SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

PUSHOVER ANALYSIS OF UNREINFORCED MASONRY BUILDINGS WITH DIFFERENT LEVELS OF DIAPHRAGM STIFFNESS

Andrea Penna[1], Ilaria Senaldi[2], Guido Magenes[3], Alessandro Galasco[4]

ABSTRACT

An extensive experimental program carried out at the EUCENTRE included shaking table tests on three full-scale building specimens representative of existing unreinforced stone masonry structures with flexible timber diaphragms. Different strengthening interventions aimed at improving the wall-to-diaphragm connection and increasing the diaphragm in-plane stiffness were implemented on the tested structures.

The experimental response of the strengthened prototypes was reproduced by means of pushover analyses, following an existing equivalent frame macro-element modelling approach .

The obtained numerical results were then compared with the experimental data in terms of the strength capacity of the system and the observed damage pattern associated with the global response of the tested prototypes.

The influence of several factors on the modelling of the global response of the building prototypes was studied by means of sensitivity analyses. The investigation focused on the effect on the global response of the discretization/geometry of the equivalent frame model and of the dispersion of the mechanical properties, which were obtained experimentally from masonry characterization tests and in-plane cyclic tests on masonry structural members.

Furthermore, since it is widely recognized that the response of unreinforced masonry structures can be influenced by the in-plane stiffness of diaphragms, which in existing historical masonry construction often consist of simply supported timber floors and roofs, the effect of the different stiffening interventions on the global behaviour of the prototypes has been studied.

INTRODUCTION

Three full-scale, two storey single-room buildings with timber diaphragms were subjected to shaking table tests as part of an extensive experimental program carried out at Eucentre devoted to the study of the seismic response of unreinforced stone masonry buildings [Magenes et al., 2010c, 2013; Senaldi et al., 2014] within the framework of the 2010–2013 Reluis Project Task AT1-1.1 funded by the Italian Department of Civil Protection. The prototype buildings are characterized by the same geometrical configuration and constructed with double layer undressed stone masory walls (Figure 1, left).

Selected strengthening interventions were applied to the second and third buildings tested, with the purpose to improve at different levels the wall-to-diaphragm connections and to increase the in-plane stiffness of diaphragms, to ensure a global type of response and to prevent the occurrence of out-of-plane local failure mechanisms. The wall-to-diaphragm connection were enhanced with the

[1] EUCENTRE and DICAR, University of Pavia, via Ferrata 1 Pavia (Italy), andrea.penna@unipv.it
[2] DICAR, University of Pavia, via Ferrata 1 Pavia (Italy), ilaria.senaldi@unipv.it
[3] EUCENTRE and DICAR, University of Pavia, via Ferrata 1 Pavia (Italy), guido.magenes@unipv.it
[4] DICAR, University of Pavia, via Ferrata 1 Pavia (Italy), alessandro.galasco@unipv.it

realisation of a steel ring beam at the floor level and a reinforced masonry ring beam at the roof level of the second building prototype. The in-plane stiffness of floor and roof pitches of Building 2 was moderately increased by means of an additional layer of diagonal cross-planks [Magenes et al., 2013]. The strengthening solutions applied to the third building were based on the experimental results of a research program relative to the study of the influence of the in-plane floor stiffness conducted by the University of Trento [Piazza et al, 2008]. A reinforced concrete ring beam was constructed at roof level and the roof pitches were also stiffened by the application of multilayer spruce plywood panels. A reinforced concrete collaborating slab was added to the original flexible floor, creating a mixed r.c.-wood structure, connected to the walls by external anchoring steel plates and through bars anchored into the concrete slab [Senaldi et al., 2014].

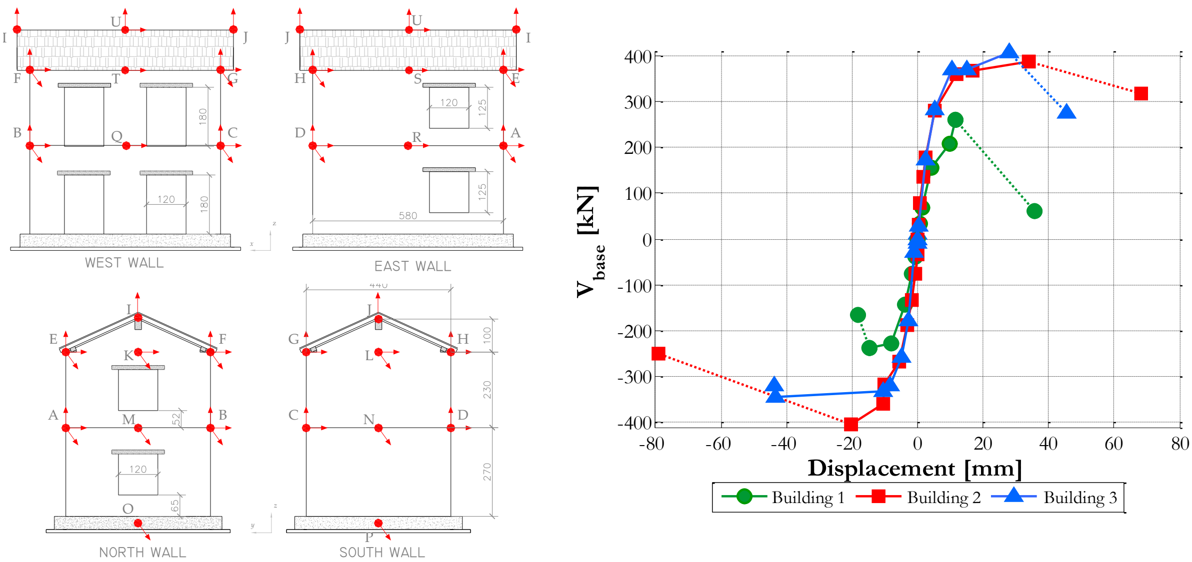


Figure 1: Front views of the walls of the reference geometry and position of the accelerometers installed for the shake table testing (red dots) (left). Experimental seismic response curves of the three prototypes (right).

