 (b).

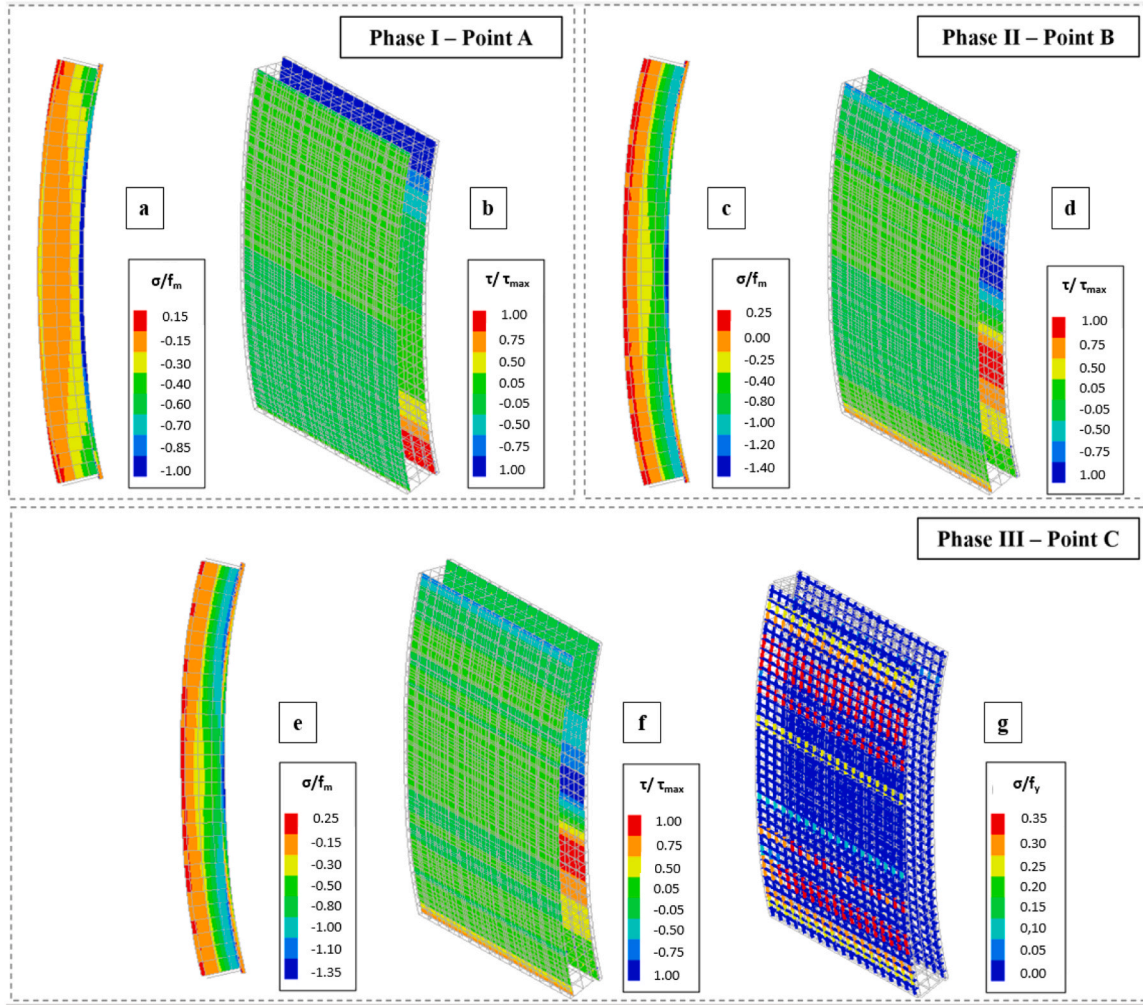


Fig. 22. Numerical response of the panel under compression. a. Compression stresses in the masonry and plaster at the end of phase I. b. Tangential stresses in the masonry-mortar interface at the end of phase I. c. Compression stresses at the end of phase II. d. Tangential stresses in the interface at the end of phase II. e. Compression stresses at the end of phase III. f. Tangential stresses in the interface at the end of phase III. g. Tensile stresses in GFRP mesh at the end of phase III.

