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RETROFITTING SOLUTIONS FOR ENHANCING THE IN-PLANE RESPONSE OF MASONRY WALLS UNDER SEISMIC LOADING

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Abstract

This study aims to numerically evaluate the beneficial effects of two retrofitting solutions for masonry walls subjected to seismic loads. The solutions investigated involve applying steel and Glass Fiber Reinforced Polymer (GFRP) grids embedded in a thin layer of plaster on both faces of the masonry panels. Two types of masonry were examined, selected to represent real construction cases in historic centres of Southern Italy: irregular stone masonry and regular soft stone masonry (tuff). Finite Element models were used with a macro-scale representation of the masonry material. The masonry panels, measuring 3000×3000×600 mm, were subjected to a constant vertical compression and an increasing in-plane horizontal displacement at the top until collapse, with rotation prevented. Both unreinforced and reinforced conditions were analysed, and the results compared to assess the effects of these retrofitting systems in terms of stiffness, strength, and ductility. This study was conducted as part of the research project "GENESIS – Seismic risk manaGEmeNt for tourist valorization of thE hiStorIcal centers of Southern Italy" and serves to lay the groundwork for upcoming experimental activities.

Keywords: Masonry walls, Retrofitting systems, Finite Element Model, Nonlinear analyses, Steel, Glass Fiber Reinforced Polymer.

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1 INTRODUCTION

Masonry buildings constitute a significant portion of the existing building stock in many parts of the world, and especially in Italy. In particular, historic centres in Italy, as well as in many countries of Southern Europe, are characterized by a high concentration of unreinforced masonry structures, often built using poor traditional techniques and local materials of scarce mechanical performances. These constructions, while architecturally valuable and culturally significant, are inherently vulnerable to seismic actions due to their brittle behaviour, limited tensile strength, and poor energy dissipation capacity.

Past earthquakes have repeatedly demonstrated the seismic inadequacy of such structures [1–3], often resulting in severe damage or collapse. The irregularity in geometry, discontinuities in construction, and degradation over time further exacerbate this vulnerability. As a result, there is a growing need to identify effective retrofitting strategies capable of enhancing the structural performance of masonry walls, particularly for improving their in-plane response under seismic loading [4–10]. Numerical analysis, in this field, are a very useful tool able to predict the behaviour of masonry structure with and without the retrofitting systems [11].

Addressing and reducing this vulnerability is one of the key objectives of the GENESIS project, a national research initiative involving around 20 academic and industrial partners, which aims to develop smart strategies for the seismic risk management of historic centres in Southern Italy [12].

In this context, the development and assessment of minimally invasive, compatible, and efficient strengthening techniques represent a key area of the research project, namely within Work Package 4 leaded by the team of University of Pisa. WP4 addresses strategies for damage prevention, mitigation, and structural retrofitting across four key stages of seismic risk management: (i) planning risk-reduction interventions before an event; (ii) applying protective measures during earthquake swarms; (iii) responding promptly after a mainshock to limit further harm; and (iv) developing reversible and compatible retrofitting solutions for historical masonry in the post-emergency phase.

This study contributes to this objective by numerically assessing the in-plane behaviour of masonry walls, in both unreinforced and retrofitted configurations, under constant vertical compression and increasing in-plane shear loading. Specifically, it investigates the effectiveness of two strengthening techniques based on the use of steel and Glass Fiber Reinforced Polymer (GFRP) grids applied on both faces of the wall. The numerical results provide valuable insights into the potential of these systems for seismic risk mitigation. This work represents a preparatory phase for future experimental testing, which will be carried out at the Laboratory of the University of Pisa.

2 UNREINFORCED MASONRY WALLS

2.1 Definition of the test

The present study focuses on two case studies representative of Italian historic centres, namely Tricarico (Matera province, Basilicata region) and Casertavecchia (Caserta province, Campania region), which were analysed within the framework of the aforementioned research project GENESIS (Figure 1).

