Modelling of stone masonry walls strengthened by RC jackets

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MODELLING OF STONE MASONRY WALLS STRENGHTENED BY R. C. JACKETS

C. Modena[1], G. Zavarise[2] and M.R. Valluzzi[3]

1. ABSTRACT

In this paper we illustrate the research activity carried out to set up diagnostic methods for the determination of mechanical parameters of the walls and to check the effectiveness of repairing and reinforcement techniques. A series of experimental tests carried out on old buildings before and after restoration are discussed, and comparisons with numerical analyses are shown.

2. INTRODUCTION

Stone masonry walls constitute one of the most common construction typologies that have been used all over Europe and the necessity to define reliable criteria for repairing and restoring them is actually taking more and more importance. Such walls are in fact quite often not responding to the current criteria of comfort and safety.

One of the most diffused techniques for consolidation is based on the application of a thin concrete slab on both the surfaces of the wall; these layers are reinforced with a welded net and connected between them by transversal ties across the wall. A noticeable experimental activity has been devoted to the analysis of the reparation with this technology and comparisons with FE numerical methods have also considered. These methods have actually the possibility to deal with various kinds of non-linearity, hence it can be used also to integrate and partially replace experimental tests. However the current experience in using numerical method in this field is still limited, hence results should be evaluated with attention.

Keywords: Stone masonry walls, Wall reinforcement, R.C. Jackets, Numerical tests

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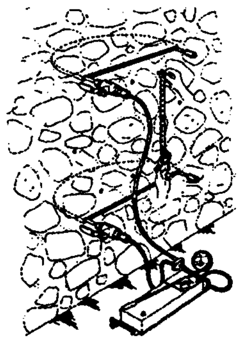
3. EXPERIMENTAL TESTS

The walls have been tested before and after restoration using both large compression and shear tests, and flat jack tests, which allowed to determine the state of stress and the deformability. The necessity to check a representative area has forced the employment of special very large flat jacks. The scheme of the test with the check points is reported in Fig. 1 [3]. In-situ compression tests have been carried out checking large panels which dimension is 1.5×1 m. A concrete beam have been created both on the top and on the bottom of the panels to distribute the applied load and a series of checking point on the sides of the panel have been placed. The tests have been carried out both for original and restored walls, see Fig. 2. A comparison of the results obtained with flat jack tests and compression tests is reported in Fig. 3 [4], which shows a good agreement between the procedures. The efficacy of the restoration with concrete slabs is evidenced in Fig. 4, where the completely different behaviour before and after restoration is depicted.



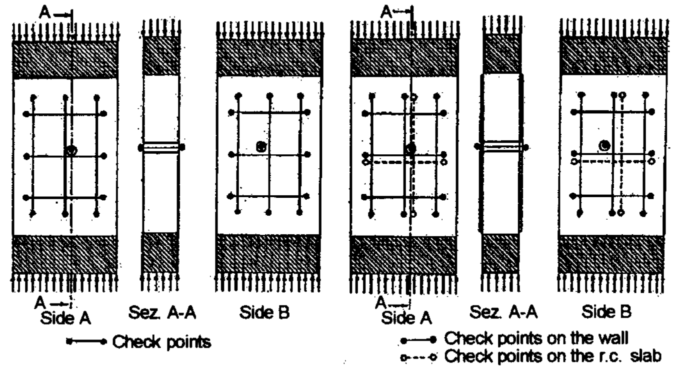


Figure 1: Flat jack test scheme   Figure 2: Compression test for original and reinforced walls.

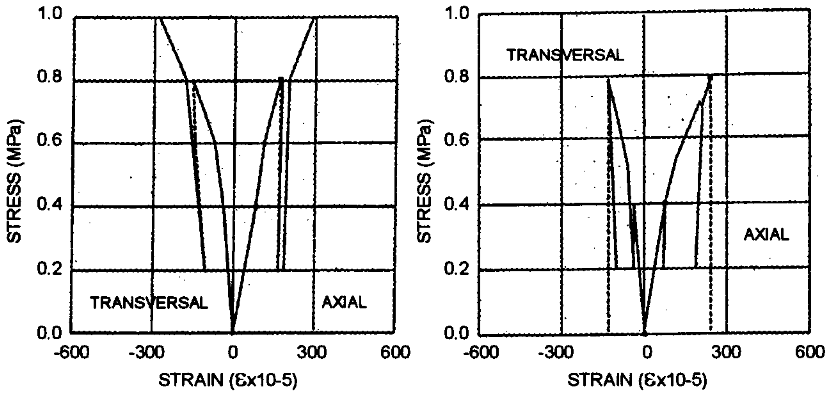


Figure 3: Stress-strain diagrams from flat jack test (left) and compression test (right).