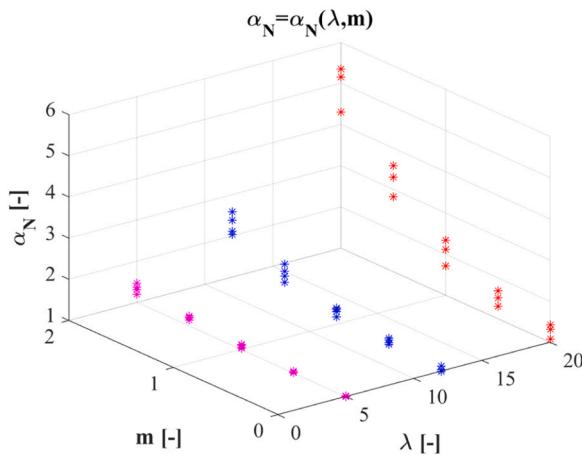


Fig. 23. Trend of amplification factors α_N by varying the load eccentricity and the slenderness of the walls.

Interface shear shifts toward mid-height as end stresses decrease according to the bilinear bond-slip law. Phase II ends at point B, when compressive stresses at both ends reach $f_{c,m}$. (iii) Phase III, Softening: lateral deflection increases with a slight load drop. Out-of-plane

buckling is restrained by the tensile-side CRM. The response ends at point C with masonry crushing.

The numerical behaviour aligns well with the experimental evidence presented in [17]. Although a direct comparison is not possible due to the different boundary condition adopted in the experimental setup, where the base section of the panel was not free to rotate but instead fully supported on the testing machine, the observed experimental failures are consistent with the trilinear response. Indeed, in the centred-compression test, a progressive detachment of the CRM from the masonry initiated at the panel ends. Under eccentric loading, debonding first occurred in the plaster on the compressed side, starting from the end section, while the plaster on the tensile side remained bonded until final failure by masonry crushing in the upper region.

The CRM efficiency in enhancing compression strength was assessed through a parametric study investigating the trend of the amplification factor α_N , defined as the ratio between the ultimate load of the reinforced wall, $N_{u,R}$, and the peak load of the unreinforced wall, $N_{u,URM}$. Fig. 23 plots the trend of α_N for the five eccentricities and the three slenderness ratios implemented in the parametric investigation. The results show a clear dependence of CRM effectiveness on out-of-plane behaviour: amplification increases with eccentricity, and for a given eccentricity, it increases with slenderness. Values of $\alpha_N \approx 1$ were obtained for centred loading in squat and moderately slender panels, indicating limited CRM contribution. The influence of masonry typology has been also investigated by monitoring the trend of α_N as a function of