The preliminary reconnaissance of the building typologies present in these areas leads to assume the prevalence of uncut stone masonry and regular tuff masonry for ordinary buildings in Tricarico and Casertavecchia, respectively.

Thus, these two masonry typologies were considered for the study of the efficiency of the two investigated retrofitting solutions. The considered masonry walls have dimensions of 3 m in length and height, with a thickness of 0.60 m. The load conditions involved the application of a uniformly distributed vertical load equal to 40% of the wall's compressive strength, along with increasing horizontal displacements applied to the upper face of the wall under fixed-end (rotation-prevented) conditions.

(a) (b)

Figure 1. View of the considered historical centers: (a) Tricarico (CS) and (b) Casertavecchia (CE).

2.2 URM walls modelling

FE models were implemented in Abaqus software [13], employing a macro-modelling strategy for the simulation of masonry panels [14]. The panels were discretized using second-order 10-node tetrahedral elements (C3D10), with an average mesh size of 0.15 m.

The upper and lower faces of the panels were rigidly connected to reference nodes, which served as the points of application for boundary and load conditions. The analyses were conducted in two sequential steps. In both steps, the lower reference node was assumed to be perfectly fixed. In the first step, a vertical load was applied to the upper reference node, while horizontal displacements, both in-plane and out-of-plane, were constrained. In the second step, the vertical displacement of the upper reference node was fixed, and a progressively increasing horizontal displacement was imposed until the collapse of the panel.

As for masonry modelling, the *Concrete Damage Plasticity* material model was adopted [15, 16]. Compressive laws were defined according to the parabolic law proposed in [17]. A fracture energy-based approach was adopted to capture the post-peak nonlinear tensile behaviour. Additionally, damage laws were defined for both compressive and tensile responses, with the damage variables increasing linearly from 0 at peak strength to 1 at the corresponding ultimate strains.

As for masonry mechanical and strength characteristics, reference to median values of the range proposed by Italian standards for the considered masonry typologies was made [18]. The adopted values are reported in Table 1, where E is the elastic modulus, f_c is the compressive strength, f_t is the tensile strength and G_f is the fracture energy. For convenience of the reader, masonry Type 1 refers to uncut stone masonry, while Type 2 to tuff masonry.

Masonry	E [MPa]	f_c [MPa]	f_t [MPa]	G_f [N/m]
Type 1	1320	2	0.0645	15
Type 2	1410	2.8	0.09	20

Table 1. Mechanical and strength values adopted for masonry modelling.

The remaining parameters required to fully define the material behaviour were: ψ=10°, ε=0.1, f_{b0}/f_{c0}=1.16 and K_c=0.66.

2.3 Responses URM walls

The results of the analyses are presented as force-displacement curves in Figure 2. In both cases, the curves are truncated at a 20% drop from the peak load, which occurred at a displacement of approximately 2.5 mm in both cases. The panels exhibited limited ductility, characterized by an abrupt post-peak strength degradation. Regarding peak strength, the Type 1 and Type 2 masonry walls reached maximum forces of 414 kN and 300 kN, respectively.