The shaking table tests showed that the strengthened prototypes could withstand much higher levels of shaking, up to a nominal PGA of 0.60g, than the original unstrengthened configuration (Building 1), by fully exploiting the in-plane capacity of the walls, as depicted in Figure 1 (right) where the comparison between the seismic resistance curves of the three buildings is presented in terms of maximum resisted base shear and of average top floor displacement.

Numerical models have been created following an existing equivalent frame macro-element approach, to replicate the experimental response of the strengthened prototype buildings by means of nonlinear static analyses and nonlinear dynamic analyses. The purpose of the numerical investigation is also to address modelling issues, such as the idealization of the geometry of the building, and consequently the model discretization, and the appropriate representation of the masonry characteristics on which a reliable assessment of the seismic response of a masonry structure depends.

Material properties adopted in the models were derived from characterization tests carried out on masonry wallettes [Magenes et al., 2010b] and in-plane cyclic tests on masonry piers and spandrels [Magenes et al., 2010a, Graziotti et al., 2012]. The geometry of the models was calibrated to reproduce the damage pattern induced by the failure mechanisms activated during the shaking table tests, as well as the hysteretic behaviour of the strengthened prototypes at levels of displacement similar to those of the shaking table tests. The influence of several factors on the modelling of the global response of the building prototypes was investigated, in particular regarding the effect of the discretization/geometry of the equivalent frame model on the replication by means of pushover analyses of the global response of the strengthened prototypes and of the damage level experienced during the experimental tests.

MODELLING APPROACH

The simulation of the nonlinear behaviour of the strengthened specimens has been based on the application of the in-plane equivalent frame modelling of walls, whose out-of-plane contribution to strength and stiffness is neglected. The structural member response was analysed by means of two alternative models consisting of a refined nonlinear macro-element and a conventional elasto-plastic beam, respectively.

In the TREMURI computer program, the nonlinear macro-element model allows representing the two main in-plane masonry failure modes, bending-rocking and shear-sliding mechanism including friction, as well as their mutual interaction. The shear-sliding damage evolution, which controls the strength deterioration and the stiffness degradation, and the rocking mechanism, with toe crushing effect, are implemented in the nonlinear macro-element mechanical model, by means of internal variables. Literature works [Lagomarsino et al., 2013; Penna et al., 2014] describe in detail the algorithms embedded in the model.

In addition to the geometrical characteristics, the macro-element is defined by eight parameters representative of an average behaviour of the masonry panel: density, elastic modulus in compression, shear modulus, compressive strength, shear strength, a non-dimensional coefficient controlling inelastic deformations, global equivalent friction coefficient and a factor controlling the softening phase.

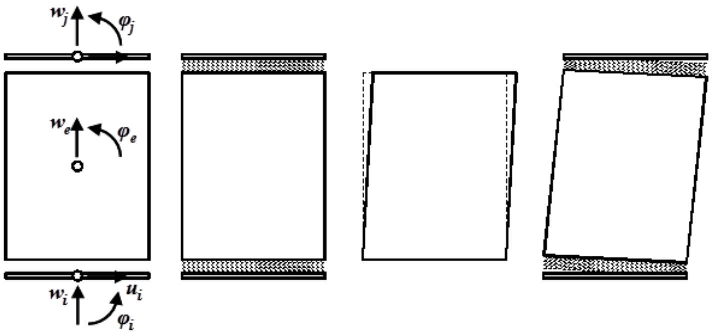


Figure 2. Kinematics of the macro-element model [Lagomarsino et al., 2013].

The results from nonlinear static (pushover) analyses of the mechanical macro-element models have been compared with those obtained by bilinear elasto-plastic beam elements. The elasto-plastic element model implemented in TREMURI is a nonlinear beam element with six degrees of freedom, with limited lateral strength. The nonlinear behaviour is activated when one of the nodal generalized forces reaches its maximum value, estimated according to the minimum of the flexural- shear criteria.

The mechanical properties assigned to the masonry macro-elements are derived from the mean values obtained from the characterization tests [Magenes et al., 2010b] (Table 1), which have not been changed in order to evaluate the effect of the modified geometry on the results of the nonlinear static analyses.

Table 1. Summary of masonry mechanical properties after characterization tests in MPa (6 specimens per test)

	Compressive strength (f_m)	Young's modulus (E)	Tensile strength for diagonal shear (f_t)	Shear modulus (G)
Mean	3.28	2537	0.138	841
St. Dev.	0.26	378.7	0.030	141.5
c.o.v.	8%	14.9%	22.1%	16.8%

The failure of the elasto-plastic element occurs at the attainment of the ultimate drift limit associated with the failure mechanism (shear or bending failure), defined according to the Italian Building Code [NTC08, 2008]. After failure the contribution of the element to the global equilibrium is considered solely for its capacity to carry vertical loads.

Nonlinear beam elements have been introduced in the models to account for the presence of both reinforced masonry and reinforced concrete ring beams and of the wall to diaphragm connections at floor level. The diaphragms are defined as orthotropic membranes finite elements with four nodes and were modelled as infinitely rigid due to the presence of the collaborating reinforced concrete slab at first floor and of the multi-layer spruce plywood panels at roof level. The mechanical properties of the floor diaphragms were defined based on the results obtained in previous experimental investigations performed at the University of Trento regarding the seismic performance of similarly stiffened timber diaphragms [Piazza et al., 2008].

