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SIMULATION OF IN PLANE REINFORCED MASONRY WALLS BEHAVIOUR BY MEANS OF SIMPLIFIED METHOD

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Abstract

Reinforced masonry is one of the most used solution that ensures good resistance from seismic loads, simplicity in the construction phase and good resilience over time. The reproduction of the mechanical behaviour of these structures with numerical strategies requires the effective simulation of the interaction between masonry elements and steel bars, objective not easy to achieve with standard methods.
This paper introduces a simplified method for the evaluation of the in plane seismic behaviour of reinforced masonry walls. The method is based on the simulation of the geometry of the panel by means of shear deformable quadrilaterals, interacting among them through non-linear 2D links, in which the stiffness and the constitutive laws of the masonry are condensed. The reinforcement steel bars are modelled as 1 D links in parallel to the masonry links, reproducing the axial behaviour of the material.
The numerical tests presented in this work are compared with experimental test present in literature, performed on walls with different geometry. The different tests highlighted the capacity of this simplified approach to reproducing the experimental failure modes, in particular the flexural behaviour and the shear-sliding mechanism observed experimentally. The results are reported as capacity curves, deformed shape and stresses contours.

Keywords: Reinforced masonry, simplified method, numerical method.

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1 INTRODUCTION

The numerical simulation of the mechanical behaviour of masonry reinforced elements, represent a complex task and requires the reproduction of the non-linearity and asintropy of the masonry and the mechanical interaction between the reinforcement steel bars and the masonry. In literature several approaches has been developed with the aim of good accuracy and limiting the computational costs for the practical application. Detailed micromodels reproduce with accuracy the elastic and inelastic behavior of the masonry using 2D plane stress Finite Elements and non-linear interfaces and utilizing trusses and beam element elements for the steel bars [1]. Simple strategies, consisting in discretizing the geometry of the masonry and the steel bars with cells which edges and diagonals are trusses elements [2], is conceptually simple to understand can lead to cheaper computational models. Another numerical strategy for the implementation of the reinforced masonry wall is the multiple-vertical-line-element model (MVLEM) that consists in several vertical springs connected to rigid edges that reproduce the axial and vertical behaviour, and a single horizontal spring that take into account the shear behaviour[3]. Limit analysis approaches are well-known methods for the estimation of the limit capacity and failure modes of masonry structures that provides good results with relative low computational cost [4].

This paper presents a simplified numerical strategy for simulating the in-plane behaviour of masonry reinforced walls. The proposed model schematizes the wall as discrete elements, in which interfaces are concentrated the elements that reproduce the mechanical behaviour of the masonry and of the steel bars. The proposed model is validated by simulating numerically lateral load test performed on two masonry reinforced walls[5]. The comparisons of force-drift capacity curves and failure mechanisms showed a good agreement between experimental and numerical results.

2 GENERAL DESCRIPTION OF THE NUMERICAL METHOD

The proposed numerical method schematizes the reinforced masonry panel through a regular mesh of shear-deformable panels with rigid edges connected by vertical and horizontal interfaces as presented in the Figure 1 [6]. In the interfaces, are arranged 2 D links with constant spacing, that reproduce the mechanical behaviour of the masonry, meanwhile 1 D link, with the role of simulating the axial behaviour in tension of the steel bar reinforcement, is positioned in parallel to the previous one in the actual position of the steel bar (Figure 2). Both the vertical and the horizontal bars are numerically simulated.

The 2 D masonry link is composed by a normal spring in which the tensile and compressive constitutive laws of the masonry are condensated and a tangential spring that reproduces the cohesive and frictional behaviour of the material. The shear spring is influenced by the normal axial force in the link through a Mohr-Coulomb correlation.