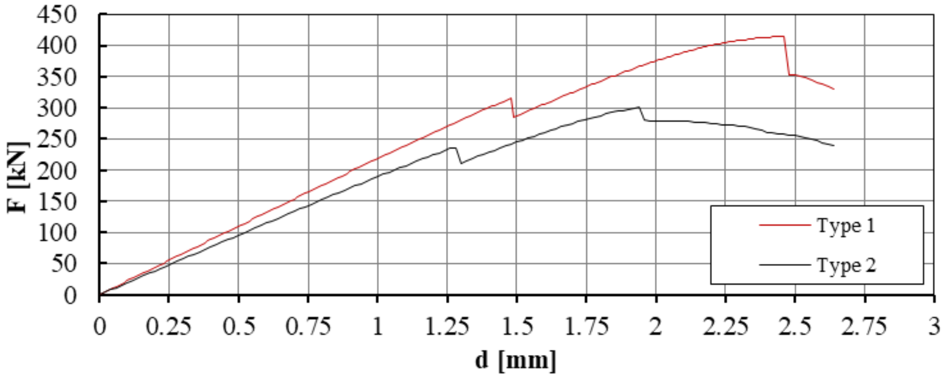
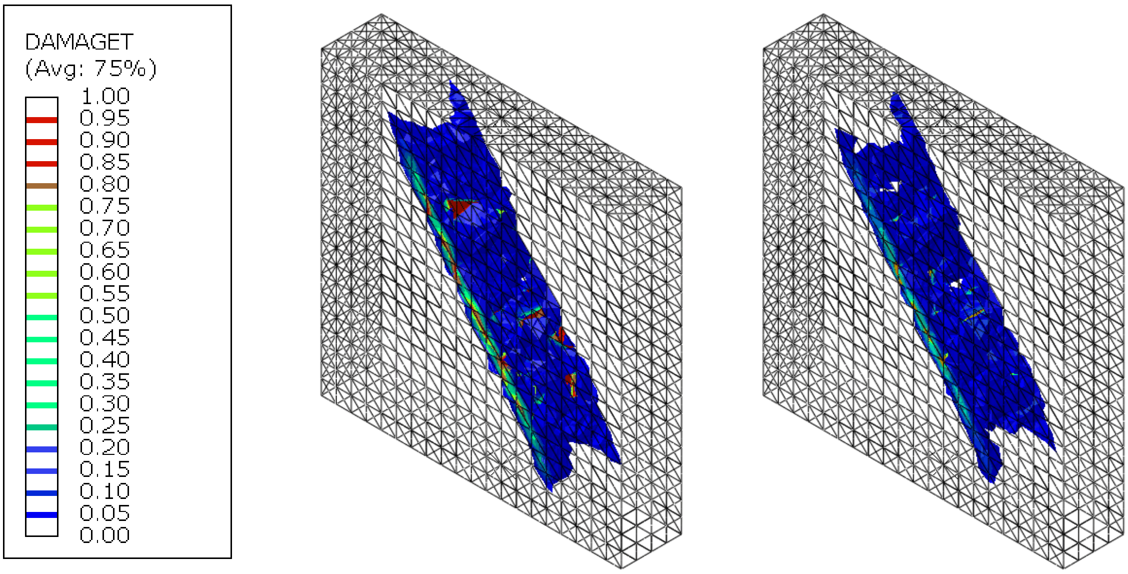


Figure 2. Force-displacement curves of URM walls

In both cases, shear failure was observed, as evidenced by the tensile damage patterns at peak forces shown in Figure 3.



(a) (b)

Figure 3. Tensile damage parameter at peak strength: (a) Type 1 masonry and (b) Type 2 masonry.

3 REINFORCED MASONRY WALLS

3.1 Description of the retrofitting strategies

The investigated retrofitting strategies aim at enhancing the in-plane behaviour of masonry walls under seismic load conditions. Both can be classified as external retrofitting systems, providing the application of grids of different materials on both faces of the masonry wall.

The first system, denoted with the acronym *S_GRID*, entails the application on both wall faces of thin steel grids with vertical and horizontal elements spaced at regular intervals. The grids are connected at each steel element intersection by transverse metal bars, which are tightened to prestress the system and ensure its effectiveness. Similar applications of this system can be found in [19].

The second system (*GFRP_MESH*), on the other hand, involves the application of a Glass Fiber Reinforced Polymer (GFRP) mesh embedded in a thin plaster of cement-based mortar. In this case as well, a transverse connection is provided through elements with higher dimensions (hooks). Examples of GFRP-retrofitting applications can be found in [20, 21].

For the cases under consideration, *S_GRID* presents the following characteristics:

- Steel grade of vertical and horizontal elements: S235;
- Steel grade of transversal metal bars: 8.8;
- Width of steel elements: 5 cm;
- Thickness of steel elements: 0.5 cm;
- Vertical and horizontal spacing: 30 cm;
- Dimensions of transversal bars: ϕ20.