NUMERICAL MODEL CHARACTERISTICS

In the equivalent frame approach each masonry wall is subdivided into an assembly of deformable masonry panels (piers and spandrels), in which the deformation and the nonlinear response are concentrated, and rigid nodes, which connect the deformable ones.

Two different geometrical discretizations of the numerical model were considered to define the deformable members, in which the effective height of the piers varies. The former discretization, namely the *STD* model, accounts for the deformability of nodal regions, as shown in Figure 3, whereas the *MOD* model is calibrated according to the damage pattern exhibited by the prototypes during the experimental tests (Figure 4). In the frame-type representation of the building of model MOD, masonry piers are assumed to have the same height of the adjacent openings, allowing better capturing the damage mechanisms observed during the shaking table tests.

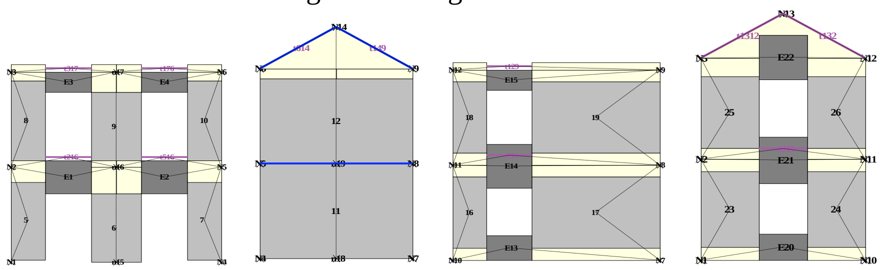


Figure 3. Geometry of the model STD, from left: West, South, East and North walls, Building 3.

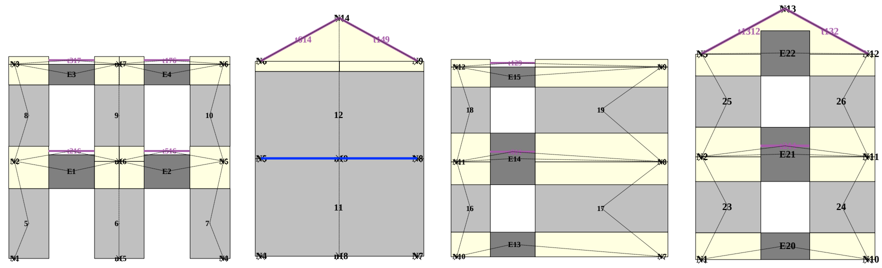


Figure 4. Geometry of the model MOD, from left: West, South, East and North walls, Building 3.

PUSHOVER ANALYSES

As proposed in the current building codes such as the Italian Building Code [NTC08] and Eurocode 8 [EC8], the pushover analyses were performed subjecting the models of Buildings 2 and 3 to horizontal nodal forces applied at the centre of mass at each floor level, parallel to the direction of analysis. The force distributions considered in the current building codes to perform pushover analyses are typically modal and uniform. The former is able to represent the structural dynamic amplification which increases the action on higher stories; the latter can describe the behavior of a building after the activation of a soft story mechanism at the base. Besides the uniform amd modal force patterns, an adaptive pushover was also performed for the nonlinear macro-element models. The adaptive pushover algorithms were developed [Galasco et al., 2006] in order to reproduce the influence of damage evolution in the force redistribution associated with the dynamic response. The evolution of damage modifies indeed the dynamic properties of the structure; hence, the force distribution, which depends on the modal amplification, should also vary according to the damage propagation into the building model. The adaptive pushover procedure adopted in this work is based on the actual deformed shape at each load step. The displacement shape automatically takes into account the effect of damage evolution in the nonlinear analysis.

A comparison of the results of the pushover analyses was made to highlight the influence of the geometrical discretization as well as of the applied force pattern on the evaluation of the capacity of the structure and on the reproduction for both modelling approaches considered. The following Figure 5 and Figure 6 present the comparison of the pushover curves obtained for the models in which the structural elements are implemented both as elasto-plastic or with nonlinear macro-elements, respectively for Building 2 and Building 3.

As shown by the pushover curves obtained for the two structures, the models in which the geometrical discretization accounts for the deformability of nodes (STD models) tend to underestimate the capacity of the structure with respect to the MOD models, in which the geometry of the structural elements is determined based on the damage pattern actually exhibited during the experimental tests. Furthermore, the STD models are not able to reproduce the damage observed during the tests, particularly in terms of damage mechanisms activated.

Considering the results obtained for the model in which the structural elements are defined as elasto-plastic beam elements, the models underestimate the capacity of the strengthened prototypes in comparison with their experimental seismic resistance, particularly when considering Building 2 where the maximum resisted base shear is approximately 72% of the base shear attained during the shaking table tests for the model MOD_B with uniform force pattern (*Unif.*). The MOD_B model of Building 3 approximates fairly well the negative branch of the envelope of the experimental seismic response of the structure, although the initial stiffness is overestimated.