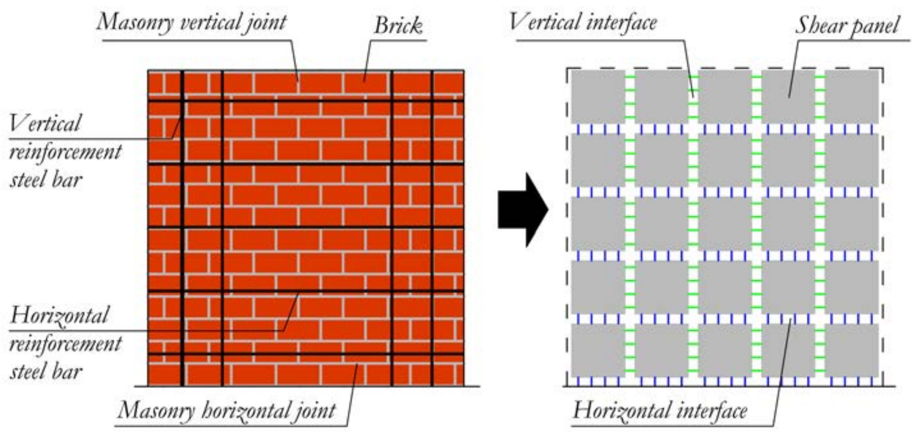

Figure 1 Real masonry reinforced wall (left); numerical representation of a panel with discrete elements (right)

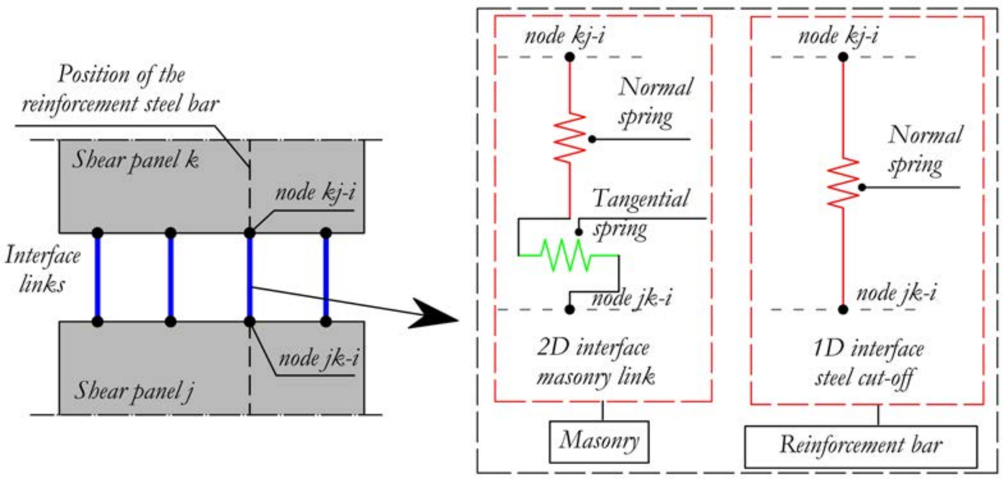

Figure 2 Discrete element interface with representation of all the nodes that are connected with 2D links (left); single 2D links numerical representation (right)

3 MODEL CALIBRATION

The normal constitutive law for the masonry is described in compression by means of a parabolic relationship reaching the peak-resistance strength, followed by a linear softening that ends to a constant residual strength. The tensile behavior is defined by a linear elastic law with linear softening after the peak-value after which a zero force residual is implemented, that corresponds to the cracking of the masonry; the link that reaches this phase is considered detached if tensile displacement occurs, and it doesn’t contribute in the normal and tangential global response. The initial normal stiffness, the peak resistance in tension and the peak resistance in compression are calibrated assigning the mechanical parameters of the influence area of each link. The tangential spring simulates the sliding behavior with an linear elasto-plastic constitutive law until the peak force, calculated considering the Mohr-Coulomb yielding surface, dependent by the cohesion of the masonry, the friction coefficient and the normal force acting in the link. After the peak follows linear softening, governed by a damage function that reduces cohesion, friction coefficient and tangential stiffness.

The links that simulates the steel bars present an only-tension linear elasto-plastic constitutive law. The peak force is the actual peak tensile force of the bar and the initial stiffness is calibrated with the influence length of the link in the direction parallel to it.

4 NUMERICAL SIMULATION OF TESTS ON REINFORCED MASONRY WALLS

The effectiveness of the proposed numerical model is validated through the simulation of experimental test executed on reinforced masonry walls. The experimental campaign [5] investigates the in plane shear and flexural behaviour of concrete hollow blocks masonry walls with several geometry and reinforcement configurations. In this work two walls are taken into account: the W2 wall with height of 3.80m, width of 2.60m and thickness of 0.14m, and the W4 wall with height of 4.00m, width of 1.40m and thickness of 0.19m. The tests are executed in horizontal control of displacement, applying the load at the corner of wall head, free to rotate and in absence of applied vertical load. The base of the walls are connected to a concrete footing from which the vertical steel bars are embedded; this footing is restrained to the rigid floor slab.