As for *GFRP_MESH* system, the following characteristics were considered:

- Tensile strength of GFRP grid: 879 MPa;
- Tensile strength of GFRP hook: 280 MPa;
- Area of grid elements: 7.05 mm^2;
- Vertical and horizontal spacing of GFRP mesh: 10 cm;
- Area of transversal connectors: 78.9 mm^2
- Vertical and horizontal spacing of transversal connector: 50 cm.

3.2 RM walls modelling

The retrofitted walls were modelled starting from the unreinforced configurations. The system *S_GRID* was assembled using 10 node 2nd order elements with an average mesh size of 5 cm for external grids and 0.5 cm for transversal metal bars. Typical *Plastic* materials were adopted for both the grids and the metal bars, assuming elastic–perfectly plastic constitutive behaviour.

The transversal bars were considered embedded within the masonry wall, while a contact law was defined for simulating the interaction between the masonry and the steel grids. In particular, *Hard* contact was considered in the normal direction such as to avoiding interpenetration while allowing for separation. In the tangential direction, a *Penalty* contact behaviour was used, assuming a friction coefficient of 0.3.

The extremity nodes of the grids were assumed to be included within the upper and lower rigid bodies. An additional analysis step was introduced prior to the application of the vertical load, in order to simulate the tightening of the transverse metal bars. This was made by conferring to transversal metal bar a temperature variation ΔT=-20°C, which combined with a thermal expansion coefficient α=1.2e-5 °C^{-1}, provided a tensile stress in the metal bars of about 50 MPa.

Regarding the *GFRP_MESH* modelling, the meshes and hooks were represented as two-node truss elements with an average element length of 1 cm. These were embedded within 2 cm thick plaster layers and masonry, respectively. The *Plastic* material model was used to simulate the GFRP components, incorporating a brittle post-peak behaviour to reflect their failure characteristics. The mortar layers were modelled using 10-node second-order solid elements, matching the mesh size of the masonry wall (average element size of 0.15 m). *Concrete Damage Plasticity* was used for reproducing the mortar response, by assuming an elastic modulus E=3200 MPa, a parabolic compressive law with a strength f_c=28MPa and a tensile strength f_t=5MPa with a fracture energy G_f=80N/m. In this case, no damage laws were defined given that low damage levels were expected in the mortar plaster.

Also in this case, the extremity nodes of both GFRP and mortar layers were included in rigid bodies and a perfect connection by means of *Tie* constraints was introduced between mortar and masonry. The adopted numerical models are shown in Figure 4.