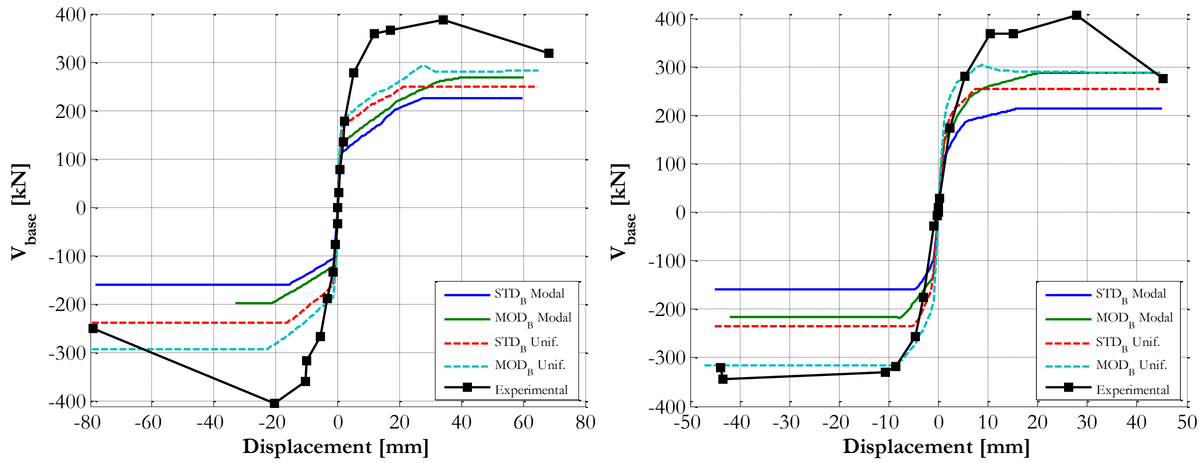


Figure 5. Comparison of numerical and experimental pushover curves obtained for models with elasto-plastic elements: Building 2 (left) and Building 3 (right).

As depicted in Figure 6, the macro-element model approximates fairly well the actual response of the Buildings 2 and 3, in terms of initial stiffness and maximum resisted base shear, the latter being for example approximately 80% to 94% of the base shear attained during the shaking table tests in which the structures reached the ultimate limit state, when applying a modal force pattern in the pushover analyses.

The MOD_M model of Building 2, when a uniform force pattern is applied, reproduces fairly well the experimental response of the prototype up to the maximum base shear resisted during the test at nominal PGA of 0.60g, in terms of initial stiffness and of strength deterioration as the damage level progresses, as presented in Figure 6 (left). However the maximum base shear attained during the last stage of experimental testing (at nominal PGA of 0.70g), when the structure reached the ultimate limit state, is still underestimated.

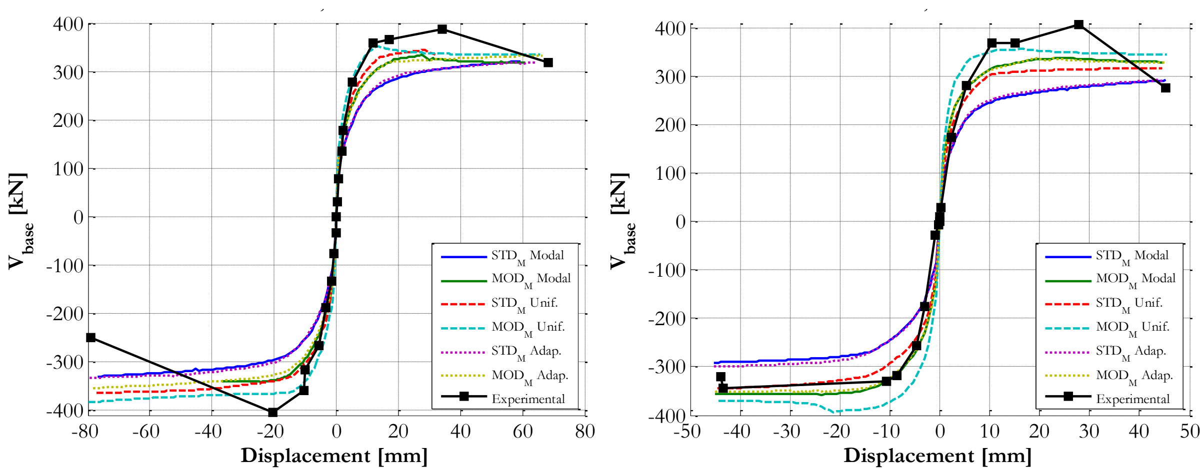


Figure 6. Comparison of numerical and experimental pushover curves obtained with nonlinear macro-elements: Building 2 (left) and Building 3 (right).

Considering the analysis of Building 3, the MOD_M model with mass proportional force pattern overestimates the initial stiffness of the structural system as well as the capacity in terms of base shear in the negative branch of the pushover curve. A better approximation in terms of initial stiffness and of stiffness degradation in the nonlinear ange is achieved by the MOD_M model when a modal force pattern is applied. The slight overestimation of the capacity observed in negative branch of the pushover curve is of approximately 15% for uniform force pattern and of 4% in the modal force pattern and in the adaptive pushover analyses.

A similarity of results is shown when comparing the adaptive pushover analyses, perfomed for the models with nonlinear macro-elements for the STD and MOD geometrical configurations, with the pushover analyses with modal force pattern, given the limited dimensions of the building models. The similarities are evident when considerding the initial stiffness and maximum resisted base shear as well as damage patterns exhibited when the ultimate limit state is reached.

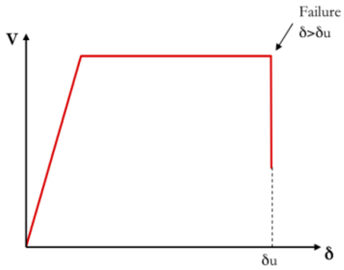
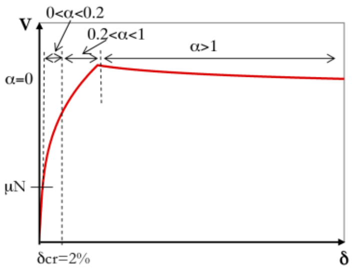
The fairly good approximation of the experimental results obtained with the nonlinear macro-element approach is also visible when considering the damage mechanisms activated and the deformations sustained by the structure. The damage level reached in each structural element is distinguished as presented in Table 2 where the different levels are reported for the elasto-plastic and the macro-element models respectively. In the case of macro-elements the damage levels refers to the attainment of failure due to shear, based on the definition of a shear damage parameter α embedded in the model that is equal to zero when the element is undamaged, it reaches the value of 1 in correspondence of the maximum shear strength and increases above 1 in the softening phase.