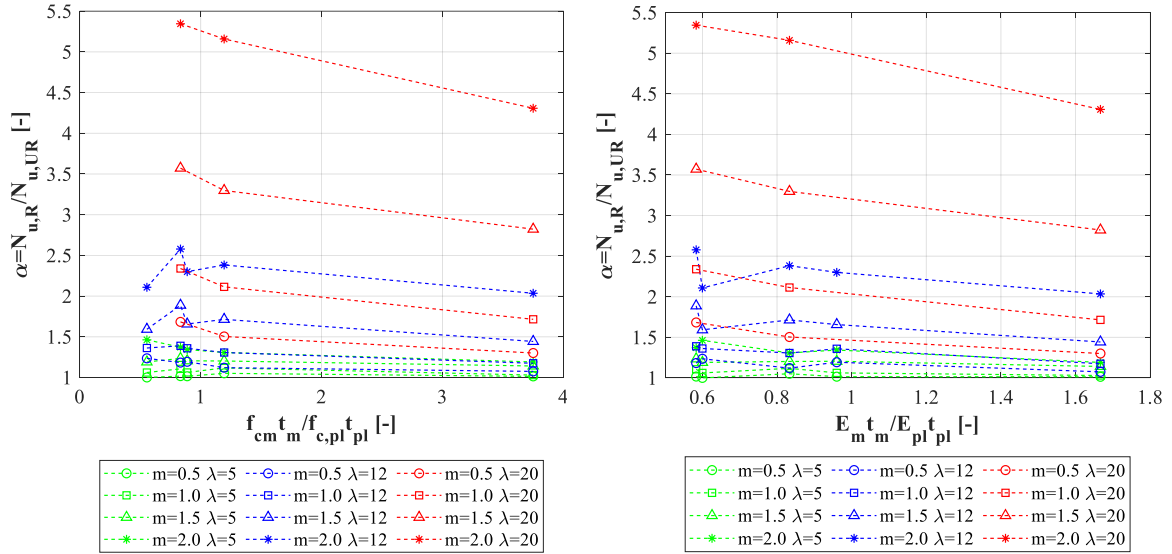


Fig. 24. Influence of masonry typology on the CRM efficiency.

the strength ratio and the stiffness ratio between the masonry and the CRM plaster. The curves $\alpha_N - f_{c,m}t_m/f_{c,pl}t_{pl}$ and $\alpha_N - E_m t_m/E_{pl}t_{pl}$ are shown in Fig. 24. For slenderness values of $\lambda = 5$ (green lines) and $\lambda = 12$ (blue lines), the curves are characterized by a pseudo-horizontal trend as the strength ratio and stiffness ratio increase, showing that the influence of the masonry mechanical properties is reduced. On the other hand, the strength and stiffness of the walls exhibit a greater influence for panels characterized by a higher aspect ratio, $\lambda = 20$ (red lines), because of the influence of masonry tensile strength in the buckling phenomenon. In any case, the mechanical properties of the masonry appear to play a secondary role in governing the effectiveness of CRM in the compressive strength and lateral stability.

4. Proposal of analytical formulations

Given the increasing use of CRM systems, the development of standardized analytical procedures for assessing the strength, stiffness, and displacement capacity of reinforced panels has become essential. Based on the previously parametric investigation, this chapter introduces new analytical formulations for predicting the shear-compression response of CRM-reinforced masonry piers, encompassing flexural strength, shear capacity, and displacement demand, for both discontinuous and continuous CRM layouts. In addition, an amplification coefficient for uniaxial compression strength is proposed to account for the beneficial effect of CRM in reducing lateral buckling.

4.1. Shear-compression response

The development of the analytical procedure begins with the definition of a bilinear force-displacement response for masonry walls. Within this formulation, the capacity curve of an individual pier is described by only three parameters: a Timoshenko beam stiffness, a lateral resistance governed by the minimum between flexural and shear capacity, and a displacement capacity dependent on the axial load level and the panel aspect ratio. Consistent with established approaches [24], the lateral strength is obtained as the minimum between the flexural and shear resistance, evaluated through the standard expressions $V_{u,flex} = \sigma_0 L^2 t_m / 2H_0 (1 - \sigma_0 / 0.85 f_{c,m})$ and $V_{u,shear} = f_{t,m} L t_m / b \sqrt{1 + \sigma_0 / f_{t,m}}$. The effect of the CRM can be quantified through the definition of equivalent stiffness and strength parameters accounting for its contribution. In the definition of the equivalent parameters, the mechanical properties of the mortar coating were considered rather than those of the