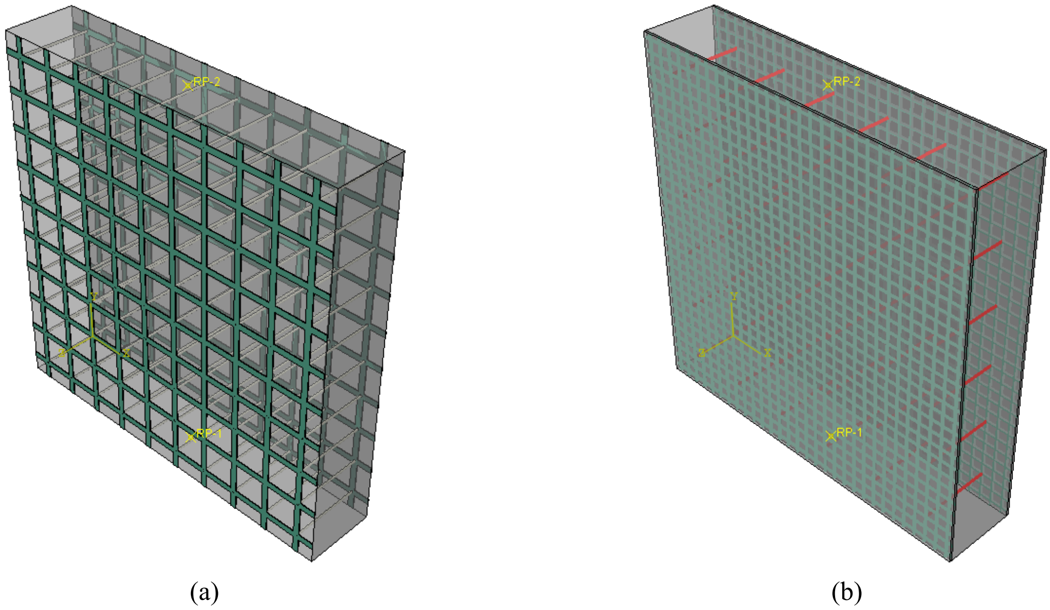


Figure 4. The adopted FE models for retrofitted configurations: (a) *S_GRID* system and (b) *GFRP_MESH* system.

3.3 RM walls responses

The responses of retrofitted walls are shown in Figure 5 in terms of force-displacement curves in comparison with URM results.

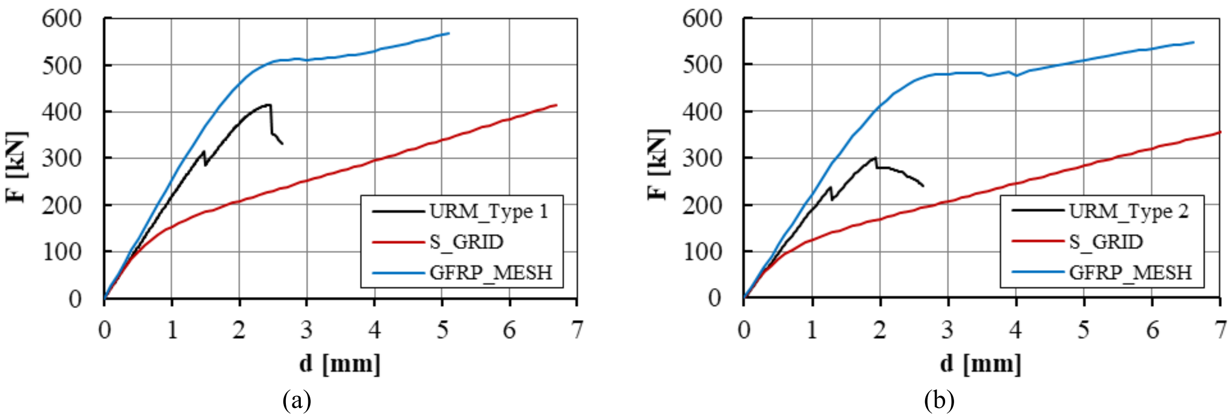


Figure 5. Force-displacement curves of RM walls: (a) Type 1 masonry and (b) Type 2 masonry.

As demonstrated, both reinforcement systems markedly improved the wall's response compared to the unreinforced configuration. For the *S_GRID* system, however, an early stiffness degradation occurs at low displacement levels. This degradation stems from contact between the transverse metal bars and the masonry, which induces localized damage. Beyond this initial loss, the force–displacement curves exhibit pronounced hardening branches: Type 1 masonry attains a peak force similar to that of the unreinforced wall, while Type 2 reaches approximately 350 kN (about a 16 % increase over the unreinforced case). Moreover, for both masonry types, the ultimate displacement more than doubled relative to the unreinforced configuration.

In contrast, the *GFRP_MESH* system imparted a modest increase in initial stiffness. More importantly, it delivered a substantial gain in strength, boosting peak capacities into the 550–570 kN range. While the additional deformation capacity it provided was less pronounced than that of the *S_GRID* system, it nonetheless offered a significant increase over the unreinforced walls.

By observing the damage patterns of the masonry obtained at the ultimate displacements shown in Figure 6 and in Figure 7 for *S_GRID* and *GFRP_MESH* systems, respectively, it can be noted a general improvement of the walls' responses also in terms of failure modes.