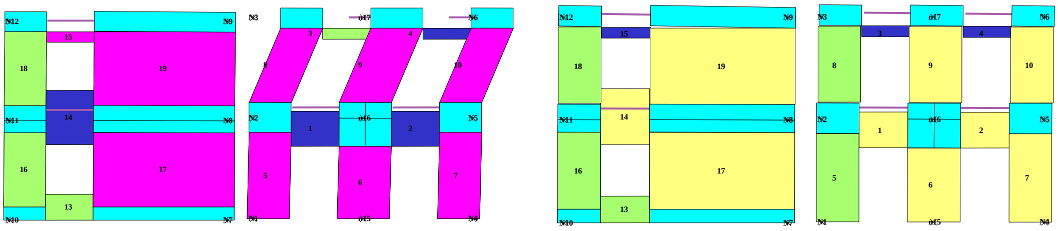
Considering the in-plane response of the piers of the longitudinal façades, the macro-element models well capture the shear failure of the squat pier at the base of the East façade, when the applied forces are directed towards South, and the rocking type of response of the slender piers in the West façade exhibited by the second and third building prototypes when reaching the ultimate limit state, as shown from Figure 7 (right) to Figure 10 (right) that represent the damage level obtained in the analyses with modal force pattern.

As depicted in Figure 7 (left) and Figure 8 (left) for Building 2, in the elasto-plastic element model a concentration of deformation of the piers at the top floor of the West façade, which are subjected to cracking due to bending, is exhibited, hence the damage mechanisms activated in the longitudinal walls during the experimental tests are not captured . As an example, the squat piers of the East façade are similarly subjected to cracking due to bending rather than to severe damage due to shear, which was the damage mechanism actually exhibited experimentally at the attainment of the maximum resisted base shear.

The nonlinear macro-element model is influenced by the geometrical discretization of the structure, in particular in the reproduction of the concentration of damage at the first storey. Only in the Mod model, the base pier of the west facade is affected by the same shear failure mechanism occurred experimentally, as shown in Figure 8 (right), whereas in the STD model the longitudinal walls are only affected by a moderate level of damage that does not corresponde to the damage pattern exhibited by the prototypes when the ultimate limit state was reached.

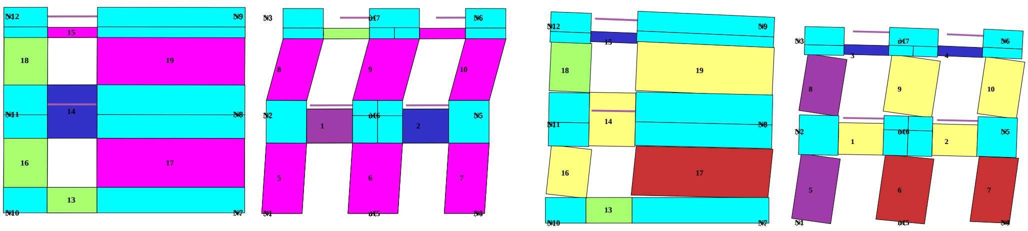
Table 2. Colour legend for damage pattern in macro-element and elasto-plastic element models.

Damage in elasto-plastic models		Shear damage in macro-element models	
	Damage level		Damage level
	negligible or slight damage		$\alpha = 0$; negligible damage
	plasticity governed by shear strength criteria		$0 < \alpha < 0.2$; slight shear cracking (stiffness degradation)
	plasticity governed by flexural strength criteria		$0.2 \leq \alpha < 1$; moderate to significant shear cracking (stiffness degradation)
	shear failure ($\delta>\delta_{u\ shear}$)		$\alpha > 1$; post-peak softening phase
	failure due to bending ($\delta>\delta_{u\ flex}$)		no compression
	no compression		
			



Elasto-plastic model Nonlinear macro-element model

Figure 7. Model STD of Building 2, damage patterns in the elasto-plastic model (left) and nonlinear macroelement model (right): East and West façades, when loaded in the South (positive) direction

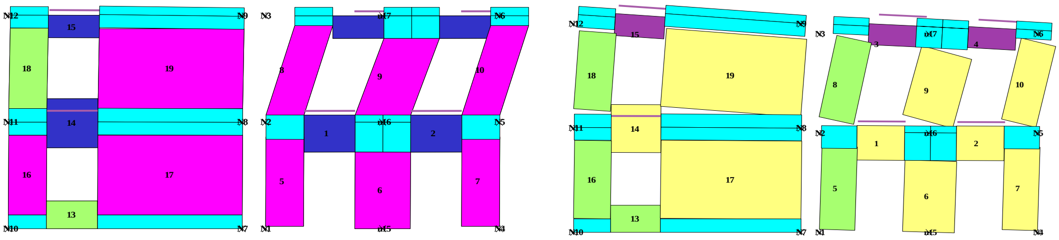


Elasto-plastic model Nonlinear macro-element model

Figure 8. Model MOD of Building 2, damage patterns in the elasto-plastic model (left) and nonlinear macroelement model (right): East and West façades, when loaded in the South (positive) direction

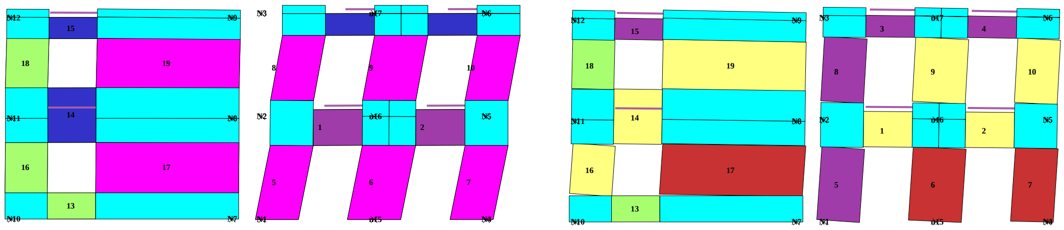
Similar damage patterns are obtained for Building 3, for the STD (Figure 9) and MOD geometrical configurations (Figure 10), in the elasto-plastic and in the nonlinear macroelement models with modal force pattern.