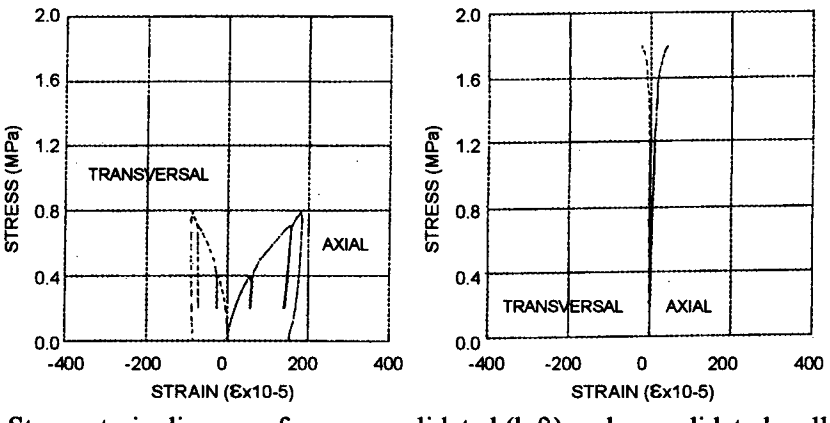


Figure 4: Stress-strain diagrams for unconsolidated (left) and consolidated wall (right).

4. NUMERICAL MODELS

The development of a methodology for the numerical modelling of the repair of existing buildings can give a considerable help to the designer in the choice of the best repairing technique. Of course, due to the high difficulty related to the characterisation of the mechanical parameters, the numerical results, at list at this stage of the research, should be used mainly for the comprehension of the global behaviour of the structure. The local state of stress depends strongly on the local characteristics of the existing walls, and it requires a different analysis carried out using a local scale.
The numerical modelling to test the effectiveness of the reinforcements of masonry walls with thin concrete slabs on the surface presents problems related to the characterisation of the shear transmission between the original walls and the new concrete. In fact, a wall reinforced with concrete slabs on the surfaces presents the same structure of a sandwich panel. The core material of the sandwich, i.e. the original wall, in reality is a non-homogeneous anysotropic material characterised by a strongly non-linear mechanical behaviour. Moreover the behaviour of the joint between the core and the outer layers presents different characteristics from the simple adhesion between two flat surfaces of homogeneous materials. In this case the wall is characterised by a superficial macroscopic roughness, hence interlocking mechanisms takes place for the stress transmission. Finally the steel reinforcements which connect the two outer slabs crossing the inner wall have a strong influence, at least on the ultimate load, because the wall benefits of the confinement state which is generated.
The complexity of the problem requires a step-by-step procedure in the development of the numerical model. One more difficult can occur when geometrical non-linearity are involved. Due to the difficulty related to the characterisation of the input parameters we start with simple models to check the basic behaviour.

4.1 Basic Model

The basic model, taken as reference for the subsequent modelling, is a 1.5 m high wall consolidated with two r.c. slabs 5 cm thick connected by $5\varnothing 6/m^2$ tie-rods.
The specimen involves four different materials, i.e. the concrete at the top and at the bottom, placed to distribute the applied load, the steel reinforcement, which cross the original masonry wall, the special concrete used for reinforcement and the wall itself. We adopted a two dimensional-plane strain model and the evaluation of input data describing mechanical characteristics has been carried out on the base of specific tests. Material parameters for both concrete and steel can be determined with a low effort whereas in-situ compressive tests have been used for the characterization of the existing wall. A curve fitting of experimental results using Drucker-Prager constitutive law permitted to find the following values: elastic modulus E_m = 400 MPa, Poisson coefficient $\nu = 0.2$, friction $\varphi = 45°$, cohesion $c = 1.5$. Concrete for reinforcement has been modelled using a linear elastic law with E_b = 3200 MPa and Poisson coefficient $\nu = 0.2$. Steel reinforcements have been modelled using an elasto-plastic law with E_s = 210000 MPa and yield stress σ_s = 500 MPa.
A crucial task concerns the modelling of the interface between the existing wall and the new concrete. The adopted scheme of the joint (Fig. 5) permits the transfer of shear forces due to the asperities interlocking and a contemporary presence of dilatancy effects. Local effects due to the interaction of steel reinforcements with the masonry have been neglected. However the presence of steel reinforcements has been taken into account and modelled with rods connected to the external sides of the added concrete.