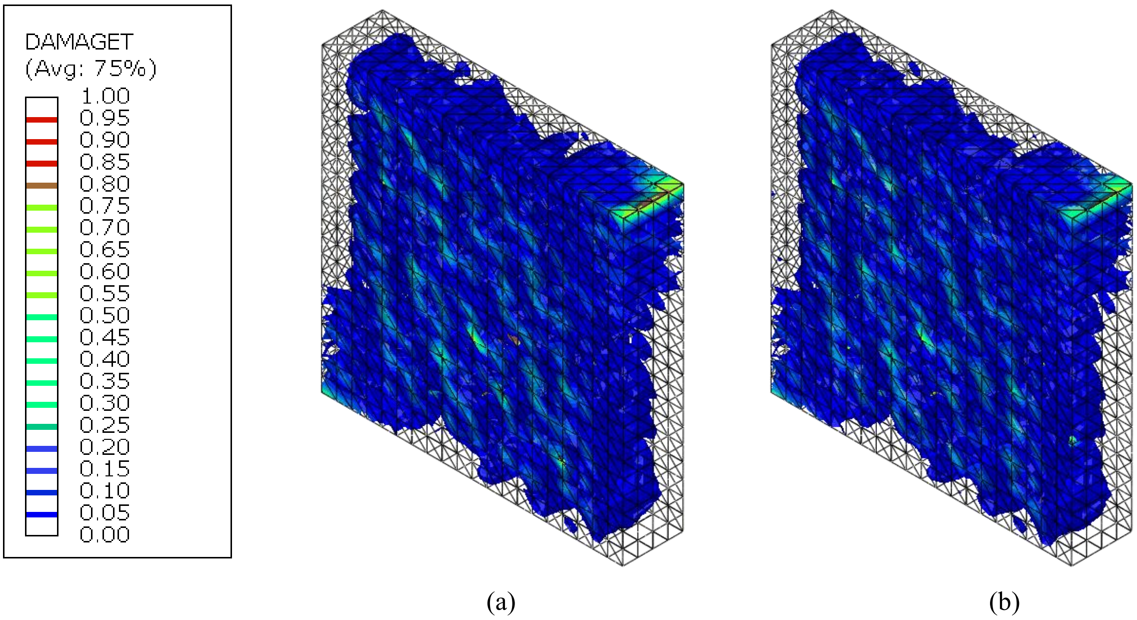


Figure 6. Tensile damage at ultimate displacement of masonry walls retrofitted with *STEEL_GRID* system: (a) Type 1 masonry and (b) Type 2 masonry.