The lower strength capacity of the elasto-plastic model is due in part to the fact that the masonry spandrels are weaker than in the macro-element model, and in part to the fact that the kinematic modelling of the macro-element is more accurate in describing the coupling between rocking and axial deformation (uplift in piers, horizontal dilation in spandrels) in the elements.



Elasto-plastic model Nonlinear macro-element model

Figure 9. Model STD of Building 3, damage patterns in the elasto-plastic model (left) and nonlinear macroelement model (right): East and West façades, when loaded in the South (positive) direction



Elasto-plastic model Nonlinear macro-element model

Figure 10. Model MOD of Building 3, damage patterns in the elasto-plastic model (left) and nonlinear macroelement model (right): East and West façades, when loaded in the South (positive) direction

Considering the displacement response of the macro-element models of the strengthened prototype buildings, the deflected shapes depict that in Building 2 and 3 deformations are characterized by a trend that is similar to the envelope of displacements in elevation attained at ultimate limit state during the experimental tests. A good approximation of the deflected shapes is obtained particularly in the case of the West façade for Buildings 2 and 3, considering for instance the results of the pushover analyses of model MOD with modal force pattern, although some differences are exhibited when comparing experimental and numerical results (Figure 11).

The displacement profiles in elevation of the longitudinal walls, obtained numerically, were produced by plotting the wall displacements at each floor level occurring synchronously to an average top floor displacement identical to that experienced by each structure at the attainment of the maximum base at the last stage of the experimental tests, in which the ultimate limit state of the structure was reached.

However, as concerns the elasto-plastic models, the deflected shapes obtained further confirm that in this case this modelling approach does not reproduce well the behaviour of the structures in terms of displacement response, in particular when considering Building 3. The deformations of the elasto-plastic model of Building 3 (Figure 11) are concentrated at the top floor showing a soft storey type of mechanism, instead of showing greater drifts at first floor due to the shear failure mechanism exhibited by the East façade during the shaking table tests. Similar trends were obtained also in the analyses performed with uniform force pattern and with the models with STD geometrical discretization.

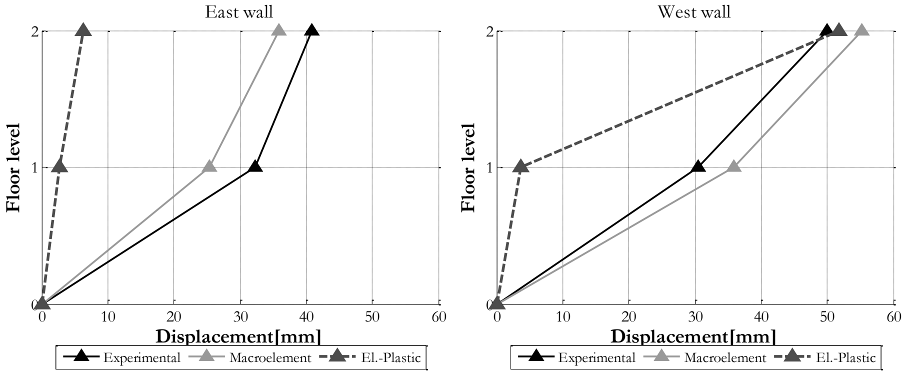


Figure 11. Comparison of the deflected shapes of the longitudinal walls, obtained from the numerical analyses with modal force pattern and the experimental test results. From left: East and West façades of Building 3.

It should be noted however, as shown for example in Figure 12 for Building 2, when comparing the experimental and numerical displacement response, that the macro-element numerical models tend to slightly overestimate the torsional response of the structure, whereas during the experimental tests the response of the buildings was characterized by small torsional components of the in-plane deformed shapes.

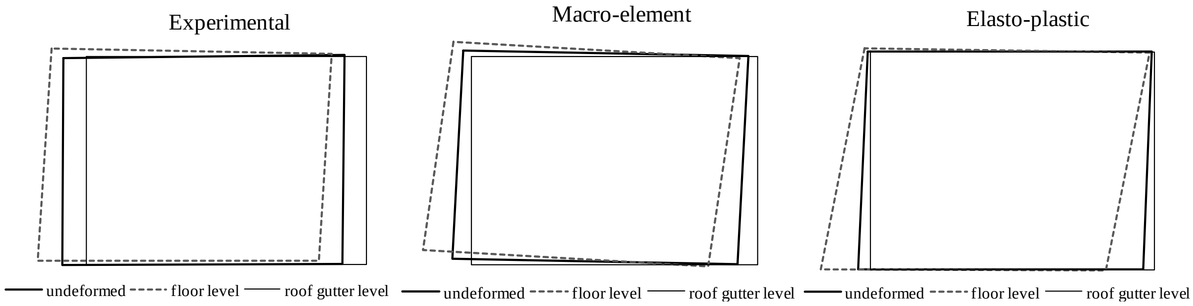


Figure 12. Comparison of the normalized deformed shapes in plan of Building 2, for modal force pattern pushover analyses.

