EQUIVALENT-FRAME MACRO-ELEMENT SIMULATION OF SHAKING TABLE TESTS ON UNREINFORCED STONE MASONRY BUILDINGS WITH STRENGTHENING INTERVENTIONS

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Abstract. Simplified nonlinear numerical models have been created to replicate the results obtained in an extensive testing program carried out at the EUCENTRE. The experimental and numerical research was aimed at the evaluation and reduction of the seismic vulnerability of existing stone masonry structures and included shaking table tests on three full-scale building prototypes, representative of existing unreinforced stone masonry structures with flexible timber diaphragms and characterized by different interventions meant to improve the wall-to-diaphragm connection and increase the diaphragm stiffness. Pushover analyses and nonlinear dynamic analyses of equivalent frame macro-element models, were performed to simulate the seismic response of the strengthened prototypes. The models were suitably calibrated to reproduce the cracking pattern induced by the failure mechanisms activated during the shaking table tests, as well as the hysteretic behaviour of the strengthened prototypes at the levels of acceleration of the shaking table tests. The nonlinear time-history analyses were performed following the experimental testing sequence, i.e. explicitly accounting for the damage cumulated at each stage of the tests. 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 variability of the mechanical properties, which were obtained experimentally from masonry characterization tests and in-plane cyclic tests on masonry structural members.
1 INTRODUCTION

Within the framework of the 2010–2013 Reluis Project Task AT1-1.1 funded by the Italian Department of Civil Protection and devoted to the evaluation and reduction of the seismic vulnerability of existing masonry structures, extensive experimental and numerical research was devoted to the study of the seismic response of unreinforced stone masonry buildings [1, 2, and 3].

The main part of the experimental program has been devoted to the shaking table tests on three full-scale, two-story, single-room prototype buildings made of undressed double-leaf stone masonry. The prototypes are all designed with the same geometrical configuration and construction materials (Figure 1, left), however selected strengthening interventions were applied to Building 2 and Building 3.

The applied interventions were meant to improve the wall-to-diaphragm connections and to increase the in-plane stiffness of diaphragms and hence permitted to ensure a global type of response and to prevent the occurrence of out-of-plane local failure mechanisms. The wall-to-diaphragm connection of Building 2 were enhanced with the 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 [2].

The strengthening solutions applied to Building 3 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 [4]. 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 [3].

The experimental results obtained from the shaking table tests showed that the original un-strengthened configuration (Building 1) could withstand much lower levels of shaking intensity with respect to the second and third prototypes that fully exploited 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.
Equivalent frame macro-element simulation of shaking table tests on unreinforced stone masonry buildings with strengthening interventions

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 and dynamic analyses. The purpose of the numerical investigation is also to address modelling issues, such as the idealization of the geometry of the equivalent frame, 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 wallets [5] and in-plane cyclic tests on masonry piers and spandrels [6, 7]. 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 acceleration similar to those of the shaking table tests. Sensitivity analyses were carried out in order to investigate the influence of several factors on the modelling of the global response of the building prototypes, such as the discretization/geometry of the equivalent frame model and the dispersion of the mechanical properties, obtained experimentally from masonry characterization tests and in-plane cyclic tests.

2 NONLINEAR MACRO-ELEMENT MODELLING APPROACH

2.1 Nonlinear macro-element implemented in TREMURI

The equivalent frame approach adopted for the simulation of the nonlinear behaviour of the strengthened specimens is based on the use of a refined nonlinear macro-element, implemented in the TREMURI computer program [8]. According to this approach, each resistant masonry wall is subdivided into a set of deformable structural elements, in which the deformation and the nonlinear response are concentrated, and rigid portions, which connect the deformable ones. The nonlinear macro-element is able to represent the two main in-plane masonry failure modes, bending-rocking and shear mechanism including friction, as well as their mutual interaction. This mechanical model considers, by means of internal variables, the shear-sliding damage evolution, which controls the strength deterioration and the stiffness degradation, and rocking mechanism, with toe crushing effect. The algorithms embedded in the model are described in detail in literature works [8, 9].

The macro-elements are defined by the geometrical characteristics as well as material properties, represented by eight parameters: 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. The macro-element parameters should be considered as representative of an average behaviour of the masonry panel. Figure 2 describes the kinematics of the macro-element.

![Figure 2: Kinematics of the macro-element model [9]](image)

Figure 2: Kinematics of the macro-element model [9].
2.2 Characteristics of numerical models

The numerical models were differently defined depending on the type of analysis to be performed, being either nonlinear static analyses (pushover) or nonlinear dynamic analyses.