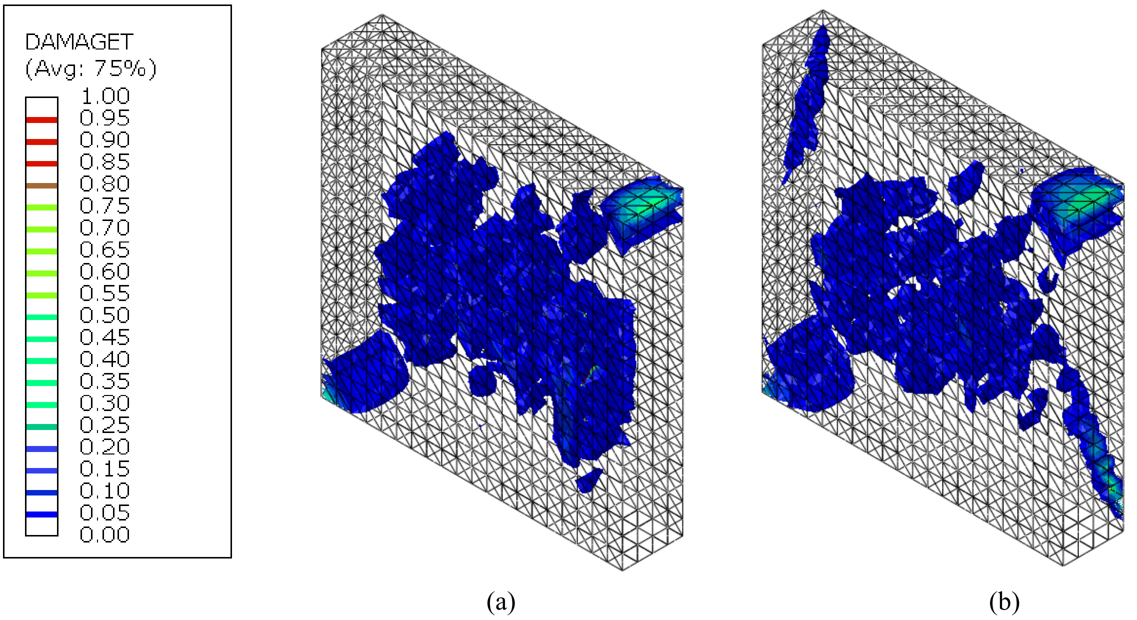


Figure 7. Tensile damage at ultimate displacement of masonry walls retrofitted with *GFRP_MESH* system: (a) Type 1 masonry and (b) Type 2 masonry.

In particular, both systems facilitate stress redistribution within the masonry, ensuring a division of the tensile ties similar to a Ritter–Mörsch mechanism. As a consequence, the most damaged zones resulted the tensile corner of the panels and the internal masonry portions in contact with transversal elements.

In all analysed cases, the reinforcing elements (i.e steel grids and GFRP meshes) remained in the elastic fields, with most stressed zones, again, in the corner proximities, as shown in Figure 8 and in Figure 9 for *S_GRID* and *GFRP_MESH* systems, respectively. Finally, it is worth mentioning that for *GFR_MESH* system no plasticized zones were observed for plaster layers.

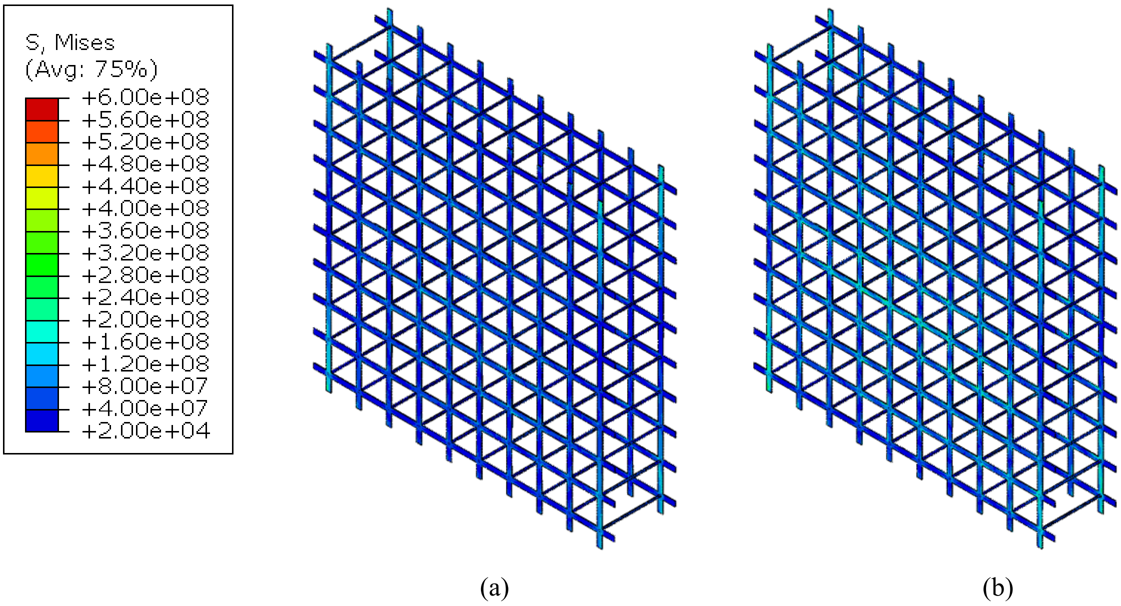


Figure 8. Von Mises stresses at ultimate displacements of steel grids: (a) Type 1 masonry and (b) Type 2 masonry.