reinforcing mesh, as the latter plays a secondary role in the response of the strengthened panels. Indeed, as observed in both experimental and numerical investigations conducted in this study, and consistent with previous findings in the literature, CRM systems with thick plaster layers ($t_{pl} \geq 30$ mm) exhibits a negligible contribution of the grid to stiffness and strength. The grid mainly contributes during the post-peak phase, where it enhances ductility after significant matrix spalling. However, both the numerical simulations and experimental shear-compression tests performed by the authors revealed detachment phenomena that limited the efficiency of the CRM before the grid could reach its ultimate tensile capacity. The only relevant exception is the evaluation of the compression-bending capacity of panels strengthened with CRM systems effectively anchored at their ends. In this case, the tensile contribution of the reinforcing grid cannot be neglected, and the flexural resistance should be evaluated by means of the resistance domain of the strengthened cross-section, explicitly accounting for the tensile force carried by the wires, rather than through an equivalent compressive strength approach. Therefore, for panels with CRM discontinuous at the specimen ends, the lateral capacity $V_{U,R}$ can be evaluated as the minimum between the shear and flexural failure mechanisms, where the equivalent compressive strength $f_{c,m,R}$ and tensile strength $f_{t,m,R}$ of CRM-reinforced masonry can be evaluated using Eqs. (3) and (4).

$$f_{c,m,R} = f_{c,m} + \beta_c \bullet f_{c,pl} t_{pl} / t_m, \beta_c = 0.09 \quad (3)$$

$$f_{t,m,R} = f_{t,m} + \beta_t \bullet f_{t,pl} t_{pl} / t_m, \beta_t = 0.87 \quad (4)$$

where $f_{c,m}$ and $f_{t,m} = 1.5\tau_0$, are the compressive and tensile strength of URM, $f_{c,pl}$ and $f_{t,pl}$ are the compressive and tensile strength of the mortar coating, t_m and t_{pl} the thickness of the panel and the plaster. The tensile strength of the coating, $f_{t,pl}$, should be evaluated through a tensile test on CRM coupons; in absence of experimental tests, $f_{t,pl}$ can be estimated starting from the compressive strength of the mortar in the range $f_{c,pl}/10 - f_{c,pl}/15$. The correction factors β_c and β_t applied to formulations (3) and (4) were calibrated through a numerical regression aimed at minimizing the discrepancy Δ between the amplification coefficients α_V obtained from the FE model and those derived analytically. Therefore, 3 and 4 were employed to evaluate the shear strengths of both URM and RM panels, considering the different masonry typologies included in the parametric investigation. Based on these calculated strength values, the analytical amplification coefficients were computed and compared with the numerical results, iteratively updating the β coefficients. The deviation between numerical and analytical estimations was quantified

through the percentage difference $\Delta = \alpha_{V,fem} - \alpha_{V,analytic} / \alpha_{V,analytic}$. After regression, the resulting average deviation Δ between the two approaches is below 4%, with a maximum value of approximately 9%, corresponding to the case of a 120 mm thick brick masonry panel, strengthened on both faces, under an axial load level of $p = 0.75$. When the CRM is continuous and effectively anchored at the panel edges, the shear strength associated with diagonal cracking $V_{u,shear}$ can be still assessed using Eq. (4) to determine the equivalent tensile strength. In contrast, for the flexural failure mechanism, Eq. (3) can no longer be adopted to estimate the ultimate bending capacity and the compression–flexural resistance of the composite CRM–masonry section should instead be derived by constructing the corresponding strength domain, based on equilibrium conditions and strain-compatibility assumptions. Regarding the lateral stiffness, the equivalent Young's modulus and shear modulus can be derived using Eqs. (5–8), depending on the connection efficiency at wall's edges.