Moreover, the existence of horizontal and vertical symmetry has permitted to discretize only a quarter of the whole specimen, as reported in Fig. 6.

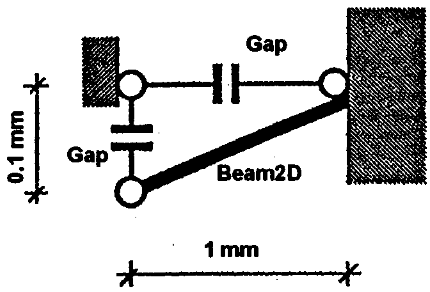


Fig. 5: Concrete-wall interface discretization.

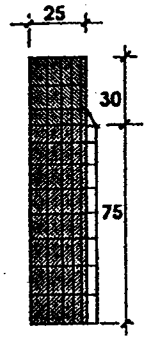


Fig. 6: Concrete and wall discretization.

Load conditions have been applied with a step-by-step procedure, with increments of P=0.1 MPa. Transversal displacements in the middle of the specimen versus the applied vertical pressure due to the load conditions have been compared with experimental tests. Comparisons reported in Fig. 7, evidence a good agreement for the linear part of the diagrams both for original and reinforced walls. The correspondence of the non-linear behaviour is also in good agreement in the first part. For the unreinforced wall the almost perfect correspondence between the curves disappear when the experimental curve present a jump, probably due to a collapse of a brittle masonry element. The numerical model adopted is not able to take into account such effects. However it is noticeable that after the jump the two curves present the same trend till the total collapse is achieved.

Numerical results evidence also that the transmission of the shear forces which takes place at the interface between the masonry and the concrete slabs are concentrated at the top. For any applied load, more than the 90% of the shear between the wall and the reinforcement is concentrated at the top in an area which length is 40 cm.

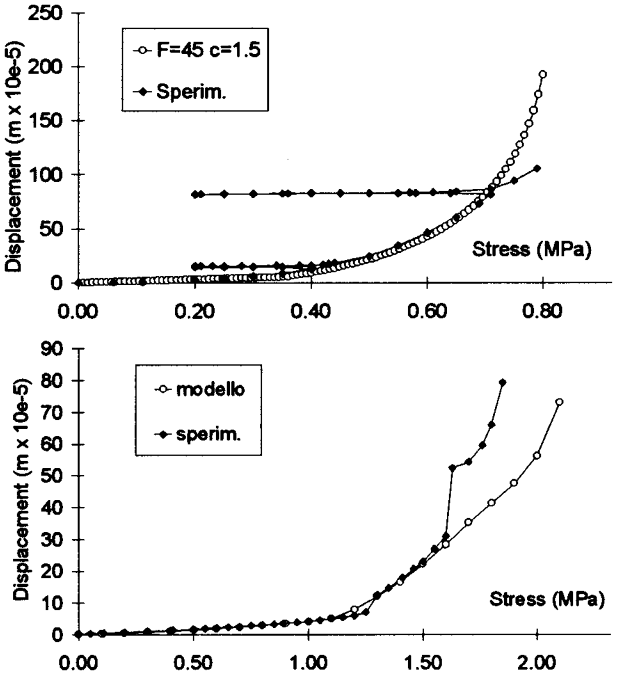


Fig. 7: Comparison of numerical and experimental stress - displacement diagrams. Original wall (top), reinforced wall (bottom).

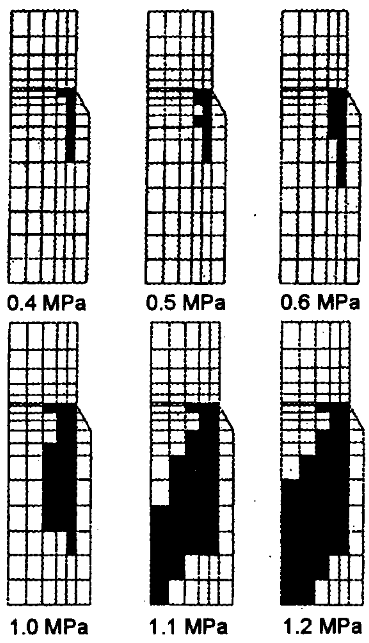


Fig. 8: Plastic zone evolution of the consolidated wall.