As pushover analyses are concerned, the mechanical properties assigned to the masonry macro-elements are derived from the mean values obtained from the characterization tests [4], which have not been changed in order to evaluate the effect of the modified geometry on the results of the nonlinear static analyses. Since a preliminary discretization of the numerical model, accounting for the supposed deformability of the nodal regions, underestimated the capacity of the strengthened prototypes in comparison with their experimental seismic resistance, the geometry of the macro-element was defined according to the damage pattern exhibited in the experimental tests. In the frame-type representation of the building (Figure 3) masonry piers are assumed to have the same height of the adjacent openings, allowing to better capture the damage mechanisms observed during the shaking table tests. In the case of nonlinear dynamic analyses, the characteristics of numerical models were defined also based on the results obtained from sensitivity analyses, which will be described in detail in the following sections.

![Figure 3: Geometry of the model, from left: West, South, East and North walls, Building 3.](image)

To account for the presence strengthening interventions on Buildings 2 and 3, nonlinear beam elements were introduced in the models to represent the ring beams and the wall to diaphragm connections at floor levels. The floor diaphragms are defined as orthotropic membranes finite elements with four nodes. The mechanical properties of diaphragms were defined based on the results obtained in previous experimental investigations performed at the University of Trento [7].

3 PUSHOVER ANALYSES

Nonlinear pushover analyses were performed to replicate the seismic response exhibited during the experimental campaign by the two strengthened building prototypes. The numerical models of Buildings 2 and 3 were subjected to a modal force pattern to represent the dynamic amplification attained in the experimental tests. The horizontal forces were applied at the centre of mass at each floor level, parallel to the longitudinal walls both towards the North and South directions.

As a result of the analyses, depicted in Figure 4, the macro-element model was able to approximate fairly well the actual response of the Buildings 2 and 3, in terms of initial stiffness and maximum resisted base shear, which is approximately 80% to 94% of the base shear attained during the shaking table tests in which the structures reached the ultimate limit state.

The damage mechanisms activated and the deformations sustained by the structures are fairly well reproduced with the nonlinear macro-element approach. The damage level reached in each structural element is distinguished as presented in Figure 5 where the different levels are reported. Based on the definition of a damage parameter $\alpha$ embedded in the software, the damage levels refer to the attainment of failure due to shear. The parameter $\alpha$ is equal to zero.
when the element is undamaged, while it reaches the value of 1 in correspondence of the maximum shear strength and increases above 1 in the softening phase.

Figure 4: Comparison of numerical and experimental pushover curves: Building 2 (left) and Building 3 (right).

<table>
<thead>
<tr>
<th>Damage level</th>
<th>Description</th>
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<tr>
<td>α = 0; negligible damage</td>
<td></td>
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<tr>
<td>0 &lt; α &lt; 0.2; slight shear cracking (stiffness degradation)</td>
<td></td>
</tr>
<tr>
<td>0.2 ≤ α &lt; 1; moderate to significant shear cracking (stiffness degradation)</td>
<td></td>
</tr>
<tr>
<td>α &gt; 1; post-peak softening phase</td>
<td></td>
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<tr>
<td>no compression</td>
<td></td>
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Figure 5: Colour legend for shear damage in macro-element models

The macro-element models well capture the in-plane response of the longitudinal façades, particularly 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 the ultimate limit state is reached, as shown in Figure 6.

Figure 6: Damage patterns in the nonlinear macroelement model: East and West façades, when loaded in the South (positive) direction.

Some differences are exhibited when comparing experimental and numerical results when comparing the deflected shape profiles attained at ultimate limit state during the experimental tests with those obtained from the macro-element models. However, the deflected shapes depict that, for example, in Building 3 deformations are characterized by a trend that is similar
to the envelope of displacements in elevation obtained experimentally. The displacement profiles in elevation of the longitudinal walls, obtained numerically (Figure 7), 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.

![Figure 7: Comparison of the deflected shapes of the longitudinal walls, obtained from the numerical analyses and from the experimental test results. From left: East and West façades of Building 3.](image)

4 **NONLINEAR DYNAMIC ANALYSES**

4.1 **Sensitivity analyses**

The global behaviour of the strengthened building prototypes was further simulated by means of nonlinear dynamic analyses, in order to reproduce the hysteretic behaviour exhibited by the strengthened prototypes during each shaking table test. The numerical models were calibrated to capture also the evolution of the exhibited damage pattern at each stage of the dynamic testing. The numerical models were hence subjected to the input signal recorded on the shaking table during each stage of the experimental campaign.

Sensitivity analyses were carried out in order to investigate the influence of several factors on the modelling of the global response of the building prototypes, in particular considering the following:

- discretization/geometry of the equivalent frame model;
- variability of the mechanical properties obtained experimentally from masonry characterization tests and in-plane cyclic tests;
- selection of an appropriate value of viscous damping.