$$E_{m,R,discontinuity} = E_m \quad (5)$$

$$G_{m,R,discontinuity} = G_m + \beta_k \cdot G_{pl} t_{pl} / t_m, \beta_k = 0.485 \quad (6)$$

$$E_{m,R,continuity} = E_m + E_{pl} t_{pl} / t_m \quad (7)$$

$$G_{m,R,continuity} = G_m + G_{pl} t_{pl} / t_m \quad (8)$$

where E_m and G_m are the Young and the shear moduli of URM and E_{pl} and $G_{pl} = E_{pl} / 2(1 + \nu)$ are the moduli of the mortar coating. As for the strength, the coefficient β_k in (6) was obtained iteratively to minimize the discrepancy between numerical and analytical results. The deviation was quantified through the percentage difference $\Delta = \alpha_{k,fem} - \alpha_{k,analytic} / \alpha_{k,analytic}$. In this case, after regression, the resulting average deviation Δ between the two approaches is below 5%. Finally, regarding the definition of the ultimate displacement, numerical modelling and literature evidence indicate that the drift capacity of masonry piers should account for both the axial compression and the panel aspect ratio H/L . Despite the importance of this parameter, particularly in nonlinear static analyses, current Italian and European codes define it primarily based on failure mechanisms, typically function of panel height alone. Orlando et al. [40] proposed an analytical formulation for the drift capacity θ_c as a function of compressive ratio and slenderness, Eq. 9:

$$\theta_c = \frac{\delta_u}{H} = \frac{a_1 + a_2 H/L}{p^{a_3}} = \frac{0.00074 + 0.00069 H/L}{p^{1.22}} \quad (9)$$

The application of this formulation to the numerical models analyzed in the present study demonstrated its reliability in predicting the

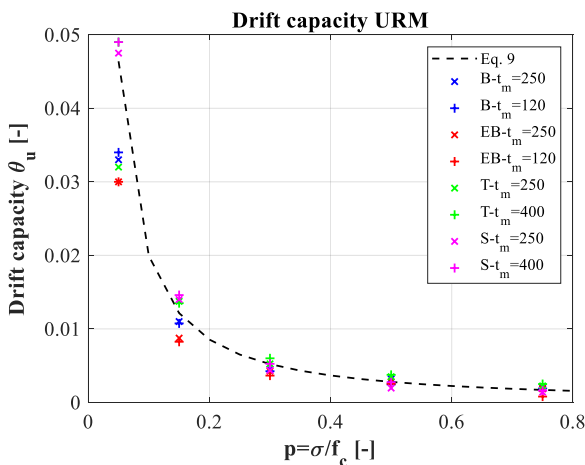


Fig. 25. Application of the analytical procedure proposed by Orlando et al. 2016 for the definition of displacement capacity of URM walls to the results of the present work.

ultimate drift of unreinforced masonry walls, as illustrated in Fig. 25 by the comparison between analytical and numerical results.

To develop an analytical procedure accounting for CRM effectiveness, Eq. (9) should first be adopted for the calculation of the displacement capacity of URM panels. Based on the results of the parametric study, the following expressions of amplification coefficient α_δ can then be used to estimate the ultimate displacement of CRM-reinforced walls:

$$\alpha_{\delta,discontinuity} = \begin{cases} 1 + p/0.3(1.64 \cdot \omega^{-0.273} - 1) & \text{for } p < 0.30 \\ 1.64 \cdot \omega^{-0.273} & \text{for } p \geq 0.30 \end{cases} \quad (10)$$

$$\alpha_{\delta,continuity} = \begin{cases} 0.3 + (p - 0.05)/0.25(1.64 \cdot \omega^{-0.273} - 0.3) & \text{for } p < 0.30 \\ 1.64 \cdot \omega^{-0.273} & \text{for } p \geq 0.30 \end{cases} \quad (11)$$

where p is the compression level acting on the wall and $\omega = \tau_0 t_m / f_{t,pl} t_{pl}$. The average difference $\Delta = \alpha_{\delta,fem} - \alpha_{\delta,analytic} / \alpha_{\delta,analytic}$ between the values obtained through the analytical procedure (10 and 11) and numerical procedure is about 6%. Fig. 26 illustrates the application of the proposed analytical method to a set of shear–compression experimental tests on CRM-strengthened masonry reported in the literature [16,17,41,42]. The experimental data considered refer to stone masonry or solid clay brick masonry specimens, with the CRM mortar coating applied on one or both faces of the panels. The mechanical and geometrical properties of the masonry substrate and the strengthening system were derived from the experimental data reported in the reference studies and were used to apply the analytical formulations. The dashed lines represent the bilinear response predicted by the analytical model, while the solid lines correspond to the experimental curves.