It have to be remarked the fact that the reinforcement sustain more than 90% of the applied load, hence it gives a great contribution to the original wall. Even when the system is close to the collapse limit such contribution is greater than 80%. This behaviour is evidenced also in Fig. 8, where the evolution of the plastic zone is represented.

4.2 Variations on the basic model

On the basis of the numerical model previously described, the effects have been investigated of varying some characteristic geometrical parameters, such as:

- area of the connecting steel bars (A), by considering 3, 8 and 10 rods Ø6/m^2;
- thickness of r.c. slabs (S), taken equal to 3, 4, 6, 7 and 8 cm;
- wall's height (H), considered equal to 3 and 4 m.

Tab. 1 shows the combinations of the investigated parameters; the figures where the results are presented as vertical compressive stress versus transversal displacement diagrams are referred to in the lost column.

Table 1: Geometrical variations of the numerical models and numbers of the related figures.

Variations compared to the reference model	Models	Number of tie-rods	Slab's thickness (cm)	Wall's height (m)	*Figures*
	Reference	5Ø6/m^2	5	1.5	
Number of tie-rods	A3	3Ø6/m^2	5	1.5	*Fig. 9*
	A8	8Ø6/m^2	5	1.5	
	A10	10Ø6/m^2	5	1.5	
R.C. slabs' thickness	S3	5Ø6/m^2	3	1.5	*Fig. 10*
	S4	5Ø6/m^2	4	1.5	
	S6	5Ø6/m^2	6	1.5	
	S7	5Ø6/m^2	7	1.5	
	S8	5Ø6/m^2	8	1.5	
Wall's height	H3	5Ø6/m^2	5	3	*Fig. 13*
	H4	5Ø6/m^2	5	4	
Tie-rods and slabs' thickness	S3A3	3Ø6/m^2	3	1.5	*Fig.12*
	S8A10	10Ø6/m^2	8	1.5	
Tie-rods and wall's height	H3A3	3Ø6/m^2	5	3	*Fig. 15*
	H3A10	10Ø6/m^2	5	3	
	H4A3	3Ø6/m^2	5	4	
	H4A10	10Ø6/m^2	5	4	
Slabs' thickness and wall's height	H3S3	5Ø6/m^2	3	3	*Fig. 16*
	H3S8	5Ø6/m^2	8	3	
	H4S3	5Ø6/m^2	3	4	
	H4S8	5Ø6/m^2	8	4	

The models characterized only by the variation of the steel areas showed an increasing ultimate strength as the steel area increases (Fig. 9). The first elastic branch does not significantly depend on the quantity variations of the steel; afterwards, for stress values around 1.1 MPa, the inside core of the masonry wall comes in the plastic field and the connecting rods bear higher stresses. After their yelding (in this case has been observed that this happens always for the upper rod first) the slab is not kept any more and the collapse happens for its sudden push out due to instability phenomena.

As regards the slab thickness, a slight decrement of the ultimate strength has been obsereved as the slab's thickness increases (Fig. 10), up to a value of 2.2 MPa for the thickness of 3 cm and 1.7 MPa for that of 8 cm.

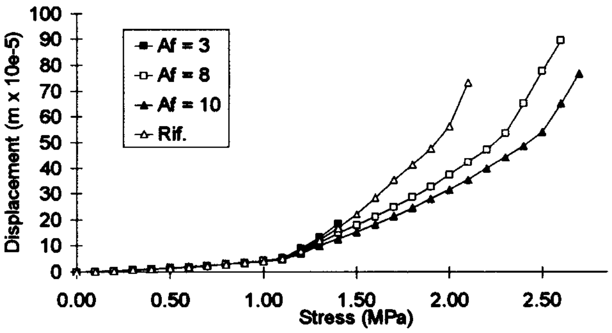


Fig. 9: Comparison among the basic model (Rif.) and the models with different steel areas.

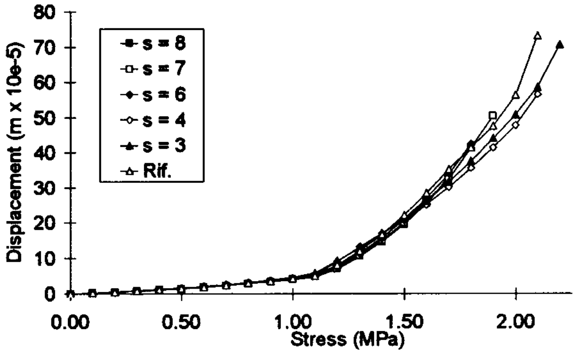


Fig. 10: Comparison among the basic model (Rif.) and the models with different slab thicknesses.