The numerical tests output are the correlation of the applied horizontal drift applied to the panel, expressed the ratio between displacement and wall height, and the sum of the horizontal reaction registered at the restrained base of the wall. To evaluate the global behaviour of the walls, monotonic test are numerically executed.

In the Figure 3 is presented the comparison between the experimental curve and the monotonic numerical output of the W2 wall, meanwhile in the Figure 4, the corresponding curves for the W4 wall are reported.

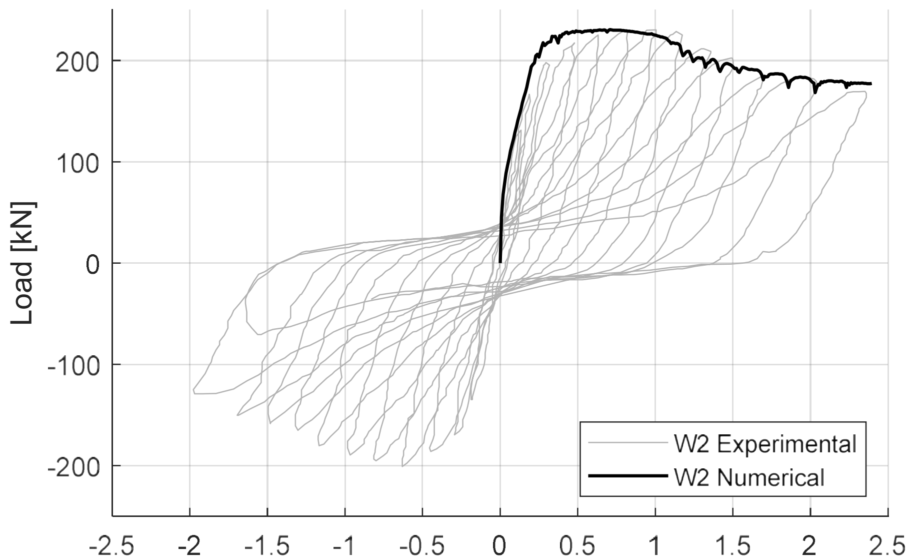

Figure 3 Comparison between the experimental and numerical results for the W2 wall

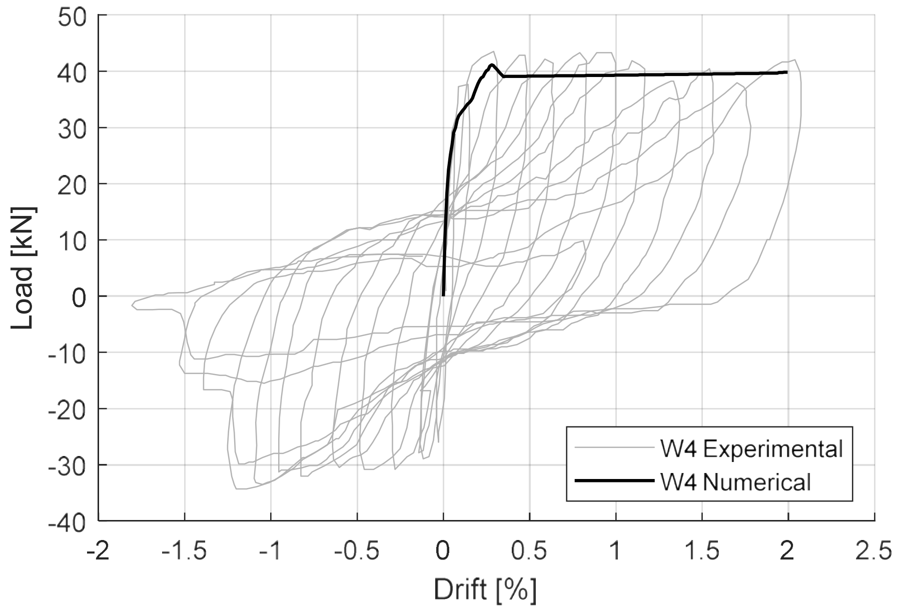

Figure 4 Comparison between the experimental and numerical results for the W2 wall

5 DISCUSSION OF THE RESULTS

The comparison of capacity curves of the W2 wall highlights the faithful reproduction of the experimental behaviour by the numerical test in terms of the initial stiffness, corresponding to the not-cracked phase of the masonry, and the variation of the stiffness around 70 kN, due to the partialization of the section of masonry because of the cracking of the material. After this point, the typical rocking mechanism is activated, with crushing phenomena of the masonry in the right corner of the base wall (Figure 5) and masonry cracking in the left side at which follows the increasing of the axial stress in tension in the steel bars until the yielding stress. The peak load is well reproduced by the numerical test, corresponding to the yielding of the left bars and the crushing of the compressed corner. The decreasing in the load registered after the peak, well-simulated in the numerical test, is due to the moving of the compressed zone toward left, since after the masonry links reach the compressive limit, the softening in compression reduces the link axial resistance.