4.2. Compression response

In this Section, an analytical framework is introduced to capture the increase in compression strength that brick walls can attain through CRM reinforcement. The parametric study of Section 3.3 showed that CRM primarily enhances strength by stabilizing slender panels prone to second-order effects, whereas the mechanical contribution of the composite itself is rarely fully mobilized. Current Italian and European codes address these instability mechanisms through reduction factors that downgrade the nominal compressive capacity according to slenderness and eccentricity, leading to design checks based on simplified assumptions. Starting from this approach, the proposed formulation aligns with established definitions of slenderness and eccentricity. Based on the results of the parametric simulations described in Section 3.3 and presented in Fig. 23, an analytical procedure was developed to compute the contribution of CRM through the definition of the amplification coefficient $\alpha_N = N_{u,R} / N_{u,URM}$. The amplification coefficient is provided as a function of the out-of-plane bending effects acting on the panels, namely the values of λ and m . A first analytical approach for the evaluation of the CRM efficiency can be provided by the direct correlation between the amplification coefficient and the reduction factor, $\alpha_N(\phi)$, as obtained from the numerical simulations. By fitting the numerical results, the correlation expressed by Eq. 12 has been obtained.

$$\alpha_N(\phi) = N_{u,R} / N_{u,URM} = \max(0.90 \cdot \phi^{-0.51}; 1) \quad (12)$$

To assess the reliability of the correlation, it is possible to refer to the value of the determination coefficient R^2 . Using Eq. 12, an $R^2 = 0.82$ has been found, showing the good quality of the regression. Although strongly correlated, the coefficient ϕ depends on both panel slenderness and load eccentricity, so the same ϕ value may correspond to high slenderness with low eccentricity or low slenderness with high eccentricity. For a fixed ϕ , CRM effectiveness increases with slenderness, as second-order effects dominate over flexural moments induced by eccentricity. Consequently, the amplification coefficient can be expressed directly as a function of λ and m . An iterative least-squares regression minimizing the discrepancy between numerical and analytical