In both the elastic and inelastic branches the global behaviour is substantially that of the masonry, thus the curves are coincident for all the models. However, the collapse happens first in the models with higher slab's thickness than in those with the lower. That is due to global instability phenomena of the slabs caused by two kinds of actions applied to the slabs themselves by the masonry. The action applied by the upper layer of the masonry causes a thrust which the two connecting rods (of which the upper is more stressed) oppose to. Consequently the slab tends to buckle as shown in Fig. 11.a. The shear load P could be considered as concentrated on the slab's upper section connected with the masonry-slab interface, so that the equilibrium of the forces causes the slab's deformation as shown in Fig. 11.b. This stresses the lower connecting rod, which is the first to yield. These two actions are combined in the loading of the slab but, while the first mechanism does not vary significantly as the thickness varies, the second tends to be prevalent over the other when the thickness increases. The results of the elaboration seem to confirm this hypothesis, since the first yelding has been found in the upper rod for the thinner walls and in the lower one in the thicker ones.

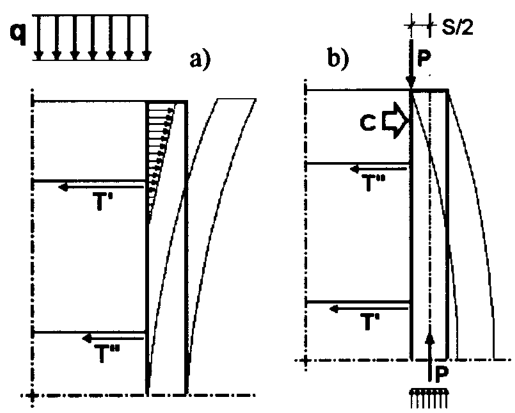


Figure 11: Masonry actions on the slab.

As regards the two numerical models obtained by combining different slabs' thickness with different steel areas (Fig. 12) a loss of stiffness has been observed in the plastic branch of the model with the lower steel area. This corresponds to a sudden instability of the slab after the simultaneous yelding of the two connecting rods. It can be noticed, however, that the failure load did increase, because of the reduction of the slab's thickness, which reduces the effects described in Fig. 11.b. Also the increase of the steel area in the model with S=8 cm produced good effects, raising the resistance from 1.7 to 2.0 MPa. These results confirm the importance of the steel quantities in the strength and stiffness increase in the plastic phase.

The height of the wall has been proved to be a very influential parameter on the global behaviour of the numerical models of the consolidated masonry. The reference model reproduces a panel resulting from an existing wall, having a reduced height (H=1.5 m) as regards those usually interested by repair interventions, since it was subjected to compressive tests. In the models with height of 3 and 4 m (Fig. 13) the ultimate strength increases from 2.1 MPa for H=1.5 m to 2.2 MPa for H=3 m, up to 2.5 MPa for the H=4 m. The increase of strength happens because of a different post-elastic phase evolution: in the reference model, in fact, the plastic zones start from the upper outer elements and join diagonally with those of the lower elements, whereas in the models of the taller panels a central core not yet in plastic phase has been found also near the failure load (Fig. 14). Hence the connecting rods set in the central zone create a restraint for the upper portion, which is more deformable and subjected to more effort. The failure happens quite in this zone and depends on a local instability phenomenon.

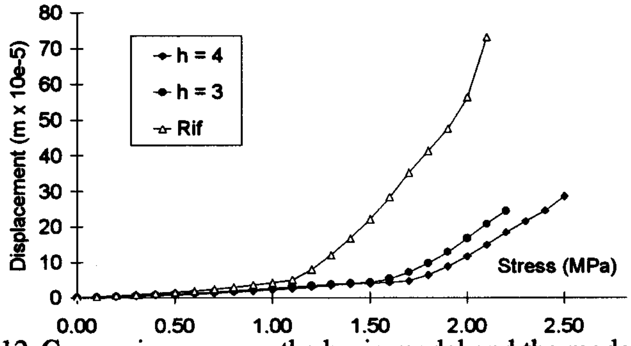


Fig. 12: Comparison among the basic model and the models with (S_{max}, A_{max}) and with (S_{min}, A_{min}).