The W4 wall is characterized by a first phase in which the flexural behaviour is predominant, reproduced by the numerical model, at which follows the partialization o the section that corresponds to the variation of the stiffness, similarly to the previous case. The load peak registered in the numerical test is coherent with the experimental one. After the peak load, both in the numerical and in the experimental curves it is possible to individuate an almost load constant phase, that corresponds to the sliding between the base wall and the concrete footing (Figure 5). For the numerical model this phenomenon is due to the shear failure of the masonry links. The shear resistance of the steel bars can be ignored due to the magnitude of the sliding registered in the experimental findings: it reaches 35 mm of slippage at the base of the wall, at which the 5.64mm diameter bars results not active in the global shear contribution.

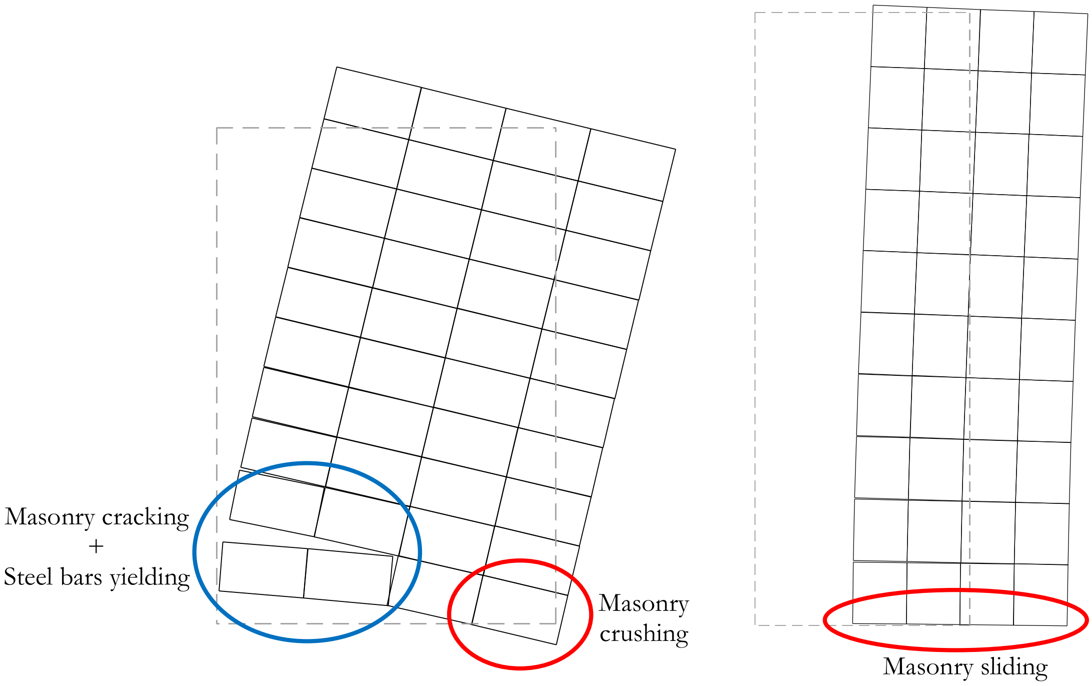

Figure 5 Numerical deformed shape registered at the end of the test for the W2 wall (left) and W4 wall(right), considering an amplification factor of 10.

6 CONCLUSION

This paper introduces a simplified numerical strategy for the simulation of the nonlinear in-plane behaviour of reinforced masonry panels. The geometry of the masonry is represented by a mesh of discrete elements able to simulate the stiffness and collapse modality of masonry material. The steel bars are implemented by a simple and efficient method. The calibration of the model is intuitive and based on the materials mechanical characteristics. The cases study presented highlight the faithfulness of this method in reproducing the global collapse modalities, in particular the flexural and the sliding behaviors. The comparison between the experimental and numerical results exhibits the capacity of this method in simulating the initial linear phase, the peak load and the post-peak phase, reproducing the actual behaviour of the masonry and the steel bars.

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