The influence of the model discretization was studied considering three different configurations (Figure 8, right), in which the height of masonry piers is varied as follows:

- **Model STD**: the height of piers is assigned taking into account the deformability of nodes;
- **Model $H_{eff}$**: the piers are assumed to have the same height of the adjacent openings;
- **Model Mixed**: the geometry of the model accounts for the deformability of the node panels as in **Model STD** excluding the West façade, in which the piers are assumed to have the same height of adjacent openings.

The masonry mechanical parameters were defined based on the calibration of the macro-elements on the experimental results from in-plane cyclic tests on piers [5], shown in and hence assumed identical in the three aforementioned models.
Equivalent frame macro-element simulation of shaking table tests on unreinforced stone masonry buildings with strengthening interventions

Figure 8 (left) shows the comparison of the experimental response of Building 3, represented as the envelope of the hysteresis curves (base shear – top displacement relationship) at each stage of testing, with the numerical response of the three models. Modelling the piers with a reduced effective height, as assumed in Model $H_{eff}$, leads to an overestimation of the stiffness of the building and of the maximum base shear sustained. All the three models were not able to reproduce the nonlinear behaviour at the last stages of testing when the structure was subjected to damage levels from moderate to high (tests at nominal PGA of 0.50g and 0.60g), hence with a significant underestimation of the displacement response (i.e. from Models STD and Mixed 15mm of average top floor displacement instead of the 45mm measured experimentally during the last test at nominal PGA of 0.60g).

![Figure 8](image)

**Figure 8**: Influence of the geometrical discretization of models on the numerical simulation of the experimental response of Building 3

The masonry characterization tests and the cyclic in-plane tests on piers and spandrels evidenced the scatter of the masonry mechanical parameters obtained, although this scatter is comparable with the dispersion generally associated to experimental tests on more regular masonry typologies. In order to investigate the influence of such variability, the mechanical properties of a single macro-element were calibrated within the range of values observed experimentally from the in-plane cyclic tests on piers (Figure 9).

As shown in Table 1 for the case of Building 3, the masonry mechanical properties of Model Mixed were defined varying the stiffness and strength parameters in order to better capture the stiffness of the structure and to reproduce its nonlinear response. Starting from the values obtained from the calibration of the single macro-element and assigned to the model Cyclic (CT01), either a single parameter or a series of parameters were varied as in the case of Mod($G$, $\tau$, $\xi$), in which the shear modulus, the tensile strength $\tau$ and the damping ratio $\xi$ differ from the parameters of model Cyclic (CT01).

The shear modulus has been reduced in order to be approximately 1/6th of the Young’s modulus. This reduction allows to better reproduce the stiffness of the structure in the elastic phase and to capture the instant when the shear mechanism is activated in the squat pier at the base of the East façade.
Figure 9: Calibration of the macro-element mechanical parameters based on the results of the cyclic tests.

Table 1: Masonry mechanical parameters for selected numerical models of Building 3

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<tr>
<td>Cyclic (CT01)</td>
<td>1700</td>
<td>610</td>
<td>0.25</td>
<td>4.5</td>
<td>0.135</td>
</tr>
<tr>
<td>Mod(G)</td>
<td>1700</td>
<td>250</td>
<td>0.25</td>
<td>4.5</td>
<td>0.135</td>
</tr>
<tr>
<td>Mod(τ)</td>
<td>1700</td>
<td>250</td>
<td>0.19</td>
<td>4.5</td>
<td>0.135</td>
</tr>
<tr>
<td>Mod(μ)</td>
<td>1700</td>
<td>250</td>
<td>0.25</td>
<td>4.5</td>
<td>0.06</td>
</tr>
<tr>
<td>Mod(G, τ, ξ)</td>
<td>1700</td>
<td>250</td>
<td>0.25</td>
<td>4.5</td>
<td>0.135</td>
</tr>
</tbody>
</table>

In the models Mod(τ) and Mod(μ) the reduction of the strength parameters, with respect to those of model Cyclic (CT01), permits a better evaluation of the base shear sustained by Building 3, as shown in Figure 10, although its displacement response at top floor is overestimated for the tests at nominal PGA of 0.40g and 0.50g while it is underestimated when the ultimate limit state is reached during the test at nominal PGA of 0.60g.