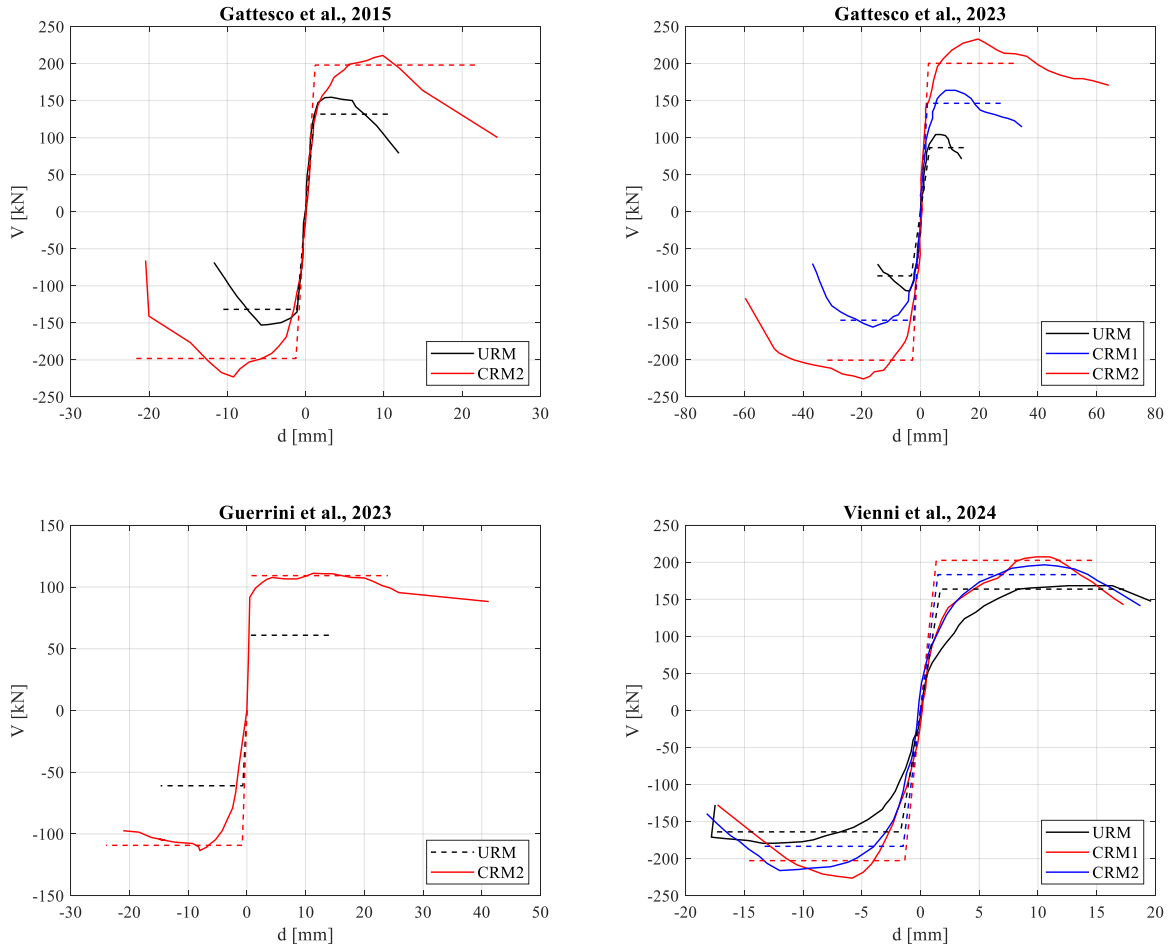


Fig. 26. Application of the analytical procedure to experimental shear-compression tests.

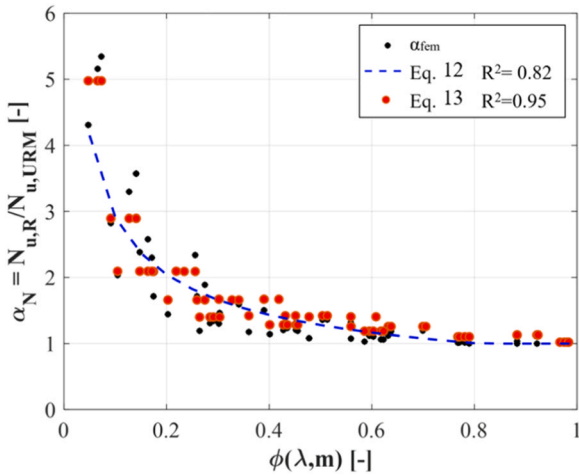


Fig. 27. Analytical procedure for the definition of the amplification coefficient of the compressive strength. Comparison between Eq. 12, Eq. 13 and numerical results.

coefficients $\alpha_N(\lambda, m)$ led to Eq. (13):

$$\alpha_N(\lambda, m) = (1 + 0.144m) \bullet e^{([0.0096\lambda / (0.33 - 0.082m)]^2)} \quad (13)$$

For the Eq. 13, an $R^2 = 0.95$ was obtained, showing a stronger correlation than the previous one. In addition, the discrepancy has been defined as the percentage difference between the numerical and

analytical procedure $\Delta = \alpha_{N,fem} - \alpha_{N,analytic} / \alpha_{N,analytic}$, where $\alpha_{N,fem}$ are the amplification coefficients obtained from the numerical models implemented in Table 6 and $\alpha_{N,analytic}$ are analytical values for the corresponding configurations derived using Eq. 13. Considering the results of the whole samples of analysis, an average difference of $\Delta = 7\%$ has been obtained. The regressions obtained using 12 and 13, and the comparison with the numerical results are shown in Fig. 27.