The macro-element and elasto-plastic modelling approaches resulted to be different also in terms of distribution of the internal forces, in particular of axial forces, as the applied horizontal loads increase. The distribution and variation of the axial forces in the structural elements were compared to investigate the role of spandrels and ring beams in the numerical evaluation of the global response of the tested structures. The axial forces have been computed for each building model at the initial step of the nonlinear static analysis and at the attainment of the ultimate limit state, while the variation ΔN is evaluated as the difference among the previous two. Figure 13 (right) shows, as an example, the comparison for Building 2 of the variation of axial forces from the initial stage up to the ultimate limit state, when the structure is pushed longitudinally from North to South.

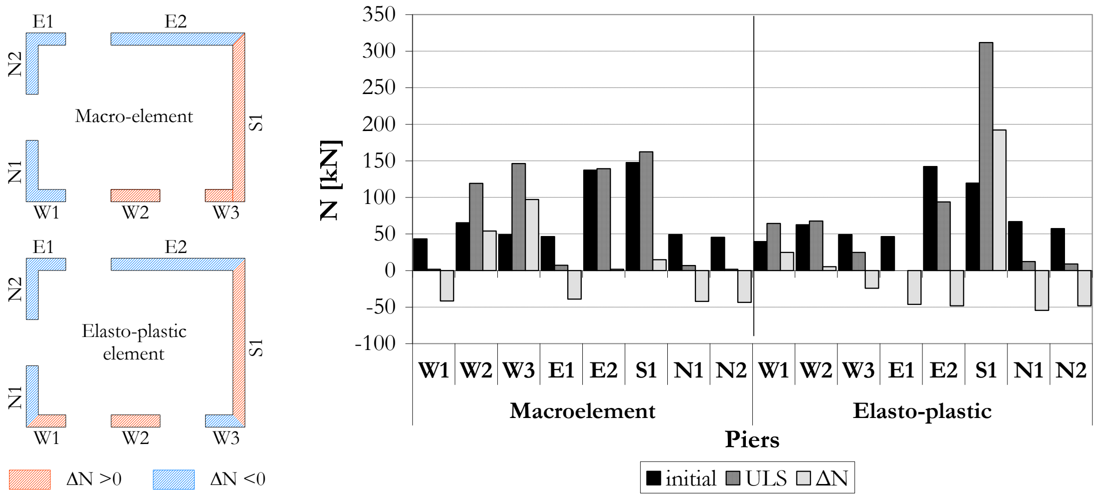


Figure 13. Building 2, schematic representation of the variation of axial forces ΔN in the piers at floor level (left) and distribution of axial forces N at the base of piers (at the initial step of the nonlinear static analysis and at ultimate limit state ULS) and variation of axial forces ΔN for macro-element and elasto-plastic models (right).

In the two buildings, when considering the macro-element modelling approach, the North façade together with the returning piers W1 and E1 are progressively unloaded while the spandrels and ring beams, when present, allow transferring the loads to the South façade and to the W2 and W3 piers of the West wall. In the elasto-plastic models instead, the axial load is increased in the South façade as well as the compression in piers W1 and W2 of the West wall, while the East and North walls are unloaded, as schematically represented in Figure 13. The macro-element model is hence able to represent the effect of a global frame response, whereas in the bilinear elasto-plastic model the masonry piers behave to some extent similarly to flanged cantilever due to the limited role of spandrels in transmitting shear forces.

NONLINEAR DYNAMIC

The global behaviour of the strengthened building prototypes was further simulated by means of nonlinear dynamic analyses, in order to accurately reproduce the hysteretic behaviour of the strengthened prototypes at levels of acceleration similar to those of the shaking table tests. The numerical models were calibrated to capture also the evolution of the exhibited damage pattern at each stage of the dynamic testing.

The influence of several factors on the modelling of the global response of the building prototypes was investigated by means of sensitivity analyses. The investigation focused in particular on the effect on the global response of the discretization/geometry of the equivalent frame model and of the dispersion of the mechanical properties, which were obtained experimentally from masonry characterization tests and in-plane cyclic tests on masonry structural members.

Based on the outcomes of the sensitivity analyses, an optimal model configuration has been set, which allows approximating fairly well the experimental behaviour of the strengthened building prototypes. Such optimal approximation of the experimental results was achieved implementing a numerical model in which the mechanical properties are derived from the calibration of the macro-element based on the result of the in-plane cyclic tests on piers. The strength parameters as well as the equivalent viscous damping coefficients had to be further reduced for the tests at higher shaking intensity to better capture the dissipative behaviour of the strengthened prototypes and to control the strength deterioration and the stiffness degradation as the damage progresses.

The nonlinear macro-element proved to be an efficient tool for the reproduction of the experimental dynamic response of the strengthened masonry building prototypes.

As a result of the nonlinear dynamic analyses, the hysteretic behaviour of the strengthened buildings was fairly well replicated, at lower level of seismic intensity as well as during the last stages of tests when a higher level of damage was accumulated as presented in Figure 14 (left) for Building 3.