Figure 10: Influence of the definition of the mechanical properties of masonry on the numerical simulation of the experimental response of Building 3 (right)

Hence, a variation of both mechanical and damping parameters was deemed necessary to reproduce numerically, with a sufficient degree of approximation, the experimental behaviour of the strengthened building prototypes exhibited during the test with higher shaking intensity. Model Mod(G, τ, ξ) combines a reduction of stiffness and strength parameters with a decrease
of the damping coefficient, the latter applied after the test at nominal PGA of 0.50g, during which the tested prototype was first subjected to a moderately high level of damage (activation of a shear damage mechanism in the squat East pier and incipient rocking of the slender piers of the West facade).

The influence of the Rayleigh damping parameters was also investigated. Figure 11 represents the results obtained with model $\text{Mod}(G, \tau, \xi)$, in which the damping coefficient was varied between 2% to 5% at the last stage of numerical analysis, without modifying the masonry mechanical properties for the four cases, in order to replicate the dissipative behaviour that Building 3 exhibited during the last experimental test at nominal PGA of 0.60g.

![Figure 11: Influence of the definition of the Rayleigh damping parameters and of masonry mechanical properties on the numerical simulation of the experimental response of Building 3 (model $\text{Mod}(G, \tau, \xi)$ with varying $\xi$).](image)

Figure 12 shows that if the damping coefficient value is set to 2% in this particular building configuration, the displacement demand is overestimated as well as the energy dissipated by the structural system, whereas if the Rayleigh damping parameters remain unchanged ($\xi=5\%$) throughout all the analyses the stiffness degradation in the nonlinear phase is not captured and the damage mechanisms are hardly activated.

![Figure 12: Building 3. Hysteresis cycles and corresponding damage patterns of longitudinal façades at nominal PGA of 0.60g for model $\text{Mod}(G, \tau, \xi)$ with damping coefficient of 2% (left) and 5% (right).](image)
4.2 Reproduction of experimental tests

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. The optimal approximation of the experimental results was achieved implementing a numerical model in which the mechanical properties are those of model \( \text{Mod}(G, \tau, \zeta) \), but with shear strength and equivalent viscous damping coefficient further reduced to better capture the dissipative behaviour of the strengthened prototypes during the last test simulation. In the case of Building 3 the parameters \( \tau \) and \( \zeta \) are reduced after the analysis with seismic input at nominal PGA of 0.50g. The nonlinear dynamic analyses of the macro-element models showed a good consistency of the numerical results with the experimental response of the building, as depicted in Figure 13.

As presented in Figure 14 for Building 3, the hysteretic behaviour of the strengthened buildings, together with the progressive stiffness degradation of the structures, was numerically reproduced with a fairly good approximation both at lower level of seismic intensity and during the last stages of tests when a higher level of damage was accumulated.

The results of the macro-element models of the strengthened prototype buildings, expressed in terms of envelope of deflected shapes, are in good agreement with the experimental displacement response, for each stage of dynamic testing (as depicted in Figure 15 for Building 3 for the tests at nominal PGA of 0.20g and of 0.60g). The maximum error of prediction of the average floor displacement is approximately equal to 15%.

Figure 13: Envelope of the hysteresis curves of Building 3: in black, the experimental curve, in blue the result from nonlinear dynamic analyses, with indication of the values of nominal PGA in [g].

Figure 14: Comparison of experimental data and numerical results from nonlinear dynamic analyses of Building 3 at different levels of nominal PGA.
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Figure 15: Building 3: envelope of deformed shapes in elevation. Tests at nominal PGA of 0.20g (left) and of 0.60g (right)

Similarly to the displacement response, Figure 16 (left) depicts the consistency of the total base shear time histories resulting in the nonlinear dynamic analyses, in particular for the tests with higher peak ground acceleration in the instants of stronger shaking. The occurrence of the shear failure mechanism in the squat pier at the base of the East façade is well captured in the numerical models as well as the significant level of damage due to 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 (Figure 16, right).

Figure 16: Building 3. Comparison of experimental and numerical total base shear time histories (left) and numerical damage pattern of the East and West façades and sketch of the shear failure mechanisms activated in the East wall, test at nominal PGA of 0.60g (right)

5 CONCLUSIONS

The numerical simulation showed a significant consistency of the nonlinear static analysis results with the experimental response of the building. The nonlinear macro-element model reproduces with sufficient approximation the global response of the prototypes, in terms of damage mechanisms activated, of shear capacity and of deformations experienced.

Nonlinear dynamic analyses allowed a simulation of the hysteretic behaviour at the levels of acceleration of the shaking table tests. The models were calibrated by means of sensitivity analyses to capture the evolution of the dynamic properties and the exhibited damage pattern at each stage of dynamic testing. The sensitivity analyses have shown 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 vary the masonry mechanical parameters, within the range of variability resulting from the characterization tests, to further control the strength de-
terioration 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 that the numerical results obtained from a suitably calibrated numerical model are in agreement with the experimental response of the building.

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