Considering the boundary conditions adopted in the numerical simulations, on the basis of which the analytical models have been developed and the significant role of the interface strength on the evolution of the buckling (Figs. 21, 22), the proposed method is formulated for double-hinged walls with a symmetric CRM application and has been validated for the specific CRM-brick system considered in this work, characterized by a specific interface strength value τ_{max} . Its extension to other masonry typologies requires further calibration that accounts for the shear bond law at the interface, valid for the individual type of substrate.

5. Conclusions

The present study examined the structural response of masonry panels strengthened with CRM systems, focusing on in-plane shear and flexural capacity, drift performance, and the contribution of CRM layers to out-of-plane stability under compression. Experimental results previously obtained by the authors (Vienni et al. [17]) were used to support the formulation and calibration of a DIANA FEA numerical model. A 3D macro-modelling strategy was adopted, combining nonlinear constitutive laws for masonry and mortar, truss elements for the GFRP grid, and an interface layer reproducing limited bond and detachment. Despite

the inherent simplifications, the model reproduced with good accuracy the experimental response and failure modes of both unreinforced and strengthened panels. The calibrated model enabled an extensive parametric analysis aimed at assessing the influence of key variables on CRM efficiency. Results confirmed that CRM continuity and anchorage at wall edges are decisive for seismic response. Without effective anchorage, the flexural contribution of CRM is negligible under low axial load and only modest under higher compression. With full anchorage, bending capacity and initial stiffness increase markedly. Conversely, shear capacity remains mainly governed by diagonal cracking and is only marginally affected by boundary conditions. A slight reduction in drift capacity was observed in rocking-dominated panels due to corner constraints. Overall, CRM effectiveness was found to scale with the ratio $\tau_{0t_m}/(f_{t,pl}t_{pl})$, reflecting the key role between masonry strength and plaster tensile capacity. Under compression, CRM primarily enhances out-of-plane stability rather than directly increasing compressive strength, with significant benefits occurring in slender or eccentrically loaded walls. In contrast to the shear-compression loading, compression behaviour proved highly sensitive to interface adhesion because detachment tends to develop during second-order deformation. Based on the numerical results, analytical formulations were proposed to quantify CRM contributions in both shear-compression and compression design. For seismic assessment, two sets of expressions were derived to account for the limit cases of reinforcement continuity or discontinuity at wall ends, enabling evaluation of shear strength by diagonal cracking, flexural capacity, stiffness, and drift within the bilinear idealization commonly adopted in technical standards. Further studies will be required to extend the evaluation also on the sliding shear failure mechanism. For compression, an amplification coefficient was introduced to incorporate CRM effects as a function of wall slenderness and eccentricity. It is worth nothing that this proposed formulation is calibrated and validated for brick masonry with bond characteristics comparable to those investigated in this study. The proposed analytical approaches offer a practical, physically grounded framework for the design and assessment of CRM strengthening interventions. Future developments should include validation on different masonry typologies, refined investigation of interface behavior and connector effectiveness, and the analysis of full wall assemblies to more accurately capture the influence of spandrels, floors, and boundary constraints.

CRedit authorship contribution statement

Carlo Vienni: Writing – review & editing, Writing – original draft, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **Luca Salvatori:** Writing – review & editing, Supervision, Formal analysis, Data curation, Conceptualization. **Maurizio Orlando:** Writing – review & editing, Supervision, Methodology, Funding acquisition, Formal analysis, Conceptualization.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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