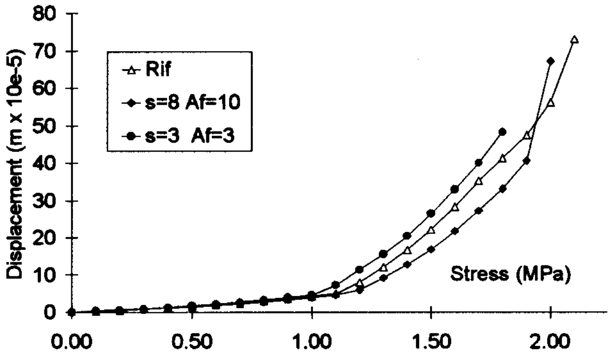


Fig. 13: Comparison among the basic model and the models with diffrent height.

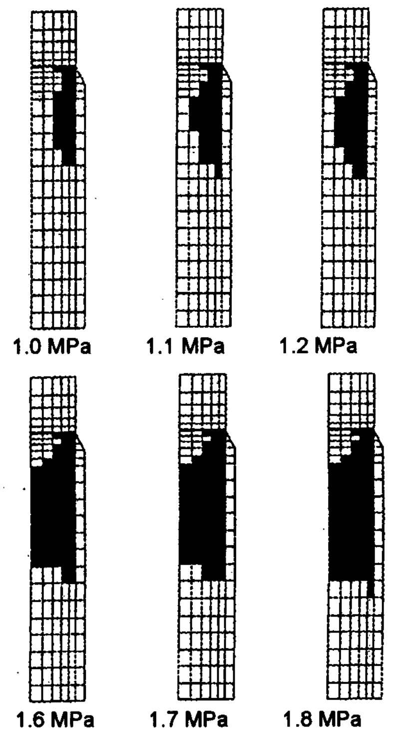


Fig. 14: Plastic zone evolution of the model H3.

Finally, in the tall panels, it has been observed that the increase of steel produces an increase of strength and stiffness (Fig. 15) and that the increase of slab's thickness implies a remarkable increase on strength (Fig. 16) due to a collapse mechanism that happens for local flexure of the slab.

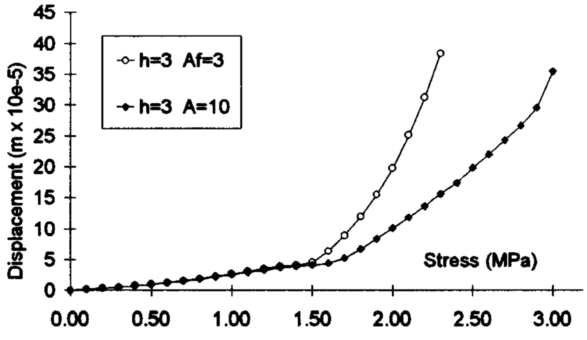


Fig. 15: Comparison between the models H3A3 and H3A10.

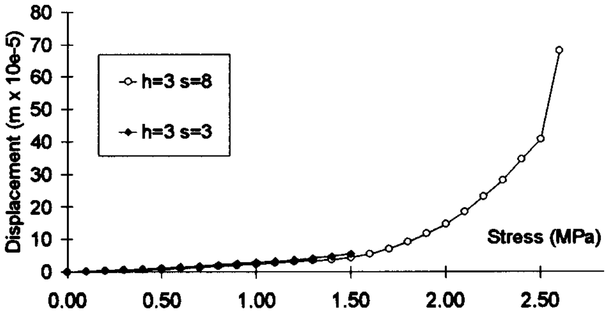


Fig. 16: Comparison between the models H3S8 and H3S3.

5. CONCLUSIONS

The research activity presented in this paper concerns experimental and numerical investigations on old stone masonry walls strengthened by reinforced concrete jackets. The experimental results have been used to verify the effectiveness of numerical models and their capacity to estimate the resistance of original and reinforced walls. Such numerical models, even though quite simple, confirmed the real possibility to integrate and complete the experimental tests.
From the results, some qualitative useful indications concerning the design of the strengthening intervention (number of tie-rods and slab thickness) can be drawn. The improvement of the mechanical characteristics of the masonry as the area of the connecting steel rods increases have been roughly quantified and the behaviour in the post-elastic phase up to the failure have been clarified. Tall masonry walls, for example, have a good behaviour when the tie-rods are closer one another in the upper and lower zones rather than in the central one. Also the slab's thickness has an important role in these walls because their flexural stiffness is increased. Decreasing the wall's height, it would be better not to increase the slab thickness, to avoid a collapse for instability rather than for flexure.

6. REFERENCES

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