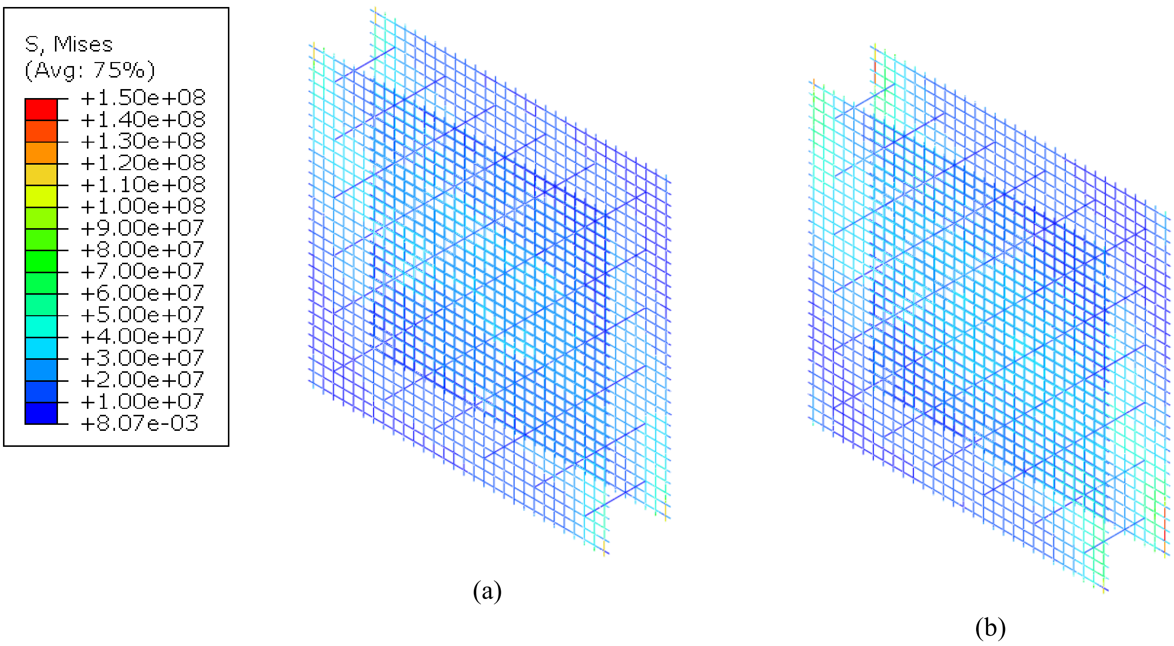


Figure 9. Von Mises stresses at ultimate displacements of GFRP meshes: (a) Type 1 masonry and (b) Type 2 masonry.

4 CONCLUSIONS

The present study investigated two retrofitting techniques designed to enhance the in-plane behaviour of masonry walls through numerical analysis. Based on case studies from Italian historic centres, two prototype masonry specimens representative of typical ordinary buildings were defined. To this purpose, two distinct masonry materials, with mechanical and strength properties specified in accordance with Italian Standards, were assumed.

Both retrofitting systems employ external grids, one made of steel and the other of glass fiber reinforced polymer (GFRP). To assess their effectiveness, advanced nonlinear numerical models were developed in Abaqus, accurately capturing the material behaviours and the interactions between masonry and reinforcement.

The results demonstrated that both systems substantially improve wall performance, increasing strength and deformation capacity. By altering the masonry response, the grids prevent crack localization along the panel diagonals and promote a more uniform stress redistribution. Future work will extend these analyses to optimize the grid configurations and dimensions, with the goal of guiding upcoming laboratory tests planned as part of the GENESIS project.

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