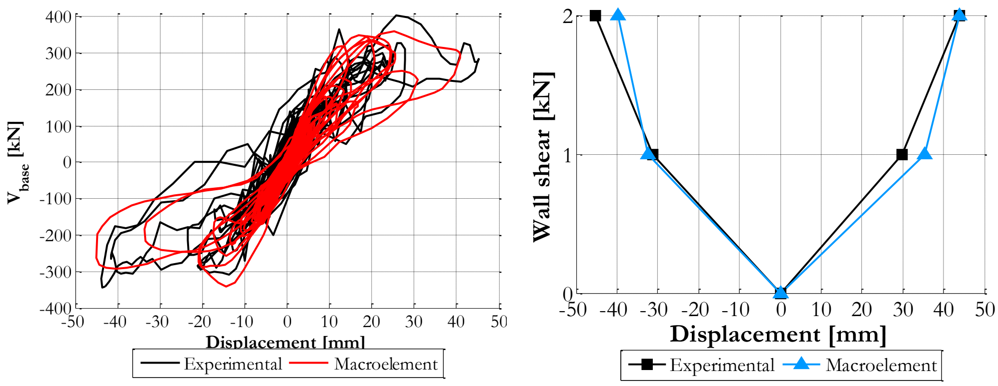


Figure 14. Comparison of experimental data and numerical results from nonlinear dynamic analyses of Building 3 at nominal PGA of 0.60g: hysteresis curves (left), envelope of deformed shapes in elevation (right).

Comparing the results of the macro-element models of the strengthened prototype buildings with their experimental displacement response, the envelope of deflected shapes is fairly well approximated for each stage of dynamic testing, for example as depicted in Figure 14 (right) for Building 3 for the tests at nominal PGA of 0.60g, with an error of prediction of the average floor displacement approximately up to 15%.

CONCLUSIONS

The numerical simulations, by means of pushover analyses, showed a good consistency of the numerical results with the experimental response of the buildings, when applying a macro-element modelling approach. The macro-element model reproduces with sufficient approximation the global response of the prototypes, also in terms of damage mechanisms activated and of deformations experienced. It has also been shown that the numerical simulations based on elasto-plastic behaviour of the structural elements tend to produce a rather conservative prediction of the strength capacity of the systems while, at the same time, it does not always reproduce satisfactorily the global response of the tested prototypes, since the damage mechanisms activated during the last stages of the experimental tests are not well captured. It is worth to mention on this regard that in the bilinear elasto-plastic approach the interaction between axial and shear forces as well as axial forces and moments are governed by the strength criteria rather than representing continuously the damage evolution and degradation of stiffness, as implemented in the macro-element approach. Furthermore, in the elasto-plastic element models issues are related to the role of spandrels in the redistribution of applied forces within the structural members.

The nonlinear macro-element model was also used to further simulate the global behaviour by means of nonlinear dynamic analyses, in order to accurately reproduce the hysteretic behaviour of the strengthened prototypes at levels of acceleration similar to those of the shaking table tests. The sensitivity analyses were performed to evaluate the impact of the geometrical discretization of the models on the overall stiffness and on the capacity of the structural systems, as well as the necessity to adjust the masonry mechanical parameters to further control the strength deterioration and the stiffness degradation, although the latter are already accounted for in the algorithms embedded in the Tremuri software. Based on the outcomes of the sensitivity analyses, the nonlinear dynamic analyses performed using the macro-element models showed a good consistency of the numerical results with the experimental response of the building filling the residual gap between the experimental results and pushover results.

ACKNOWLEDGEMENT

The numerical research project was funded by the Italian Department of Civil Protection, through the 2010-2013 Reluis Project Task AT1-1.1.

REFERENCES

EN 1998-3 (2005) Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings, CEN, Brussels.

Galasco, A., Lagomarsino, S., Penna, A. (2006). "On the use of pushover analysis for existing masonry buildings" *Proceedings of the 1st European Conference on Earthquake Engineering and Seismology*, Geneva

Graziotti, F., Magenes, G., & Penna, A. (2012). "Experimental cyclic behaviour of stone masonry spandrels", *Proceedings of the 15th World Conference on Earthquake Engineering*, Lisbon

Lagomarsino S, Penna A, Galasco A, Cattari S., 2013. "TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings." *Engineering Structures*, 56: 1787-1799

Magenes, G., Della Fontana, A. (1998). "Simplified non-linear seismic analysis of masonry buildings", *Proceedings of the British Masonry Society*, 8: 190-195.

Magenes G., Galasco A., Penna A., Da Paré M. (2010a) "In-plane cyclic shear tests of undressed double leaf stone masonry panels", *Proceedings of the 14th European Conference of Earthquake Engineering*, Ohrid.

Magenes G., Penna, A., Galasco A., Rota M. (2010b) "Experimental characterisation of stone masonry mechanical properties", *Proceedings of the 8th International Masonry Conference*, Dresden.

Magenes, G., Penna, A., and Galasco, A., (2010c). "A full-scale shaking table test on a two storey stone masonry building", *Proceedings of the 14th European Conference on Earthquake Engineering*, Ohrid

Magenes, G., Penna, A., Rota M., Galasco, A., and Senaldi, I. (2013). "Shaking Table Test of a Strengthened Full-Scale Stone Masonry Building with Flexible Diaphragms", *International Journal of Architectural Heritage: Conservation, Analysis, and Restoration*, 8(3): 349-375

NTC08 (2008) Norme tecniche per le costruzioni, Ministero delle Infrastrutture e dei Trasporti, Decreto Ministeriale del 14 gennaio 2008, Supplemento ordinario alla G.U. n. 29 del 4 febbraio 2008 – in Italian.

Penna A, Lagomarsino S, Galasco A. (2014). "A nonlinear macro-element model for the seismic analysis of masonry buildings", *Earthquake Engineering and Structural Dynamics*, 43(2):159-179

Piazza, M., Baldessari, C. and Tomasi, R. (2008). "The role of in-plane floor stiffness in the seismic behaviour of traditional buildings." *Proceedings of the 14th World Conference on Earthquake Engineering*. Beijing

Senaldi, I. Magenes, G., Penna, A., Rota M., and Galasco, A. (2013). "The effect of stiffened floor and roof diaphragms on the experimental seismic response of a full scale unreinforced stone masonry building", *Journal of Earthquake Engineering*, 18(3):407-443